NATIONAL REPORT OF THE CZECH REPUBLIC

NÁRODNÍ ZPRÁVA ČESKÉ REPUBLIKY

# STRUCTURAL CONCRETE IN THE CZECH REPUBLIC 2022–2025

# KONSTRUKČNÍ BETON V ČESKÉ REPUBLICE 2022–2025

7th *fib* CONGRESS
LISBON 2026

7. *fib* KONGRES
LISABON 2026

Česká betonářská společnost ČSSI
Czech Concrete Society

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## FOOTBRIDGES
## LÁVKY



## TUNNELS
## TUNELY



## BUILDINGS
## BUDOVY



## TECHNOLOGY
## TECHNOLOGIE



## PARTICIPANTS
## ORGANIZACE

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## NATIONAL REPORT OF THE CZECH REPUBLIC 7TH *fib* CONGRESS – LISBON 2026

The Czech national *fib* group is delighted again to present its national report published on the occasion of the *fib* Congress. It is traditionally prepared by the team of the Czech Concrete Society (CCS). The report represents a summary of the most important concrete structures built in the period from the last *fib* Congress until now. It covers structures completed from 2022 to 2025 built in the Czech Republic and also abroad, which were designed or executed by Czech engineers and companies. As usual, bridges make up a great majority of the presented structures. After the covid period, when construction activity was reduced, the investment in traffic infrastructure increased steadily in the last 3 years and many structures were built in the last few years. It is expected that this trend will also continue in 2026. Interesting technologies were applied and also advanced materials were used in new structures and also in the rehabilitation of existing structures.

In the report, 17 bridges, 8 footbridges and 5 other projects are presented. 4 bridges and 1 footbridge were built abroad. Czech engineers participated in large projects in Slovakia. The highway viaduct Kriváň–Mýtna is more than 4 km long (page 042–047) There are interesting semi-integral structures built by different technologies (cantilever casting, application of movable scaffolding systems and incremental launching). The viaducts with central spine box girders and long cantilevers fit very well into the beautiful landscape. An interesting construction technology was applied at the highway bridge close to Uhersko (page 020–023) where the completed part of the bridge rotated over the railway so that the limitation of railway traffic was minimized. A cable stayed bridge close to Pardubice (page 014–015) was built by the incremental launching method and then the stays were installed and temporary supports removed. The railway bridge at Cervena has a nice arch with the record span (156 m) in Czechia at the moment (page 016–019). The new bridge over the Vltava River in Prague (Dvorecky bridge) is an example of a structure with an extraordinary architectural appearance (page 006–013).

Footbridges offer possibilities for a wide range of structural systems from simple beam elements to cable stayed and suspension structures. UHPC was applied in several cases for improving the durability or for reduction of the self-weight of the structure. Lightweight aggregate concrete was used in the stands of a new multipurpose arena in Brno.

Great attention is paid to rehabilitation of existing bridges. A thorough repair of the heavily loaded Barrandov bridge in Prague (page 058–061) was designed and executed with the minimum impact into the traffic on the bridge. Finally, application of UHPC contributed to acceleration of construction works and to increased durability of the strengthening of the bridge deck.

## NÁRODNÍ ZPRÁVA ČESKÉ REPUBLIKY 7. KONGRES *fib* – LISABON – 2026

Česká národní skupina *fib* s potěšením opět prezentuje svou zprávu u příležitosti kongresu *fib*. Národní zpráva je tradičně připravována týmem České betonářské společnosti (ČBS) a představuje přehled nejvýznamnějších betonových staveb realizovaných v období od posledního kongresu *fib*. To pokrývá projekty dokončené v letech 2022 až 2025 v Česku nebo také v zahraničí, pokud byly navrženy nebo postaveny českými inženýry a společnostmi. Jako obvykle tvoří velkou většinu prezentovaných staveb mosty. Po covidové epidemii, kdy stavební činnost byla redukována, se investice do dopravní infrastruktury během posledních tří let stabilně navyšovaly, a proto bylo postaveno mnoho staveb. Očekává se, že podobný trend potrvá i v roce 2026. Pro nové konstrukce i pro opravy těch stávajících byly použity zajímavé technologie i nové materiály.

Národní zpráva představuje 17 mostů, osm lávek pro pěší a cyklisty a pět dalších projektů. Z toho čtyři mosty a jedna lávka byly realizovány v zahraničí. Čeští inženýři se účastnili velkých projektů zvláště na Slovensku. Dálniční viadukt Kriváň–Mýtna je dlouhý více než 4 km (str. 042–047) Obsahuje semiintegrované konstrukce stavěné různými technologiemi (letmou betonáží, betonáží na posuvné skruži a vysouváním). Viadukty se středním páteřovým nosníkem a dlouhými konzolami zapadají velmi dobře do krásné krajiny. Zajímavá technologie byla použita pro výstavbu mostu u obce Uhersko (str. 020–023) kde byly zhotovené části mostu otočeny přes železniční trať, aby bylo minimalizováno omezení železničního provozu. Zavěšený most u Pardubic (str. 014–015) byl stavěn vysouváním a teprve pak byly instalovány závěsy a odstraněny dočasné podpory. Železniční most u Červené má pěkný oblouk s nyní rekordním rozpětím v Česku (156 m) (str. 016–019) Nový most přes Vltavu v Praze (Dvorecký most) je příkladem stavby s mimořádným architektonickým výrazem (str. 006–013).

Lávky nabízejí možnosti pro široké spektrum konstrukčních systémů od jednoduchých trámů po zavěšené a visuté systémy. V několika případech byl použit UHPC pro zlepšení trvanlivosti nebo k redukci vlastní tíhy konstrukce. Lehký beton byl uplatněn na tribunách multifunkční areny v Brně.

Velká pozornost je též věnována opravám stávajících mostů. Důkladná oprava velmi zatíženého Barrandovského mostu v Praze (str. 058–061) byla navržena a realizována s minimálním omezením provozu. Použití UHPC nakonec přispělo k urychlení stavebních prací a k vyšší trvanlivosti zesílení mostovky.

Články v národní zprávě ilustrují schopnosti českých inženýrů navrhovat a realizovat širokou škálu betonových staveb inovativními způsoby s ohledem na architektonické, environmentální a technické požadavky.

Česká betonářská společnost publikováním této zprávy a technických specifikací, organizací kurzů a konferencí

The articles in the report illustrate the ability of the Czech engineers to design and build structures in an innovative way considering architectural, environmental and technical requirements in a great variety of structures.

The Czech concrete society supports the development of construction technologies by publishing this report and technical guidelines, by organizing courses and conferences and by dissemination of knowledge among their members and beyond.

Jan L. Vítek
Head of the Czech National *fib* Group

a rozšiřováním znalostí nejen mezi svými členy podporuje rozvoj stavebních technologií.

Jan L. Vítek
vedoucí české národní skupiny *fib*

# DVORECKÝ BRIDGE ACROSS THE VLTAVA RIVER IN PRAGUE
# DVORECKÝ MOST PŘES VLTAVU V PRAZE

**01**



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**Upon its completion this year, Dvorecký Bridge became Prague's 19th bridge across the Vltava River. It is designated for use by public transport tram and bus lines, vehicles of the integrated emergency response system, and pedestrians and cyclists; it is not intended for private automobile traffic. Its design emerged from an international architectural and structural engineering competition. The winning design is a continuous box-girder bridge structure made of white prestressed concrete, featuring six spans and two dominant piers in the river, with distinctive atypical shaping of all parts of the bridge, referencing Prague's famous Cubism (Fig. 1).**

**Dvorecký most se stal po svém dokončení v letošním roce 19. pražským mostem přes řeku Vltavu. Je určen pro provoz tramvajových a autobusových linek městské hromadné dopravy, vozidel integrovaného záchranného systému a pro pohyb chodců a cyklistů, neslouží tak individuální automobilové dopravě. Jeho podoba vzešla z mezinárodní architektonicko-konstrukční soutěže. Vítězným návrhem se stala spojitá mostní komorová konstrukce z bílého předpjatého betonu o šesti mostních polích a dvou dominantních pilířích v řece, s výrazným netypickým tvarováním všech částí mostu, odkazujícím na slavný pražský kubismus (Obr. 1).**


Fig. 1 Render of the winning design from the architectural competition
Obr. 1 Vizualizace vítězného návrhu architektonické soutěže

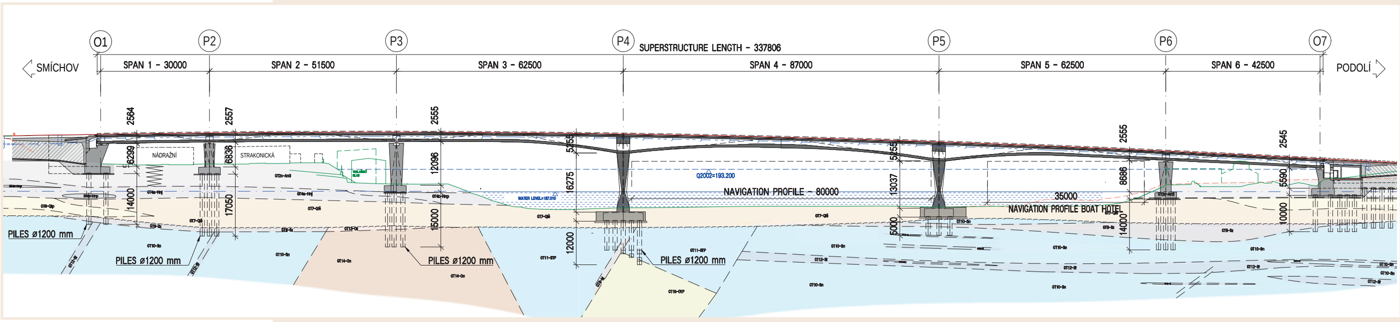
## DESIGN OF THE BRIDGE

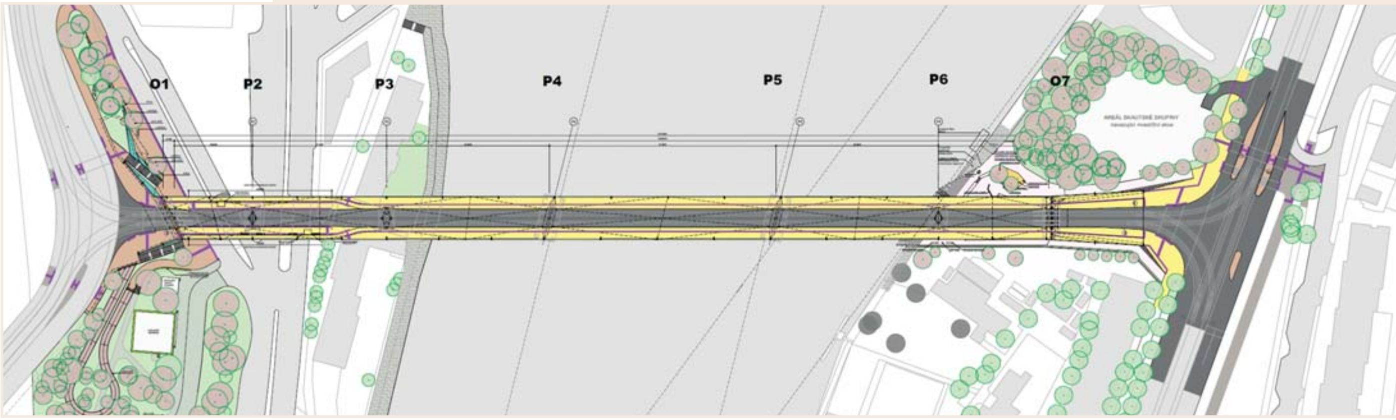
The bridge is designed as a continuous 337.8m long box girder structure made of prestressed concrete with six spans, each 30 + 51.5 + 62.5 + 87 + 62.5 + 42.5 m (Fig. 2).

The layout of the winning design conceptually honors the typology of Prague's arched bridges across the Vltava; in the main spans across the river, it employs an arch motif, while the river piers are intentionally made visually, when looking along the bridge, more substantial, supporting the entire width of the structure. The actual shaping of the substructure and the superstructure is unusual and distinctly contemporary, systematically utilizing the interplay of triangular surfaces with emphasized edges, thereby also referencing the original and famous Cubist structures beneath the nearby Vyšehrad Basilica.

Fig. 2 Longitudinal section
Obr. 2 Podélný řez

Fig. 3 Plan view
Obr. 3 Situace / Půdorys

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The two main piers in the river (water piers) are skewed, follow the angle of the river's flow relative to the route (bridge axis) (Fig. 3) and are rigidly connected to the superstructure, thus together forming a frame. The main span between the water piers is 87 m long, allowing navigation on the river through a single 80 m wide navigation channel. In the adjacent section near the Podolí bank, the bridge further crosses a 40 m wide navigation channel for handling the Racek boat hotel. On the left bank, the structure crosses Nádražní and Strakonická Streets in its first two spans; behind the abutment, the route then smoothly connects to Na Zlíchově Street with the existing tram line. The main span of the bridge safely crosses the river's 7 m high navigation profile; toward the right bank, the bridge level then descends to smoothly connect with the tram line on Podolské nábřeží. Despite the significant drop in level on the right bank, the superstructure is located above the prescribed standard reserve above the Q2002 level (the extreme flood level in Prague in 2002, corresponding to approximately a thousand-year flood).

## FOUNDATION OF THE BRIDGE AND COFFERDAMS IN THE RIVERBED

The structure is located in the river valley floodplain, where the existing banks are covered by thick, often highly heterogeneous fill that was placed here in the past to protect the area against high water levels in the Vltava River. The Quaternary cover consists mainly of sandy-clay alluvium with low bearing capacity and relatively high load bearing terrace gravels. The bedrock massif consists of shales with tuff inserts; near the surface, the shales are irregularly weathered, and the massif is folded with variable layer inclination. A key factor complicating the foundation is the presence of a geological fault zone in the river area of pier P4. Here, strongly tectonically fractured shales and quartzites were encountered, and a lamprophyre vein was also documented marginally. Significant tectonic fracturing in the shales is manifested by local degradation into rock with low to extremely low strength, frequent weathering, crushing, and abundant clay filling in numerous cracks. In such significantly fractured rock, discontinuous blocks of significantly stronger rock (shale, quartzite, or even lamprophyre) of various sizes often float freely. In the quartzite zones, tectonics manifests as significant disruption, and the rock shows a fragmentary decay, with clay filling in the cracks also being common.

The feasibility of using deep foundation elements in the fault zone was the subject of numerous consultations with geotechnical engineers and foundation experts during the design phase. Some recommendations suggested abandoning pile foundations and replacing them with spread foundations combined with extensive subsoil grouting, while others proposed foundations on small-profile piles up to 300 mm in diameter. Still others, on the contrary, suggested using piles of the largest possible diameter so that any boulders encountered could be removed or solid rock layers broken up. In the tender specifications, the designer opted for a larger number of 900 mm diameter piles for pier P4. Due to the significantly poorer quality of the rock (class R5/R6), the pile length was designed to be 20 m long here, whereas for pier P5, also located in the river, but outside the fault zone, short piles 5 m long were planned, embedded in sound bedrock of class R3-R2.

