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Design Concept of the Footbridges V-025 and V-026

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Content

1 Summary

The historical Railway Bridge (Vyšehradský Bridge) carries two railway tracks over the Vltava. Three riveted steel trusses with a span of 72m each bridge the river. Four 20m spans with single span plated girders and a series of masonry arches form the approach on the Vyton side. The footbridges V-025 and V-026 are laterally attached to steel trusses and connect Smíchov and Vyton. The railway bridge is an icon of important national heritage and part of the UNESCO World Heritage Site.

The lack of maintenance of the railway bridge has led to a dire state of the bridge with regards to corrosion and to a reduced structural safety. At the same time, railway traffic demands for a third track across the Vltava. To get an external input for its future strategy, the City of Prague as owner of the footpaths but not of the main bridge itself, asked the authors to outline possibilities for the development of the set-up of railway and pedestrian infrastructure over the Vltava and to give an independent assessment of the repairability of the steel truss bridges complementing the previous work by SUDOP and by Prof. Brühwiler.

The investigations carried out confirm that the bridge requires repair and strengthening to reestablish an acceptable level of structural safety. Overall, the rail stringers and the cross beams as well as the two first inclined bracing members of the lateral trusses are critical. In addition, localized corrosion could require localized measures. Possible concepts of how the measures could be carried out to minimize impact on the rail traffic are outlined with a feasibility study level of detailing. The repair work required is limited and does not stand in the way of the preservation of the bridge as historic monument.

Different options for a river crossing of three railway tracks and footpaths and/or cycling lanes with and without the existing bridge are described on a concept level. The comparison confirms that a preservation of the existing bridge not only is feasible but also generally leads to better solutions than its disposal and replacement with a new structure. Depending on the weight of clearance profile requirements and the extent of repair work, retaining the bridge for one railway track only and supplementing with a new two can be attractive.

It is noted that the study is based on drawings and third-party descriptions of the current condition and therefore relies on certain simplifications and assumptions. The study indicates potential feasible solutions that could serve as a roadmap to a detailed planning study, but it cannot replace such a detailed planning study before any firm decision on measures is made.

2 Parties Involved

Parties	Address	Represented by
Client	TSK a.s. Řásnovka 770/8 CZ-110 00 Praha 1	Mgr. Jozef Sincák
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Consultant		Prof. Petr Tej
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Tab. 1: Parties Involved

3 Introduction

The footbridges V-025 and V-026 belong to the City of Prague. They are of strategical importance for the pedestrian network of the City as these footpaths connect Smíchov and Vyton and because they are the southernmost pedestrian crossing over the Vltava within the city center. The steel footbridges are attached to the sides of the Railway Bridge (Vyšehradský Bridge), itself a riveted steel structure, see Fig. 1 below.

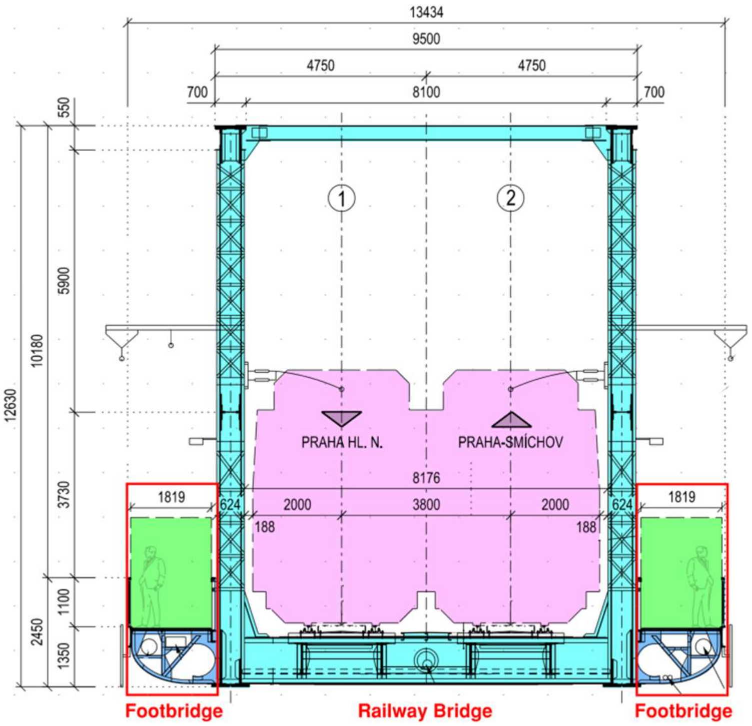


Fig. 1: Cross-section with definitions.

The footbridges have been repaired recently, but the City of Prague is concerned about their structural condition because the railway bridge to which they are attached is not in a good condition. Thus, even after the careful repair of the footbridges themselves the overall structural condition is not good, and the safety of the footbridges remains questionable and will deteriorate further unless action is taken in relation to the condition of the railway bridge.

The railway bridge is an icon of important national heritage and part of the UNESCO World Heritage Site, but its future is uncertain. Because of the speculations about future plans for the railway bridge, the City of Prague wants to be prepared and consider the options for the future of the footbridges as well as the railway bridge which provides their structural support.

For this reason, possible options and design concepts for the future of the footbridges need to be identified and outlined. Fundamentally, there are two scenarios:

a) The pedestrian and the railway crossing are separated. The question then is where to build an additional bridge from architectural, heritage and technical points of view, and what will be the consequences?

b) The current layout with a combined railway and pedestrian crossing is maintained. The question then is to establish the condition of the railway bridge and devise methods for its repair so it can continue to support both the railway and the footbridges?

With regards to the second scenario, various investigations have been carried out already. An important question, however, remains unanswered:

The central, and in terms of heritage value the most important part of the Railway Bridge (Vyšehradský Bridge) consists of three identical simply supported truss spans over the Vltava River. These spans have a well-loved and identifiable character as part of the historic city scape of Prague. Each span is made up of two identical truss girders with arched top chords along the sides, connected by crossbeams that carry the two railway tracks. There is portal frame at each end and wind bracing in plan that connects the two top chords and the two bottom chords. The footpaths are attached on the sides of the bridge.

The condition of the bridge was investigated and consequently judged beyond repair by the Railway Authority based on the work by SUDOP. In 2018 it was suggested that the bridge should be replaced by a new structure that is either a direct copy of the existing or a mimic. However, the heritage value of the bridge and its historic significance as part of an important UNESCO world heritage site requires it to be preserved if possible. Therefore, Prof. Brühwiler was appointed to undertake a structural assessment in 2019. Prof. Brühwiler concluded that a rehabilitated bridge will be able to carry modern train loads and claimed that the bridge can be rehabilitated with no exceptional difficulties.

The safety of the footpaths attached to the railway bridge depends on the railway bridge, and their safety cannot be higher than the safety of the main structure. In order to better understand the possible repair requirements, it is necessary to close the gap between the SUDOP report and Prof. Brühwiler's review. This gap essentially consists of the answer to two questions:

1. What is the likely extent of the required repairs?
2. How could the bridge be repaired while maintaining railway traffic?

The marked difference between the conclusions of the SUDOP and Brühwiler reports raises questions about the extent of the problem, and the absence of a more detailed repair concept has led to doubts about whether repair is possible at all.

The focus of the present investigation is a study to assess the structural condition of the Railway Bridge thus providing a third opinion alongside SUDOP and Prof. Brühwiler, and to consider the feasibility of repairing it, especially the main trusses on each side.

The existing historical bridge comprises three riveted steel trusses over the river, each with a span of 72m. Four 20m spans with single span plated girders and a series of masonry arches form the approach on the Vyton side, see Fig. 2.

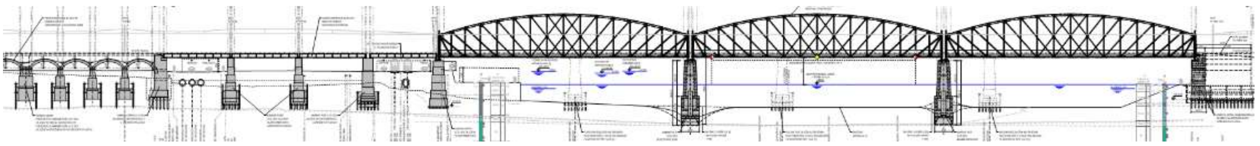


Fig. 2: Longitudinal section.

The present study focuses on the three main riveted steel trusses. As the three spans are identical, only one span will be studied.

It is noted that the study is based on drawings and third-party descriptions of the current condition. It has not been possible to carry out any detailed inspection on site of the bridge condition. In the absence of any particular detailed inspections on site and due to the limited information available, the study relies on certain simplifications and assumptions. The level of detailing in the proposals included in the report is the one of a feasibility study. The study indicates potential feasible solutions that could serve as a roadmap to a detailed planning study, but it cannot replace such a detailed planning study which will be needed in a follow-up phase before any firm decision on the repairs could be made.

4 Basis and Structural Assessment

4.1 Information received

The structural analysis and assessment has been carried out on the basis of the information which has been received, which principally comprises the following:

- Drawings referred to as "Archival documentation 1900" dating from the original construction.
- Drawings referred to as "Archival documentation 1969" dating from the time when major renovation and modification works were carried out.
- Photographs and supplementary information provided by Prof. Dr. P.Tej and his team.
- Photographs taken in connection with a study for a new single-track bridge in 2019 (report "Ideological study of railway bridges under Vyšehrad, 18.3.2020")

Reference has also been made to the following reports and documents:

- Powerpoint presentation entitled "Renewal of Railway Bridge over Vitava River in Prague" by Ing. Martin Vlasak of SUDOP (in English translation) which summarises the conclusions of the SUDOP structural assessment.
- Report by Nikolaï Martin and Professor Eugen Brühwiler entitled "Renovation du viaduct ferroviaire de Vyšehrad, Prague" dated spring 2019.
- Report entitled "Vyšehradem Railway Bridge in Prague – Preservation of the existing bridge" by Professor Eugen Brühwiler dated 1st July 2019

Some information is incomplete, and it has been necessary to make certain assumptions in order to build a complete model to represent the bridge for the structural analysis and assessment. For example, it is clear from the SUDOP presentation mentioned above that the railway girders themselves were strengthened in 1987 but the details of that strengthening cannot be confidently determined from the drawings in our possession. Therefore, we have estimated the size of the additional flange plates added at that time from what information we have been able to find in order to derive the section properties of the railway girders. However, it is our opinion that our conclusions would not be substantially different if our assumptions on these issues turned out to be incorrect.

We have also had to make assumptions about the extent and depth of corrosion currently evident on the bridge. Without the facility to be able to carry out a close detailed inspection and measurement of the corroded sections, we have had to rely on our superficial inspection from the footways in 2019 and photographs provided by Professor Tej and his team. A generous overall corrosion allowance has been made in the analysis to account for the general loss of section, but the detailed analysis of the implications of specific local concentrations of corrosion in particular locations is beyond the scope of this report.

4.2 Design basis and assumptions

The structural assessment of the railway bridge design has been carried out in accordance with the structural Eurocodes, and in particular the following:

- EN 1991-2 Eurocode 1 – Actions of structures – Part 2: Traffic loads on bridges
- EN 1993-2 Eurocode 3 – Design of steel structures – Part 2: Steel Bridges
- EN 1993-1-9 Eurocode 3 – Design of steel structures – Part 1-9: Fatigue

In addition, British Standard BS5400 part 3, the code of practice for the design of steel bridge structures, has been used in places.

It is noted that these are design codes for new-build structures and not assessment of existing structures. This implies an inevitable conservatism since design standards tend to have built-in safe-sided assumptions which can result in over-conservative conclusions when applied to the assessment of existing structures. Various draft assessment codes using Eurocodes are in existence at the time of writing, but these have not been applied since it is not known whether such an approach would be acceptable to the Czech railway authority. It is to be expected that a thorough assessment in accordance with such assessment standards may well conclude that the margins for strength are even higher than reported herein. Amendments to load factors for assessment purposes of a historic structure are discussed in the relevant section below.

