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Expert report no. 2000 J 190	**Date of report** August 2020	**KI Department** Mechanics Unit tel. no. +420 224 353 512

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Expert report:

V009 LIBEŇ BRIDGE, PRAGUE 7, X-656 U LODĚNICE, PRAGUE 8

RECALCULATION OF ARCHES nos. 1 and 5 ON BRIDGE V009

ASSESSMENT OF DETAILED SLT OF BRIDGE X-656

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ABSTRACT

This report deals with establishing the expected load capacity of the arches designated 1 and 5 after structural modifications have been conducted as part of the planned renovation of the Libeň bridge set. These modifications consist of eliminating the effect of the bridge's framing bridgeheads having settled, which is adversely affecting the load capacity of the structure.

The report also presents the expert assessment of results obtained from a static load test of the connected flood bridge X-656 (designated as arch 6), which is part of the Libeň bridge set. The report deals with creating a computational model of the structure and validating it for deflection values measured during the static load test and the impact of these modifications on the bridge's load capacity values.

The report was compiled by employees of the Klokner Institute at the Czech Technical University, which is registered on the list of institutions qualified to provide expertise under the provisions of Section 21 (3) of Act No. 36/1967 Coll. and Decree No. 37/1967 Coll., as amended, published in the Official Journal of the Czech Republic, Volume 2004, Part 2, of 14 October 2004, Annex to the Ministry of Justice Communication of 13 July 2004, Ref. No. 228/2003–Zn.

Figure 1: Location of the Libeň Bridge.

Bridgehead of Flood Bridge X-656 is in segment Voctářova – Štorchova

TABLE OF CONTENTS

1 INTRODUCTION

1.1 DESCRIPTION OF WORK

As part of the task, adjustments were made to the computational models of arches 1 and 5 of the main bridge over the Vltava. These modifications consisted of eliminating the adverse effect of the bridge's framing bridgeheads having settled. Using this adjusted model, the value was calculated of the expected load capacity after the above modifications have been carried out as part of the planned renovation of the Libeň bridge complex.

RÁM HOLEŠOVICE KLENBOVÝ MOST

HOLEŠOVICE FRAME ARCH BRIDGE

BRIDGEHEAD (BH) ARCH 1 ARCH 2

ABUTMENT H PILLAR 0 PILLAR 1

DESIGNATION OF FRAME PARTS

ARCH BRIDGE LIBEŇ FRAME

ARCH 3 ARCH 4 ARCH 5 BRIDGEHEAD (BP)

PILLAR 2 PILLAR 3 ABUTMENT L

DESIGNATION OF FRAME PARTS

OZNAČENÍ ČÁSTÍ RÁMU

Figure 2: Diagram of arch section of Libeň Bridge (V009)

Also as part of the task, the results of load testing (hereinafter SLT) produced by the company INSET were studied. In order to assess the impact of these results on the load capacity of the bridge, a new and refined computational model of the structure was produced and validation was conducted for the deflection values measures during the SLT. After the model was validated, a new assessment of the structure's load capacity was conducted. The calculation of the load capacity takes into account the elimination of the negative effect of the adjacent frame bridgeheads settling on the arch section of the bridge, as planned as part of renovations to the Libeň bridge complex.

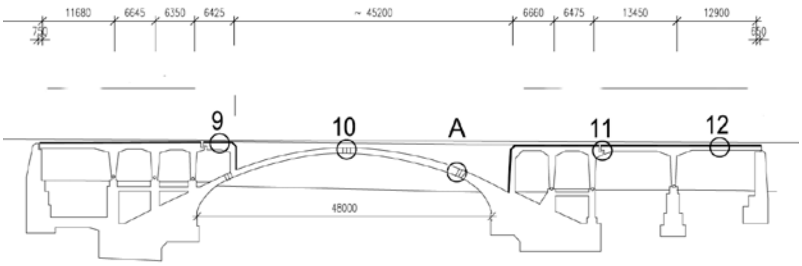


Figure 3: Diagram of flood bridge X-656 (arch 6)

1.1. BACKGROUND DOCUMENTS

[1] Expert Report No. 8301500J316 "LIBEŇ BRIDGE, PRAGUE 7 AND 8, A. NO. 999 984 Analysis and assessment of the current technical condition of the bridge complex and possibilities for repairs or construction of a new bridge based on submitted diagnostic inspections and project documentation", CTU Klokner Institute, December 2015 – Prague

[2] Expert Report No. 8301600J072 "Libeň Bridge, Prague 7 and 8, Flood Bridge X-656 – arch KL 6 and adjacent frame structure", CTU Klokner Institute, June 2016, Prague

[3] Expert Report No. 1700 J 019-01 "Establishing the load capacity of Libeň Bridge V009 and assessing the individual structural elements in terms of feasibility, usability, durability or potential action", CTU Klokner Institute, January 2018 – Prague

[4] Load test of the arch section of Libeň Bridge over the Vltava and the flood bridge – computational groundwork and test program, CTU Klokner Institute, January 2020 – Prague

[5] Supplementary diagnostic study including static and dynamic testing of bridges V009 and X-656 on the street Libeňský most – Final report from bridge load test – arches K2, K3, K4 and Kl6, INSET s.r.o., April 2020 – Prague

[6] Libeň Bridge, Prague 7 and 8, Static load test (SLT) of X656 – Report on static load testing of bridge, INSET s.r.o., August 2020 – Prague

1.3 BRIEF DESCRIPTION OF STRUCTURE

The first section of the Libeň bridge complex assessed is the bridge with the designation V009 and its arches 1 and 5. This is an arch bridge consisting of five arches with backfill. The static effect of the arches is simple – joints are attached at the crown and the abutment. Taking a cross-section, the load-bearing structure consists of four arch segments of an approximate width of 4.85 m. Attached to the outer segments are the front walls, which support the sidewalk cantilevers equipped with a balustrade. Above the pillars, the outer walls are reinforced with ribs.

The transverse configuration of the bridge is symmetrical and is made up of a space 14.5 m wide for tram and road traffic abutted by sidewalk swaths 3.25 m wide.

The arch structures are made of simple concrete, with the exception of the immediate surroundings of the abutment and crown joints, which are slightly reinforced with regard for the occurrence of transversal pressure.

On the Libeň side of the bridge it is adjoined by flood bridge X-656 of a similar construction. The arch of the flood bridge used to cross a branch of the Vltava. The flood bridge begins with a reinforced concrete frame structure of three spans with an outhanging end attached to the next part of the bridge, a three-jointed arch made of simple concrete with a clearance of 48 m (the bridge's largest arch), on the abutment of which the frame is partially founded. The arch ends with another reinforced concrete frame structure of two spans, which is founded on the abutment of the arch; this is immediately followed by a further reinforced concrete structure of two spans. Most of the spaces around the frame structures are currently closed.

Arch	Clearance	Span	Rise	Rise / Span
1	28.0 m	22.0 m	3.43 m	3.43/22.0 = 0.156
2	38.5 m	31.4 m	3.84 m	3.84/31.4 = 0.122
3	42.8 m	34.8 m	3.81 m	3.81/34.8 = 0.109
4	42.8 m	34.8 m	3.81 m	3.81/34.8 = 0.109
5	38.5 m	31.4 m	3.84 m	3.84/31.4 = 0.122
6	48.0 m	39.0 m	3.70 m	3.70/39.0 = 0.095

Table 1: Indicative dimensions of arch part of Libeň Bridge V009 and bridge X-656

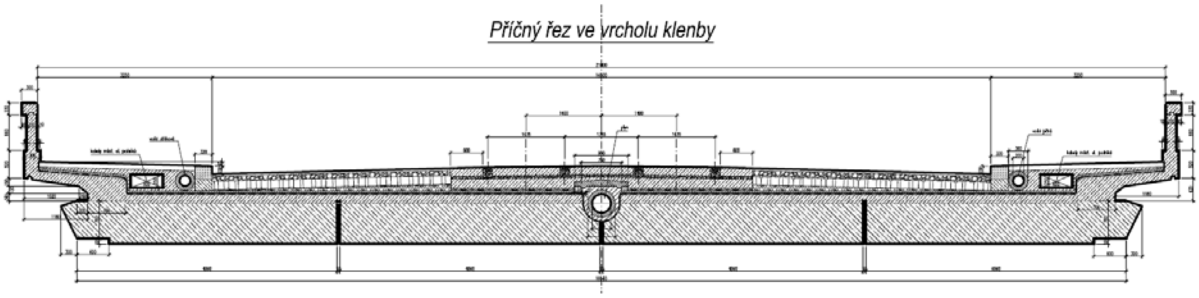


Figure 4: Cross-section at peak of arch

2 COMPUTATIONAL MODELS

Under the task, attention is paid solely to the arched part of the Libeň bridge complex. The renovations will see the frame structures demolished and replaced with new structures with adjusted geometry that respects the necessary modifications in the foundations outside the arched part of the bridge.

In order to assess the load capacity, linear computational models were created in the program MIDAS Civil for arches 1, 5 and 6, respecting to the greatest possible extent the actual geometry of the construction and the static workings thereof. The models were created using a combination of rod, surface and volume elements. In order to assess the expected load capacity of arches 1 and 5, the computational model used in background material [3] was taken and adjusted. To evaluate the load capacity of bridge X-656 and validate it for the results of the last load test [6], a new and detailed computational model was produced.

According to the results of the SLTs conducted, the pillar brackets on which the segments of arch are founded show very low values of deformation, with the time log not recording any discontinuity, jumps or significant changes in the deformation curve for the whole period of the test. Their impact on assessing the load capacity of the arch sections is ignored. The results of the deflections listed in background material [6] provide values after subtracting the bracket deflections.

2.1 Flood Bridge

The flood bridge is the longest and also flattest arch of the Libeň bridge system. Due to its geometry, the decisive cross-sections are those at the peak of the arch, in contrast to the other arches. A frame structure is founded on the existing structure near one of the abutment joints, but due to its distance from the structure's abutment joint and the overall dimension of the bridge's arch portion, it does not affect the global behaviour of the construction. This assumption is based on an assessment of the measured deflections at the quarter points of the span of the Libeň and Holešovice bridge openings, which based on the background material [6] are approximately comparable. From the perspective of a global assessment of the load capacity of the arch portion of the flood bridge, its influence is thus ignored in the calculations. The impact of the foundation of the frame strut was also ignored in the previous computational model made in SCIA Engineer, in which the expected deflections for conducting the SLTs were established. The results in the background material [6] show that despite ignoring this influence, a clear agreement was achieved between the actual action of the structure and the computational model. In terms of local effects however, the foundation of the frame strut adversely influences the area around the abutment joints and we recommend removing it as part of the renovations.

The arch segments of the model consist of a grid made of 1-D bars, which at the base and crown are equipped with joints identical to the static action of the structure.