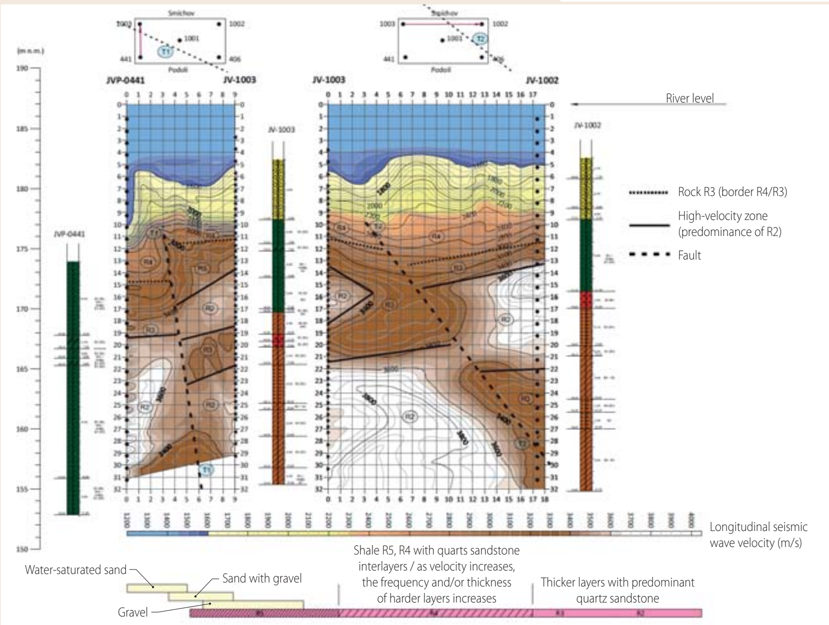



Fig. 4 Seismic tomography of the subsoil of pier P4
Obr. 4 Seismická tomografie podloží pilíře P4

With the exception of the aforementioned pier P4 all other piers and both abutments were designed to be founded on 1200 mm diameter piles, generally with the pile bottoms embedded in rock subsoil of class R4–R3 (R2).

One of the most demanding parts of the bridge construction was the execution of piles P4 and P5, along with their construction pits. The drilled piles for both piers in the riverbed were constructed from a floating pontoon, on which a drilling rig was placed, and drilling was carried out through the water column to the riverbed. Pier P5 is based on a total of 23 drilled piles 1200 mm in diameter, with a total length of 5.0 m. The foundation joint of pier P5 is at a depth of approx. 3.5 m below the level of the bottom of the Vltava riverbed, so the boreholes were approx. 8.5 m long. Although hard rock up to class R2 was encountered about 6 m below the riverbed, the required minimum pile length of 5.0 m was ultimately achieved with considerable effort.

The pontoon with the drilling rig then moved to the location of pier P4 to construct 20m long piles. Although this was a fault zone, the actual construction of the piles almost replicated the scenario at pier P5, where hard rock layers, difficult to drill, were encountered relatively early on. After several attempts at various locations on the foundation with similar results, work on the piles for pier P4 was suspended, and it was decided to prepare a more detailed supplementary geological survey. As part of this, five additional core drillings were carried out in the ground plan of the P4 pier foundation, along with other supplementary tests and measurements (logging measurements and seismic tomography between drillings).The results of the work refined the interpretation of the geological composition of the subsoil and the

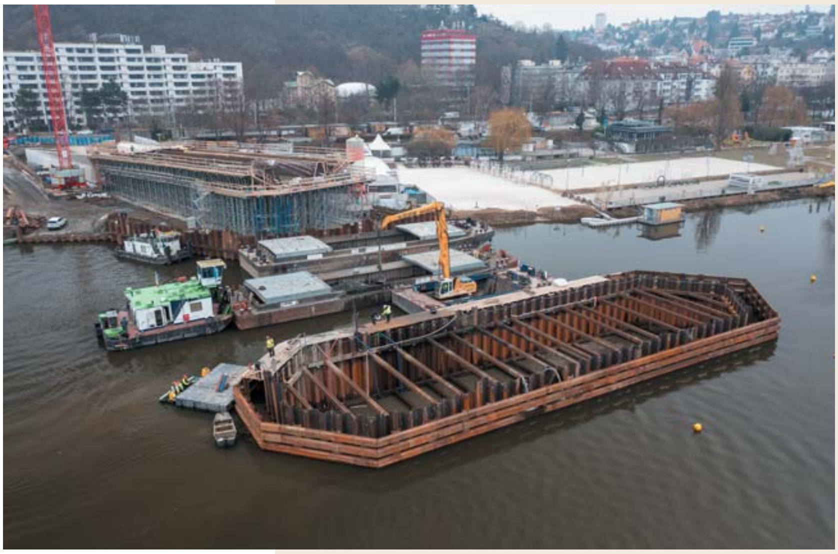


Fig. 5 Sealed cofferdam of pier P5
Obr. 5 Těsněná jímka pilíře P5

characteristics of the geotechnical types encountered, and provided information on the distribution of tectonic disturbance areas in the form of a 3D subsoil model (Fig. 4). The basic characteristics of the disturbed massif were specified as class R4 rock. Based on the additional survey, the technical solution for the foundation was recalculated and modified. Pier P4 is ultimately founded on a total of 48 piles Ø 1200 mm, with a total length of 8.0–12.0 m, where the variable length of the piles is designed with regard to the detected inclination of the layers of firmer positions from the 3D model of the subsoil. With the help of a special drill head, the piles were successfully implemented in accordance with the modified design.

At the same time, grouting of the bases of all piles of pier P4 was designed and subsequently carried out in order to eliminate possible settling at the bases caused by possible loosening of the rock on the contact surface of the pile base due to the drilling technology and difficult cleaning of the bases in rocks of varying quality, small thickness and significant inclination of the layers. This limited the possible differential settlement of the foundations of piers P4 and P5, which would have been unfavourable for the frame structure.

Even before work began on the P4 and P5 water piers, several sheet pile driving tests were carried out to verify the feasibility of constructing sealed sheet pile cofferdams for the foundations and shafts of piers P4 and P5. The results of the tests showed that the originally planned design of sealed double-shell cofferdams, constructed using conventional or vibratory pile driving, with subsequent sealing of the space between the two shells with a clay-cement mixture, appeared to be unfeasible. During the tests, it was not possible to drive the sheet piles down to the weathered edge of the rock subsoil, which was intended to prevent water from flowing under the feet and thus ensure the functionality of the cofferdam. Based on this finding, the technical solution for sealed sheet pile cofferdams was changed to single-shell cofferdams, with sheet piles driven into pre-drilled holes 1200 mm in diameter, and the holes subsequently filled with a clay-cement mixture up to the riverbed level.

The pre-drilled holes were made to a depth of approximately 6.5 m from the bottom level, so the sheet piles were embedded at least 1.5 m into the bedrock under the gravel terrace. The cofferdams were then gradually secured statically during pumping water out using massive steel spacer frames on two levels, and reinforced with concrete at the bottom (Fig. 5). The outer shell of the cofferdams is protected against impact from vessels by a fender system installed on an independent set of sheet piles. During construction, the cofferdams proved to be sufficiently watertight, and residual inflows into the pit were easily pumped out. Only once during construction did the water level in the Vltava River rise to a height that required flooding the cofferdams for safety; after the water level dropped, they were drained again.

## SUBSTRUCTURE

All elements of the substructure are monolithic reinforced concrete. The shapes of the substructure, of both the abutments and piers, follow the architectural design of the entire bridge; all visible surfaces are composed of triangles with accentuated sharp edges.

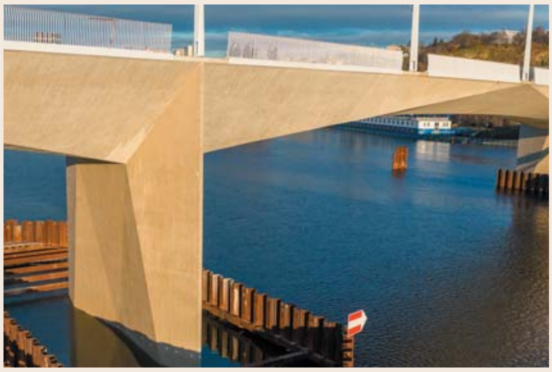
The abutments of the bridge follow the shape of the superstructure (Fig. 6), as do the adjoining wings in the

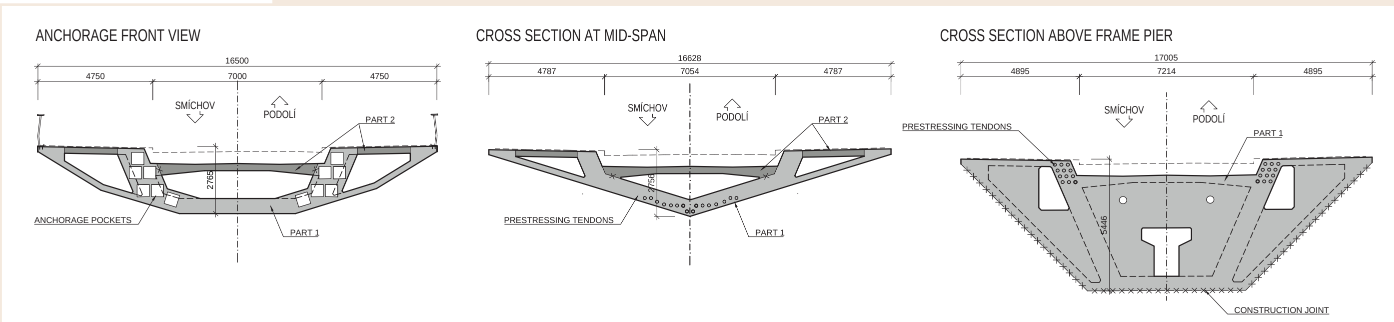
Fig. 6 Formwork of abutment O7
Obr. 6 Bednění opěry O7

Fig. 7 Shaping of abutment O7
Obr. 7 Tvarování opěry O7

Fig. 8 Shaping of pier P4
Obr. 8 Tvarování pilíře P4

Fig. 9 Typical cross sections
Obr. 9 Typické příčné řezy





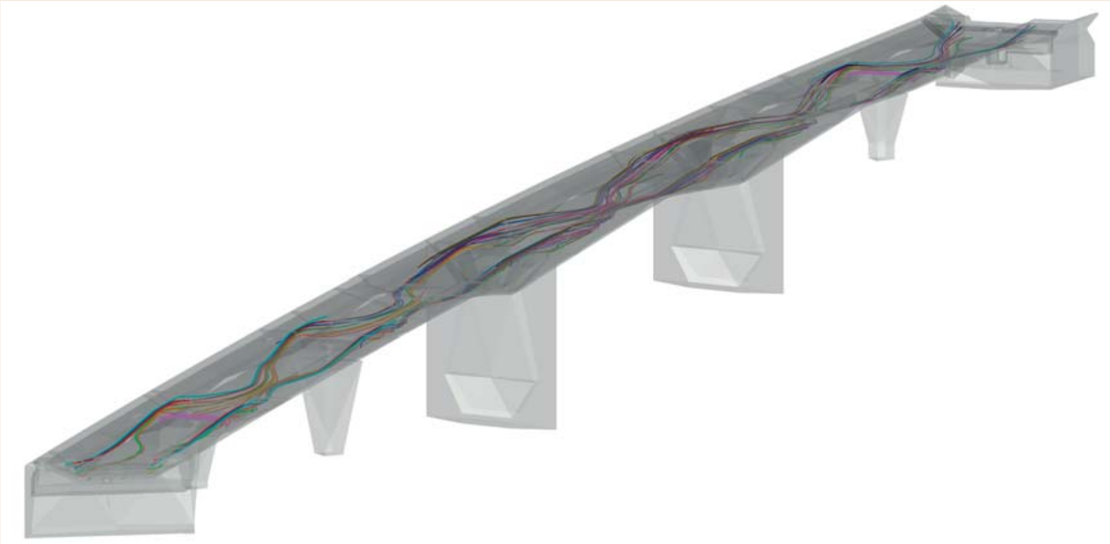


Fig. 10 Detailed 3D model of the structure with prestressing
Obr. 10 Detailní 3D model konstrukce s předpětím

form of retaining walls. Safety doors are installed in the abutment walls, or rather in the side surface of the wing, allowing access to the superstructure box.

The O1 abutment on the Smíchov side is significantly skew in its ground plan, following the course of Nádražní Street under the bridge. The O1 abutment consists of a massive shaft with a bearing sill and screen walls smoothly connecting to the shaft. The front face of the abutment shaft is significantly inclined forward and further divided into triangular sections. Along both wings, there are separate structural elements, such as access staircases to the bridge.

The O7 abutment on the Podolí bank is perpendicular to the bridge axis. The main part of the abutment consists of a massive shaft with a bearing sill, and screen walls smoothly connecting to the shaft, and wing beginnings. The front face of the abutment shaft is composed of triangular surfaces at various angles (Fig. 7). The abutment also includes an internal divided space behind the closing wall, which serves partly as a publicly accessible facility with toilets and partly as non-public space used for the operation of the arena under the last bridge span (see below). The abutment's bearing sill has two height levels: the higher one at the level of the bearing pads that support the superstructure, and the lower one, allowing access from the facilities to the inner chamber of the abutment. The adjacent wings are then formed by separate monolithic reinforced concrete angle walls.

Piers P2, P3 and P6, located on both banks, have a similar structural design, consisting of a single slender shaft. In the horizontal section, the shafts have a trapezoidal shape, variable in height in both directions, and, transversely, they widen with height, allowing bearings to be placed on them at the top. The surfaces are again formed by triangular planes. In addition, the individual edges of the P2 pier shaft rotate in one direction, creating the impression of a helix, so that the pier is not symmetrical to any of the axes in a horizontal section.

Piers P4 and P5, located in the bed of the Vltava River, have a different shape and character than the piers located on the banks. They are oriented parallel to the river flow and thus are skewed to the bridge axis. They are frame-connected to the superstructure and significantly influence the bridge's architectural expression.

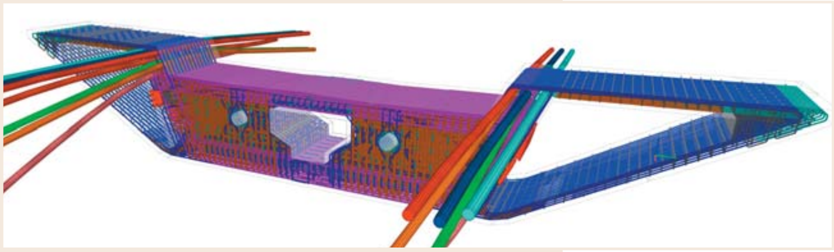


Fig. 11 Prestressing in the central part of the main span
Obr. 11 Předpětí střední části hlavního pole

Fig. 12 Detailed 3D reinforcement of the diaphragm above P2
Obr. 12 Detailní 3D model výztuže příčníku nad P2

The piers rest on a massive foundation slab laid under the riverbed. This is a wall shaft across the entire width of the bridge's superstructure. The wall surfaces are faceted, again forming triangles. Overall, the walls of these piers are very slender in the longitudinal direction of the bridge, tapering to a point at the upstream and downstream edges, thus achieving a hydraulically favourable shape. The central triangular surface of the wall has its apex at the normal water level of the Vltava River at an elevation of 187.00 m above sea level and thus naturally indicates the water level in the river (Fig. 8).

Spherical bearings for supporting the superstructure are located on both abutments and all bank piers (P2, P3, P6).

The complexity of the project and the subsequent construction of the substructure, caused by the atypical shaping and variability of the cross-sections, was further compounded by the architect's requirement to minimise, or even eliminate, construction joints and holes for fixing the formwork in exposed, visible areas. This leads to the casting of larger units in a single stage and more demanding formwork assemblies with higher support requirements.

Fig. 13 Shaping of the superstructure
Obr. 13 Tvarování nosné konstrukce
Fig. 14 Transport of falsework for the main spans
Obr. 14 Přeprava skruže pro hlavní pole

## SUPERSTRUCTURE

The structure consists of a three-box cross-section with two main load-bearing internal walls of variable height. The outer sloping walls define the structure's outer shape and serve as struts for the pavement cantilevers. These struts are inclined and kinked, and their linear shift of the kink point defines the change in the shape of the cross-section along the length of the bridge span, with the cross-section changing smoothly from trapezoidal above the piers to purely triangular in the middle of the span (Fig. 9).

The outer surface is thus again composed of triangles, the length of which is always half the length of the span. Concrete pavement slabs are integrated into the superstructure and fully utilise the available structural height. The top slab of the central box is forcibly (and structurally unsuitably) placed in a lowered position in order to create space for the construction of an independent fixed concrete tram track. The depth of the superstructure in the outer spans (outside the river profile) is a constant 2.7 m; in the spans above the river, it changes smoothly with parabolic haunges from 3.1 m in the main span (or 2.7 m in the adjacent spans) to 5.4 m above the main piers.

