Detailed Examination of Myslinka Stone Railway Bridge, Czech Republic

Sarka Nenadalova, Tomas Bittner, Milan Rydval, Lukas Balik, and Milan Hrabanek
Klokner Institute, CTU in Prague, Prague 6, Czech Republic
Email: {sarka.nenadalova, tomas.bittner, milan.rydval, lukas.balik, milan.hrabanek}@cvut.cz

***Abstract*—A detailed examination of the stone railway bridge Myslinka, built approximately in 1878, was performed. The Bridge is on the railway line Plzeň – Tachov, the Czech Republic. The bridge is created by two parallel vaulted tubes made from sandstone where a frequented road goes through under one of them and Myslinsky brook flows under the second one. In 2006 a reinforced concrete frame structure was inbuilt to the bridge portal. Lengths of the both tubes are about 45.5 m, widths are about 5.7 m, height of the tube above the road is about 6.0 m and the one above the brook is 7.8m. The arch is made as an annular vault. On the basis of the diagnostic works the structure is evaluated from the point of view of moisture, amount of water soluble salts, strengths of sandstone. There is also a visual observation of a condition of the structure itself performed. There are also recommendations for a consequent procedure in case of a revitalization stated in the end of the paper.**

***Index Terms*—examination, diagnosis, railway stone bridge, vault, sandstone, mechanical tests, moisture, water soluble salts, recommendations**

I. Bridge Description

The railway stone bridge no. 43-38-03 was built approximately in 1878. It means in the time as the construction of the railway track Plzeň - Tachov. The bridge is created by two parallel vault tubes made from sandstone where there is a road of the second class under one tube and Myslinsky brook (Fig. 1) under the second one. The lengths of these tubes are about 45.5 m and widths are approximately 5.7m. The height of the tube over the road is about 6.0m and the one over the brook approximately 7.8m. The vaults are constructed from blocks of fine-grained sandstone. The middle support and the both outer supports are a combination of blocks made from pudding stone, fine-grained sandstone and coarse-grained sandstone. The fine-grained sandstone is fair colour whereas the pudding stone as well as coarse-grained sandstone are dark red color. The vaults are divided into three approximately same dilative parts where the dilative joints are not exposed into the supports. Due to this fact there a spontaneous dilatation of the whole structure even in the supports occurs. A frame structure made from reinforced concrete was inbuilt towards to the portal to Myslinka village in 2006. The structure made possible to widen the track from one-lane to two-line in the bridge section. This modification unfortunately contributes to a significant damage of the adjoined portal (Fig. 2).

Figure 1. Bridge above road and Myslinsky Brook.

Figure 2. Damaged bridge portal.

II. Visual Investigation

A visual inspection, even though it is not possible to deny its subjectivity, is one of the most important diagnostic procedures because only this procedure enables to reveal shortages of the whole examined area. Within the diagnostics the detailed visual inspection of stone structures of the bridge focused on a crack locating, degradation and corrosive impacts, a determination of masonry wetness, a masonry crushing etc. were performed. The record of discovered failures was taken down into sections of bridge tubes and both portals. On the basis of the results of the detailed visual inspection and other observed facts it is possible to state followings. The portal with an orientation to Kozolupy village does not evince more significant visible damages while the masonry of the portal of the subsequently built reinforced

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concrete frame (oriented to Myslinka village) is significantly damaged (Fig. 2). The cause of the masonry damage is a force action of the unstable reinforced concrete frame. There is an open joint with width of even several tens mm between the stone masonry of the portal and the reinforced concrete frame. Transverse cracks with typical width of 0.3 – 2 mm were detected in the both vaults. According to a visual aspect, an extent of pollution, a rounding of crack edges etc. it is possible to presume that the cracks, apart from exceptions, are very old. These cracks are spreading through stones as well as joints. Mainly thermal or moisture or volumetric changes are the cause of the formation of these cracks. The vaults of the first dilative part (elements of the reinforced concrete frame) are not damaged by cracks, except recently created cracks that are caused by force actions of the unstable reinforced concrete frame. Degradation and falling down of surface layers with thickness 10 – 30 mm was locally observed at particular stones. Especially vertical cracks with typical 0.2–3 mm widths were detected at all supports in locations where the dilative joints of the vaults are not exposed.