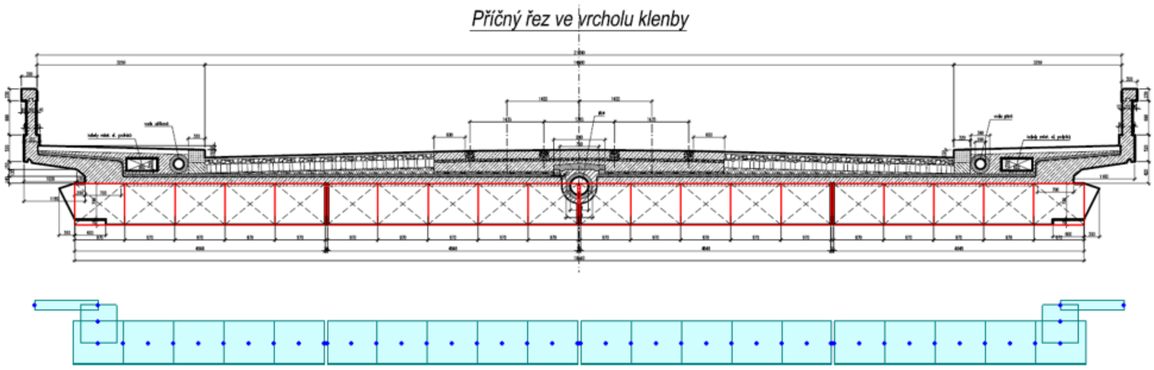


Figure 5: Computational model – arch segments

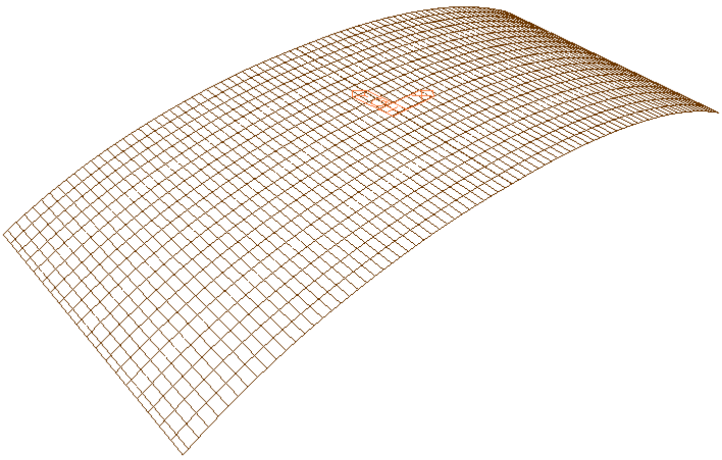


Figure 6: Grid model of arch segments

The outer walls of the arch structure are modelled as planar 2-D plate elements, to which plane elements are attached simulating the sidewalk brackets and wall elements simulating the balustrade of the bridge structure. The elements of the outer wall, brackets and balustrades are placed in the model in order to properly calculate the own weight of the construction, but with their minimum stiffness they do not contribute to the overall stiffness of the model. This set-up was chosen with regard for validation of the computational model, as it best corresponds to the measured results. At the site of the joints both walls and brackets are mutually dilated (not passing continuously through the whole length of the arch).

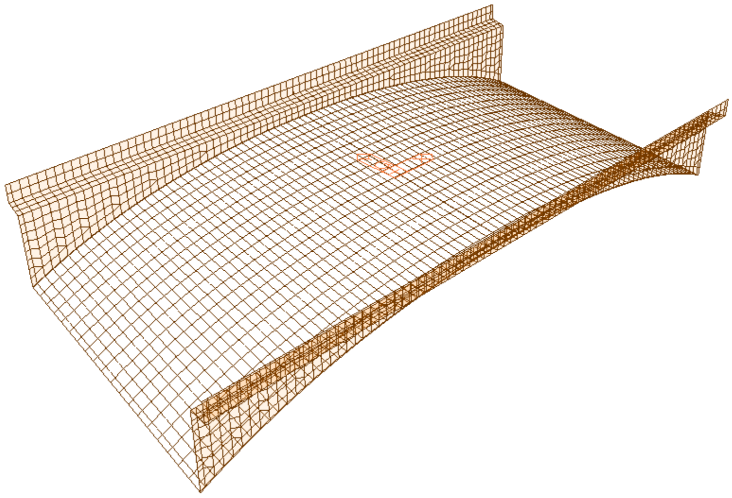


Figure 7: Arch segments and outer walls

The longitudinal and transverse distribution of the load is provided for by the backfill, which in the model is simulated by solid 3-D elements of low stiffness corresponding to the properties of soil. This backfill is supplemented in the upper part by a planar 2-D element simulating the contribution of the road surface composition to distributing the load. Based on the diagnostic study, the plane is made of concrete and its stiffness is set so as to correspond to the material properties of concrete.

Figure 8: View of overall model

2.2 Arches 1 and 5

The computational models of arches 1 and 5 are designed in a similar manner to that of the flood bridge. The difference between the two models is the action of the upper plane, which in the case of bridge V009 over the Vltava takes place continuously across the whole width of the construction and thus acts on the load distribution in a transverse direction.

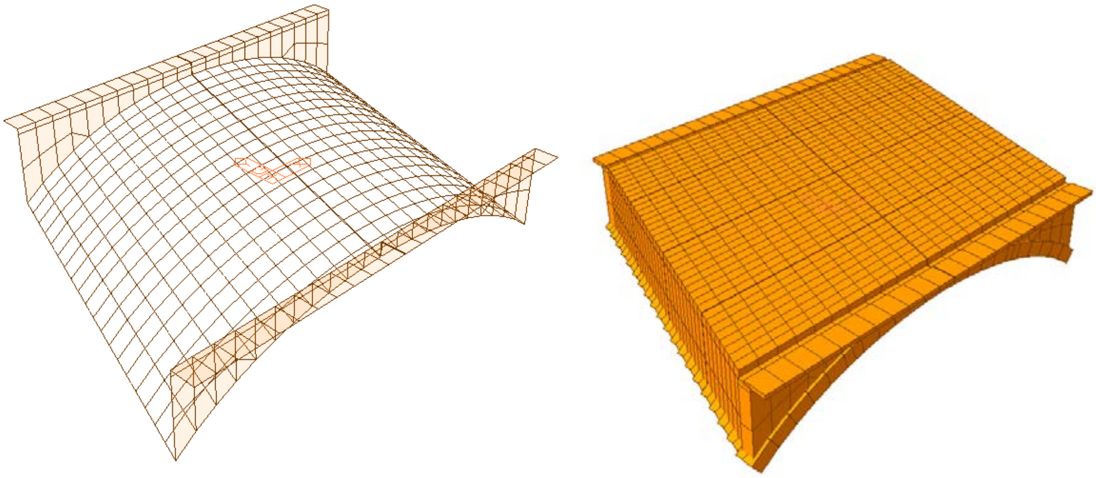


Figure 9: View of overall model of arch 1

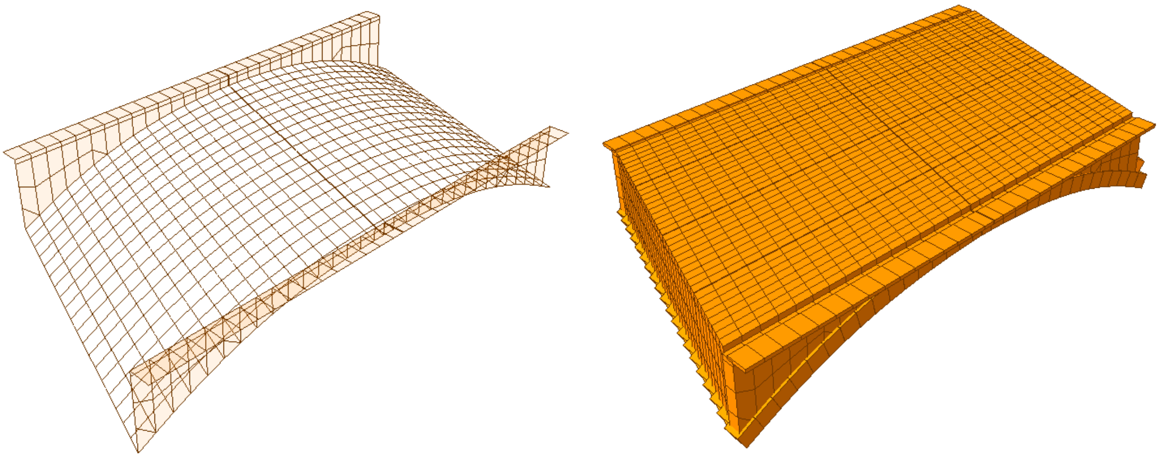


Figure 10: View of overall model of arch 5

2.3 MATERIALS

The basic material characteristics are taken from the diagnostic study of the bridge [2, 3].

Arch 6		C16/20	
Characteristic tensile strength	f_{ck}	16.0	MPa
Reduction factor of concrete compressive strength	α_{cc}	0.9	
Design compressive strength	f_{cd}	= 0.9*16/1.5=9.6	MPa
Poisson's ratio	ν	0.2	
Bulk weight	γ_c	24.3	kN/m^3
Modulus of elasticity (mean value)	E_{cs}	27.3	GPa
Coefficient of thermal expansion	α	$10*10^{-6}$	K^{-1}

Arches 1 and 5		C16/20	
Characteristic tensile strength	f_{ck}	16.0	MPa
Reduction factor of concrete compressive strength	α_{cc}	0.9	
Design compressive strength	f_{cd}	= 0.9*16/1.5=9.6	MPa
Tensile strength, mean value	f_{ctm}	1.9	MPa
Modulus of elasticity for short-term loads	E_{cs}	21.0	GPa
Poisson's ratio	ν	0.2	
Bulk weight	γ_c	22.7	kN/m^3
Coefficient of thermal expansion	α	$10*10^{-6}$	K^{-1}

Plate under the road		C16/20	
Characteristic tensile strength	f_{ck}	16.0	MPa
Reduction factor of concrete compressive strength	α_{cc}	0.9	
Design compressive strength	f_{cd}	= 0.9*16/1.5=9.6	MPa
Tensile strength, mean value	f_{ctm}	1.9	MPa
Modulus of elasticity for short-term loads	E_{cs}	21.0	GPa
Poisson's ratio	ν	0.2	
Bulk weight	γ_c	22.9	kN/m^3

2.4 VALIDATION OF COMPUTATIONAL MODEL

The computational models for arches 1 and 5 were validated in the previous expert report [3] and in this work only geometrical adjustments were made. The validation procedure is described in detail in the expert report [3].

The computational model for the flood bridge has been validated by adjusting the modulus of elasticity for the individual parts of the structure so that the resulting deflections reflect as accurately as possible the actually measured values. This is a very complicated process by which the elasticity modulus values for the basic material of the arch, the outer walls with the brackets, the backfill and the deck under the road are gradually adjusted. After these modifications had been made to the computational model, the impact of the most recent changes on the correlation of the calculated and measured results is further assessed. If this adjustment has a positive influence on agreement between the sets of results, further adjustments to the model are moved on to. The modifications to the modulus of elasticity are in the range of ±20% from the values recommended by structure diagnostic.

Considering the overall span of the arch, the impact of the foundation of the frame strut near one of the abutment joints has a negligible influence on the global action of the structure (see the description of the computational model) and its impact is thus ignored in calculating the load capacity, as with the previous computational model used for establishing the expected deflections for carrying out the SLTs.

Validation of the computational model is conducted so that the resulting deflections correspond to the measured results as per the background material [6]. In order to validate the linear model of the load-bearing structure, the results of the load condition LC-4 were used (taking into account the results of conditions LC-1 through LC-3), which in terms of stabilisation of deformation was assessed as the last load condition where the response of the structure was elastic in background material [6]. For load conditions LC-5 and LC-6 the deformation no longer stabilised and the response of the structure thus manifested as partially plastic.

The deflection values measured in the load test [6] are listed in the following tables.

