CZECH TECHNICAL UNIVERSITY IN PRAGUE

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Expert report:

EVALUATION OF THE DIAGNOSTIC SURVEY
OF THE BRIDGE STRUCTURES AT KM 3.706 – POD VYŠEHRADEM
immovable cultural monument, CLCM Register No. 101315 (part)

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ANNOTATION

The report presents the results of the assessment of the diagnostic survey of the railway bridge under Vyšehrad SO 20-20-05 RAILWAY BRIDGE AT KM 3.706, Výtoň, prepared as part of the preparatory work for the reconstruction of the bridge by SUDOP PRAHA a. s.

The report was prepared by the employees of the CTU in Prague, the Klokner Institute, which is registered in the list of institutes qualified for expert activities, according to the provisions of Section 21(3) of Act No. 36/1967 Coll. and Decree No. 37/1967 Coll., as amended, published in the Central Bulletin of the Czech Republic, year 2004, No. 2, dated 14 October 2004, annex to the Communication of the Ministry of Justice dated 13 July 2004, No. 228/2003-Zn.

Fig. 1: View of the railway bridge under Vyšehrad

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1 INTRODUCTION

A partial diagnostic survey of the railway bridge under Vyšehrad SO 2020-05 RAILWAY BRIDGE AT KM 3.706, Výtoň, on the basis of an order for the assessment of the execution and results of a detailed diagnostic survey prepared as part of the preparatory work for the reconstruction of the bridge by SUDOP PRAHA a. s.; order number of the client: 18/618000405, dated 5 September 2018.

Within the diagnostic and related work, the following was carried out and found:

- Identification of areas for inspection based on SUDOP diagnostics,
- Visual inspection of the bridge, measurement of corrosion losses of selected elements,
- Steel sampling for material testing,
- Sampling of corrosion products for chemical analysis,
- Analysis of corrosion products and evaluation of corrosion evolution over time,
- Chemical analysis of steel and evaluation of the possibility of welding steel,
- Microscopic steel analysis and steel type assessment,
- Mechanical tensile strength tests of steel,
- Charpy impact tests of steel at different temperatures,
- Assessment of the relevance and scope of the surveys carried out by SUDOP, Photographic documentation and report writing, including test results.

2 SOURCE DOCUMENTATION

[1] Rekonstrukce železničních mostů pod Vyšehradem, Stavební část E.1.4, SUDOP PRAHA a. s., Návrh PD k projednání 04/2020

[2] ČSN EN 1993-2, Eurocode 3: Design of Steel Structures – Part 2: Steel bridges

[3] TQC – Road and Motorway Directorate. ch. 19, Technické kvalitativní podmínky staveb pozemních komunikací
Chapter 19: Steel bridges and structures, Ministry of Transport OPK, 2015

[4] ČSN ISO 9223: 1992 Corrosion of metals and alloys – corrosivity of atmospheres – classification

[5] ČSN EN ISO 9223 Corrosion of metals and alloys – corrosivity of atmospheres – classification, determination and estimation

[6] ČSN EN ISO 9224 Corrosion of metals and alloys. Corrosivity of atmospheres. Guiding values for the corrosivity categories

[7] ČSN EN ISO 12944 – 1 Paints and varnishes: Corrosion protection of steel structures by protective paint systems – Part 1: General introduction

[8] ČSN EN ISO 12944 – 2 Paints and varnishes: Corrosion protection of steel structures by protective paint systems – Part 2: Classification of environments

[9] ČSN EN ISO 12944 – 3 Paints and varnishes: Corrosion protection of steel structures by protective paint systems – Part 3: Design considerations

[10] ČSN EN ISO 12944 – 4 Paints and varnishes: Corrosion protection of steel structures by protective paint systems – Part 4: Types of surface and surface preparation

[11] ČSN EN ISO 12944 – 5 Paints and varnishes: Corrosion protection of steel structures by protective paint systems – Part 5: Protective paint systems

[12] Skácel F., Tekáč V., Trendy vývoje kvality ovzduší severozápadních Čech 1990 – 2008, Paliva 3 (2001), 28-36

[13] ČSN EN ISO 6892-1: Metallic materials – Tensile testing – Part 1: Method of test at room temperature 2009.

[14] ČSN EN ISO 148-1: Metallic materials – Charpy pendulum impact test – Part 1: Test method

[15] ČSN 05 1311: Welding. Weldability of steel for arc welding. Testing and evaluation. 1991

3 METHODS AND PROCEDURES USED

3.1 VISUAL INSPECTION

The visual inspection, although it cannot be denied subjectivity, is one of the most important diagnostic procedures, as only this procedure allows to detect deficiencies in practically the entire area under examination. The visual inspection was mainly focused on verifying the typical construction defects documented by SUDOP and checking the current condition in selected and most damaged areas. The side footbridges, the track area and especially the STRABAG construction footbridges used for the reconstruction of the footbridge on the left side of the bridge were used for inspection purposes, which enabled a detailed inspection of the lower chord of the truss on the left side of the bridge, including the perpendicular and lacing connections.

3.2 ELEMENT DIMENSIONS AND SHEET THICKNESS

The cross-sectional dimensions of selected elements of the load-bearing structure were measured with a steel tape measure and a caliper. Due to the irregular corrosion losses along the width and length of the individual elements of the structure (weakening of the cross-section highly variable, rather local character), “swelling” of the cross-sections due to the growth and development of corrosion products, the determination of the undisturbed residual cross-sectional area is difficult. Without the removal of corrosion residues, this is an expert estimate, the actual condition is only visible after the corrosion residues have been blasted off. As part of the reconstruction of the side footbridge on the left side of the bridge, the lower chord of the truss structure was partially blasted from the outside (the blasting is rather secondary – it was done locally in the places of the side footbridge connections, and it is in these places that the corrosion weakening is generally the greatest). The control measurements of corrosion weakening and loss were therefore carried out on the outside of the lower chord, along the entire length of the NK1 span on the left side of the bridge, where the STRABAG footbridge was located.

3.3 STEEL SAMPLING FOR MATERIAL TESTING

The steel samples were taken by cutting out a portion of the rolled angles from the perpendiculars of the truss structure with an abrasive saw. In view of the independent production of the load-bearing structures in the three bridgeworks, samples were taken from all three structures. According to [1], material from at least six different melts (angles NK1-NK3, plates NK1NK3) was supplied for the construction of the bridge. Angles were chosen for the control sampling for material tests because according to [1] they showed slightly worse material properties than plates and also because most of the elements of the main load-bearing structure are made of angles. A total of 5 samples of 400 mm x 40 mm were taken from the angles of the least stressed perpendiculars on the left side of the bridge. The samples were cut into test specimens, each specimen was used to test the tensile strength of the steel 1x and the notch toughness 3x (V notch at -20, 0, +20°C) and each NK (load-bearing structure) was subjected to 1x chemical + 1x microscopic analysis. As the bridge is under listed building protection, the sampling sites have been approved by the National Heritage Institute.

Table 1 Identification of the samples taken and the test specimens prepared.