The complexity of the superstructure is further increased by its varying skew layout in plan above the individual supports, where the cross-section arrangement changes smoothly from a distinctly skewed support above abutment O1 to a perpendicular arrangement above piers P2 and P3, then again to be rotated above the frame of piers P4 and P5 in the Vltava riverbed, so that the cross-section returns to perpendicular above pier P6 and abutment O7.

Due to continuous changes in the superstructure's skew layout and depth, and to different prestressing alignments along the length of the bridge, almost every cross-section of the superstructure is unique (Fig. 10).

The C45/55 concrete structure is longitudinally prestressed by 27 Y1860 S7-15.7 bonded prestressing units. The prestressing tendons are connected at the joints between individual construction stages or anchored in blisters within the box. The geometry of the tendon routing is extremely complicated, especially in the ground plan, as the tendons must follow the geometry of the main walls, which, with their kinks, shape the outer surface of the superstructure, transitioning to a triangular shape in the spans.

Standard lifting tendons, which must be placed in the box wall above the support at the upper fibres and inside the span at the lower fibres of the cross-section, thus fundamentally change their transverse position from the usual "outer" position at the wall-upper slab joint to a position at the very centre of the cross-section at the apex of the triangle in the centre of the span (Fig. 11).

Above piers P2, P3 and P6, transverse prestressing of the diaphragms is also designed using 7-strand lifting tendons, which compensate for the increased stress in the diaphragms from the indirect setting of the structure on the bearings.

Based on a corrosion survey, the bridge was designated for protective measures of grade 4 against stray-current effects. Taking into account the tram line running directly on the bridge, the proximity of the railway line and the metro line on the Zlíchov bank, a number of measures were designed for the bridge to limit the impact of stray currents on the bridge structure, including the use of a fully electrically insulated prestressing system (PL3).

During elaboration of the detailed design documentation for the superstructure, it became apparent that a large amount of mild reinforcement would be required, up to approximately 300 kg/m$^3$. The reasons for this high consumption include, in particular, the reduced bending and torsional stiffnesses of the triangular cross-section in the middle sections of the bridge spans, the greater need to transfer radial forces from the curved tendon routing, and the greater number of cross-sectional changes in both the transverse and longitudinal directions, which the individual rebars must follow in shape. Given the need for strong structural reinforcement (Fig. 12), the complex prestressing and its electrical insulation, and the concreteability of the cross-section, it was necessary to design the layout of prestressing tendons and rebars for each section with unusual detail and precision. Due to potential collisions between prestressing tendons and rebars, the cross-section shape within the box was further refined locally.

The bridge's architecturally distinctive appearance (Fig. 13) distinguishes it from conventional box girder bridges of similar dimensions and significantly impacts both the scope and labour intensity of the design work and the labour intensity of the construction itself. Nevertheless, the consumption of basic materials for the superstructure was kept at usual levels (concrete: 1.0 m$^3$/m$^2$, prestressing reinforcement: 36 kg/m²). Considering the demanding requirements of the assignment, in particular the load from tram traffic on the bridge, the prescribed tram track construction with an independent fixed track on the bridge, and the relative slenderness of the superstructure given the limited space between the prescribed route elevation and the obstacles to be crossed, the design can be considered economical overall in terms of the consumption of concrete for the superstructure and prestressing. On the other hand, the above-described increase in the consumption of mild reinforcement compared to conventional box structures is significant, but not significant in terms of the total construction costs.

## CONSTRUCTION OF THE SUPERSTRUCTURE

Due to the structure's complex geometry and the spatial conditions and limitations on both banks, a gradual construction on fixed falsework was chosen.

The construction was divided into six stages, with work expected to proceed in individual spans (each with a short overhang into the next span) from both abutments towards the river, with the central section of the main span across the river being the last to be constructed. Above the river, it was originally envisaged that the formwork would be placed on a set of intermediate trestles, placed on steel structures embedded in the riverbed.

The fixed falsework on both abutments was partly made up of spatial falsework (partly in Span 1 – PERI UP Rosset Flex and then completely on the Podolí bank – DOKA Staxo 100) and partly heavy falsework (steel beams on trestles in Span 2 and part of Span 3).

But the falsework above the river was finally implemented in a different, yet interesting way. The dimensions and positions of the cofferdams were adjusted to accommodate the placement of the falsework support towers within the cofferdams next to the piers. Similarly, the shoring on both riverbanks was prepared for the placement of the falsework towers. The falsework in Spans 3 and 5 above the river was then made of massive steel truss girders, which had previously been used to construct temporary railway bridges. These structures were transported in parts to the assembly yard about 5 km away, where they were assembled into pairs, each to the required lengths of 39 m and 54 m, respectively, and gradually moved onto Helios pontoon boats and transported down the river to the site (Fig. 14). Here, they were lifted directly from the pontoons into the desired position on the support towers using hydraulic jacks. In total, these structures weighed almost 900 tonnes. The main parts of Spans 3 and 5 were then constructed on these structures.

The outer parts of the central span at the piers in the river were constructed at the same time on short girders resting on only two intermediate trestles set up in the

Fig. 15 Casting of stage 4
Obr. 15 Betonáž etapy 4

Fig. 16 Construction of the central part of the main span (stage 6)
Obr. 16 Výstavba střední části hlavního pole (etapa 6)

Fig. 17 Prestressing and mild reinforcement of stage 6
Obr. 17 Předpětí a výztuž etapy 6

Fig. 18 Artwork under the bridge (Heavy Head Boy)
Obr. 18 Umělecká díla pod mostem (Heavy Head Boy)

main river flow between piers P4 and P5 (Fig. 15). After concreting, the girders from the Podolí side were moved on pontoons to the main span to execute the final stage, which connected both parts of the bridge into a single unit (Fig. 16). During the construction of the final stage, when the formwork was positioned in the middle of the river, navigation was routed under the already completed third span of the bridge near the Zlíchov bank.

The complexity of the construction of the superstructure of Dvorecký bridge lay primarily in projecting the designed bridge's shape into the formwork design, which had to conform not only to the given geometry but also to the architect's requirements for minimizing the number of visible construction joints and assembly openings for formwork fastening. The formwork design was carried out exclusively in 3D in collaboration with the bridge designer, and the reusability of the formwork elements was minimal, partly due to time constraints. A major challenge was the placing of the concrete and prestressing reinforcement within the formwork. Given the large amount of concrete reinforcement, it was necessary to work with increased precision (Fig. 17) so that the smooth geometry of the prestressing tendons was not disrupted even locally and so that the contact between the reinforcement and the tendon ducts, which could impair the electrical insulation properties of the prestressing system, was minimised (and provided with special inserts). The prescribed use of white concrete also significantly contributed to the complexity of the concrete work. The desired colour was achieved using white cement. This was custom-made and imported from Slovakia. Due to the increased amount of cement in the mix, the rate of heat release during hydration was higher than in standard concrete. For the concrete pouring of the more massive parts of the structure (diaphragms, but also abutments), a cooling system was therefore installed.

## BRIDGE ACCESSORIES AND EQUIPMENT

The roadway with tram tracks on the bridge is designed to be covered with mastic asphalt so that it can also be used by road traffic. The independent tram track structure (fixed track with W-tram fastening system) is embedded in the bridge and rests on a lowered upper deck slab, which is separated from it by asphalt insulation bands and anti-vibration rubber mats.

Given the length of the bridge, the transition of the tram track from the bridge to both approaches is achieved using track expansion devices. The bridge expansion joints are of the finger type. The bridge insulation is full-surface, the insulation from the trough under the roadway is continuously extended to the pavement cantilevers.

The surface of the pavements is covered with a coloured layer from ACO (asphalt concrete), and the kerbs are made of stone. Water is drained from the road surface and pavements on the bridge by means of longitudinal and transverse slopes into bridge gullies located on both sides of the road. In the area of zero longitudinal slope, the gullies are replaced by a continuous trench drainage along the kerb. Water from the bridge is then drained through longitudinal drainpipes in the bridge box via end walls into the sewer system on both approaches.

On both outer edges of the bridge, there is a post-free railing, which is locally interrupted in designated places by steel poles of the tramway traction line. These also serve to mount public lighting fixtures. The railing structure itself, with a solid white artificial stone handrail at a height of 1.30 m above the pavement surface, and the shapes of the traction line poles with a square cross-section tapering conically along the height of the pole, are designed according to the architect's requirements and follow to the triangular surfaces of the superstructure. The white handrail itself is illuminated from below by an inserted continuous lighting strip with spot LED elements, which illuminate the adjacent pavement surface and serve for ceremonial lighting of the bridge and supplement the public lighting fixtures on the traction line poles.

The bridge is equipped with monitoring devices and systems that allow to monitor the temperature and deflection behaviour of the structure, penetration of aggressive substances into the concrete, tension in the

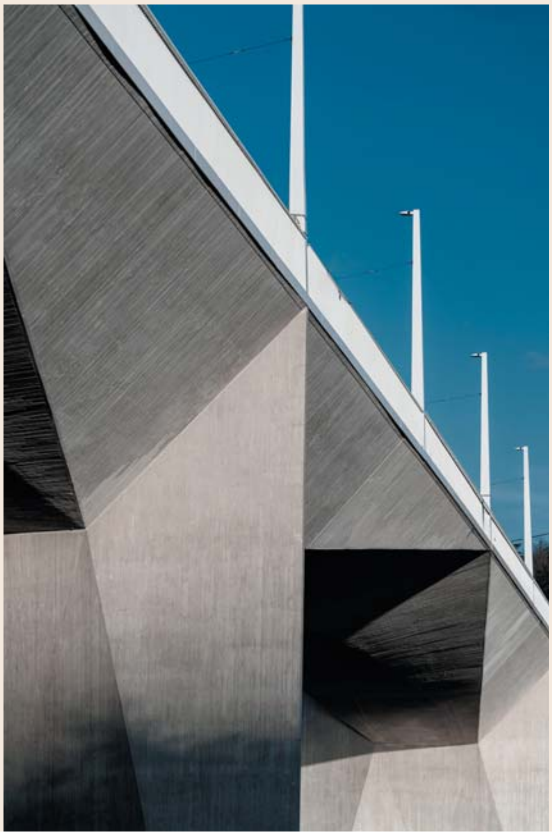


Fig. 19 Completed bridge
Obr. 19 Dokončený most

prestressing tendons, electrical insulation properties of the prestressing system and corrosion potential of prestressing and mild reinforcement.

## OTHER STRUCTURES AND ART INSTALLATIONS

The Dvorecký Bridge project also includes adjacent structures. Among these we should mention the PID Information Center, located on the lower level of Nádražní Street near the bus stop. It is embedded in the embankment of the newly widened Na Zlíchově Street. The facility will serve as a transit hub for passengers, connecting the bus stops on the lower level in Nádražní Street and the new tram & bus stops, located on the upper level directly on the bridge in close proximity to bridge abutment O1. Direct pedestrian connections between both levels are provided by staircases situated on both sides of abutment; on the southern side, the staircase is complemented by a lightweight steel footbridge in the shape of a loop winding along the slope of the embankment of Na Zlíchově Street, which provides barrier-free access between the two levels.

From the very beginning of the Dvorecký Bridge feasibility study, considerable attention was paid to the design of the public space on both approaches; plans included the installation of artworks and the use of the area for cultural and recreational activities. Various artists were also involved in these designs, notably the sculptor and visual artist Krištof Kintera. On the Zlíchov side, a so-called "light garden" is being prepared according to his design, featuring lights lanterns collected from around the world – the installation is named "Light Removes Darkness". On the Podolí side, an urban landscape is created beneath the bridge structure – a multipurpose arena for cultural and recreational activities – including theater performances, a skatepark, a bouldering zone, a staircase leading down to the river, a picnic area with a mobile, artistically designed café, etc., complemented by the installation of individual pieces of artwork (Fig. 18).

## CONCLUSION

The unusual and unique bridge design (Fig. 19, 20) based on a proposal from an architectural and structural design competition, along with the foundation of the bridge in the river, presented an extraordinary challenge for both designers and builders from the very beginning. The construction was accompanied by certain difficulties, particularly regarding the actual geological conditions encountered in the Vltava River, which affected the foundation work and the excavation of pits for piers P4 and P5. Local restrictive conditions also contributed to the high complexity of the construction.

Dvorecký Bridge was opened to service in April 2026 and became a new addition to Prague's collection of bridges across the Vltava River, the vast majority of which are exceptional. Due to its distinctive design, it is likely be impossible to overlook. The designers and builders believe that the future will prove the new bridge to be a worthy addition to the list.

## MATERIAL USAGE (SUPERSTRUCTURE)

| | TOTAL | PER $1 M^2$ |
|---|---|---|
| CONCRETE C45/55 | 5752,7 $m^3$ | 1,03 $m^3$ |
| PRESTRESSING STEEL (LONGITUDINAL) | 202,9 t | 36,4 kg |
| REINFORCING STEEL | 1770,9 t | 307,8 kg |

## SPOTŘEBA MATERIÁLU (NOSNÁ KONSTRUKCE)

| | CELKEM | NA $1 M^2$ |
|---|---|---|
| BETON C45/55 | 5752,7 $m^3$ | 1,03 $m^3$ |
| PŘEDPÍNACÍ VÝZTUŽ (PODÉLNÁ) | 202,9 t | 36,4 kg |
| BETONÁŘSKÁ VÝZTUŽ | 1770,9 t | 307,8 kg |

Fig. 20 Completed bridge
Obr. 20 Dokončený most

# CABLESTAYED BRIDGE OVER THE ELBE RIVER IN PARDUBICE NEAR FABLOVKA

# ZAVĚŠENÝ MOST PŘES LABE V PARDUBICÍCH U FÁBLOVKY

02

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**The cable-stayed bridge is part of the north-eastern bypass of Pardubice. The project was procured under a design-and-build arrangement. As the key structure along the route, the bridge – featuring a main span arrangement of 122 + 135 metres and a single pylon – was redesigned extensively compared to the original tender documentation. The final structural system was developed in close alignment with the selected construction method.**

**Zavěšený most je součástí Severovýchodního obchvatu Pardubic. Stavba komunikace byla zadána formou design and build. Most o rozpětích 122 + 135 m s jedním pylonem jakožto nejvýznamnější objekt trasy byl oproti zadání zcela přepracován. Konstrukční systém je navržen v souladu s technologií výstavby.**

Fig. 1 Completed bridge
Obr. 1 Dokončený most

## DESIGN SOLUTION

The bridge designed in the tender documentation had a steel main span and a concrete smaller span. However, at the time of the tender, the price of structural steel was very high, making the original design uncompetitive.

Several construction methods were considered during the preparation of the bid, with the launch of the bridge deck on temporary supports, followed by the construction of the pylon and installation of the stay cables, emerging as the most favourable option. The structural system was aligned to this construction method. Launching to a span of 40 metres on temporary supports corresponded to a design cross-sectional height of 2.4 metres and required centric prestressing. To facilitate the launching process, it was recommended that a constant cross-section be maintained along the entire length of the bridge. This resulted in the stay cables being set up symmetrically, which increased the adjacent span to 122 m compared to the 90 m stated in the tender documentation. The main span is 135 m, and the weight of the missing 13 m of bridge deck compared to the main span was compensated for by a massive support crossbeam into which the longest stay cable was anchored. The pylon experiences considerable bending stress, so it was designed with two stems to significantly increase its rigidity; the reinforcement of the stems was standard. The pylon and bridge deck are embedded in a segmented pillar (Fig. 2 and 3).

The structure's load-bearing behaviour is therefore very clear: it is rigid in the area of the central pillar and has sliding bearings on the abutments. Together with the one-sided anchoring of the longest stay cable to the crossbeam, this creates a robust construction system that can withstand asymmetrical payloads and eliminates the need for a prop in one of the spans, unlike some other single-pylon bridges. The sufficiently rigid, pre-tensioned bridge deck is anchored by only seven pairs of stay cables, simplifying the construction process technologically. Increasing the length of the bridge also saved material from the high embankment behind the abutment.