Point		LC-1 meas.	LC-2 meas.	LC-3 meas.	LC-4 meas.	LC-5 meas.	LC-6 meas.
Outer left segment	Left	-0.69	-0.79	-1.01	-1.38	-1.56	-1.55
	Mid	**-0.82**	**-0.95**	**-1.20**	**-1.65**	**-1.84**	**-1.88**
	Right	-0.94	-1.12	-1.42	-2.00	-2.20	-2.23
Inner left segment	Left	-1.17	-1.47	-1.75	-2.45	-2.58	-2.58
	Mid	**-1.11**	**-1.48**	**-1.78**	**-2.51**	**-2.67**	**-2.68**
	Right	-1.22	-1.63	-1.97	-2.75	-2.91	-2.86
Inner right segment	Left	-1.14	-1.55	-1.90	-2.70	-2.88	-2.84
	Mid	**-1.08**	**-1.50**	**-1.85**	**-2.59**	**-2.70**	**-2.69**
	Right	-1.03	-1.46	-1.84	-2.64	-2.71	-2.74
Outer right segment	Left	-0.75	-1.16	-1.51	-2.13	-2.24	-2.51
	Mid	**-0.49**	**-0.85**	**-1.22**	**-1.73**	**-1.72**	**-1.88**
	Right	-0.60	-0.91	-1.29	-1.78	-1.80	-1.90

Table 2: Deflections at crown of arch

Point		LC-1 meas.	LC-2 meas.	LC-3 meas.	LC-4 meas.	LC-5 meas.	LC-6 meas.
Outer left	Left	-0.31	-0.34	-0.52	-0.72	-0.78	-0.82
	Mid	**-0.32**	**-0.37**	**-0.61**	**-0.85**	**-0.92**	**-1.01**
	Right	-0.33	-0.39	-0.69	-0.97	-1.07	-1.20
Inner left	Left	-0.41	-0.47	-0.81	-1.06	-1.20	-1.23
	Mid	**-0.36**	**-0.44**	**-0.80**	**-1.08**	**-1.24**	**-1.32**
	Right	-0.32	-0.41	-0.78	-1.10	-1.28	-1.41
Inner right	Left	-0.38	-0.46	-0.87	-1.20	-1.35	-1.43
	Mid	**-0.34**	**-0.41**	**-0.80**	**-1.14**	**-1.23**	**-1.34**
	Right	-0.29	-0.36	-0.73	-1.08	-1.11	-1.26
Outer right	Left	-0.34	-0.45	-0.79	-1.06	-1.14	-1.26
	Mid	**-0.28**	**-0.38**	**-0.66**	**-0.90**	**-0.95**	**-1.07**
	Right	-0.21	-0.31	-0.52	-0.74	-0.77	-0.84

Table 3: Holešovice bridge opening – deflections at ¼ span

Point		LC-1 meas.	LC-2 meas.	LC-3 meas.	LC-4 meas.	LC-5 meas.	LC-6 meas.
Outer left	Left	-0.24	-0.24	-0.31	-0.47	-0.65	-0.66
	Mid	**-0.34**	**-0.39**	**-0.48**	**-0.73**	**-0.91**	**-1.00**
	Right	-0.43	-0.53	-0.65	-0.99	-1.18	-1.33
Inner left	Left	-0.44	-0.56	-0.68	-1.07	-1.28	-1.42
	Mid	**-0.47**	**-0.62**	**-0.74**	**-1.19**	**-1.46**	**-1.56**
	Right	-0.50	-0.68	-0.81	-1.30	-1.63	-1.69
Inner right	Left	-0.47	-0.64	-0.74	-1.28	-1.63	-1.68
	Mid	**-0.44**	**-0.62**	**-0.72**	**-1.25**	**-1.52**	**-1.64**
	Right	-0.41	-0.60	-0.71	-1.23	-1.42	-1.59
Outer right	Left	-0.32	-0.51	-0.66	-1.02	-1.16	-1.39
	Mid	**-0.28**	**-0.45**	**-0.60**	**-0.91**	**-1.02**	**-1.23**
	Right	-0.24	-0.40	-0.55	-0.81	-0.89	-1.06

Table 4: Libeň bridge opening – deflections at ¼ span

In validating the computational model, very good agreement was achieved between the measured and calculated results, which are listed in the table below. In terms of assessing the results it is necessary to respect the fact that agreement for the load conditions with lower efficiency (LC-1 and LC-2) cannot perfectly correspond to the calculated results. For assessing validation of the computational model, credible regard can be taken for the load conditions LC-3 and LC-4, where the efficiency is higher than 50%. For load conditions LC-5 and LC-6, the structure demonstrated plasticity and it is thus logical that the deflections calculated on the elastic model will report somewhat lower values. For the decisive load condition LC-4 however, nearly perfect agreement was achieved between the measured and calculated values.

Point		LC-1		LC-2		LC-3		LC-4		LC-5		LC-6	
		meas.	calc.	meas.	calc.	meas.	calc.	meas.	calc.	meas.	calc.	meas.	calc.
Outer left	Mid	-0.82	-0.74	-0.95	-1.12	-1.20	-1.09	-1.65	-1.64	-1.84	-1.59	-1.88	-1.56
Inner left	Mid	-1.11	-1.16	-1.48	-1.74	-1.78	-1.69	-2.51	-2.53	-2.67	-2.51	-2.68	-2.49
Inner right	Mid	-1.08	-1.19	-1.50	-1.78	-1.85	-1.73	-2.59	-2.60	-2.70	-2.59	-2.69	-2.57
Outer right	Mid	-0.49	-0.78	-0.85	-1.17	-1.22	-1.16	-1.73	-1.74	-1.72	-1.70	-1.88	-1.68

Table 5: Comparison of deflections (measured / calculated) at crown of arch

In assessing the results, it is appropriate to retroactively establish the efficiency of the load used in regards to the calculated load capacity result in the new computational model. The efficiency is compared using the deformation displayed by the structure – deflection at the peak of the arch, and is applied to the normal load capacity regime of the structure calculated in the following chapters.

Load Condition	LC-1	LC-2	LC-3	LC-4	LC-5	LC-6
Measured deflection [mm]	1.1	1.5	1.9	2.6	2.7	2.7
Calculated deflection [mm]	3.1					
Efficiency [%]	35	48	61	84	87	87

Table 6: Efficiency of load used in SLT

2.5 LOAD

The following chapter describes the loading of the model for flood bridge X-656 (arch 6). The action of arches 1 and 5 is described in detail in the expert report [3].

2.5.1 Permanent action

The composition of the carriageway for flood bridge X-656 was measured using diagnostic methods [2] and at the peak of the arch was captured in the following composition:

- Asphalt layers 160 mm
- Concrete in 3 layers 260 mm
- Backfill 320 mm

Own weight[1]	Concrete	see material characteristics	
	Fill material	γ_s	$= 19.5\ kN/m^3$
Carriageway 320 mm		=26.0 * 0.32	$= 8.3\ kN/m^3$
Sidewalk	Lower part	= 0.24 * 25.0	$= 6.0\ kN/m^3$
	Upper part	= 0.57 * 25.0	$= 14.3\ kN/m^3$

The effects of concrete shrinkage and creep are, in light of the type of construction (statically secure triple-joint arch) and age of the structure (approx. 100 years), ignored in the calculation.

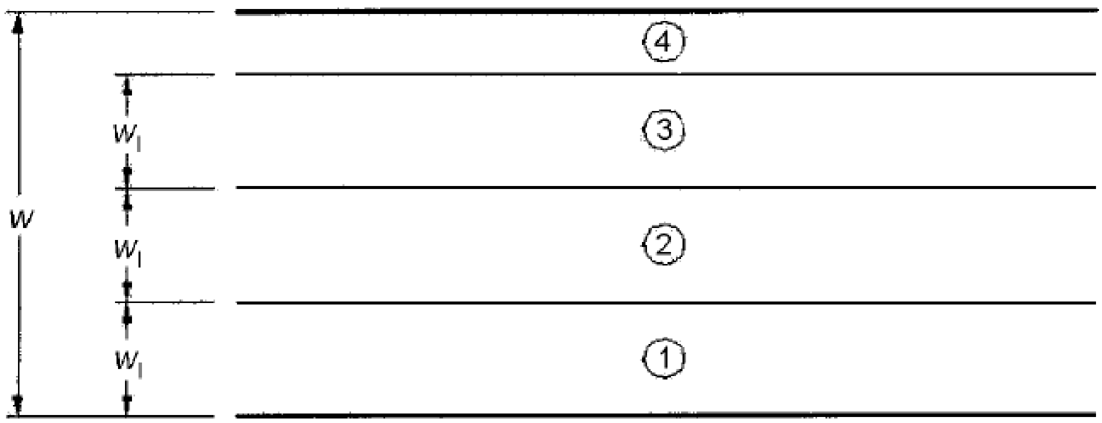
[1] Own weight taken into account directly by MIDAS and contains the load from the own weight of the structure, its backfill, outer walls including brackets and balustrades and the concrete slabs under the carriageway.

2.5.2 Variable actions

It is a combined bridge with tram and road traffic.

2.5.2.1 Number and width of lanes

The lanes will be placed on the structure so as to take into account the position of the remaining space by the median and shoulder.



Legend

w width of carriageway

w_l width of load lane

1 load lane no. 1

2 load land no. 2

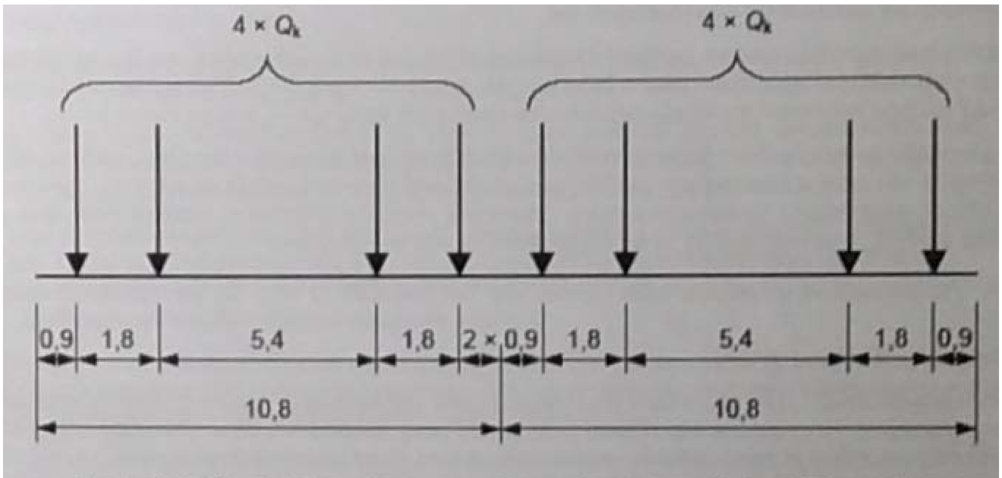
3 load lane no. 3

4 remaining space

Overall road width			= 14.5 m
Width of tram lane	= 2*2.8		= 5.6 m
Traffic area	= 2*4.45		= 8.9 m
Number of lanes in single direction			= 1
Remaining width	= 4.45-3.0		= 1.45 m

2.5.2.2 Trams

Considered in accordance with ČSN EN 1991-2/Z1 – national annex NB.



Obrázek NB.1 – Zatěžovací souprava tramvajových vozidel, vzdálenosti v m

Figure NB.1 – Loading set of tram cars, distance in m

According to the commentary provided in [5], the values of the dynamic coefficient can be considered very low (close to a value of 1.000). In order to assess the load capacity, the calculated safe value under ČSN EN 1991-2/Z1 – national annex NB, Art. NB.2.2 will be left.