Sample No.	Designation	Place of collection	Test specimens

1	NK1-1	NK1-L-S1	1x tensile test, 3x Charpy test (-20°C, 0°C, +20°C), 1x chemical analysis, 1x microscopic analysis
2	NK1-2	NK1-L-S15	1x tensile test, 3x Charpy test (-20°C, 0°C, +20°C)
3	NK2	NK2-L-S3	1x tensile test, 3x Charpy test (-20°C, 0°C, +20°C), 1x chemical analysis, 1x microscopic analysis
4	NK3-1	NK3-L-S3	1x tensile test, 3x Charpy test (-20°C, 0°C, +20°C)
5	NK3-2	NK3-L-S14	1x tensile test, 3x Charpy test (-20°C, 0°C, +20°C), 1x chemical analysis, 1x microscopic analysis

3.4 MICROSCOPIC ANALYSIS OF STEEL

The samples for microscopic analysis were first ground (sandpaper P 240, P 360, P 600, P 800 and P 1200) and then polished with diamond paste of 1 μm grit and then polished with Eposal suspension of 0.06 μm grit. To obtain the structure, the samples were etched with Nital 5%. Images of the structure were taken on a Nikon Eclipse ME 600 light microscope at magnifications of 100x (unetched structure) and 200x (etched structure). The aim of the analysis was to determine whether the steel was shear steel or mild steel.

3.5 CORROSION ANALYSIS

As part of the investigation of the condition of the steel load-bearing structure of the bridge under Vyšehrad, a detailed corrosion survey was carried out, which included visual characterisation and localization of significant corrosion damage (photo documentation and measurement of residual thickness), as well as sampling of corrosion products (5 sites in total) and their elemental (XRF – X-ray fluorescence analysis) and phase (XRD – diffraction analysis) chemical analysis. The corrosion products were cleaned with ethanol and spread (friction pan with pestle) before the actual analysis. Furthermore, the subsequent uniform corrosion rate was predicted (numerical linear analysis) based on the climatic data.

4 DIAGNOSTIC WORK

4.1 BRIEF DESCRIPTION OF THE LOAD-BEARING STRUCTURE

The railway bridge bridges the Vltava River with three bridge openings. The load-bearing structures were manufactured in 1901. The load-bearing structures are designed as closed truss multiple systems with a curved upper chord with an identical span of 71.72 m. The individual sections are graded according to the expected stresses. The bridge is double-lined with an elemental rail-track consisting of supporting cross bars and unconnected longitudinal trusses that are inserted between the supporting cross bars. The axial distance between the main beams is 8.80 m. The height of the main beam varies from 7.136 m at the portal to 12.347 m in the centre of the span. The shape of the upper chord is polygonally broken in the place of the centres. The main beam is divided into 16 trusses with lengths of 3.46 m + 4.00 m + 4.40 m and 5 x 4.80 m at mid-span. Pedestrian bridge cantilevers are attached to both main beams with a clear width between the railings of 1,820 mm.

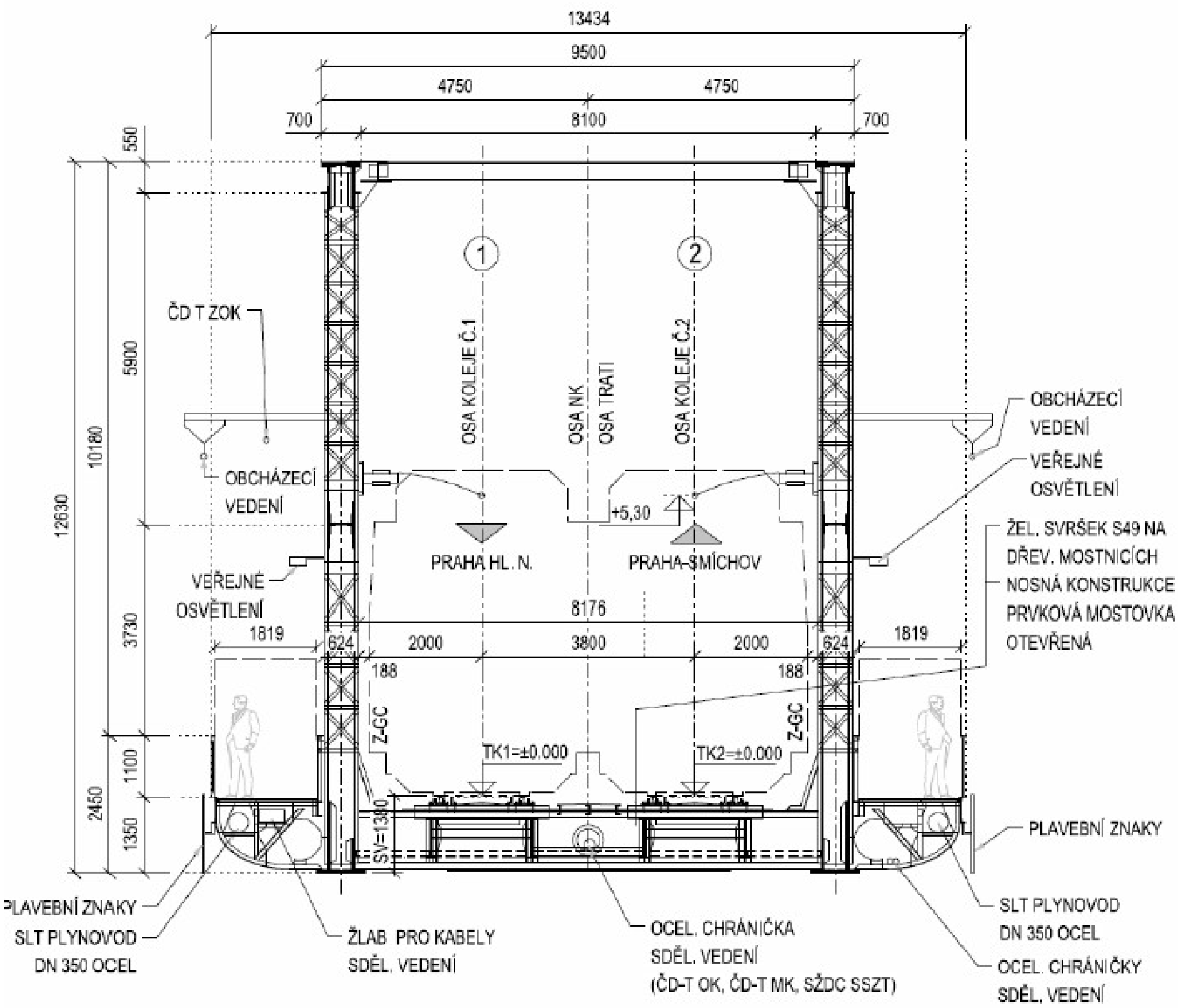


Fig. 2: Cross section in the centre of the span [1]