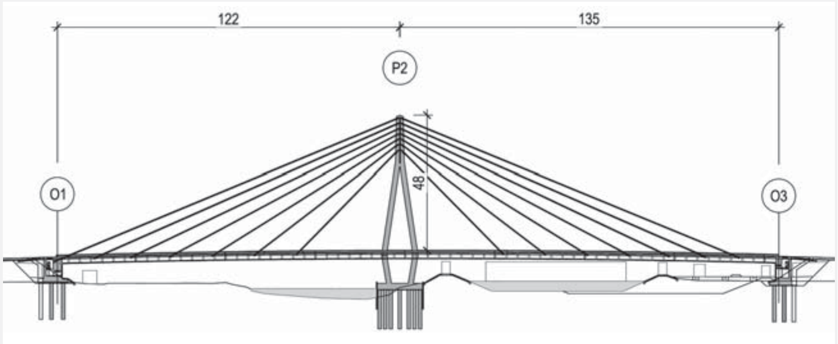
The bridge's structural design has therefore been carefully optimised to achieve competitive pricing, while also paying attention to aesthetic effects and future maintenance. The top of the pylon can be reached from the platform. The stay cables are made up of replaceable galvanised monostrands.

## BRIDGE CONSTRUCTION

Concreting of the bridge deck took place in two adjacent bays. In the first bay, the bottom and walls of the box were concreted. In the second bay, inclined precast struts were installed and the slab was concreted. The bridge was then launched using a rope system anchored to the abutment. A total of eight launches of the standard 32 m length took place (Fig. 4). The pylon was constructed in stages using climbing formwork. The stay cables were transferred through the pylon in saddles, with a separate teardrop-shaped channel created for each rope. The cable was then wedged into place by symmetrical pretension (Fig. 5).

## CONCLUSION

The cable-stayed bridge was built on time, within budget and to a high standard. In addition, the design was created with the aim of minimising future maintenance costs. From an aesthetic point of view, the bridge is generally very well received (Fig. 1).



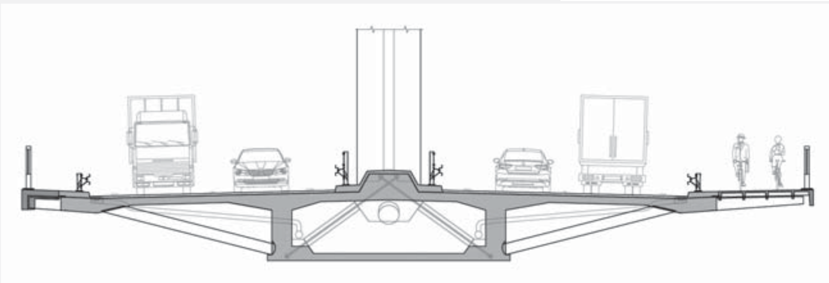



Fig. 2 Longitudinal section of the bridge
Obr. 2 Podélný řez mostem

Fig. 3 Cross-section of the bridge
Obr. 3 Příčný řez mostem

## KONSTRUKČNÍ ŘEŠENÍ

Most navržený v zadání měl hlavní pole ocelové, zatímco vedlejší kratší pole bylo betonové. V době veřejné soutěže byla velmi vysoká cena konstrukční oceli, čímž se původní návrh stal nekonkurenceschopným.

Během přípravy soutěžního návrhu se zvažovalo několik technologií výstavby, při čemž výsuv mostovky na dočasných podporách s následnou výstavbou pylonu a montáží zavěšení vycházel nejpříznivěji. S technologií výstavby souvisel konstrukční systém. Vysouvání na rozpětí 40 m po dočasných podporách odpovídalo konstrukční výšce průřezu 2,4 m a vyžadovalo centrické předpětí. Pro výsuv je vhodné ponechat konstantní průřez po celé délce mostu. Z toho vycházela symetrická osnova závěsů, a to vedlo ke zvětšení rozpětí vedlejšího pole na 122 m oproti zadání, kde bylo jen 90 m. Hlavní pole má rozpětí 135 m. Tíha chybějící délky mostovky 13 m oproti hlavnímu poli se kompenzovala mohutným podporovým příčníkem, do kterého se zakotvil nejdelší závěs. Ohybové namáhání pylonu je značné, proto byl navržen rozkročený do dvou dříků, čímž se výrazně zvýšila jeho tuhost a vyztužení dříků bylo na běžné úrovni. Pylon s mostovkou jsou vetknuty do členěného pilíře (obr. 2 a 3).

Statické působení konstrukce je tedy velmi transparentní, v oblasti středního pilíře tuhé vetknutí, na opěrách posuvná ložiska. Spolu s jednostranným kotvením nejdelšího závěsu do příčníku vzniká robustní konstrukční systém, který dobře odolává i nesymetrickému užitnému zatížení, a nevyžaduje tak stojku v jednom z polí, jak je tomu u jednopylonových mostů obvyklé. Dostatečně tuhá a předepnutá mostovka je zavěšena jen sedmi páry závěsů, což vedlo k technologickému zjednodušení procesu výstavby. Zvětšení délky mostu umožnilo úsporu materiálu vysokého násypu za opěrou.

Konstrukční řešení mostu je tedy pečlivě optimalizované s cílem dosáhnout konkurenceschopné ceny díla, ale rovněž bylo dbáno na estetické působení a v neposlední řadě i na budoucí údržbu. Vrchol oproti zadání sníženého pylonu je dosažitelný z plošiny. Závěsy jsou sestaveny z pozinkovaných monostrandů, které jsou vyměnitelné.

## VÝSTAVBA MOSTU

Betonáž mostovky probíhala ve dvou navazujících výrobnách, v první se betonovalo dno a stěny komory, ve druhé se osazovaly šikmé prefabrikované vzpěry a betonovala se deska. Vysouvalo se lanovým systémem kotveným do opěry. Celkem proběhlo osm výsuvů standardní délky 32 m (obr. 4). Pylon se budoval do po taktech do překládaného bednění. Závěsy jsou přes pylon převáděny v sedlech, kde je pro každé lano vytvořen separátní kanálek kapkovitého tvaru, do něhož se lano symetrickým předepnutím zaklíní (obr. 5).

## ZÁVĚR

Zavěšený most se podařilo postavit v rámci rozpočtu, v požadovaném termínu a ve vysoké kvalitě. Kromě toho se při návrhu dbalo na minimalizaci budoucích údržbových nákladů. Po estetické stránce je most všeobecně přijímán velmi příznivě (obr. 1).

Fig. 4 Launching of the bridge deck
Obr. 4 Průběh výsuvu mostovky

Fig. 5 Installation of the stay cables
Obr. 5 Montáž závěsů

03

# RAILWAY BRIDGE OVER THE VLTAVA RIVER NEAR ČERVENÁ

# ŽELEZNIČNÍ MOST PŘES VLTAVU U ČERVENÉ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**The railway arch bridge near Červená carries a single-track railway connecting the southern Bohemian cities Tábor and Písek. The new bridge replaces the old steel bridge from the 19th century, which does not satisfy the requirements of the Railway administration, mainly the load carrying capacity, due to severe corrosion. It was necessary to avoid the piers in the river, therefore an arch bridge with arch footings outside the river was designed. The arch was built by the free cantilever method with temporary pylons and temporary stays. The prestressed bridge deck was cast on fixed scaffolding. The bridge was completed in 3 years.**

**Železniční obloukový most u Červené převádí jednokolejnou trať spojující jihočeská města Tábor a Písek. Nový most nahrazuje starý ocelový most z 19. století, který nesplňuje požadavky drah na únosnost z důvodu rozsáhlé koroze. Bylo požadováno, aby pilíře nového mostu nebyly umístěny v řece, proto byl navržen obloukový most s patkami na březích mimo říční tok. Oblouk se stavěl letmou betonáží s dočasnými pylony a závěsy. Předpjatá mostovka byla betonována na pevné skruži. Celý most byl postaven během tří let.**

Fig. 1 The old steel railway bridge from 1889
Obr. 1 Starý ocelový železniční most z roku 1889

## 1. INTRODUCTION

The railway connecting the cities Tábor and Písek in southern Bohemia crosses the Vltava River. The original steel truss bridge was completed in 1889. At that time the Vltava River was a rather small watercourse in a deep valley. The steel bridge with 3 spans, each 84.4 m long, appeared as an interesting landmark high above the water level. In the 1960's a dam was built downstream from the bridge and the water level significantly increased. The piers of the old bridge had to be protected against water and because of flooding they appeared to be shorter and too big, and the favourable impression of the high bridge was significantly reduced (Fig. 1). After more than 130 years of operation the steel structure was heavily corroded. It was decided to build a new bridge and demolish the old steel bridge. Railway traffic had to be maintained on the old bridge during the construction of the new bridge. The axis of the new bridge is parallel to the axis of the old bridge at a distance of only about 10 m.

The new bridge is an arch bridge, a suitable design without piers in the river. The footings of the arch are located above the water level on the river banks. The arch is at the moment the longest concrete arch in Czechia with a span of 156 m. It carries the bridge deck made of prestressed concrete (Fig. 2).

After the construction of the new arch bridge started, some local people began to fight for preservation of the old bridge. They were successful in that the Ministry of culture declared the old bridge to be a cultural monument, which cannot be demolished. Now there is a strange situation of two bridges very close each other, which resulted in a very bad appearance of the two structures. The new bridge is in the ownership of the railway administration. The maintenance of the old bridge is not resolved, and the future owner is also not known.

## 2. BASIC PARAMETERS OF THE NEW BRIDGE

The concrete bridge carries a single-track railway. The span of the reinforced concrete arch which is fixed into

Fig. 2 The new concrete arch bridge
Obr. 2 Nový betonový obloukový most


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large footings is 156 m and its elevation is 34.7 m. The bridge deck has a double T section, it is only 5.9 m wide. The spans of the bridge deck vary in the range 20 to 24 m. The total length of the bridge deck is 299.2 m.

The bridge has mostly flat foundations. There is hard rock in the vicinity of the bridge. Only the abutment on the right-hand side river bank (direction Tábor) is founded on piles, because of its location on the railway embankment.

Large foundations support the arch and the tallest piers. On the right-hand side of the river, the foundation pit was excavated according to the design of the bridge. On the left-hand side, the rock base was founded deeper, the soft ground had to be replaced by additional concrete filling under the foundation.

The bridge deck has 12 spans. On each side, there are 3 spans above the slopes of the valley and 3 spans above the arch. The piers above the slopes have a rectangular cross-section 4 × 1.5 m and their height varies from 8.5 to 35 m. The four piers on the arch are from 5.8 to 18 m high and their cross-sections are slightly smaller, 4 × 1.1 m.

The arch has a rectangular hollow cross-section with a constant width of 5 m and a variable depth. At the footings, the arch is 3.3 m deep, while at the midspan it is only 2.5 m deep. The two webs have a constant thickness of 0.6 m and the thicknesses of the top and bottom slab are variable from 0.85 m at the footings to approx. 0.5 m at the midspan. The dimensions of the cell inside the arch are constant 3.8 × 1.5 m.

The bridge deck has a double T section, 5.9 m wide and 1.35 m deep. The piers on the arch and the footings are connected with the bridge deck by concrete hinges. On piers above the slopes and on abutments, the bridge deck is supported on sliding spherical bearings. The fixed point of the bridge deck is in the centre of the arch, where the bridge deck was cast directly on the arch. The deck and the arch are connected along a length of 22 m. The bridge is a semi-integral structure which has only 12 bearings on supports with easier access and two expansion joints at the ends of the bridge. The bridge deck is prestressed by 4 tendons composed of 15 strands, Ø15.7 mm. The central part of the bridge deck above the arch crown is not prestressed, and is only reinforced by mild reinforcement.

## 3. CONSTRUCTION OF THE NEW BRIDGE

The works on the site started in December 2021. Complex excavation works had to be done especially on the Tábor side (Fig. 2 left), where the slope is very steep. The slope is about 30 m deep and rocky. Blasting was necessary for excavation of the foundation pit for the footing of the arch and the tall pier. The machines for all works had to be transported to the foundation pit by crane. During 3 months, 9000 m$^3$ of rock was excavated. At the same time the foundations of shorter piers were erected. It was necessary to drill and assemble the ground anchors and micro piles to the foundations. Without them the tensile forces induced by temporary stays during the erection of the arch could not be anchored. Two teams worked on each side of the valley; the left and right part of the bridge were built independently until the arch was connected at the midspan. Fig. 3 shows the excavation works on the Tabor side of the river. The steep slope had to be stiffened by a concrete transversal beam anchored to the subbase by ground anchors.

The arch footings were rather large structures with dimensions 12 × 11 × 6.4 m. They were cast in 3 parts. For the largest part, it was necessary to pour 650 m$^3$ of concrete. The increased temperatures due to the heat of hydration had to be kept under the limit of 70°C, which is given by the technical specifications of the Czech railway administration. Concrete C30/37 90d was tested and its composition was modified. The cement CEMII/B-S 32.5 was used in the amount of 335 kg/m$^3$ of concrete. The temperature gauges were embedded in the core of the cast parts of the large footings. The maximum temperature measured in the core of the footing was 57.4°C.

After casting the foundations, the piers in the slope area were built. Standard formwork was used in steps 4.5 m high. It was necessary to install the anchors of temporary stays into some foundations and piers and also to leave the

Fig. 3 Excavation works on the Tabor side, anchored stiffening beam guarantees the stability of the slope.
Obr. 3 Výkopové práce na táborské straně, kotvený ztužující trám zajišťuje stabilitu svahu

Fig. 4 Completed piers on the slopes with openings for temporary stays
Obr. 4 Dokončené pilíře ve svazích s otvory pro dočasné závěsy

openings in piers for temporary stays (Fig. 4), which were necessary for the erection of the arch. The construction continued by casting of the bridge deck above the slopes until reaching the highest pier above the arch footing.

Access into the valley was practically excluded because of steep slopes and deep water in the river. The arch was cast by the cantilever method, using temporary pylons and stays. A simple scheme is shown in Fig. 5. The initial part of the arch at the footing was cast on fixed scaffolding (Fig. 6). This scaffolding was a part of the form traveller which was used later for casting of subsequent segments of the arch. The segments were about 5.2 m long. The form traveller (Fig. 7) was operated by a hydraulic system, which allowed for its movement forwards and for adjustment of its precise position. The transversal steel beam which supported the form in the temporary stays had to be adjusted, so that the correct shape of the arch was achieved.

After casting of the last segment of one half of the arch, this form traveller was suspended on bars and lowered on the pontoon and then transported to the bank. The last segment of the other half of the arch could be cast using the remaining form traveller. Then it was also lowered to the pontoon. The gap between the completed two halves of the arches was 2.4 m wide. The ends of the arch cantilevers were temporarily connected by steel elements, and the formwork was assembled for casting the closing gap. Each segment of the arch was cast in 12 days on average.