Q_k	= 120.0 kN
Dynamic coefficient:	= 1.05

2.5.2.3 Normal load capacity

TYPE OF LOAD

<u>TWO-AXLE</u> : Load lanes no. 1 and no. 2

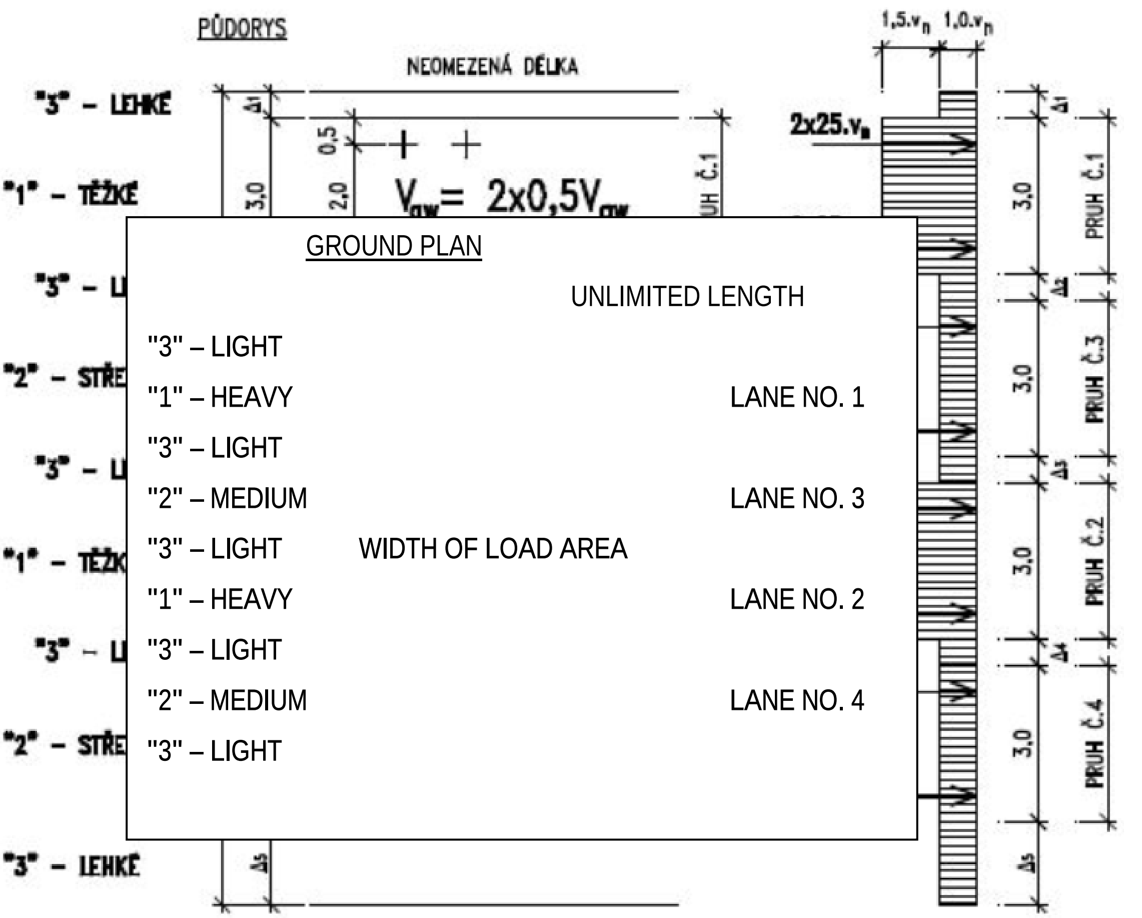
"1" – HEAVY (per wheel)

<u>SINGLE-AXLE</u> : Load lanes no. 3 and no. 4

"2" – MEDIUM (per wheel)

<u>REMAINING SPACE OF LOAD AREA</u>

"3" – LIGHT



GROUND PLAN

UNLIMITED LENGTH

"3" – LIGHT

"1" – HEAVY LANE NO. 1

"3" – LIGHT

"2" – MEDIUM LANE NO. 3

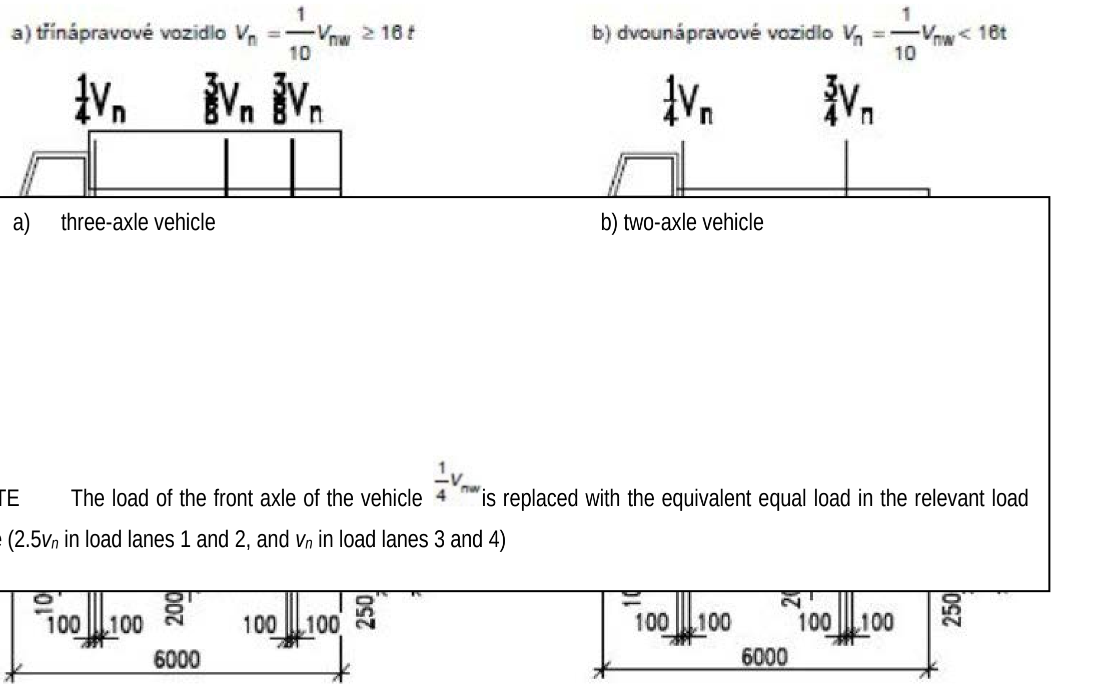
"3" – LIGHT WIDTH OF LOAD AREA

"1" – HEAVY LANE NO. 2

"3" – LIGHT

"2" – MEDIUM LANE NO. 4

"3" – LIGHT



a) three-axle vehicle b) two-axle vehicle

NOTE The load of the front axle of the vehicle $\frac{1}{4}V_{nw}$ is replaced with the equivalent equal load in the relevant load lane (2.5v_n in load lanes 1 and 2, and v_n in load lanes 3 and 4)

POZNÁMKA Zatížení přední nápravou vozidla $\frac{1}{4}V_{nw}$ je nahrazeno ekvivalentním rovnoměrným zatížením v příslušném zatěžovacím pruhu (2,5v_n v zatěžovacím pruhu č. 1 a č. 2, resp. v_n v zatěžovacím pruhu č. 3 a č. 4)

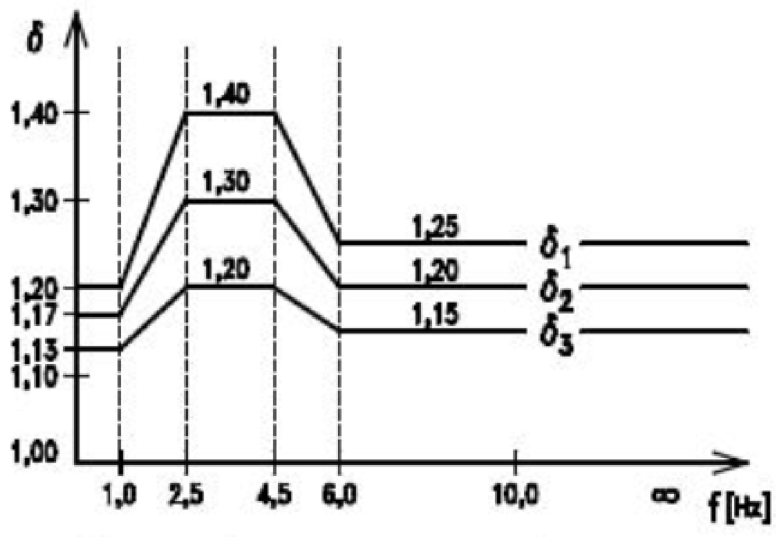
Dynamic effects:

c) Loading with two load lanes and lanes Δi $\delta = \delta_2$

For arch bridges the spare length L_d is equal to half of their span $L_d = 39/2 = 19.5$ m

8.7.1 If measurements are not entirely exact, the natural frequency of the bridge's load-bearing structure or part thereof can be established with a spare length L_d (see Table 8.1) from the formula:

$f = 90.6\ L_d^{-0.923}$ (Hz) (1)



Natural frequency	$= 90.6 * 19.5^{-0.923}$		= 5.84 Hz
Dynamic coefficient δ_2			= 1.21

Horizontal load:

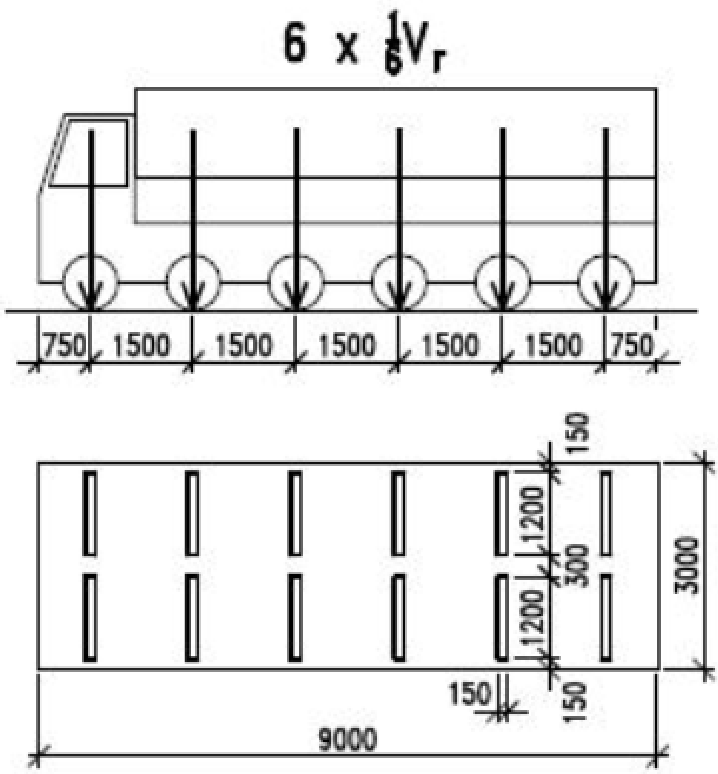
The braking and acceleration forces will be ignored in light of the nature of the structure.

Load sets:

Load set	**Normal load**	**Horizontal force**	**Load of sidewalks and bicycle lanes**
n_1	Characteristic value as per 7.1 [2)]	-	Reduced value w_f = 2.5 kN/m^2
n_2	Frequent value (i.e. $\psi_{1.1}$ times the characteristic value as per 7.1)	Characteristic value [2)] as per 7.4	-
n_3[1)]	Frequent value (i.e. $\psi_{1.1}$ times the characteristic value as per 7.1)	-	-
NOTES	[1)] For assessment of fatigue [2)] Most efficient load		

2.5.2.4 Exclusive load capacity

The mass of a six-axle vehicle must be greater than 50.0 t. The vehicle drives in any lane to the exclusion of other automobile traffic.



Dynamic effects:

b) Action with two, three or four axles; action with whole vehicle $\delta = \delta_2$

Natural frequency	$= 90.6 * 19.5^{-0.923}$		= 5.84 Hz
Dynamic coefficient δ_1			= 1.27

Horizontal load:

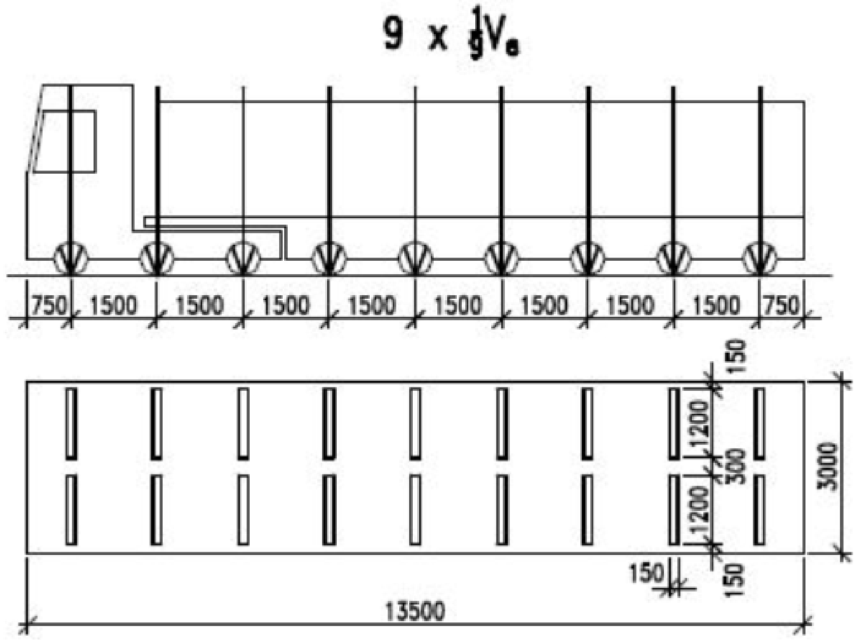
The braking and acceleration forces will be ignored in light of the nature of the structure.