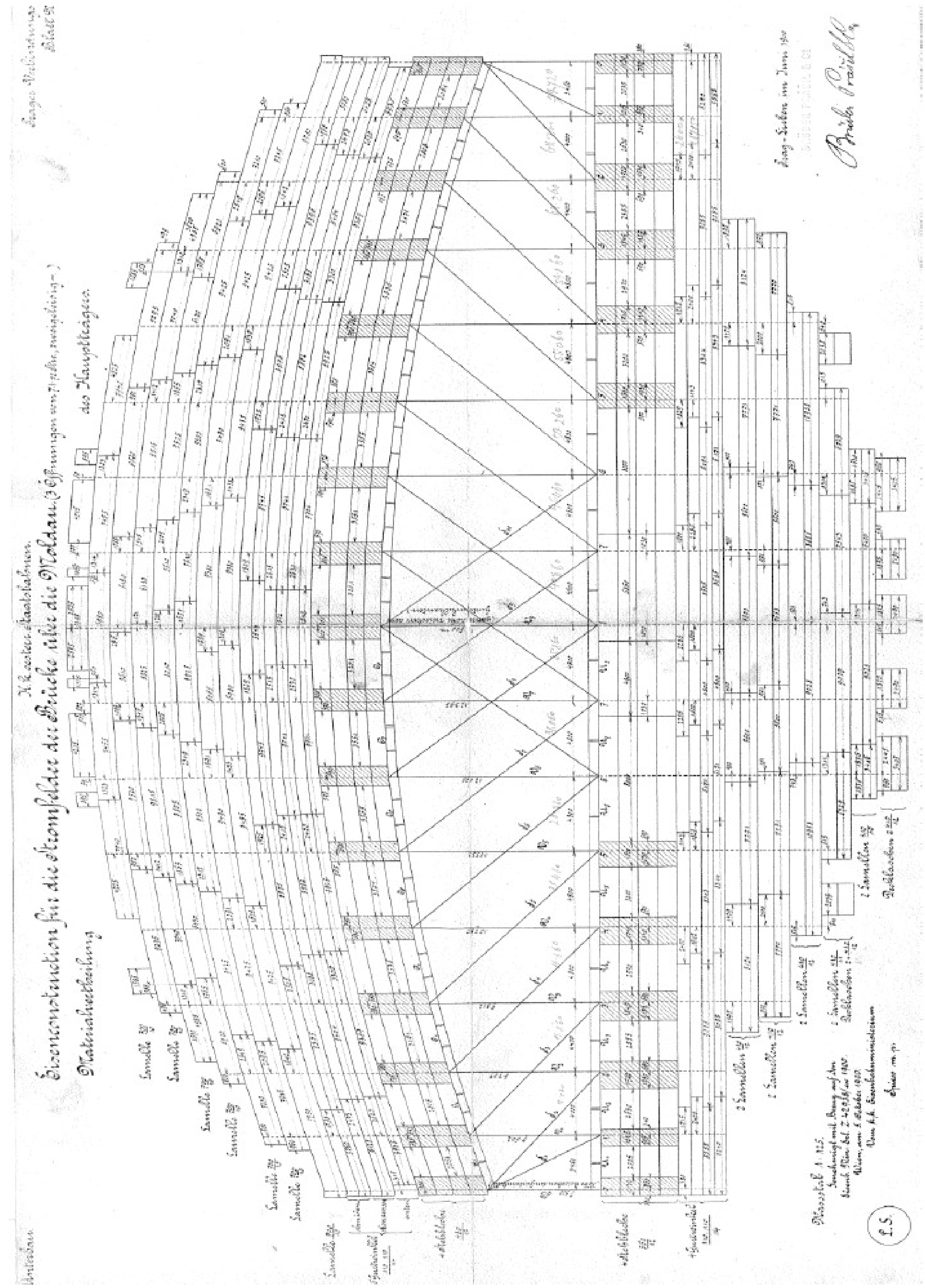


Fig. 3: Diagram of the grading of the chords of the upper and lower flange of the truss beam according to archival documentation [1]

As part of the strengthening of the longitudinal trusses in 1987, the longitudinal trusses were supplemented with bridge rail-track and a brake stiffener. The stiffener was located in the centre of the load-bearing structure and to the edges of the 2nd truss. During the reconstruction, pavement plates and pavement plates longitudinal trusses were installed. The upper stiffening over the rail-track was comprehensively reconstructed in 1970 together with the electrification of the railway. The upper stiffening is formed by a rhombic system with mullions (perpendiculars). The original cross-section stiffening was completely removed and replaced with a mullion at the level of the upper chord made of welded asymmetrical I section. The reconstruction included the outermost portals.

The load-bearing structures are supported on steel bearings. The dilatation movement of all constructions is from Smíchov towards Vyšehrad. The moving bearings are cylindrical roller bearings with five Ø 160 mm rollers and a bascule. Fixed bearings are rack mounted. The substructure is solid of coursed rubble, with concrete infill. The method of foundation in the case of abutment O01 and piers P01 and P02 is flat. The piers are based on steel riveted caissons. The Smíchov abutment O02 from 1871 is based on a wooden pile sleeve. As part of the installation of new structures in 1901, the upper part of the abutment was modified in place of the storage blocks and ledges on the wings.

4.2 <u>DIAGNOSTIC SURVEY BY SUDOP</u>

As part of the preparatory documentation for the reconstruction of the railway bridges under Vyšehrad [1], a detailed diagnostic survey of bridges SO 20-20-04 (4-bay-steel beam bridge) and SO 20-20-05 (3-bay truss bridge) was carried out by SUDOP.

The diagnostic survey for bridge structure SO 20-20-05, which is addressed in this report, contains the following parts:

- Material testing of steel (70 pages)
- Detailed inspection of the steel NK (89 pages + annexes 747 pages)
- Detailed inspection of the substructure (24 pages + annexes 76 pages + video recording of the underwater inspection)
- Static and dynamic load test (104 pages)

Material tests of the steel were carried out on a total of 18 samples. In view of the independent production of the load-bearing structures in the three bridgeworks, samples were taken from all three structures (3 samples of plates + 3 samples of angles each). Chemical structural analysis of the steel was carried out. Mechanical tests were carried out to determine the tensile strength in the longitudinal and transverse direction, Charpy impact tests at room temperature.

The detailed inspection of the steel NK included in particular the documentation of corrosion weakening, which is the basis for the static recalculation of the bridge's load-bearing structures and for determining the design of the scope of reconstruction of the bridge's load-bearing structures, i.e. the possibility of partial replacement of an element or complete replacement of a corroded element. The corrosion weakening of the steel structure elements was documented using the so-called Element Cards, where the corrosion weakening of the cross-section is documented in detail for each element of the load-bearing structure.

The substructure was also inspected in detail and individual defects documented. In order to record the defects in the substructure, surface layouts of the individual supports were created. The surface was divided by a rectangular grid into parts (relative raster), where the description of defects was made. The inspection also included a detailed underwater survey of Piers P01 and P02 in the Vltava River. A geotechnical and structural engineering survey of the substructure was carried out in order to verify the material properties of the stone masonry of the piers of the railway bridge for the structural recalculation of the substructure. Diagnostic borings were made into the structure to verify the material characteristics and hidden dimensions of the substructure.

The static and dynamic load test was carried out in order to verify the conformity of the measured quantities determined on the calculation model of the bridge for its possible modification (calibration according to the actually measured values) and to determine the fatigue effects of the traffic on the bridge (determination of the spectra of the traffic load). During the static load test, the following was measured: vertical deflection (by radar interferometry), deformation of the end supporting cross bar, normal stresses on selected elements of the bridge structure (upper and lower chords, lacings, supporting cross bars, longitudinal trusses). During the dynamic load test, the response of the structure to dynamic loading by passing the test load was measured: acceleration of vertical deformation u_z and transverse deformation u_y in the middle of the span and in about ¼ of the span, normal stresses on selected elements of the bridge structure in accordance with the static test.

