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DRAFT PD for discussion

Change number:	*Content of the change:*	*Date of change:*
01	-	-
02	-	-
03	-	-

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Designation of the project: **RECONSTRUCTION OF POD VYŠEHRADEM RAILWAY BRIDGES**	*Contract number:* 16 354 201 *Project level:* PD	
Part: CONSTRUCTION PART BRIDGES, CULVERTS AND WALLS	*Date:* 04/2020 *Part number:* E.1.4	
Name of annex: **TECHNICAL REPORT**	*Scale:* -	*Number of formats:* 101 x A 4
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"Reconstruction of Pod Vyšehradem Railway Bridges"

Preparatory documentation 05/2018 – to be discussed

SO 20-20-04 Railway bridge at km 3.545 – Výtoň
SO 20-20-05 Railway bridge at km 3.706 – Pod Vyšehradem
SO 20-20-05.1 Railway line at km 3.706 – footbridges
SO 20-20-05.2 Railway at km 3.706 – day beacons SO 20-20-05.3 Railway at km 3.706 – securing the bottom at the piers

Technical Report – Bridge Structures

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1. INTRODUCTORY DATA

1.1 Construction identification data

Name of the construction: **"Reconstruction of Pod Vyšehradem Railway Bridges** "**(Construction 2**) section within the Reconstruction of the Line Praha hl. n. (excl.) – Praha-Smíchov (incl.)

Level of documentation: Preparatory Documentation (PD) / Documentation for Zoning Decision (DZD) and Project Plan (PP)

Characteristics of the construction: Linear railway construction, modernisation of the railway line

ISPROFIN number: **511 352 0019**

Client's CfW number: E618-S-12006/2016/Šim

Contractor's CfW number: 16 354 201

Place of construction: Railway line 0201 Praha hl. n. – Praha-Smíchov

Railway line 1703 Praha-Vršovice os. n. – Praha-Vyšehrad

The line according to the Railway Declaration 2017 PPraha hl. n. – Praha-Smíchov (as per KJŘ 171 Praha – Beroun)

Praha-Vršovice – Praha-Vyšehrad (as per KJŘ 122 Praha – Hostivice –

Rudná u Prahy)

both lines are part of the national railway of European importance (E)

Region: City City of Prague

Municipality / Municipal district: Prague 2, Prague 5

Cadastral territory: cadastral territory of Vyšehrad, cadastral territory of Smíchov

Designated municipal authorities: Prague 2, Prague 5

Municipalities with extended powers: Capital City of Prague

Start of construction: km Reconstruction of the Line Praha hl. n. (excl.) – Praha-Smíchov (incl.) at km 3.500 of the stationing

End of construction: at km 3.850 of the stationing

Date of documentation preparation: **May 2018** (DRAFT PD for discussion)

1.2 Identification Data of the Client (Builder)

Client: **Správa železniční dopravní cesty, státní organizace**
with registered office: Prague 1003/7, 110 00, Nové Město, Dlážděná 1/7, 110 00
Registered in the Commercial Register maintained by the Municipal Court in Prague, Section A, Insert 48384
ID No.: 70994234, VAT No.: CZ70994234

Organisational component for the Client: **Civil Engineering Administration West**
Sokolovská 278/1955
190 00 Prague 9

Supervising authority: **Ministry of Transport**
Nábřeží L. Svobody 12
110 00 Praha 1

for the investor in technical matters: Ing. Petr Vaníček, SŽDC, s.o., Construction Administration West

1.3 Identification Data of the Documentation Author

Author: **“SP+MTP+SPEU_Praha hl. – Praha-Smíchov“**

established by the Company Agreement dated 04/08/ 2016
Partners of the Company
Company name: **SUDOP PRAHA a.s.**
Registered Office: Prague 3, Žižkov, Olšanská 2643/1a, 130 00
ID No.: 25793349, VAT No.: CZ25793349
and
Company name: **METROPROJEKT a.s.**
and
Company name: **SUDOP EU a.s.**

Chief Project Engineer: Chief Project Engineer: Ing. Michal Mečl
AI in the field of transport construction No. 0009519

Chief Project Engineer (Construction 2): Ing. Tomáš Martinek, SUDOP PRAHA, a.s.

Responsible designer of the structure: Ing. Martin Vlasák, SUDOP PRAHA, a.s.
AI in the field of Bridges and IK and in the field of Transportation Construction
ČKAIT No. 0009271

Cooperation: Ing. Jaroslav Voříšek, SUDOP PRAHA, a.s.
Bc. Filip Kramoliš, SUDOP PRAHA, a.s.

1.4 Identification Data of the Structure

Bridge name, structure number:

SO 20-20-04 Pod Vyšehradem Bridges, railway bridge at km 3.545 – Výtoň

SO 20-20-05 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem

SO 20-20-05.1 Pod Vyšehradem Bridges, railway bridge at km 3.706 – footbridges

SO 20-20-05.2 Pod Vyšehradem Bridges, railway bridge at km 3.706 – day beacons

SO 20-20-05.3 Pod Vyšehradem Bridges, railway bridge at km 3.706 – securing the bottom at the piers

Common name SO 20-20-04 Výtoň

SO 20-20-05 Pod Vyšehradem

Track section: LS 0201 Praha hl. n. (excl.) – Praha- Smíchov (excl.)

Definitional section: DÚ 04 Praha- Vyšehrad – Praha- Smíchov

(station section)

View of the railway bridge over the Vltava River in the direction of Smíchov

2. Purpose of Construction

2.1 Subject of the Preparatory Documentation and Project Plan

The subject of the contract is the preparation of the preparatory documentation and project plan for the project “Reconstruction of the Line Praha hl. n. (excl.) – Praha-Smíchov (incl.)" (hereinafter referred to as the PD and the PP), based on the prepared and approved feasibility study in the variant **Centre 1.1 SH**, in the following scope:

- reconstruction of all tracks along the entire length of the construction, with the track layout respecting the design from the feasibility study,
- The solution will allow future modification to the condition of **CENTRE 2.1** according to the approved feasibility study with prospective triple-tracking of the section from the newly located junction of the Praha-Smíchov railway station around Vyšehrad in the direction of the Praha-Smíchov railway station,
- noise and vibration mitigation measures will be part of the design of the railway superstructure,
- the railway superstructure will be designed in accordance with Directive No. 28/2005 of the SCZC Directorate General on sleepers with flexible baseless fastening, with emphasis on noise and vibration reduction.

The main objective of the project is the reconstruction of the railway bridge under Vyšehrad. The existing railway structures and equipment will be brought to a construction and operational condition that corresponds to the current required technical parameters for increasing the capacity, efficiency and safety of railway operation within the framework of the complete reconstruction of the line.

The objective must be the most appropriate technical and economic solution that will be “**negotiable**" in the area of interest.

The project includes technical proofs of the possibility of building a new Praha – Výtoň stop for both double-track and triple-track variants of the track design.

3. Underlying Materials

3.1 Initial Underlying Materials Submitted by the Investor, according to the CfW

[1. 1] Tender documentation dated 04/02/2016, updated 20/04/2016,
[1. 2] Feasibility study of the TRC III to the railway junction Prague (contractor SUDOP PRAHA a.s., update 2015), discussed and approved by the Ministry of Transport at the meeting of the Central Commission on 18 September 2015 with the recommendation of the Centre 1.1 SH variant,
[1.3] Assessment Report No.: 13 224/2015-SŽDC-SSZ-ÚTl-Frk of 18 August 2015,
[1.4] Approval Report No. 50705/2015-SŽDC-07,

3.2 LITERATURE USED

[2.1] Ocelové konstrukce 20, Zatížení staveb, ČVUT 1999
[2.2] Ocelářské tabulky, ČVUT 1995
[2.3] Závěrečná zpráva projektu COST CZ – LD15127 – Pokročilé metody posuzování degradovaných ocelových konstrukcí, ČVUT in Prague, 2017
[2.4] GARCÍA M. O. The Impact of the Connection Stiffness on the Behaviour of a Historical Steel Railway Bridge. Thesis. Faculty of Civil Engineering, ČVUT in Prague, 2017.
[2.5] Statický přepočet mostu km 41,791 trati Tábor – Písek včetně návrhu řešení opravy, SUDOP PRAHA a.s., 2015
[2.6] Ekvivalentní rozkmit napětí železničních mostů, Dizertační práce, Ing. L. Žemličkové, ČVUT in Prague, 2004.
[2.7] Prof. L. Frýba, Dynamika železničních mostů, Academia, 1992, ISBN 80-200-0262-6
[2.8] Statický přepočet mostu km 41,791 trati Tábor – Písek, včetně návrhu řešení opravy, TP, SUDOP PRAHA a.s., 2014
[2.9] Interakce koleje a mostů s velkými dilatačními délkami – závěrečná zpráva, VUT v Brně, doc. Ing. Otto Plášek, 2015
[2.10] Preliminary report from the SŽDC s.o. project „Pokročilé metody posuzování existujících ocelových mostů na účinky zatížení větrem, brzdných a rozjezdových sil“, ČVUT in Prague, 2018

3.3 ARCHIVAL DOCUMENTATION AND OTHER DOCUMENTS

[4.1] Archival documentation of the substructure from 1872
[4.2] Archival documentation of the substructure from 1900, including the construction of the steel caissons
[4.3] Archival documentation of the structure from 1900, main drawings including material distribution, Brüder Prašil & Co.
[4.4] Archival documentation of the upper stiffening replacement from 1969, North-Western Railway in Prague, Design Office Ústí n. Labem,
[4.5] Static recalculation of the bridge at km 3.706, Annex C.1, TOPCON servis s.r.o., 2004
[4.6] Fischer J., Fischer O. – Pražské mosty, 1985,
[4.7] Soukup J. – Obrazy z pražských břehů a vod, Díl I., Pražské mosty, 1904

Note: archive documentation for the rail-track modifications (stiffening of the rail-track, braking stiffener) from 1987 has not been found

3.4 TRACK AND BRIDGE DOCUMENTS

[5.1] Report on Detailed Inspection of the Bridge at km 3.706, SŽDC, 2014
[5.2] Report on Detailed Inspection of the Bridge at km 3.706, SŽDC, 2017
[5.3] Survey of corrosion weakening of OK and substructure, SUDOP PRAHA a.s., 2017
[5.4] Static and dynamic verification load test, ČVUT in Prague, 2017
[5.5] Static and dynamic verification load test, Evaluation by radar interferometry, Vintegra s.r.o., 2017

[5.6] Monitoring of vertical deformations of the Pod Vyšehradem Bridge during normal operation, Vintegra s.r.o., 2017
[5.7] Material testing of steel, FERMET CZ, 2017
[5.8] Annual Report of the Czechoslovak State Railways for 1946. Prague: Josef Pacl, 1947
[5.9] Eisenbahn Verkehrs Jahrbuch 1917. Wien: Compassverlag, 1917
[5.10] Transport statistics of the Czechoslovak State Railways for the year 1928. Prague: Josef Pacl, 1929
[5.11] Yearbook of State and Private Railways 1920. Prague, 1921

4. Existing Condition of the Bridge

4.1 Description of the Existing Bridge

4.1.1 SO 20-20-04 Pod Vyšehradem Bridges, Railway Bridge at km 3.545 – Výtoň

Type of supporting structure	beam, steel, riveted, plate girder with recessed element rail-track
Description of the substructure including wings	stone abutments, stone piers foundation of the area on wooden piles (P3) without wings (connecting bridges)
Number of bridge openings	4
Number of tracks	2
Length of bridging	76.735 m
Bridge length	80.33 m
Span of the supporting structure	18.88 m under track Nos 1 and 2
Construction height	1.345 m to the LT under track Nos. 1 and 2
Decisive height of the railway bed contour	flat-laid bridge deck (vertical bolt) (building without railway bed)
Free height under the bridge	4.04 m – road (according to the survey)
Perpendicular hole clearance	bay 1 17.484 m bay 2 17.574 m bay 3 17.365 m bay 4 17.115 m
Bridge inclination (right/left, angle of inclination)	90°
Angle of crossing with a bridged obstacle	approx. 80°
Oblique opening clearance	19.14 m
Bridge width	9.950 m (outside the railing)
Year of construction (production)	NK: 1901 under tracks Nos. 1 and 2 O01: 1871 (modifications 1901 to 1907) P01: 1901 P02: 1901 P03: 1871 (modifications 1901) O02: 1901 (modifications 1901)
Year of last reconstruction or repair of the structure	1997 repair 1998 renewal of paint
Load data to date:	$\mathbf{Z_{LM71} = 0.82}$ (NK1 - NK3, main beam – bending) $\mathbf{Z_{LM71} = 0.95}$ (NK4, main beam – bending)
Construction condition of the structure	load-bearing structure - **Grade 2** substructure - **Grade 2**

4.1.2 SO 20-20-05 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem

Type of supporting structure	steel riveted parabolic truss with lower element bridge common for both converted tracks
Description of the substructure including wings	stone abutments, stone piers surface foundation (P02 and P03 on caisson) on wooden piles (O02) stone wings parallel and perpendicular at O02

Number of bridge openings		3
Number of tracks		2
Length of bridging		215.550 m
Bridge length		234.450 m
Span of the supporting structure		71.72 m under track Nos. 1 and 2
Construction height		1.380 m (to LT) under track Nos. 1 and 2
Decisive height of the railway bed contour		flat-laid bridge deck (vertical bolt) (building without railway bed)
Free height under the bridge		3.74 m (right bank pavement) 7.73 m (Vltava – max. navigation level 188.28 m asl Bpv
Perpendicular hole clearance	Hole 1	69.045 m
	Hole 2	69.145 m
	Hole 3	69.450 m
Bridge inclination (right/left, angle of inclination)		90°
Angle of crossing with a bridged obstacle		approx. 80°
Bridge width		13,580 m (including footbridge brackets)
Clear width on the bridge:		8,108 (between portal perpendiculars)
Year of construction (production)		NK: 1901 (RZ 1901) O01: 1901 (RZ 1901) P01: 1901 (RZ 1901) P02: 1901 (RZ 1901) O02: 1871 (modifications 1901)
Year of last reconstruction or repair of the structure		1987 repair (MES) 1957 renewal of paint (MES) 1912 repair of the substructure (MES)
Load data to date:		Z_{LM71} **= 0,61** (P1 to P8 – common supporting cross bars)
Construction condition of the structure		load-bearing structure - **Grade 3** substructure - **Grade 2**
Bridge equipment:		footbridge sidewalk brackets are administered and owned by the Capital City of Prague (SO 20-20-5.1) The day beacons, including lighting, are managed by and owned by Povodí Vltavy s.p.
River km:		Vltava river km 55.35 (data from SPS Praha)

4.2 Existing Spatial Organisation

In the area of the foreland is

The following data are from the record of the REPORT ON DETAILED INSPECTION OF THE BRIDGE 2017:

Transverse displacement of the track axis relative to the NK axis:

NK	beginning	end	Note

K01	+22	+42	Track No.1/Track No.2
K02	+39/+22	+17+24	
K03	+22/+25	+5/-19	
K04	+6/+20	-2/+30	

NK	beginning	end	Note
K01	-12/+12	+1/+10	Track No.1/Track No.2
K02	0/-1.5	-2/0	
K04	+7/+4	-2/+8	

- left/ + right

Distance of the handrail/barrier from the track axis:

NK	beginning left	right	end left	right	Note
K01	2,980	2,970	/	/	railings
K02	/	/	/	/	railings
K03	/	/	/	/	railings
K04	/	/	2,940	2,920	railings

NK	beginning left	right	end left	right	Note
K01	1,910	1,900	1,920	1,910	perpendicular – corner reinforcement
K02	1,910	1,900	1,910	1,910	perpendicular – corner reinforcement
K03	1,910	1,910	1,900	1,900	perpendicular – corner reinforcement

The distance of the end portal perpendicular to the track axis is 2,154 mm in the basic value. The distance from the inner perpendiculars is 2,188 mm in the basic value. Due to the eccentricity of the track axis, see table above, the minimum perpendicular distance from the track axis is **2,142 mm** for track No.1 and **2,144 mm** for track No.2.

In terms of the minimum requirements in the station according to DG Directive 16/2005, the section of the bridge over the Vltava River (SO 20-20-05) **does not comply with VMP 2.5** or **MPP 2.2** according to the original ČSN 73 6201 (status before the change in 2008). In terms of spatial clearance, the bridge **is suitable for the Z-GC cross-section**.
There are recesses in the truss structure with a minimum depth of 0.5 m for the movement of persons along the track

4.3 Bridge Description – General

4.3.1 Historical Development of the Pod Vyšehradem Bridges

The area of the bridge structure, which is currently made up of five bridges, underwent a major transformation during its development in the late 19th and early 20th centuries.

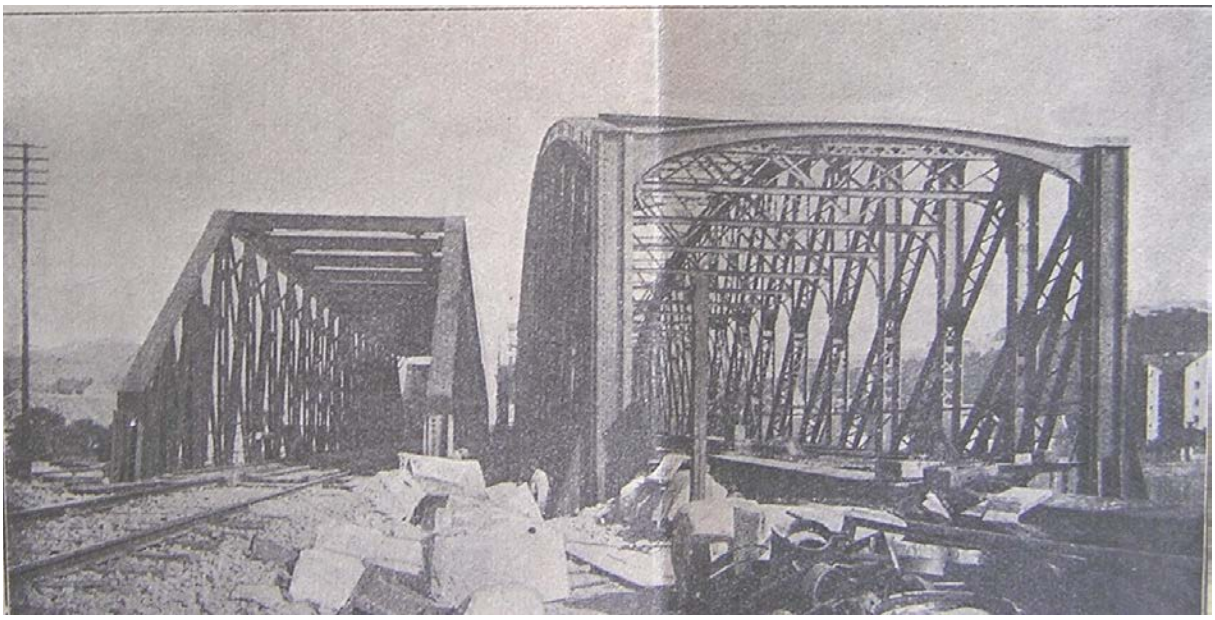
The construction of the original Pod Vyšehradem Railway Bridges took place in 1871. The bridge structures were part of the so-called Connecting Railway, which connected the then Czech Western Railway (Praha-Smíchov – Plzeň) with the Railway of Emperor Franz Joseph I (Vienna – České Velenice – Praha).
Historically, the first bridge in this section was built on a single-track line in 1871. Behind the Praha-Vyšehrad railway station, the line ran along a stone arched foreland, which was divided by a bridge for bridging Vyšehradská street with a steel riveted bearing structure. The main bridge over the Vltava River had five bridge openings with a span of 56.9 m. The single-track superstructure was a truss closed straight-span multiple trapezoidal system with inclined end portals, see photo.

Vltava River bridge from 1874 (photo by F. Fridrich)

After a flood in 1891, the right bank was provided with an embankment wall with a floating depot. Together with these modifications of the entire bank, the Botič stream, which passed through the first arched bridge opening behind the Praha-Vyšehrad railway station, was relocated along the left side of the bridge structure. In the given section from Neklanova street was covered with the use as a local road Vnislavova street. The roofing was later extended further to the crossing of the brook with Na Slupi street. The original Botič stream bed was also used as a local road, Svobodova street. The riverbed modifications continued until 1910. With the development of rail transport, there was a need to build a higher capacity transport connection.

First, in 1901, the load-bearing structures of the bridge over the Vltava River, including the foreland section, were replaced by double tracks and then the stone arched section for the second track was extended until 1907
(track No. 1).
A new bridge structure was built over the Vltava River with three bridge openings and a right bank Výtoň foreland with four bridge openings. The design of the structures was carried out by the Prášil Brothers Bridgeworks. The production and delivery of steel structures was provided by the Prášil Brothers

Bridgeworks, První Českomoravské strojírny and Pražské akciové strojírny. The Výtoň foreland was provided by Těšínské železárny.



Vltava River bridge from 1901 – before the replacement

Only one track was put into operation (today's track No. 2). The second track (today's track No.1) was opened in 1907 after the completion of the arched flyover.

In the contemporary publication Prague Bridges, by Jiří Soukup, the aesthetics of the bridge is not perceived very positively, see below:

The total cost of construction was about 4,000,000 K.
We do not intend here to criticise the aesthetic aspect of this bridge; we would have preferred to see a neater construction in the bosom of a large city; it is difficult to prescribe everything to the railways, and besides, a semi-parabolic construction is always, according to present experience, relatively very cheap.

excerpt from contemporary press (Epochy, Jiří Soukup, 1904

Within the framework of the electrification of the railway network, in 1969–1970 the upper stiffening including the end portals was modified on the load-bearing structures. The catenary brackets were attached directly to the perpendicular sections of the main beam. In terms of the aesthetic and static effect of the bridge, these modifications were very insensitive.

More extensive structural modifications were made to the rail-track in 1987, when the longitudinal trusses were strengthened, sub-bridge stiffening and braking stiffening were added at the edges and in the centre of the structure. Structurally and statically, the design of the braking stiffening is not optimal in terms of limiting the transverse stresses on the cross members and longitudinal trusses from the interaction of the rail-track with the main bearing system.

The Výtoň forebay (SO 20-20-04) was comprehensively reconstructed in 1997–1998, when the longitudinal trusses were reinforced and the lower chord was replaced in the structure in the 4th bay. For NK1 and NK3 under track No.1, the original lower angles of the longitudinal trusses have been retained.

Since December 2004, the set of bridge structures of the Vltava River bridging has been a cultural immovable monument "Railway Bridge – a set of railway bridges on the line Praha hl. n. – Praha Smíchov" (CLCM Reg. No. 101 315), which is part of the monument reservation.

4.3.2 Summary of Historical Events

1871 Connecting track with monorail Pod Vyšehradem Railway Bridges:

- Bridges over the Vltava River and the original delta of the Botič stream (in the place of today's SO 20-02-05 and SO 20-20-04):
- load-bearing structures: straight-chord steel trusses made of welded iron, L = 5 x 56.90 m,
- substructure: stone abutments and piers based on wooden grates and wooden piles,
- single-track structure, width ~4.30 m, substructure prepared for 2 tracks,
- Other right bank bridges (in place of today's SO 20-02-03, SO 20-20-02, SO 20-20-01):
- 8 stone arches + Vyšehradská avenue bridge + 5 stone arches,
- monorail bridges,
- stone masonry of Saxon sandstone, founded on wooden grids and wooden piles,
- steel truss structure over Vyšehradská avenue, perpendicular clearance approx. 11 m.

~1888 Beginning of passenger transport on the Connecting Railway, Pod Vyšehradem stop

1901 Reconstruction of bridges over the Vltava River (in the place of today's SO 20-02-05 and SO 20-20-04):

- new stone piers in the Vltava River (in place of SO 20-20-05), based on caissons,
- new stone piers on the right bank (in place of SO 20-20-04), based on wooden piles,
- the Botič delta is limited by the curbing and raising of the terrain at the new piers,
- temporary trestles in the Vltava River, founded on wooden piles,
- new NK over the Vltava River: steel truss with parabolic top flange, L = 3 x 71.72 m,
- (production took place in three bridgeworks)
- NK width ~8.10m for 2 tracks + double-sided cantilevers ~2.32m with footbridges,
- new NK on the right bank: steel beams, L = 4x ~18.9 m,
- construction of new NK on temporary barges, cross-loading into the final position,
- grade line raised by approx. 0.5 m,
- Removal of the original piers and makeshift berms down to the river bottom.

~1901–1907 Extension of other bridges (in place of today's SO 20-02-03, SO 20-20-02, SO 20-20-01):

- newly built left (south) part of the arched bridges, material: granite (see IGP),
- foundations on concrete in concrete pits (according to archival documentation),
- reconstruction and extension of Vyšehradská avenue:
- New NK: steel beam plate girder riveted, perpendicular clearance approx. 19 m,
- construction of new abutments partly in the last opening of the adjacent arch bridges, these openings completely walled up with foundation on wooden piles, arches demolished and connected to the new abutments, original abutments demolished,
- left (southern) part of the bridges: atypical small arches, abutments connected to the right (northern) part.

~1901–1910 so-called Boticbachcorrection – relocation of the original Botič:

- new Botič to the left (south) of the railway bridges, bridged by a reinforced concrete structure on stone abutments under today's Vnislavova Street, under the floating depot, flows into the Vltava,
- the original Botič to the right (north) of the railway bridges and under the 1st arch filled in,
- the original delta of the Botič stream is disappearing with the construction of the floating depot, Rašín's Embankment and the rise of the terrain.

1902–1905 Construction of the tunnel under Vyšehrad Rock

~1904–1905 Construction of Vyšehrad railway station
1907–1910 Construction of Rašín’s Embankment: retaining walls, floating depot, raising the terrain
1994 Replacement of NK through Vyšehradská street (SO 20-20-01): steel chamber, L = 21.00 m (type of temporary structure KN 21 with direct rail placement on the bridge
1997–1998 Reconstruction of the Výtoň Bridge (20-20-04)

2004 declaration of the cultural immovable monument "Railway bridge – a set of railway bridges on the line Praha hl. n. – Praha Smíchov"

4.3.3 DESCRIPTION OF BRIDGE STRUCTURES

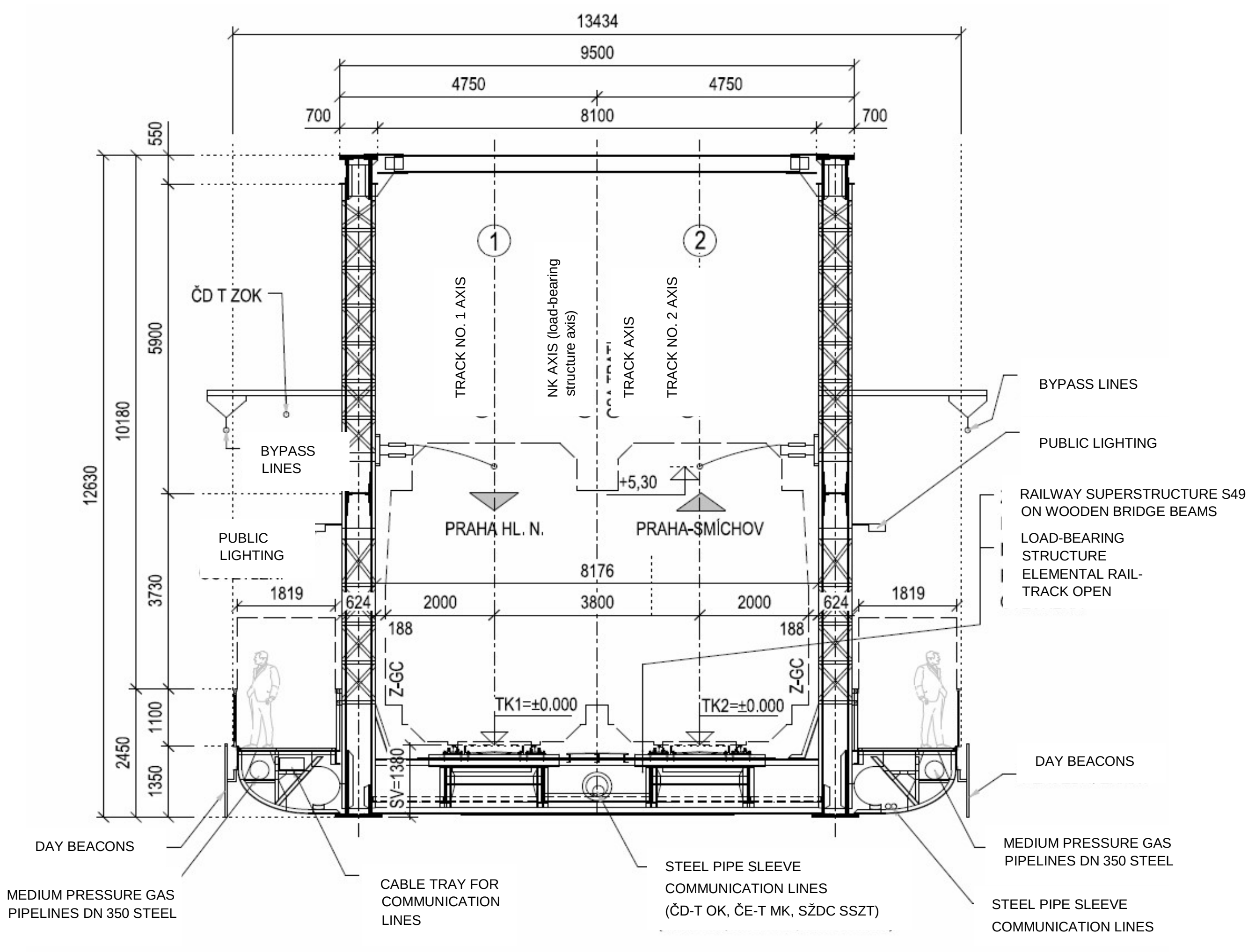
4.3.3.1 Pod Vyšehradem Bridge (km 3.706)

The railway bridge bridges the Vltava River with three bridge openings. The load-bearing structures were made in 1901 from plow steel. According to the 2015 Methodological Guideline, this is a steel with a guaranteed yield strength of 230 MPa, which corresponds approximately to today's S235JR steel.

The load-bearing structures are designed as closed truss multiple systems with a curved upper chord with a uniform span of 71.72 m, which was the most economical solution at the time. The structural arrangement of the bridge corresponded to the time of its construction and the effort to reduce the weight of the structure. The individual sections are graded according to the stresses expected at the time. The details of the truss rods were not designed with regard to the risk of steel corrosion in case of failure of the corrosion protection (especially crevice corrosion). This problem particularly affects the lower chord, lacings and perpendiculars.

The bridge is double-lined with an open 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 at mid-span. The shape of the upper strip is polygonally broken in the place of the panel points. The riveted steel structure is divided into 16 trusses with lengths of 3.46 m + 4.0 m + 4.40 m and 5 x 4.80 m at mid-span.