After closure of the arch, the pylons and temporary stays were removed. The piers on the arch were cast (Fig. 8) and finally the bridge deck above the arch was

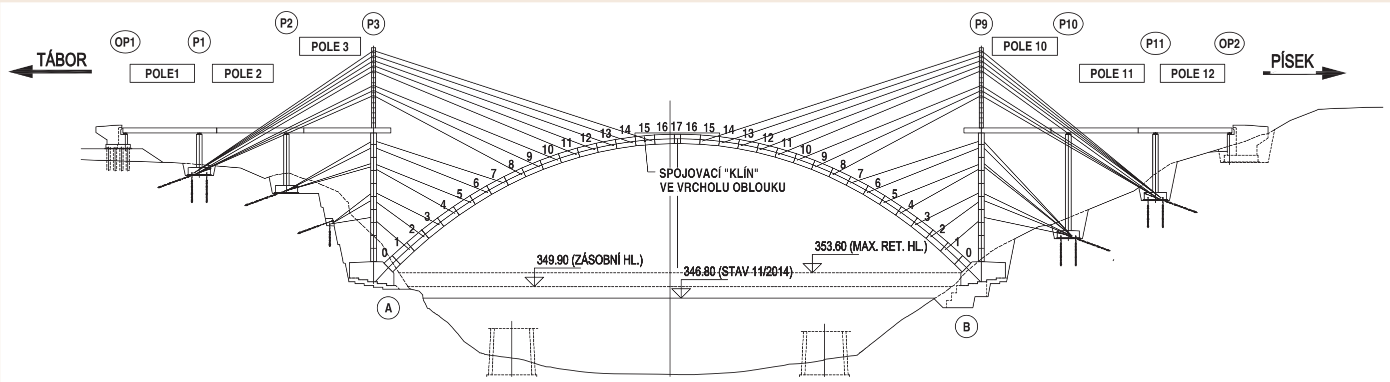


Fig. 5 Scheme of the arch construction
Obr. 5 Schéma výstavby oblouku

Fig. 6 Formwork for the first part of the arch
Obr. 6 Bednění první části oblouku

Fig. 7 Cantilever casting of the arch using a temporary pylon and stays
Obr. 7 Letmá betonáž oblouku s dočasným pylonem a závěsy

traveller in front had to be installed by crane. Large tower cranes were located on the arch foundation and reached up to the midspan of the arch. The stays were composed of 12 or 19 strands Ø15.7 mm. The temporary pylons installed on the bridge deck above the highest piers were 21.6 m high and were made of reinforced concrete. They were cast in segments 1.35 m high separated with plastic foil. The reinforcement located along the perimeter of the cross-section continued through the joints between segments. It made it possible to more easily demolish the temporary pylons after completion of the arch.

The two halves of the arch were erected approximately symmetrically from each bank of the river. Very close co-operation between the designer and the contractor was necessary. After casting of each segment, the geometry of the arch was measured several times, and the future position of the form traveller was specified. The forces completed. A fixed scaffolding was used for casting of the bridge deck. In the spans above footings, temporary towers located beside the piers supported steel beams carrying the formwork. In spans close to the midspan a light scaffolding was used (Fig. 9).

When the bridge deck was completed and prestressed, the edge beams were cast, and the waterproofing and railway track were installed.

The traffic on the railway was interrupted only in about the last 3 months of the construction. During this period, the existing railway was connected to the track on the new bridge.

Static and dynamic loading tests were executed in November 2024. Heavy steam and diesel locomotives and a loaded carriage were used for the static loading test (Fig. 10). The dynamic test followed. All tests have proven the expected behaviour of the bridge structure.

Fig. 8 Completed piers on the arch
Obr. 8 Dokončené pilíře a oblouk

Fig. 9 Tower supporting the formwork (right), light scaffolding on the arch (middle)
Obr. 9 Věž podporující bednění (vpravo), lehká skruž na oblouku (uprostřed)

The bridge was opened to traffic on Nov. 29, 2024, which is slightly less than 3 years after the construction started. The costs were adequate to the expectation of the client, about 22 mil. EUR.

## 4. CONCLUSIONS

The construction of the railway arch bridge in Červená crossing the Vltava River was a successful project. The design documentation was prepared by an experienced design office. The cantilever construction of the arch was supervised by skilled engineers. Their experience guaranteed the safety and functionality of the structure and also the correct geometry after the complex construction process. The contractor already had experience with a similar highway bridge with a span of 135 m, which was built earlier.

The team on the site had to face many challenges given by local conditions, like steep slopes inducing sophisticated transport of materials and variable quality of the soil and rock. The arch bridge construction using free cantilevering and temporary stays required very close co-operation with the designer. The form travellers also represented a complex device equipped with hydraulic systems controlling their movement to the next position and detailed adjustment of the geometry. It was necessary to pay a lot of attention to the concrete technology. Casting of massive foundations required limitations of the heat of hydration. Casting of heavily reinforced arch segments with additional anchors of temporary stays and anchors for the suspension of form travellers required concrete with an appropriate workability. Finally, all activities on the site were completed successfully and on time and the client could be satisfied with the new bridge. Now the bridge has the longest concrete arch in Czechia. What happens with the old bridge will be decided in the future.

## 5. PARTICIPANTS OF CONSTRUCTION

| | |
|---|---|
| CLIENT: | Railway administration (Správa železnic), state organization |
| DESIGN OF THE BRIDGE: | SUDOP PRAHA a.s. |
| CONTRACTOR: | Association Metrostav a.s. and Metrostav TBR a.s. |
| SUPERVISOR OF CONSTRUCTION: | PONTEX, Ltd. |
| SCAFFOLDINGS, FORM TRAVELLERS: | ROLAND CZ, Ltd. |
| PRESTRESSING: | VSL Systems CZ, Ltd. |

## MATERIAL USAGE

| STRUCTURAL ELEMENT | MATERIAL | Consumption | Consumption /m² |
|---|---|---|---|
| FOUNDATIONS – ABUTMENTS | CONCRETE C25/30 | 163 m³ | 0.081 m³ |
| | STEEL B500B | 25 t | 12 kg |
| FOUNDATIONS – PIERS | CONCRETE C30/37 | 1802 m³ | 0.890 m³ |
| | STEEL B500B | 213 t | 105 kg |
| ABUTMENTS | CONCRETE C30/37 | 261 m³ | 0.129 m³ |
| | STEEL B500B | 28 t | 14 kg |
| PIERS | CONCRETE C35/45 | 841 m³ | 0.415 m³ |
| | STEEL B500B | 92 t | 45 kg |
| ARCH | CONCRETE C45/55 | 1430 m³ | 0.706 m³ |
| | STEEL B500B | 501 t | 247 kg |
| BRIDGE DECK | CONCRETE C45/55 | 1429 m³ | 0.706 m³ |
| | STEEL B500B | 226 t | 112 kg |
| | PRESTRESSING STRANDS Y1860S7 – 15.7 – A | 41 t | 20 kg |

## SPOTŘEBY MATERIÁLŮ

| STAVEBNÍ PRVEK | MATERIÁL | Spotřeba | Spotřeba /m² |
|---|---|---|---|
| ZÁKLADY OPĚR | BETON C25/30 | 163 m³ | 0,081 m³ |
| | VÝZTUŽ B500B | 25 t | 12 kg |
| ZÁKLADY PILÍŘŮ | BETON C30/37 | 1802 m³ | 0,890 m³ |
| | VÝZTUŽ B500B | 213 t | 105 kg |
| OPĚRY | BETON C30/37 | 261 m³ | 0,129 m³ |
| | VÝZTUŽ B500B | 28 t | 14 kg |
| PILÍŘE | BETON C35/45 | 841 m³ | 0,415 m³ |
| | VÝZTUŽ B500B | 92 t | 45 kg |
| OBLOUK | BETON C45/55 | 1430 m³ | 0,706 m³ |
| | VÝZTUŽ B500B | 501 t | 247 kg |
| MOSTOVKA | BETON C45/55 | 1429 m³ | 0,706 m³ |
| | VÝZTUŽ B500B | 226 t | 112 kg |
| | PŘEDPÍNACÍ VÝZTUŽ Y1860S7 – 15.7 – A | 41 t | 20 kg |

Fig. 10 Loading test of the completed bridge
Obr. 10 Zatěžovací zkouška hotového mostu

# BRIDGE ON THE D35 OVER UHERSKO RAILWAY STATION
# MOST NA D35 PŘES ŽELEZNIČNÍ STANICI UHERSKO



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**The D35 motorway in the section Časy–Ostrov runs through flat countryside east of Pardubice. At the location of Uhersko railway station, the motorway crosses the Prague–Česká Třebová line at a low elevation and at a sharp angle. To bridge the six railway tracks, a prestressed concrete bridge with a single superstructure for both motorway directions was designed. The continuous three-span superstructure was divided into two parts, which were constructed parallel to the railway line and then rotated in opposite directions into their final positions, including part of the ancillary structures. The construction method using rotation was already defined in the building permit design stage in order to minimise the time required for works above the main railway tracks.**

**Dálnice D35 v úseku Časy–Ostrov prochází rovinatou krajinou východně od Pardubic. V místě železniční stanice Uhersko kříží dálnice s nízkou niveletou a pod ostrým úhlem koridorovou trať Praha – Česká Třebová. Pro přemostění šesti kolejí železniční trati byl navržen předpjatý betonový most se společnou nosnou konstrukcí pro oba směry dálnice. Spojitá nosná konstrukce o třech polích byla rozdělena na dvě části, které byly budovány rovnoběžně s železniční tratí a následně i s částí příslušenství byly protisměrně otočeny do definitivní polohy. Způsob výstavby otáčením byl zadefinován již v projektovém stupni dokumentace pro stavební povolení tak, aby byl minimalizován čas pro práce nad hlavními kolejemi koridoru.**

## BASIC PROJECT DATA

| | |
|---|---|
| TYPE OF CONSTRUCTION | Concrete prestressed continuous girder |
| SPANS | 50 + 72 + 50 m |
| WIDTH | 26,7 m |
| INVESTOR | Ředitelství silnic a dálnic ČR |
| BRIDGE DESIGNER | Link projekt s.r.o. |
| BRIDGE CONTRACTOR | EUROVIA CZ a.s. |
| CONSTRUCTION TIME | 2019–2022 |

## DESCRIPTION OF THE BRIDGE

The bridge crosses the railway at an angle of 34° and spans it with three spans of 50 + 72 + 50 m. The superstructure has a constant height of 3.40 m and a width of 26.35 m. The cross-section consists of a cast-in-situ central box girder, prefabricated cantilever struts, and a cast-in-situ deck slab. The resulting cross-section forms a three-cell configuration.

The bridge superstructure is longitudinally prestressed with bonded prestressing cables consisting of 18 strands. Another part of the longitudinal prestressing consists of unbonded cables with 25 strands, which are placed in the main box of the superstructure. In the transverse direction, the superstructure is prestressed with bonded cables consisting of 5 strands. The Freyssinet prestressing and anchoring system was used. For the installation of the prefabricated struts, threaded bars with a diameter of 40 mm from SAH/SAS SYSTEMS were used.

At both ends, the bridge connects to the motorway embankment, approximately 10 m high, via shaped concrete abutments with parallel wing walls. The confined conditions caused by the existing class III road required trimming the corner of the southern abutment by approximately 4 m and resulted in a significant cantilevering of the backwall nose.

Both abutments are founded on bored piles with a diameter of 1.2 m and are supplemented with inclined ground anchors 30–35 m in length.

The internal piers are massive wall-type concrete elements equipped with a pair of spherical bearings with a capacity

Fig. 1 View of the bridge
Obr. 1 Celkový pohled

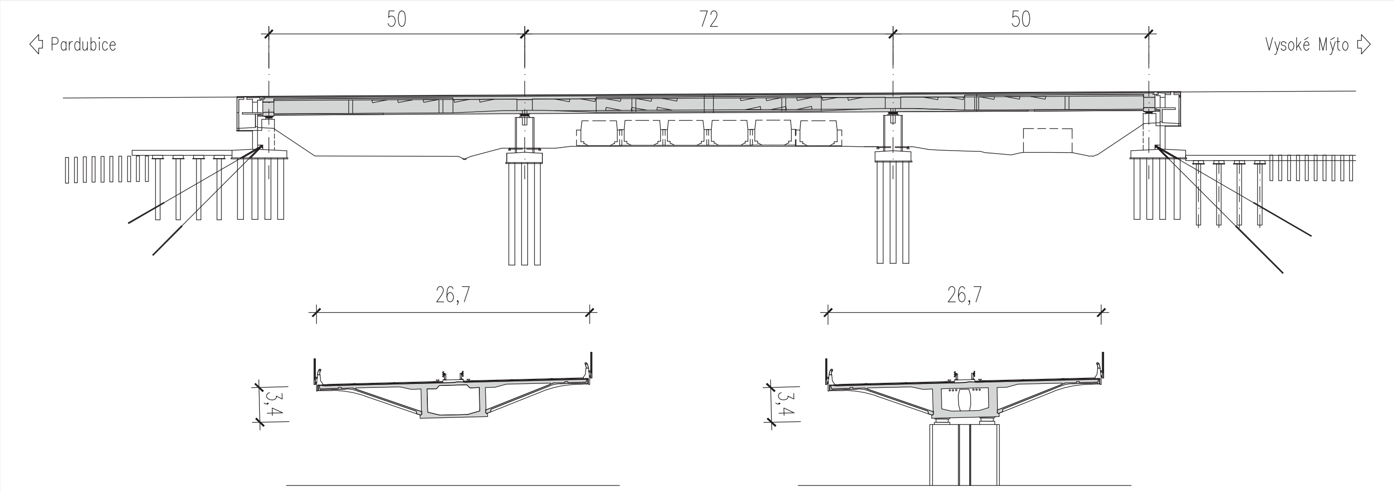



Fig. 2 Longitudinal and cross sections
Obr. 2 Podélný a příčný řezy

## ZÁKLADNÍ DATA PROJEKTU

| | |
|---|---|
| TYP KONSTRUKCE | Spojitý nosník z předpjatého betonu |
| ROZPĚTÍ | 50 + 72 + 50 m |
| ŠÍŘKA | 26,7 m |
| INVESTOR | Ředitelství silnic a dálnic ČR |
| PROJEKTANT MOSTU | Link projekt s.r.o. |
| ZHOTOVITEL MOSTU | EUROVIA CZ a.s. |
| DOBA VÝSTAVBY | 2019–2022 |

## POPIS MOSTU

Most kříží koridorovou trať pod úhlem 34° a přemosťuje ji třemi poli s rozpětími 50 + 72 + 50 m. Nosná konstrukce má konstantní výšku 3,40 m a šířku 26,35 m. Příčný řez je složen z monolitické páteřní komorové konstrukce, z prefabrikovaných konzolových vzpěr a monolitické horní desky. Výsledný příčný řez tak tvoří tříkomorový průřez.

Nosná konstrukce mostu je v podélném směru předepnuta soudržnými předpínacími kabely složenými z 18 lan. Další část podélného předpětí tvoří volně vedené předpínací kabely složené z 25 lan, které jsou umístěné v hlavní komoře nosné konstrukce. V příčném směru je nosná konstrukce mostu předepnuta soudržnými předpínacími kabely složenými z pěti lan. Byl použit předpínací a kotevní systém fy Freyssinet. Pro montáž prefabrikovaných vzpěr byly použity závitové tyče s průměrem 40 mm fy SAH/SAS SYSTEMS.

Most na obou koncích navazuje na cca 10 m vysoké těleso dálnice pomocí tvarovaných betonových opěr s rovnoběžnými křídly. Stísněné poměry ze strany stávající silnice III. třídy vedly k seřezání rohu jižní opěry o cca 4 m a k významnému vykonzolování nosu závěrné zídky. Současně bylo nutné zrealizovat šikmou opěrnou zeď z vyztužené zeminy o délce 45,5 m a výšce až 10,5 m.

Obě opěry jsou založeny na vrtaných pilotách průměru 1,2 m a jsou doplněny o šikmé zemní kotvy délky 30–35 m. Pro eliminaci sedání přechodových oblastí byly v prostoru za opěrami realizovány roznášecí platformy, které byly sestaveny z pole vrtaných pilot průměru 0,9 m a z roznášecích matrací tl.1,0 m.