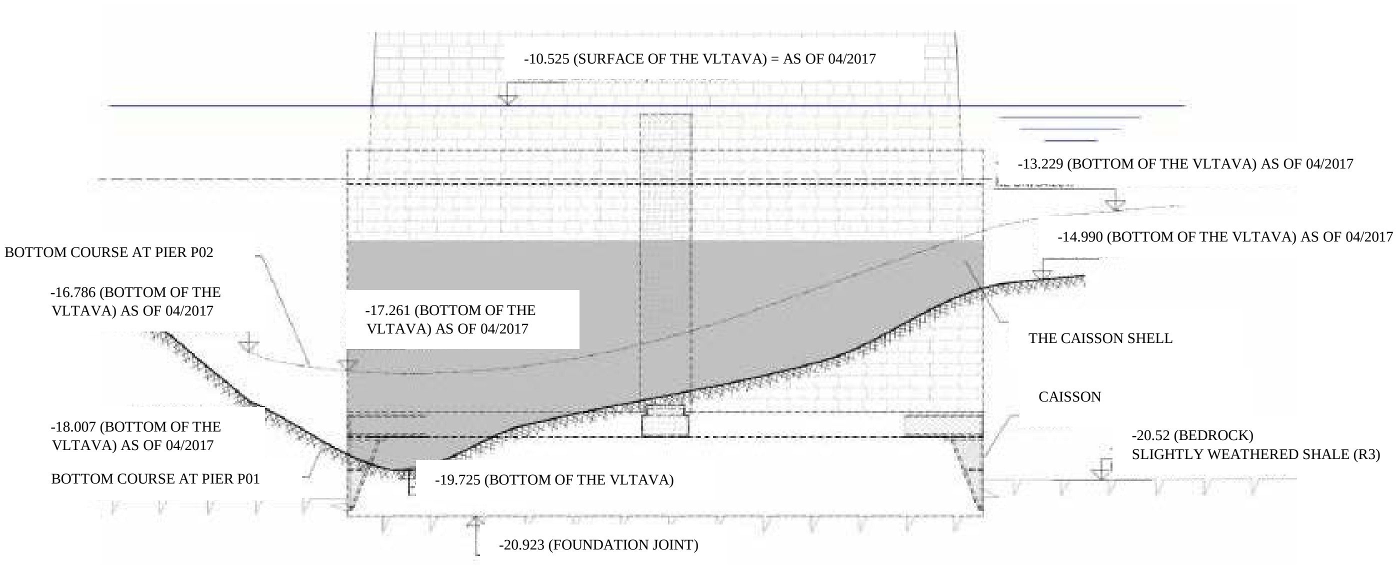
It can be concluded that the survey prepared by SUDOP is of a good standard in terms of content and detail. The number of samples taken is sufficiently representative. The steel load-bearing structure and the substructure of the bridge were documented in detail. It is only necessary to state that on an uncoated steel load-bearing structure it is rather an expert estimate of the actual state of corrosion losses. On the basis of the partial control survey, the conclusions of the diagnostic survey developed by SUDOP [1] can be confirmed. The results of the SUDOP survey [1] are compared with the results of the control survey in the individual chapters of this document.

The diagnostic survey [1] served as a basis for a detailed static recalculation of the load-bearing structure, which determined the load capacity and residual life of the bridge. On the basis of the results of the static recalculation, preparatory documentation was prepared for the reconstruction of the bridge to maintain the existing compatibility for the next 30 years. The conclusions of all parts are described in the technical report in [1].

The following is a summary of important conclusions taken from the SUDOP documentation [1]:

From the inspection of the substructure, the overall condition can be summarized:

- the upper surface of the joint is cracked, sporadically hollowed out. Upper surface lightly soiled, good condition.
- shanks joint occasionally frayed, good condition.
- between the stones the binder is leaking in places and forms a weak crust on the masonry, good condition. At the top – graffiti.
- The survey of the substructure shows that the non-functioning of the movable bearings is causing distortion of the block masonry of the abutments. The anchors implemented in about 1987 caused the influence to shift down a number. The pairing must therefore be continuously repaired.
- Furthermore, the underwater survey showed that the bottom around Piers P01 and P02 is significantly scoured on the upstream side. The cavern reaches a depth of – 5.0 m, i.e. down to bedrock. The substructure of the caisson is also exposed. The cladding of the caisson shows extensive damage and deformation due to the effects of corrosion.
- remediation of these faults is necessary as part of the repair work as soon as possible.



Bottom course (cavity on the upstream side) at Pier P01 and P02 as determined by underwater survey

A detailed inspection of the steel load-bearing structures revealed that:

- detailed inspection of the corrosion weakening revealed **faults that are limiting for the residual life of the bridge structure** Particularly the detail at the connection point of the truss connector of the segmented rod between the pair of neck angles and the actual rods to the splice plates or directly to the lower chord. Dirt settles in the narrow space of the crevice between the neck angles and the constant moisture causes corrosion of the entire neck angle flanges or significant corrosion loss,
- from the point of view of repairability, this is an unrepairable fault that can only be solved by **replacing the entire element**. The corrosion limitation at the point of failure cannot be reduced in any way because corrosion protection repair is not practicable given the layered corrosion growth at the crevice that causes permanent deformation of the outer flanges of the angles. Over time, the corrosion of these faults will continue to worsen. From the point of view of load-bearing capacity, the identified defects are significant and reduce the load-bearing capacity of the rods. The inspection revealed a high frequency of these failures. It applies to virtually all perpendiculars and lacings.
- Replacement of all these affected elements in the above-mentioned range is only possible in a lightened state **on the mounting frame outside the construction opening.**

It should be noted here that these are characteristic **"inherent"** defects of early 20th century riveted truss structures of medium and larger spans with segmented rods and a lower chord section ± ± (a pair of inverted T-sections). The above-mentioned failures are mainly due to **inappropriate structural design** that corresponds to the knowledge, design possibilities and effectiveness of the bridge structures at the time of their occurrence. Even with regular maintenance, the corrosion protection options for these details are very limited and **cannot be reliably achieved in the long term.** In inaccessible gaps and crevices, the damaged corrosion protection cannot be effectively repaired, i.e. the degradation of the structure due to corrosion over time is still ongoing and it is only a matter of time before it reaches the limit values in terms of the load-bearing capacity of the structure.

- another element that is weakened by corrosion are the neck angles of the lower chord and the splice plates of the lower horizontal stiffening including the above bearing splice plates. Here, due to the constant moisture in the panel points, corrosion loss of the neck angles occurs, which is due to the small gap between the lower chords, which does not allow the spontaneous fall of dirt. The failure can only be repaired by replacing these angles and splice plates,
- 2 new cracks of 185 mm and 580 mm in the upper chords of the longitudinal trusses, which were not detected in the structure during the detailed inspection in 2014, were diagnosed during the corrosion weakening inspection,
- Compared to the regular inspection in 2014, there is an increasing deterioration in the structural condition of the bridge structure, as evidenced by newly diagnosed cracks in the longitudinal trusses.

Overall, the current condition of the bridge elements can be characterised as being at the limit of their service life and in many cases beyond this limit.

In order to ensure the current compatibility of the C3/40 TTZ for the next 30 years, it is necessary to replace most of the load-bearing structure, see the following tables.