Cross section in the centre of the span

The **upper belt** consists of a double-walled Π-shaped section with a wall-to-wall clearance of 416 mm. The wall has a constant height of 470 mm along the entire length of the strip and a constant thickness of 24 mm composed of a pair of 2 x 12 mm sheets. The progression of the increasing axial force is taken into account by changing the thickness of the upper chord, which is graded from a basic thickness of t1 = 10 mm in the first chord by 10 mm up to a thickness of 70 mm in the middle of the span. The connection between the wall and the upper chord is made by means of a pair of L 110x14 right angles for each wall and Ø 22 mm rivets in the walls and 24 mm in the chords. The shape of the cross-section of the rods of the upper chord is ensured by plate girder diaphragms riveted to the walls and chords in thirds of the span of the trusses.
The **lower chord** of the main beam has a double-walled open section in the shape of a pair ⊥ ⊥ (inverted T) with the same clearance as the upper flange, i.e. 416 mm with a constant height of 560 mm and a width of 410 mm. The connection of walls and chords is again by means of neck angles L 110x14 and rivets Ø 22 mm in walls and 24 mm in chords. The shape of the cross-section is ensured by plate girder diaphragms riveted to the walls at the mid-span of the trusses.

Lacings D1 to D3 have a segmented cross-section consisting of 2 quadrants of L 80 x 9 angles, each reinforced with a pair of P14 x 360 mm (D1), P14 x 400 mm (D2) and P12 x 340 (D3) sheet metal chords.
The **lacing D4** has a segmented cross-section, which consists of two quadrants of L 80 x 8 angles reinforced again by a pair of P10 x 320 mm sheet metal chords.
Lacings D5 to D7 also have a segmented cross-section made of only two quadrants L 90 x 130 x 12 (D5), L 80 x 120 x 11 (D6) and L 80 x 100 x 10 (D7) without chords.

The **cross-section of the lacings D8 to D10** is made up of only pairs of angles L 80 x 100 x 13 (D8), L 90 x 10 (D9) and L 80 x 8 (D10).

The connection of quadrants and pairs of angles into a segmented cross-section is ensured by truss connectors made of P8 x 60 mm strip. The connection to both strips is made by riveted joints with Ø 22 mm rivets.

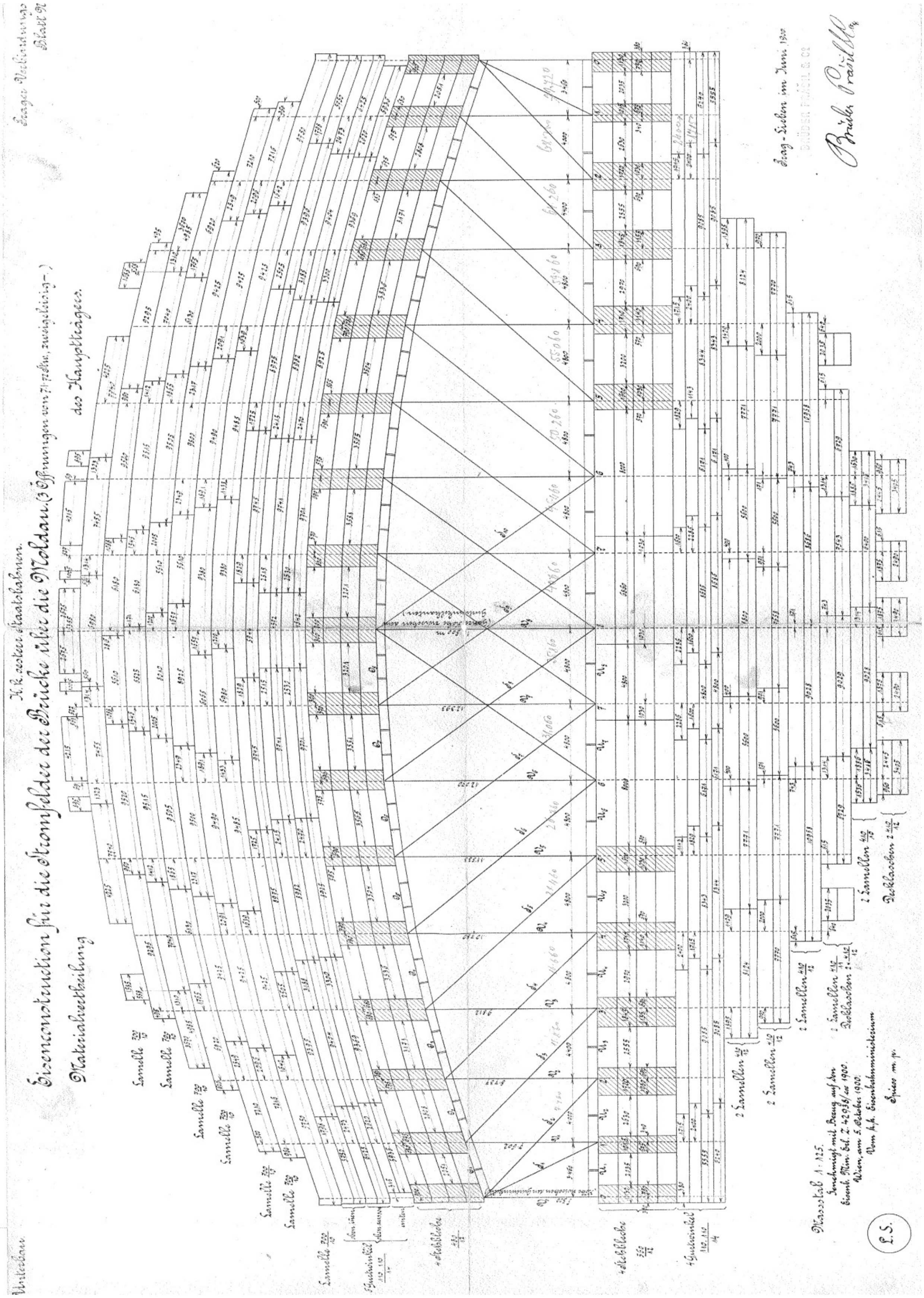
The **inner perpendiculars V1 to V8** are of similar design to the central lacings. **Verticals V1 to V3** have a segmented cross-section consisting of two quadrants of angles L 100 x 150 x 14 mm (V1), L 90 x 130 x 14 mm (V2) and L 90 x 130 x 11 (V3) connected to the segmented cross-section by truss connectors made of P13 x 60 mm strip. **Verticals V4 to V8** have a segmented cross-section consisting of two pairs of angles L 100 x 150 x 14 mm (V4), L 90 x 130 x 14 mm (V5) and L 80 x 120 x 12 (V6), L 80 x 100 x 12 mm (V7), L 80 x 10 mm (V8), which are connected by truss connectors made of P13 x 60 mm strip.

The connection of the perpendiculars to the two strips by means of Ø 20 mm rivets is made through the splice plates, which form the diaphragms of the cross-section of the lower and upper strip. At the bottom flange, the splice plate of the frame corner of the crossbar is connected to the perpendicular.

The cross-section of the extreme **portal perpendicular V0** consists of 16 angles L 100 x 12 mm folded into the H section. The chords are always made up of 8 angles and a filler plate 2 x P12 x 600 mm. The straps are connected along the entire height by a 13 mm thick vertical plate. On the front side there is a covering (fairing) made of P8 sheet and on the inside there is a connection of the chords with truss connectors into a segmented rod. The gantry perpendiculars create a structural chamber cross-section but with little torsional stiffness.

The **lower elemental rail-track** consists of cross members and inserted unconnected longitudinal trusses. The **longitudinal trusses** have a differentiated cross-section due to the different lengths of the trusses. **Longitudinal truss L1** in truss 1 consists of a P10 x 690 mm sheet metal wall. The straps are made of a pair of angles L 80 x 8 mm. The connection of the angles to the wall is made with rivets Ø 20 mm with 120 mm spacing. The longitudinal truss was reinforced in 1987 by the addition of chords. The upper chord was reinforced with P10 x 250 plate with holes for the vertical bridge bolt and the lower chord with P10 x 190 mm plate. The L2 longitudinal in the 2nd truss has the same arrangement but with L 80 x 10 mm neck angles and the **L3 longitudinal** in the 3rd truss with L 90 x 10 mm angles.

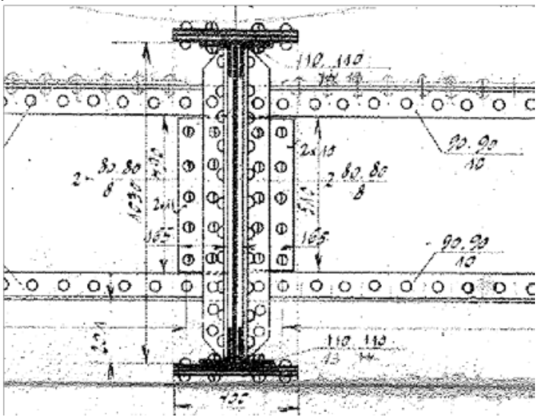
The **longitudinal trusses** in the other trusses have a reinforcement of P10 x 280 mm plate for the upper and P10 x 220 for the lower s. in the other trusses have a reinforcement of P10 x 280 mm plate for the upper and P10 x 220 for the lower chord. The neck angles are made of L100 x 12.



Eisenconstruction für die Stromfelder der Brücke über die Moldau.
Materialvertheilung
des Hauptträgers.

Diagram of the grading of the chords of the lower and upper chords of the main beam (archive documentation) **The supporting cross bars** also have a differentiated cross-section with the grading of the chords towards the main beams. The **outermost cross member P0** consists of a P13 x 1030 mm wall connected to 2 x P10 x 400 mm sheet metal chords by L 110 x 13 neck angles. The neck rivets have a section Ø 22 with a pitch of 100 mm. This cross-section has a supporting cross bar in the central part and under the inner longitudinal trusses. It is graded to a P10 x 400 profile at the outer longitudinal truss up to the connection to the main beam. The cross-sections of the cross-sections P1 and P2 differ in the thickness of the chords, which is 26 mm thick in the central part made of P12 + P14 x 400 mm sheets and in the outermost part it transitions into a cross-section with one chord made of P14 x 400 mm sheets.

Crossbars P3 to P8 have a 30 mm thick chord made of three P10 x 400 mm plates and L 110 x 13 mm neck angles. The neck rivets have the same section Ø 22 with a pitch of 100 mm. The grading of the supporting cross bar chord is towards the main beam, i.e. under the outer longitudinal trusses the grading is to the 2 x P10 chord section in connection to the main beam to the 1 x P10 chord section.
The mounting joints of all crossbars consist of 27 rivets Ø 22 mm arranged in 3 rows of 9 rivets. Connection of the crossbars to the main beams is by rivets Ø 20 mm.
The connection of the longitudinal trusses to the wall of the supporting cross bar is identical for all longitudinal trusses and is solved by a pair of connecting angles L 80 x 8, which connect the walls of the supporting cross bar and the longitudinal truss. The number of rivets in the longitudinal truss wall is 7, the outermost rivets are Ø 20 mm and the inner ones are Ø 22 mm. The joint is completed by 18 rivets in the supporting cross bar wall Ø 22 mm.



The connection of the longitudinal truss to the supporting cross bar (longitudinal cut along the longitudinal truss axis)

As part of the strengthening of the longitudinal trusses in 1987, the longitudinal trusses were supplemented with bridge stiffening and a braking stiffener. The stiffener was located in the centre of the bearing structure and to the edges of the 2nd truss.
The perimeter sections of the braking stiffener are made of a pair of 2 x L 125 x12 mm angles. The inner lacings are made of the angle L 125 x 12 mm in the part between the longitudinal trusses or L 90 x 12 mm in the central part. The height of the braking stiffener truss is 1,700 mm in the 2nd truss and 2 x 1,600 mm in the 8th and 9th truss.
The **lower horizontal stiffening** is a composite system consisting of a pair of L 110 x 12 mm angles riveted together in trusses 1 to 4.
The lacings of the lower stiffening are made of 2 x L100 x 12 mm cross section in the 5th truss, 2 x L100 x 10 mm cross section in the 6th truss, 2 x L 90 x 10 mm in the 8th truss and 2 x L 80 x 10 mm in the 8th 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 by a mullion

at the level of the upper chord made of welded asymmetric I section with a P12 x 300 plate wall, a P14 x 180 mm bottom chord and a P14 x 200 mm upper chord.

The reconstruction included the outermost portals. The portal mullion is made of symmetrical welded I section with P14 x 600 mm plate wall and P20 x 300 mm chords.
The lacings of the upper horizontal stiffening are formed from a double-sided section of a pair of angles L 90 x 8 mm. Pedestrian bridge cantilevers are attached to both riveted steel structures with a clear width between the railings of 1,820 mm. The brackets are connected via the splice plate to the section of the perpendiculars and then to the lower chord of the lower chord. In the longitudinal direction, the cantilevers are connected on the outside by a continuous U-shaped cornice beam consisting of a P7 x 450 mm wall and 70 x 7 mm L angle chords. On the inner side there is a longitudinal truss of U-section of 260 mm height. The outer ledge longitudinal truss and the inner longitudinal truss are connected to each other in the middle of the trusses by an intermediate supporting cross bar.
The height of the railing on both sides of the footbridge is approximately 1,130 mm above the walking surface, which is made of wooden planks of thickness of 50 mm.

The 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.

From the archive documentation, the weight change during the service life of the bridge structure was traced. From the steel statements, an overview is created for the actual supporting structure and for the bridge equipment. The weight overview is an important basis for the static recalculation.

PART: LOAD-BEARING STRUCTURE – NK1

PERMANENTLY INSTALLED ITEMS – NET WEIGHT

YEAR OF MOUNTING	DESCRIPTION	STEEL QUALITY ACCORDING TO EN			DIMENSIONS	UNIT WEIGHT.	TOTAL WEIGHT
		Martin plow steel S235 JR+N	Castings and Roheisen's BEARINGS	RIVETS 4%	LENGTH		
					[m]	[kg/type]	[kg]
1901	ORIGINAL STEEL. OF STRUCTURE	488,210		19,528	71,720	7,079	507,738
1901	BEARINGS		16,218	649			16,867
1901	SIDEWALKS – CONSOLES, RAIL. AND THE LONGITUDINAL TRUSSES	35,962		1,438	71,720	521	37,400
1970	UPPER STIFFENING – DISASSEMBLED PARTS	-28,002		-1,120	71,720	-406	-29,122
1970	UPPER STIFFENING – ASSEMBLED PARTS	18,377		735	71,720	266	19,112
1987	RAIL-TRACK – REINFORCEMENT	15,641		626	71,720	227	16,266
1987	RAIL-TRACK – DISASSEMBLED PART	-6,452			71,720	-90	-6,452
TOTAL BY STEEL		523,735	16,218	21,856	71,720	**7,598**	**561,809**

PART: BRIDGE EQUIPMENT – NK1

PERMANENTLY INSTALLED ITEMS – NET WEIGHT

YEAR OF MOUNTING	DESCRIPTION	STEEL QUALITY ACCORDING TO EN			SIZE	UNIT WEIGHT	TOTAL WEIGHT
		Martin plow steel S235 JR+N	Castings and Roheisen's BEARINGS	RIVETS 4%	LENGTH		
					[m]	[kg/type]	[kg]
1987	FLOOR SHEETS + LONGITUDINAL TRUSSES	30,008		1,200	71,720	435	31,208
TOTAL BY STEEL		30,008	0	1,200	71,720	**435**	**31,208**

In its current state, one load-bearing structure of the bridge over the Vltava River weighs **593 t** including bridge equipment, which corresponds to **8.0 t.m^{-1}.**

4.3.3.2 Výtoň Bridge (km 3.545)

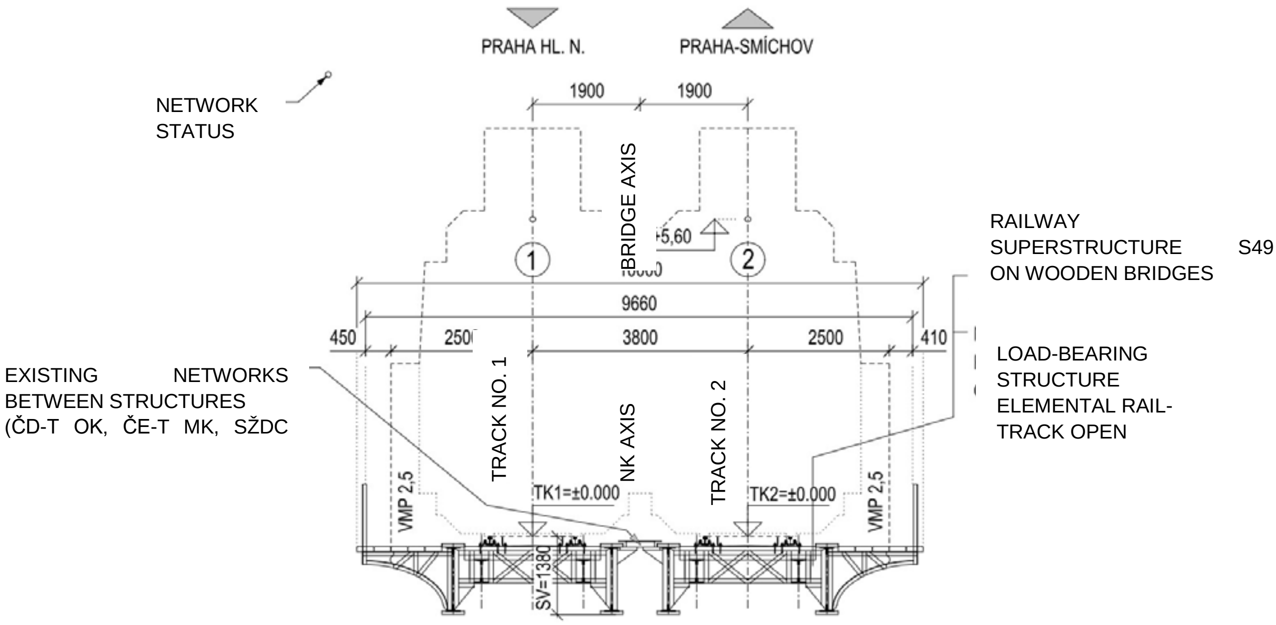
A railway bridge with four bridge openings bridges the local roads on the foreland. The load-bearing structures were made in 1901 from plow steel at the same time as the bridge over the Vltava River (SO 20-2005). According to the 2015 Methodological Guideline, this is a steel with a guaranteed yield strength of

230 MPa, which corresponds to today's S235JR steel. The supporting structure of the bridge is separate under each of the converted tracks.

The **main beams** are plate girder riveted with a span of 18.88 m in all four bridge openings.

The axial distance of the main beams is 2.80 m. The section of the main beam consists of a wall made of P 14 x 1130 sheet metal. Chords made of P10 x 400 sheet metal are connected with L 110 x13 neck angles. Chords of P10 sheet are gradually added according to the increasing bending stress towards the centre of the span, where it reaches a total thickness of 50 mm + 10 mm for overcladding.

As part of the reconstruction in 1997, the bottom chord of the supporting structure in hole 4 was replaced with P16 x 400 mm + P45 x 400 mm sheets (total thickness of the bottom chord 61 mm). The neck angles were replaced with L 160 x 100 x 14 with the addition of a welded plate 120 x 14 mm, which also forms the overcladding of the lower part of the main beam wall.



Sample Cross section in the centre of the span

The **intermediate elemental rail-track** is composed of longitudinal trusses and supporting cross bars. The **longitudinal trusses** are non-continuous at an axial distance of 1.80 m and are inserted between the supporting cross bars. The span of the longitudinal trusses is 2.36 m. The longitudinal truss consists of a wall made of P10 x 390 mm sheet metal and 2 x L 80 x 8 angle chords for the lower chord and 2 x L 70 x 8 for the upper chord, connected by Ø 20 mm neck rivets. As part of the reconstruction in 1997, the existing neck angles were replaced and the upper chord of the longitudinal trusses made of P16 x 250 mm sheet metal was added.

The **supporting cross bars** are in the truss structure connected to the main beams via splice plates that directly transfer the loads from the longitudinal trusses. The static function of the supporting cross bar is therefore rather stiffening, which is also reflected in the subtle sections used. The height of the intermediate supporting cross bars is 0.57 m. The chord of the supporting cross bar is made of a pair of angles L 80 x 10 mm. The supporting cross bars are 0.89 m high with chords made of a pair of L 80 x 10 mm angles.

The **bridge horizontal stiffening** of the rhombic system is made of angles L70x8 mm to L90x9 mm.

Outside the main beams, the **cantilevers of the inspection walkways** are connected. In openings 1 to 3 there is a walking surface of the inspection walkway made of wooden planks. In bay 4, the brackets were replaced with welded brackets with a walking surface made of sheet metal as part of the reconstruction.

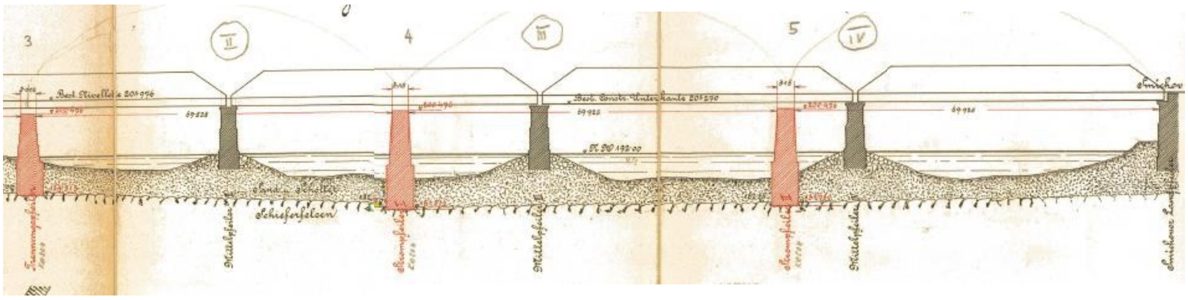
The **bearings** are tangential/plank bearings except for the fixed bearings in hole 4 on the OP2 abutment in connection with the bridge over the Vltava River, where the bearings are fixed rack bearings.

4.3.4 SUBSTRUCTURE

4.3.4.1 Pod Vyšehradem Bridge (km 3.706)

The substructure is solid of coursed rubble, with concrete infill. The method of foundation in the case of abutment O01and 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. The original shafts of the piers were demolished to the level of the Vltava River bed.

<= Praha hl. n. *Smíchov =>*



O01 P01 P02 O02

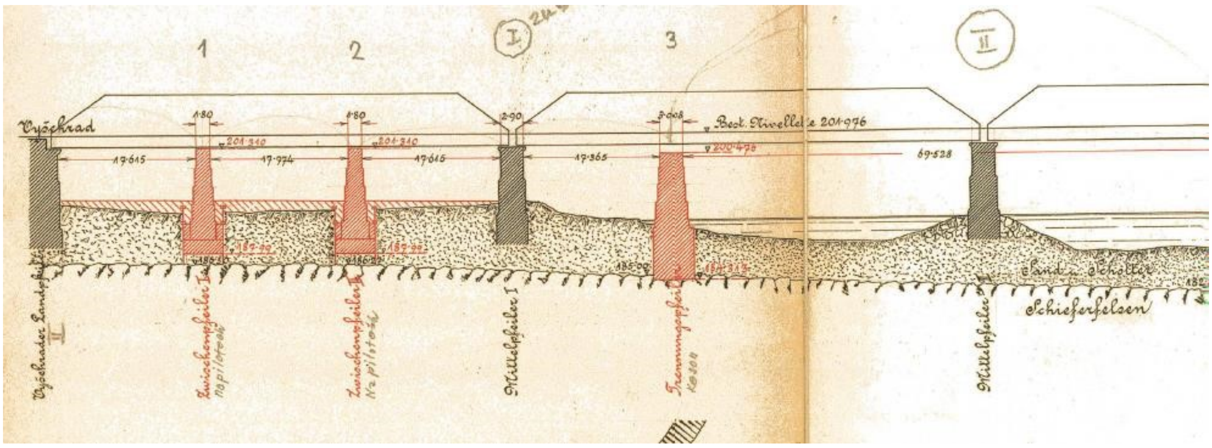
Reconstruction of the lower bridge structure at km 3.706

4.3.4.2 Výtoň Bridge (km 3.545)

The original sandstone stone arches were completed with vaults of more durable granite, but the O01 abutment was built as a double-track in 1871. The load on the abutment was originally considerably higher with the first span of the original 56.9 m bridge. Pier P03 was retained from the original bridge (originally P01). Pier P03 has been extended in shape. Piers P01 and P02 were newly built from granite row masonry.

<= Praha hl. n.

Smíchov =>



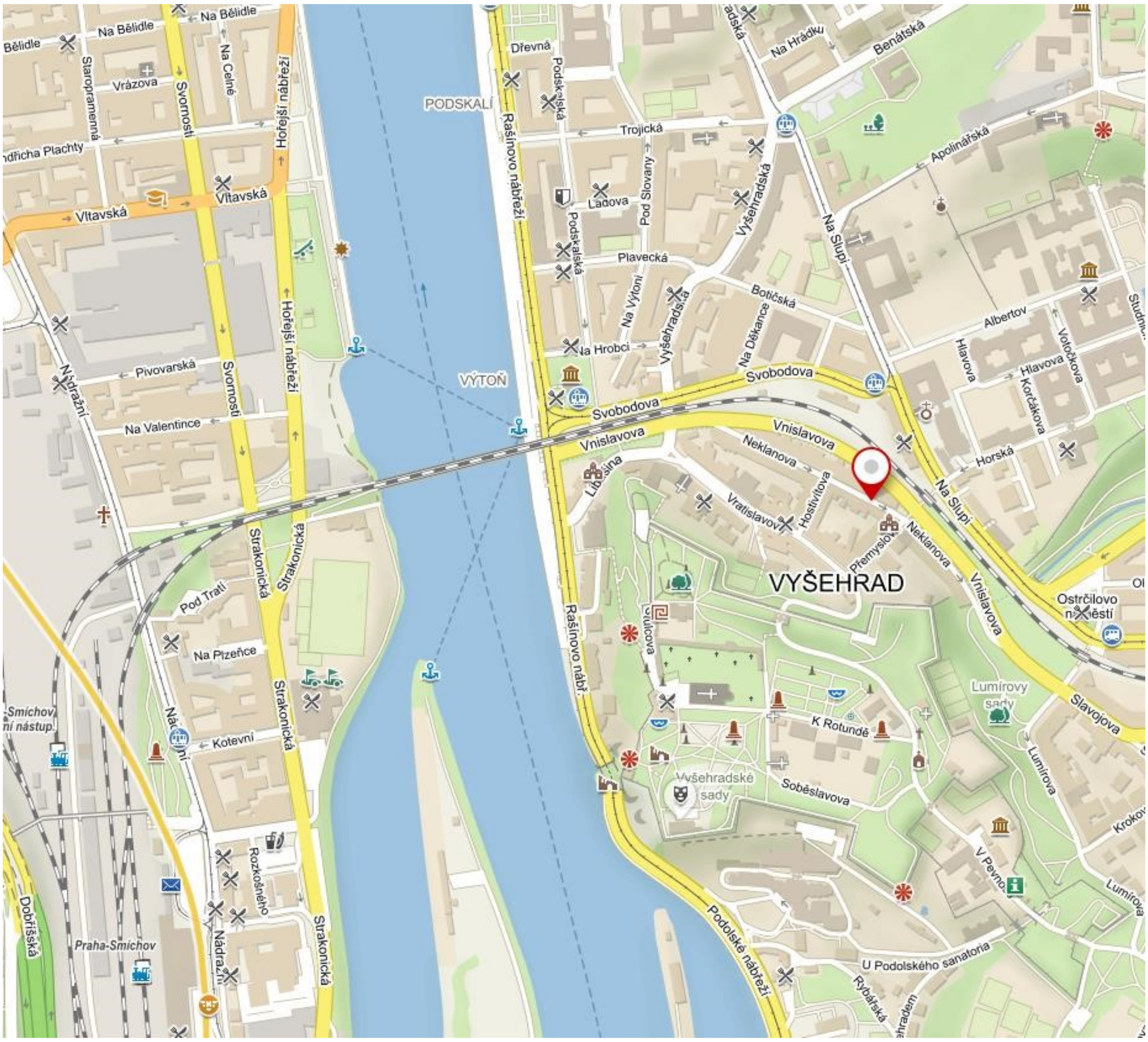
O01 P01 P02 P03 O02 (O01 3.706)

Reconstruction of the lower bridge structure at km 3.545

4.4 Territorial Conditions

The bridge is located on the southern edge of the centre of the City of Prague. For over 100 years, the bridge structure has been co-creating the panorama of Prague, both in the northern view of Prague Castle and in the southern view of the Basilica of St. Peter and Paul (Vyšehrad). The bridge structure is part of the conservation area. The design of the reconstruction of the bridge should be approached with this in mind.

The accesses to the bridge are given by the passage on local roads in the centre of Prague. The other option is the on-axis access along the track. It is also possible to use boat transport on the Vltava River to supply the construction with material.



Situation of the Vltava Railway Bridge

4.5 Existing Technical Condition of the Bridge

An extensive diagnostic survey was carried out to verify the current status. A detailed description is given in the following chapter.

The basic document that characterises the structural condition of the bridge structure is the Report on Detailed Inspection of the Bridge 2017. The conclusions are presented below.