Load sets:

Load set	Exclusive load	Horizontal force	Vertical action of sidewalks and bicycle lanes
r_1	Characteristic value as per 7.2 [1)]	-	Reduced value w_f = 2.5 kN/m^2
n_2	Frequent value (i.e. $\psi_{1.1}$ times the characteristic value as per 7.2)	Characteristic value [1)] as per 7.4	-
NOTE [1)] Most efficient load.			

2.5.2.5 Exceptional load

The vehicle moves along the axis of the bridge ±0.5 m to the exclusion of other traffic on the bridge and with a low speed of up to 5 km/h.



Dynamic effects:

b) Action with multiple axles; action with whole vehicle $\delta = 1.05$

Horizontal load:

7.4.3 In establishing exceptional load, horizontal actions are not considered.

Load sets:

7.5.4 For establishing exceptional load, a single set of actions is used with characteristic values of vertical action as per 7.3.

2.6 LOAD COMBINATION FOR ESTABLISHING LOAD CAPACITY

The described loads are combined in the sense of standards ČSN 73 6209 and ČSN EN 1990.

2.6.1 Ultimate limit state

10.1.1 The load combination for establishing bridge load capacity with regard to ultimate limit state is determined in accordance with ČSN EN 1990 and the relevant European design standards.

In these combinations $Q_{k,1}$ is the characteristic value of the variable load for the most efficient traffic load set established for the appropriate load capacity V_{n1} V_{r1} V_e according to chapter 7. The coefficient of the combination for establishing the relevant load capacity is set with the value $\psi_{0,1}$ = 0.75.

Basic combinations:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \quad \dots \quad (6.10)$$

Alternatively:

$$\sum_{j\geq 1}\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}\psi_{0,1}Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \quad \dots \quad (6.10a)$$

$$\sum_{j\geq 1}\xi_j\gamma_{G,j}G_{k,j}\text{"+"}\gamma_P P\text{"+"}\gamma_{Q,1}Q_{k,1}\text{"+"}\sum_{i>1}\gamma_{Q,i}\psi_{0,i}Q_{k,i} \quad \dots \quad (6.10b)$$

2.6.3 Serviceability limit state

10.2.1 The load combination for establishing the bridge load capacity with regard to the serviceability limit state is determined in accordance with ČSN EN 1990.

In these combinations $Q_{k,1}$ is the characteristic value of the variable load for the most efficient traffic load set established for the appropriate load capacity V_{n1} V_{r1} V_e according to chapter 7. The coefficient of the combination for establishing the relevant load capacity is set with the value $\psi_{1,1}$ = 0.75.

Characteristic combination:

$$\sum_{j\geq 1}G_{k,j}\text{"+"}P\text{"+"}Q_{k,1}\text{"+"}\sum_{i>1}\psi_{0,i}Q_{k,i} \quad \dots \quad (6.14b)$$

Frequent combination:

$$\sum_{j\geq 1}G_{k,j}\text{"+"}P\text{"+"}\psi_{1,1}Q_{k,1}\text{"+"}\sum_{i>1}\psi_{2,i}Q_{k,i} \quad \dots \quad (6.15b)$$

Quasi-permanent combination:

$$\sum_{j\geq 1}G_{k,j}\text{"+"}P\text{"+"}\sum_{i\geq 1}\psi_{2,i}Q_{k,i} \quad \dots \quad (6.16b)$$

2.6.3 Values of combination coefficients

Action	Symbol		ψ_0	ψ_1	ψ_2
Traffic loads (see EN 1991-2, Table 4.4)	gr1a (LM1+ pedestrian or cyclist loads)[1)]	TS (two-axle)	0.75	0.75	0
		UDL (equal load)	0.40	0.40	0
		Pedestrian + cyclist loads[2)]	0.40	0.40	0
	gr1b (single axle)		0	0.75	0
	gr2 (horizontal forces)		0	0	0
	gr3 (pedestrian loads)		0	0	0
	gr4 (LM4 – crowd loading)		0	0.75	0
	gr5 (LM3 – special vehicles)		0	0	0
Wind forces	F_{wk} - persistent design situations - execution		 0.6 0.8	 0.2 -	 0
	F_w*		1.0	-	-
Thermal actions	T_k		0.6[3)]	0.6	0.5

2.6.3.1 Design load values (STR/GEO) – Set B

Basic combination:

Persistent and transient design situations	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Most efficient (if any)	Other
(Eq. (6.10))	$\gamma_{Gj_1 sup} G_{kj_1 sup}$	$\gamma_{Gj_1 inf} G_{kj_1 inf}$	$\gamma_P P$	$\gamma_{Q,1} Q_{k_1 1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

Alternatively:

Persistent and transient design situations	Permanent actions		Prestress	Leading variable action (*)	Accompanying variable actions (*)	
	Unfavourable	Favourable			Most efficient (if any)	Other
(Eq. (6.10a))	$\gamma_{Gj_1 sup} G_{kj_1 sup}$	$\gamma_{Gj_1 inf} G_{kj_1 inf}$	$\gamma_P P$		$\gamma_{Q,1} \psi_{0,1} Q_{k,1}$	$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$
(Eq. (6.10b))	$\xi \gamma_{Gj_1 sup} G_{kj_1 sup}$	$\gamma_{Gj_1 inf} G_{kj_1 inf}$	$\gamma_P P$	$\gamma_{Q,1} Q_{k_1 1}$		$\gamma_{Q,i} \psi_{0,i} Q_{k,i}$

(*) Variable actions are those listed in tables A2.1 through A2.3.

NOTE 1 The choice between (6.10) or (6.10a) and (6.10b) will be in the National annex. In case of (6.10a) and (6.10b), the National annex may in addition modify (6.10a) to include permanent actions only.[NP20)]

NOTE 2 The γ and ξ values may be set by the National annex. The following values for γ and ξ are recommended when using expressions (6.10) or (6.10a) and (6.10b):[NP20)]

$\gamma_{Gsup} = 1.35$ [1)]

$\gamma_{Ginf} = 1.00$

$\gamma_Q = 1.35$ if Q represents an unfavourable load from road traffic or pedestrians; (0 for favourable);

$\gamma_Q = 1.45$ if Q represents an unfavourable load from rail traffic, for load sets 11 to 31 (with the exception of 16, 17, 26[3)] and 27[3)], load model 71, SW/0 and HSLM and actual trains if considered as individual main traffic loads; (0 for favourable);

$\gamma_Q = 1.20$ if Q represents unfavourable loads from rail traffic, for load sets 16 and 17 and SW/2; (0 for favourable);

$\gamma_Q = 1.50$ for other traffic loads and other variable loads; [2)]

$\xi = 0.85$ (so $\xi\gamma_{G_1 sup} = 0.85 \times 1.35 \cong 1.15$).

$\gamma_{Gset} = 1.20$ in the case of linear elastic analysis and $\gamma_{Gset} = 1.35$ in the case of non-linear analysis, for design situations where uneven settlements can have unfavourable effects. For design situations where actions caused by uneven settlements can have favourable effects, these actions are not to be taken into account.

See also EN 1991 through EN 1999 for γ values that are used for imposed deformations.

γ_P = the recommended values defined in the applicable Eurocodes for design.

2.6.4 <u>Combinations used in computational model</u>

1 G+G0 Active Add
Dead Load(1.000) + Erection Load_1(1.000) + Erection Load_2(1.000)
\+ Erection Load_3(1.000) + Erection Load_4(1.000) + Erection Load_5(1.000)
\+ Erection Load_6(1.000) + Erection Load_7(1.000) + Erection Load_8(1.000)
\+ Erection Load_9(1.000) + Erection Load_10(1.000)

----

2 Norm_S_CHAR Active Add
Norm_T_S_L_CHAR(1.000) + Norm_T_S_P_CHAR(1.000) +
Norm_L_S_CHAR(1.000)

----

3 Norm_S_FREQ Active Add
Norm_T_S_L_FREQ(1.000) + Norm_T_S_P_FREQ(1.000) +
Norm_L_S_CHAR(0.400)

----

4 Norm_K_CHAR Active Add
Norm_T_K_L_CHAR(1.000) + Norm_T_K_P_CHAR(1.000) +
Norm_L_K_CHAR(1.000)

----

5 Norm_K_FREQ Active Add
Norm_T_K_L_FREQ(1.000) + Norm_T_K_P_FREQ(1.000) +
Norm_L_K_CHAR(0.400)

----

6 Norm_CHAR Active Envelope
Norm_S_CHAR(1.500) + Norm_K_CHAR(1.500)

----

7 Norm_FREQ Active Envelope
Norm_S_FREQ(1.500) + Norm_K_FREQ(1.500)

----

8 MSU_normalni Active Add
G+G0(1.350) + Tramvaje_CHAR(1.350) + Q_Chodci(0.675)
\+ Norm_CHAR(1.350)

----

9 CHAR_normalni Active Add
G+G0(1.000) + Tramvaje_CHAR(1.000) + Q_Chodci(0.500)
\+ Norm_CHAR(1.000)

----
10 FREQ_normalni Active Add
G+G0(1.000) + Tramvaje_CHAR(0.800) + Q_Chodci(0.200)
\+ Norm_FREQ(1.000)
----
11 Vyhradni_ENV Active Add
Vyhradni_CHAR(500.000)
----
12 MSU_vyhradni Active Add
G+G0(1.350) + Tramvaje_CHAR(1.350) + Q_Chodci(0.675)
\+ Vyhradni_ENV(1.350)
----
13 CHAR_vyhradni Active Add
G+G0(1.000) + Tramvaje_CHAR(1.000) + Q_Chodci(0.500)
\+ Vyhradni_ENV(1.000)
----
14 FREQ_vyhradni Active Add
G+G0(1.000) + Tramvaje_CHAR(0.800) + Q_Chodci(0.200)
\+ Vyhradni_ENV(1.000)
----
15 Vyjimecna_ENV Active Add
Vyjimecna_CHAR(1960.000)
----
16 MSU_vyjimecna Active Add
G+G0(1.350) + Vyjimecna_ENV(1.350)
----
17 CHAR_vyjimecna Active Add
G+G0(1.000) + Vyjimecna_ENV(1.000)
----
18 FREQ_vyjimecna Active Add
G+G0(1.000) + Vyjimecna_ENV(1.000)
----
19 QUASI Active Add
G+G0(1.000)
----

tramvaje = trams chodci = Pedestrians vyhradni = exclusive vyjimecna = exceptional

Notes on listed combinations:

It is evident from the above combinations that for tram actions in the ULS combination, the γ value 1.35 is used. It is not clear from the ČSN EN 1990 standard whether tram actions are included among rail traffic for which a γ value of 1.45 applies. If we were to use the coefficient 1.45 for tram actions, no change in the structure's load capacity would occur as the ULS assessment is not the deciding factor for arch structures.

It is furthermore evident that tram loads are also taken into account for exclusive load capacity. It is not evident from the ČSN 73 6222 standard whether the exclusive vehicle is the sole vehicle on the bridge or the only road vehicle on the bridge. To be safe we thus consider that tram traffic is not *limited for exclusive load capacity and the exclusive vehicle is thus the sole road vehicle on the bridge moving in any lane. Tram and pedestrian traffic is excluded for exceptional load capacity.*

It is furthermore evident that for frequent combination of actions a safe value of 1.0 is used for the coefficient ψ for tram actions. As a whole the normative regulations do not specify load sets for composite bridges and refer only to the regulation ČSN EN 1990, specifically the part that defines combinatory coefficients for rail actions. The safest (maximum) value of $\psi = 1.0$ is applied.

3 LOAD CAPACITY

The calculation of the load capacity is based on the validated computational models according to the results of the SLTs conducted. Calculating the bridge's load capacity is carried out by assessing the exclusion of tensile stress in the arch structure for the SLS (for the frequent combination of actions) and restricting the size of tensile stress for the characteristic combination of actions, in keeping with the prior procedure for establishing load capacity as per [3] as well as in the ultimate limit state by testing on the interactive diagrams for simple concrete.