NOTES

N - COATING
V - REPLACEMENT – COMPLETE
VC - PART REPLACEMENT
O - REPAIR OF THE ELEMENT

RECAPITULATION OF THE NECESSITY OF REPLACEMENT DUE TO CORROSION WEAKENING OF THE OK

ELEMENT: MAIN BEAM PAGE: L/P NK NO.: 1, 2 , 3

TRUSS	LEFT MAIN BEAM					RIGHT MAIN BEAM				
	UPPER CHORD	LOWER CHORD	PERPENDICULAR	LACING	LACING	UPPER CHORD	LOWER CHORD	PERPENDICULAR	LACING	LACING
	O.L	U.L	V.L	D.L	Z.L	O.P	U.P	V.P	D.P	Z.P
0			O					O		
1	N	VC	V	V		N	VC	V	V	
2	N	VC	V	V		N	VC	V	V	
3	N	VC	V	V		N	VC	V	V	
4	N	VC	V	V		N	VC	V	V	
5	N	VC	V	V		N	VC	V	V	
6	N	VC	V	V	V	N	VC	V	V	V
7	N	VC	V	V	V	N	VC	V	V	V
g	N	VC	V	V	V	N	VC	V	V	V
9	N	VC	V	V	V	N	VC	V	V	V
10	N	VC	V	V	V	N	VC	V	V	V
11	N	VC	V		V	N	VC	V		V
12	N	VC	V		V	N	VC	V		V
13	N	VC	V		V	N	VC	V		V
14	N	VC	V		V	N	VC	V		V
15	N	VC	V		V	N	VC	V		V
16	N	VC	O			N	VC	O		

RECAPITULATION OF THE REPLACEMENT OF THE RAIL-TRACK ELEMENTS DUE TO THE CORROSION WEAKENING OF THE OK AND STATIC ACTION

ELEMENT: RAIL-TRACK PAGE: **L/P** NK No.: 1,2,3

TRUSS	LEFT SIDE					RIGHT SIDE				
	SUPPORTING CROSS BAR	LONGITUDINAL TRUSS	LONGITUDINAL TRUSS	LONGITUDINAL TRUSS STIFFENING	LOWER STIFFENING	SUPPORTING CROSS BAR	LONGITUDINAL TRUSS	LONGITUDINAL TRUSS	LONGITUDINAL TRUSS STIFFENING	LOWER STIFFENING
	P	LI	L2	WL	WU	P	L3	L4	WL	WU
0	VC					VC				
1	VC	V	V	V	VC	VC	V	V	V	VC
2	VC	V	V	V	VC	VC	V	V	V	VC
3	VC	V	V	V	VC	VC	V	V	V	VC
4	VC	V	V	V	VC	VC	V	V	V	VC
5	VC	V	V	V	VC	VC	V	V	V	VC
6	VC	V	V	V	VC	VC	V	V	V	VC
7	VC	V	V	V	VC	VC	V	V	V	VC
8	VC	V	V	V	VC	VC	V	V	V	VC
9	VC	V	V	V	VC	VC	V	V	V	VC
10	VC	V	V	V	VC	VC	V	V	V	VC
11	VC	V	V	V	VC	VC	V	V	V	VC
12	VC	V	V	V	VC	VC	V	V	V	VC
13	VC	V	V	V	VC	VC	V	V	V	VC
14	VC	V	V	V	VC	VC	V	V	V	VC
15	VC	V	V	V	VC	VC	V	V	V	VC
16	VC	V	V	V	VC	VC	V	V	V	VC

From the assessment of the individual elements of the bridge's load-bearing structure it is clear that the bridge deck elements are at the end of their service life in terms of fatigue stresses. The load capacity of the structure is about 61% of the LM71 load model for the design of railway bridges according to the valid standards, therefore the compatibility of the bridge had to be reduced to TTZ C3/40.

A summary of the calculation results for **NK1 to NK3** is given in the following table:

Element	**Ultimate value / utilisation v %**		**Load-bearing capacity**	**Compatibility**	**Note**
	carrying capacity	**fatigue**	Z_{LM71}	**TTZ/ PRTTZ**	
MAIN BEAM					
Upper chord – 0	78%	-	1.44		
Lower chord – U	109%	3%	0.87	C3/60	
Lacings – pushed – D	85%	-	1.26		
Lacings – drawn – D	: V:	**95%**	0.98	C3/60	
Perpendicular – V	117%	35%	0.77	C3/60	
TOTAL – MAIN BEAM	**117%**	**95%**	**0.77**	**C3/60**	**fatigue life until 2055**
RAIL-TRACK					
Longitudinal truss L1 to L8	133%	**278%**	0.72	C3/60	fatigue life until 2024
Supporting cross bars – supporting PO	124%	11%	0.69	C3/60	
Supporting cross bars – standard P1 to P8	146%	**188%**	0.61	C3/40	fatigue life until 2029
TOTAL – RAIL-TRACK	**146%**	**278%**	**0.61**	**C3/40**	**fatigue life until 2024**
TOTAL – Bridge at km 3.706 Pod Vyšehradem	**146%**	**278%**	**0.61**	**C3/40**	**fatigue life until 2024**

Note:

1) compatibility assessed for Z_{LI}^,<1.0
2) the service life of the element until failure is one year, when the accumulation of fatigue damage reaches 100% (without reconstruction)

The conclusions for the reconstruction of the bridge, which should include the replacement of damaged elements, their reinforcement and rehabilitation of the substructure and which would ensure the current compatibility of TTZ C3/40 for the next 30 years, are as follows:

After evaluating the scope of the proposed modifications, in relation to the investor's intention to reconstruct while maintaining the existing structure for the given SO, it is necessary to state that the proposed scope of reconstruction of the steel structures of the bridges at km 3.545 and km 3.706 **is disproportionate to the overall implementation time, long-term limitation of operation, financial costs and the resulting parameters with a limited life of 30 years and we recommend the client to reconsider the intention to reconstruct the steel structures of the bridges and to consider the replacement of the load-bearing structures** that will ensure a service life of 100 years for the bridge structure. However, this proposal would require the removal of the listed building protection on these affected parts of the bridge.

Furthermore, with regard to the conclusions of the static recalculation, it is necessary to carry out the construction in the short-term horizon of 5 years in order to maintain the compatibility parameters of TTZ C3/40 on this line section.

Until the reconstruction is completed, the **intensity of** the traffic load on the bridge structure **must not be increased**!

It can be concluded that the survey prepared by SUDOP as part of the documentation [1] is at a good level in terms of content and detail and the intervention procedure proposed in the SUDOP project [1], i.e. the recommendation for the total replacement of the load-bearing structure, is based on relevant information.

4.3 MARKING OF THE ELEMENTS OF THE LOAD-BEARING STRUCTURE

The designation of the elements of the load-bearing structure was taken from the original diagnostic survey carried out by SUDOP [1]. The corrosion weakening of the steel structure elements was documented by means of the so-called Element Cards. It is a system of documentation of corrosion weakening of elements, where the structure is divided into groups of elements. The individual elements are thus clearly identified. To identify the position of the defect, the local stationing of the element is also used, which is relative to the length of the element from 0 at the beginning to 1 at the end of the element. Within an element, defects are identified by a serial number. An unambiguous code designation is used to describe the defect. The photo documentation has the same code designation. In this way, defects can be monitored during subsequent inspections and retrospectively located.

V.W.X.Y.Z

V – load-bearing structure (1 – NK1, 2 — NK2, 3 – NK3)

W – Element type (1. Perpendicular, 2. Lower chord panel points, ...8. Upper chord, 9. Lower chord...)

X – Truss (1 to 30)

Y – Bridge side (1 – left, 2 – right)

Z – Cross-sectional defect number (1 to 999)

Table 2 List of steel construction element groups [1]

Component No. (code – W)	Component name	Mark
1	Upper chord	01 to 016
2	Lower chord	U1 to U16
3	End perpendiculars	V0 and V16
4	Inner perpendiculars	V1 - V4a V12-V16
5	Centre perpendiculars	V5- V8a V9-V12
6	Lacings – lateral	D1 - D4 and Z12 -Z15
7	Lacings – inner	D5 - D8 and Z8 - Z11
8	Lacings – central	D9-D10aZ6-Z7
9	Supporting cross bars	PO - P16
10	Longitudinal trusses	L1 -L16
11	Upper stiffening	WO
12	Lower stiffening	WU

4.4 FAULT REGISTRATION SYSTEM

Due to the very irregular weakening of the elements, it was decided to write the largest weakening in the given element length. The marking of the individual rods is made on the outside and inside and the top and bottom of the element according to the actual position to the ground. For perpendiculars on the front and rear of the element (directional arrow km).