4.5.1 Conclusions from the Report on Detailed Inspection of the Bridge 2017 (SŽDC, TÚDC)

4.5.1.1 Evaluation of the Bridge Superstructure at km 3.706:

Structure K 01 – Grade 3 rating

For the following reasons:

- 180 mm long transverse crack in in the upper chord in the 4th longitudinal truss between bridge beams Nos. 6 and 7
- Significant corrosion weakening of individual parts of the structure, corrosion of elements, increase in slice
- and crevice corrosion
- Individual loose and missing rivets
- Missing screws in bearing mounting, 3 pcs in total
- Damaged CPC structure
- Cracked and rotten bridge beams

Deterioration since PPM 2014 – transverse crack in the upper chord of the 4th longitudinal truss, multiple loose and missing rivets

Structure K 02 – Grade 3 rating
For the following reasons:

- In the upper chord of the 2nd longitudinal truss under bridge beam **No. 75 an oblique crack of approx. 550 mm** (under the bridge beam in the place of the splice plate)
- Significant corrosion weakening of individual parts of the structure, corrosion of elements, increase in slice
- and crevice corrosion
- Individual loose and missing rivets
- Missing screws in bearing mounting, 2 pcs in total
- Damaged CPC structure

Deterioration since PPM 2014 – oblique crack in the upper chord of the 2nd longitudinal truss, multiple missing bearing bolts

Structure K 03 – Grade 3 rating
For the following reasons:

- Transverse crack on the bottom plate of the left bearing on P 02 in the middle part
- Significant corrosion weakening of individual parts of the structure, corrosion of elements, increase in slice
- and crevice corrosion
- Individual loose and missing rivets
- Missing screws in the bearing mounting, 2 pcs in total, on the lower plate of the left bearing on P 02 in - central part of the transverse crack
- Damaged CPC structure

There has been a significant deterioration since PPM 2014

4.5.1.2 Evaluation of the Substructure of the Bridge at km 3.706:

Abutment O 01 – Grade 2 rating
For the following reasons:

- Individual water leaks with binder leachates through the masonry of the abutment
- Individually cracked and fallen masonry joints
- Displaced corner blocks and dropped jointing see PPM 2014 rehabilitated – new jointing in these places

Pier P 01 – Grade 2 rating
For the following reasons:

- Individual visible leaching of the binder through the masonry of the pier
- Cracked and individually fallen masonry joints in places

Pier P 02 – Grade 2 rating
For the following reasons:

- Block under left bearing K 03 on upper surface 2 x cracked

- Individual visible leaching of the binder through the masonry of the pier
- Cracked and individually fallen masonry joints in places

Abutment O 02 – Grade 2 rating

For the following reasons:

- Water leaks with binder leachates through the masonry of the abutment
- Locally cracked and fallen joints in the masonry of the support and wings
- Storage threshold heavily clogged with gravel and dirt

No visible deterioration since PPM 2014

4.5.1.3 Evaluation of the Bridge Superstructure at km 3.545:

Structure K 01 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Dents and grooves in the lower chord of the left main beam
- Condition of bearings

Structure K 02 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Condition of bearings

Structure K 03 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Missing rivet heads at left main beam
- Condition of bearings

Structure K 04 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Bearing condition (cracked bearing on P 01 left)

Structure K 05 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Condition of bearings

Structure K 06 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Condition of bearings

Structure K 07 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Condition of bridge decks
- Condition of bearings

Structure K 08 – Grade 2 rating

For the following reasons:

- Corrosion weakening of individual parts of elements and individual OK elements (treated with CPC)
- Condition of bearings

4.5.1.4 Evaluation of the Substructure of the Bridge at km 3.545:

Abutment O 01 – Grade 2 rating

For the following reasons:

- Storage threshold status

Pier P 01 – Grade 2 rating

For the following reasons:

- Elevated masonry
- Storage threshold status

Pier P 02 – Grade 2 rating

For the following reasons:

- Elevated masonry
- Storage threshold status

Pier P 03 – Grade 2 rating

For the following reasons:

- Binder secretions
- Storage threshold status

Abutment O 02 – Grade 2 rating

- Binder secretions
- Storage threshold status

5. Surveys, including results and conclusions of surveys, influencing the solution

5.1 Geotechnical and Civil Engineering Investigation of the Substructure

The aim of the survey was to verify the material properties of the stone masonry of the piers of the railway bridge for the static recalculation of the substructure. Diagnostic borings were made into the structure to verify the material characteristics and hidden dimensions of the substructure. The full report is presented in section B.14 of this documentation.

From the final reports, the critical information and findings are set out below.

Findings for structure SO 20-20-04 (railway bridge at km 3.545 – Výtoň):

- masonry that is not properly protected from the effects of ground moisture can be damaged by the leaching of lime from the mortar, which loses strength and can be further mechanically damaged by water. Masonry with a reduced mortar content is gap-like, has low strength and is more prone to failure,
- stone masonry elements show an average compressive strength of **66.7 MPa**, the binder shows an indicative compressive strength of 35.3 MPa,
- According to the water pressure tests carried out, the masonry of the substructure is assessed as finely porous, with the exception of borehole V5. From the values found, there is no a priori necessity to grout the substructure. However, it is assumed that below ground level where the binder is exposed to soil moisture, the binder will be more degraded and weathered. For this reason, we recommend considering grouting the piers below ground level.

Findings for structure SO 20-20-05 (railway bridge at km 3.706 – Pod Vyšehradem):

- According to the inclined diagnostic borehole Š1, the existing Smíchov abutment is based at the level of 183.96 m above sea level on a wooden grid in the environment of Quaternary sandy fluvial sediments of geotechnical type Q1, sometimes even gravelly sediments of geotechnical type Q2,
- the width of the Smíchov abutment is 4.00 m according to the horizontal diagnostic borehole,
- the existing Vyšehrad abutment is based on the inclined diagnostic borehole Š4 at the level of 181.97 m above sea level on a wooden grid in the environment of Quaternary gravelly fluvial sediments of geotechnical type Q2,
- according to archival laboratory tests, the existing bridge foundations are permanently within the reach of groundwater, which has an aggressiveness of XA2 according to ČSN EN 206 (pH, agr. CO2),
- The masonry of the piers and abutments is assessed as moderately to coarsely porous according to newly conducted water pressure tests, and based on these findings it is recommended to grout the existing substructure,
- the stone masonry elements used have medium to high simple compressive strength.
- The approximate average mortar strength determined on the drill core taken from the Smíchov abutment is
  8.9 MPa. The approximate average mortar strength found in the drill cores taken from P01 and P02 is 37.5 MPa. The equivalent strengths found have a large variance depending on the amount of cement component (especially for the Sv2 borehole). Mortar is inhomogeneous with regard to the data found.

5.2 Detailed Inspection of Steel Load-Bearing Structures

5.2.1 Description of the Inspection of the Steel Structure

A detailed inspection of the corrosion weakening is the basic basis for the static recalculation of the bridge's load-bearing structures. Furthermore, the inspection serves as a basis 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 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 positioning 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.

THE CODE DESIGNATION OF THE DEFECT:

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 - Trusses (1 to 30)
Y - Bridge side (1 – left, 2 – right)
Z - Cross-sectional defect number (1 to 999)

List of steel construction element groups

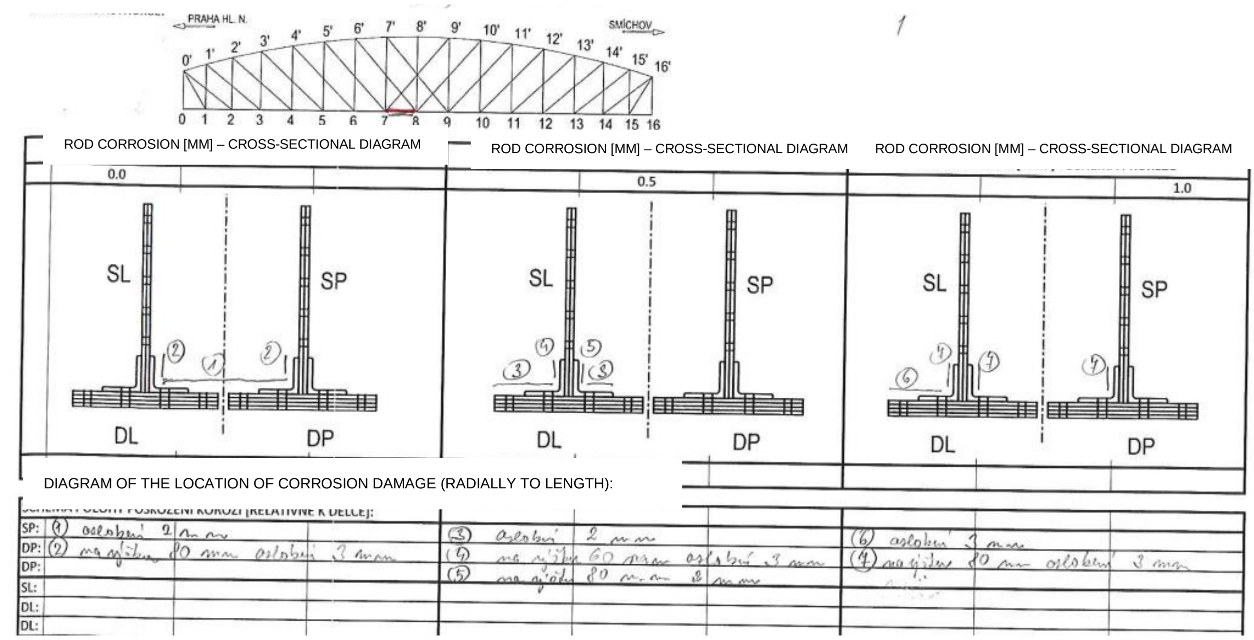
Component No. (code – W)	Component name	Mark
1	Upper chord	O1 to O16
2	Lower chord	U1 to U16
3	End perpendiculars	V0 and V16
4	Inner perpendiculars	V1–V4 and V12–V16
5	Centre perpendiculars	V5–V8 and V9–V12
6	Lacings – lateral	D1–D4 and Z12–Z15
7	Lacings – internal	D5–D8 and Z8–Z11
8	Lacings – central	D9–D10 and Z6–Z7
9	Supporting cross bars	P0–P16
10	Longitudinal trusses	L1–L16
11	Upper stiffening	WO
12	Lower stiffening	WU

Damage to the CPC corresponds to more than 10% rusting of the paint in the entire OK area **(highest damage level Ri 5** according to EN ISO 4628-3).

INVESTIGATION OF CORROSION WEAKENING OF MESH ELEMENTS
ELEMENT: LOWER CHORD U1-16
CONSTRUCTION DIAGRAM:

NK NO.: 2 (2) TRUSS NO.: 8

RAILWAY BRIDGE AT KM 3.706 OVER THE VLTAVA RIVER ON THE LINE PRAGUE HL.N. – SMÍCHOV
PAGE: L/P DATE:

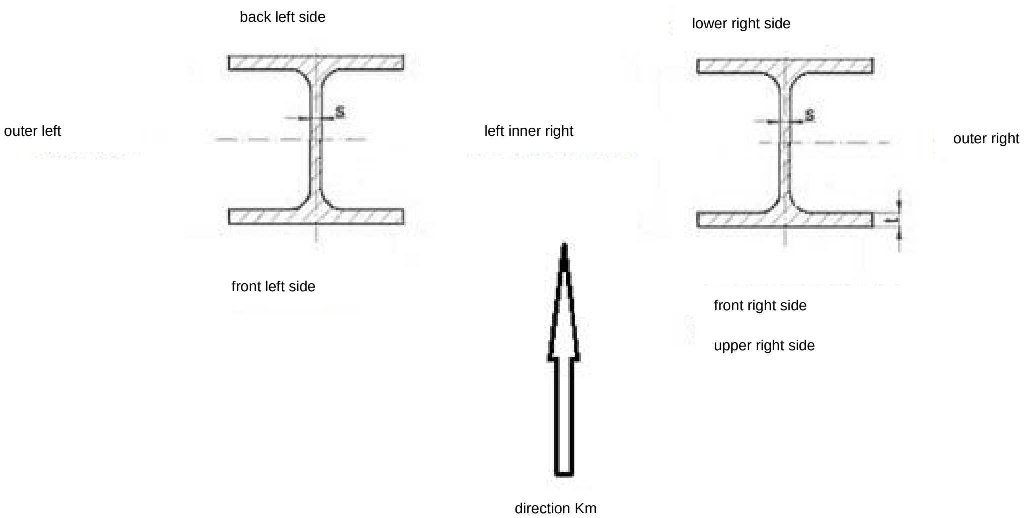


Example of Element Card (2-2-1-8-IE) to record corrosion weakening (NK2, Lower chord, left, 8th truss)

OK fault registration system

Fault records in individual sheets are recorded according to the supplied documents. Due to the very irregular weakening of the elements, it was decided to write the **largest weakening** in the given element length. Due to the stress lock-out of only one track, the upper parts of the left main beam (from the first crosses of the perpendicular – lacing) up to the main beam are not checked. For these parts, the condition corresponding to the diameter of the faults occurring on the right main beam is considered**.**

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).



Orientation of the description of the fault/defect

Fault marking system on OK

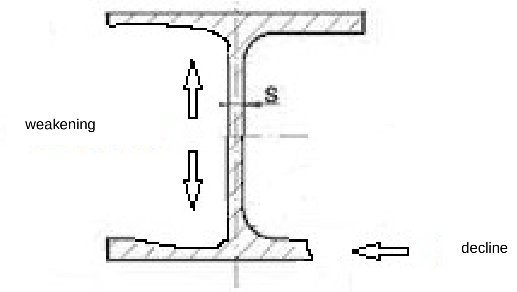
The fault is recorded in the element card using the double number section **weakening / material loss**

Section 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 section 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.



Fault / defect marking

5.2.2 Steel Structure Inspection Summary

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

- Detailed inspection of the corrosion weakening revealed **failures that are limiting to the residual service life of the bridge structure**. In particular, this is the detail at the point of connection of the truss connection of the split rod between the pair of neck angles and the rods themselves to the splice plates or directly to the bottom flange. 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 of the 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.

Note: *The above is confirmed by the current domestic and foreign experience, where the most frequent failures of steel bridges are failures caused by fatigue loading and corrosion of articulated details due to limited maintenance of truss structures.*

5.2.3 Recapitulation of the Evaluation of the Need to Replace the Element due to Corrosion Weakening

RECAPITULATION OF THE NECESSITY OF REPLACEMENT DUE TO CORROSION WEAKENING OF THE MESH ELEMENTS

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

TRUSS	LEFT MAIN BEAM					RIGHT MAIN BEAM					NOTES
	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	U – chords and neck angles
8	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			

WO – REPLACEMENT OF SPLICE PLATES IN THE PORTAL AREA
CH – REPLACEMENT OF PAVEMENT BRACKETS

NOTES

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

RECAPITULATION OF THE NECESSITY OF REPLACEMENT DUE TO CORROSION WEAKENING OF THE MESH ELEMENTS

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

TRUSS	LEFT SIDE					RIGHT SIDE					NOTES
	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	L1	L2	WL	WU	P	L3	L4	WL	WU	
0	N					N					
1	O	N	O	N	VC	O	N	O	N	VC	NK1/L4 – upper chord crack repair
2	N	N	N	N	VC	N	N	N	N	VC	
3	N	N	N	N	VC	N	N	N	N	VC	
4	N	N	N	N	VC	N	N	N	N	VC	
5	N	N	N	N	VC	N	N	N	N	VC	
6	N	N	N	N	VC	N	N	N	N	VC	
7	N	N	N	N	VC	N	N	N	N	VC	
8	O	N	N	N	VC	O	N	N	N	VC	P – wall at the connection point of the braking stiffener
9	N	N	N	N	VC	N	N	N	N	VC	
10	N	N	O	N	VC	N	N	O	N	VC	NK2/L2 – upper chord crack repair
11	N	N	N	N	VC	N	N	N	N	VC	
12	N	N	N	N	VC	N	N	N	N	VC	
13	N	N	N	N	VC	N	N	N	N	VC	
14	N	N	N	N	VC	N	N	N	N	VC	
15	O	N	N	N	VC	O	N	N	N	VC	P – wall at the connection point of the braking stiffener
16	N	N	N	N	VC	N	N	N	N	VC	

WU - REPLACEMENT OF THE SPLICE PLATES OF THE LOWER HORIZONTAL STIFFENING

NOTES

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

After the static recalculation, it was necessary to increase the extent of replacement of the steel structure elements in the rail-track area based on the limiting load capacity of the rail-track elements and the limited residual fatigue life. The revised scope is shown in the table below.

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

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

TRUSS	LEFT SIDE					RIGHT SIDE					NOTES
	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	L1	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	

WU - REPLACEMENT OF THE SPLICE PLATES OF THE LOWER HORIZONTAL STIFFENING

NOTES

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

5.2.4 Description of Typical Defects in Steel Structure

The corrosion survey of the weakening of individual elements of the structure revealed typical defects that **cannot be repaired other than by replacing the whole or part of the element**. It should be noted that the truss structure of the bridge at km 3.706 is primarily composed of tensioned elements (lacings, lower chord and parts of the perpendiculars). Tensile elements generally have higher accumulations of fatigue damage and a higher risk of failure by sudden brittle fracture. From the point of view of structural reliability, these are risky elements that should be given special attention during inspections. Furthermore, as a result of corrosion weakening, the stresses are redistributed in the cross-section of the cord, i.e. the stresses increase in the remaining part.

A secondary problem is the possibility of replacing a part of the cord itself, when it is necessary to dismantle the downstream parts of the structure, which were assembled after the replacement part was fitted. The truss structure was assembled on site from pre-assembled parts of the rods from the bridgeworks along the entire length of the rod, i.e. from the lower chord to the upper chord. This procedure must always be taken into account when designing the reconstruction. The perpendiculars and lacings were assembled in two phases for this structure. First the inner parts of the cross-sections and then the outer parts of the cross-sections (e.g. if the outer part of the lacing needs to be replaced, the outer parts of the perpendiculars must be removed first).

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.

Typical corrosion of the lower chords of the main beam

On the outside, corrosion weakening occurs at the sidewalk brackets, where water flows down the lower strip of the bracket to the lower strip (see figure below), which together with debris on the splice plate causes large corrosion losses. In some places the cantilevers of the footbridges are completely broken.

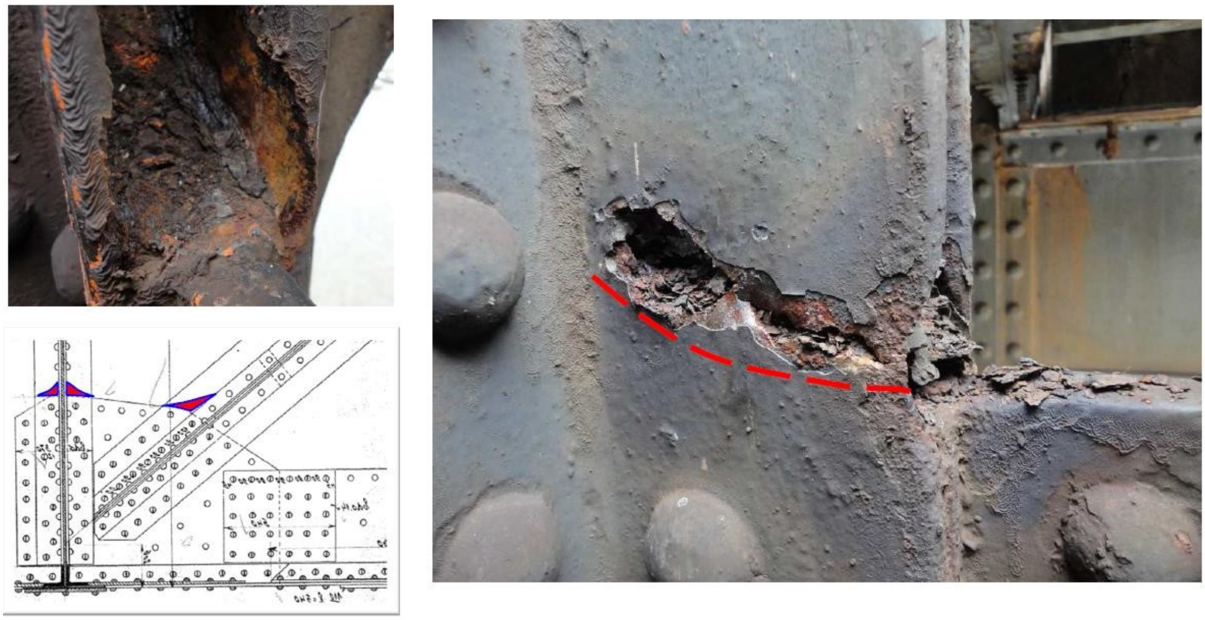
Corrosion of the neck angles of the lower chords in areas with constant moisture

Corrosion of the lower chords in the pavement bracket connections

On the inside, there is a corrosion weakening at the point of the splice plate for the connection of the lower horizontal stiffening, where again the continuous action of moisture corrodes the lower chord and the splice plate itself.

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. The high frequency of these failures along the length of the rods is also a problem. This defect **can only be repaired by replacing part of the cross-section along the entire length of the rod**, as local repair of the damaged area is not structurally possible.

These failures cannot be practically prevented even by intensive maintenance and it is only a matter of time before the corrosion damage reaches a limit state in terms of railway safety.

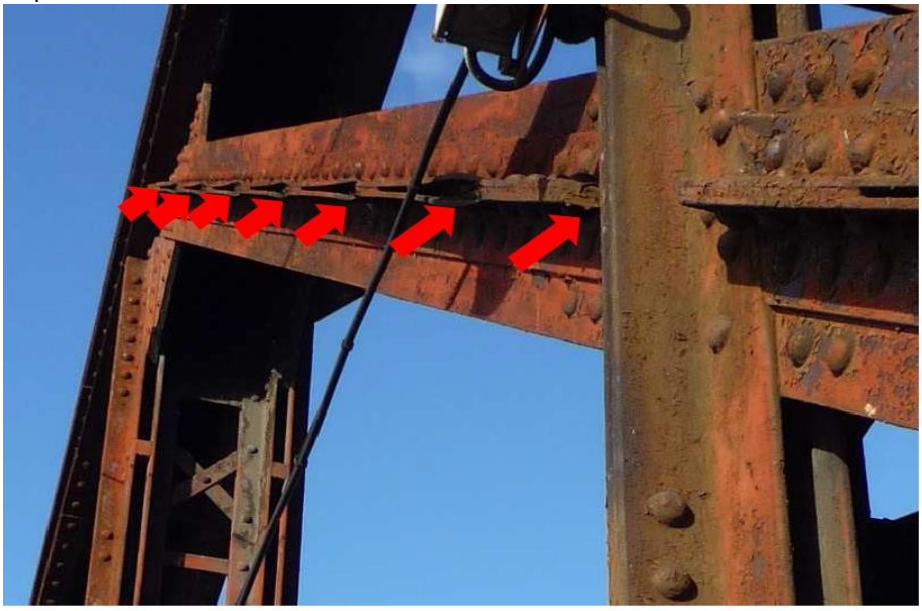


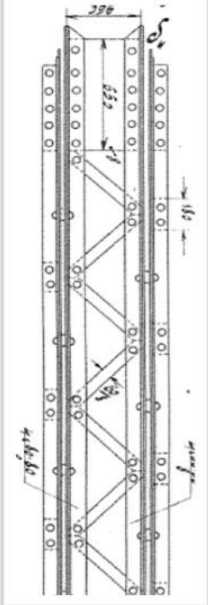
Corrosion in the connections of perpendiculars and lacings to the splice plates of the lower chord

Corrosion at the connection of the lacing (NK2 – right D.7) to the splice plate of the lower chord U.7

Corrosion at the connection of the lacing (NK1 – right D.7) to the splice plate of the lower chord U.7

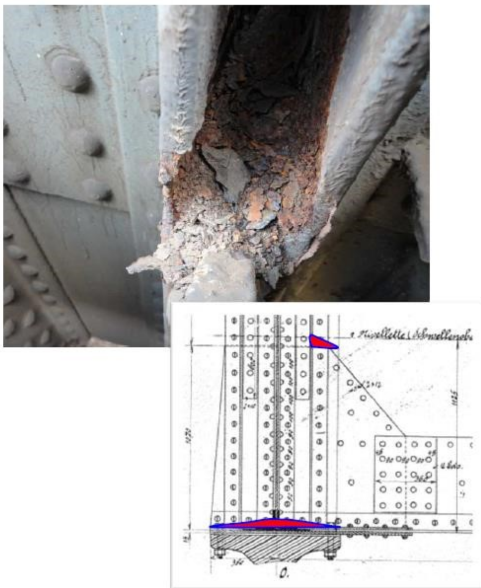
On the outside of the lacings, crevice corrosion is increasing to such an extent that the flanges are permanently deformed. The problem is again the high frequency of defects along the length of the element. The steel material reached its yield strength and was further permanently deformed (the steel material locally plasticised). Damaged parts of the rods must be replaced with new parts that are capable of acting flexibly. This defect can only be repaired by replacing part of the cross-section along the entire length of the rod, and the outer parts of the cross-section of the perpendiculars must also be removed for replacement.





Severe crevice corrosion in the connections of the couplings of the segmented rods of perpendiculars and lacings

Massive corrosion loss occurs in the area of the end portal perpendiculars where the overlay splice plates are. Here, permanent humidity, high dirt fallout and minimal possibility of ventilation lead to significant corrosion losses. These elements also exhibit crevice corrosion at the connection to the splice plate of the bottom chord.



Corrosion of superstructure areas – portal perpendicular

In the area of the rail-track, pitting corrosion of the supporting cross bar chords occurs, where the surface layer of the chord lamellas is corroded by the action of bird droppings and moisture. At the point where the braking stiffeners are connected to the supporting cross bars, the supporting cross bar wall is corroded, again due to the effects of bird droppings and moisture. The splice plates of the lower horizontal stiffening and the braking stiffening create conditions ("birdhouses") for nesting birds, especially pigeons. In the case of longitudinal trusses, corrosion of the upper chords occurs at the location of the bridge beams and the vertical bridge bolt hole.

Corrosion of the upper supporting cross bar chords due to pigeon droppings

5.2.5 Recapitulation of the Evaluation of the Corrosion Weakening Survey of OK elements

The evaluation of the corrosion weakening survey of the cross-sections is carried out for the purpose of static recalculation.

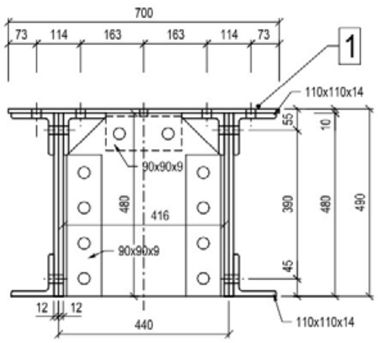
The recapitulation is given by individual elements in aggregate for the load-bearing structures in bays 1 to 3 (NK1 to NK3).

The section weakening is expressed as the ratio of the corrosion weakening to the original unweakened part in percent (corrosion loss = % of the original area) for each basic part of the section.

The amount of weakening is related to the section where the design of the cross-section is carried out, i.e. with the effect of weakening by rivets, so as to eliminate duplication of consideration of weakening by corrosion and holes for fasteners. From the maximum values of corrosion weakening in the part outside the holes (see the element cards), the values of weakening in the critical section are at the level of about 60%.

The recapitulation also lists the critical weakenings for the section under consideration that are present on the structure at the same time at the point (section under consideration).

ELEMENT: **UPPER CHORD O1-O16** PAGE: **L/P** NK NO.: **1, 2, 3**

TRUSS			0–1		NK		NOTES
O.1/O.16			1/0%		1/ 2/ 3		
O.2/O.15			1/0%		1/ 2/ 3		
O.3/O.14			1/0%		1/ 2/ 3		
O.4/O.13			1/0%		1/ 2/ 3		
O.5/O.12			1/0%		1/ 2/ 3		
O.6/O.11			1/0%		1/ 2/ 3		
O.7/O.10			1/0%		1/ 2/ 3		
O.8/O.9			1/0%		1/ 2/ 3		

TYPICAL CORROSION OF THE UPPER CHORD

- DEFORMATION OF THE SPLICE PLATE STIFFENING CONNECTION CONNECTION

TYPICAL CORROSION OF THE UPPER CHORD

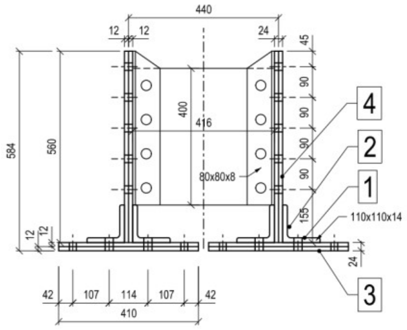
- TRANSVERSE CRACK IN THE UPPER SPLICE PLATE OF THE STIFFENING

NOTES

1–1. THE FIRST SHEET OF THE UPPER CHORD

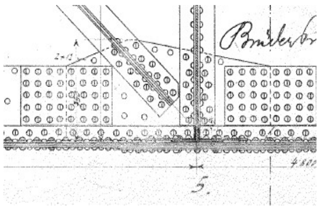
ELEMENT: **LOWER CHORD U1-16** PAGE: **L/P** NK NO.: **1, 2, 3**

TRUSS	0.5		0–1		NK	NOTES
U.1/U.16	1/20%, 2/5%				1_P	
U.2/U.15	1/25%, 2/6%				1_P	
U.3/U.14	1/45%, 2/34%, 3/12%				1_L	
U.4/U.13	1/50%, 2/33%, 3/7%				1_L	
U.5/U.12			1/36%, 2/27%, 3/5%		1_L	
U.6/U.11			1/43%, 2/22%, 3/4%		1_L	
U.7/U.10			1/36%, 2/29%, 3/4%		1_L	
U.8/U.9			1/29%, 2/17%, 3/3%		1_L	

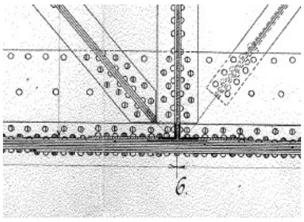


TYPICAL CORROSION OF THE LOWER CHORD AT THE PANEL POINT (LACING/PERPENDICULAR)

SPLICE PLATE (NO. 0–5)



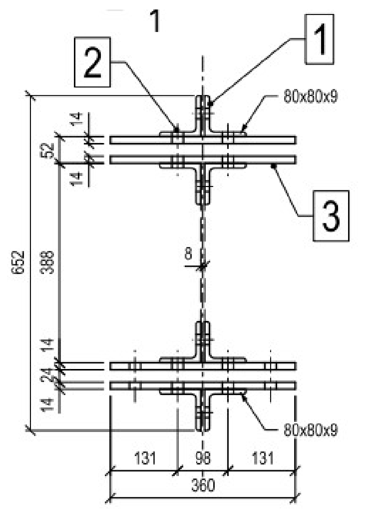
WITHOUT SPLICE PLATE (NO. 6–8)



TYPICAL CORROSION OF THE LOWER CHORD IN THE SECTION BETWEEN THE PANEL POINTS

NOTES
1 - HORIZONTAL FLANGES OF THE NECK ANGLES
2 - VERTICAL FLANGES OF THE NECK ANGLES
3 - LOWER CHORD TOTAL
LACING D \ - DESCENDING
LACING Z / - ASCENDING

ELEMENT: **LACINGS D1–D10 AND Z6–Z15** PAGE: **L/P** NK NO.: **1, 2, 3**

TRUSS	0	NK	0.5	NK	1	NK	NOTES
D.1/Z.15	1/40%	1_L	1/20%	2_L			
D.2/Z.14	1/35%	1_L	1/25%	3_P			
D.3/Z.13	1/35%	2_P	1/25%	3_L			
D.4/Z.12	1/40%	2_L	1/30%	2_P			
D.5/Z.11	1/35%	3_P	1/20%	3_P			
D.6/Z.10	1/30%	3_P	1/15%	3_P			
D.7/Z.9	1/35%, 2/25%	3_P (1_P)	1/20%	3_P			
D.8/Z.8	1/20%	3_P	1/20%	1_P			
D.9/Z.7	1/20%	2_P	1/15%	3_P			
D.10/Z.6	1/20%	2_P	1/10%	1_P			

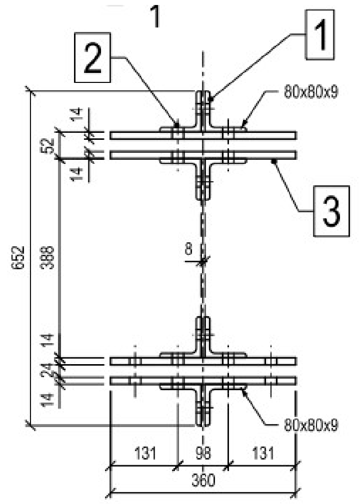
TYPICAL CORROSION OF THE LACING AT THE PANEL POINT WITH THE LOWER CHORD

TYPICAL CREVICE CORROSION OF THE LACING

NOTES

1 - EXTREME FLANGES OF ANGLES (VERTICAL)
2 - ANGLE FLANGES (HORIZONTAL)
3 - DIAGONAL SHEET
LACING D **** - DESCENDING
LACING Z **/** - ASCENDING