Fig. 3 Section No.20
Obr. 3 Vahadlo 20

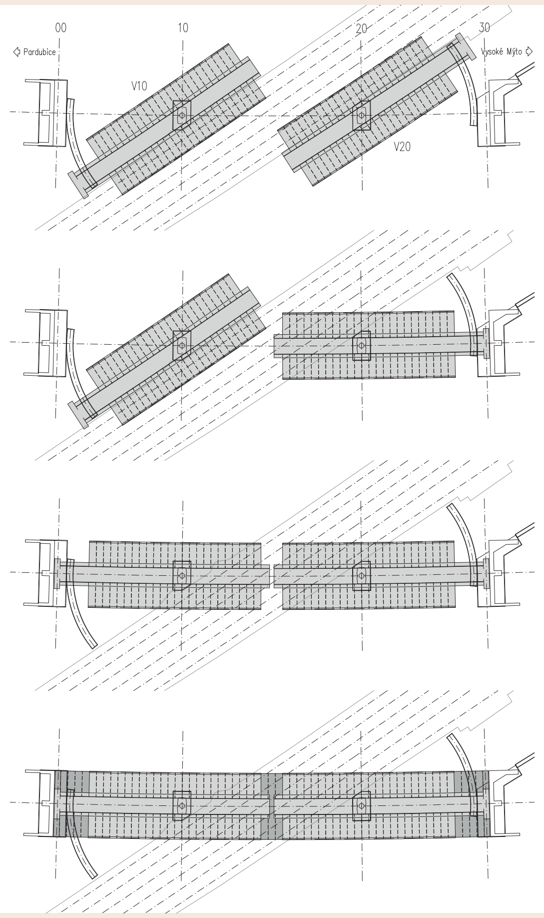
of 47 MN. A space was created between the bearing pedestal blocks to install sliding plates for the bridge superstructure rotation equipment.

## CONSTRUCTION

The key part of the bridge construction was the erection of the superstructure by rotation. The bridge structure was divided into two cantilevering sections with a length of 86.3 m. In their starting position, the two bridge sections were built parallel to the railway line at the locations of the inner piers and were partially equipped with accessories.

The temporary support of each section was provided at two locations—at the piers and at the turning tracks in front of the end cross-beams. The front temporary support on the piers consisted of four pots made of steel tubes filled with high-strength concrete. These pots rested on a sliding steel plate with a rotation radius of 1.1 m. The rear temporary support was located at the turning tracks, whose axis was 45.1 m from the rotation point on the piers. The rear support consisted of a steel structure fixed at its top into the bridge superstructure and sliding at its base on a temporary turning track. The track consisted of a reinforced-concrete foundation strip 1.3 m high, founded on bored piles with a diameter of 0.9 m.

Fig. 4 Construction diagram
Obr. 4 Schéma výstavby




The two sections were rotated into their final positions – by 34.7° (section 10) and by 32.4° (section 20 – using a hydraulic power unit with a 200-ton capacity cylinder. The power unit was positioned under the rear temporary support, with a movement step of 1.0 m. Afterward, both sections were lifted by 40 mm in their final position, the spherical bearings on the supports were activated, and the front and rear temporary supports were removed. Subsequently, the closing segment and the end parts of the superstructure were completed.

## CONCLUSION

The rotation of each section was carried out over two nights during railway possession periods in April and May 2022. The superstructure was completed and prestressed in June 2022. The entire D35 motorway section Časy–Ostrov was put into operation and officially opened in December 2022. The construction of a concrete superstructure by rotation is unique within the Czech Republic and represented a demanding task for all parties involved. It was successfully accomplished not only from a technical standpoint but also fully met the requirements of the construction schedule.

## MATERIAL USAGE (SUPERSTRUCTURE)

| | TOTAL | PER 1 M$^2$ |
|---|---|---|
| CONCRETE C45/55 | 3039 m$^3$ | 0,66 m$^3$ |
| PRECAST CONCRETE C50/60 | 548 m$^3$ | 0,12 m$^3$ |
| PRESTRESSING STEEL | 163 t | 35.5 kg |
| REINFORCING STEEL | 733 t | 160 kg |

Fig. 5 View of the rotated section No. 20 (photo: EUROVIA CZ a.s.)
Obr. 5 Pohled na otočené vahadlo 20 (foto: EUROVIA CZ a.s.)

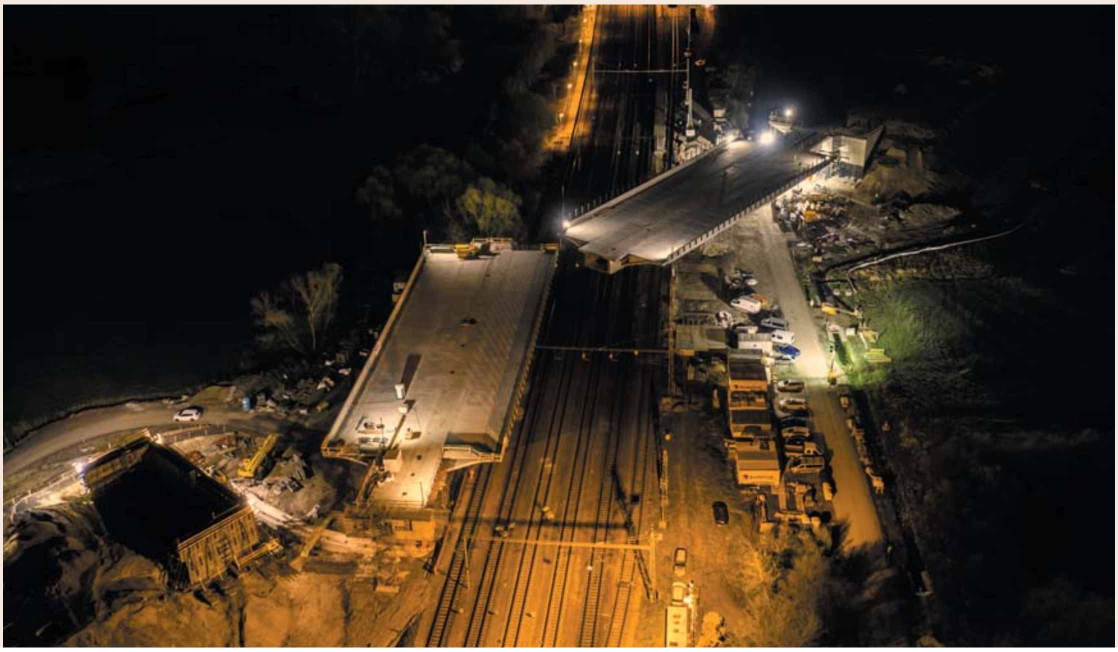


Fig. 6 Completed bridge
Obr. 6 Dokončený most

Vnitřní pilíře tvoří masivní stěnové betonové prvky s dvojicí kalotových ložisek s únosností 47 MN. Mezi podkladními bloky pod ložiska byl vytvořen prostor pro osazení kluzných plechů pro otáčecí zařízení nosné konstrukce.

## VÝSTAVBA

Klíčovou částí realizace mostu byla výstavba nosné konstrukce otáčením. Konstrukce mostu byla rozdělena na dvě vahadla délky 86,3 m. Vahadla ve startovací poloze byla umístěna rovnoběžně se železniční tratí v místech vnitřních pilířů a byla částečně vybavena příslušenstvím.

Montážní podepření vahadel bylo provedeno na dvou místech – v místě pilířů a v místě otáčecích drah před koncovými příčníky. Přední montážní podepření na pilířích tvořila čtveřice hrnců vytvořených z ocelové trubky s výplní z vysokopevnostního betonu. Hrnce byly uloženy na kluzné ocelové desce s poloměrem otáčení 1,1 m. Zadní montážní podepření se nacházelo v místě otáčecích drah, jejichž osa byla vzdálena 45,1 m od bodu otáčení na pilířích. Zadní podpěru tvořila ocelová konstrukce, která byla v hlavě vetknuta do nosné konstrukce mostu a v patě kluzně uložena na dočasnou otáčecí dráhu. Dráhu tvořil železobetonový základový pás vysoký 1,3 m založený na vrtaných pilotách průměru 0,9 m.

Vahadla byla otočena do své finální polohy o 34,7° (vahadlo 10) a o 32,4° (vahadlo 20) pomocí hydraulické pohonné jednotky s válcem nosnosti 200 t. Pohonná jednotka byla umístěna pod zadní montážní podpěru s krokem pohybu 1,0 m. Po následném nadzdvižení vahadel ve finální poloze o 40 mm byla aktivována kalotová ložiska na podpěrách a odstraněna přední a zadní montážní podepření. Následně byla dobudována uzavírací lamela a koncové části nosné konstrukce.

## ZÁVĚR

Otočení každého z vahadel proběhlo ve dvou dnech v nočních výlukách v průběhu dubna a května 2022. Nosná konstrukce byla dobudována a předepnuta v červnu 2022. Celý úsek dálnice D35 Časy–Ostrov byl zprovozněn a oficiálně otevřen v prosinci 2022. Výstavba otáčením betonové nosné konstrukce je v České republice unikátní a představovala pro všechny účastníky stavby náročný úkol, který byl zvládnut nejen technicky, ale byly dodrženy i požadavky časového harmonogramu výstavby.

Fig. 7 South abutment
Obr. 7 Jižní opěra

Fig. 8 View of the soffit at the south abutment
Obr. 8 Podhled u jižní opěry

Fig. 9 View into the side chamber
Obr. 9 Pohled do krajní komory

### SPOTŘEBA MATERIÁLU (NOSNÁ KONSTRUKCE)

| | CELKEM | NA $1M^2$ |
|---|---|---|
| BETON C45/55 | 3039 $m^3$ | 0,66 $m^3$ |
| PREFA BETON C50/60 | 548 $m^3$ | 0,12 $m^3$ |
| PŘEDPÍNACÍ VÝZTUŽ | 163 t | 35,5 kg |
| BETONÁŘSKÁ VÝZTUŽ | 733 t | 160 kg |

# BRIDGE OVER THE SKALICE RIVER
# MOST PŘES ŘEKU SKALICI

05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**The bridge on the D4 motorway spans the Skalice River, a railway and local roads. The bridge, with a total length of 404.10 m, carries both directions of the motorway on a single bridge structure consisting of a spine box girder with very large overhangs. The 29.10 m wide deck, and a variable depth from 2.60 to 4.50 m, consists of a continuous girder of nine spans of lengths from 34.80 to 70.00 m. The bridge deck was built incrementally, first the spine box girder was cast on stationary scaffolding, then precast struts were suspended on the box girder, and the overhangs were incrementally cast into movable formwork supported by these struts. The motorway was built as a PPP project.**

**Most na dálnici D4 přemosťuje řeku Skalici, železnici a místní komunikace. Most celkové délky 404,10 m převádí oba směry dálnice na jedné mostní konstrukci, tvořené páteřovým komorovým nosníkem s velmi vyloženými konzolami. Mostovku šířky 29,10 m a proměnné výšky od 2,60 až 4,50 m tvoří spojitý nosník o devíti polích s rozpětími od 34,80 do 70,00 m. Mostovka byla vytvářela postupně, nejdříve se na pevné skruži vybetonoval páteřní komorový nosník, poté se na nosník zavěsily prefabrikované vzpěry a do pojízdného bednění podepřeného těmito vzpěrami se postupně vybetonovaly vnější konzoly. Dálnice byla postavena jako PPP projekt.**

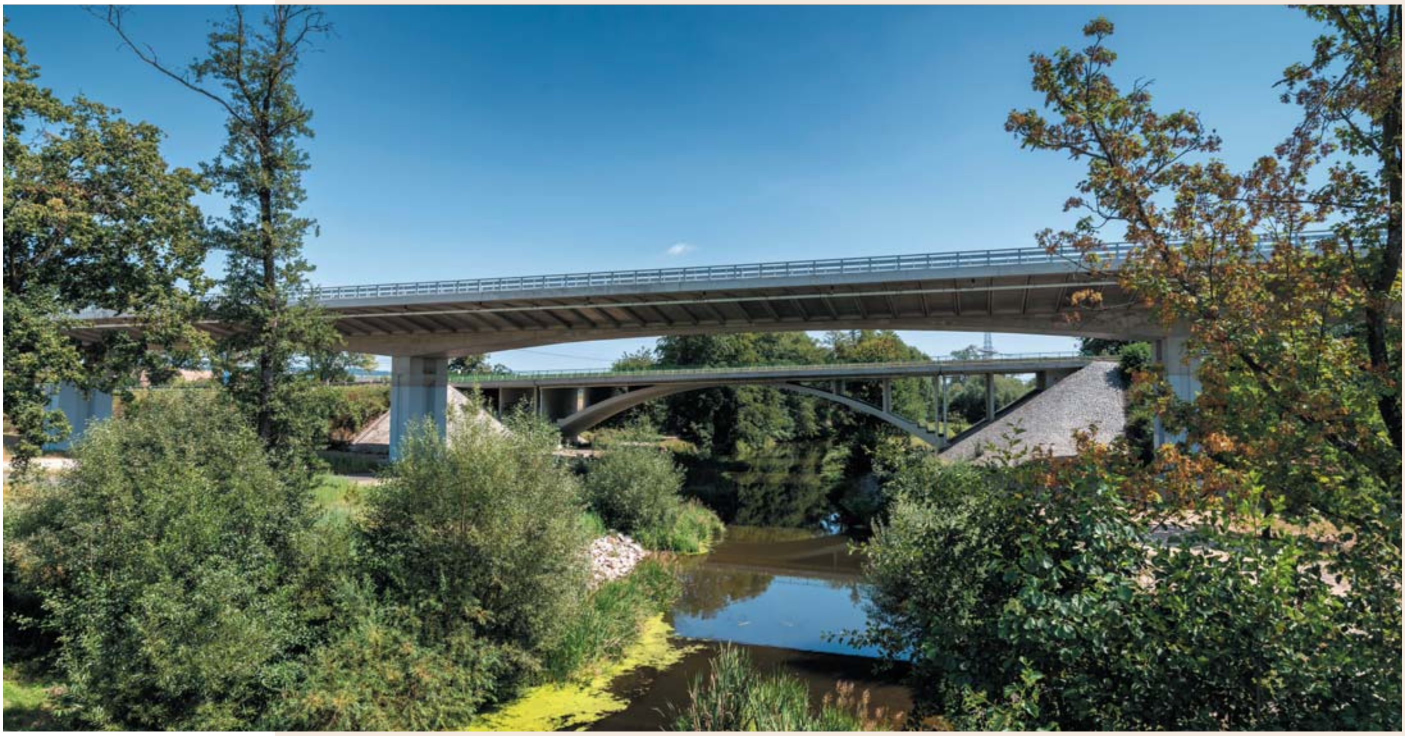


Fig. 1 Bridge over the Skalice River
Obr. 1 Most přes řeku Skalici

## INTRODUCTION

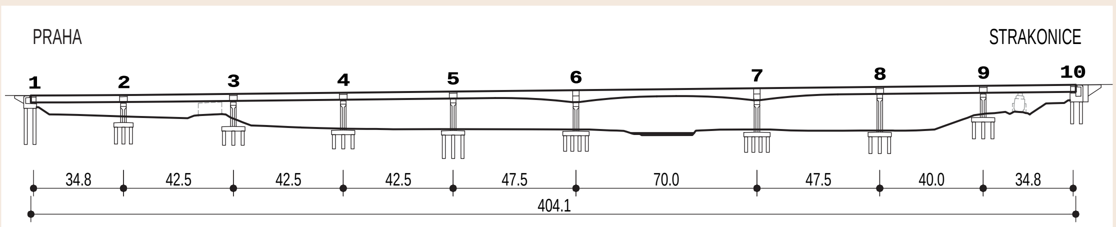
Near the small village of Nerestce the motorway D4 bridges the Skalice River, a railway and several local roads. The motorway is situated between a local road connecting two parts of the village (Horní and Dolní Nerestce) and the highway II/604 – see Fig. 1. While the local road crosses the Skalice river by a beautiful three span Empire arch bridge built in 1814, the highway crosses the river on a slender concrete arch bridge. Both bridges represent outstanding structures of the time where they were constructed. Not to disturb the views of both bridges, the motorway bridge crosses the river by a bold 70.00 m long haunch span, which opens the space above the river.

## DESIGN AND CONSTRUCTION

The bridge, which carries the motorway in both directions, forms a continuous girder of nine spans of lengths 34.8 + 4×42.5 + 47.5 +70.0 + 47.5 + 40.0 + 34.5 m – see Fig. 2. The deck with a width of 29.10 m is assembled of a 8.30 m wide spine girder and 2 × 10.4 m long overhangs – see Fig. 3. The approach spans have a constant depth of 2.66 m, the central three spans have a variable depth from 2.66 to 4.56 m. The width of the bottom slab of 6.5 m is constant along the whole length.