The calculated load capacity does not take into account the possibility of occurrence of hidden structural faults that could not be expected based on the surveys and diagnostics carried out due to limited access to the load-bearing structure from the upper surface and potentially the insufficient amount of input data for calculation. These include the following risks and influences:

- the effect of lower actual mass of the load-bearing structure (and backfill) reducing the natural prestressing of the bridge's arch sections
- the effect of a change in geometry of the arch's centreline that adversely influences the development of bending moments
- the effect of local weakening of the cross-section through a fault (degradation) of the concrete

An intensified load test conducted on the arch part of Libeň Bridge V009 and X-656 did not show the existence of any of the above faults, but their occurrence in the future as a result of ongoing degradational processes cannot be ruled out.

Evaluation of the load capacity of the arch sections of the bridge has been conducted for the structures with the eliminated negative influence of the frame struts founded on the outer parts of the arch near the abutment joint.

3.1 SERVICEABILITY LIMIT STATE (SLS)

In the serviceability limit state, testing is done on the restricting the emergence of tensile stress for the frequent combination of actions and testing of restricting the size of tensile stress in the structure and compliance with the design tensile strength of simple concrete for the characteristic combination of actions. The design tensile strength of simple concrete is set at a value of 0.3 MPa for class C16/20.

3.1.1 Arches 1 and 5 of Libeň Bridge V009

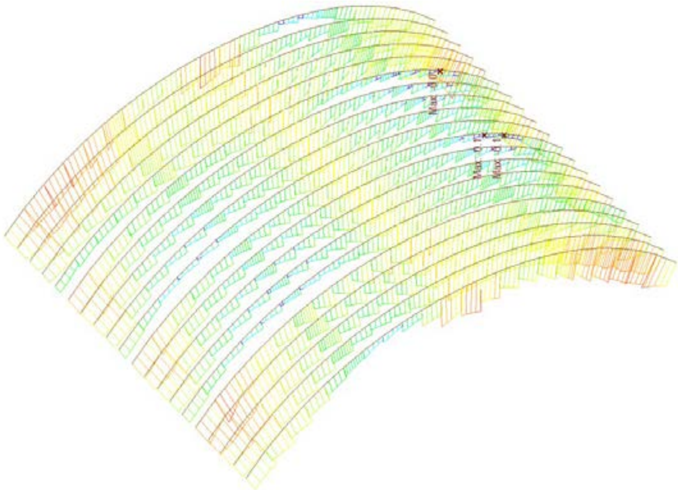


Figure 11: Arch 1 – Elimination of tensile stress for frequent combination of actions (Vn=32 t)

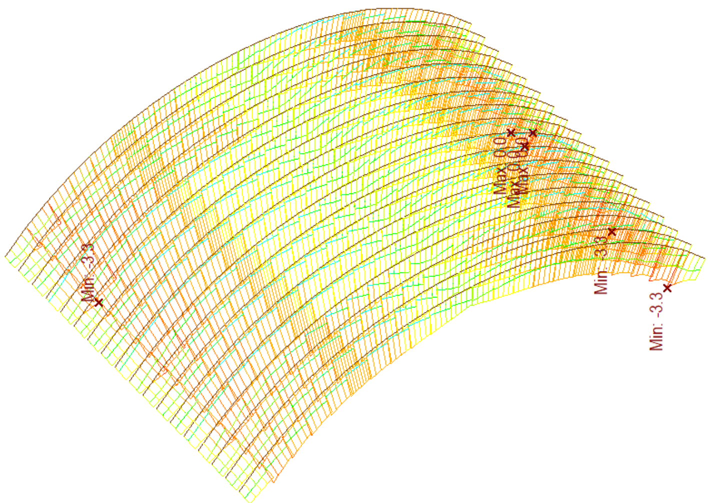


Figure 12: Arch 1 – Elimination of tensile stress for frequent combination of actions (Vr=80 t)

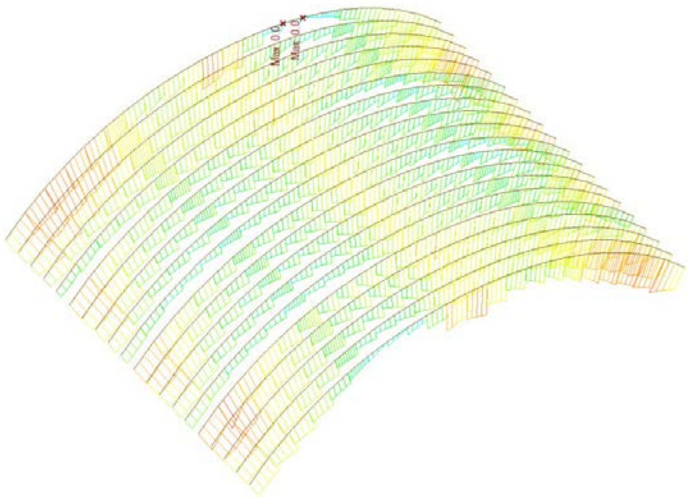


Figure 13: Arch 1 – Elimination of tensile stress for frequent combination of actions (Ve=196 t)

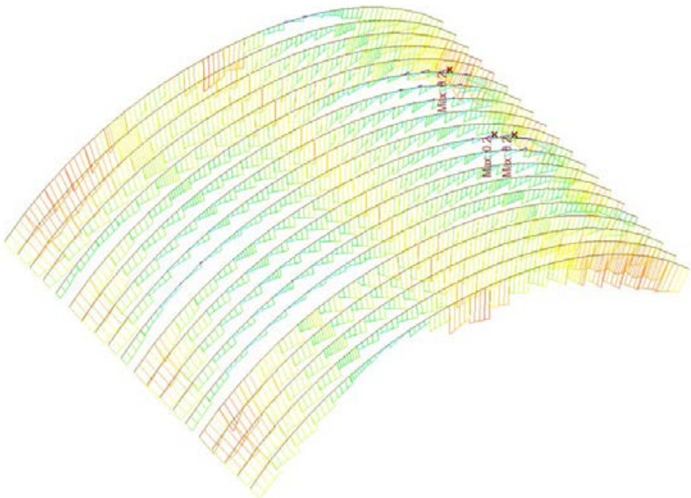


Figure 14: Arch 1 – Reducing tensile stress for characteristic combination of actions (Vn=32 t)

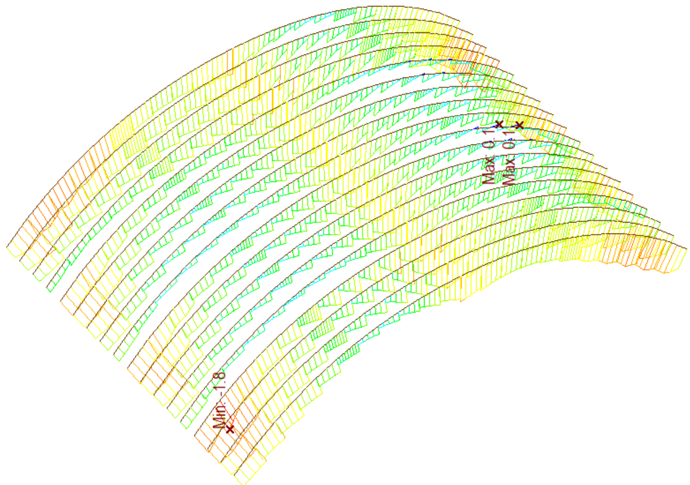


Figure 15: Arch 1 – Reducing tensile stress for characteristic combination of actions (Vr=80 t)

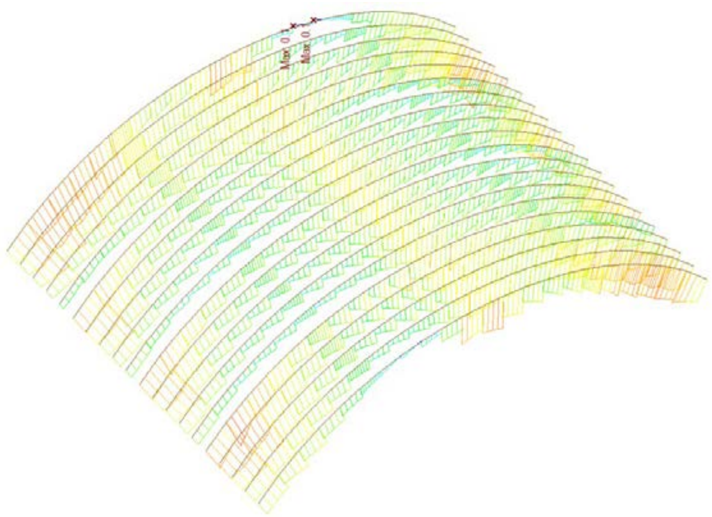


Figure 16: Arch 1 – Reducing tensile stress for characteristic combination of actions (Ve=196 t)

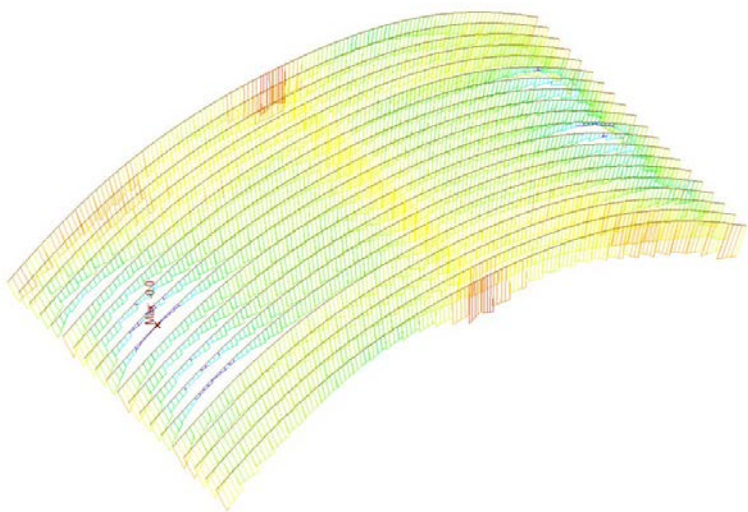


Figure 17: Arch 5 – Reducing tensile stress for frequent combination of actions (Vn=20t)

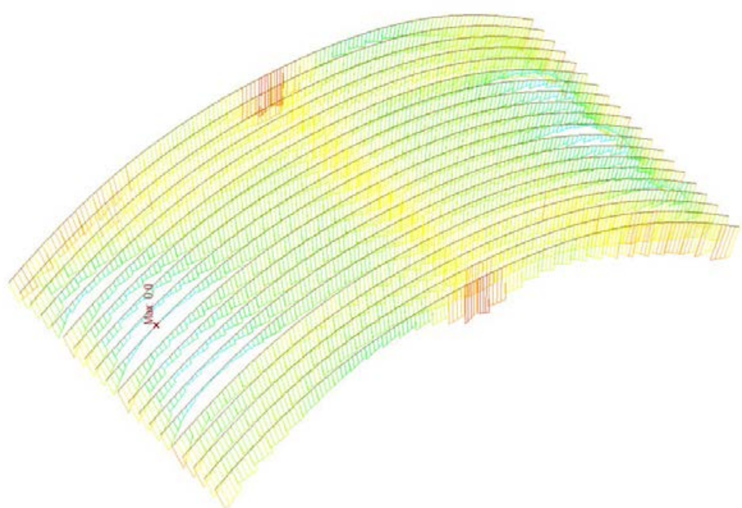


Figure 18: Arch 5 – Reducing tensile stress for frequent combination of actions (Vr=50 t)