ELEMENT: **LACINGS D1–D10 AND Z6–Z15** PAGE: **L/P** NK NO.: **1, 2, 3**

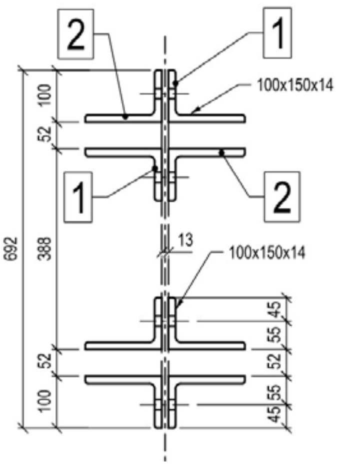
TRUSS	0	NK	0.5	NK	1	NK	NOTES
D.1/Z.15	1/40%	1_L	1/20%	2_L			
D.2/Z.14	1/35%	1_L	1/25%	3_P			
D.3/Z.13	1/35%	2_P	1/25%	3_L			
D.4/Z.12	1/40%	2_L	1/30%	2_P			
D.5/Z.11	1/35%	3_P	1/20%	3_P			
D.6/Z.10	1/30%	3_P	1/15%	3_P			
D.7/Z.9	1/35%, 2/25%	3_P (1_P)	1/20%	3_P			
D.8/Z.8	1/20%	3_P	1/20%	1_P			
D.9/Z.7	1/20%	2_P	1/15%	3_P			
D.10/Z.6	1/20%	2_P	1/10%	1_P			

TYPICAL CORROSION OF THE LACING AT THE PANEL POINT WITH THE LOWER CHORD TYPICAL CREVICE CORROSION OF THE LACING

NOTES

1 - EXTREME FLANGES OF ANGLES (VERTICAL)
2 - ANGLE FLANGES (HORIZONTAL)
3 - SHEET OF LACING CHORD
LACING D **** - DESCENDING
LACING Z **/** - ASCENDING

ELEMENT: **PERPENDICULAR V0-V8 (V9–V16)** PAGE: **L/P** NK NO.: **1, 2, 3**

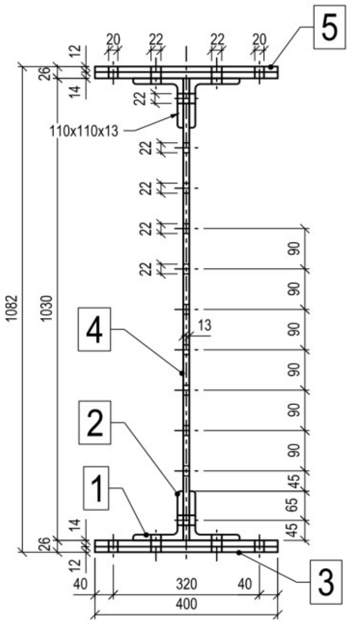
TRUSS	0.1–0.5	NK	0.5–1	NK		NOTES
V.0/V.16	4/10%	1_P	1/0%	1/ 2/ 3		
V.1/V.15	1/10%	3_P	1/10%	3_P		
V.2/V.14	1/10%	2_P	1/10%, 2/5%	2_P		
V.3/V.13	1/10%	3_P	1/10%	3_P		
V.4/V.12	1/10%	1_P	1/15%	1_P		
V.5/V.11	1/10%	3_P	1/10%, 2/5%	3_P		
V.6/V.10	1/15%	1_P	1/20%	1_P		
V.7/V.9	1/15%	1_P	1/20%	1_P		
V.8	1/15%	1_P	1/20%	1_P		

TYPICAL CORROSION OF THE PERPENDICULAR AT THE PANEL POINT WITH THE LOWER CHORD

TYPICAL CREVICE CORROSION OF THE PERPENDICULAR

NOTES
1 - VERTICAL FLANGES OF ANGLES (OUT OF PLANE)
2 - HORIZONTAL FLANGES OF ANGLES (IN PLANE)

ELEMENT: **SUPPORTING CROSS BAR P.0 - P.16** PAGE: **L/P** NK NO.: **1, 2, 3**

SPLICE	0 – edge to main beam	0.25 – outer longitudinal truss	0,5 - inner longitudinal truss/centre	NOTES
P.0/P.16	5/5%	3/1%, 5/3%	3/4%, 5/3%	
P.1/P.15	5/5%	2/15%, 5/3%	2/10%, 5/3%	
P.2/P.14	5/5%	2/15%, 5/3%	2/10%, 5/3%	
P.3/P.13	5/5%	1/15%, 2/15%, 5/3%	1/15%, 2/10%, 5/3%	
P.4/P.12	5/5%	1/15%, 2/15%, 5/3%	1/15%, 2/10%, 5/3%	
P.5/P.11	5/5%	1/15%, 2/15%, 5/3%	1/15%, 2/10%, 5/3%	
P.6/P.10	5/5%	1/15%, 2/15%, 5/3%	1/15%, 2/10%, 5/3%	
P.7/P.9	5/5%	1/15%, 2/15%, 5/3%	1/15%, 2/10%, 5/3%	
P.8	5/5%	1/15%, 2/15%, 5/3%	1/15%, 2/10%, 5/3%	

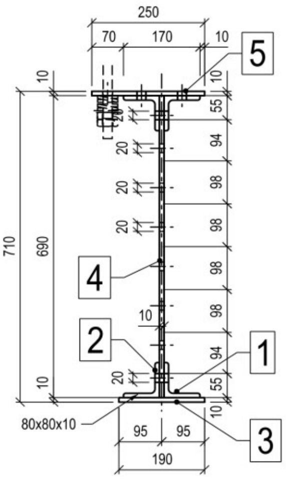
TYPICAL CORROSION OF THE NECK ANGLES OF THE SUPPORTING CROSS BAR

Photo: 2-9-7-02-2.JPG
- CORROSION OF THE WALL ABOVE THE NECK ANGLES

NOTES
1 - HORIZONTAL FLANGES OF THE NECK ANGLES
2 - VERTICAL FLANGES OF THE NECK ANGLES
3 - LOWER CHORD TOTAL
4 - LOWER BELT WALL
5 - TOTAL UPPER CHORD

FOR A CORROSION SURVEY:
- upper chords with pitting corrosion up to 3 mm. Globally, it represents a 3-5% weakening.

ELEMENT: **LONGITUDINAL TRUSS L1–L16** PAGE: **L/P** NK NO.: **1, 2, 3**

TRUSS	0	0.5	1	NOTES
L.1 (L.16)		5/5%		NO.1 (NO.3) NO.2 (NO.4) 
L.2/L.15		2/5%, 5/5%		
L.3/L.14		5/5%		
L.4/L.13		5/5%		
L.5/L.12		5/5%		
L.6/L.11		5/5%		
L.7/L.10		5/5%		
L.8/L.9		1/5%, 2/5%, 5/5%		

TYPICAL CORROSION OF THE NECK ANGLES OF THE LONGITUDINAL TRUSS AT THE BRAKING STIFFENER

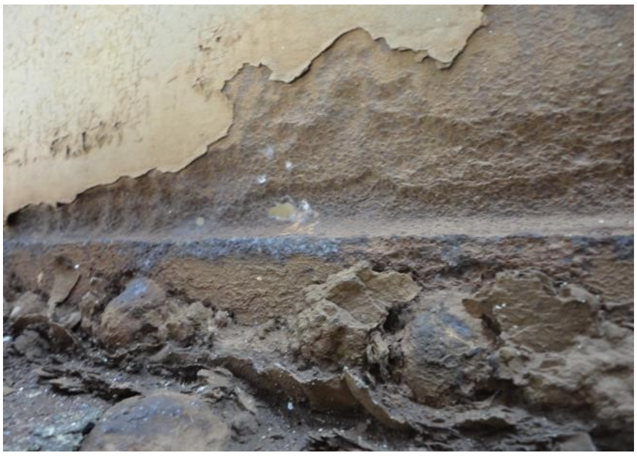
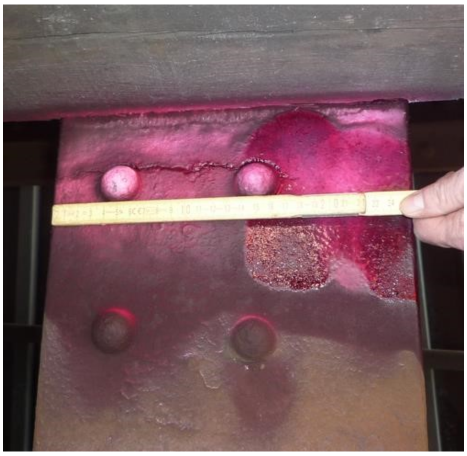


photo 2-10-15-05

CRACK IN THE UPPER CHORD



crack in the longitudinal truss chord L4.1 (NK1)

NOTES

1 - HORIZONTAL FLANGES OF THE NECK ANGLES
2 - VERTICAL FLANGES OF THE NECK ANGLES
3 - LOWER CHORD TOTAL
4 - LOWER BELT WALL
5 - TOTAL UPPER CHORD

FOR A CORROSION SURVEY:

- According to the Detailed Inspection 2017, the upper chords under the bridge beams are weakened by about 2–3 mm. Globally, it may be a 5% to 10% weakening

5.3 Documentation of Bridge Structure Dimensions

The dimensions of the individual elements and their sections were verified from archival documentation as part of the steel structure survey. In cases of absence of archival documentation or contradictions, they were measured. Global dimensions were checked from the S-JTSK coordinates (see Part I – Geodetic Documentation).

The invisible dimensions of the substructure were verified by exploratory borings at abutments O01 and O02 (see Section B.14). For Piers P01 and P02, verification of the bottom level was carried out as part of the underwater survey.

5.4 Detailed Inspection of the Substructure

A detailed inspection of the substructure is the fundamental basis for the static recalculation of the individual supports. Furthermore, the survey serves as a basis for determining the design of the scope of the reconstruction of the substructure and its foundation, i.e. the necessity of remediation works, grouting works, reinforcement, etc.

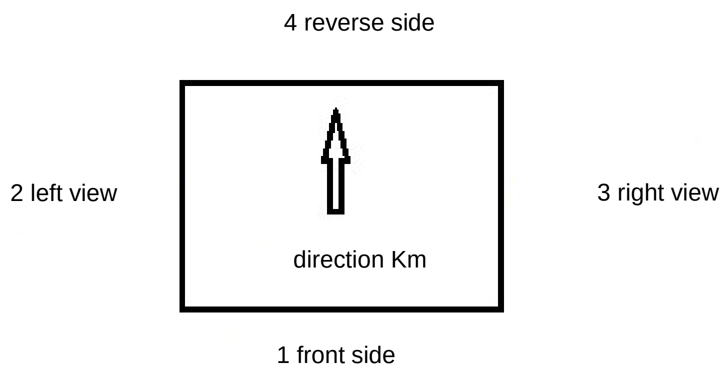
The inspection also included a detailed underwater survey of Piers P01 and P02 in the Vltava River. The above ground visible parts of the substructure were inspected and individual defects documented.

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 defects are marked with a serial number within the inspection. 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 in a similar way to a steel structure.

Marking System of the Substructure

DEFECT CODE DESIGNATION: **A – B – C – D**

whereas:
A - denotes the component number (indicated on the front – **14** – substructure)
B - indicates the relevant element (abutment/pier) in direction km
C - the number indicates the position of the view according to km
D - the number of the photo in the respective view



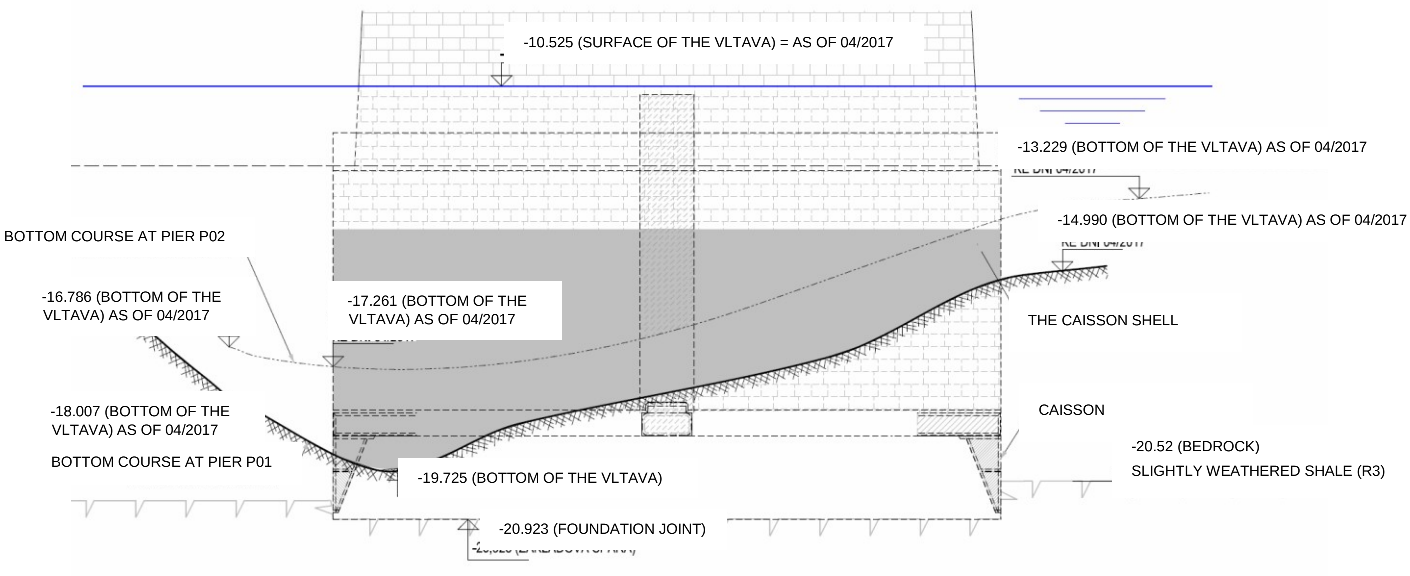
position of the view for the defect entry

The fault is indicated by one or two entries in the relevant grid of the surface of the abutment or pier.
The **first figure** with % + number is related to the water and binder flow of the respective area in the grid
The second figure itself is related to the splice buckling in the total length of the joints in the grid

The underwater survey is plotted with descriptions of the defects and depths in each section of the piers. The exploration work includes HD video of the underwater survey.

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

- the upper surface of the joint is cracked, sporadically hollowed out. Upper surface lightly soiled, good condition.
- paired shanks 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

5.5 Material Tests of the Load-bearing Structure Samples

5.5.1 Material Tests in General

The laboratory work was carried out in the testing room of CZ FERMET s.r.o, Kladno. The sampling was carried out in agreement with the bridge manager of the SZCZ, DG Prague. The location of the sampling points was determined according to the static and structural possibilities so as to avoid permanent damage to the bridge structure. The sampling was carried out on rolled plate and rolled L section. It was not possible to take samples of rivets and strip steel of the filler elements of the sectioned rods. The properties of these elements were checked by means of hardness tests. Using these tests, check verifications were also carried out on the critical elements of the structure, thus providing a sufficient set of measured points to describe the material properties of the steel used for the structure.

With regard to the independent production of the load-bearing structures in the three bridgeworks, samples were taken from all three load-bearing structures. At a minimum, material from six different smelters was supplied for the bridge. The tests were set in the following range:

Sample designation	Description of the sample	Number of test bodies				Microstructure	Chemical composition	Note
		Longitudinally	Transversely	Cutter longitudinally	Cutter transversely			
1P	NK1 – Sheet metal	3	2					melt 1
2P	NK1 – Sheet metal	3	2	1	1	1	1	
3P	NK1 – Sheet metal	3	2					
4P	NK2 – Sheet metal	3	2					melt 2
5P	NK2 – Sheet metal	3	2	1	1	1	1	
6P	NK2 – Sheet metal	3	2					
7P	NK3 – Sheet metal	3	2					melt 3
8P	NK3 – Sheet metal	3	2	1	1	1	1	
9P	NK3 – Sheet metal	3	2					
1U	NK1 – Angle	3						melt 4
2U	NK1 – Angle	3		1		1	1	
3U	NK1 – Angle	3						
4U	NK2 – Angle	3						melt 5
5U	NK2 – Angle	3		1		1	1	
6U	NK2 – Angle	3						
7U	NK3 – Angle	3						melt 6
8U	NK3 – Angle	3		1		1	1	
9U	NK3 – Angle	3						
Total		**54**	**18**	**6**	**3**	**6**	**6**	

5.5.2 Results of Mechanical Tests

From the material tests of the steel samples, it was found that the yield strength corresponds to the characteristic values according to the Methodological Guideline for Plow Steel. High ductility **ranging from 33% to 41%** was achieved in the tests. The minimum value of ductility according to today's delivery standards is **26%** for S235 JR steel. The properties in the transverse **Q** and longitudinal **L** directions do not differ significantly, which is a typical feature of plow steel. The modulus of elasticity E was measured from **192 GPa to 207 GPa** (average 197 GPa).
Charpy impact tests were carried out at room temperature +23 °C. The values of impact work for sheet metal were **8 J to 42 J** and for rolled section **18 J to 155 J**. The impact work values correspond approximately to **JR** grade steel.

The impact work values correspond approximately to **JR** grade steel, which is **completely unsuitable** for **dynamically stressed structures**. This is due to the higher susceptibility to **sudden breakage by brittle fracture**. According to the current EN 1993-2 Table 3.1, for thicknesses up to 30 mm, the value of notch toughness (absorbed energy in bending impact test) is required to be **at least 27 J at -20°C**, which is **not met** in this case.
Furthermore, in view of the current requirements for the basic material of railway bridges, which are defined in the TKP SSD Chap. 19, Art. 19.2.1, the use of **JR** quality steel for bridge structures is not **permissible**.

Results of SUDOP 64 ex A:

	ReH [kN/m]	Rp0.2 [MPa]	Rm [MPa]	A* [%]	Z* [%]	E [GPa]
1P-1	264	207	410	37,8	62,7	207,621

Working diagram of steel Sample 1 - P1

Year of production	**Material strength classes**	**Permissible stresses σ_{adm} [MPa]**	**Guaranteed yield strength f_y [MPa]**	**Strength limit f_u [MPa]**	**Standard**
until 1894	welding iron	130	210	340	
1895– **1904**	welding iron	130	210	340	Regulation 97/1904
	Plow steel	**140**	**230**	**360**	

The basic values of the guaranteed yield strengths according to the Methodological Guideline (MG) were used for the design of the cross-sections in the framework of the determination of the load capacity and the assessment of compatibility. The tests show a slightly higher yield strength value, but the strength value corresponds to the MG value. Also the number of samples and their distribution along the length of the structure is not entirely conclusive for statistical evaluation because of the possibility of considering higher yield strength values than those specified in the Methodological Guideline.

5.5.3 Results of Mechanical Tests – Hardness Tests

The quality of the steel was verified by hardness testing. The parameter investigated corresponds to the strength of steel f_u. The calibration of the hardness tests was carried out on the samples tested in the testing room as part of the mechanical tests.

From the measurements, it can be concluded that the steel corresponds to plow steel, and is close to today's S235 steel. The measured values are always higher in strength than the value fu=340 MPa given in MG 2015. It can also be seen that the plates generally have better strength values than the rolled sections (angles), which is common in these structures. At the same time, it is important that no location with a significantly lower strength value was discovered that would have entailed the risk of using lower quality steel.

In the case of the supporting cross bar and longitudinal truss, it cannot be ruled out that in places better quality steel than the then used Series 37 steel was used for reinforcement, as the tests were carried out on the upper chord.

Bay 1						
	Location	Type of element	1 series-f_u [MPa]	2 series-f_u [MPa]	3 series-f_u [MPa]	Average - f_u [MPa]
Lacing	GD021	Sheet	381	374	387	381
	GD022	Sheet	381	404	369	385
	GD023	Sheet	380	375	374	376
	GD024	Sheet	375	380	338	386
Lacing	GD071	Angle	385	401	414	401
	GD072	Angle	416	409	417	414
	GD073	Angle	390	417	405	404
	GD074	Angle	364	372	381	372
Lower chord	GB0S1	Sheet	416	419	451	429
	GB082	Sheet	445	424	406	425
	GB083	Sheet	479	479	492	431
	GB084	Sheet	452	436	458	465
Lower chord	GB085	Sheet	396	414	410	407
	GB086	Sheet	427	427	449	434
	GB087	Sheet	474	473	473	475
	GB088	Sheet	490	505	497	497
Along niches	S1U	Sheet	365	370	366	367
	S2U	Sheet	385	371	373	376
	S3U	Sheet	405	413	383	400
	54U	Sheet	404	407	413	408
	S5U	Sheet	431	417	413	420
	S6U	Sheet	418	429	410	419
Supporting cross bar	C1	Sheet	360	375	374	370
	C2	Sheet	383	365	362	370
	C3	Sheet	454	438	444	445
	C4	Sheet	435	452	447	445
	C5	Sheet	434	441	445	440
	C5	Sheet	461	437	459	452
	C7	Sheet	379	373	389	380
	C8	Sheet	365	374	378	372

Note: The values in green were measured in poorly accessible conditions on a corroded, uneven surface, and its predictive power is limited.

5.5.4 Results of Chemical Composition Tests

In order to compare the properties, 6 samples were analysed for chemical composition.
The chemical composition is similar for all steel samples. The carbon content ranges from 0.07 to 0.2%. The manganese content Mn is higher among the alloying elements. The low sulphur S and phosphorus P content is a confirmation of the plow steel.

Results of chemical analysis for sample 2P – sheet metal

Order number	Sample number	Brand of steel	Melt	Product
16 354 209 K36	2P	--------------	1	NK1 – Sheet metal

Element	[%]	Element	[%]
C	0.15	**Cu**	0.01
Si	0.002	**Nb**	0.002
Mn	0.24	**Ti**	0.001
P	0.016	**V**	0.003

S	0.028	**W**	0.012
Cr	0.01	**Pb**	0.003
Mo	0.01	**Sn**	0.002
Ni	0.02	**As**	0.017
Al	0.008	**Sb**	0.002
Co	0.011	**B**	0.0012

Results of chemical analysis for sample 2U – angle

Order number	**Sample number**	**Brand of steel**	**Melt**	**Product**
16 354 209 K36	2U	---------------	4	NK1 – Angle

Element	**[%]**	**Element**	**[%]**
C	0.16	**Cu**	0.05
Si	0.002	**Nb**	0.002
Mn	0.31	**Ti**	0.001
P	0.011	**V**	0.002
S	0.010	**W**	0.017
Cr	0.05	**Pb**	0.003
Mo	0.01	**Sn**	0.003
Ni	0.04	**As**	0.007
Al	0.004	**Sb**	0.003
Co	0.003	**B**	0.0011

From the chemical analysis, the carbon equivalent value CEV is calculated:

CEV = C+Mn/6+(Cr+Mo+V)/5+(Cu+Ni)/15

Sheet metal:
CEV = 0.15+0.24/6+(0.01+0.01+0.003)/5+(0.01+0.02)/15 = **0.20 < 0.35 C** = **0.16 < 0.17**

Angle:
CEV = 0.16+0.31/6+(0.05+0.01+0.002)/5+(0.05+0.04)/15 = **0.23 < 0.35 C = 0.16 < 0.17**

Sheet and angle correspond to the chemical composition according to the currently valid delivery standard according to EN 10025-2.

The carbon equivalent values show a low content of alloying elements, which corresponds to non-alloyed structural steel. Comparison with steel produced today would be possible with **S235JR steel according to EN 10025-2.** The chemical composition of this steel is shown below.

Mark		Method of deoxi-dation[b]	C in % max. for product of nominal thickness in mm			Si % max.	Mn % max.	P[d)] % max.	S[s), e)] % max.	N[f)] % max.	Cu[g)] % max.	Other ele-ments[h)] % max.
According to EN 10027-1 and	According to EN 10027-2		< 16	> 16 <40	> 40[c]							

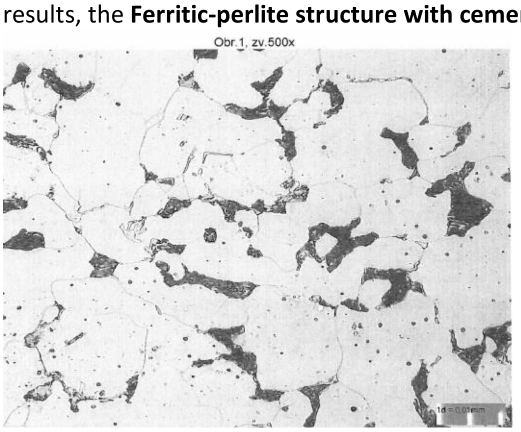
CR 10260												
S235JR	1.0038	FN	0.17	0.17	0.20	-	1.40	0.035	0.035	0.012	0.55	—

Chemical composition of steel samples

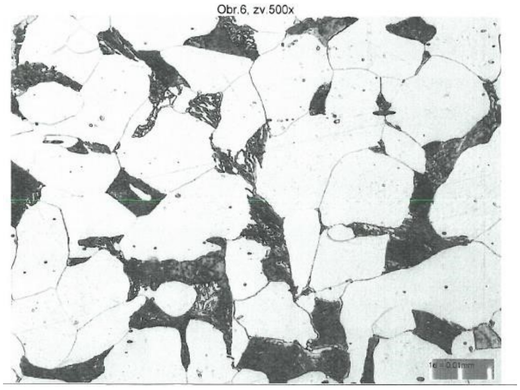
For steel S 235JR for t< 30 mm the limit value of carbon equivalent CEV < 0,35 according to EN 10 025-2

5.5.5 Metallographic Test Results – Microstructure

After grinding and polishing, a photo microstructure of the steel grains was taken on the samples. From the results, the **Ferritic-perlite structure with cementite** was evaluated for the sheet and angle samples.

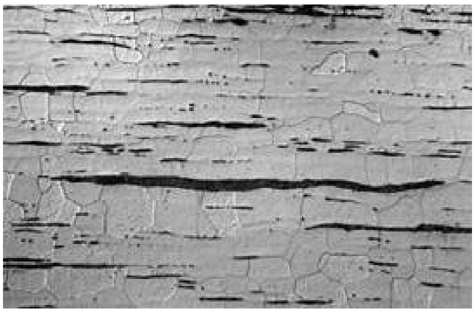


Sample sheet metal – 500 x enlarged



Angle sample – 500 x enlarged

For comparison, a microscopic image of the welding steel is shown below, where the layered positions of the rolled slag (dark layers) are visible.



Microscopic image of welding steel

It was confirmed by metallographic tests that it is **plow steel**.

5.6 Static and Dynamic Verification Load Test

On 11/05/2017, a **static and dynamic load test** was carried out on the bridge structure. The purpose is to verify the conformity of the measured quantities determined on the calculation model of the bridge for its possible adjustment (calibration according to the actually measured values). Furthermore, the fatigue effects of traffic on the bridge (determination of traffic load spectra) are determined.

The test was carried out by ČVUT in Prague, Faculty of Civil Engineering.

From the test results, the conformity of the assumption of deformations and stresses with the measurement results and the first eigenmodes and frequencies (torsion and bending) will be evaluated. It is therefore not a load test according to ČSN 73 6209 or according to the construction and technical regulations for railways (Decree No. 177/95, Section 6e).

This document provides a basic evaluation of the static and dynamic testing of the bridge. Due to the extent of the data obtained, the evaluation is continuing, the data are mainly used to calibrate the computational model.

During the static load test, the following was measured:

- vertical deflection (radar interferometry),
- deformation of the end cross member

- normal stresses on selected bridge structure elements (upper and lower chords, lacings, supporting cross bars, longitudinal trusses),

In 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 1/4 of the span
- normal stresses on selected elements of the bridge structure in accordance with the static test. The frequency of sensing of the measured quantities will be adjusted for individual load conditions. Due to interference from the nearby electrified tram line, a frequency of 50 Hz was used.

Load conditions according to purpose:

- **Static load conditions:**

SZS0 Static load condition L/0 unsymmetrical (in track 2) – above the supporting cross bar (2:10)
SZS1 Static load condition L/2 unsymmetrical (in track 2) – excluded track 2 (0:56)
SZS2 Static load condition L/2 unsymmetrical (in track 1) – no-track operation (1:27)
SZS3 Static load condition L/2 symmetrical (both tracks) – no-track operation (1:50)

- **Dynamic load conditions:**

DZS0 Dynamic load condition – slow crossing at 0 km/h in the direction of Smíchov and back in track 1 (2:40)
DZS1 Dynamic load condition – level crossing 30 km/h in track 1 direction of Smíchov
DZS2 Dynamic load condition – level crossing 60 km/h (in track 1) direction of Smíchov
DZS3 Dynamic load condition – level crossing 5 km/h (in track 1), to Smíchov and back to Prague (3:41)
DZS4 Dynamic load condition – level crossing 30 km/h (in track 1), direction of Smíchov (3:51)
DZS5 Dynamic load condition – level crossing 60 km/h (in track 1) (3:59)

- **Braking load conditions:**

BZS1 Braking load condition – braking from 40 km/h to 0 in track 2, direction of Smíchov, departure direction of Smíchov (4:11)

BZS2 Braking load condition – braking from 40 km/h in track 2, direction of Prague, departure direction of Prague, unsuccessful condition (4:18)

BZS3 Braking load condition – braking from 60 km/h in track 1, direction of Smíchov (inappropriate stop, continued braking from 40 km/h) (3:04)

BZS4 Braking load condition – braking from speed 40 km/h in track 1 in the direction of Prague, then starting in direction of Prague (3:19),

BZS5 Braking load condition – braking from 40 km/h towards the main railway station, subsequent departure in the direction of Smíchov

During the dynamic load test, the frequencies and shapes of the bridge's own oscillations were also evaluated. Due to the limited number of sensors and the time available for the test, where it was not possible to move the sensors during the test, it was only possible to evaluate a few basic frequencies and shapes of the natural oscillation. Acceleration sensors located on the lower chords and on the perpendiculars of the main beams were used for the evaluation. The corresponding eigenmodes $\{r_{(j)}\}$ were also evaluated for the frequencies from the 9 measured points.

The frequencies and shapes of natural vibrations evaluated in the dynamic load test were compared with the calculated frequencies and shapes of natural vibrations determined in the dynamic calculation of the bridge. The aim of this comparison was to refine the characteristics of the MKP model.
For comparison, the criteria from ČSN 736209 change Z1 were used. The deviation of the compared natural frequencies $\Delta_{(j)}$ is calculated based on the formula given in ČSN 736209

The evaluation of the dynamic response of the real structure is the basis after the verification of the data from the computational model.