The bridge deck was built incrementally, first the spine box girder was cast on stationary scaffolding, then precast

Fig. 2 Elevation
Obr. 2 Podélný řez



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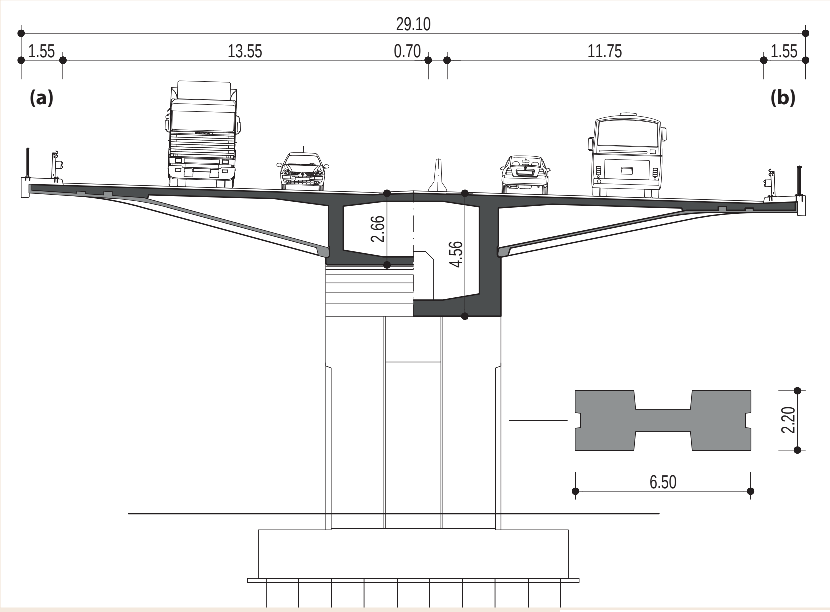
struts were suspended on the box girder (see Fig. 4), and the overhangs were incrementally cast into movable formwork supported by these struts. The spine girder was cast span-by-span with overhanging cantilevers in formwork supported by a stationary scaffolding. The precast struts were erected, and overhangs were cast two spans beyond the casting spans.

The deck was incrementally prestressed by continuous internal bonded tendons formed of 2 × 6 × 19-0.6″ strands which were coupled at the construction joints. Above piers 6 and 7 four short tendons of 19-0.6″ strands were added at the top slab. When the overhangs were cast, the transverse tendons formed by 5 0.6″ strands situated at a distance of 1.1 m were installed and post-tensioned. Once all spans were completed, the 2×2 external cables of 31-0.6″ strands situated inside the box and running along the whole bridge length were installed and post-tensioned. These cables are deviated at the pier and span diaphragms and are anchored at the end cross beams. In spans 5 through 7 an additional two external cables of 31-0.6″ strands are added.

The deck is supported by piers formed by two rectangular columns mutually connected by shear walls. While the piers 6 and 7 are frame connected with the deck, the piers 5 and 8 are hinge connected with the deck. On abutments and remaining piers, the deck sits on couples of pot bearings. All supports are founded on drilled piles.

## CONCLUSIONS

The viaduct forms a structurally efficient structure – see Fig. 5. By using one structure for both motorway directions allows to design axial piers which minimally impact the surroundings. It allows opening of the space under the bridge and to have an undisturbed view on beautiful neighbouring engineering structures.




The bridge, which is a part of the motorway D4 section Skalka-Krašovice, was built as a PPP project from June 2021 through to December 2024. Concessionaire is Via Salis, Praha, its contractor was DIVia stavební s.r.o., Praha. The viaduct was designed by the firm Stráský, Hustý a partneři, Brno, the bridge was constructed by Eurovia CZ a.s., Praha.

Fig. 3 Cross sections: (a) mid-span, (b) at pier 6
Obr. 3 Příčný řez: (a) uprostřed rozpětí, (b) u podpěry 6

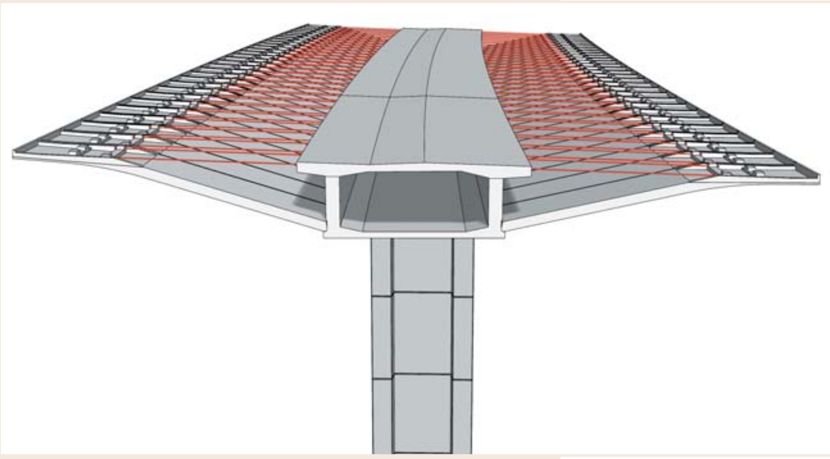


Fig. 4 Suspension of the struts on the spine girder
Obr. 4 Zavěšení vzpěr na páteřním nosníku

Fig. 5 Bridge over the Skalice River
Obr. 5 Most přes řeku Skalici

# D35 MOTORWAY VIADUCT ACROSS THE LITOMYŠL – MORAVSKÁ TŘEBOVÁ HIGHWAY

# VIADUKT DÁLNICE D35 PŘES SILNICI LITOMYŠL – MORAVSKÁ TŘEBOVÁ

06



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**The 429 m long viaduct, which bridges a busy road at an acute angle of 19.270, consists of two parallel bridges with a maximum span of 62 m. The crossing angle determined the length of the span bridging the highway and the staggered arrangement of supports. The bridge decks of both bridges are made of single-cell box girders, which were incrementally launched over obstacles. The economic design of the viaduct was made possible by its detailed static and dynamic analysis.**

**Viadukt délky 429 m, který v ostrém úhlu 19,270 překlenuje rušnou komunikaci, se skládá ze dvou souběžných mostů s maximálním rozpětím 62 m. Úhel křížení určil rozpětí pole přemosťujícího silnici a odstupňované uspořádání podpěr. Mostovky obou mostů jsou tvořeny jednokomorovými nosníky, které byly postupně vysouvány přes překážky. Ekonomický návrh viaduktu byl umožněn jeho podrobnou statickou a dynamickou analýzou.**

Fig. 1 Bridge incremental launching
Obr. 1 Postupné vysouvání mostu

## INTRODUCTION

The viaduct, which is a part of the recently completed D35 motorway section Janov-Opatovec, under an angle of skew crossing of 19,270 bridges the busy highway I/35 Litomyšl – Moravská Třebová, a local road and the Mikulečský Creek – see Fig. 1. The acute angle of crossing needs a minimum span length of 62 m and staggered pier arrangement. Heavy highway traffic required construction technology without any supporting members situated in the highway – see Fig. 2.

## ARCHITECTURAL AND STRUCTURAL DESIGN

The 429 m long viaduct is formed by two parallel bridges with eight bays. Due to the staggered arrangement of the piers, the span lengths of the right and left bridges are different. They are from 43.0 m to 62.0 m – see Fig. 3. Since the viaduct's axis is in a plan circle with a radius of 1,450 m and a constant longitudinal slope of 0.75%, it was possible to utilize technology of incremental launching for bridge construction.

The prestressed concrete deck of both bridges consists of 8-span continuous one cell box girders with a width of 13.80 m and depth of 2.93 m. During construction the girders were prestressed by straight Tensacciai bonded tendons of 7, 12 and 19 of 0.6″ strands situated in the top and bottom slabs. The tendons are led over two segments so that approximately 50% of them were tensioned and coupled in each construction joint.

When he bridge launching was completed, the girders were prestressed by external cables formed 19 of 0.6″ strands which are led along the whole bridge length and are anchored at the end diaphragms. In spans longer than 60 m additional cables of 19-0.6″ strands are situated. These cables are anchored at span diaphragms of neighboring spans. All external cables are deviated at the pier and span diaphragms.

The girders are stiffened by pier diaphragms in which the manholes were made. To reduce the weight of the launched structure, and simplify formwork of the top slab, only the bottom half of their depths were cast in the yard. When launching was completed, the diaphragm top parts were cast. The span diaphragms are formed by bottom blocks stiffened by additionally cast ribs transferring the cable radial forces into the girder webs.

The bridge decks are supported by spherical bearings. The fixed bearings are on piers 4, 5 and 6. All other bearings are longitudinally sliding bearings. The piers are formed by columns of the rectangle cross section lightened by vertical ribs, which transfers into column caps. Due to poor geotechnical conditions all supports are founded on 15 to 22 m long drilled piles.

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The viaduct's economical design was made possible by its detailed static and dynamic analysis. The bridge, which was analyzed by the MIDAS software system, was modeled as a 3D structure assembled from beam elements. A detailed time-dependent analysis of the gradually erected structure was performed for the designed and actual construction process. Reinforcement of important details were checked by a strut and tie analysis. Function of the viaduct was verified by static loading tests.

Fig. 2 Launching over the Litomyšl – Moravská Třebová Highway
Obr. 2 Vysouvání nad silnicí Litomyšl – Moravská Třebová

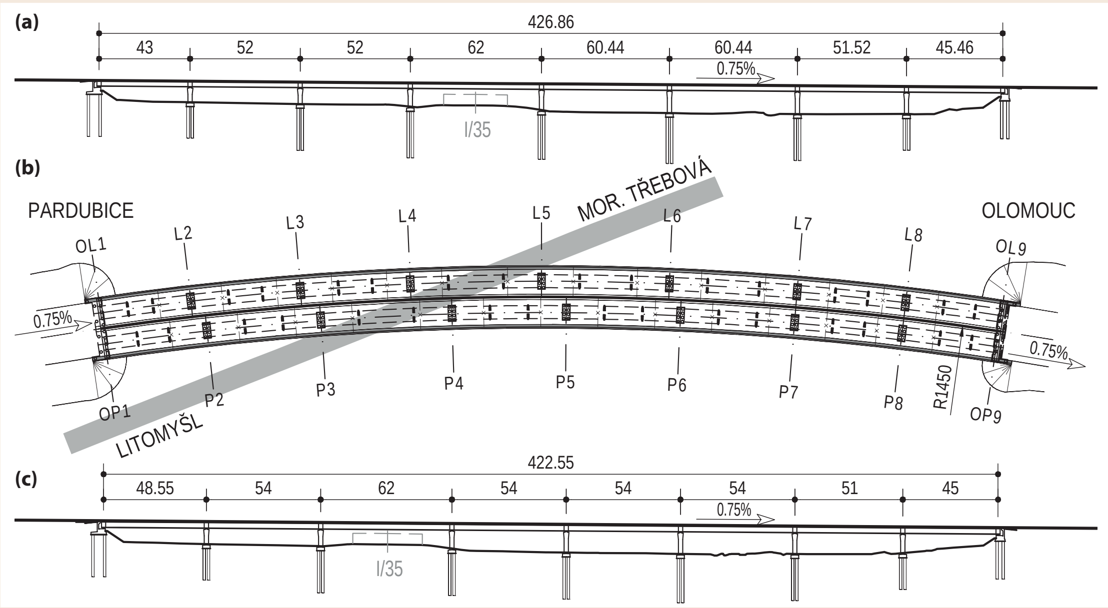



Fig. 3 Viaduct: (a) elevation of the left bridge, (b) plan, (c) elevation of the right bridge
Obr. 3 Viadukt: (a) podélný řez levým mostem, (b) půdorys, (c) podélný řez pravým mostem

## VIADUCT CONSTRUCTION

The technology of construction was developed by the contractor. The bridge decks were incrementally launched in the direction from the abutment 1 to 9, static effects in the launched girders were reduced by a steel nose prestressed to the decks. Both bridges were divided into 15 segments of lengths from 24.00 to 32.55 m.

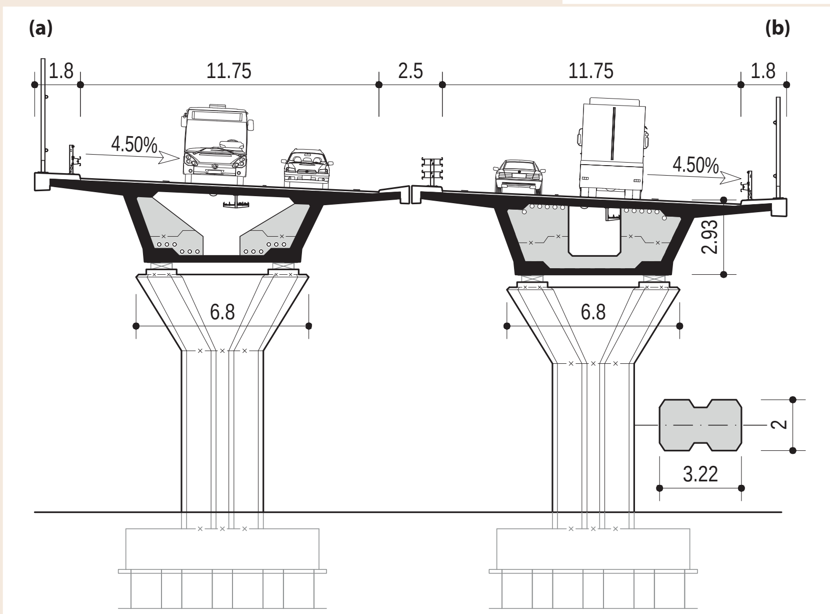
The deck was incrementally cast in a formwork situated beyond abutment 1. The box girder was cast in two stages, at first, the bottom slab and webs, then the top slab. As soon as coupled continuous tendons were post-tensioned, the whole structure was incrementally pushed by an Eberspächer AH190 device across the obstacle. As soon as the girder reached its final position, the diaphragms were completed and the structure was prestressed by external cables. At first, the left bridge was constructed, then the right bridge. Construction of the viaduct started in November 2023, and was completed in December 2025.

## CONCLUSIONS

The viaduct forms a structurally efficient and architecturally pleasing structure. The slender deck (the ratio of the deck depth d = 2.93 m to span length L = 62 m is 1/21.20) was constructed with minimum highway interruptions and minimum influence on the environment.

The client is Ředitelství silnic a dálnic (Directorate of Highways and Motorways) s. p., Praha. The viaduct was designed by the firm Stráský, Hustý a partneři, Brno, the motorway's section was built by Společnost (Joint Venture) D35 Janov – Opatovec, the viaduct was constructed by MI Roads a.s., Praha.