4.3 Structural modelling and analysis

We have carried out a global structural analysis of one of the spans of the bridge. The three spans are identical and act as simply supported truss girders. The analysis has included all of the structural elements of the bridge, including the secondary elements associated with the railway girders and crossbeams. Elements were represented by linear beam elements with six degrees of freedom at all nodes. An illustration of the computer model is given in Fig. 3. Analysis was carried out using COWI's proprietary in-house software NODLE.

The geometry and section properties have all been taken from the drawings and data available to us, and where the information is missing or unclear we have made certain assumptions based on interpretations of photographs and other information.

The analysis has considered the full gross cross section of all elements, which is appropriate to the analysis of truss structure, but in the assessment, the stresses have been calculated at the weakest net section through the rivets, making deductions for the rivet holes for elements acting in tension.

The analysis initially considered all members to be in an un-corroded condition as a starting point, with the section properties derived from the dimensions shown on the drawings. Subsequently, an overall corrosion allowance was applied, reducing the section dimensions by 15% to allow for the general loss of thickness. Higher levels of section loss are expected to occur in some localised areas but we do not have specific information on this to enable a detailed assessment. The implications of this limitation are referred to later.

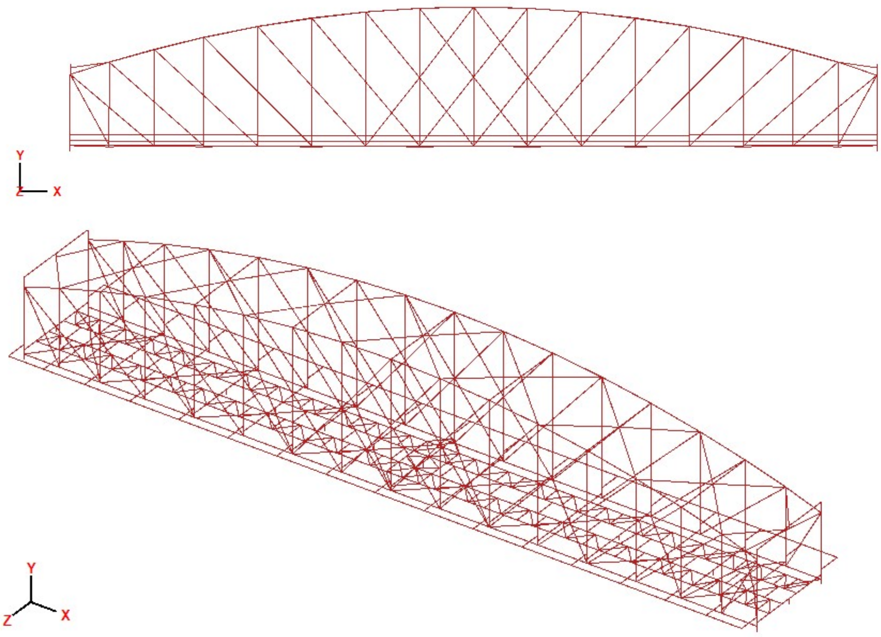


Fig. 3: 3-D computer analysis model of the structure.

The analysis has adopted the following basic modelling assumptions.

- Linear elastic behaviour.
- Geometric non-linearity accounted for but had little effect.
- Geometry defined as on the drawings without local or global imperfections.
- All sections modelled with gross sectional properties with elements lying generally on the centroidal axis of the member.
- Where eccentric connections occur, such as where the plan bracing is attached to the flange and not the centroid of end portal crossbeam, the eccentricity of the connection has been modelled.
- Single span, simply supported structure, with roller bearings free to move.
- Large rivet group connections modelled as moment carrying joints but assumed to be capable of redistributing moment and becoming effectively pinned connections at ULS if necessary.

4.3.1 Structural verification method

All load factors and loadings adopted in this assessment have been estimated from the limited information currently available. Following a more detailed condition survey of the bridge in due course, it is recommended that these are re-reviewed with the local railway authority to establish updated values for the final assessment and strengthening design.

The railway loading in the analysis has assumed loading model LM71, as illustrated in Fig. 4. An alpha factor (α) equal to 1.0 has been applied to the loads. The α factor is usually greater than one for the design of a new bridge, but a value of 1.0 is considered appropriate for the assessment of an existing bridge.

The load cases that have been considered are:

- Dead load + Superimposed deadload + 2 LM71 trains, one on each track, with the point loads located at midspan.
- Dead load + Superimposed deadload + 1 LM71 train on one track with the point loads located at midspan.
- Dead load + Superimposed deadload + 2 LM71 trains, one from the abutment to midspan on one track, and one from midspan to the other abutment on the opposite track.
- Dead Load + Superimposed dead load + 2 LM71 trains, one on each track, with the point loads located at midspan, and with both footways loaded with pedestrian loading.

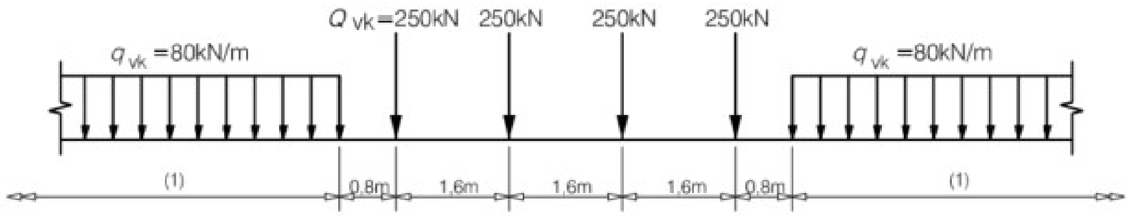


Fig. 4: Load Model 71 and characteristic values for vertical loads, EN 1991-2, Figure 6.1.

For both ULS and the fatigue verification the forces are extracted from the model for the members and imported into excel. Basic engineering principles are then used to calculate the stresses. The stresses were calculated using gross sectional properties, corroded sectional properties and net corroded sectional properties. For all sections, a general allowance of 15 % corrosion on the thickness of each plate was taken into account.

4.3.2 Partial load factors and material factors

The load and material factors assumed in the assessment are given in Tables 4.1, 4.2 and 4.3 below. Note that these values are based on the limited information known at the time of the assessment and would need to be reviewed and agreed with the railway authority in due course.

Table 4.1 – ULS Load Factors assumed in the assessment

	ULS Factor	Comments / Background Assumptions
Steel self-weight, γ_G	1.2	A γ_G value of 1.16 for steel would be a typical value in assessment codes for the assessment of an old steel bridge. Adopting the Eurocode default of 1.2 allows for contingency for weight of ancillary steelwork items, rivets and splice plates.
Other superimposed load, γ_G	1.2	Includes the weight of the timber decking and track.
Rail traffic, LM71 γ_Q	1.45	The default Eurocode γ_Q of 1.45 has been adopted for future LM71 train traffic shown in the figure below. The additional 'α' factor for LM71 loading to EN 1991-2 Clause 6.3.2 has assumed to be 1.0. Larger values would be needed if future proofing of train loading is required. The additional Dynamic factors 'Φ' have been derived from EN 1991-2 Clause 6.4.5.2
Pedestrian walkway traffic	0.54	Derived from an assumed γ_Q of 1.35 for pedestrian loading applied with a ψ_0 value of 0.4 as it will always be an accompanying action for the leading action rail traffic. Resulting ULS factor = 1.35 x 0.4 = 0.54. Pedestrian loading has been estimated using 'crowd loading' rules with a max applied load of 5.0kN/m² - using EN 1991-2 Cl. 5.3.2.1. Crowd loading is considered to be a possibility for the bridge structure on account of the location over the river in central Prague.

Table 4.2 – Partial Load Factors assumed for the fatigue checks

	Fatigue Safety factor	Comments / Background Assumptions
Fatigue Loading, γ_{Ff}	1.00	The fatigue assessment has assumed a partial load factor of for fatigue loading γ_{Ff} = 1.00. Following the guidelines adopted using the guidelines of EN 1993-2 Clause 9.3 (2). This has been combined with the worst case, which has been taken as 75% of 2 x LM71 train loading and appropriate impact factors.
Fatigue Strength γ_{MF}	**1.1**	The fatigue assessment has adopted the fatigue material factor of γ_{Mf} =1.1 used for the fatigue checks, which is the default value in the UK NA for all bridges.

Table 4.3 - Material Factors

	Material Safety factor	Comments / Background Assumptions
Steel Yield, γ_{M0}	1.05	1.05 has been assumed as an appropriate value for the 120 year old steelwork based on values in similar assessment codes and observations in photographs. The default value in the Eurocode for new-build structures is 1.0
Rivets in shear γ_{M2}	1.33	1.33 has been assumed as an appropriate value for the 120 year old rivets based on values in similar assessment codes. The default value in the Eurocode for new-build structures is 1.25

4.3.3 Strength of Existing Materials

In the absence of any material testing data being made available, the following values in Table 4.4 below have been assumed for material strengths. These figures are taken from UK railway authority document NR/GN/CIV/025 and are typical values to be expected from early twentieth century steels, but would need to be reviewed and correlated against any actual material test data if this is available.

Table 4.4 – Assumed strength of existing plates and rivets

	Min. Yield Strength "R_{eH}"	Min. Tensile Strength "R_m"
Existing Steel Plates (N.B an estimated corrosion loss of 15% has been taken from all elements based on limited photographs. This will need to be confirmed more accurately in due course following a detailed corrosion survey).	205 MPa	370 MPa
Steel Rivets	300 MPa	430 MPa

4.3.4 ULS verification

Checking the structural members at the Ultimate Limit State (ULS) reveals that the main truss members are acceptable, with strength utilization factors all less than 1.00. This is assuming a uniform 15% loss of section throughout due to corrosion.

The railway stringers are also found to be acceptable at ULS, but a relatively small overstress is identified in the crossbeams which form the bottom transverse member of every crossframe.

The worst case results from the ULS assessment are summarized below in Table 4.5.

Table 4.5 – Maximum ULS utilization assuming 15% corrosion allowance

Bottom Chord	Top Chord	Vertical Posts	Diagonal Bracing	Crossbeam	Stringers
0.73	0.89	0.98	0.70	**1.05**	0.65

The critical section for each member in tension is typically the net cross section through the riveted connection where a deduction has been made to allow for the rivet holes. Buckling is the governing design condition for members in compression, ie. mainly the top chord and vertical posts.

In the analysis, the riveted joints have been modelled as moment carrying connections, able to transmit bending moments across the joint. But at the ultimate limit state it has been assumed that some moment redistribution can occur such that the joints can behave effectively as pinned joints, and this has been taken into account where necessary.

Fig. 5 shows the calculated ULS utilization ratios in the truss for the load case with two LM71 trains passing at midspan plus associated full pedestrian live load.

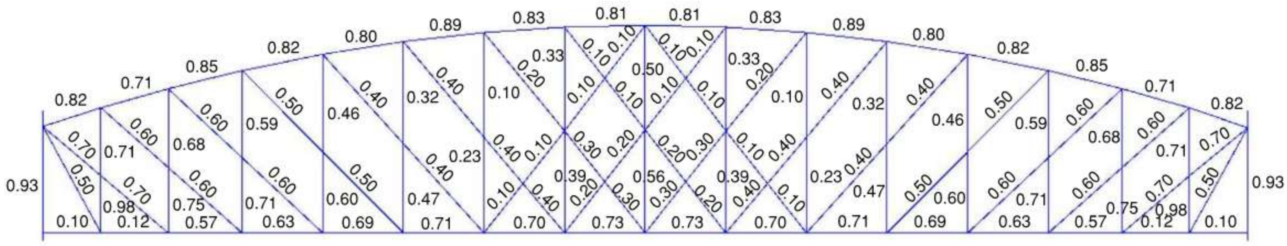


Fig. 5: ULS utilization for two LM71 trains at midspan plus pedestrian live load, with a 15% corrosion allowance.