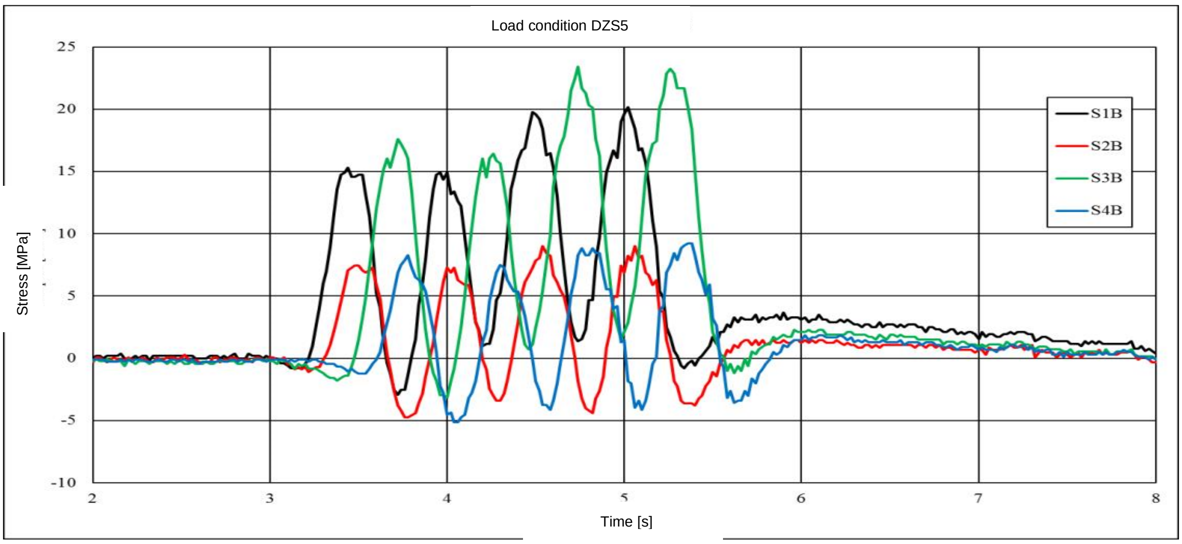
Natural frequencies calculated		Natural frequencies measured			Natural frequency deviation	Permissible limit deviation of frequencies
Order no. (j)	f(j) [Hz]	Order no. (j)	f(j) [Hz]	Extended uncertainty $U_{k=2}$ [Hz]	$\Delta_{(j)}$ [%]	$\Delta_{(j)}$ [%]
(1)	2.03	(1)	2.09	+/- 0.06	-3.0 +/- 2.9	+10 ; -15
(2)	2.85	(2)	3.34	+/- 0.06	-17.2 +/- 1.8	+10 ; -15
(3)	4.15		x	+/- 0.06		+10 ; -15
(4)	4.35	(3)	4.41	+/- 0.06	-1.4 +/- 1.4	+/- 15.0
(5)	5.15		x	+/- 0.06		+/- 15.2
(6)	5.28	(4)	5.78	+/- 0.06	-9.5 +/- 1.0	+/- 15.3
(7)	6.41	(5)	6.84	+/- 0.06	-6.7 +/- 0.9	+/- 15.5

Comparison of the corresponding calculated and measured natural oscillation frequencies using the deviation $\Delta_{(j)}$ of *the monitored bridge structure.*

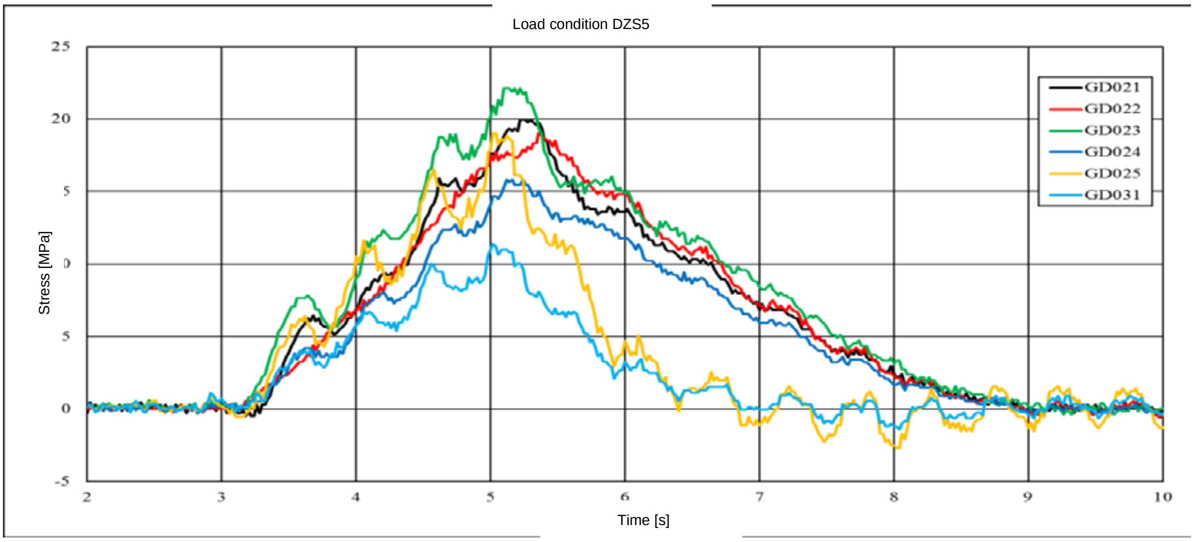
The theoretical eigenmodes have the same number of nodal lines as the test eigenmodes and the lines lie in the same bays of the structure, all the pairs of **eigenmodes** compared **show very good agreement** with the computational model.
The deviation $\Delta_{(j)}$ exceeds the limiting deviation for the pair of compared frequencies corresponding to the 2nd natural oscillation shape, for the other compared frequencies this criterion is met.

5.7 Evaluation of Stress Spectra from Traffic Loading

The basis for creating the stress spectra was data obtained from monitoring the bridge, which took place between 11 May 2017 and 28 May 2017, a total of 18 days. A total of 7 days (Sunday, 21 May 2017 to Saturday, 27 May 2017) were selected and evaluated to ensure that the spectra produced represent the stress ranges and their corresponding cycle counts from traffic on the bridge during 1 typical week. The data were in the form of relative strain values, which were detected by strain gauges placed on the bridge structure.
Next, the stresses at the measured locations were evaluated. The following figures show a typical stress pattern when crossing the longitudinal truss and the main beam.



Time course of stresses on the longitudinal trusses during the crossing of the test railway vehicle.



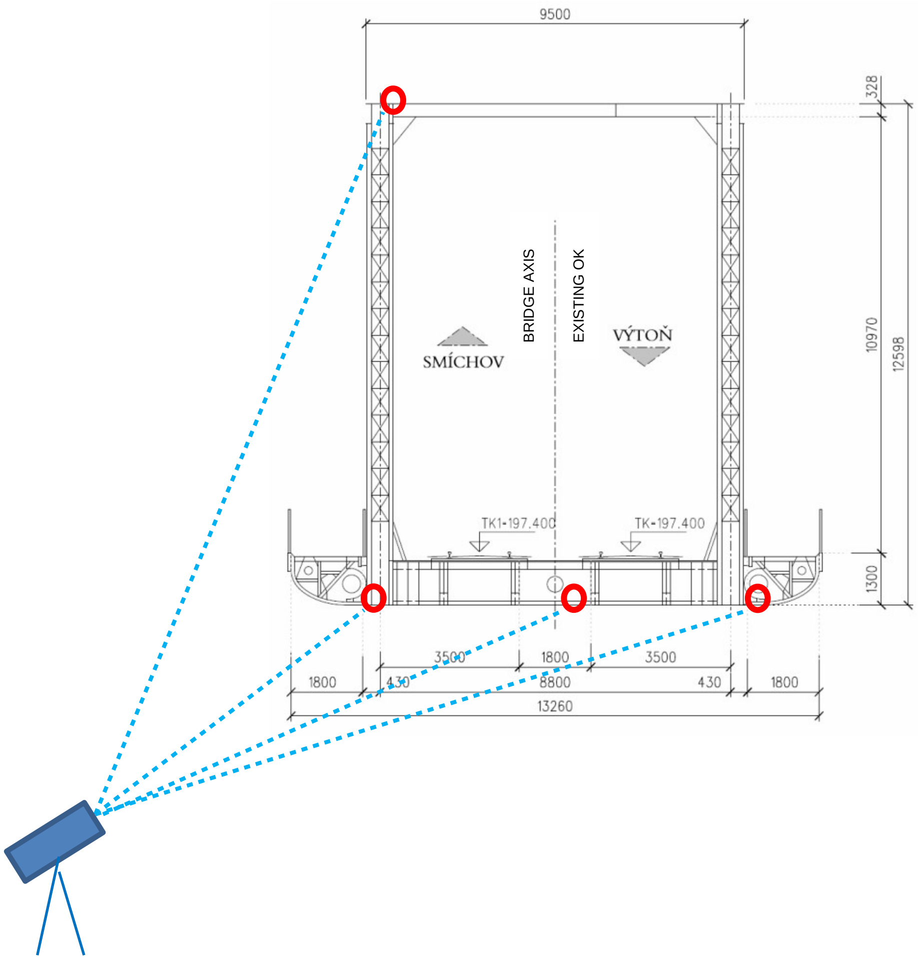
Time course of stresses on the lower chord of the main beam when the test vehicle passes over it

Stress spectra were evaluated for all the monitoring sites mentioned above (32 sites in total), and the results are tabulated for better overview. For each element, the resulting tables with stress spectra (stress ranges and their corresponding numbers of cycles) for each element from the effects of all trains in one week were compiled.

5.8 Measurement of Absolute Deformations from Traffic Load

The control measurement of the absolute deformations of the main beam was carried out by Vintegra s.r.o. using the non-contact method of radar interferometry.

The interferometric radar was used to scan 3 corners (1 upper and 2 lower) and the centre of the supporting cross bar also at locations close to the centre of the NK. Given the position of the radar, the measurement was taken at the nearest panel point to the centre of the NK.

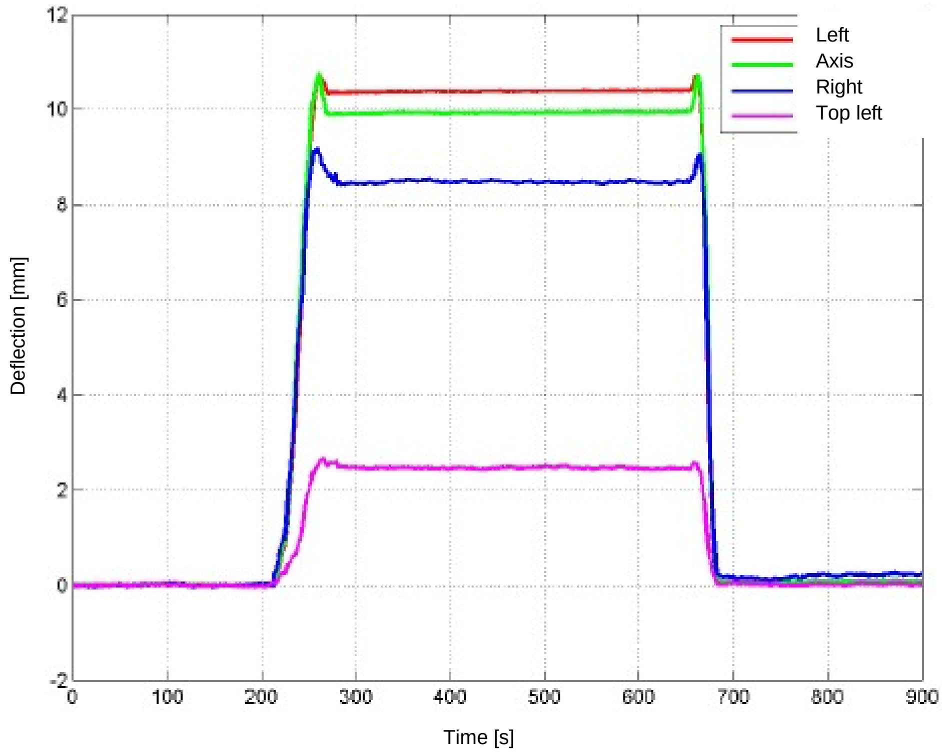


Cross section – measured locations by radar interferometry

The monitored points on the lower part of the structure are fitted with metal corner reflectors to increase the reflectivity and accuracy of vertical displacement measurements. The point on the upper arch of the bridge structure could not be fitted with a corner reflector due to the catenary, the accuracy of the displacement measurement of this point is unfortunately lower (affected by noise)

Measurements were taken during the load test and then during the monitoring of one section of the day from 14:00 to 22:00 to evaluate the traffic load spectra.

The evaluations of the recording of the deformation of the bridge cross-section under the loading condition are shown in the following graph, where the deformation at the individual monitored points is distinguished by colour.



The natural frequencies of the structure were also evaluated from the data obtained during the measurement of the passage of the test load, see the following table.

stress state	Detected natural frequencies [Hz]				
DZS1	2.063	3.334	4.336		
DZS2	2.063	3.334			
DZS3	2.063	3.334	4.334	4.438	5.77
DZS4	2.063	3.334	4.334		5.77
DZS5	2.063	3.334			
DZS6	2.063	3.334	4.334	4.438	5.75
DZS7	2.063	3.334	4.334		5.77
BZS1	2.063	3.334	4.334	4.438	
BZS2	2.063	3.334			
BZS3	2.063	3.334			5.77
BZS4	2.063	3.334		4.438	
resulting	2.063	3.334	4.334	4.438	5.77

5.9 Traffic Load Survey on the Line

For the purpose of the fatigue limit state assessment using the "fatigue damage accumulation" method, the so-called **Palmgren-Miner** hypothesis, it was necessary to obtain information on the traffic load from the origin of the bridge to the planned end of service life, i.e. from 1901 to 2055 (30 years after the bridge reconstruction).

This method of fatigue damage accumulation is one of the most accurate methods and one of the most widespread of the possible approaches. It is a linear hypothesis of fatigue damage accumulation. In the case of this method, the input data is in the form of stress spectra.

The bridge at the location of the line Prague – Main Station – Smíchov is specific with regard to its associated function with the Branický Bridge, which crosses the Vltava River on the southern edge of Prague and carries the so-called Southern Link Radotín – Krč – Vršovice. In 1964, most of the freight traffic was transferred to this line. Some freight trains in the direction of Kladno were left on the bridge together with passenger traffic.

Traffic data was traced in historical documents, which was very time consuming. The historical data were then divided into intensities attributable to the Branický Bridge and intensities attributable to the Pod Vyšehradem Bridge. From the point of view of the overall transport intensity, both lines must be seen as one section.
Historical data is available for the whole network. Such data should be calculated in relation to the length of the lines within Austria-Hungary, from 1918 within Czechoslovakia and then from 1993 within the Czech Republic. The two world wars and the crisis periods after their end are also reflected in the evolution of intensities. The basic source of information is the yearbooks, where traffic data are given.

The split by line has been traced back to 1986, one of the key milestones, as this is where the peak of traffic movements occurred (top of the chart). Subsequently, there was a decline, especially after 1989, when the economy underwent a fundamental change in transport needs. Since 2005, data can be obtained split between freight and passenger transport. The prospective load was considered according to the assumptions of the traffic technology.

For the assessment of the fatigue limit state, the relative spectra of stress variations according to CSN 73 6203:86 in the period 1901 to 2000 were used. These normalised relative spectra related to the load train ČD-Z (LM 71 with α =1.1) were transformed for the load scheme of track load class C3 and then applied to the double-track bridge.
For the period 2001 to 2055, spectra were determined by evaluating the dynamic response from current and forecast trains that were evaluated from the existing traffic mix.

Methodologically, the traffic intensity data were evaluated according to the procedures of Prof. L. Frýba summarised in the dissertation of Ing. L. Žemličková "Equivalent Stress Range of Railway Bridges", 2004 [2.6].

The basis of the methodology is data on the development of the average operational load across the network, which is taken as the total load in millions of gross tonne-kilometres per year depending on the total length of track [2.6].

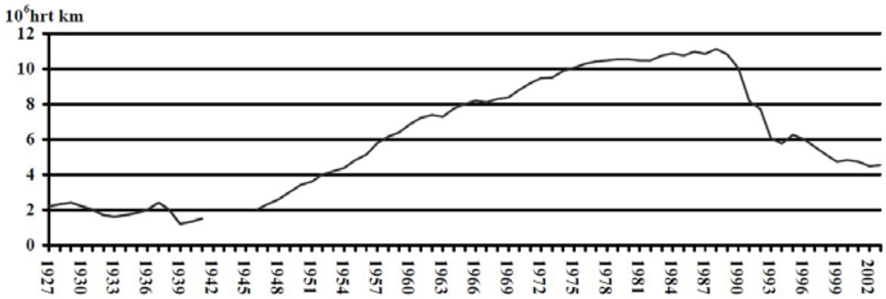
$P_{pz} = c_z / s,$

where P_{pz} average operating load,
c_z total traffic load,
s total track length

The total length of the tracks has changed significantly only once since the 1950s and that was during the break-up of the ČSD into the ČD and ŽSR. The other changes are insignificant in terms of the whole network and therefore only 2 different values are neglected and taken into account.

time interval	network length s [km]
1954–1992	16175
1993–2004	11430

Average operating load



Evolution of the average operational load in the whole network of the Railway Infrastructure Administration (ČD) in the period 1927 to 2002 [source 2.6]

As already mentioned in the introduction, the estimation of the load development from traffic on railway bridges (especially steel bridges) is very important for the determination of the fatigue life of the bridge material, from which the residual fatigue life of selected details is subsequently determined.

The development of the traffic load on the Pod Vyšehradem Bridge at km 3.706 is determined from the tables of traffic load, annual reports from SZCZ and documents from the Central Technical Library of Transport. In order to determine the development of traffic on the Pod Vyšehradem Bridge, the development of traffic on the Branický Bridge is also monitored, which after its commissioning in 1964 takes over a large part of the traffic load, especially from freight trains.

Between 2005 and 2015, both recalculated and not recalculated values are known, while between 1901 and 2004, with the exception of 1990, 2000 and 2001, only not recalculated values are known. The recalculated values include partly the dynamic loading of the track, which is taken into account in sub-factors such as the effect of the maximum permissible speed on the line of passenger and freight traffic, the effect of weight and adverse axle effects from freight traffic and also the proportion of the weight of the driving vehicles on the total weight of the train. However, the dynamic load is introduced using the dynamic coefficient in a later part of the calculation. In order not to multiply the values twice due to dynamic loading, the **unconverted values** are used.

The exact values of the transported million gross tonnes for the Pod Vyšehradem Bridge and the Branický Bridge are known from the tables of operational load from SZCZ. On the Pod Vyšehradem Bridge, the total load is also divided into individual tracks. There is no such division on the Branický Bridge because it is a single-track bridge. These values are known from 2000, 2001 and 2005–2015. From these years, apart from 2015, it is also known how much of the load was passenger trains and how much was freight trains. For the years 2002–2004, the traffic load is determined by interpolation between the known values of 2001 and 2005. The total performance of carriers in the years 2003–2015 is known from SZCZ annual reports, where the values are recalculated. The performance of carriers between 1954–2003 was known from the dissertation [2.6]. The performance is divided into passenger and freight transport. The values in this dissertation [2.6] are unconverted. Based on these data, in years when the load values of both the Branický Bridge and the Pod Vyšehradem Bridge are known, the ratio between the two Prague bridges together and the total capacity in the network is determined, which is a value that shows how much traffic load crosses both of these bridges and is very accurately determined in the years 2000–2015. The percentage of freight and passenger traffic in the total performance of the entire rail network is also calculated. Based on the ratio of the two bridges together to the total capacity of the carriers on the entire network, the total load on both bridges was calculated.

The split between freight and passenger transport is distributed using the ratio between passenger and freight transport performance on the entire rail network in a given year. Traffic load in 1990, where the value of the load on each track on both bridges is known, both converted and unconverted. The distribution of the traffic on the bridges into freight and passenger traffic was calculated assuming that all passenger traffic travelled on the Pod

Vyšehradem Bridge and the total load on the Branický Bridge consisted only of freight traffic. The performance of freight traffic on the Pod Vyšehradem Bridge was then determined as the difference between the performance from freight traffic determined for both bridges together and the freight traffic that crossed the Branický Bridge.

The loading of individual tracks on the Pod Vyšehradem Bridge is divided according to the average ratio of the 1986–1990 total loading of these tracks.
The 1991–1999 load values for both bridges are calculated by interpolation between the known years 1990 and 2000.
Further, in 1986, the value of the load on both bridges is known, but only the calculated value for each track on both bridges. The unconverted value was determined using the ratio between the converted and unconverted 1990 value. The distribution of traffic was again such that all passenger traffic went over the Pod Vyšehradem Bridge and only freight traffic went over the Branický Bridge. The freight traffic load on the Pod Vyšehradem Bridge was again determined as the difference between the total freight traffic for both bridges and the one that crossed the Branický Bridge.
The years 1987–1989 were again calculated by linear interpolation between 1986 and 1990. Then, from 1986–1990 determined the average power on the Pod Vyšehradem Bridge in both tracks and on the Branický Bridge to the total power in the whole railway network. The average loads in the years 1986–1990 on the Pod Vyšehradem Bridge in each track are calculated. The load on both bridges and individual tracks is then determined as the load on the entire network times the average percentage, from 1986–1990, of the performance on the entire network. This is how the years 1964–1985 were calculated.
In 1964, the Branický Bridge was built, so before this year it is assumed that all railway traffic went over the Pod Vyšehradem Bridge. Before this year, the percentages of performance from both bridges were added together and it was assumed that everything was running on the Pod Vyšehradem Bridge.
The years 1954 to 1963 are calculated as network-wide power times the average power in 1986–1990 for both bridges, which equals the total load on the Pod Vyšehradem Bridge. The distribution among the individual tracks was using the average ratio from 1986–1990.
The split between passenger and freight was again made using the ratio of passenger and freight performance across the network to the total. Before 1954, there are no more data from the dissertation [2.6]. The data was then retrieved from the Central Technical Library of Transport. In the library were found data on the performance of the entire network in 1946, 1928, 1927, 1921, 1920, 1919 and also data from 1901–1913, which are from the Austro-Hungarian period.

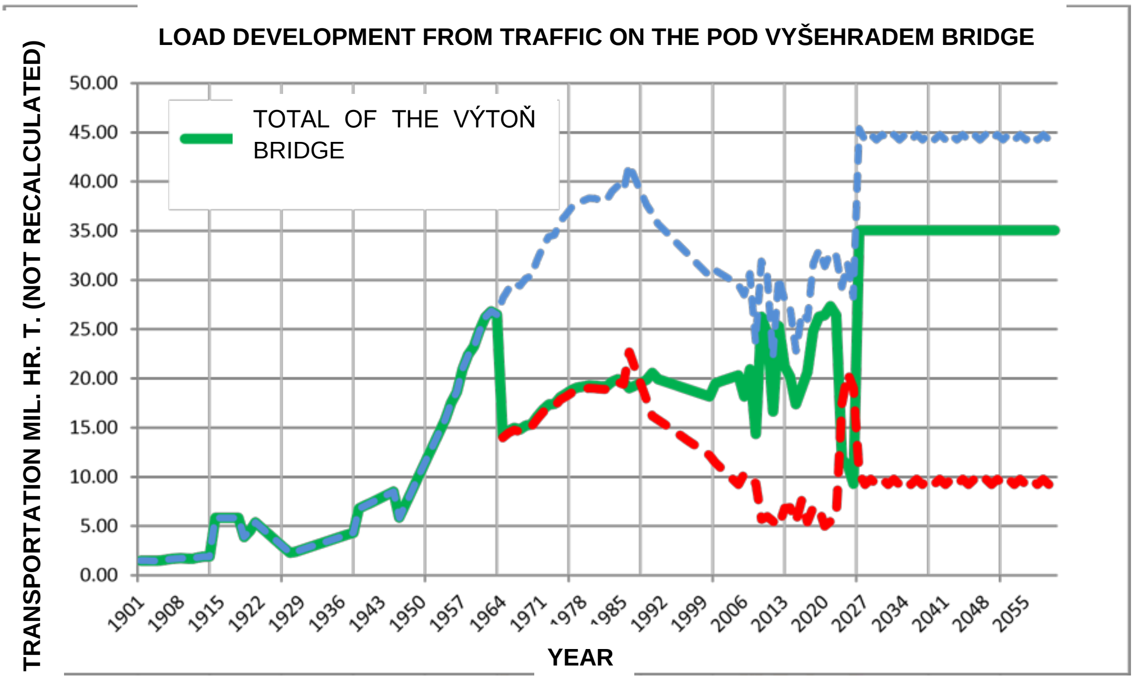
The years 1907–1953 were calculated by interpolation between known years. From the construction of the bridge in 1901 to 1913, the known values were multiplied by the relative length of the lines in Austria-Hungary and later Czechoslovakia. The values during the World Wars, i.e. in 1914–1918 and 1939–1945, were increased by 50% due to the increased need to transport military equipment and supplies. The total load and the load of individual tracks on the Pod Vyšehradem Bridge were again calculated as the total load in the whole railway network in a given year times the average percentage load from the years 1986–1990. In the years 1901 to 1906, according to contemporary documents, only one track was counted on the bridge, so all traffic on the bridge was running over the track in the direction of Smíchov. The 1901–1953 split between passenger and freight traffic is based on the average 1954–1963 split.

Recapitulation of the development of traffic intensity in the Prague junction [m.hr. tonnes/year] – not recalculated

Year	Average traffic load [million hr.t/year]		Note
	Výtoň Bridge	track No.1	
1921	5.365	**1.533**	from 1901 to 1921
1927	2.276	**1.781**	
1937	4.130	**1.633**	

1941	7.347	**3.188**	
1953	14.600	**4.754**	
1969	15.328	**9.757**	
1987	19.289	**9.333**	
1997	18.765	***9.489***	*from 1987 to 1997*
1999	18.376	***9.188***	*from 1987 to 1999*
2000	18.182	**9.637**	
2017	20.623	**10.127**	
2022	26.424	**13.126**	
2025	*9.281*	***10.754***	*reconstruction*
2055	35.040	**17.520**	outlook

The resulting graph of the development of the traffic load intensity on the bridge over the Vltava River and the Branický Bridge is shown in the following graph.



Development of traffic load in the Prague junction (direction Plzeň/Kladno) in the period 1901 to 2055

A review of the cyclical load history shows that the forecast traffic will be nearly double the average traffic to this point. The load on the bridge will therefore increase enormously, which has implications for the design of the bridge reconstruction (design concept). **Compared to the original intensity at the time of the bridge construction, the intensity is 10 times higher**, which has implications for the accumulation of fatigue loads.

5.9.1 Current Rail Traffic on the Bridge

5.9.1.1 Passenger Trains

On the basis of the train schedule provided by SZCZ, all possible variants of the composition of individual passenger trains were identified. These trains can be pulled by 13 different locomotives. The number of carriages is the same for most trains, but for trains pulled by locomotive 362 the number of carriages is very variable. These different variants of train composition were classified into groups according to the axle forces of locomotives and wagons, total weight and length of the train. Then one train with a given locomotive was selected to represent the group. Subsequently, its exact dimensions were retrieved from the website www.zelpage.cz.

Based on the wagon markings, the weight of an empty and a full wagon was found on the website www.atlasvozu.cz . The weight of all carriages was then calculated with full carriages, only Regiojet's trains were calculated with empty carriages, as these trains have their final station at the Main Station in Prague and only go over the Pod Vyšehradem Bridge due to the train stoppage at Smíchovské nádraží. On the basis of their weight and the number of individual train crossings per day, the total load in millions of tonnes per year was determined. However, this figure did not coincide with the data from SZCZ, so the weight of trains that contribute significantly to the total load was reduced by a coefficient. The introduction of the reduction is based on the assumption that the trains are not completely full.

For the CityElefant trains, it was found from the train schedule that the train is usually made up of two trainsets, hence its group is also represented by two linked trainsets, with one trainset being 471+071+971. This train contributes significantly to the total load of the bridge, however, it is not assumed that the train is completely occupied when crossing the bridge, therefore its total weight was reduced by the already mentioned coefficient so that the resulting weight was 2x180=360 t, which is a figure determined by the transport technologist.

5.9.1.2 Freight Trains

Again, based on the train schedule, it was found that an empty freight train passes over the bridge daily, which is not as significant due to its low weight in total and per axle, but is still included in the calculation. Far more important is the Metrans Rail s.r.o. train. This train runs 5 times a week and its weight and length is variable depending on the type of goods being transported. These values were based on data from the transport technologist for 2,000 t and a length of 590 m. The train consists of one locomotive and 29 wagons.

Freight trains are then shown in the table in two groups that do not contain any passenger trains, namely the empty freight train and the fully loaded freight train.

5.9.1.3 Forecast of Traffic Development in the Section

From the train schedule, it is known what quantity of individual trains is running on the bridge now and according to the projections prepared by the traffic technologist it has been assumed what quantity of trains will run here after the reconstruction. On the basis of the ratio of a given group of trains to all trains, to the total number of trains of the same use and to the prospects, the number of trains of each group that should run there after the reconstruction was determined. On the basis of current conditions, it was also determined how many trains will run along the direction of the station, i.e. in the direction of Smíchovské nádraží, and how many trains will run against the direction of the station, i.e. in the direction of Hlavní nádraží or Vršovice railway station.

5.9.1.4 Distribution of Traffic into Train Groups

Trains are divided according to their purpose of use into:

- express train
- fast trains
- multiple unit trains
- locomotive trains
- passenger trains – freight.

Express trains are groups No.6 and No.14, fast trains No.5 and No.7, multiple unit trains No.2, locomotive trains No.8, passenger trains No.1, 3, 4, 11, 12, 13 and freight trains No.9 and No.10.

OVERVIEW OF EVALUATED TRAIN GROUPS

CHARACTERISTIC TRAIN GROUPS						NUMBER OF TRAIN CROSSINGS		
GROUP NUMBER	TYPE	TRAIN DESIGNATION	BASIC WEIGHT [t]	MULTIPLIER	RESULTING WEIGHT [t]	PERIOD 2017 [number/day]	2018–2022 [number/day]	2025–2055 [number/day]
1	Passenger		298.00	1.00	298	3	3	**6**
2	Multiple Unit	REGIOJET	550.00	0.85	469	28	28	**14**
3	Passenger	REGIONOVA	47.00	1.00	47	22	22	**41**
4	Passenger		219.00	1.00	219	2	2	**4**

5	Fast		146.00	1.00	146	11	11	**25**
6	Ex		529.00	0.61	320	29	64	**32**
7	Fast		484.00	0.60	290	18	18	**41**
8	Locomotive	LK	64.00	1.00	64	6	6	**6**
9	Mn	MN	500.00	1.00	500	1	1	**1**
10	Nex	NEX	2066.00	1.00	2066	0.7	0.7	**1**
11	Passenger	CE – FULL	208.50	1.00	209	0	0	**0**
12	Passenger	CE – AVERAGE	180.00	1.00	180	128	128	**238**
13	Passenger	CE – EMPTY	155.40	1.00	155	0	30	**0**
14	Ex	PENDOLINO – full	417.00	1.00	417	4	4	**4**
					TOTAL	**253**	**318**	**412**

OVERALL DEVELOPMENT OF TRAFFIC VOLUME

Total per day	52,054	tonnes	Total per day	96,000	tonnes
Total per year	**19.0**	**million tonnes**	Total per year	**35.04**	**million tonnes**

Note: train schedule dates for 2017
trains that have an abbreviation starting with "Rus" (Disruptive connections) are not included
not included trains that run only a few times a year

These groups were used to determine the spectra of the oscillations using the dynamic response. The traffic overview shows a **large projected increase in traffic after the bridge reconstruction**.