Figure 3. Type of stones taken by core drills

III. Verification of Vault Thickness and Composition

Thicknesses and compositions of vaults were observed using Ø 70 mm core drills in each dilative part of the both openings of the bridge, it means in total six core drills were performed. The core drills were performed always into stones of the fourth line from the top of the vault. Thickness of the vault was observed by Ø 10 mm core drills. On the ground of the observation from the performed core drills it is possible to state that the bridge vaults are assembled from blocks made of fair fine-grained sandstone. Stone masonry from pudding stone, coarse-grained sandstone and migmatite (Fig. 3) was found out on the reverse side of the vault. The stones used for this masonry are various sizes and shapes. The pudding stone and fine-grained sandstone included in the masonry are same type as in the support masonry. The observed thicknesses of the blocks of fair fine-grained sandstone are ranged from 640 to 960 mm. The discovered total thickness of the vault of the first dilative part (outer part) including the masonry on the reverse side of the vault is about 1200 mm. The discovered total thickness of the vault of the second dilative part (middle part) including the masonry on the reverse side of the vault is about 1600 mm. The core drills are described and showed in photos (Fig. 4) and the summary of the observed facts is shown in Table I

Figure 4. Samples for laboratory tests

TABLE I. Vault Thicknesses

Opening (VAULT)	Dilative part	Core drill	Drill top	Vault thickness [mm]	
				Block made from fine-grained sandstone	Total including masonry on vault reverse side
1 Over road	1-Outer	JV3		640	About 1250
				No detection	-
	2-Middle	JV2		800 - 830	About 1500
			2	About 800	-
	3-Outer	JV1		960	About 1220
			1	> 950	-
2 Over brook	1-Outer	JV6		680	About 1200
			4	About 750	-
	2-Middle	JV5		780	About 1600
				No core drill	-
	3-Outer	JV4		760	-
			3	About 750	About 1300

Notes:
1 – Fine-grained sandstone; vault (core drills JV2 - JV6)
2 – Fine-grained sandstone; vault (core drill JV1)
3 – Coarse-grained sandstone; masonry on vault reverse side (core drills JV1 a JV4)
4 – Pudding stone; masonry on vault reserve side (core drills JV1 a JV5)
5 – Migmatite; masonry on vault reserve side (core drills JV2 a JV4 - JV6)

IV. Non-Destructive Tests of Stone in Compressive Strengths

Non-destructive hardness test by Schmidt hardness-tester (type N-34) was used for the determination of in-situ compressive strength of stone, fine-grained sandstone. Conversion coefficient αc for the conversion of the strengths gained using non-destructive method to the real strengths was determined form the results of the destructive and non-destructive tests performed on the same stones as $\alpha c = 0.79$. The summary of the result of the non-destructive tests of stones in compressive strengths is stated in Table II.

TABLE II. Non-Destructive Tests in Compression Strength of Stone

Opening (vault)	Dilative part	Test number	Average compressive strength f_c [MPa]
1-Over road	1 - 3	12	31.2
2-Over brook	1 - 3	12	33.4
Total average [MPa]		32.7	

V. Tests of Compressive Strengths of Vault Stone

Compressive strength of stone was determined by destructive tests of the core drills as well as the non-destructive tests in-situ. Testing samples with $\lambda = 1$ [1] were made from Ø70 mm core drills for the purposes of the destructive tests in compressive strengths. The tests in compressive strength were performed under various states of wetness of stone, it means in the steady state (after 100 hours in the laboratory) and in saturated state (after 72 hours in water). Bulk density of the stone was determined in the dried state. An assessment of the tests in compressive strength of the fine grained sandstone is in Table III.

TABLE III. Destructive Tests in Compressive Strength of Stone

Opening (vault)	Dilative part	Core drill	Bulk density [kg/m^3]	Test number	Compressive strength f_c [MPa]	
					Steady	Saturated
1-Over road	1 Outer	JV3	2080	6	29.2	-
				5	-	25.8
	2 Middle	JV2	2167	5	32.6	-
				4	-	22.6
	3 Outer	JV1	2157	4	25.8	-
				3	-	21.5
2-Over brook	1 Outer	JV6	2122	4	31.7	-
				3	-	23.3
	2 Middle	JV5	2197	5	32,6	-
				4	-	24.7
	3 Outer	JV4	2159	5	30.2	-
				4	-	24.6
Total average [MPa]					30.4	24.0
Characteristic strength f_{ck} [MPa]					25.2	19.1
Lower critical value E [MPa]					24.5	17.7

VI. Softening Coefficient of Stone

Stone wetness usually unfavourably influences its compressive strength. The level of the influence depends on many factors, for example on a stone structure, a content of clays, bulk density, granularity, a binder type etc. The softening coefficient in this case means, ratio of stone strength in saturated state to stone strength in steady state. The summary of the determined softening coefficients is showed in Table IV.

The particular determined values of the softening coefficients ranged from 0.69 to 0.88. The average value of the softening coefficients is K = 0.79. It follows that the stone saturated by water evinces in average about approximately 20 % lower compressive strength than the stone in the steady state.