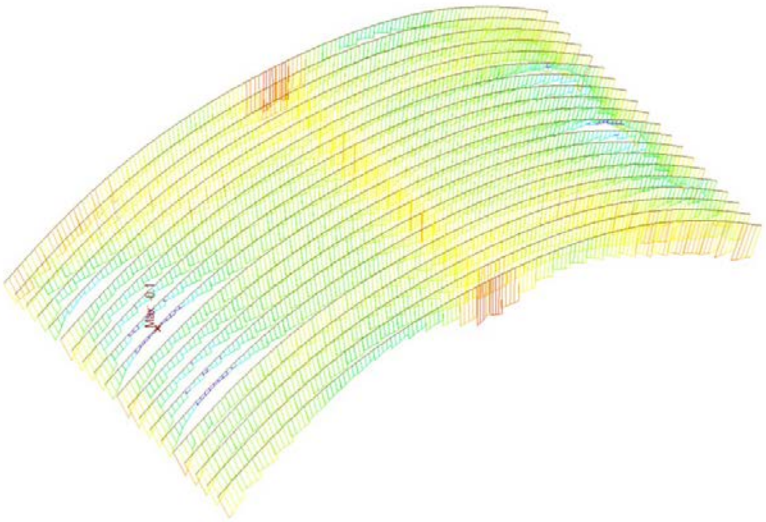


Figure 19: Arch 5 – Reducing tensile stress for frequent combination of actions (Ve=196 t)

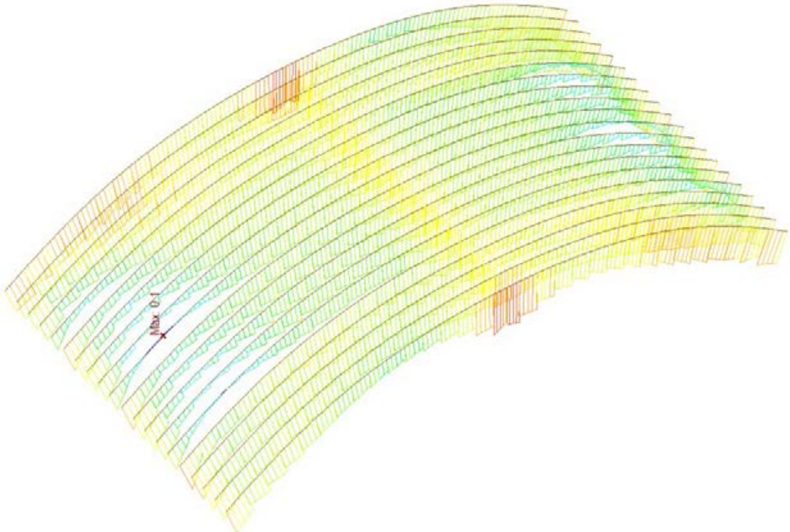


Figure 20: Arch 1 – Reducing tensile stress for characteristic combination of actions (Vn=20 t)

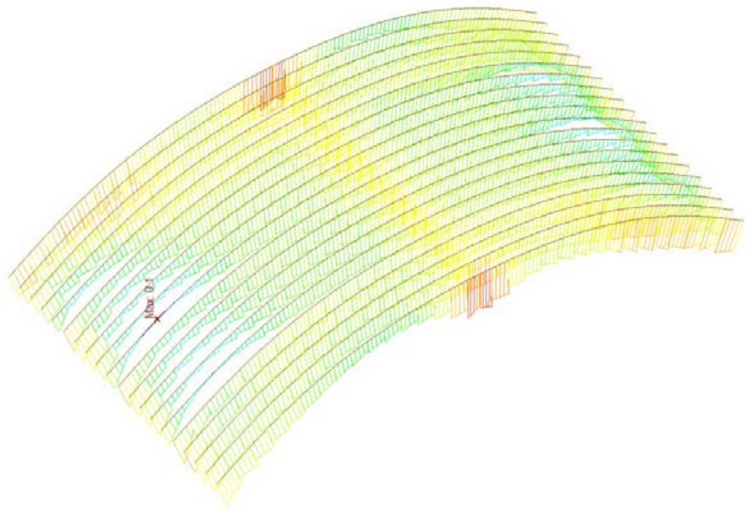


Figure 21: Arch 1 – Reducing tensile stress for characteristic combination of actions (Vr=50 t)

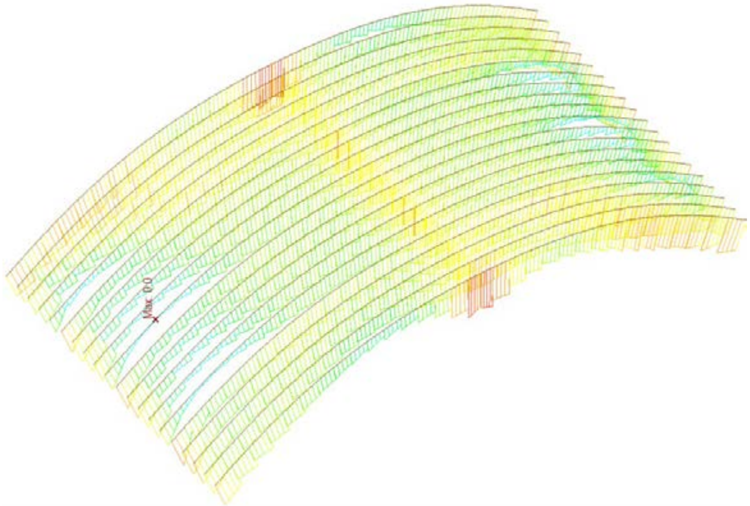


Figure 22: Arch 1 – Reducing tensile stress for characteristic combination of actions (Ve=196 t)

3.1.2 <u>Flood Bridge X-656</u>

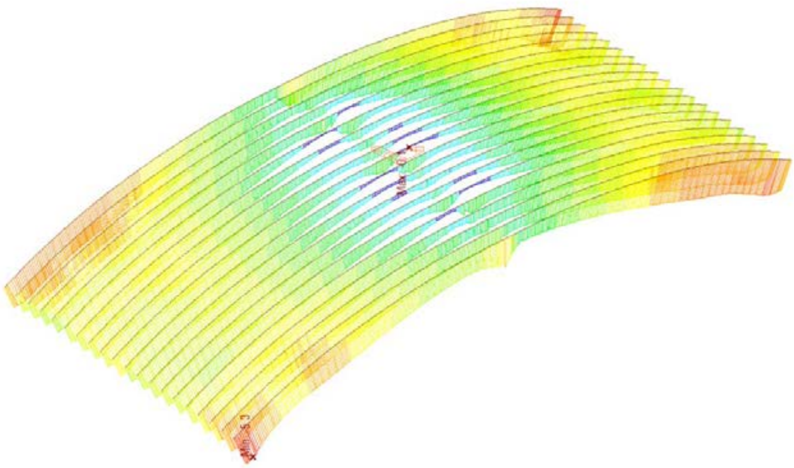


Figure 23: Arch 6 – Reducing tensile stress for frequent combination of actions (Vn=20 t)

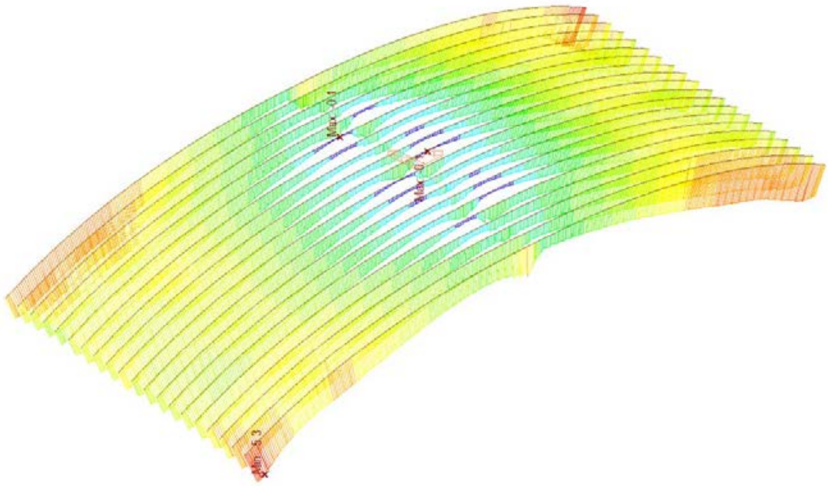


Figure 24: Arch 6 – Reducing tensile stress for frequent combination of actions (Vr=50 t)

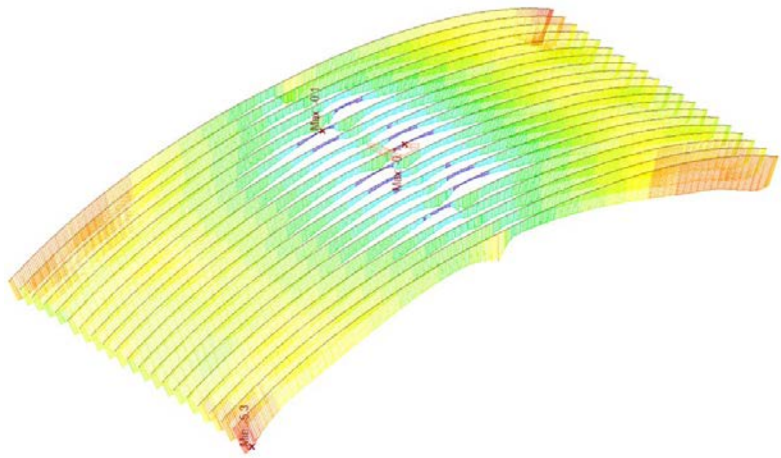


Figure 25: Arch 6 – Reducing tensile stress for frequent combination of actions (Ve=196 t)

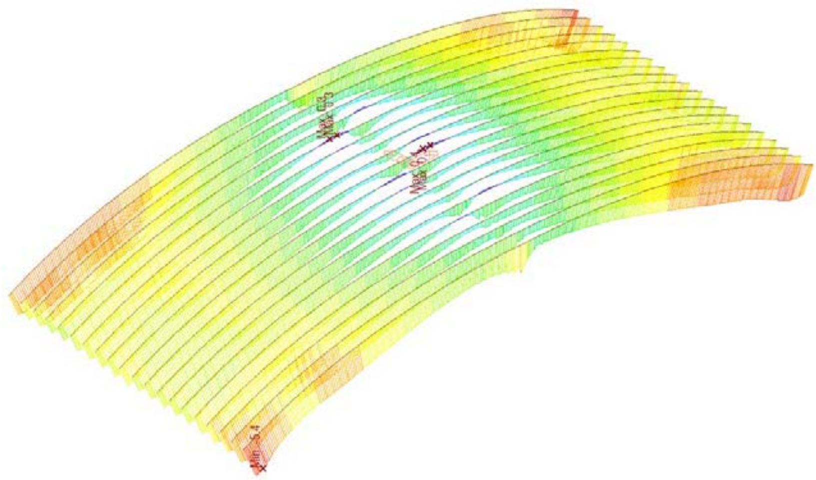


Figure 26: Arch 6 – Reducing tensile stress for characteristic combination of actions (Vn=20 t)

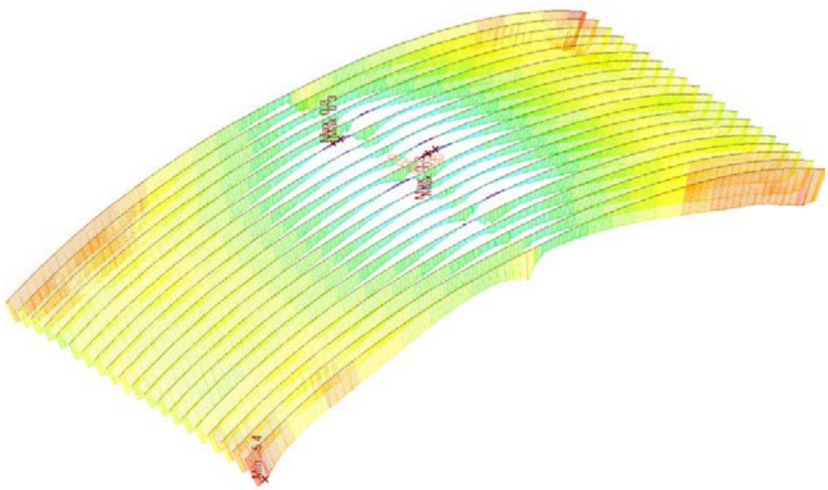


Figure 27: Arch 6 – Reducing tensile stress for characteristic combination of actions (Vr=50 t)

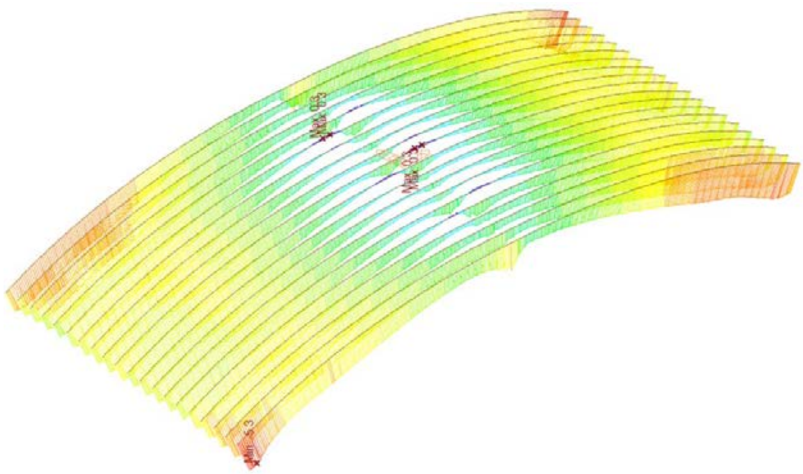


Figure 28: Arch 6 – Reducing tensile stress for characteristic combination of actions (Ve=196 t)

3.2 ULTIMATE LIMIT STATE (ULS)

In the ultimate limit state, testing of structure reliability is carried out in an interactive diagram for simple concrete in the areas with the most pronounced bending moments.