5.10 Static Recalculation of Bridge Structures

The objective of the structural recalculation was to determine the load capacity and to assess the compatibility of the existing 1901 steel structure, taking into account its current structural condition in terms of ultimate limit state, fatigue limit state and serviceability limit state. From the results of the static recalculation, the residual service life of the structure was determined, i.e. the time for which the monitored compatibility of the bridge structure can be confirmed.

The most accurate current procedures in the field of railway bridge design were used to determine the load carrying capacity and the compatibility assessment. In the framework of the assessment of the cross-sections, all the concessions given in the 2015 Methodological Guideline were applied, taking into account the currently upcoming changes.

The static calculation is the basis for the design of the reconstruction of the bridge, which is conceived with the assumption of using the existing bridge structure with the extension of operation for the next **30 years** while maintaining at least the existing compatibility of the line load class **C3/60**, which will, however, allow a prospective increase in the number of train capacities to almost double.
The recalculation of existing bridge structures is carried out according to the new principles given by the Methodological Guideline for Determining the Load-Carrying Capacity of Railway Bridge Structures in **Category D**, which is based on the set of valid standards ČSN EN 1990 – ČSN EN 1996.

5.10.1 Recapitulation of the Static Recalculation of the Bridge at km 3.545

5.10.1.1 Load-Bearing Structure

A summary of the calculation results is shown in the following table:

RECAPITULATION OF STATIC RECALCULATION – NK1 to NK4

Element	Limit state / utilisation in %		Load-bearing capacity	Compatibility	Note
	carrying capacity	fatigue	ZLM71	TTZ / PRTTZ	

MAIN BEAM					
Main beam NK1 to NK3	117%	107%	0.82	D4/70	fatigue life until 2037
Main beam NK4	104%	98%	0.95	-	NK replacement required after 2055!
LONGITUDINAL TRUSS					
Longitudinal truss NK1 to NK4	108%	264%	0.92	D4/70	fatigue life until 2019
TOTAL – Bridge at km 3.545 Výtoň	**117%**	**264%**	**0.82**	**D4/70**	**fatigue life until 2019**

Note:

1) compatibility assessed for $Z_{LM71}<1,^{0}$

The structure **meets the requirements** of the 2015 Methodological Guideline in terms of the serviceability limit state. In the serviceability limit state, the deformation criteria using **73%** deflection for the track load class **D4/70**. For the basic LM71 standard scheme, the utilisation is **104%**, which indicates a lower longitudinal stiffness of the structure.

Another very important aspect is the depletion of the service life of the rail-track elements due to cyclic loading, which is strongly influenced by the ever-increasing intensity of the traffic load. The assessment of the fatigue limit state shows that:

- **the residual fatigue life of the bridge – longitudinal trusses is 1 year (i.e. until 2019)**

From the fatigue damage assessment of the bridge structure it is obvious that the rail-track elements are at the end of their **service life** and it is necessary to pay increased attention to these elements during detailed inspections with regard to the possible development of fatigue failures, i.e. cracks. Out of the elements of the main load-bearing system, the central lacings are the most fatigue-damaged, with higher stress ranges and significant corrosion weakening. These elements should also be given special attention during inspections.

The compatibility of TTZ **D4/70** in the ultimate limit state is assessed for a residual service life of 30 years, however in terms of fatigue limit state the residual fatigue life is only 1 year.
Until the reconstruction is completed, **the intensity of the traffic load on the bridge structure must not be increased**!

In the **short term of 5 years** it is therefore absolutely necessary to ensure the reconstruction of the bridge structure at least in the area of the rail-track in bays 1–3, because the reliability in the fatigue limit state cannot be guaranteed in the long term.
A reassessment is required after the expiry of the limited lifetime. It is to be expected that, in view of the deteriorating structural condition of the steel structure, it will be necessary to reduce the compatibility which would lead to a reduction in the number of train connections in the section in question.

The **fatigue damaged** elements of the main load-bearing system are the **lower chords of** the main beam in the bay
1-3 that must be replaced to meet the 30-year life requirement.

It should also be noted that after the required service life of **2055**, the fatigue life of the main beam wall will be reached and **replacement of the load-bearing structures in bays 1–4 should be considered**.

5.10.1.2 Description of the Scope of the Modifications

The results of the static recalculation of the bridge show that the following modifications are necessary to ensure the required track load class **C30/60** with a residual service life of 30 years:

in the area of the rail-track

-replacement and reinforcement of the longitudinal trusses

-replacement of the connecting supporting cross bars – transverse diaphragms (following the change of the longitudinal trusses) **in the area of the main beams**

-replacement of the lower chord of the main beams of bays 1–3 **in the stiffening area**

-replacement of horizontal stiffening (in connection with the change of longitudinal trusses – increase of transverse stiffness)

5.10.2 Recapitulation of the Static Recalculation of the Bridge at km 3.706

5.10.2.1 General

The results of the calculation were verified with experimentally determined measurements made during the static and dynamic verification test and with an independently performed computational model for the dynamic analysis of the structure.

5.10.2.2 Bridge Load-bearing Structure at km 3.706

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

Element	**Limit state / utilisation in %**		**Load-bearing capacity**	**Compatibility**	**Note**
	carrying capacity	**fatigue**	**ZLM71**	**TTZ / PRTTZ**	
MAIN BEAM					
Upper chord – O	78%	-	1.44		
Lower chord – U	109%	3%	0.87	C3/60	
Lacings – pushed – D	85%	-	1.26		
Lacings – drawn – D	101%	**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 P0	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_{LM71}<1.0

2) the service life of the element until failure is one year, when the accumulation of fatigue damage reaches 100%
(determined without reconstruction)

The structure **meets the requirements** of the 2015 Methodological Guideline in terms of the serviceability limit state. In the serviceability limit state, the deformation criteria using **37%** deflection.

Exceeding the upper limit of the natural frequency limit is taken into account in the assessment of the fatigue limit state by means of a dynamic analysis for a characteristic train composition.

The compatibility of the **C3/40** TTZ is assessed for a **residual service life of 5 years**, i.e. in the short term it is absolutely necessary to ensure the reconstruction of the bridge structure because the compatibility for the **C3/40** line class **cannot be guaranteed in the long term**.
Until the reconstruction is completed, **the intensity of the traffic load on the bridge structure must not be increased**!

A reassessment is required after the expiry of the limited lifetime. It is to be expected that, in view of the deteriorating structural condition of the steel structure, it will be necessary to reduce the compatibility which would lead to a reduction in the number of train connections in the section in question.

In addition, a compatibility assessment was performed for the C2D2/40 hybrid TTZ assuming residual life 5 years

It is related to the classification of vehicles and trainsets into track load classes according to ČSN EN 15 528, where existing traction vehicles with axle pressure over 20 t correspond to track load class D2 and according to the currently applied legislation are classified into track class C3.

Another very important aspect is the depletion of the service life of the rail-track elements due to cyclic loading, which is strongly influenced by the ever-increasing intensity of the traffic load. The assessment of the fatigue limit state shows that:

- **the residual fatigue life of the bridge – longitudinal trusses is 6 years (i.e. until 2024)**
- **the existing crack on the L2 longitudinal truss at the 75th bridge beam on NK2 must be repaired by 2022**

From the fatigue damage assessment of the bridge structure it is obvious that the rail-track elements are at the end of their **service life** and it is necessary to pay **increased attention to these elements during detailed inspections with regard to the possible development of fatigue failures, i.e. cracks.** Out of the elements of the main load-bearing system, the central lacings and perpendiculars are the most fatigue-damaged, with higher stress ranges and significant corrosion weakening. These elements should also be given special attention during inspections.

5.10.2.3 Description of the Scope of Bridge Modifications at km 3.706

The results of the static recalculation of the bridge show that the following modifications are necessary to ensure the required track load class **C30/60**:

in the area of the rail-track

- replacement and reinforcement of the longitudinal trusses
- replacement and reinforcement of the supporting cross bars chords, including a change in the position of the end of the lamellas **in the area of the main beams**
- replacement of centre perpendiculars V.4 to V.8
- replacement of the centre lacings D.5 to D.10 **in the stiffening area**
- Reinforcement of the braking B – (increase in the stiffness of the longitudinal force distribution)
- reinforcement of the upper horizontal stiffening mullion (increase in cross-section stiffness)

6. New Condition of Bridges

6.1 Bridge Characteristics (new Condition)

The reconstruction does not change the characteristics of the bridge structure. The load capacity parameter is modified by replacing or reinforcing the elements of the load-bearing structure.

Design load (standard):	generally α.LM71, α.SW/0, α = 1.21 - 2nd class according to Z4 to ČSN EN 1991-2 Assessment of compatibility for line load class C3 or D2 for verification of local stresses in rail-track sections
Load-bearing capacity – existing condition:	SO 20-20-04 Railway bridge at km 3.545 – Výtoň Z_{LM71} = **0.82** – main load-bearing system Z_{LM71} = 0.92 – local bearing system (rail-track) Compatibility of **TTZ D4/70** **Residual fatigue life until 2019** SO 20-20-05 Railway bridge at km 3.706 – Pod Vyšehradem Z_{LM71} = 0.77 – main load-bearing system Z_{LM71} = **0.61** – local load-bearing system (rail-track) Compatibility of **TTZ C3/40** **Residual life until 2023 – 5 years** **Residual fatigue life until 2024**
Load-bearing capacity – new condition:	SO 20-20-04 Railway bridge at km 3.545 – Výtoň Z_{LM71} = 0.95 – main load-bearing system Z_{LM71} = **0.95** – local load-bearing system (rail-track) Compatibility of **TTZ D4/120** SO 20-20-05 Railway bridge at km 3.706 – Pod Vyšehradem Z_{LM71} = 0.87 – main load-bearing system Z_{LM71} = **0.85** – local load-bearing system (rail-track) Compatibility of **TTZ C3/60** **Residual life 30 years until 2055**

The proposed measures achieved the required parameters of the construction specification.

6.2 Scope of Modifications

The main construction target is the Vltava River bridge. According to a detailed survey carried out in 2017, the bridge structure from 1901 is in a **technically unsatisfactory condition**. In particular, it is a corrosion weakening of the steel support structure, which is accelerating in nature and accelerates over time. Overall, the current condition of the bridge elements can be characterised as being at the limit of its service life and in the short term of **about 5 years** it will be necessary to **arrange for the reconstruction** of the bridge structure.

Another very important aspect is the depletion of the service life of the rail-track elements due to cyclic loading, which is strongly influenced by the ever-increasing intensity of the traffic load. The assessment of the fatigue limit state shows that:

- the residual fatigue life of the bridge at **km 3.545** is **1 year** (i.e. until 2019)
- the residual fatigue life of the bridge at **km 3.706** is **6 years** (i.e. until 2024)

From the fatigue damage assessment of the bridge structure it is obvious that the rail-track elements are at the end of their **service life** and it is necessary to pay increased attention to these elements during detailed inspections with regard to the possible development of fatigue failures, i.e. cracks. Out of the elements of the main load-bearing system, the central lacings are the most fatigue-damaged, with higher stress ranges and significant corrosion weakening. These elements should also be given special attention during inspections.

The scope of the modifications is designed taking into account the findings of the detailed survey and the conclusions of the static recalculation. In particular, it is the limitation of service life due to cyclical loading, which is greatly influenced by the prospective increase in traffic intensity to almost double.
According to the Report from the regular inspection for 2017, the construction condition of the supporting structure is in Stage 3 and the substructure in Stage 2.
The aim of the modifications was to maintain the existing C3 class of line with an associated speed of 60 km.h^{-1} for a planned service life of 30 years.
Furthermore, the width arrangement on the existing bridge **does not comply with the conditions for operation** of existing bridge structures according to the Directive 16/2005 for the station district, i.e. 2.5 m. Furthermore, it is a requirement for compatibility by line load class D4/120,
The free width on the bridge cannot be adjusted and the compatibility parameter **cannot be adjusted without replacing the load-bearing structures**.

On the basis of the above and with regard to the current monument protection, the following is proposed within the construction: **reconstruction of the bridge structure**

Note: the reconstruction in the scope of the construction specification includes replacement of damaged elements of the supporting structure, their reinforcement and rehabilitation of the substructure

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 period 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**!

Note on the implementation of SO:

After the reconstruction, the medium pressure gas pipeline cannot be placed on the bridge structure, which is currently a source of danger to the railway and contrary to the requirements of ČSN 73 6201

For the bridge structure SO 20-20-05, sub-structures that are directly related to the bridge structure are kept for the sake of management and property:

SO 20-20-05.1 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem – Footbridges

SO 20-20-05.2 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem – Day beacons

There is also a separate structure that deals separately with securing the bottom of the piers, which is significantly eroded by the erosion of the water flow. In this way, it will be possible to carry out separate implementations in time and possibly even in advance of the actual construction of the bridge structure.

SO 20-20-05.3 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem – Securing the bottom at the piers

The sub-structures are described in the following chapter **6.5 - Related Construction Sub-structures to SO 20-20-05**

6.3 Basic Data

6.3.1 Design Load and Interoperability (TSI)

For the design of new bridge structures, the load corresponding to the category of **Class 2** lines according to the Category of railway lines in terms of bridges according to the amendment Z4 to ČSN EN 1991-2 is valid. This corresponds to the LM71 load model with a load classification factor of α=1.21.
According to the Commission Regulation (EU) No 1299/2014 (TSI 1299/2014/EU), Paragraph 4.2.7.1., a minimum classification factor α **=0.91** is required for the given category of the reconstructed line section **P5** (20t/axle) and the connecting **F4** (18t/axle).

For the assessment of existing structures, the principles of the Methodological Guideline for Determining Load-Carrying Capacity 2015 are followed.

6.3.2 Track on the Bridge

The bridge is located in the station district of the railway station Praha-Smíchov and the Praha Vyšehrad. The line is double-tracked. The line speed in the given section is limited by the adjacent directional curves and by the compatibility of the bridge structure and is 60 km/h.
The track on the bridge is in a straight line without an elevation change in the section in question. The bridge grade line corresponds to the balanced existing condition throughout the entire bridge section. The new grade line is horizontal 0,00 ‰.

6.3.3 Spatial Organisation on the Bridge

For the bridge in the station perimeter, the free bridge cross-section **VMP 2.5** is applied in accordance with the **SZCZ DG 16/2005**.
In the section of the Vltava River bridge it is possible to provide only the **Z-GC** cross-section, with the proviso that the recesses in the openings of the trusses, in which the required VMP 2.5 is met, can be used to ensure safety during the detour. These recesses must be added to the bridge structure at distances of up to 20m at the location of the NK trusses.

Approval is being secured from the SZCZ DG, OTH (O13) for the above condition.

6.3.4 Spatial Organisation under the Bridge

In the area of the Vltava River bridge, a navigation profile of 7.0 m is ensured at the highest navigation level. In the area of the foreland on the local road Rašínovo nábřeží, in the existing condition, due to the concurrence with the tram line, the height of the passage section is limited to 3.1 m. The reconstruction will slightly improve the condition to about 0.20 m, which is still outside the requirement of ČSN 736201.
With regard to the technical possibilities of the solution, the parameter of the lack of height must be discussed with the road owner and the relevant transport department.

6.4 Description of the Technical Solution

6.4.1 Basic Concept

Within the reconstruction of the OK bridge over the Vltava River and on the Vltava foreland it is planned to replace degraded and insufficiently load-bearing parts. In addition, the steel structures will be reinforced to meet the client's input requirements.

The protective coating system will be renewed on all structures.

A condition for putting the reconstructed bridge into operation will be to perform a technical safety test in accordance with Decree No. 177/1995 Coll. in the form of a main inspection according to SZCZ S5 and a static load test according to ČSN 73 6209.
The load test will test the load-bearing structures in all seven bays.

6.4.2 Load-bearing Structure

6.4.2.1 Pod Vyšehradem Bridge (km 3.706)

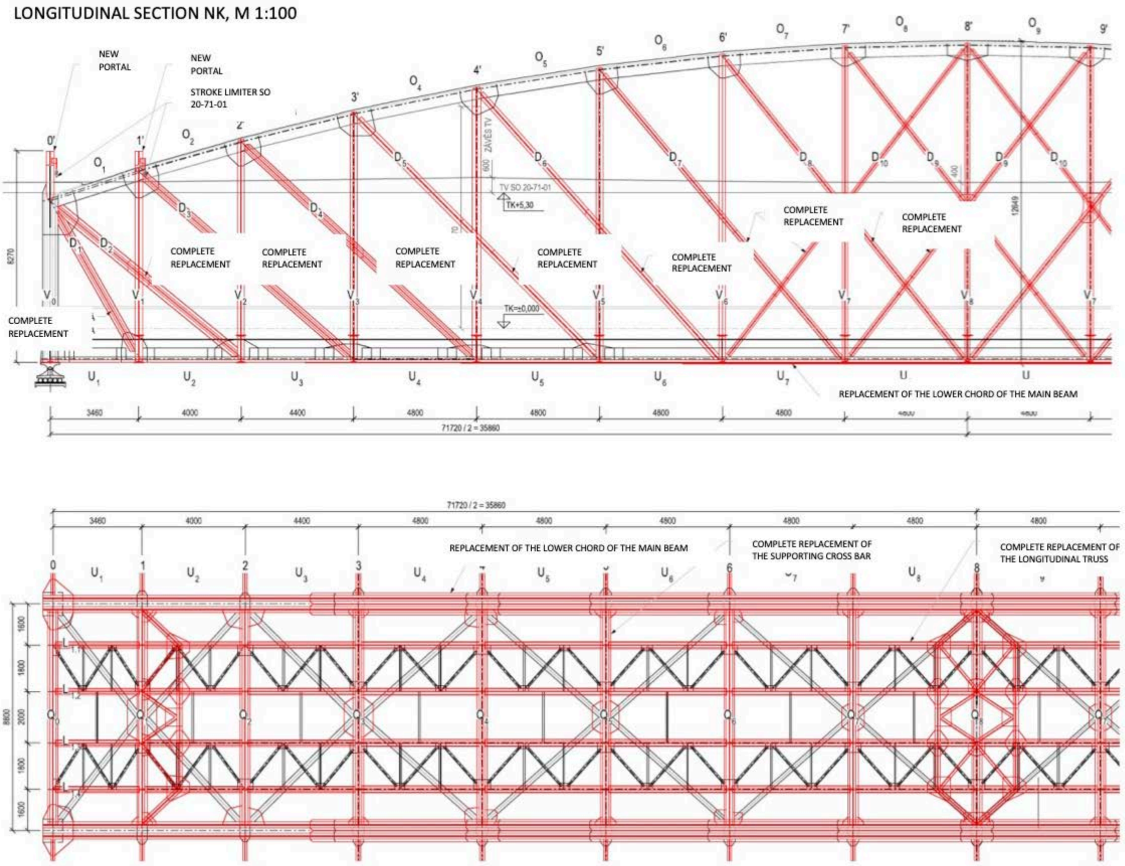
Corrosion-damaged elements of the bridge's supporting steel structures will be replaced. In particular, the drawn elements are more susceptible to destructive failure.
The scope of the replacement is assumed as follows:

- the lower chord of the lower flange,
- lacings D1 to D10 completely
- perpendiculars V1 to V8 completely,
- replacement and reinforcement of the chords on the supporting cross bar P0 to P8,
- replacement of the longitudinal trusses complete with adjustment for the horizontal bridge beam bolt,
- replacement of the upper horizontal stiffening including the end portals (partial return of the bridge to its original historical appearance)
- reinforcement of braking stiffeners,
- replacement of the bridge stiffening between the longitudinal trusses,
- replacement of the splice plates of the horizontal sub-bridge stiffening.

In total, **approximately 63%** of the steel structure elements will be replaced, which will require adequate time for the actual implementation, which will take place at the construction site above the watercourse. The construction is expected to take 42 months to complete over 4 construction seasons.
The total expected weight of the parts to be replaced will be 3 x 376t ~ **1,130t**.

In connection with the replacement of parts of the OK, another **8%** of the elements will have to be dismantled and refitted. This applies in particular to the floors of the inspection walkways and the lower stiffening.

The reconstruction also includes the replacement of the pedestrian bridge brackets (see SO 20-20-05.1). The consoles will be delivered completely new in the external shape corresponding to the condition. An external ledge with a railing and an internal string course will be used in reverse. The inner railings will be replaced with new ones meeting the requirements of ČSN 73 6201 for the separation of railway and pedestrian traffic. The walking surface will be replaced with a steel walking surface, possibly with the addition of a hardwood walking surface.



Reconstruction design – elements to be replaced (in red) of the steel bridge structure at km 3.706

6.4.2.2 Výtoň Bridge (km 3.545)

For structures in bays 1 to 3, the lower chord will be replaced similar to the 1997 repair in bay 4. The rail-track will be reinforced in all structures. A lower chord will be added to the longitudinal trusses. The supporting cross bar will be reinforced with chords and filler rods. The bridge horizontal stiffening will be replaced.

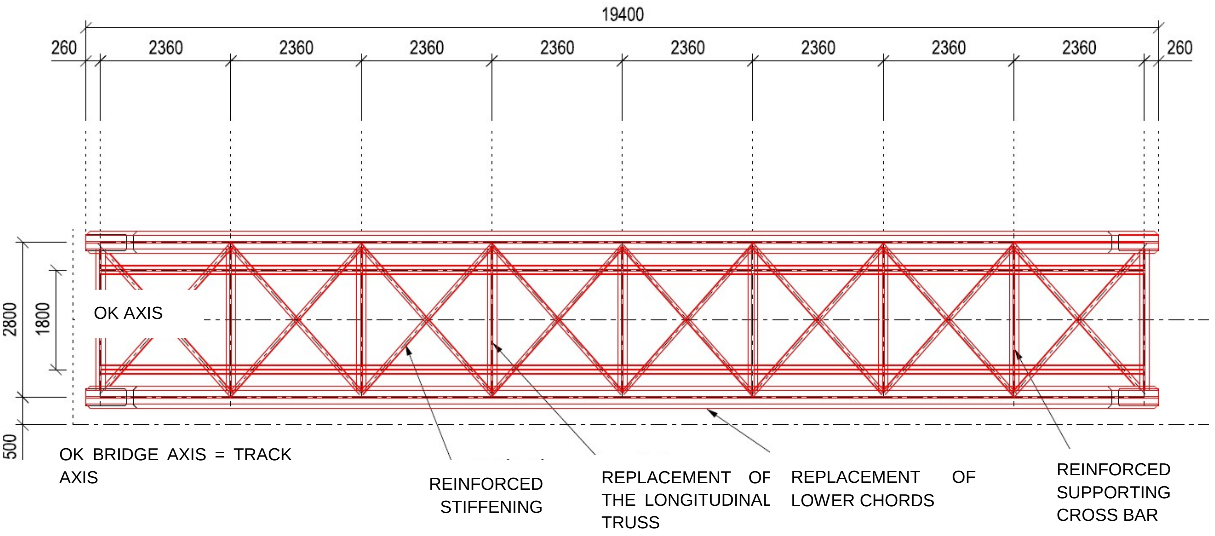
The bridge beams will be adapted for fixing with a horizontal bolt.

The bearings will be replaced.

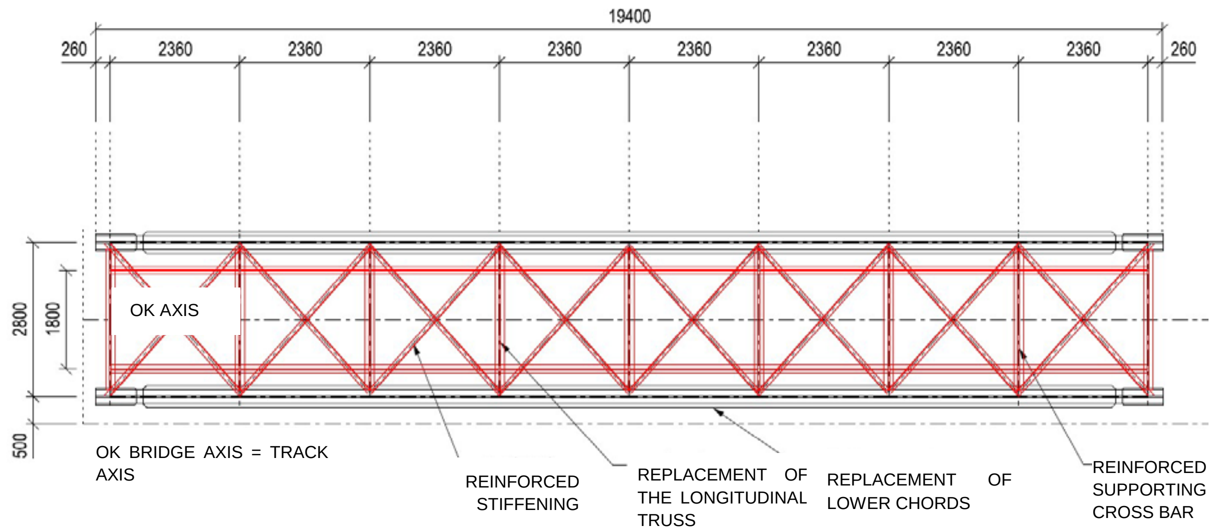
The reconstruction of the OK also includes the replacement of the brackets of the inspection footbridge. The consoles will be delivered completely new in the external shape corresponding to the current one. The floor part in bay 4 will be used retrospectively.

In bays 1 to 3, the floor section will be supplied in new sheet metal to replace the existing timber floor.

The total weight of the parts to be replaced is expected to be 53% i.e. 3 x 32 t **~ 96t** for NK1 to NK3 and 22% i.e. 14t for NK4. In total, **~110 t** of elements will need to be replaced, which is on average **46%** of the total weight.



Reconstruction design – elements to be replaced (in red) of the steel structure NK1–NK3 of the bridge at km 3.545



Reconstruction design – elements to be replaced (in red) of the steel structure of the NK4 bridge at km 3.545

6.4.3 Substructure

For the substructure, surface rehabilitation including jointing is foreseen. The shafts of abutments and piers will be grouted. For the new deposits, sub-base reinforced concrete blocks will be established.

6.4.4 Foundation of the Substructure

In order to ensure the foundation of the substructure in the parts built in 1871, the underground was grouted, i.e. the area of the wooden pile grid, which is at the boundary of the groundwater level.
The jet grouting columns will be used to rehabilitate the sub-base.
The rehabilitation works concern the abutment O01 and pier P03 of the bridge on the foreland at km 3.545 and the abutment O02 of the bridge over the Vltava River at km 3.706.

6.4.5 Bridge Equipment

Railway utilities are located on the bridge structure. The communication and signalling lines are located in the space between the penetrations in the supporting cross bars similarly to the existing situation. This position is suitable in terms of limiting theft and damage to cable lines.

On the left side there is only a bypass line (part of the catenary), which serves as the third power supply track between TCS Chuchle and TCS Třešňovka. The line is designed as it is in the current state – brackets attached to the side of the bridge structure. The amplification line is not designed according to the energy calculations.
On the bridge structure SO 20-20-05, the TCL is solved by means of SIK brackets, which are fixed on the bridge structure.
In case the construction "Reconstruction of the Line Praha hl. n. (excl.) – Praha-Smíchov (incl.)" (Construction 1) is not yet implemented, it will be necessary to temporarily erect anchor poles in km approx. 2.60 (replacement for the existing No. 103, 104). These masts will be only temporary and will be dismantled after the construction of the "Reconstruction of the Line Praha hl. n. (excl.) – Praha-Smíchov (incl.)". The existing masts 99, 100, 101, 102 and 109A will also have to be temporarily retained during this concurrent construction. It will be necessary to keep the 109A mast in case the construction in the Smíchov section (Construction 3), in which this mast is cancelled, is not yet implemented.

Only two feeding tracks will be maintained during construction. One will be a catenary line and the other will be a bypass line (2x120Cu) mounted on brackets that will be attached to a temporary, single-track bridge.
The foundations for the masts are designed within the following structures:
- SO 20-20-04 – mast at km approx. 3.520 and 3.579
- SO 20-20-05 – masts at km 3.60
- SO 20-23-01 – masts at km 3.848

The footbridge lighting is located on brackets on the supporting structure and the actual wiring is located on the brackets of the outer walkways.

The cantilevered exterior sidewalks are addressed under SO 2020-05.1 – Sidewalk Footbridges from the perspective of a different property owner and management. From the structural point of view, they are an integrated part of the bridge structure SO -20-20-05.

The railing of the footbridges are equipped with day beacons, which are addressed within SO 20-2005.2.

6.4.6 External Equipment

There are no non-railway networks on the bridge structure SO 20-20-05 that are not related to the operation of the bridge.

On the footbridges, a high-voltage cable will be placed in a protector on the right side of the bridge
6 kV (or prospectively 22 kV), which is managed by SZCZ, RD Prague, SSE.

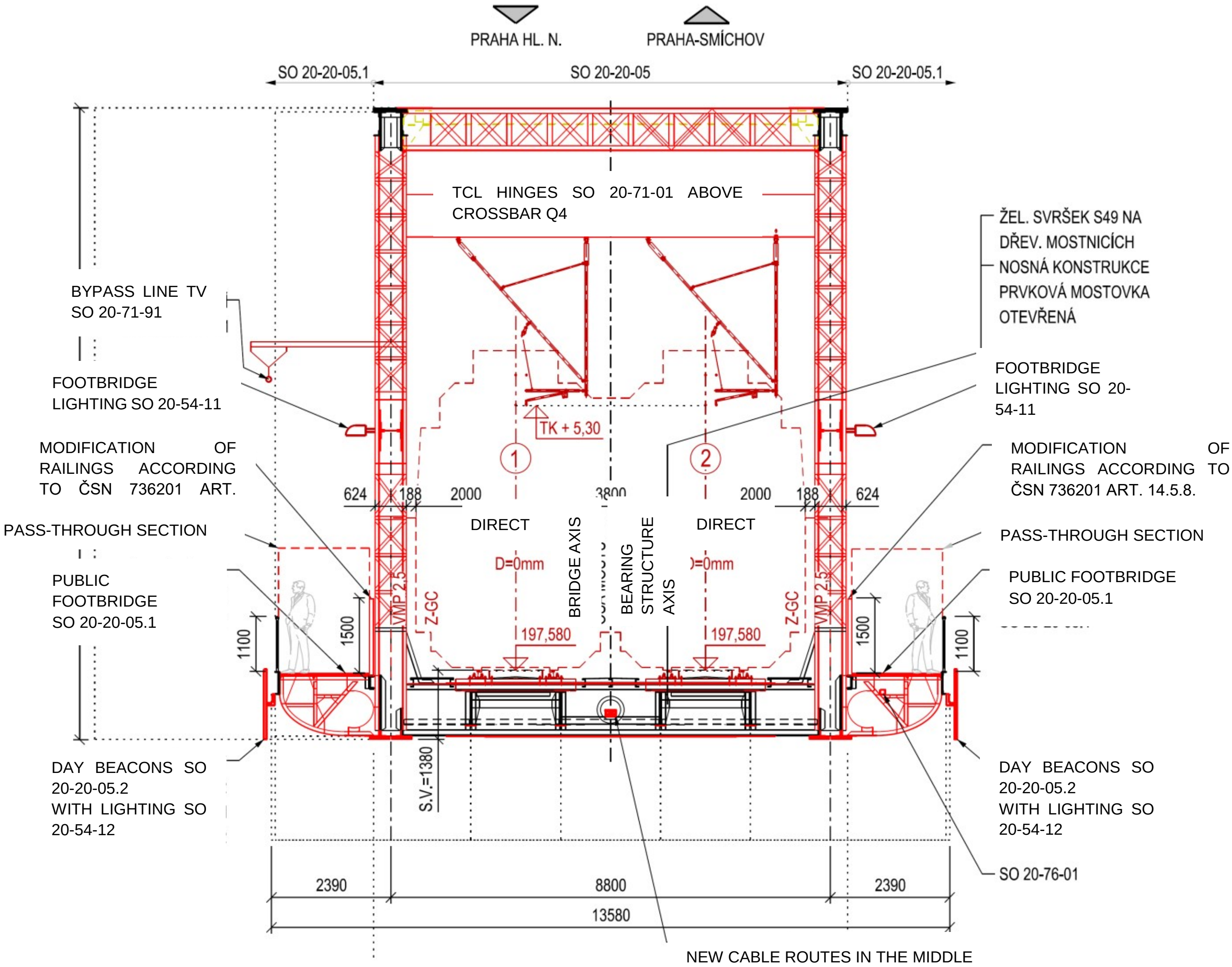
Note: The existing gas pipelines and non-railway networks have been moved from the bridge to the boreholes under the Vltava River.