Fig. 6shows the worst case results for the crossbeams and longitudinal rail stringers. These are governed by local loads applied from the heavy bogies of the LM71 trains, so the stress levels are the same everywhere, with slight reductions in the end bays where the transverse frame spacing reduces slightly. The crossbeams are slightly overloaded in every case, assuming a 15% loss of section due to corrosion.

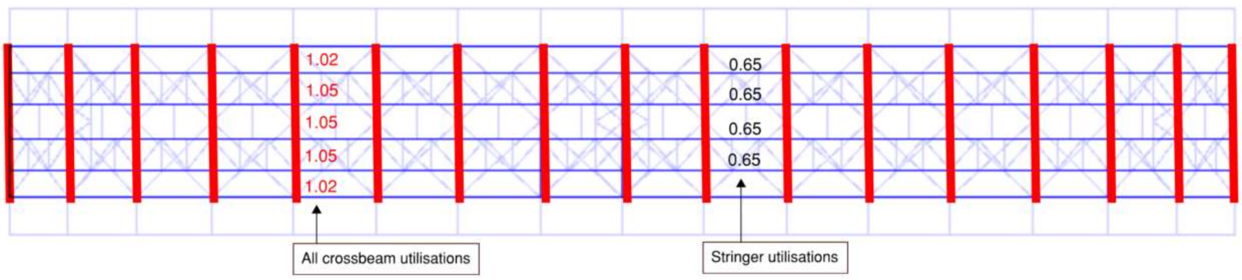


Fig. 6: Summary of ULS utilization for the crossbeams and stringers, with a 15% corrosion allowance.

A 15% loss of section due to corrosion has been allowed in the analysis. As already noted, this assessment is being carried out without the opportunity for a detailed visual and measured inspection of the bridge. We know from having seen photographs of a few isolated areas of damage that severe corrosion has caused some significant loss of section in some localized areas and that this may have resulted in greater than 15% overall loss of section in those locations. However, without further detailed knowledge of exactly where such details occur, it is not possible to say whether or not the damage is serious enough to require extensive repairs. It is possible that localized repair is all that is required, and we recommend that each defect is considered on its merits and suitable repair methods devised accordingly. In those damaged areas, locally higher stresses in the net section will be experienced, thus increasing the utilization ratios quoted here. For this reason, the conclusions from this assessment must be treated as preliminary until such time that a thorough detailed corrosion survey can be carried out.

4.3.5 Fatigue assessment

Fatigue damage on a steel railway bridge is the gradual development of cracks within the steel elements as a consequence of repetitive train loading. If left unchecked, the growing cracks can become large enough to induce a complete fracture through a truss element. On a steel truss bridge this is likely to happen when a train is passing over the bridge and the implications can be very serious and even catastrophic.

The usual way of assessing fatigue damage on a steel bridge is to carry out a 'Miner Summation', which needs an assessment of the likely maximum loading the bridge has seen throughout its history and the number of times that these have occurred.

On account of the uncertainty in accurately predicting the 120 year load history on the bridge to date, it is considered that the results from a detailed fatigue damage history and Miner Summation would be extremely difficult to justify with confidence.

A preferable approach for justifying the fatigue performance of the existing bridge is to review train loading stresses against the 'endurance limit' for the fatigue details in question. The endurance limit is the fatigue stress, below which a detail can withstand an infinite number of loading cycles. If the maximum train loading stress ranges are below the endurance limit, then this gives confidence that no fatigue damage has been accrued in the steel bridge to date. This assessment has conservatively checked the stress ranges due to the passage of two LM71 trains, at 75% of full loading, against the threshold stress levels defined below.

In the terms of Eurocode 1993-1-9, the endurance limit that can be used in this instance is the 'constant amplitude fatigue limit' – $\Delta\sigma_D$ from EN 1993-1-9 Clause 7.1(2). Provided the maximum train loading has always been below this value, then this term can be taken as the effective endurance limit.

If the assessment finds that maximum trains loadings have exceeded $\Delta\sigma_D$, then a full Miner summation calculation is required and the fatigue endurance limit reduces to the EN 1993-1-9 Clause 7.1 'cut-off limit' $\Delta\sigma_L$ on account of the weaker strength after the damage at the higher stress cycles.

Table 4.6 – Allowable fatigue stress (fatigue damage threshold) used in the assessment

	Detail Category	**Endurance limit = $\Delta\sigma_D$ /γMF**
All steelwork containing rivet holes	90 MPa	66 / 1.35 = 49 MPa

Table 4.7 summarises the worst case results from the fatigue assessment, and Fig. 7 illustrates the worst utlizations derived from the fatigue analysis for the main truss members.

Table 4.7 – Maximum fatigue utilization assuming 15% corrosion allowance

Bottom Chord	Top Chord	Vertical Post	Diagonal bracing members		Crossbeam	Stringers
			d1/d2	All others		
0.99	0.78	0.67	1.27	0.94	1.73	1.1

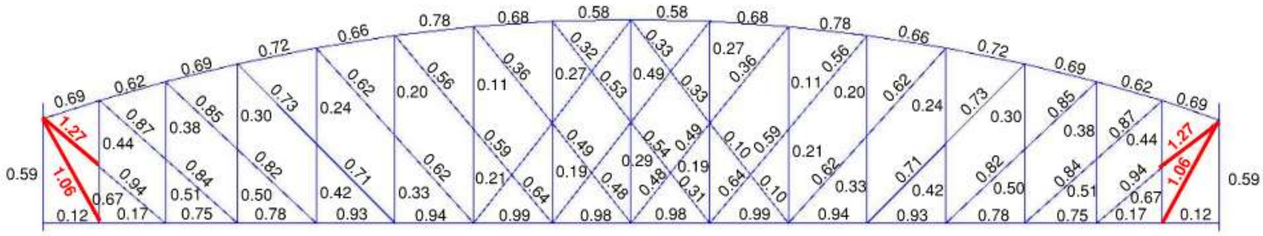


Fig. 7: Fatigue stress threshold utilizations in the main truss, assuming a 15% corrosion allowance.

This shows that under the cyclic loading model assumed in the analysis, the diagonal bracing members at the end of the bridge experience cyclic stress levels higher than the threshold level above which some fatigue damage accumulation can occur. The critical section is the reduced net cross section in the parent plate through the rivet holes. The parent plate in these areas is covered by the riveted cover plates, so if any fatigue cracking is ocurring it may not be visible to inspection.

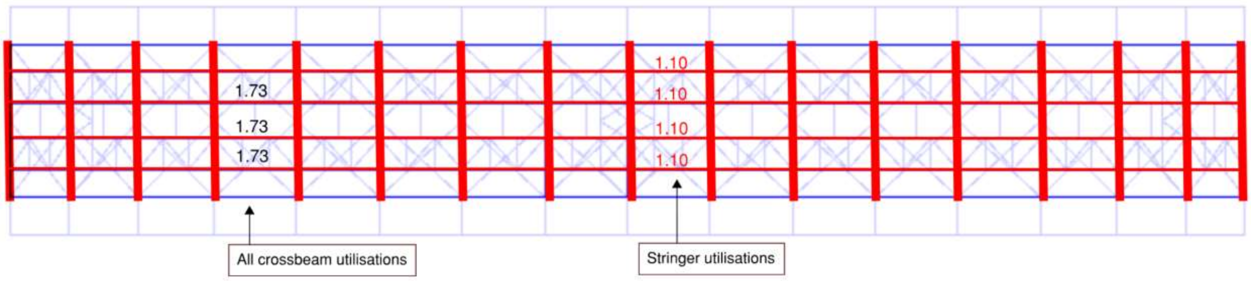


Fig. 8: Fatigue stress threshold utilizations for the crossbeams and stringers, assuming a 15% corrosion allowance.

Fig. 8 shows the worst case utilisations derived from the fatigue analysis for the crossbeams and longituidinal rail stringers. These are subject to local loading from the train axles, and are consequently bound to be the elements in the bridge most susceptible to potential fatigue damage.

This analysis reveals high levels of potential overstress in the crossbeams, indicating a high likelihood that some fatigue damage accumulation is occurring in these members. Again, as before, the worst potential damage is predicted to occur in the parent plate at the position of the rivet holes. These areas are covered by cover plates making it very difficult to inspect the parent plate to detect cracks that may be occurring there.

As previously noted, this analysis assumes a 15% loss of section everywhere due to corrosion. It is known that some local areas or more extreme corrosion and other potential damage are present in some discrete locations, resulting in higher risks of fatigue damage. Accordingly, these results must be considered as preliminary until such time that a more detailed investigation of the actual bridge condition can be carried out on site, and such local details can be repaired and/or a monitoring process instigated to control and future damage.

4.4 Assessment Findings

The main conclusions from the analysis are summarised below and discussed further in Section 7.

4.4.1 Deck Support – Cross beams and longitudinal stringers

The main railway stringers and cross beams supporting the tracks experience significantly high fluctuating fatigue stresses under railway loading. The stress levels exceed the threshold below which fatigue damage is considered unlikely in a 120 year old steel structure, so it is likely that fatigue damage is occurring. It is to be expected that close inspection would reveal that fatigue cracking is already evident in these areas.

The inference is that all the crossbeams and longitudinal railway stringers should be replaced. This can be done one bay at a time to minimise disruption to the railway, as described in Section 7 below.

4.4.2 Main Truss Elements

Calculated stresses in the main truss members are acceptable at the ultimate limit state, with utilization ratios less than 1.00 everywhere, but the diagonal bracing members nearest to the ends of each span are found to fail the fatigue assessment criteria adopted for this study.

The fatigue assessment has adopted a conservative approach because of the difficulty in evaluating the accumulated fatigue damage in the 120 year life of the bridge to date. It is possible that a more detailed fatigue assessment, coupled with close inspection of the vulnerable areas on site, will produce a more acceptable result leading to a conclusion that the affected diagonal bracing members need not be strengthened or replaced.

It is therefore recommended that the fatigue analysis is repeated in more detail, using more accurate assumptions for cross-section corrosion based on site survey and fatigue loads appropriate for the past-history of the bridge. In particular, there should be a focus on those localised areas where more severe isolated corrosion has cause local loss of section that may result in higher fatigue stress ranges.

LM71 loads (equivalent to UIC 71) is considered to be a conservative fatigue loading for assessing past fatigue damage, and the further study should review the published fatigue loading criteria for relevance to this particular bridge and if necessary, derive a fatigue load assessment basis that is more appropriate to this structure

If the bridge is reconfigured to operate in future with only a single rail track instead of two, as discussed as one of the potential options in Section 6 below, then this significantly reduces the risk of any future ongoing fatigue damage in the bridge.

5 Separation of Pedestrian and Railway Crossing

5.1 Background and Challenges

The City's footpaths are attached to the railway bridge structure and as such to some degree depending on the railway owner / operator. Challenges for the railway that concern the bridge structure are affecting the footpaths, too. Currently, the main challenges on the railway side seem to be the lack of maintenance causing the deficiencies described in the previous chapter and requiring rehabilitation with significant costs, the need for a third track, the new Vyšehradský station and the clearance, see Fig. 9.