VII. STONE ABSORPTION

Stone absorption under atmospheric air pressure was also performed. The summary of the results of sandstone absorption is possible to see in Table V.

TABLE IV. SOFTENING COEFFICIENT OF STONE CAUSED BY WETNESS - SUMMARY

Opening (vault)	Dilative part	Core drill	Bulk density [kg/m^3]	Test number	Softening coefficient fine-grained sandstone	
					$f_{c,des,n}$ / $f_{c,des,w}$	K
1-Over road	1 - Outer	JV3	2080	6 + 5	25.8 / 29.2	0.88
	2 - Middle	JV2	2167	5 + 4	22.6 / 32.6	0.69
	3 - Outer	JV1	2157	4 + 3	21.5 / 25.8	0.84
2-Over brook	1 - Outer	JV6	2122	4 + 3	23.3 / 31.7	0.73
	2 - Middle	JV5	2197	5 + 4	24.7 / 32.6	0.76
	3 - Outer	JV4	2159	5 + 4	24.6 / 30.2	0.81

TABLE V. ABSORPTION OF STONE UNDER ATMOSPHERIC AIR PRESSURE

Opening (vault)	Dilative part	Core drill	Bulk density [kg/m^3]	Test number	Average absorption A_b [% wt.] fine-grained sandstone
1-Over road	1 - Outer	JV3	2080	5	5.6
	2 - Middle	JV2	2167	4	5.1
	3 - Outer	JV1	2157	3	5.0
2-Over brook	1 - Outer	JV6	2122	3	6.7
	2 - Middle	JV5	2197	4	5.0
	3 - Outer	JV4	2159	4	5.1
Total average [% weight]					5.4

VIII. WETNESS OF STONE IN-SITU

Stone wetness in-situ was determined by a gravimetric analysis and then a measuring using a wetness-measuring device was made. The assessment of the gravimetric analysis and the measuring of stone wetness using the device can be seen in Table VI.

On the basis of the assessment of these results and other findings it is possible to say the followings. For decisive can be considered the results of the gravimetric analysis that proved the excessive wetness of the stone of the both vaults only in the first dilative parts, it means in the subsequently built reinforced concrete frame (excessive wetness w = 5.0–7.5% weight), whereas wetness of the stone from the vaults of the second and the third dilative part is possible to classify as very low or low.

With regard to the fact that the determined stone absorption ranges from 5.0 to 6.7%wt. and the determined in-situ wetness of the stone from the first dilative parts ranges from 5.1 to 6.9%wt., it is possible to state that the stone from the first dilative parts of the vaults is almost water saturated.

The reason of the saturation of the stone from the first dilative parts of the vaults is very probably water retained by the subsequently built reinforced concrete frame. These significant facts were found out during day. The works were performed during days when morning temperatures ranged about 11 °C and afternoon temperatures achieved to 29 °C.

Water vapour condensation on the stone surface of the first dilative parts of the both vaults was visible during afternoons when temperatures achieved even 29 °C and under the vault over the road was temperature about approximately 8 - 10 °C lower and under the vault over the brook about approximately 10 - 12 °C lower.

On the stone surface of the first dilative parts of the vaults over the brook the drops of a condensate, that fallen in drops subsequently, were created. The condensate started to be created mainly on the parts of the vaults with lower surface temperature, it means on the cooler parts where the excessive stone wetness was detected.

When air with temperature 28 °C and relative humidity 60% flows under the bridge vaults then dew point, it means the surface temperature when water vapour starts to condensate on a stone surface, is about 19.5 °C.

TABLE VI. GRAVIMETRIC ANALYSIS OF STONE WETNESS

Opening (vault)	Dilative part	Number of analysed samples	Stone wetness [% wt.]	Wetness classification according to ČSN P 73 0610 [2]
1 Over road	1 - Outer	4	5.6 – 6.2	Increased
	2 - Middle	4	2.0 – 3.2	Very low to low
	3 - Outer	2	1.2 – 2.7	Very low
2 Over brook	1 - Outer	4	5.1 – 6.9	Increased
	2 - Middle	4	2.1 – 3.0	Very low to low
	3 - Outer	2	0.9 – 1.4	Very low

IX. Conclusion, General Proposals and Recommendations

General proposals and recommendations for repairs of the stone bridge are formulated on the basis of the results of the diagnostic works, the assessment their technical state and the contemporary state of cognition [3]. All the performed measures have been done according to the standard [5].