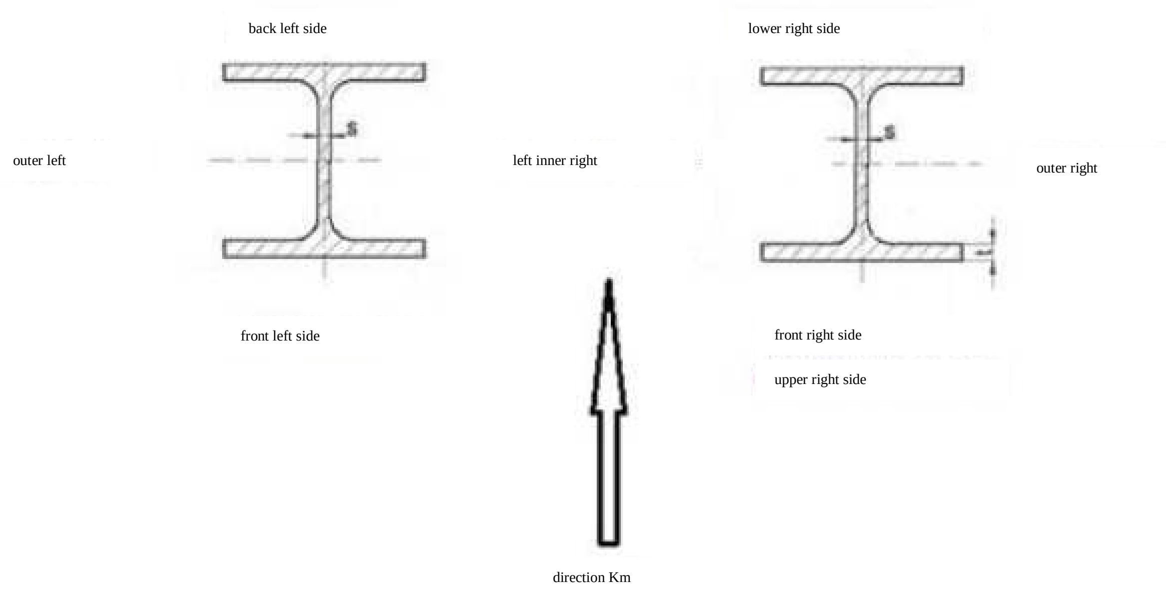


Fig. 4: Fault registration system according to [1]

The fault is recorded in the element card using the double number profile weakening / material loss. Profile weakening – represents weakening of the shoulder by deep corrosion in part or in the whole width by a given value. E.g.: written in the format – by 5 mm in 50 mm means a weakening of the profile in the width of 50 mm by 5 mm in the lower/upper part of the total thickness of the section. Material loss – represents the missing part of the section shoulder of the specified width. E.g.: 20 mm loss **–** means 80 x 60 mm is left from 80 x 80 section.

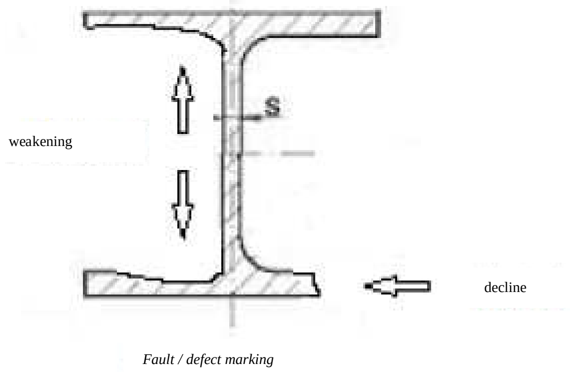


Fig. 5: Fault registration system according to [1]

4.5 <u>VISUAL INSPECTION</u>

The inspections were carried out by the KI staff on two dates:

a) on 03/09/2018 – inspection and sampling,
b) on 20/09/2018 – inspection and measurement of corrosion weakening and losses on the blasted parts of the lower chord of NK1.

The inspection included taking steel samples for laboratory tests and measuring corrosion weakening. The inspection of the bridge revealed all the typical defects of the structure, which were already described in detail in the original document from SUDOP [1].

For the upper structure, there is crevice corrosion of lacings and perpendiculars, local corrosion in the area of the connections of perpendiculars and lacings to the lower chord (significant corrosion weakening of cross-sections), sheet corrosion especially in the area of the connections at the lower chord of the truss, loose and missing rivets, rivets with corroded heads, broken CPC structure, cracks in the upper chord of the longitudinal truss.

In the case of the lower chord, there are failures in the area of the panel points, where dirt accumulates and permanent corrosion occurs due to moisture and dripping water from the elements above the panel point. The insufficient gap between the lower chords does not allow dirt to fall off spontaneously. The most damaged parts are the flanges of the neck angles. There is also extensive damage to the horizontal splice plates.

On the outside of the diagonals, crevice corrosion is increasing to such an extent that the flanges are permanently deformed. The steel material reached its yield strength and was further permanently deformed (the steel material locally plasticised).

In terms of the steel structure failures, the most serious failure is the crevice corrosion of the lacing and perpendicular chords at the connection to the splice plates and also at the connection points of the joints of the segmented rods. In these problematic details, complete corrosion, i.e. breakage of the chords of the connected rods, occurs, which affects the load-bearing capacity of the element.

Massive corrosion loss occurs in the area of the end portal verticals where the splice plates are above the bearing (due to persistent moisture, high dirt fallout and minimal ventilation).

The actual corrosion weakening can only be detected after blasting the corrosion fumes. Without blasting the structure, this is only an expert estimate of the actual condition, which is largely subjective. Based on several control measurements at unblasted sites, no major discrepancies with the original diagnostic inspection were found.

As part of the reconstruction of the side footbridge on the left side of the bridge, the lower chord of the truss structure was partially blasted from the outside (the blasting is rather secondary – it was done locally in the places of the side footbridge connections, and it is in these places that the corrosion weakening is generally the greatest). The control measurements of corrosion weakening and loss were therefore primarily carried out on the outside of the lower chord, along the entire length of the NK1 span (always in each span of the truss) on the left side of the bridge, where the STRABAG footbridge was located. On the basis of control measurements carried out on the outside of the bottom flange of the NK1 span, it was found that the actual corrosion weakening is in several cases slightly worse after (partial) blasting than expected on the basis of a detailed inspection of the uncleaned bridge's load-bearing structure. The measurement sites are documented in Annex P1 – Photo documentation and Annex P2 – Element Cards, the results are summarised in Table 3.