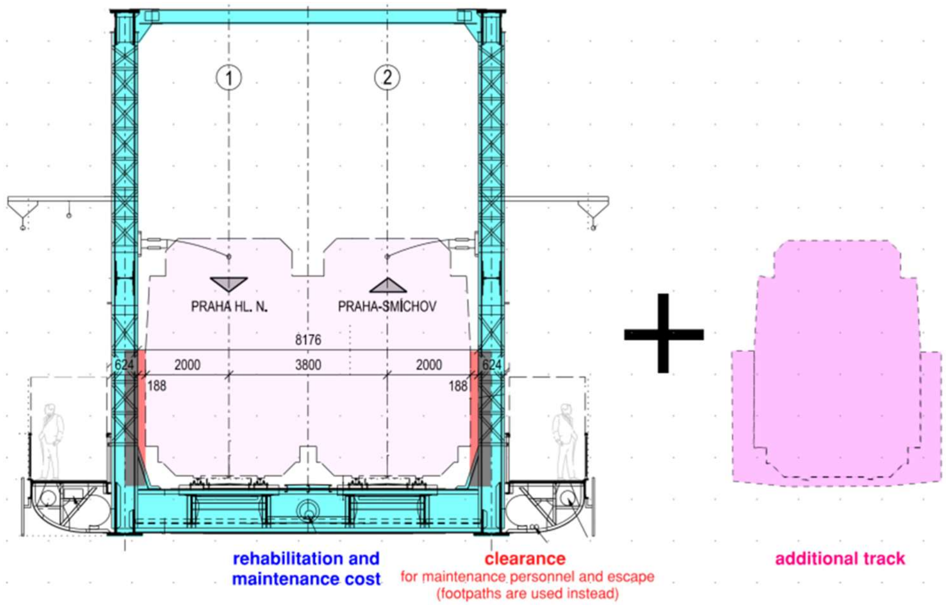


Fig. 9: Railway challenges.

The clearance requirements are linked to the possibilities to pass the lateral truss girder. If an opening of b x h = 1 m x 2 m is provided at least every 10 m along the track, the clearance can be less than without (clearance profile VMP2,5 instead of VMP3,0 or 3,5). Fig. 10 shows the current situation for half of a span. The green rectangles depict openings of standard size (1 m x 2 m) and the orange rectangles openings with substandard height or width. With a spacing of the posts of 4.8 m the number of openings over the length of the bridge is significantly higher than the minimum required. Currently, the handrail between footpath and longitudinal truss is an obstacle for an easy evacuation over the footpaths. For service personnel, the openings are deep enough to serve a recess.

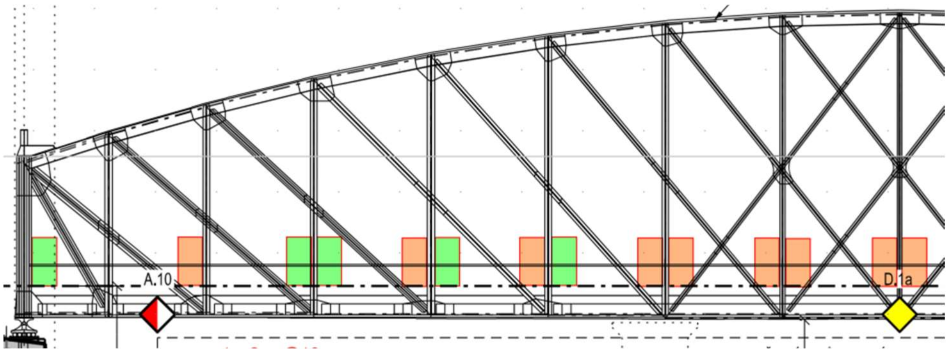


Fig. 10: Openings through side truss.

The high heritage value of the bridge significantly raises the limit for what cost and constraints on the railway operations are acceptable for a rehabilitation compared to a replacement with a new bridge. Nevertheless, it will not be possible to fit three tracks on the existing bridge and a new station will also require substantial modifications of the historic structure on the Vyšehradský side. Therefore, this chapter investigates the solution space on a concept level and gives an overview of possibilities and their advantages and disadvantages.

5.2 Strategic Options

The strategic options are driven by the requirements of the railway operation and the railway station and the boundary conditions formed by the tramways, by the road traffic and by the neighbouring buildings, especially the heritage sites (historic station, toll house, cubistic villa), as well as by the subterrain river Botic. Furthermore, there is a strategic choice in combining railway traffic and cycle and pedestrian traffic on the same bridge or not – they have different characteristics and owners.

Apart from the immediate technical preferences, requirements and boundary conditions from each traffic type and owner, there is an overlying public interest in the city scape and in the historic fabric justifying the UNESCO World Heritage status. This public interest should weigh heavily in the appreciation of the different options described hereafter.

5.3 Option 2 + 1

This option consists of rehabilitating the existing bridge for continued use as up to this day with two railway tracks and footpaths on either side and an additional new bridge carrying the additional third railway track, see Fig. 11.

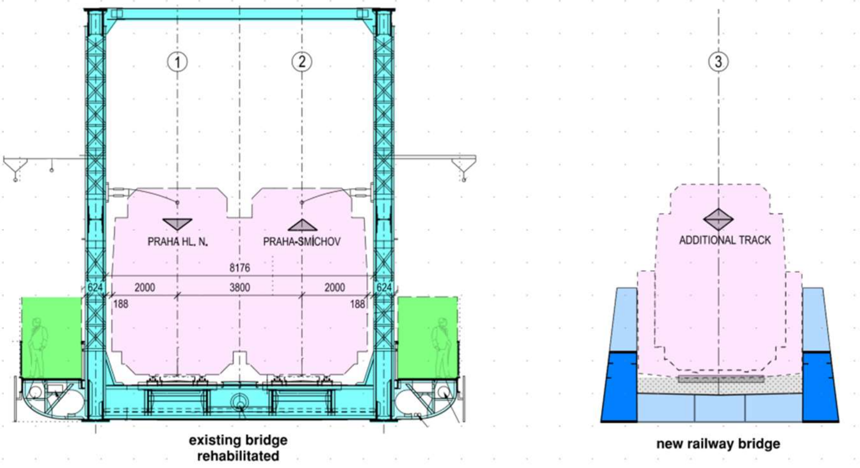


Fig. 11: Option 2 + 1 a. Cross-section.

As a variant it can be imagined to widen one of the footpaths so it also can accommodate cycle traffic by relocating it onto the new bridge as illustrated in Fig. 12. The existing footpath between bridges for service personnel can then be used exclusively for maintenance access and services.

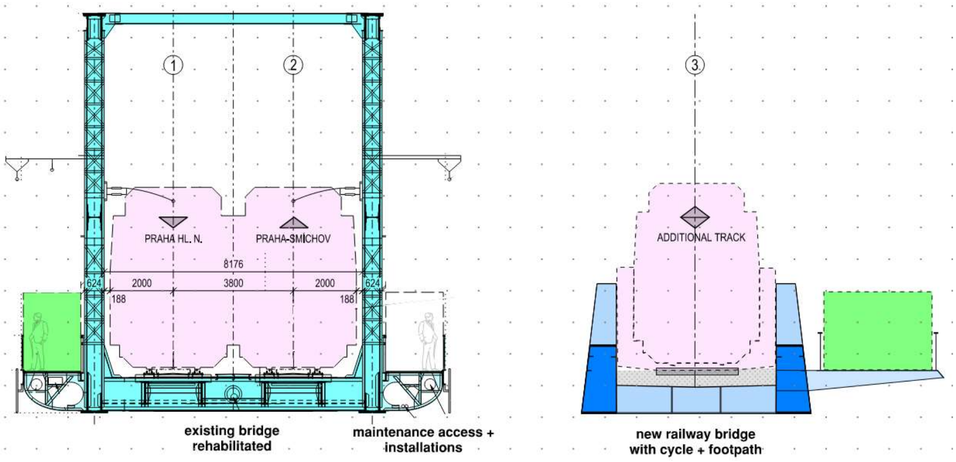


Fig. 12: Option 2 + 1 b. Cross-section.

Locating the new bridge on the north side of the existing bridge, as illustrated in Fig. 13, seems more advantageous than on the south because of the already tight radii of the railway tracks in front of the historic station and because of the Botic river in the south of the existing approach bridge.

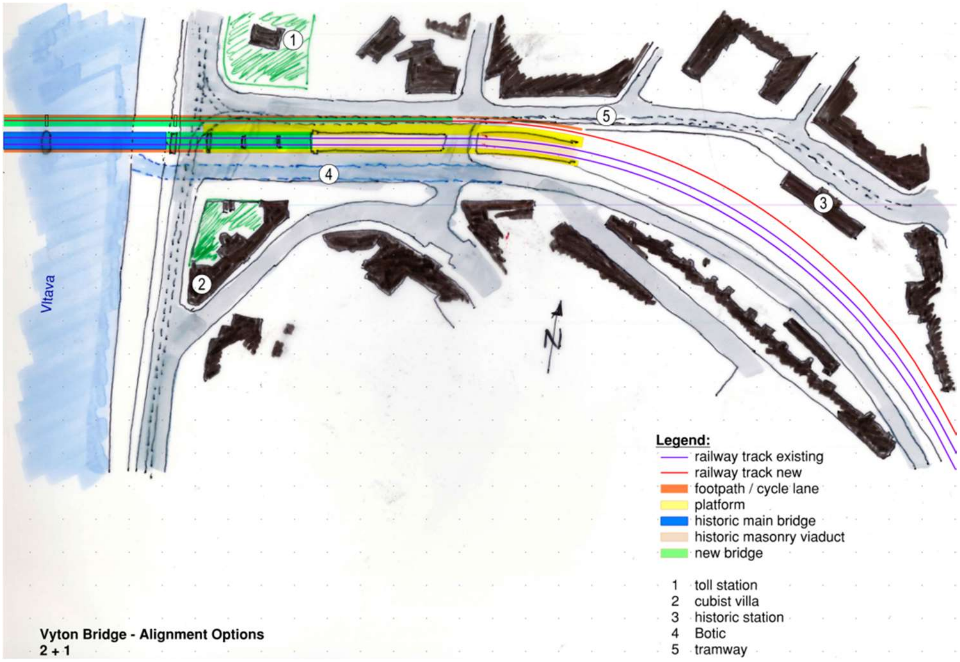


Fig. 13: Option 2 + 1. Alignment.

This option has been studied in more detail in a feasibility study in 2019. The feasibility study demonstrated the advantages of this option in terms of heritage value, city scape and traffic relations of the area.

To allow for a larger clearance (both between track and longitudinal truss and between the tracks themselves) and to thereby improve the operational safety of the railway it would be necessary to replace the existing crossbeams and railway beams as well as the top wind bracing, see Fig. 14. Large parts of the portal frames at the span ends will also have to be replaced. Contrary to the rehabilitation discussed in the next chapter, this cannot be done in-situ under traffic. The lateral truss girders need to be entirely separated from the connecting horizontal elements. This can be done either by lifting a span off its position and float it to an installation area nearby or by the installation of a working platform under the existing bridge. Either way, the moved bridge superstructure and in particular the supports will not align with the piers as originally. This will require a modification of the piers or at least of the pier top. This solution has been chosen recently as a compromise between heritage and railway considerations for the Weissbad crossing, a riveted truss bridge of similar age and condition but with a far smaller scale.

Works in-situ with a working platform can either be conceived using the working platform as stabilising element for the bridge elements (e.g. lateral truss girders under wind loads) or combining the bridge elements with additional temporary elements to spatial truss structures that carry the working platform. The jacking operation for moving the lateral truss girders apart is challenging and even more so for the spatial truss solution..