The first basic and necessary step is a stabilization of the reinforced concrete frame. During the stabilization it is necessary to carry out such measures to the frame do not be forced active on the bridge stone masonry, it means to the both structures were independent each other and divided by a real dilative joint. It is essential to carry out a proper drainage of the embankment of the frame not to happen water retaining over the vaults of the first dilative parts. It is imperative to perform some insulation against moisture penetration of the reverse sides of the vaults of the first dilative parts. These measures also decrease the possibility of water vapour condensation on the masonry surface because the lower masonry wetness means higher surface temperature of masonry.

Then it is recommended a local replacement of the masonry (damaged pieces of stone) of the portal. Due to the falling down of the surface stone layers discovered in a relatively small extent of the both tubes, in our opinion the repair of these failures is not immediately acute and necessary. For future (in horizon 5–10 years) it is recommended to carry out local repairs not to occur a progressive deterioration of the state [4]. It is advices to the both tubes be washed by pressure water and subsequently repaired joints, mainly by the supports.

With regard to the fact that in very old cracks were not observed any traces of water seepages (salt infusion, efflorescence, dripstones etc.) it is possible to deduce that no water penetrated and penetrating through the cracks. On the ground of this fact it is not considered a sealant injection of the cracks to be needed. Even a deterioration of the balanced state could be caused in case of an improper injection.

A potential hydrofobization of stone surface should be taken into consideration very cautious because according to our opinion it could very unfavourably influence the natural evaporation of the moisture penetrating into the masonry from the reverse side of the vaults because a moisture closure in the masonry can happened as well as subsequent degradation actions of moisture itself supported by climatic effects. The hydrofobization does not also prevent water vapor condensation on the masonry surface. These are the reasons according to them it is not recommended the hydrofobization of the stone surface without substantiated thoughts of a hydrofobization agent type and a verification of the stone reaction to a treatment by the agent, for instance on a long-term monitored reference area directly on the bridge masonry. [6]

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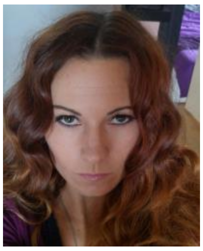

Šárka Nenadálová was born in Prague, Czech Republic on 7th January 1986. Šárka Nenadálová got master's degree at the Czech Technical University in Prague, Faculty of Civil Engineering, branch Civil Engineering Structures in 2011 before her study of a doctoral degree at the Czech Technical University, Klokner Institute, branch Nonmetal and Building Materials.
She has worked as a scientific researcher at the Czech Technical University in Prague, Klokner Institute since the end of the master's degree. Her research interests include special concretes, fly ash concretes and diffusion of bull burnt bricks.
Mrs. Šárka Nenadálová is a membership of the Czech Concrete Society as well as a member of organizing committee of the Concrete Days 2015, 2016 conferences.

Tomáš Bittner was born in Prague, Czech Republic on 7th September 1985. Tomáš Bittner got master's degree at the Czech Technical University in Prague, Faculty of Civil Engineering, branch Civil Engineering Structures in 2011 before his study of a doctoral degree at the Czech Technical University, Klokner Institute, branch Nonmetal and Building Materials.
He has worked as a scientific researcher at the Czech Technical University in Prague, Klokner Institute since the end of the master's degree. His research interests include textile concretes and diagnostics of structures and bridges.
Mr. Tomáš Bittner is a membership of the Czech Concrete Society.

Milan Rydval was born in Jaroměř, Czech Republic on 4th December 1985. Milan Rydval got master's degree at the Czech Technical University in Prague, Faculty of Civil Engineering, branch Civil Engineering Structures in 2011 before his study of a doctoral degree at the Czech Technical University, Klokner Institute, branch Nonmetal and Building Materials.
He has worked as a scientific researcher at the Czech Technical University in Prague, Klokner Institute since the end of the master's degree. His research interests include Ultra High Performance Concretes.
Mr. Milan Rydval is a membership of the Czech Concrete Society.

Lukáš Balík was also born in the Czech Republic. Lukáš Balík got master's degree as well as doctoral degree at the Czech Technical University in Prague, Faculty of Civil Engineering, branch Building Materials.

He has worked as a scientific researcher at the Czech Technical University in Prague, Klokner Institute since the end of the master's degree. His research interests include electroosmotic methods and diagnostics of buildings from the point of view of moisture and dampness.
Mr. Lukáš Balík is a membership of the Czech Concrete Society.

Milan Hrabánek was born in the Czech Republic, too. Milan Hrabánek got master's degree as well as doctoral degree at the Czech Technical University in Prague.
He has worked as a scientific researcher at the Czech Technical University in Prague, Klokner Institute since the end of the master's degree. His research interests include diagnostics of various building and structures.
Mr. Milan Hrabánek is a membership of the Czech Concrete Society.