3.2.1 Arches 1 and 5 of Libeň Bridge V009

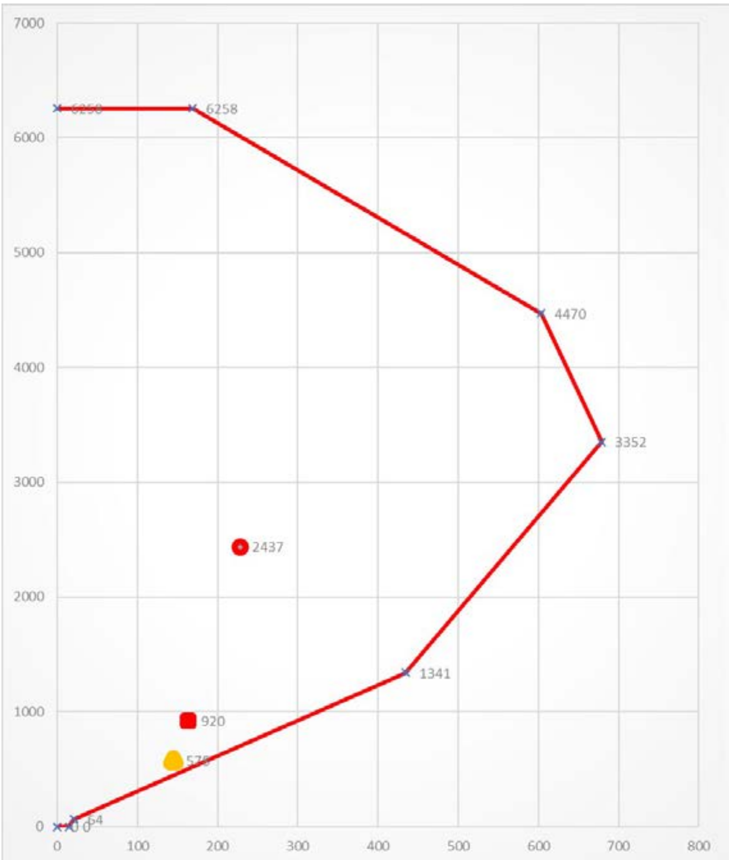


Figure 29: Arch 1 – Interactive diagram of ULS envelope

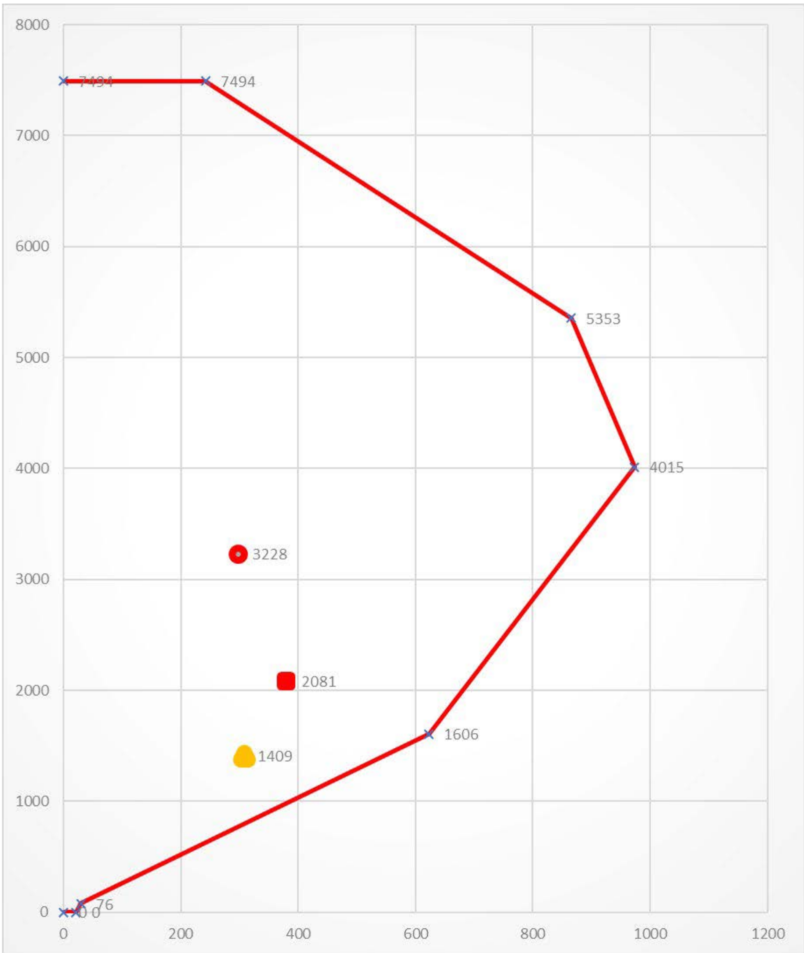


Figure 30: Arch 5 – Interactive diagram of ULS envelope

3.2.2 Flood Bridge X-656

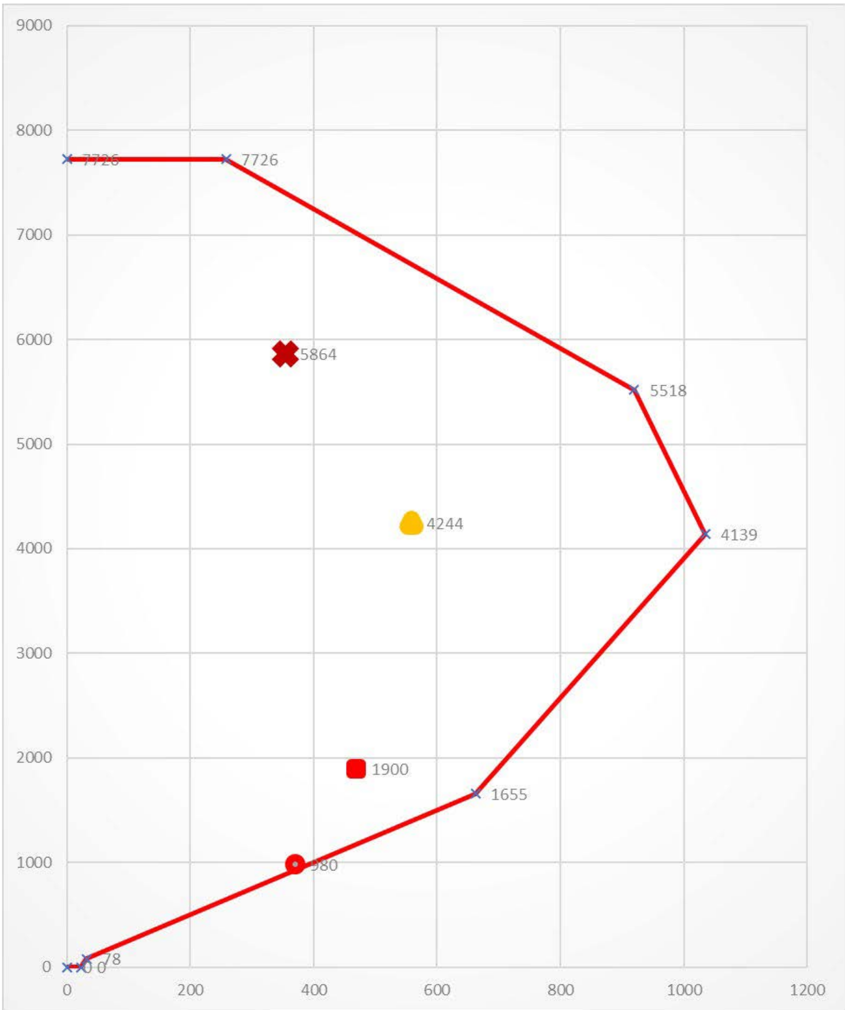


Figure 31: Arch 6 – Interactive diagram of ULS envelope

4 CONCLUSION

A calculation was made of the load capacities of arches 1 and 5 of bridge V009 after modifications to eliminate the negative effect of the bridge's framing bridgeheads having settled around the abutment joints.

On the basis of an SLT performed on the flood bridge (arch 6) in order to test the structure's response to increased stress, a new computational model thereof was produced. Based on the measured results, validation of the computational model was conducted and an update made to the evaluation of bridge X-656's load capacity. Agreement between the results of the static load test and the calculated expected deflections was very high and demonstrated the quality of the computational model.

Load capacity	Arch 1	Arch 5	Arch 6
Normal (Vn)	32 t	20 t	20 t
Exclusive (Ve)	80 t	50 t	50 t
Extraordinary (Vr)	196 t	196 t	196 t

Table 7: Result of recalculating load capacity without considering risks

The load capacity of the outer arches 1 and 5 of the main bridge over the Vltava V009 after recalculation and the planned modifications (removing the frame struts at the site of the joints) was determined to be Vn=32t / Vr=80t / Ve=196t for arch 1 and Vn=20t / Vr=50t / Ve = 196t for arch 5.

The load capacity of bridge X-656 after recalculating for the planned modifications (removing the frame struts at the joint site) was determined to be Vn=20t / Vr=50t / Ve=196t.

Report [5] defines the load capacity of arches 2,3 and 4 of the bridge over the Vltava V009.

A summary of the load capacity values of the arch sections of the Libeň bridge set of the bridge over the Vltava V009 (arches 1-5) and bridge X-656 (arch 6) is provided in Table 7.

Load capacity	arch 1	arch 2	arch 3	arch 4	arch 5	arch 6
Vn	32	20	32	32	20	20
Vr	80	50	80	80	50	50
Ve	196	196	196	196	196	196

Table 8: Load capacity of arch sections

The total load capacity of the Libeň set of bridges with the planned modifications (removal of the frame structures including the mounted frame struts on the arch structures of both renovated bridges V009 and X-656) is defined by the values Vn=20t / Vr=50t / Ve196t.

5 RECOMMENDATIONS

Leaving the current values for traffic load capacity as listed in the BMS unchanged for both bridges because:

- The current total load capacity of both bridges is influenced by the state of the frame structures and is Vn = 11 tonnes for V009 and Vn = 6 tonnes for X-656, as stated in previous reports and findings and as recorded in the BMS
- Removing the negative impact of the settling of the frame bridgeheads on the arch parts of bridges V009 and X-656 for arches 1, 5 and 6 as part of renovations.

The report includes:

1. Expert Report No. 2000 J 190-1 LIBEŇ BRIDGE X-656 (FLOOD BRIDGE) DIAGNOSTIC OF JOINTS, 17 July 2020
2. Expert Report No. 2000 J 190-2 LIBEŇ BRIDGE X-656 (FLOOD BRIDGE) DIAGNOSTIC OF BRIDGE SUPERSTRUCTURE, 11 August 2020