Table 3 Comparison of corrosion losses before blasting (SUDOP) and after blasting (KI)

Deterioration – Comparison					
Element designation	Total area of 1/4 cross-section				Final total area loss:
	Original	SUDOP	KI	Deterioration after blasting by:	
	A	A	A		
	[mm^2]	[mm^2]	[mm^2]		
U1 - 0	9604	7972	7696	3%	20%
U2 - 1	9604	7624	7624	0%	21%
U3 - 1	9604	7876	7876	0%	18%
U4 - 0,5	14524	12416	12416	0%	15%
U5 - 1	16984	15452	15260	1%	10%
U6 - 0	19444	17816	17528	2%	10%
U7 - 0	21494	19964	19007	5%	12%

U8 - 0	21494	20538	19962	3%	7%
U9 - 0	21494	20538	19771	4%	8%
U10 - 0	21494	20347	19963	2%	7%
U11 - 1	19444	18871	17913	5%	8%
U12 - 0	16984	15931	15931	0%	6%
U13 - 1	14524	13951	12994	7%	11%
U14 - 0,5	9604	9316	8644	7%	10%
U14 - 1	9604	9028	8660	4%	10%
U15 - 0	9604	9124	8548	6%	11%
U15 - 0,5	9604	9604	8452	12%	12%
U15 - 1	9604	9156	8884	3%	7%
U16 - 0	9604	9604	8980	6%	6%

In the measurements made after blasting, corrosion weakening of the measured ¼ cross-sectional area of the lower chord was recorded in several cases to be about 5% higher than that found in the original SUDOP [1] measurements without blasting. In one case, a deterioration of up to 12% was observed (U15-0.5). However, in terms of load capacity design, this is not the most weakened cross-section of this type, because in terms of internal forces and cross-sectional area, U15 = U2 and the overall weakening is higher for U2 (21%) than for U15 (12%).

The nature of the corrosion weakening of the lower chord of the truss is basically similar throughout the bridge span – weakening of the neck angles and the top side of the lower chord, which is localised mainly at the panel points. Since the thickness of the lower chord is graded from the support to the centre of the bay and the lower chord consists only of angles in the outermost bays of the truss, the proportion of corrosion weakening is greatest here relative to the total cross-sectional area.

At the substructure there are visible water leaks with leaching of the binder through the masonry, in places cracked and individually fallen masonry joints at supports and piers.

Typical defects are documented in Annex P1 – Photo documentation.

4.6 CORROSION ANALYSIS

Based on the corrosion survey (Annex 4), the most important conclusions can be drawn:

a) the steel load-bearing structure of the bridge is in a state of disrepair in terms of corrosion development,
b) in many places there is significantly developed localised corrosion damage reducing the cross-section of the element, for which the residual service life of the structure cannot be predicted with sufficient predictive ability,
c) Local corrosion of fasteners (rivets) due to crevice corrosion damage is also common,
d) the slotted corrosion mechanism and especially the differential aeration cell mechanism for corrosion under the deposits are responsible for the numerous local corrosion damages
(the deposits also retain moisture and extend the wetting time τ),
e) the steel load-bearing structure was originally protected by a coating system that was renewed several times on the original coatings, but currently the layered organic coating system is heavily corroded or completely rusted and its barrier protection mechanism is essentially zero and therefore negligible.

f) In riveted load-bearing structures with segmented cross-sections of this type, it is not possible to ensure sufficient corrosion protection in hard-to-maintain and inaccessible places (especially joints of segmented cross-sections and panel points), i.e. places that currently show the occurrence of significantly developed localised corrosion damage, even with regular renewal of corrosion protection.

4.7 MATERIAL PROPERTIES OF STEEL

The results of the original diagnosis by SUDOP [1] were confirmed by a control material survey. The structural analysis confirmed that the load-bearing structure of the bridge was made of mild steel, the collected samples have a ferritic-perlitic structure. On the basis of chemical analysis, it was shown to be a low carbon non-alloy weldable steel. In terms of mechanical properties, the steel can be classified as S235 JR (minimum tensile strength 26%, yield strength 235 MPa, notch toughness 27 J at +20°C). The steel taken from all three bridge bays shows similar material characteristics. From the room temperature tensile test of the steel test pieces it was found that test pieces 3 and 5 show a significant yield strength, while test pieces 1, 2 and 4 do not show a significant yield strength.

Table 4 Tensile test at room temperature.

Sample number	F_{eH}	$F_{0.2}$	F_m	R_{eH}	$R_{e0.2}$	R_m	A_{gt}	A_5	Z
	[kN]	[kN]	[kN]	[MPa]	[MPa]	[MPa]	[%]	[%]	[%]
1		39.3	55.1		248	348	17.6	33.3	54.7
2		37.8	53.5		237	336	21.7	37.6	58.7
3	47.0		58.1	295		365	18.4	31.4	58.5
4		40.7	57.7		251	356	20.1	33.6	51.8
5	40.4		61.7	251		384	19.6	35.1	51.8

*Note: $\boldsymbol{A_5}$ ductility, $\boldsymbol{A_{gt}}$ total extension in percentage measured by an extensometer at maximum load, $\boldsymbol{F_{e0,2}}$ force on the proof stress, $\boldsymbol{F_{eH}}$ force on the upper yield strength, $\boldsymbol{F_m}$ maximum force achieved during loading, $\boldsymbol{R_{e0,2}}$ proof stress, $\boldsymbol{R_{eH}}$ upper yield strength, $\boldsymbol{R_m}$ breaking strength, **Z** contraction.*

Table 5 Charpy pendulum impact test.

Test specimen	Temperature	L	B	W	E_c	KV_2	Proportion of shear fracture
	[°C]	mm	mm	mm	kg·m	J	[%]
1-1	+20	55.00	10.05	9.99	6.8	67	incomplete fracture
2-1		55.00	10.03	10.00	6.0	59	incomplete fracture
3-1		55.00	10.12	9.89	2.2	22	35
4-1		55.00	10.07	9.56	4.0	39	incomplete fracture
5-1		55.00	9.99	9.90	2.6	26	43
1-2	0	55.00	10.05	9.83	1.4	14	incomplete fracture
2-2		55.00	9.94	9.96	0.9	9	incomplete fracture
3-2		55.00	9.87	9.76	1.0	10	10
4-2		55.00	10.18	9.65	2.0	20	5

5-2		55.00	10.04	9.82	2.2	22	incomplete fracture
1-3	-20	55.00	9.98	10.02	0.6	6	0
2-3		55.00	10.03	10.07	0.8	8	0
3-3		55.00	9.96	9.79	0.6	6	0
4-3		55.00	10.13	9.51	0.7	7	0
5-3		55.00	10.00	9.92	0.8	8	0

*Note: **B** thickness of the test specimen, **E_c** energy required for breaking, **KV_2** absorbed energy in the case of a V-notched test rod using a 2 mm radius cutting edge, **L** length of the test rod, **W** width of the test rod.*

In addition, compared to the original SUDOP diagnostics, Charpy impact tests were also performed at -20 °C, because the energy required for failure of the test specimens usually decreases as the temperature drops in steels. According to the test results of these removed specimens, the average value of the notch toughness at -20°C was only 7 J and this unfavourable expectation was confirmed. According to the currently valid standard for the design of steel bridges, ČSN EN 1993-2 (Table 3.1) [2], J2 steel with a notch strength of at least 27 J at -20°C is required for bridge structures. The evaluated average notch toughness value of 7 J is therefore only about a quarter of the standard required value of 27 J and corresponds to JR steel. The test specimens no longer exhibited shear fracture (tough) at -20°C and broke with brittle fracture throughout their cross-section. **A fragile quarry is a dangerous type of fault that occurs suddenly without previous visible symptoms.** E.g. according to TQC – Road and Motorway Directorate, ch. 19 (19:A.2.2.1.1 (4)) [3] the use of JR quality steel is not allowed for welded bridge structures and is only possible for riveted elements up to a thickness of 10 mm. However, the riveted elements of the load-bearing structure of the bridge have a higher plate thickness (min. 12 mm).