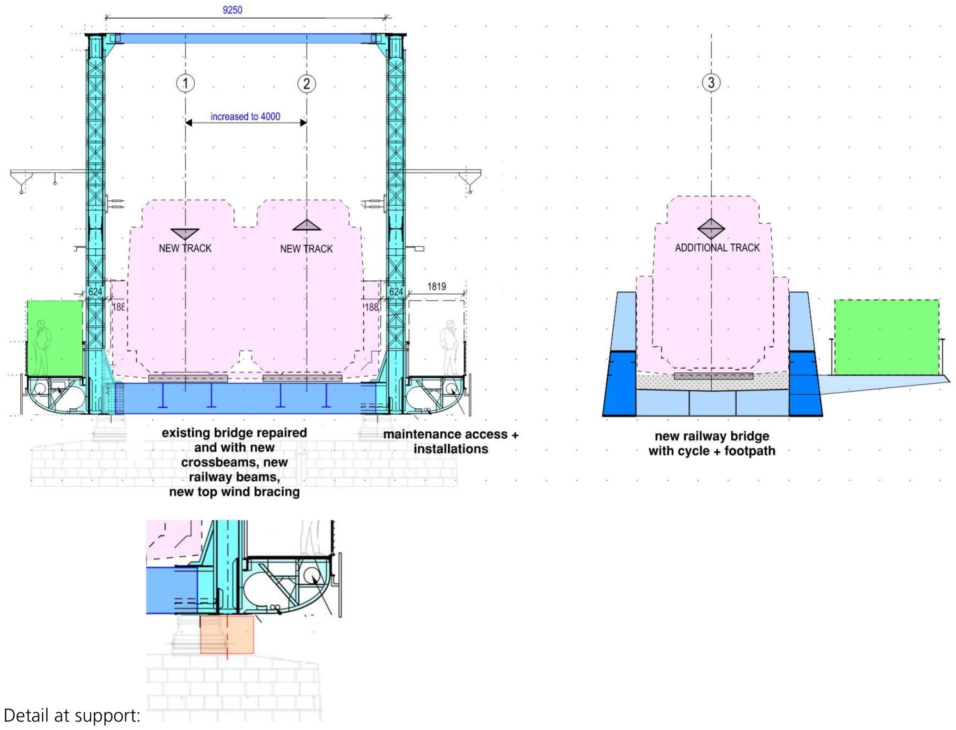


Fig. 14: Option 2 + 1c. Cross-section and detail at support.

Replacing the entire bridge deck, the wind bracing and the portal frames is a massive intervention on the historical substance. Historically, the elements that are to be replaced in this option are also the elements, that have been worked on already (interventions 1969) and that, at least for the longitudinal stringers under the rails, can be viewed as wear part.

While replacing the bridge deck opens the possibility for a fatigue-optimised redesign, widening also means increasing the dead load and the bending moment in the cross beam. Care has to be taken that an increased bending moment in the crossbeam does not increase the transverse bending in the lateral truss girders on top of the generally increased sectional forces due to increased dead load. In view of the condition of the bridge, this option entails the largest/most/deepest interventions on the lateral trusses. It does not allow traffic on the bridge to be continued. Service ducts likely need temporary solutions or permanent relocation. It technically the most challenging option for the bridge part and one with an elevated risk on the cost side. On the other hand, taking the bridge out of service for a while also opens the opportunity to be more progressive on the extent of the rehabilitation and strengthening works, thereby opening up for higher-durability-solutions.

5.4 Option 1 + 2

This option is similar to the previous option in the combination of the refurbished existing bridge and an additional new bridge. However, here, the existing bridge is only to carry 1 railway track while the new bridge carries 2 tracks.

Due to the width of the new bridge, it seems advisable to relocate the inner side footpath to the outer side of the new bridge, as illustrated in Fig. 15. On the existing bridge, the railway track is moved to the centre. The existing tracks can be retained (out of order) to ease future maintenance work (e.g. repainting the side truss structure).

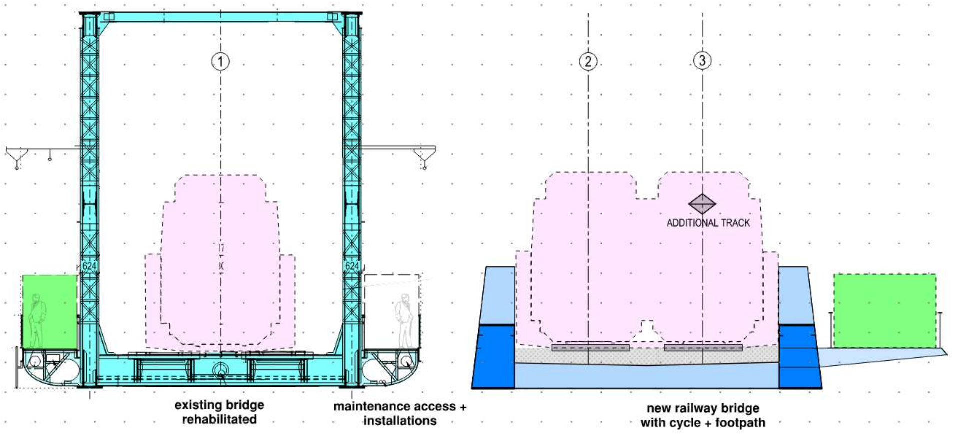


Fig. 15: Option 1+ 2. Cross-section.

With the same arguments as for Option 2 + 1 locating the new bridge to the north of the existing seems the apparent choice (Variant a in Fig. 16), but the station comes rather close to the toll house.

Switching the positions of the new and the old bridge (Variant b in Fig. 16) would not only ease this situation but it would also allow the current railway operation to continue after a short interruption for side launching while refurbishing the historic bridge in a comfortable situation without traffic. The new bridge girder would be installed to the south of the final position on temporary piers.

Positioning the new bridge south of the existing bridge (Variant c in Fig. 16)has the advantage that the bridge and a large part of the station can be built without interfering with the railway operations. The width of the two tracks renders spanning across the Botic river a plausible structural solution. However, the track curve radii might be too tight, especially considering the curved section in the new station, the clearance of the road might not be sufficient (area marked with a red circle) and the station moves rather close to the cubist villa.

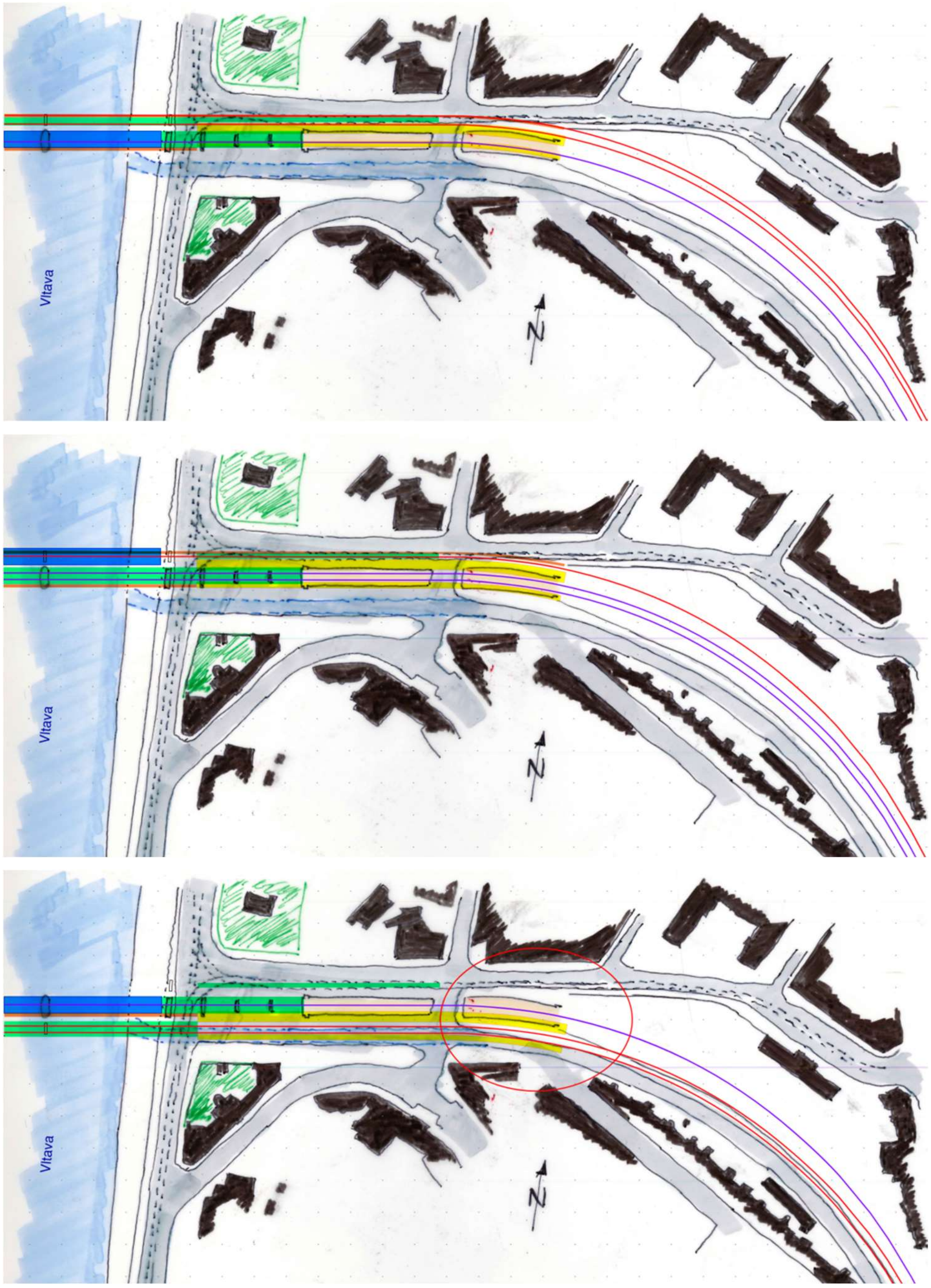


Fig. 16: Option 1 + 2. Alignment Variants a (top), b (middle) c (bottom).

The Option 1 + 2 is attractive, because the future traffic reduction allows to reduce the amount of structural steel repair work on the existing bridge (smaller ultimate load and far smaller fatigue load and cycles). Variant b seems to be the likely choice, especially, because it would allow a slight raise of the track level, thereby easing the clearance situation of the tramway and the road traffic under the bridges along the Vltava.

5.5 Option 3

This option consists of replacing the existing bridge entirely and building a new bridge with 3 railway tracks or a new bridge with 2 tracks plus a new bridge with 1 track. It has been suggested by the SUDOP and others on the grounds of cost and simplicity. The pedestrian traffic might be kept up by means of an attached solution as with the existing bridge or discontinued on the ground that there will be a station on either side of the river. The City of Prague would likely have to consider building a separate cycle- and footbridge. Technically, this option is the least complicated and future decisions would be simplified without shared ownership. However, it completely negates the heritage aspects and can therefore not be recommended.

In the same spirit as for option 2+1c it could be asked whether it might be possible, to separate the lateral truss girders and add a new structure with all three tracks in-between. Because the lateral truss girders do not have sufficient capacity, additional elements, for example additional longitudinal trusses, would have to be placed between the tracks. These new elements would alter the historic fabric to an extent where the remaining heritage value is questionable. Furthermore, such a hybrid structure would still require a lot of maintenance without the full benefits of a new structure. This option will therefore not be commented further in the present report.

5.6 Option 1 + 3

If the railway issues are solved by means of an entirely new structure of its own (one or two bridges), the existing bridge could be taken over by the City of Prague and turned into a new part of city, being a platform that hosts cycle pedestrian traffic, as well as areas for leisure, see Fig. 17.

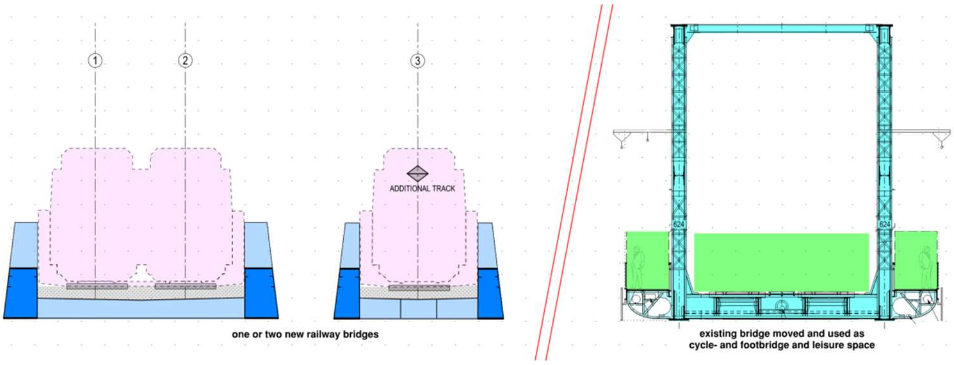


Fig. 17: Option 3 + 1. Cross-section.