6.5 Related Construction Sub-structures to SO 20-20-05

6.5.1 SO 20-20-05.1 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem – Footbridges

As part of the reconstruction of the NK bridge SO 20-20-05, the existing footbridges will be replaced with new ones of similar external shape. Only the outer ledge beam and the inner longitudinal beam will be used from the original footbridge structure. The railings will be refurbished and the corroded parts will be replaced with new ones. A new walking cover will be installed on the footbridges. The nature of the new cover will be determined as part of the documentation discussion. The basic solution is a walking layer of orthotropic panels with a straight-through insulation skim coat. Another alternative is a long-lasting, weather-resistant tropical hardwood surface. The existing cover made of coniferous wooden beams does not meet the criteria of durability and maintenance-free operation. Furthermore, in terms of pedestrian safety, the existing cover does not meet the requirement for surface roughness (shear friction value of 0.6).

The ownership and management of the new footbridges will be handed over to the Capital City of Prague.



Cross section in the middle of the span – footbridges on the outer sides of the NK bridge

The placement of external equipment on the footbridges is not considered, i.e. the placement of cable lines that are not related to the operation of the bridge and footbridges.
On the outer railing, there will be day beacons, see SO 20-20-05.2.

Lighting of pavements, which is part of SO 20-54-11 Pod Vyšehradem Bridges, modification of public lighting on footbridges.

A crosswalk will be provided on the upstream side of the temporary bridge during construction. Pedestrian traffic will be interrupted during the rehabilitation works. The total period of service interruption will be **approximately 2.5 months** for each track closure. The temporary footbridge will be illuminated.

6.5.2 SO 20-20-05.2 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem – Day beacons

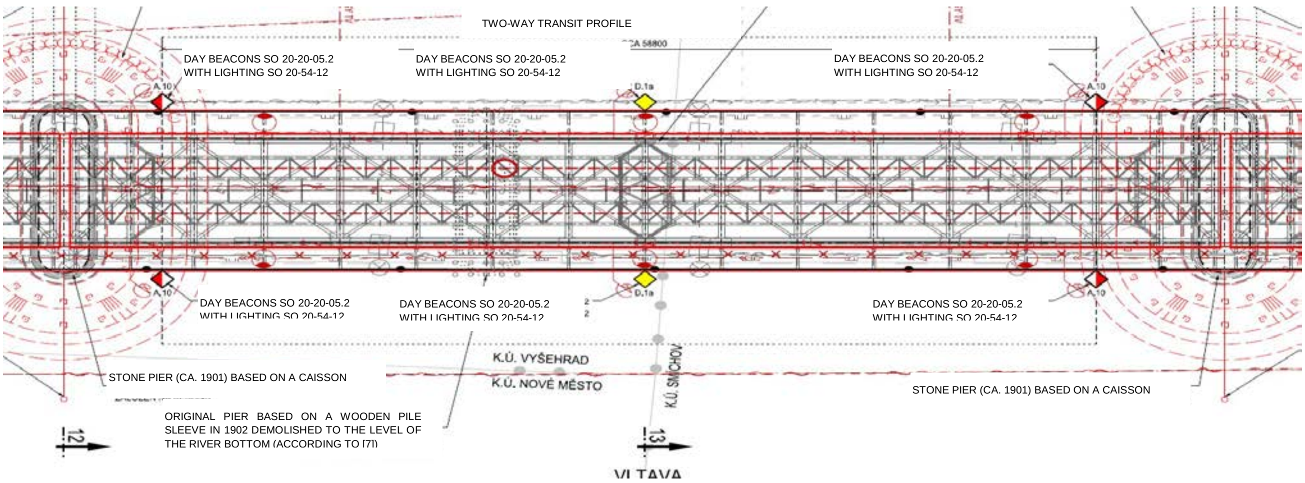
To ensure safe navigation, day beacons are installed on the railing of the footbridges in the central bridge opening. The position and extent of the day beacons will correspond to the existing situation in accordance with Decree 67/2015 Coll.
The day beacons will be illuminated by a led perimeter strip. Within the SO, radar rod reflectors of fixed construction will be mounted on both sides of the piers.

Furthermore, the structure solves all temporary conditions of day beacons that will be in the construction and reconstruction of the bridge over the Vltava River. These include:

- placement of temporary day beacons on the bridge makeshift and the assembly platform for split navigation traffic,
- placement of radar reflectors on the mounting trestles,
- protective buoys and other shore day beacons to warn of a change in the navigation regime,
- day beacons in the assembly procedures for the construction of sheet pile sumps around piers and for mounting trestles.

Temporary day beacons will be illuminated.
The power supply of permanent and temporary day beacons is designed in SO 20-54-12 Pod Vyšehradem Bridges, modification of the power supply of the navigational lane signalling.

Note: river km 55,35, max. navigable level 188,28 m above sea level Bpv



Ground plan of the middle opening of the P1/P2 pier – navigational lane state

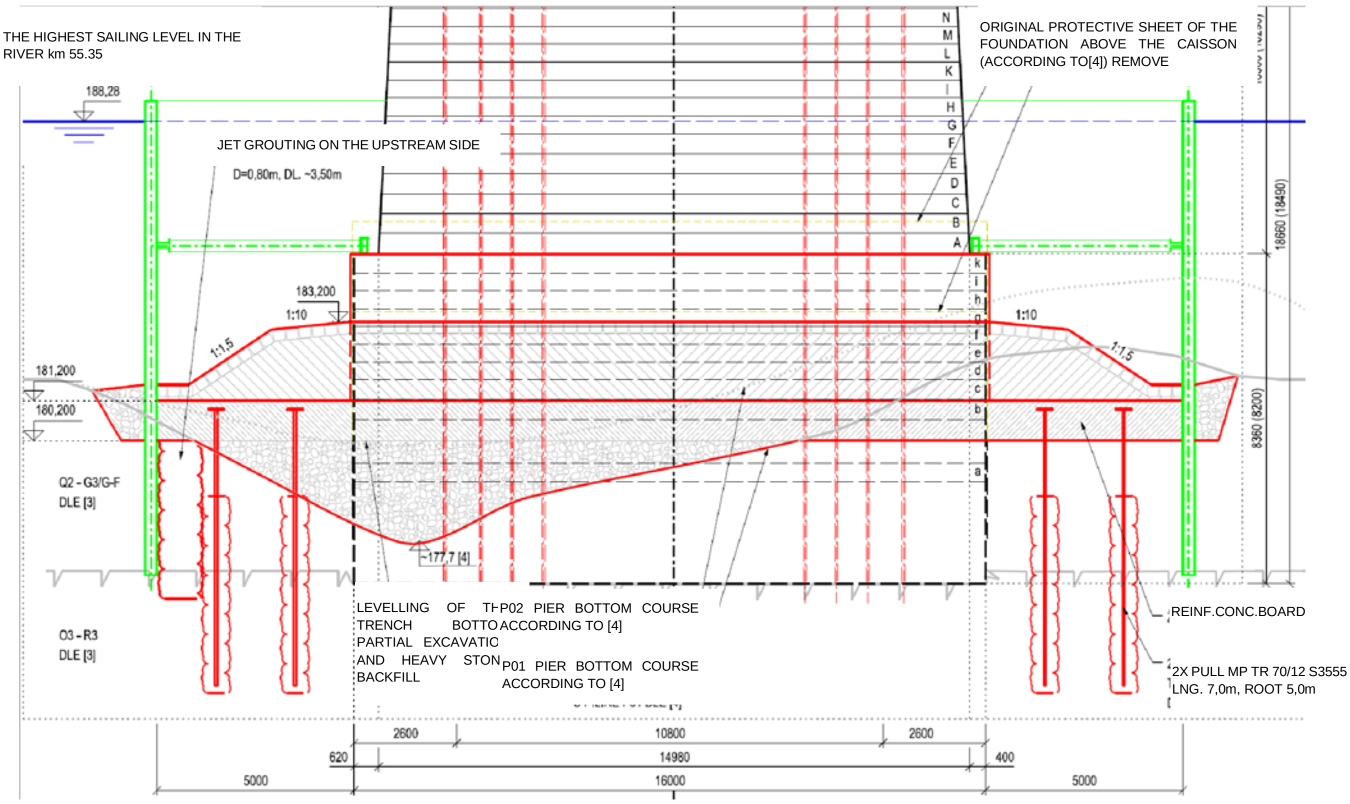
6.5.3 SO 20-20-05.3 Pod Vyšehradem Bridges, railway bridge at km 3.706 – Pod Vyšehradem – Securing the bottom at the piers

The sub-structure deals with the rehabilitation of the Vltava River bed, which is heavily degraded by the erosive action of water eddies (caverns up to 5m deep). This involves securing the bottom surface around the piers and creating a barrier to protect against further erosion.

An anchored concrete slab will be made in the sealed sheet pile sump around the piers (due to buoyancy) and then the bottom around the piers will be modified with protective heavy stone paving in concrete. In the face of the fortification a protective shield will be made of jet grouting columns.

At the same time as the bottom treatment, the stonework will be cleaned and grouted. To protect against the effects of water, a protective layer of shotcrete reinforced with stiffening nets will be installed.

From the point of view of implementation, it is necessary to note that the cavern in the bottom reaches down to the bedrock, i.e. almost to the foundation joint of the pier (the base of the caisson). This unfavourable situation needs to be addressed in the short term, approximately **3-5 years.** Due to the possibility of separate implementation, the activities related to the rehabilitation of the bottom are conducted under this sub-structure. It is owned and managed by SZCZ, RD Praha.



Longitudinal section in the face of piers P1/P2 with proposed modification of the bottom securing around the piers

Warning:

This SO must be implemented before or simultaneously with the installation of the sumps for the installation trestles.

7. Implementation of the Structure

7.1 Technological Principles of Construction, Reconstruction of the Bridge Structure, Construction Procedure

Reconstruction of the steel structure to the extent proposed requires the structure to be lightened by supporting it on trestles.

The reconstruction will be carried out on an assembly platform in an displaced position (in the direction of the river).

To ensure the operation during the construction period, a temporary bridge span of 3 x 72 m **type ŽM16 in 2p2sz configuration** will be built. The makeshift will be assembled in the area of Vnislavova Street and extended longitudinally to the left bank of the Vltava River.
A 1.5m wide footbridge for public traffic will be placed on the left side of the temporary bridge. A footbridge with a width of 0.8 m for non-public traffic will be placed on the right side of the temporary bridge (SZCZ line administration).

The load-bearing structures of the Výtoň foreland will be reconstructed out of the locality in the contractor's bridgeworks. Disassembly and reassembly will be carried out by crane equipment from Vnislavova Street (disassembly from Svobodova Street is not suitable due to the railway TCL and tram TCL.

7.2 Requirements for Closures, Speed Restrictions and Other Operational Restrictions

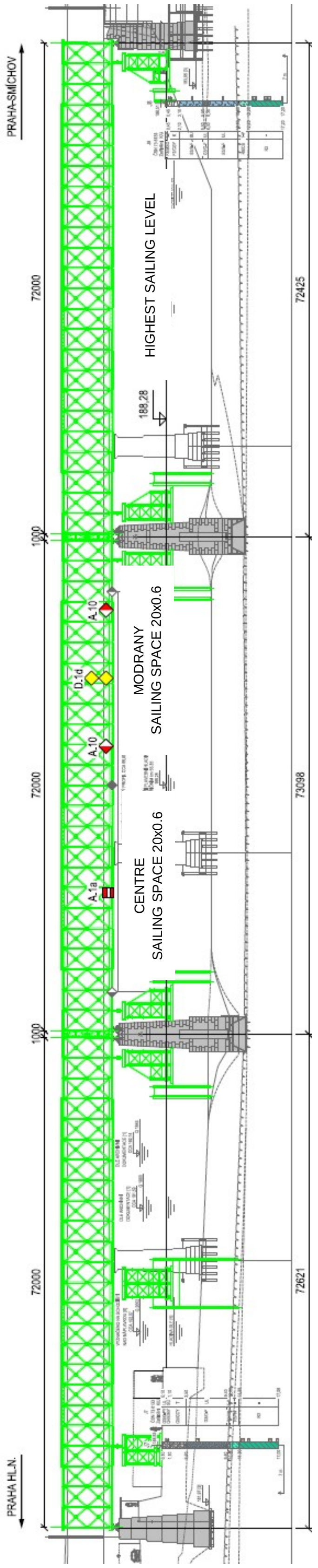
7.2.1 Requirements for Limitation of Operation on the SZCZ Line (closures)

For the reconstruction of the bridge, single-track operation will be ensured for **4 construction seasons**, i.e. **42 months**, which represents a major limitation of the traffic capacity of this section.
The total time will be specified on the basis of the discussion of the construction with the Hygiene Station of the Capital City Prague, when the limit for the noise load due to the construction will be determined.

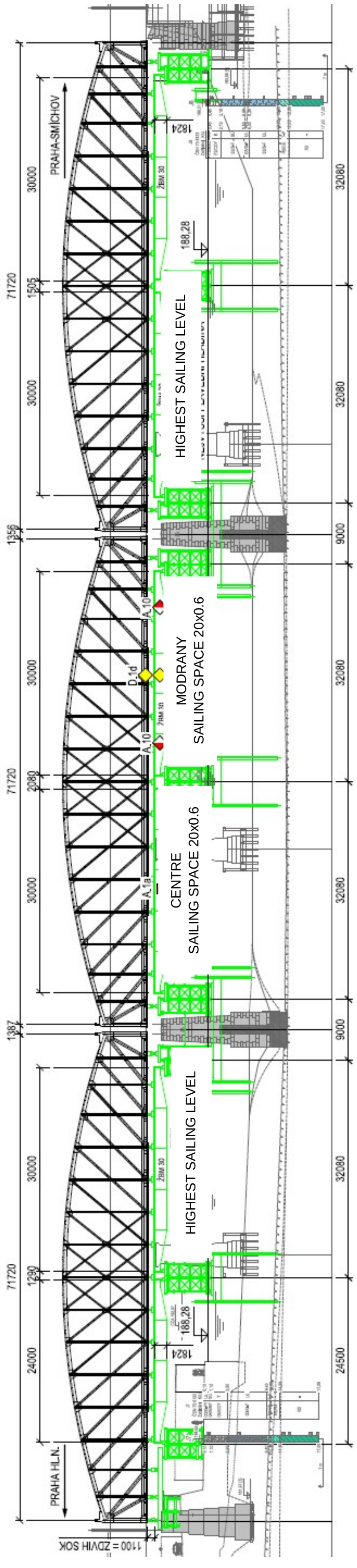
7.2.2 Restrictions on Navigation

During the construction period of approx. 42 months, the central navigation opening will be divided into two with a width of at least **2 x 20m and a section height of at least 6.00m.** During the construction of the barriers, there will be a narrowing to only one section with two-way traffic (short-term for a period of about 14–21 days).
For limit construction operations above the watercourse, navigation will be completely interrupted for 24 hours (max. 48 hours).

All temporary conditions will be marked with day beacons in accordance with Decree 67/2015 Coll., Rules of Navigation.



OPERATIONAL BRIDGE



TEMPORARY SUPPORT FOR SOK DURING RECONSTRUCTION

7.2.3 Road Traffic Restrictions

Road and tram traffic will be significantly restricted at the bridge site due to the extent of the reconstruction.

The assembly area is planned in Vnislavova street. The dismantling and reassembly of SO 20-20-04 will be carried out using a crane located in Vnislavova Street. In SO 20-20-05, the installation of a temporary bridge will take place, which will be extended longitudinally over the Vltava River in the direction of Smíchov.

7.2.4 Restrictions on Pedestrian Traffic

Road and tram traffic will be significantly restricted at the bridge site due to the extent of the reconstruction.

7.2.5 Limitation of the Floating Depot Area

In the length of approximately 80m in front of and beyond the bridge over the Vltava River, mounting trestles will be placed on the floating depot to support the bridge structure. Pedestrian, cycling and automobile traffic will be restricted during construction. **The exit to the floating depot by the bridge will be completely closed for trucks over 2.5 t.** Passage under the bridge will be possible in both directions in the basic width of 4.0 m + 2.0 m pavement, i.e. the opening clearance is 6.0 m. The floating depot area will be completely closed for the necessary period of time (carrying out risky assembly activities.)

8. Main Related Structures

1. Construction

SO 20-20-01 Pod Vyšehradem Bridges, railway station at km 3.390 – Garages I
SO 20-20-02 Pod Vyšehradem Bridges, railway bridge at km 3.415 – Vyšehradská
SO 20-20-03 Pod Vyšehradem Bridges, railway station at km 3.470 – Garages II

2. Construction

SO 20-20-04 Pod Vyšehradem Bridges, railway station at km 3.545
SO 20-20-05 Pod Vyšehradem Bridges, railway bridge at km 3.706

SO 20-23-01 Pod Vyšehradem Bridges, retaining walls at km 3.834–3.849
- is connected to the parallel wings of the OP2 abutment and runs along the pavements on both sides

SO 20-31-01 Pod Vyšehradem Bridges, modification of pavements (access to the footbridge)
- solves the connection of the pavements along the abutments and the connection to the footbridges on the bridge (SO 20-20-05.1)

Railway Superstructure and Substructure
SO 20-10-01 Pod Vyšehradem Bridges, railway superstructure
SO 20-11-01 Pod Vyšehradem Bridges, railway substructure
SO 20-15-01 Pod Vyšehradem Bridges, track alignment

Interlocking Equipment (SZZ)
PS 20-01-11 Railway station Praha-Smíchov, connecting tracks, SZZ

Trunk cable (DK), distance optical cable (DOK), suspension optical cable (ZOK)
PS 20-02-51 Pod Vyšehradem Bridges, modifications of existing DOK and TK SŽDC s.o.
PS 20-02-52 Pod Vyšehradem Bridges, modifications of existing Connecting cables
PS 20-02-53 Pod Vyšehradem Bridges, modifications of existing ZOK ČD-Telematika a.s.

Other utility structures – Power supply
SO 20-54-11 Pod Vyšehradem Bridges, modification of public lighting on pedestrian footbridges
SO 20-54-12 Pod Vyšehradem Bridges, modification of power supply for the fairway signalling

Traction and power equipment
SO 20-71-01 Pod Vyšehradem Bridges, TCL modifications
SO 20-71-02 Pod Vyšehradem Bridges, 6kV overhead cable

Distribution of HV, LV, lighting and remote control of sectional disconnecting switches
SO 20-76-01 Pod Vyšehradem Bridges, Vyšehrad – Praha Smíchov, 6kV distribution line

Connecting metal structures to rail
SO 20-77-01 Pod Vyšehradem Bridges, connecting conductive structures to rail

Link to adjacent parts of construction 1 and 3:
During the construction process, it is necessary to monitor and coordinate the links especially to the overhead lines, where due to various procedures there is a need to leave the masts until the implementation of the related construction.

9. Requests for Supplementary Documents

The following additions are required as part of the processing of the next stage of the project documentation:

SO 20-20-04

- dia boreholes with core to assess the condition of masonry and spacing – 4 x 4 boreholes = 16 boreholes (2 x horizontal 3.0m long into the shank and 2 x inclined into the base, 5.0m long)
- inspection of the condition of the stonework under the terrain (to a depth of 1.0 m) – 4 x excavated probe 0.8 x 1.0 1.0m deep
- 2 x IG boreholes in the bottom at the location of piers P1 and P2,
- 2 x IG boreholes at the location of OP1 and OP2 abutments,

SO 20-20-05

- 4 x IG boreholes in the bottom at the location of piers P1 and P2 (will also serve for IS relocation under the Vltava river bed), – 2 x IG boreholes at the location of piers OP1 and OP2,
- technical inspection – survey of the Vltava riverbed in the area 50m in front of and 50m beyond the bridge (fairway and surrounding area),
- survey of the hidden dimensions of the OP2 abutment

- survey of protective coating (chemical composition + PCB according to SZCZ's requirements)
- measurement of the dimensions of the substructure, including the joints of the masonry in the area of the storage threshold (level of caving)

SO 20-20-05.3

- updating the condition of the bottom around the piers – underwater survey including the positions of the bottom of the original bridge from 1871,

10. Standards and Regulations

Note: The relevant standards and regulations are considered in the version in force at the time of commencement of work on the project documentation.

No. 266/1994 Coll.	Act of the Parliament of the Czech Republic on Railways,
177/1995 Coll.	Decree of the Ministry of Transport, which issues the construction and technical regulations of railways, as amended,
137/1998 Coll.	Decree of the Ministry of Regional Development on General Technical Requirements for Construction, as amended,
No. 66/2015 Coll.	Decree of the Ministry of Transport on Waterways, Navigation In Ports, Joint Crash and Transport of Dangerous Goods
TQC	Technical Qualitative Conditions of State Railway Buildings, 3rd updated edition, 2000, incl. 1/2001, 2/2002, 3/2002, 4/2004, 5/2007, 6/2008
DG SŽDC s. o. 16/2005	Directive of DG SŽDC s. o, Documentation for the Preparation of Constructions on Railway of National and Regional Lines
DG SŽDC s. o. 11/2006	Directive of DG SŽDC s.o., Principles of Modernisation and Optimisation of Selected Railway Network of the Czech Republic,
SŽDC S 3	Railway Superstructure
SŽDC (ČD) S 3/2	Jointless Track
SŽDC S 4	Substructure,
SŽDC S 5	Management of Bridge Structures,
SŽDC S 5/4 (S)	Corrosion Protection of Steel Structures
MP 2015	Methodological Guideline for Determining the Load Capacity of Railway Bridges, 2015
SŽDC SR 5/7 (S)	Protection of Railway Bridge Structures against the Effects of Stray Currents,
SŽDC MVL 102	Transition between Load-bearing Structures. Transition between the Load-bearing Structure and the Abutment. Transition between the Substructure and the Earth Body, 1996,
ČSN EN	A set of standards for the design of bridge structures,

11. Deviations from Regulations and Standards

Exceptions to standards and regulations are addressed within the construction. During the elaboration of the documentation, limit cases were solved in relation to the standard requirements and to the requirements given by the SZCZ regulations, especially in relation to the spatial clearance and railway operation safety.

In the station, the section of the bridge over the Vltava River (SO 20-20-05) **is not suitable for VMP 2.5.** For the movement of authorised persons along the track, there are no recesses in the truss structure 10.0 m x 2.5 m min. depth 0.5 m at a distance of < 10 m.

The proposed reconstruction of the existing bridge structure **cannot ensure** the standard parameters required for the design of new bridge structures.

12. LOAD CAPACITY OVERVIEW – SO 20-20-04

Overview of the load capacity of the bridge parts

Bridge at km 3.545 – Výtoň, LS Praha hl. n. – Praha-Smíchov **load-bearing structure NK1 – NK3 – existing condition**

A. Identification of the Bridge

TS: **LS 0201 Line section 0201 Praha hl. n. (excl.) – Praha-Smíchov 1, 2, 3 (incl.)** SAO: **B1** km: **. 3.545**

B. Identification of the bridge section

Part of the bridge: **load-bearing structure NK1–NK3** Order no.: **1, 3, 5 (NK1, NK2, NK3)** under track no.: **1**
Order no.: **2, 4, 6 (NK1, NK2, NK3)** under track no.: **2**

C. Additional data of the bridge section

Load capacity cat.: **D** Computational Model: **space rod**

Track geometry, considered in the recalculation of the bridge section (in the direction of stationing): *Track No 1 and Track No 2*

		at beg.	middle	at the end	
radius of arc	[m]	**-**	**-**	**-**	**(in direct)**
track elevation	[mm]	**0**	**0**	**0**	
eccentricity of the track axis	[m]	**0**	**0**	**0**	

Description of the defects considered in the recalculation of the bridge section:
- corrosion weakening according to the Evaluation of the corrosion weakening survey of OK elements (2018)
...

Date of finding the technical condition of the bridge: SŽDC s.o. **2017**
by the author of the recalculation **2017**

Note on the bridge section:
- partial reliability coefficients in the load capacity calculation are considered according to MG 2015, Annex F for a residual life of 30 years
...

Order no.	Element	Detail	Stress	ki	type	L_p	Φ_i	L_Φ	γ_Q, LM71	γ_Q, LM71,	see recalculati	Z_{LM71}	$Z_{LM71,E}$	Notes
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	HN1.2-outer main beam – bay 1	POINT K3 – SL	ULS σ_x MY	0.94	S		1.25	18.88	1.30		P3-19	**0.82**		2)
			ULS σ_x MZ	0.06	S		1.25	18.88	1.30					
2	HN2 – main beam – bay 2	POINT K3 – SL	ULS σ_x MY	0.99	S		1.25	18.88	1.30		P3-31	**0.89**		2)
			ULS σ_x MZ	0.01	S		1.25	18.88	1.30					
3	HN3 – main beam – bay 2 (in front of panel point 2)	POINT K3 – SL	ULS σ_x MY	0.99	S		1.25	18.88	1.30		P3-37	**0.86**		2)
			ULS σ_x MZ	0.01	S		1.25	18.88	1.30					
4	HN4 – main beam – bay 3	POINT K3 – SL	ULS σ_x MY	1.00	S		1.25	18.88	1.30		P3-43	**0.85**		2)
			ULS σ_x MZ	0.01	S		1.25	18.88	1.30					
5	HN5 – main beam – bay 4	POINT K3 – SL	ULS σ_x MY	0.99	S		1.25	18.88	1.30		P3-49	**0.87**		2)
			ULS σ_x MZ	0.01	S		1.25	18.88	1.30					
6	L2.1 – Longitudinal truss – bay 2 – section LI	POINT K1 – HL	ULS σ_x N	0.07	S		1.75	5.36	1.30		P3-105	**0.92**		2)
			ULS σ_x MY	0.80	S		1.75	5.36	1.30					
			ULS σ_x MZ	0.13	S		1.75	5.36	1.30					
7	L2.6 – Longitudinal truss – bay 2 – section LI	POINT K4 – SP	ULS σ_x N	-0.06	S		1.75	5.36	1.30		P3-111	**0.93**		2)
			ULS σ_x MY	1.00	S		1.75	5.36	1.30					
			ULS σ_x MZ	0.06	S		1.75	5.36	1.30					

8	L4.1 – Longitudinal truss – bay 4 – section LI	POINT K1 – HL	ULS σ_x N	0.10	S		1.75	5.36	1.30		P3-125	**1.15**		1)
			ULS σ_x MY	0.86	S		1.75	5.36	1.30					
			ULS σ_x MZ	0.04	S		1.75	5.36	1.30					
9	HNO – outer main beam – support	POINT S6 – HP	ULS τ_z VZ	1.00	S		1.25	18.88	1.30		P3-8	**1.41**		1)
10	L0.1 – Longitudinal truss – supporting cross bar – cross-section L1	POINT S6 HP	ULS τ_z VZ	1.00	S		1.75	5.36	1.30		P3-137	**1.33**		1)

Minimum load capacity of the bridge section:	**load-bearing structure NK1–NK3**	Z_{LM71} =	**0.82**

Notes:

1) Z_{LM71} > 1.00, i.e. D4-70km/h is transient
2) Transitional for D4-70km/h

Author of the recalculation: **Ing. Martin Vlasák, SUDOP PRAHA a.s.**
Date: **21/05/2018**, load-bearing capacity determined: **Ing. Jaroslav Vořížek, SUDOP PRAHA, a.s.**

Overview of the load capacity of the bridge parts

Bridge at km 3.545 – Výtoň, LS Praha hl. n. – Praha-Smíchov **bearing structure NK4 – existing condition**

A. Identification of the Bridge

TS: **LS 0201 Line section 0201 Praha hl. n. (excl.) – Praha-Smíchov 1, 2, 3 (incl.)** SAO: **B1** km: **. 3.545**

B. Identification of the bridge section

Part of the bridge: **load-bearing structure NK4** Order no.: **7 (NK4)** under track no.: **1**
Order no.: **8 (NK4)** under track no.: **2**

C. Additional data of the bridge section

Load capacity cat.: **D** Computational Model: **space rod**

Track geometry, considered in the recalculation of the bridge section (in the direction of stationing): *Track No 1 and Track No 2*

		at beg.	middle	at the end	
radius of arc	[m]	-	-	-	**(in direct)**
track elevation	[mm]	**0**	**0**	**0**	
eccentricity of the track axis	[m]	**0**	**0**	**0**	

Description of the defects considered in the recalculation of the bridge section:
- corrosion weakening according to the Evaluation of the corrosion weakening survey of OK elements (2018)
…

Date of finding the technical condition of the bridge: SŽDC s.o. **2017**
by the author of the recalculation **2017**

Note on the bridge section:
- partial reliability coefficients in the load capacity calculation are considered according to MG 2015, Annex F for a residual life of 30 years
…

Order no.	Element	Detail	Stress	ki	type	L_p	Φ_i	L_Φ	γ_Q, LM71	γ_Q, LM71,	see recalculati	Z_{LM71}	$Z_{LM71,E}$	Notes
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	HN4_3 - NK4 – main beam – bay 2	POINT K3 – SL	ULS σ_x MY	0.99	S		1 25	18.88	1.30		P3-79	**1.04**		1)
			ULS σ_x MZ	0.01	S		1.25	18.88	1.30					
2	HN4_3 - NK4 – main beam – bay 2	POINT S6 – HP	ULS σ_x MY	1.00	S		1.25	18.88	1.30		P3-80	**0.95**		2)
			ULS σ_x MZ	0.00	S		1.25	18.88	1.30					
3	L2.1 - Longitudinal truss bay 2 – cross section L1	POINT K1 – HL	ULS σ_x N	0.07	S		1.75	5.36	1.30		P3-105	**0.92**		2)
			ULS σ_x MY	0.80	S		1.75	5.36	1.30					
			ULS σ_x MZ	0.13	S		1.75	5.36	1 30					
4	L2.6 – Longitudinal truss – bay 2 – cross section L6	POINT K4 – SP	ULS σ_x N	-0.06	S		1.75	5.36	1.30		P3-111	**0.93**		2)
			ULS σ_x MY	1.00	S		1.75	5.36	1.30					
			ULS σ_x MZ	0.06	S		1.75	5.36	1.30					
5	L4.1 – Longitudinal truss – bay 4 – cross section L1	POINT K1 – HL	ULS σ_x N	0 10	S		1.75	5.36	1.30		P3-125	**1.15**		1)
			ULS σ_x MY	0.86	S		1.75	5.36	1.30					
			ULS σ_x MZ	0.04	S		1.75	5.36	1.30					
6	HN0 – outer main beam – support	POINT K4 – SP	ULS τ_z VZ	1.00	S		1.25	18.88	1.30		P3-8	**1.41**		1)
7	L0.1 – Longitudinal truss – supporting cross bar – cross-section L1	POINT K4 – SP	ULS τ_z VZ	1.00	S		1.75	5.36	1.30		P3-137	**1.33**		1)

Minimum load capacity of the bridge section:	**load-bearing structure NK4**	Z_{LM71} =	**0.92**

Notes:

1) Z_{LM71} > 1.00, i.e. D4-70km/h is transient
2) Transitional for D4-70km/h

Author of the recalculation: **Ing. Martin Vlasák, SUDOP PRAHA a.s.**
Date: **21/05/2018**, load-bearing capacity determined: **Ing. Jaroslav Voříšek, SUDOP PRAHA, a.s.**

13. LOAD CAPACITY OVERVIEW – SAT 20-20-05

Overview of the load capacity of the bridge parts

Bridge at km 3.706 – Pod Vyšehradem, LS Praha hl. n. – Praha-Smíchov **load-bearing structure NK1 – NK3 – existing condition**

A. Identification of the Bridge

TS: **LS 0201 Line section 0201 Praha hl. n. (excl.) – Praha-Smíchov 1, 2, 3 (incl.)** SAO: **B1** km: **. 3.706**

B. Identification of the bridge section

Part of the bridge: **load-bearing structure NK1–NK3** Order no.: **1, 2, 3** under track no.: **1 and 2**

C. Additional data of the bridge section

Load capacity cat.: **D** Computational Model: **space rod**

Track geometry, considered in the recalculation of the bridge section (in the direction of stationing): *(Track No. 1 / Track No. 2)*

		at beg.	middle	at the end	
radius of arc	[m]	**- / -**	**- / -**	**- / -**	**(in direct)**
track elevation	[mm]	**0/0**	**0/0**	**0/0**	
eccentricity of the track axis	[m]	**-1.90/+1.90**	**-1.90/+1.90**	**-1.90/+1.90**	

Description of the defects considered in the recalculation of the bridge section:

- corrosion weakening according to the Evaluation of the corrosion weakening survey of OK elements (2018)
- upper chord of the L4.2 longitudinal truss – crack weakening considered by the absence of the upper chord, crack according to local investigation (2018/04)

...