Comparison with the results presented in the SUDOP report [1]:

- The steel taken from all three bridge bays shows similar material characteristics.
- From the tensile test, it was found that specimens 1 and 2 taken from the NK1 structure showed significantly lower values of the breaking strength R_m and Z contraction than the test specimens 1U, 2U and 3U presented in the documents [1].
- From the tensile test, it was found that the specimen 3 taken from the NK2 structure has lower values of tensile strength A_5 and Z contraction than the test specimens 4U, 5U and 6U given in documents [1].
- From the tensile test, it was found that specimens 4 and 5 taken from the NK3 structure showed lower values of the tensile strength A_5 and the Z contraction than the test specimens 4U, 5U and 6U presented in the documents [1].
- From the Charpy pendulum impact test at +20°C, it was found that specimens 1 and 2 taken from the NK1 structure showed significantly higher values of absorbed energy KV_2 than the 2U test specimen [1].
- From the Charpy pendulum impact test at +20 °C, it was found that the specimen 3 taken from the NK2 structure showed a significantly lower value of the absorbed energy KV_2 than the 5U test specimen [1].

- From the Charpy pendulum impact test at +20 °C, it was found that specimens 4 and 5 taken from the NK3 structure showed comparable values of absorbed energy KV_2 to the 8U test specimen [1].
- By comparing the chemical composition of the samples presented in Chapter 3 with the data given in the documents [1], it was found that the composition of the steel is similar and in both cases it is a low carbon non-alloy weldable steel.
- By comparing the microscopic analysis, it is clear that the structure of the steel samples presented in Chapter 4 is the same as that of the 2U, 5U and 8U samples presented in [1].

In summary, we conclude that despite some differences explained by natural variability of the material and different sampling locations, we reach the same conclusion as SUDOP [1]. **In terms of mechanical properties, the steel can be classified as S235 JR i.e. minimum tensile strength 26%, yield strength 235 MPa, notch toughness 27 J at +20°C.**

5 CONCLUSIONS AND RECOMMENDATIONS

The results of the original diagnostics performed by SUDOP [1] were confirmed by a material control survey. Structural analysis confirmed that the load-bearing structure of the bridge was made of mild steel. On the basis of chemical analysis, it was shown to be a low carbon non-alloy weldable steel. **In terms of mechanical properties, the steel can be classified as S235 JR i.e. minimum tensile strength 26%, yield strength 235 MPa, notch toughness 27 J at +20°C. In this respect, our work confirms the conclusions of the SUDOP diagnosis.**

According to the currently valid standard for the design of steel bridges [2] J2 steel with a notch strength of at least 27 J at -20°C is required for bridge structures. Therefore, in contrast to the original diagnosis [1], Charpy impact tests at -20 °C were also performed. The evaluated average value of the notch toughness at -20 °C was only 7 J for the sampled specimens, i.e. only about a quarter of the standard required value of 27 J. The test specimens at -20°C no longer showed shear fracture (tough) at all and broke with dangerous brittle fracture along their entire cross-section.

The inspection of the bridge revealed all the typical defects of the structure, which were already described in detail in the original document from SUDOP [1].

Due to the irregular corrosion losses along the width and length of the individual elements of the structure (weakening of the cross-section highly variable, rather local character), "swelling" of the cross-sections due to the growth and development of corrosion products, the determination of the undisturbed residual cross-sectional area is difficult. Without the removal of corrosion residues, this is an expert estimate, the actual condition is only visible after the corrosion residues have been blasted off.

On the basis of a check measurement of the partially blasted outer side of the lower chord of the truss structure (carried out within the reconstruction of the side footbridge by STRABAG in 08-09/2018), corrosion weakening of the cross-sections was found in several cases to be approximately 5% greater than that determined in the original SUDOP inspection [1], which was carried out on the unblasted structure. It is therefore very likely that the actual corrosion weakening will be higher for some other elements of the structure than determined by the detailed bridge inspection. At the same time, localised corrosion damage occurs on the elements. The most significant corrosion damage with perforation consequences is caused by the influence of localised forms of corrosion (crevice corrosion, cells with differential aeration under the deposits). This type of corrosion damage has a completely unpredictable course and may also have a close relationship to the initiation and growth of cracks or fatigue cracks in the steel.

The load-bearing capacity and residual life of some elements of the load-bearing structure may therefore be even lower in reality than determined in the static calculation by SUDOP [1].

Corrosion weakening of the structure occurs primarily in the tensioned elements (lacings, lower chord and parts of the perpendiculars). Significant local corrosion weakening of some elements causes indentations in the structure, which are very unfavourable in terms of fatigue stresses. As a result of corrosion weakening, the stresses are redistributed in the cross-section and the tenseness increases in the remaining part of the element. Tensile elements generally have higher accumulations of fatigue damage and a higher risk of failure by sudden brittle fracture.

Based on the above mentioned facts, it can be concluded that the survey prepared by SUDOP as part of the documentation [1] is of a good standard in terms of content and detail. The alternative intervention procedure proposed in the SUDOP project [1], i.e. total replacement of the steel load-bearing structure, is based on relevant information. Replacement of the steel elements of all 3 arch truss bridges would affect nearly two-thirds of all bridge components. Dismantling and reassembling the structure to this extent and quantity in order to preserve some elements would be technically impracticable. In this situation, this proposal can be supported in this particular case and further accelerated development can be recommended, especially due to the increased risk of sudden breaching by brittle fracture. In the case of a new structure and a requirement for the same or similar appearance, a “looser form of replica” may be recommended, as the existing structural arrangement of details of the load-bearing structure and elements cannot provide sufficient corrosion protection even after its reconstruction (in the form of an exact replica) due to difficult-to-maintain details (joints of segmented cross-sections) and freely inaccessible parts (the lower chord of the lower flange, parts of the bridge in the area of the traction catenary lines).

Due to the established condition of the load-bearing structure of the bridge, it is necessary to reconstruct the bridge in the shortest possible time frame. The limiting time for reconstruction within 5 years given by SUDOP is “borderline” in our view, mainly because of the increased risk of sudden failure by brittle fracture, the advanced development of localised corrosion and the approaching end of the fatigue life of the rail-track elements.

Until the bridge is reconstructed, more frequent monitoring of the load-bearing structure for possible damage development should take place. We recommend to carry out a visual inspection of the load-bearing structure at least twice a year.

During the inspection, it is necessary to focus more closely on areas with advanced development of local corrosion damage where cracks are most likely to develop. In addition, the rail-track should be monitored in more detail for possible cracking due to the approaching end of fatigue life (fatigue life determined on the basis of the SUDOP recalculation [1]). Any newly detected faults must be consulted immediately with experts.

6 LIST OF ANNEXES

ANNEX 1 – Photographic Documentation

ANNEX 2 – Element Cards

ANNEX 3 – Material Tests

ANNEX 4 – Corrosion Analysis

The conclusions presented in this report have been formulated on the basis of the documentation provided and the results of our own diagnostic work carried out in certain areas, i.e. the findings of visual inspections and laboratory analyses. The author reserves the right to make corrections and additions to the conclusions if additional material facts are discovered which are beyond the scope of the diagnostics performed or are subsequently discovered outside the scope of the work performed and commissioned, or were unknown to the author at the time the report was prepared, or were falsely communicated to the author or withheld from the author.