Similar reuse of railway infrastructure has successfully been implemented in New York (High Line, see Fig. 18), Zürich (Lettenviadukt) and elsewhere.

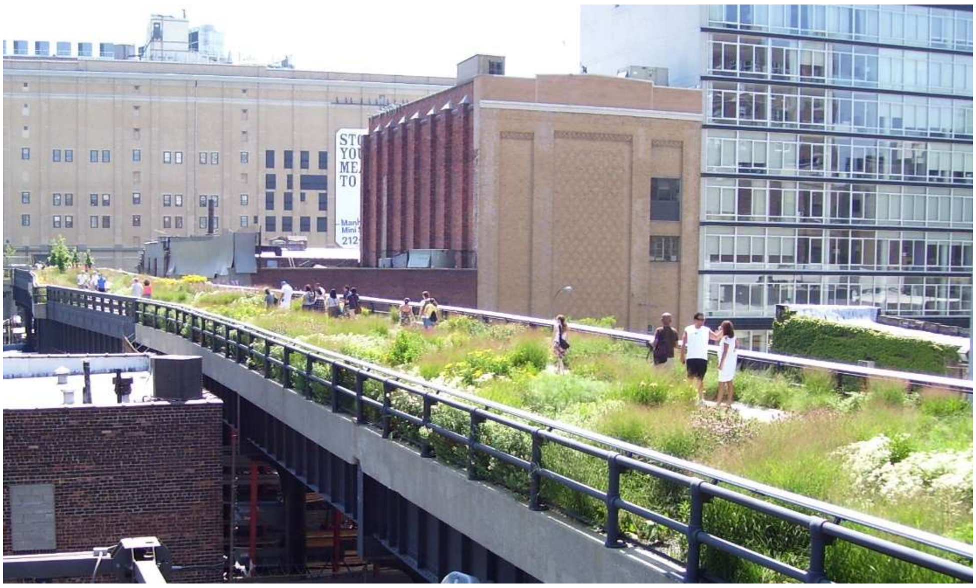
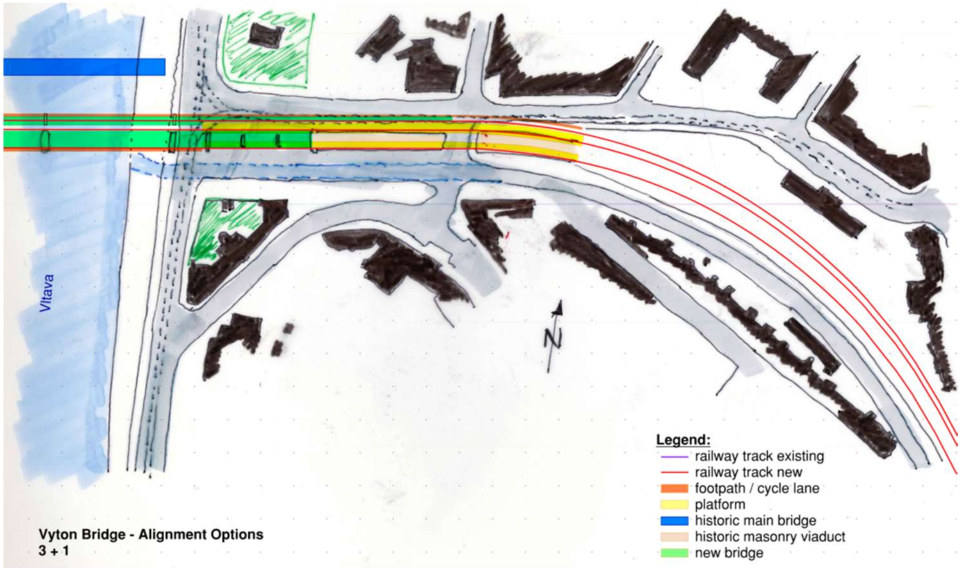


Fig. 18: High Line New York. Foto by Beyond My Ken, 2010.

While the existing bridge does not require notable repair of the steel structure in this option and while the space created and the experience of the heritage structure can be very attractive, the location in the city situation proves a challenge. Two possibilities are illustrated in Fig. 19. Variant a moves the existing bridge (girder) onto new piers on the hight of the toll station. This is an attractive option for pedestrians continuing along the river or taking the tram, but direct access to the new station can only be achieved by means of an additional bridge across the traffic node.



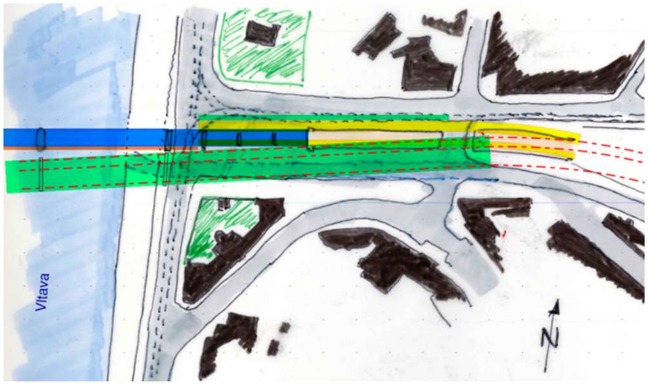


Fig. 19: Option 3 + 1. Alignment Variants a (top) and b (bottom).

Retaining the existing bridge's location requires the new railway structures to move south, thereby having to span over the Botic, moving close to the cubist villa and causing (too) tight radii at the historic station.

Locating the new structure to the north of the existing bridge would solve above issues, but cause the new bridge to block the free view from the public space on the existing bridge towards the city centre and vice versa which does not seem meaningful.

5.7 Comparison

A large number of variants have been outlined. Even though not exhaustive, they give a good idea of the possibilities and their characteristics. On a concept level, the following figure and table give an overview of their relative strengths and weaknesses.

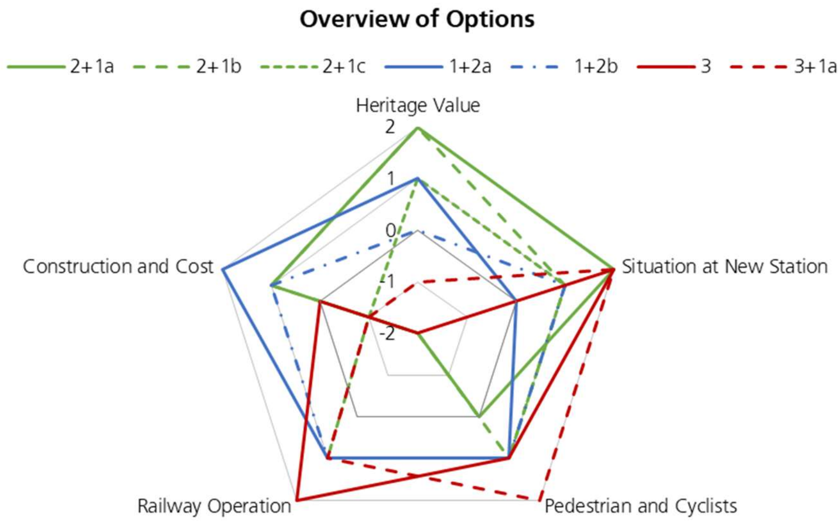


Fig. 20: Relative strengths and weaknesses of options.

Table 5.1 – Comparison of Options

	2+1a	2+1b	2+1c	1+2a	1+2b	3	3+1a
Heritage Value	++	++	+	+	0	--	-
Situation at New Station	++	+	+	0	+	++	++
Pedestrian and Cyclists	0	+	+	+	+	+	++
Railway Operation	--	--	+	+	+	++	+
Construction and Cost	+	+	-	++	+	0	-
Ranking	4	5	3	1	2	5	7

The comparison shows that there is no solution that is ideal with regard to all aspects but also that there clearly are solutions that are more adequate overall than others. When weighting "railway operation" and "heritage" with a factor 3, "situation at station" and "construction" with a factor 2 and pedestrians/cyclists with a factor 1 the 1+2-solutions score best overall.

6 Rehabilitation of Railway Bridge

6.1 Bridge Condition and Scope of Rehabilitation

The bridge is currently in poor condition, having been allowed to deteriorate through corrosion over many years. The amount of corrosion varies and there are some local details where severe corrosion has caused a localised loss of material which affects the structural integrity in the immediate vicinity of the defect. But such local defects do not necessarily affect the overall global capacity unless there is a general loss of section across the full width of the element. These local details will each need to be considered individually taking into account the particular local configuration and degree of damage. It has not been possible to assess each of these in turn because to do would require close inspection and detailed survey of the area in order to derive the appropriate repair solution. But such solutions are possible, in principle at least, without affecting the overall integrity of the bridge.

In many places corrosion has caused flanges of made-up sections to "buckle" outwards. This is not generally a problem in the bracing members which are in tension with no tendency for local buckling. But where this occurs on members in compression, it is necessary to carry out a survey to determine whether there may be a potential problem of local buckling.

Our analysis has shown that the main trusses themselves have adequate reserves of strength at the ultimate limit state, even with a 15% net loss of section due to corrosion. We do not consider it necessary to replace the majority of the truss members, unlike the conclusions of the SUDOP report, and in this respect our conclusions echo those of Professor Brühwiler.

We have however identified a potential fatigue problem in the diagonal members closest to the ends of the bridge. These members are indicated in Fig. 21. The assessment has shown that calculated fatigue stresses in these members exceed the threshold stress range above which fatigue damage can be expected to occur. However, this is based on a relatively conservative fatigue loading assumption. It is recommended that further investigation is carried out, including detailed on-site inspection to check for visible signs of fatigue damage, before concluding whether or not these elements need strengthening or replacement.

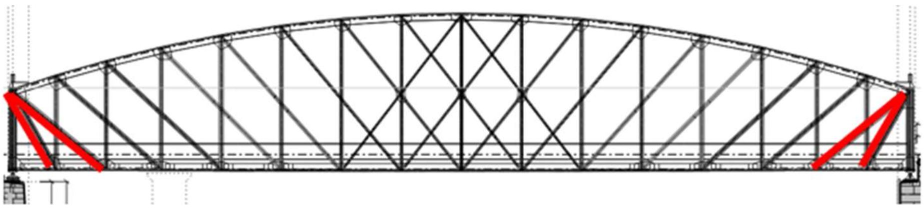


Fig. 21: Diagonal bracing members with potential fatigue damage requiring further investigation.

If it is concluded that these diagonals need replacement, it would be necessary to provide an alternative load path to carry the shear while the work is carried out. This is considered in the next section.

The biggest potential problem experienced by the bridge affects the crossbeams and longitudinal stringers supporting the railway. It is no surprise that these areas are subject to the highest potential fatigue damage due to the passage of the trains. The fluctuating local stresses occurring in certain joints between the rail stringers and the crossbeams due to the passage of trains are high enough to cause fatigue damage. The details worst affected are indicated in Fig. 22. The crossbeams are also found to experience high stresses which result in marginal overstressing in the section at ULS.

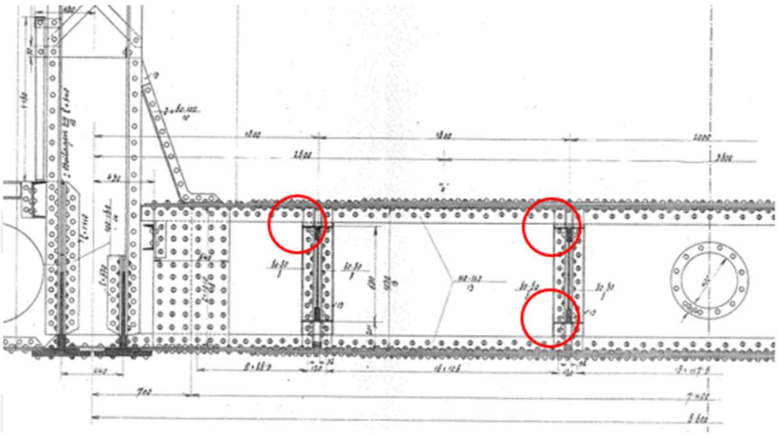


Fig. 22: Rail stringer to crossbeam joints most prone to potential fatigue damage.