Date of finding the technical condition of the bridge: SŽDC s.o. **2017**; by the author of the recalculation **2017**

Note on the bridge section:

- partial reliability coefficients in the load capacity calculation are considered according to MG 2015, Annex F for a residual life of 30 years

...

Order no.	Element	Detail	Stress	ki	type	L_p	Φ_i	L_Φ	γ_Q, LM71	γ_Q, LM71,	see recalculati	Z_{LM71}	$Z_{LM71,E}$	Notes
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
1	NK – HL. BEAM UPPER CHORD O1	POINT K4 – SP	ULS σx N – track 1	0.24	S		1.00	72.00	1.30		P3-10	**1.44**		1)
			ULS σx MY – track 1	0.06	S		1.00	72.00	1.30					
			ULS σx MZ – track 1	0.02	S		1.00	72.00	1.30					
			ULS σx N – track 2	0.54	S		1.00	72.00	1.30					
			ULS σx MY – track 2	0.12	S		1.00	72.00	1.30					
			ULS σx MZ – track 2	0.02	S		1.00	72.00	1.30					
2	NK – HL. BEAM UPPER CHORD U3	POINT K4 – SP	ULS σx N – track 1	0.26	S		1.00	72.00	1.30		P3-51	**0.87**		2)
			ULS σx MY – track 1	0.07	S		1.00	72.00	1.30					
			ULS σx MZ – track 1	0.07	S		1.00	72.00	1.30					
			ULS σx N – track 2	0.39	S		1.00	72.00	1.30					
			ULS σx MY – track 2	0.14	S		1.00	72.00	1.30					
			ULS σx MZ – track 2	0.07	S		1.00	72.00	1.30					
3	NK MAIN BEAM LACING D2	POINT K4 – SP	ULS σx N – track 1	0.22	S		1.00	72.00	1.30		P3-87	0.98		2)
			ULS σx MY – track 1	0.04	S		1.00	72.00	1.30					
			ULS σx MZ – track 1	0.04	S		1.00	72.00	1.30					
			ULS σx	0.51	S		1.00	72.00	1.30					

			N – track 2											
			ULS σ_x MY – track 2	0.10	S		1.00	72.00	1.30					
			ULS σ_x MZ – track 2	0.09	S		1.00	72.00	1.30					
4	NK – HL. BEAM LACING D7 D7.2	POINT K3 – SL	ULS σ_x N – track 1	0.24	S		1.00	72.00	1.30		P3-123	1.26		1)
			ULS σ_x MY – track 1	0.18	S		1.00	72.00	1.30					
			ULS σ_x MZ – track 1	0.01	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.57	S		1.00	72.00	1.30					
			ULS σ_x MY – track 2	-0.01	S		1.00	72.00	1.30					
			ULS σ_x MZ – track 2	0.02	S		1.00	72.00	1.30					

1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
5	NK – MAIN BEAM PERPENDICULAR V7 V7.2	POINT K2 – HP	ULS σ_x N – track 1	0.09	S		1.00	72.00	1.30		P3-243	**0.96**		2)
			ULS σ_x MY – track 1	0.45	S		1.00	72.00	1.30					
			ULS σ_x MZ – track 1	0.01	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.23	S		1.00	72.00	1.30					
			ULS σ_x MY – track 2	0.22	S		1.00	72.00	1.30					
			ULS σ_x MZ – track 2	0.01	S		1.00	72.00	1.30					
6	NK – MAIN BEAM PERPENDICULAR V8 V8.2	POINT K2 – HP	ULS σ_x N – track 1	0.08	S		1.00	72.00	1.30		P3-255	**0.77**		3)
			ULS σ_x MY – track 1	0.46	S		1.00	72.00	1.30					
			ULS σ_x MZ – track 1	0.00	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.23	S		1.00	72.00	1.30					
			ULS σ_x MY – track 2	0.23	S		1.00	72.00	1.30					
			ULS σ_x MZ – track 2	0.00	S		1.00	72.00	1.30					
7	NK – RAIL-TRACK LONGITUDINAL TRUSS L2.E- EDGE L2.1	POINT K1 – HL	ULS σ_x N – track 1	0.19	S		1.61	7.00	1.30		P3-285	0.86		2)
			ULS σ_x MY – track 1	0.08	S		1.61	7.00	1.30					
			ULS σ_x MZ – track 1	0.11	S		1.61	7.00	1.30					
			ULS σ_x N – track 2	0.24	S		1.61	7.00	1.30					
			ULS σ_x MY – track 2	0.22	S		1.61	7.00	1.30					
			ULS σ_x MZ – track 2	0.17	S		1.61	7.00	1.30					
8	NK – RAIL-TRACK LONGITUDINAL TRUSS L2.E – CENTRE L2.2	POINT K4 – SP	ULS σ_x N – track 1	0.07	S		1.61	7.00	1.30		P3-292	0.74		3)
			ULS σ_x MY – track 1	0.08	S		1.61	7.00	1.30					
			ULS σ_x MZ – track 1	0.05	S		1.61	7.00	1.30					
			ULS σ_x N – track 2	0.08	S		1.61	7.00	1.30					
			ULS σ_x MY – track 2	0.64	S		1.61	7.00	1.30					
			ULS σ_x MZ – track 2	0.08	S		1.61	7.00	1.30					
9	NK – BRIDGE-DECK LONGITUDINAL TRUSSL4.E - CENTREL4.2	POINT K4 – SP	ULS σ_x N – track 1	0.05	S		1.56	7.80	1.30		P3-334	0.72		2)
			ULS σ_x MY – track 1	0.03	S		1.56	7.80	1.30					
			ULS σ_x MZ – track 1	0.01	S		1.56	7.80	1.30					
			ULS σ_x N – track 2	0.08	S		1.56	7.80	1.30					
			ULS σ_x MY – track 2	0.82	S		1.56	7.80	1.30					
			ULS σ_x MZ – track 2	0.01	S		1.56	7.80	1.30					
10	NK – RAIL-TRACK SUPPORTING CROSS BAR PO – OUTER LONGITUDINAL TRUSS PQ.1	POINT K3 – SL	ULS σ_x N – track 1	-0.01	S		2.00	3.60	1.30		P3-340	0.69		3)
			ULS σ_x MY – track 1	0.26	S		2.00	3.60	1.30					
			ULS σ_x MZ – track 1	0.23	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	-0.03	S		2.00	3.60	1.30					
			ULS σ_x MY – track 2	0.57	S		2.00	3.60	1.30					

			ULS σ_x MZ – track 2	-0.02	S		1.00	72.00	1.30					
11	NK – RAIL-TRACK SUPPORTING CROSS BAR PI – OUTER LONGITUDINAL TRUSS P1.1	POINT K2 – HP	ULS σ_x N – track 1	0.00	S		1.27	17.60	1.30		P3-359	0.91		2)
			ULS σ_x MY – track 1	0.19	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 1	0.01	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.00	S		1.27	17.60	1.30					
			ULS σ_x MY – track 2	0.80	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 2	0.00	S		1.00	72.00	1.30					
12	NK – RAIL-TRACK SUPPORTING CROSS BAR P2 – OUTER LONGITUDINAL TRUSS P2.1	POINT K2 – HP	ULS σ_x N – track 1	0.02	S		1.27	17.60	1.30		P3-374	0.72		3)
			ULS σ_x MY – track 1	0.15	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 1	0.19	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.01	S		1.27	17.60	1.30					
			ULS σ_x MY – track 2	0.57	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 2	0.05	S		1.00	72.00	1.30					

Order no.	Element	Detail	Stress	ki	type	L_p	Φ_i	L_Φ	γ_Q, LM71	γ_Q, LM71,	see recalculati	Z_{LM71}	$Z_{LM71,E}$	Notes
1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
13	NK – RAIL-TRACK SUPPORTING CROSS BAR P3 - BASIC PART P3.1	POINT K3 – SL	ULS σ_x N – track 1	0.01	S		1.27	17.60	1.30		P3-394	**0.62**		4)
			ULS σ_x MY – track 1	0.16	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 1	0.15	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.00	S		1.27	17.60	1.30					
			ULS σ_x MY – track 2	0.63	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 2	0.05	S		1.00	72.00	1.30					
14	NK – RAIL-TRACK SUPPORTING CROSS BAR P4 – BASIC PART P4.1	POINT K2 – HP	ULS σ_x N – track 1	0.03	S		1.27	17.60	1.30		P3-413	**0.61**		3)
			ULS σ_x MY – track 1	0.14	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 1	0.04	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.04	S		1.27	17.60	1.30					
			ULS σ_x MY – track 2	0.60	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 2	0.15	S		1.00	72.00	1.30					
15	NK – RAIL-TRACK SUPPORTING CROSS BAR P5 – BASIC PART P5.1	POINT K3 – SL	ULS σ_x N – track 1	0.00	S		1.27	17.60	1.30		P3-434	0.64		2)
			ULS σ_x MY – track 1	0.19	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 1	0.03	S		1.00	72.00	1.30					
			ULS σ_x N – track 2	0.00	S		1.27	17.60	1.30					
			ULS σ_x MY – track 2	0.68	S		1.27	17.60	1.30					
			ULS σ_x MZ – track 2	0.10	S		1.00	72.00	1.30					
16	NK – BRIDGE DECK LONGITUDINAL L2.E – EDGE L2.1	POINT S5 – HL	ULS τ_z VZ – track 1	0.06	S		1.61	7.00	1.30		P3-287	**1.34**		1)
			ULS τ_z VZ – track 2	0.94	S		1.61	7.00	1.30					
17	NK – RAIL-TRACK SUPPORTING CROSS BAR PO – OUTER LONGITUDINAL TRUSS P0.1	POINT S6 – HP	ULS τ_z VZ – track 1	0.27	S		2.00	3.60	1.30		P3-341	**0.92**		2)
			ULS τ_z VZ – track 2	0.73	S		2.00	3.60	1.30					
18	NK – RAIL-TRACK SUPPORTING CROSS BAR P4 – BASIC PART P4.1	POINT S5 – MAIN	ULS τ_z VZ – track 1	0.29	S		1.27	17.60	1.30		P3-415	**1.04**		1)
			ULS τ_z VZ – track 2	0.71	S		1.27	17.60	1.30					

Minimum load capacity of the bridge section:	**load-bearing structure NK1–NK3**	Z_{LM71} = **0.61**

Notes:

1) Z_{LM71} > 1.00, i.e. C3-60km/h-30 years is transient
2) Transitional for C3-60km/h-30 years

3) Transitional for C3-60km/h-5 years
4) Transitional for C3-40km/h-5 years

Author of the recalculation:	**Ing. Martin Vlasák, SUDOP PRAHA a.s.**
Date: **04/05/2018**, the load capacity determined by:	**Ing. Jaroslav Voříšek, SUDOP PRAHA, a.s.**

14. Bill of Quantities

SO 20-20-04 — Pod Vyšehradem Bridges, railway bridge at km 3.545 – Výtoň

JKPOV, JKSO: *821 21*
SKP, KSD: *45.21.21*
Prices 2017

future owner of assets % share of SO assets	Percentage of the cost of the structure for:			*name of another owner*
	SŽDC, s.o.	***ČD, a.s.***	*other*	
	100			

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	u/measure	number of units	unit price	total price
10: Earthworks				
ADJUSTMENT OF SURFACES WITH COMPARISON OF AREAS UP TO 0.50M DEEP (Vnislavova Street – adjustment of green areas, adjustment of the surface in front of O02)	m2	375.00		
20: Foundations				
Reinforced concrete foundations up to C30/37 (TCL foundations)	m3	8.80		
Micropiles, including boreholes in rock III (reinforcement O01, P03, O02)	m	554.16		
DRILL HOLES FOR ANCHOR, GROUT, MICROPILE ON THE SURFACE CL IV D UP TO 200MM (DRILL HOLES FOR MICROPILES)	m	554.16		
JET GROUTING OF THE SUB-BASE UP TO 600MM DL OF THE DRILL UP TO 14M ON THE SURFACE (JET GROUTING OF THE SUB-BASE O01,P03,O02)	m3	433.93		
BOREHOLES FOR ANCHORING, GROUTING AND MICROPILES ON THE SURFACE CL. III D UP TO 150MM (boreholes for jet grouting of sub-base O01, P03, O02)	m	1 680.00		
Grouting of masonry, including boreholes (masonry of abutments and piers)	m3	1 536.06		
Remediation of masonry surface by pressure water blasting, silica sand and deep grouting (masonry abutments and piers)	m2	454.26		
30: Vertical structures				
Stone masonry cladding with grouting (reconstruction of TCD P01, P02, O02)	m3	31.86		
Abutments, wings, retaining and frame walls reinforced concrete, monolithic, solid (reconstruction of TCD O01, P03, O02)	m3	67.50		
40: Horizontal structures				
BRIDGE BEARINGS OTHER FOR LOADS UP TO 1,0MN (bay 1+2+3+4 - bearing repairs)	piece	32.00		
50: Roads				
Paved areas (assembly only, etc.), construction and removal (areas in Vnislavova and Svobodova Streets)	m2	2 004.00		
REMOVAL OF THE COVER OF PAVED AREAS WITH ASPHALT BINDER	m2	1 543.50		

(areas in Vnislavova and Svobodova Streets and under bridges)		
ASPHALT CONCRETE FOR CIRCULAR ROWS MODIFIK ACO 11 TH. 50MM (Vnislavova street - reconstruction of pavements)	m2	1 543.50
ASPHALT CONCRETE FOR BED LAYERS ACL 16 50MM THICK (Vnislavova Street – reconstruction of pavements)	m2	1 543.50
REPAVING OF THE COVER MADE OF LARGE CUBES (Vnislavova Street – reconstruction of the roadway)	m2	825.00

14.1 Bill of Quantities SO 20-20-04

SO 20-20-04 — **Pod Vyšehradem Bridges, railway bridge at km 3.545 – Výtoň**

JKPOV, JKSO:	*821 21*	**Prices 2017**
SKP, KSD:	*45.21.21*	

future owner of assets % share of SO assets	Percentage of the cost of the structure for:			*name of another owner*
	SŽDC, s.o.	***ČD, a.s.***	*other*	
	100			

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	**u/measure**	**number of units**	**unit price**	**total price**
60: Surface modifications				
711: Waterproofing				
Watertight insulation system for supporting structure / substructure against free flowing water, with soft protection (reverse O01)	m2	10.4		
90: Other constructions and works				
Steel railing with vertical infill (replica of existing railing) (delivery, galvanising, painting, fitting, anchoring)	m	168.69		
Load test of the load-bearing structure, static, including test load (bay 1, bay 2, bay 3, bay 4)	piece	8.00		
Load test of the load-bearing structure, static, including test load (bay 1, bay 2, bay 3, bay 4)	piece	8.00		
TEMPORARY STRUCT. FROM STEEL. BEAMS INCL. REMOVAL (Vnislavova Street – assembly area)	t	650.16		
Dismantling of the steel load-bearing structure (lifting from bearings and moving SOK)	t	344.0		
Railway bridge load-bearing structures steel plate girder (delivery, assembly, fitting, corrosion protection by painting and metallisation) (reconstruction of SOK)	t	344.0		
96: Demolition and dismantling				
Demolition of stone structures, including loading and stacking (reconstruction of ÚP piers and abutments)	m3	141.78		
Dismantling of steel railings, including loading and folding	t	5.06		
Horizontal relocation of debris and demolished materials for every 1 km	km	2 684.33		

14.2 Bill of Quantities SO 20-20-05

SO 20-20-05 **Bridges Pod Vyšehradem, railway bridge at km 3.706 – Pod Vyšehradem**

JKPOV, JKSO: *821 21* **Prices 2017**
SKP, KSD: *45.21.21*

	Percentage of the cost of the structure for:			*name of another owner*
future owner of assets	***SŽDC, s.o.***	***ČD, a.s.***	*other*	
% share of SO assets	**100**			

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	u/measure	number of units	unit price	total price
Tests and revisions				
Charges for waste disposal				
10: Earthworks				
Backfilling of pits and trenches with compacted material, including loading and composition (backfilling of ditches)	m3	15 315.79		
Excavation of buried and unburied pits in the rock of class II, incl. loading and composition (removal of backfill)	m3	15 315.79		
Horizontal relocation of the excavation of cl. II for every 1 km (24km)	m3	735,158		
Excavation of buried and unburied pits in the rock of cl. I, incl. loading and folding (reverse of O02)	m3	275.40		
Horizontal relocation of the excavation of cl. I for every 1 km (24km)	m3	6,610		
Slope preparation, incl. humus removal, incl. loading and folding	m2	450.00		
SURFACE ADJUSTMENT BY LEVELLING THE AREA UP TO 0.50M (Vnislavova Street – green areas, surface treatment in front of O02)	m2	988.00		
WATER PUMPING UP TO 4,000 L/MIN (construction and dismantling of temporary trestles)	h	3 360.00		
Reinforcement of steel barriers with removal of the trench depth up to 10 m (reinforcement on O02)	m2	82.62		
20: Foundations				
Drainage of the bridge abutment – drainage plastic pipe HDPE DN 200,	m	15.00		
including casing and aggregate backfill (O02 – back drainage)	m	784.32		
Micropiles, including boreholes in rock III (reinforcement of O01 and piers)				
DRILL HOLES FOR ANCHOR, GROUT, MICROPILE ON THE SURFACE CL IV D UP	m	784.32		
TO 200MM (DRILL HOLES FOR MICROPILES)	m2	7 492.80		
Shield walls, temporary (at piers, at trestles, in front of O02)				
Jet grouting of the sub-base up to 600MM DL of the drill up to 14M on the surface (jet grouting of the sub-base O01,O02)	m3	622.83		
BOREHOLES FOR ANCHORING, GROUTING AND MICROPILES ON THE	m	1 670.00		
SURFACE CL. III D UP TO 150MM (boreholes for jet grouting of sub-base O01, O02)	m3	1 172.52		
Grouting of masonry, including boreholes (masonry of abutments and piers)	m2	966.35		
Remediation of masonry surface by pressure water blasting, silica sand and deep grouting (masonry abutments and piers)	m3	5.51		
Wooden temporary full support boundaries, establishment and removal (temporary reinforcement of the track bed on O02)	m	135.00		
Rope anchors, temporary, including boreholes in rock III (reinforcement on O02 - anchoring)				
	m3	3.14		
30: Vertical structures	m3	72.46		
Reinforced concrete bridge ledges, C30/37 (ledges on O02)				
Stone masonry cladding with jointing (reconstruction of ÚP piers and supports)	m3	159.94		
Abutments, wings, retaining and frame walls reinforced concrete, monolithic, solid (reconstruction of ÚP O01, O02, O02)				

SO 20-20-05 Bridges Pod Vyšehradem, railway bridge at km 3.706 – Pod Vyšehradem

JKPOV, JKSO: *821 21* **Prices 2017**
SKP, KSD: *45.21.21*

future owner of assets % share of SO assets	Percentage of the cost of the structure for:			*name of another owner*
	SŽDC, s.o.	***ČD, a.s.***	*other*	
	100			

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	u/measure	number of units	unit price	total price
Tests and revisions				
Charges for waste disposal				
40: Horizontal structures				
BRIDGE BEARINGS OTHER FOR LOADS OVER 5.0MN (bay 1+2+3 – bearing repair)	piece	12.00		
Backfill behind abutments compacted, with purchased material (according to SZCZ S4) (reverse O02)	m3	236.16		
Fractured stone plane (reverse O02)	m3	8.64		
Paving with quarry stone into the underlying concrete (wings O02)	m2	50.00		
50: Roads				
Paved areas (assembly only, etc.), establishment and removal (areas in Vnislavova Street, under the trestles, at O02)	m2	5 064.00		
REMOVING THE COVER OF ASPHALT BINDER (Vnislavova Street – reconstruction of pavements)	m2	500.00		
ASPHALT CONCRETE FOR CIRCULAR ROWS MODIFIK ACO 11 TH. 50MM (Vnislavova street - reconstruction of pavements)	m2	500.00		
ASPHALT CONCRETE FOR BED LAYERS ACL 16 TH. 50MM (Vnislavova street – reconstruction of pavements)	m2	500.00		
REPAVING OF THE COVER FROM LARGE CUBES (Vnislavova Street – reconstruction of the roadway)	m2	1 500.00		
60: Surface modifications				
Reprofiling with remedial mortar up to 100mm thick (reverse O02)	m2	213.0		
711: Waterproofing				
Watertight insulation system for supporting structure / substructure against free flowing water, with soft protection (reverse O02)	m2	205.2		

SO 20-20-05

Bridges Pod Vyšehradem, railway bridge at km 3.706 – Pod Vyšehradem

JKPOV, JKSO: *821 21* **Prices 2017**
SKP, KSD: *45.21.21*

future owner of assets % share of SO assets	Percentage of the cost of the structure for:			*name of another owner*
	SŽDC, s.o.	***ČD, a.s.***	*other*	
	100			

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	u/measure	number of units	unit price	total price
Tests and revisions				
Charges for waste disposal				
90: Other constructions and works				
Steel railing with vertical infill (replica of existing railing) (delivery, galvanising, painting, fitting, anchoring)	m	437.28		
Load test of the structure, static, including test load (bay 1, bay 2, bay 3)	piece	3.00		
Load test of the structure, dynamic, including test load (bay 1, bay 2, bay 3)	piece	3.00		
Provisional support PIŽMO height up to 12 m, assembly, rental, dismantling (temporary trestles)	t	2 348.71		
TEMPORARY STRUCT. FROM STEEL. BEAMS INCL. REMOVAL (Vnislavova Street – assembly and extension platform for ŽM16)	t	650.16		
TEMPORARY STRUCT. FROM STEEL. BEAMS INCL. REMOVAL (SOK reconstruction platform)	t	1 515.77		
Provisional bridge ŽM16 (transport, assembly, rental, removal, 3x cross-insertion, 2x installation, back-removal, dismantling)	t	1 017.00		
Dismantling of the steel load-bearing structure (lift from bearings and 2x transverse movement of SOK)	t	1 779.0		
Steel truss bearing structures of railway bridges (delivery, assembly, fitting, corrosion protection by painting and metallisation) (reconstruction of SOK)	t	1 666.8		
96: Demolition and dismantling				
Demolition of stone structures, including loading and stacking (reconstruction of ÚP piers and abutments)	m3	322.91		
Dismantling of steel railings, including loading and folding	t	13.12		
Horizontal relocation of debris and demolished materials for every 1 km	km	6 115.99		
TOTAL				

14.3 Bill of Quantities SO 20-20-05.1

SO 20-20-05.1

Pod Vyšehradem Bridges, railway bridge at km 3.706 – footbridges

JKPOV, JKSO: *821 21*
SKP, KSD: *45.21.21*

Prices 2017

	Percentage of the cost of the structure for:			*name of another owner*
	SŽDC, s.o.	***ČD, a.s.***	*other*	
future owner of assets % share of SO assets			**100**	*Capital City of Prague / TSK*

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	u/measure	number of units	unit price	total price
Tests and revisions				
Charges for waste disposal				
10: Earthworks				
Backfilling of pits and trenches with compacted material, including loading and composition (O02 – pavement surface elevation)	m3	11.30		
20: Foundations				
Foundations of plain concrete up to C25/30 (O01 – steel staircase foundations)	m3	2.40		
Rehabilitation of masonry surface by blasting with pressure water, silica sand and deep grouting (O02 – reconstruction of the wall)	m2	6.98		
30: Vertical structures				
Stone masonry cladding with grouting (O02 – reconstruction of the wall)	m3	11.60		
Reinforced concrete bridge ledges, C30/37 (O02 – outer edge of pavement)	m3	3.93		
40: Horizontal structures				
RAIL-TRACKS AND PERMANENT WOOD FLOORS (wooden bridge deck of footbridges and staircases)	m3	33.85		
STEEL STAIRWAY CONSTRUCTION S 235 (delivery, assembly, CPC, anchoring) (O01 – steel staircase – renovation)	t	5.2		
Paving with quarry stone into the underlying concrete (O02 - along the walls under the pavement)	m2	26.60		
50: Roads				
REMOVING THE COVER OF ASPHALT-FLOORED AREAS (O02 – pavement on the abutment, O01 – under the stairs)	m2	120.88		
ASPHALT CONCRETE FOR MODIFIC ACO 11 TH. 50MM (O02 – pavement on abutment, O01 – under stairs)	m2	120.88		
ASPHALT CONCRETE FOR BED LAYERS ACL 16 TH. 50MM (O02 – pavement on abutment, O01 – under stairs)	m2	120.88		
60: Surface modifications				
711: Waterproofing				
90: Other constructions and works				
Steel railing with vertical infill (delivery, galvanising, painting, fitting, anchoring) (replica of existing railing)	m	500.08		
Steel truss bearing structures of railway bridges (delivery, assembly, fitting, corrosion protection by painting and metallisation) (reconstruction of SOK – footbridges)	t	112.2		
96: Demolition and dismantling				
Dismantling of steel railings, including loading and folding	t	12.50		
Demolition of stone structures, including loading and stacking (O02 – reconstruction of the wall)	m3	8.53		

14.4 Bill of Quantities SO 20-20-05.2

SO 20-20-05.2 — **Bridges Pod Vyšehradem, railway bridge at km 3.706 – day beacons**

JKPOV, JKSO:	*821 21*	**Prices 2017**
SKP, KSD:	*45.21.21*	

	Percentage of the cost of the structure for:			*name of another owner*
future owner of assets	***SŽDC, s.o.***	***ČD, a.s.***	*other*	
% share of SO assets			**100**	*Povodí Vltavy s.p.*

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	u/measure	number of units	unit price	total price
Tests and revisions				
Charges for waste disposal				
existing boards up to 2x2m on the structure – dismantling	pc	6.00		
new signal board up to 2x2m fixed on the structure	pc	6.00		
(delivery)	pc	6.00		
new signal board up to 2x2m – mounting on the structure	pc	4.00		
Radar reflector – bar	pc	12.00		
temporary signal board up to 2x2m fixed on the structure				
temporary signal board up to 2x2m – assembly	pc	22.00		
(3 times on ŽM16, 1 time on SOK)				
temporary signal board up to 2x2m – dismantling	pc	22.00		
(3 times on ŽM16, 1 time on SOK)	pc	6.00		
Provisional radar reflector – bar				

14.5 Bill of Quantities SO 20-20-05.3

SO 20-20-05.3

Pod Vyšehradem Bridges, railway bridge at km 3.706 – securing the bottom at the piers

JKPOV, JKSO: *821 21* **Prices 2017**
SKP, KSD: *45.21.21*

	Percentage of the cost of the structure for:			*name of another owner*
future owner of assets % share of SO assets	***SŽDC, s.o.***	***ČD, a.s.***	*other*	
	100			

The cost of acquiring an operating set, a structure: **in CZK thousand**

Item	**u/measure**	**number of units**	**unit price**	**total price**
Tests and revisions				
Charges for waste disposal				
10: Earthworks				
Excavation of buried and unburied pits in the rock of class II, incl. loading and folding (bottom adjustment at piers)	m3	321.77		
Horizontal relocation of the excavation of cl. II for every 1 km (24km)	m3	7 722.43		
Backfilling of pits and grooves with compacted, dredged material,	m3	529.75		
incl. loading and composition (bottom adjustment at piers, heavy stone backfill outside)	h	5 760.00		
WATER PUMPING UP TO 4000 L/MIN (at piers)				
	m	672.0		
20: Foundations				
Micropiles, incl. boreholes in rock III (securing the bottom at the piers)	m	595.2		
DRILL HOLES FOR ANCHOR, GROUT, MICROPILE ON THE SURFACE CL IV D UP TO 200MM (DRILL HOLES FOR MICROPILES)	m3	141.2		
JET GROUTING OF COLUMN UP TO 800MM BOREHOLE TO 6M ON	m	280.8		
THE SURFACE (securing the bottom at the piers)	m3	804.42		
BOREHOLES FOR ANCHORING, GROUTING AND MICROPILES ON	m2	2 529.72		
THE SURFACE CL. VI UP TO 300MM (boreholes for jet grouting)	m3	667.52		
Reinforced concrete foundations up to C30/37 (securing the bottom of the piers)				
Sheet pile walls, temporary (for piers to secure the bottom)	m3	282.94		
Grouting of masonry, incl. boreholes (grouting of the pier foundation)	m3	643.54		
30: Vertical structures	m2	697.16		
Envelope of the pier foundation made of concrete reinf. kari mesh				
Lightweight concrete infill (above the base slab around the piers)				
40: Horizontal structures				
Paving with quarry stone into the underlying concrete				
50: Roads				
60: Surface modifications				
711: Waterproofing				
90: Other constructions and works				
96: Demolition and dismantling				