Further investigation is needed to ascertain the level of fatigue damage that may already have occurred and to establish the best solution for their repair or replacement. It is also important to establish more accurately what loss of section due to corrosion should be applied. Nevertheless, is seems probable that some or all of the crossbeams and stringers will need to be replaced and an outline of the replacement method is given in the next section. Replacement is likely to be a much quicker, simpler, less disruptive and more durable solution than attempting to repair and strengthen the existing members in-situ.

The Railway Bridge is an important feature of the city scape and a valuable part of the Prague city centre UNESCO World Heritage Site. It is essential that all repairs and replacement works are carried out with sensitivity to the historical character and importance of the structure. The repair proposals therefore strive to retain as much of the original structure and detailing as reasonably possible without exorbitant cost.

If the decision is made to change the railway configuration and only run a single track on the bridge, partly because of the sub-standard separation distance between the two tracks as noted above, then the structural rehabilitation work described below is unlikely to be necessary, and the work will be limited mainly to restoring the paintwork and preventing further corrosion damage.

6.2 Methods of Rehabilitation

The riveted nature of the structure means that it will be quickest to simply cut through the existing sections to be removed and prepare the cut ends with new holes drilled to accept either a bolted or new riveted connection. If it is considered necessary for heritage reasons to use riveted connections this can be done, but a bolted solution would be much quicker, cheaper and safer to install.

6.2.1 Replacing the crossbeams and rail stringers

The longitudinal railway stringers and the crossbeams can be replaced by working on one bay at a time during night time railway possessions. The crossbeams will be cut as shown in Fig. 23, and the railway stringers will be cut midway between adjacent crossbeams so that a complete section, typically 4.8m long and about 6.5m wide, could be removed and replaced with a new piece of identical shape and size.

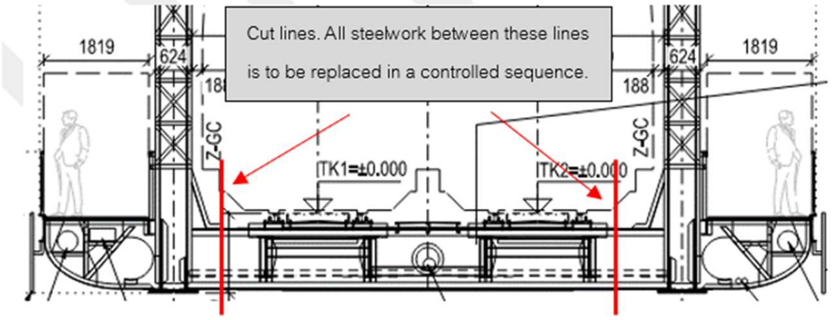
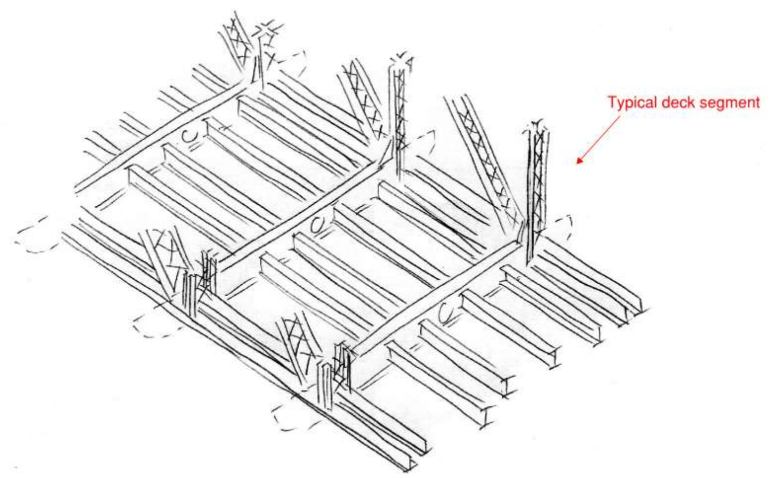


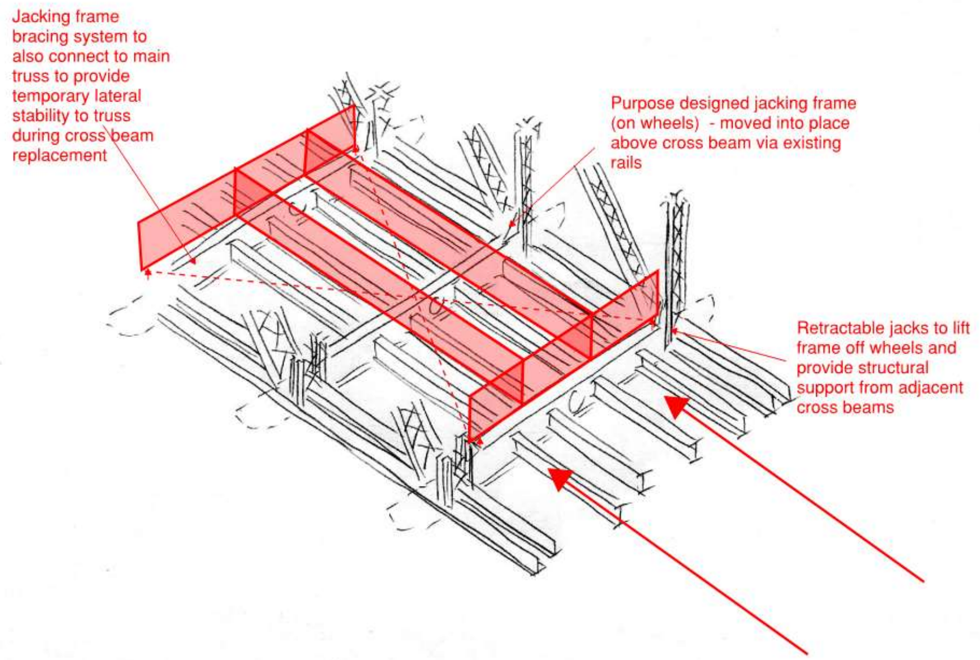
Fig. 23: Approximate positions of cut lines for crossbeams and rail stringer replacement.

The suggested strategy will be to incrementally replace the cross girders and stringers using river access and a purpose-designed rail-mounted jacking frame. The proposed concept methodology is illustrated on Fig. 24 below and assumes that segments of the deck can be replaced incrementally during a series of regular railway possessions.

This kind of procedure has been used in the past for replacing bridge decks and can be done whenever the principal load carrying structure (in this case the main trusses) is not affected and can remain in place. Previous examples include the complete replacement of the deck girder of the Lion's Gate bridge in Vancouver and the Angus MacDonald bridge in Halifax. In those cases the principal load carrying structure was a suspension cable system, but the principle remains the same here.



POTENTIAL CONCEPT FOR DECK REPLACEMENT - Sheet 1 of 3



POTENTIAL CONCEPT FOR DECK REPLACEMENT - Sheet 2 of 3

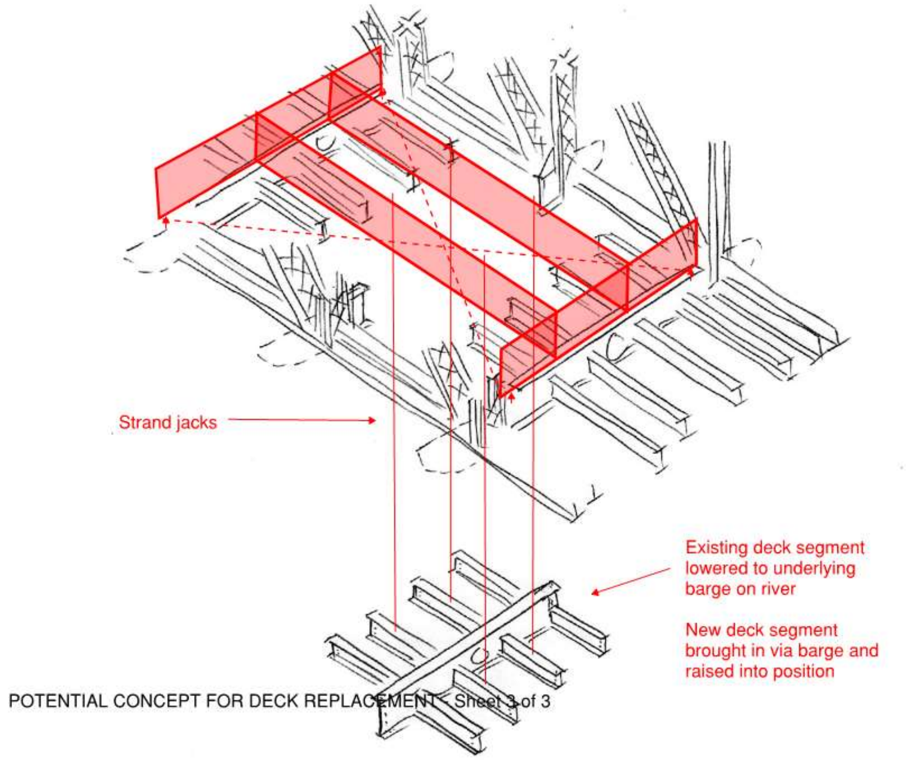


Fig. 24: Sketch illustrating the proposed methodology for replacing the deck structure.

It may be possible to carry out this work without replacing the rails themselves. The rails would be simply unclipped from the sleepers so that the old steel deck section can be lowered onto the barge below. They will be able to span across the 4.8m gap without difficulty. Then when the new steel section is lifted up from the barge and bolted into place, complete with sleepers already attached, the rails are simply clipped back on, ready for rail traffic the following morning.

It would be possible to work on multiple workfronts, perhaps one in each span, in order to complete the works in a short timescale.

Such procedures require meticulous planning and well-trained teams working efficiently in order to complete each cycle within a single night time possession, but it is our experience that they work well and can be done with minimal disruption to the railway. Obviously if longer duration railway possessions are allowed, such as over the Christmas period for example, then longer lengths of bridge can be replaced in a single possession.

A development of this replacement procedure which should be considered further at the next stage would be to incorporate a lightweight UHPC deck slab in the replacement deck structure, similar to that proposed by Professor Brühwiler. This could help to provide a longer life for the bridge in future by reducing the ongoing potential for future fatigue damage.

6.2.2 Replacing the main truss end diagonals

If necessary to replace the diagonal bracing members at the ends of the span, it will be necessary to provide an alternative load path while the replacement is carried out. Again, the actual replacement of each member would be done during a short railway possession, although the installation of the temporary works and preparation for the replacement would need to be done in advance.

The principle of the member replacement is as follows. A temporary steel bracket would be attached to the node at the top of the end post and another to the bottom chord under the bottom end of the bracing member to be replaced. Then high strength Macalloy Bars or tendons will be attached between the brackets and stressed to take the tension out of the bracing member. Then during the railway possession, the old bracing member would be removed and a new one installed. This is illustrated in Fig. 25.

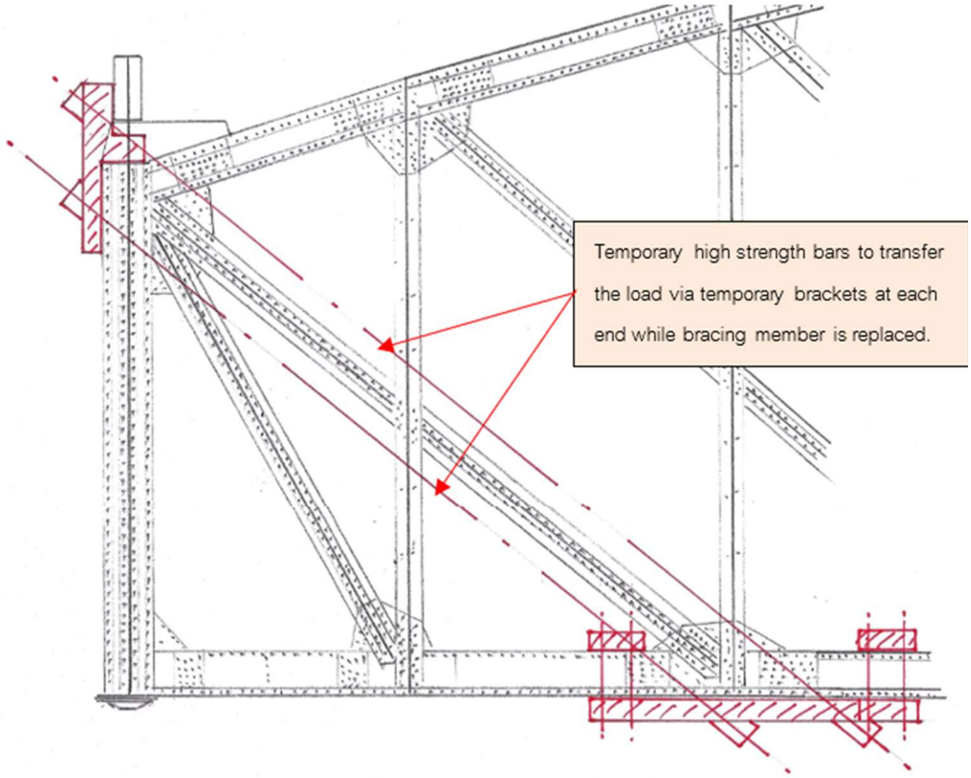


Fig. 25: Sketch illustrating the proposed methodology for replacing the end bracing members.

It is possible that part of the end node may need to be replaced as well as the bracing member itself. This requires further investigation based on detailed inspection to determine whether or not this is necessary. If it is, then the brackets would need to be designed to enable this to happen, and the replacement process would be more complex and time consuming, possibly requiring an extended railway possession.

6.2.3 Replacing main truss posts

How to replace entire diagonals has been outlined above. According to the analysis, no replacement of posts is required. However, it could be done in a similar manner as for the diagonals: introducing two temporary props alongside of the post, ensure load transfer to the temporary props by means of hydraulic jacks, removal of old post and placing of new post, bolting or riveting new elements to existing structure, releasing jacks to ensure load transfer, removal of temporary props.

6.2.4 Replacing main truss member components

Local corrosion might lead to a local capacity reduction that requires remedial measures. As the truss members are composed of multiple components, see examples in Fig. 26, it will often be sufficient to apply measures on a specific component rather than on the entire member. As such it will often be enough to stabilise the remaining components by temporary measures while replacing or strengthening the component or sections of it.

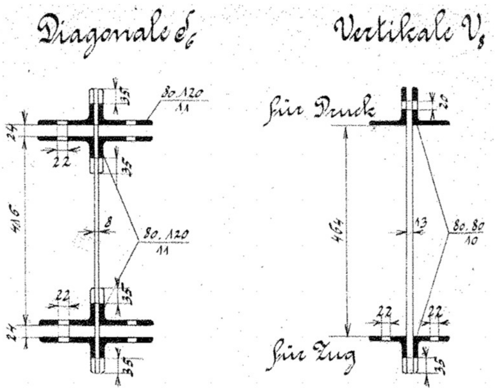


Fig. 26: Typical built-up sections of main truss diagonal (left) and post (right).

Replacing of a section of a component of riveted bridges is a procedure that is done frequently throughout the world. From a heritage point of view an important question is whether rivets are replaced by fitting bolts or rivets. While the traditional knowhow of riveting might be lost in many parts of the world, automatic machines have been developed recently, such that riveting is a real possibility again. It is to be noted though that riveting is substantially more costly.

Strengthening of a component can often be done efficiently by the addition of plated steel elements. Again, the main challenge is the bolting or riveting of the new element to the existing components. For the short-term temporary state where the existing rivets are removed temporary stabilising measures as described previously can be required.

Fig. 27: Example of strengthening by adding component using a riveting device (Foto: F.Looser).

6.2.5 Repainting

The bridge needs to be thoroughly cleaned of rust and repainted. There are areas where corrosion products have accumulated and these need to be cleared out and the bare metal surfaces cleaned and properly prepared to receive a new paint treatment. In some places, such as the gaps between flanges of built-up sections where corrosion has been occurring, it may be necessary to inject inaccessible voids with a suitable resin and then paint over to prevent future water ingress.

This work will require the structure to be partially or even fully shrouded to enable the work to be carried out within a controlled and enclosed environment. This would also prevent unwanted materials from entering the river and polluting the water.

It is likely that some train traffic controls will be needed during this work. When working on the inside faces of one of the trusses, for example, the adjacent track will probably need to be closed for safety reasons. Thus, it is expected that work on the trusses would continue one side at a time, and when completed on the first side work can move to the other side. This can be quite disruptive of rail services, so the work may need to be delayed until the new bridge is constructed alongside so that at least two tracks are available for use across the river at all times.

6.3 Condition after Rehabilitation

On completion of the rehabilitation works, the railway bridge will be able to carry trains on both tracks and the footpaths will once again be supported by a structure that can be considered reliable for many years to come.

There remains the question of the sub-standard railway clearance envelope which may dictate reduction to a single track, with the proposed new bridge alongside being designed to carry two tracks and not just one. This may prove to be the most economic solution and is worthy of further investigation. A new twin track railway bridge will not cost twice as much as a single track bridge, and there should be a considerable saving in the strengthening requirements for the existing bridge.

As noted above, this assessment has made certain assumptions regarding the bridge condition and the amount of fatigue damage that may or may not have accumulated over the years. We believe that those assumptions are safe-sided, but it is important that further detailed inspection and investigation is carried out to establish (a) whether the assessment has been too conservative and in fact there is no need to replace or strengthen any sections, or (b) whether the bridge has already suffered more damage than has been assumed and that further remedial action is in fact required.

7 Conclusion and Recommendations

7.1 Summary and Conclusions

The future of the footbridges V-025 and V-026 depends upon the continued integrity of the railway bridge to which they are attached. Therefore, an assessment of the railway bridge has been carried out, taking into account its long history and important heritage value, in order to:

- establish whether the bridge can continue to safely carry railway traffic in future, given its current condition,
- determine what rehabilitation measures may be necessary, if any, and
- indicate how, in principle, such rehabilitation measures might be carried out.

It has not been possible to carry out independent condition surveys or inspections of the bridge, so the assessment has relied upon photographs and other information that has been provided. Assumptions have been made about the levels of corrosion and the general bridge condition, but it is necessary to carry out further inspections to check those localised areas where excessive corrosion has caused isolated loss of section which requires localised remedial work. It is important to note, therefore, that further investigation would be necessary, including detailed inspection and surveys of the existing structure, before our conclusions and recommendations could be implemented.

The bridge has been assessed for compliance with the Ultimate Limit State loading and capacity criteria of the relevant structural Eurocodes, and found to comply with adequate margins for safety everywhere except as follows:

- The crossbeams that carry the longitudinal railway stringers are found to be marginally overstressed, with a maximum utilization factor of 1.05. This implies an 5% excess of load over capacity, based upon the net section at one of the riveted connections, making allowance for the deduction in section due to the rivet holes.

A fatigue assessment has also been carried out to determine whether fatigue damage may be occurring in critical details due to repeated passage of trains on the bridge. A conservative approach has been adopted, comparing the calculated stress levels with the threshold stress level below which fatigue crack propagation is assumed not to occur. If the stress levels remain below this threshold then fatigue damage is considered unlikely. This approach avoids the need to derive the historical stress ranges and accumulated potential fatigue damage over the 120 year life of the bridge using a Miners Summation method, which would have been difficult given the absence of historical data. The assessment has found that the bridge is unlikely to be at risk of suffering fatigue damage except in the following locations:

- the longitudinal railway stringers, which typically have a utilization of 1.10, implying that the applied stress is 10% higher than the no-damage threshold stress,

- the crossbeams which carry the longitudinal railway stringers, which typically have a utilization of 1.73, and
- the diagonal bracing members in the end bay of the truss nearest the supports, which have a maximum calculated utilization of 1.27.

These findings suggest that some fatigue damage of these elements is likely, demanding that some action is taken to investigate further and possibly replace or strengthen the affected members.

In view of the identified levels of overstress in the fatigue assessment, methods for replacing the affected components have been considered. An outline proposal has been presented for the incremental replacement of the entire deck structure comprising the crossbeams and railway stringers, and the replacement of the affected end bay diagonals in the main trusses. These methods are based on adopting the following basic principles:

- respect for the important heritage nature of the historic railway bridge, replacing like-for-like components where possible,
- minimising disruption to the railway, enabling component replacement work to be carried out during short railway possessions where possible, and
- using established technologies for the repair and rehabilitation of such historic bridges over water.

The assessment has been based upon the assumption that the bridge continues to carry two railway tracks and the two footbridges. But some possible scenarios have also been considered whereby the bridge only carries one track in future, with one of the tracks moved onto the new bridge which is being proposed to stand alongside. This would radically reduce the imposed loading on the bridge, and also facilitate future management and maintenance of the bridge by providing more working space for safe access to the inside face of one truss while trains on the opposite track. But in view of the bridge's long history to date and the possibility that fatigue damage may already have occurred, particularly in the crossbeams, it is probable that some or all of the recommended deck replacement works would still be required.

However, another option exists which is to remove trains from the bridge entirely and place both existing tracks onto the new bridge. Then there is the opportunity to open up the bridge as a public space for pedestrians and cyclists to enjoy as a new amenity, celebrating the heritage value of the historic bridge. Apart from providing a suitable new walking surface in place of the rails and sleepers, there would be no need for any further structural rehabilitation or modification.

Whichever option is selected, the bridge will need repainting, dealing with the corrosion problem and giving it a new lease of life for many years to come. Repainting would require either reducing the use of the bridge to a single track at a time while the opposite truss is sheeted and painted, or removal of trains entirely, as with the option to convert the bridge into a pedestrian only environment.

7.2 Recommendations (next steps)

We recommend that the bridge is retained and not replaced. The historic and heritage character of the bridge is of considerable value to the city of Prague, and there is no justification for its removal because it can be retained with the rehabilitation proposals outlined in this report.

Further investigation should be conducted to verify the assumptions on which this assessment has been made, because it has not been possible to carry out detailed on-site inspections of the critical areas of the bridge in reaching our conclusions. These investigations will help to confirm that the extent of rehabilitation that is being proposed is appropriate.

If the bridge is to continue to carry rail traffic on two tracks, then we recommend that a scheme to replace the crossbeams and railway stringers should be developed in detail, along the lines suggested in this report. The scheme should also address the need to replace or strengthen the two diagonal bracing members in the end bays of each span. The precise extent and nature of those detailed proposals will depend upon the further investigation referred to above.

We recommend that serious consideration is given to the option of either removing trains from the bridge altogether and converting it into a pedestrian environment, or operating just a single rail track on the bridge. Either of these options would modify the extent and nature of the rehabilitation works required, as discussed earlier. Both of these options require additional train tracks to be carried by the new railway bridge currently being considered to be constructed alongside.

In any case, the bridge needs to be repainted, and we recommend that this should be carried out as soon as is reasonably practicable.

Zurich, 17. June 2021

Revisions

Index	Date	Change	Author
A	17.06.2021	More detailed comments on Option 2+1c. Minor adjustments in the description of clearance in Chapter 5.1.	GaAn
	05.06.2021	Final Report	IPTF/GaAn
	13.05.2021	Draft Report	IPTF/GaAn
	04.03.2021	Establishment of document	GaAn