Fig. 4 Cross section: (a) at midspan, (b) at piers
Obr. 4 Příčný řez: (a) uprostřed rozpětí, (b) u podpěr



# II/268 KLÁŠTER HRADIŠTĚ N. J., MOST EV. Č. 268-007 PŘES JIZERU PŘED OBCÍ KLÁŠTER HRADIŠTĚ NAD JIZEROU

07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**Construction of the new bridge on road II/268 between the municipalities of Mnichovo Hradiště and Klášter Hradiště nad Jizerou was carried out between 2023 and 2024. The structure transfers road II/268 in a category S9.5/90, spanning the Jizera River floodplain, the main river channel, the Zábrdka stream, and two field access roads aligned with the original bridge route. This nine-span bridge features a post-tensioned concrete double-T girder superstructure that transitions into a closed box girder section over the main span. The bridge thereby spans the Jizera River with a single 60.1 m haunched span. At 312 m total length, it ranks among the longest bridges under the administration of the Central Bohemian Region.**

**Realizace nového mostu na silnici II/268 mezi obcemi Mnichovo Hradiště a Klášter Hradiště nad Jizerou probíhala v letech 2023 až 2024. Převádí silnici II/268 v šířkovém uspořádání S9,5/90 přes inundační území řeky Jizery, vlastní koryto řeky, potok Zábrdka a dvě polní cesty v trase původního mostu. Jedná se o most o devíti polích s železobetonovou, předpjatou dvoutrámovou nosnou konstrukcí, která v nábězích hlavního pole přechází do uzavřeného komorového průřezu. Most tak překonává řeku Jizeru jedním klenutým polem s rozpětím 60,1 m. Se svojí délkou 312 m se most řadí mezi nejdelší ve správě Středočeského kraje.**

Fig. 1 Aerial view of the bridge
Obr. 1 Pohled shora

## BASIC PROJECT DATA

| | |
|---|---|
| TYPE OF STRUCTURE: | Continuous post-tensioned concrete girder with variable cross-section and longitudinal haunch. Basic double-T girder cross-section transitions to box girder in spans 6, 7, and 8 over riverbank piers. |
| TOTAL LENGTH: | 312 m |
| SPAN ARRANGEMENT: | 29.50 + 30.00 + 2×30.05 + 2×30.00 + 60.10 + 30.00 + 29.50 m |
| WIDTH: | 12.60 m |
| CLIENT: | Regional Road Administration and Maintenance of the Central Bohemian Region, public organization |
| BRIDGE DESIGNER: | Pragoprojekt, a.s. |
| CONTRACTOR: | OHLA ŽS, a.s. |
| CONSTRUCTION PERIOD: | 12/2022 – 11/2024 |

## INTRODUCTION

The original bridge between the municipalities of Mnichovo Hradiště and Klášter Hradiště nad Jizerou transferred road II/268 across the Jizera River floodplain, the main channel, the Zábrdka stream, and two field access roads. Due to its unsatisfactory structural and technical condition, it was replaced by a new structure.

The new nine-span bridge follows the original alignment on road II/268 in category S9.5/90. It features a continuous post-tensioned concrete girder with variable cross section and longitudinal haunch. Compared to the original design, pier P8 in the Jizera River was removed, creating a single 60.10 m long haunched span over the river. This modification allows unobstructed use of the river channel beneath the bridge, eliminating the risk of debris accumulation (such as logs, branches, or other driftwood) on the upstream face of the pier, as occurred with the previous structure. Pier P8 in the Jizera channel was fully demolished down to the existing riverbed level.

Furthermore, the superstructure, originally divided into three independent expansion joints separated by bridge expansion joints, was redesigned and constructed as a single continuous unit with fixed bearings at the riverbank piers P7 and P9.

## STRUCTURAL DESIGN

The new bridge utilizes existing pile foundations reinforced with additional bored piles beneath pier foundations. The bridge abutments incorporate three original 1.8 m diameter piles, 9.1 m long at OP1, 12.5 m at OP11. For piers P7 and P9 adjacent to the Jizera riverbanks, between which spans the new 60.10 m span, four 1.2 m diameter piles were added to the pair of original 1.8 m diameter piles. For piers P2–P6 and P10, only two 1.2 m diameter piles were added to the pair of original 1.8 m diameter piles. Pile lengths range from 8.0 to 10.0 m, depending on the depth of the load bearing stratum.

Fig. 2 Main span of the bridge across the Jizera River
Obr. 2 Hlavní pole mostu přes Jizeru

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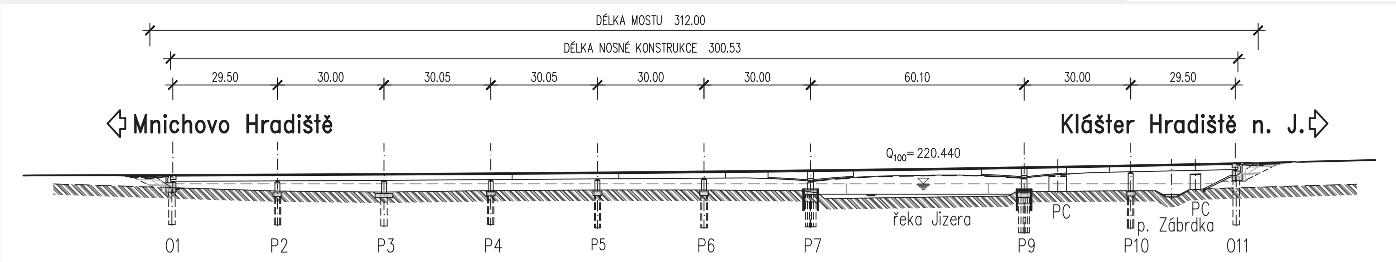


Fig. 3 Longitudinal section
Obr. 3 Podélný řez

## ZÁKLADNÍ DATA PROJEKTU

| | |
|---|---|
| TYP KONSTRUKCE: | Spojitý nosník z předpjatého betonu s proměnným příčným řezem a podélným náběhem. Základní příčný řez je dvoutrámový, který přechází v polích 6, 7 a 8 do komorového průřezu nad břehovými pilíři. |
| DÉLKA: | 312 m |
| ROZPĚTÍ POLÍ: | 29,50 + 30,00 + 2 × 30,05 + 2 × 30,00 + 60,10 + 30,00 + 29,50 m |
| ŠÍŘKA: | 12,60 m |
| INVESTOR: | Krajská správa a údržba silnic Středočeského kraje, p.o. |
| PROJEKTANT: | Pragoprojekt, a.s. |
| DODAVATEL: | OHLA ŽS, a.s. |
| DOBA VÝSTAVBY: | 12/2022–11/2024 |

## ÚVOD

Původní most mezi obcemi Mnichovo Hradiště a Klášter Hradiště nad Jizerou sloužil pro převedení silnice II/268 přes inundační území řeky Jizery, vlastní koryto řeky, potok Zábrdka a dvě polní cesty a byl z důvodu nevyhovujícího stavebně-technického stavu nahrazen mostem novým.

Nový most o devíti polích byl navržen v trase původního mostu na silnici II/268 v šířkovém uspořádání S9,5/90. Jedná se o spojitou konstrukci z předpjatého betonu proměnného průřezu a s podélným náběhem. Oproti původnímu řešení byl odstraněn pilíř P8 v korytě Jizery, čímž vzniklo nad řekou jedno spojené klenuté pole s rozpětím 60,10 m. Toto opatření umožnilo využití říčního koryta pod mostem v plném profilu, bez rizika zachycení kmenů nebo větví stromů a dalších naplavenin na návodní straně pilíře, jako tomu bylo u původního mostu. Pilíř P8 v korytě Jizery byl kompletně odstraněn až na stávající úroveň dna řeky. Dále byla nosná konstrukce navržena a zrealizována jako jeden dilatační celek s pevnými body uložení na břehových pilířích P7 a P9, oproti původnímu členění na tři samostatné dilatační části oddělené mostními dilatačními závěry.

## KONSTRUKČNÍ NÁVRH

Pro stavbu nového mostu bylo využito stávající pilotové založení, které bylo pod základy pilířů posíleno o nové vrtané piloty. Na opěrách mostu jsou tak tři původní piloty průměru 1,8 m. Jejich délka je na OP1 9,1 m a na OP11 12,5 m. Na pilířích P7 a P9 přiléhajících ke břehům koryta Jizery, mezi nimiž je nové pole délky 60,10 m, byly ke dvojici původních pilot průměru 1,8 m doplněny čtyři piloty o průměru 1,2 m. Na pilířích P2–P6 a P10 byly ke dvojici původních pilot průměru 1,8 m doplněny jen dvě piloty o průměru1,2 m. Délky pilot se pohybují od 8,0 do 10,0 m v závislosti na hloubce únosného skalního podloží.

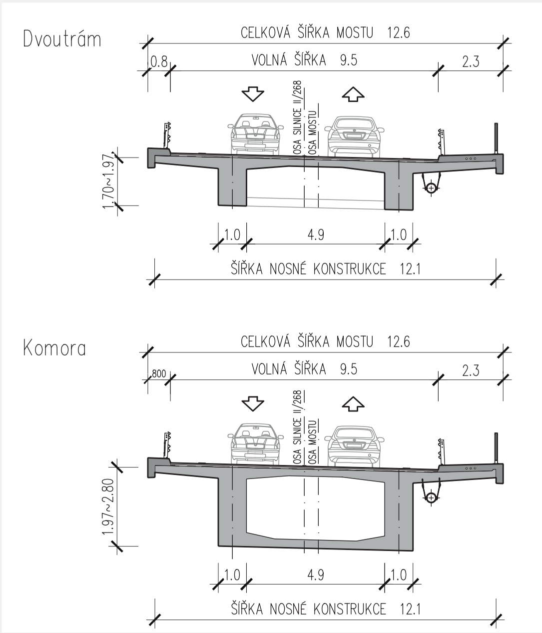


Fig. 4 Cross section
Obr. 4 Příčný řez

Dříky opěr jsou masivní s rovnoběžnými křídly a byly betonovány přímo na odbourané a očištěné hlavy pilot. Dříky pilířů tvoří dvojice železobetonových sloupů půdorysně obdélníkového tvaru rozměrů 1,4 × 1,2 m, vetknutých do základů a dále do pilot. Beton dříků pilířů je C30/37 – XF2, XD3, XC4.

Uložení nosné konstrukce je provedeno přes kalotová ložiska. Pevné body uložení jsou na pilířích P7 a P9. Ostatní

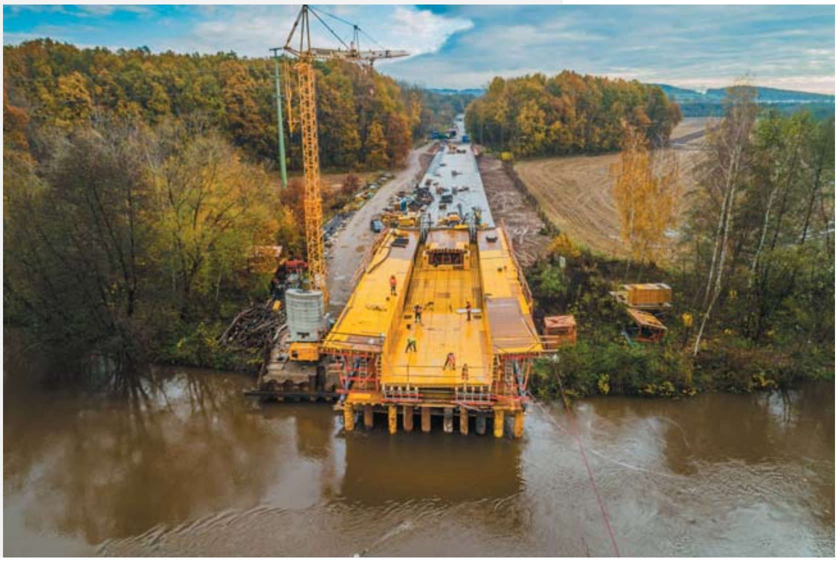

Fig. 5 Staged construction of the bridge
Obr. 5 Postupná výstavba mostu

Fig. 6 Overview of the site
Obr. 6 Celkový pohled na staveniště

The abutments are massive with parallel wings cast directly onto the demolished and cleaned pile heads. The piers consist of pairs of columns with a rectangular plan measuring 1.4 × 1.2 m, embedded into the foundations and further into the piles. The concrete class used for piers is C30/37 – XF2, XD3, XC4.

The superstructure is supported on spherical bearings. Fixed bearings are at piers P7 and P9. Other bearings are longitudinally guided with factory-pre-set positions corresponding to their distance from the theoretical fixed point, anticipated temperatures during the concreting period, and expected concrete creep and shrinkage.

The superstructure consists of a post-tensioned concrete double-T girder, transitioning to a closed box girder section in spans 6, 7, and 8 over the riverbank piers. The double girder height varies from 1.70 to 1.97 m, while the box girder height ranges from 1.97 to 2.80 m. Longitudinal post-tensioning uses continuous bonded 19 strand tendons of Ø15.7 mm with strength 1660/1860 MPa.

The total length of the new bridge is 312 m, with span arrangement 29.50 + 30.00 + 2×30.05 + 2×30.00 + 60.10 + 30.00 + 29.50 m. Multi-gap expansion joints are installed over the abutments. The overall width including edge beams is 12.6 m. The clear bridge carriageway width for 1+1 lanes is 9.5 m, with a 1.5 m wide public footpath on the right edge beam. The deck waterproofing system consists of a full-surface asphalt waterproofing membrane. The bridge carriageway features a double-layer of asphalt, 100 mm thick.

## CONSTRUCTION SEQUENCE

The start of construction was conditional on protection of hibernating wildlife in the locality. Two winter periods with adverse climatic conditions thus affected the construction process. The construction sequence was therefore adapted to these conditions – from demolition of the existing bridge, through sequential concreting of the substructure and superstructure, to the final concrete pour of the connecting main span over the Jizera River without requiring extreme actions to ensure quality requirements.

In each individual stage, the abutments and piers of the substructure were concreted and bearings were installed. Subsequently, work on the next stage's substructure began during formwork preparation, reinforcement tying, and tendon installation of the superstructure. This overlapping approach maximized process efficiency and achieved maximum time savings through staged construction.

Total concrete volume of the bridge superstructure was 2215.73 m$^3$. Concreting was therefore divided into seven stages – each cast separately in sequence, post-tensioned, and attached to the previous segment to create a continuous structure. Construction stages progressed from abutment OP1 to pier P7 on the riverbank, then reversed from abutment OP11 to pier P9 on the opposite bank. The final stage was the concrete pour between piers P7 and P9, forming the 60 m river span. Non-integrated reinforced concrete edge beams were then cast along the entire bridge.

The bridge superstructure was constructed using C35/45 – XF2, XD1, XC4 – Cl 0.10 – Dmax 22 – S4 concrete, with requirements for determining the actual static modulus of elasticity for post-tensioning and its real value after 28 days for precise static calculations in the project. Given the low pier heights, the superstructure was cast using falsework. For the connecting span over the river between piers P7 and P9, the foundation of the demolished pier P8 in the middle of the river was utilized to support a temporary structure of steel girders ŽBM 30, which spanned the river in both directions. At piers P7 and P9, the falsework was hinged on the already-cast cantilevered segments of the superstructure.

## CONCLUSION

Construction in the riverbed required river vessels and specialized underwater technologies operated by divers. Despite these challenging conditions, environmental impact was minimal, and the nearby natural monument 'Skalní sruby Jizery' remained completely undisturbed. The bridge sensitively blends into the landscape; its subtle profile prevents visual disruption despite the low clearance. The structure was handed over to the client several weeks ahead of the contract schedule. Following successful verification of static assumptions and quality through a static load test on November 6, 2024, the bridge was ceremonially opened to traffic in mid-November 2024.

## CONSUMPTION OF MATERIALS (SUPERSTRUCTURE)

| | TOTAL | PER 1 M$^2$ |
|---|---|---|
| CONCRETE C35/40 | 2215.73 m$^3$ | 0.568 m$^3$ |
| PRESTRESSING STEEL | 113.41 t | 29.1 kg |
| REINFORCING STEEL | 377.81 t | 96.9 kg |

ložiska jsou podélně posuvná s přednastavením z výroby odpovídajícím vzdálenosti od teoretického pevného bodu, předpokládané teplotě v návaznosti na období betonáže a předpokládanému smrštění a dotvarování betonu. Nosnou konstrukci tvoří železobetonový, dodatečně předepjatý dvoutrám, který přechází v polích 6, 7 a 8 do uzavřeného komorového průřezu nad břehovými pilíři. Výška dvoutrámové konstrukce je 1,70–1,97 m a komorové konstrukce 1,97–2,80 m. K podélnému předpětí jsou použity průběžné kabely složené z 19 lan o průměru 15,7 mm a s pevností 1660/1860 MPa.. Celková délka nového mostu je 312,0 m s rozpětím polí 29,50 + 30,00 + 2 × 30,05 + 2 × 30,00 + 60,10 + 30,00 + 29,50 m. Nad opěrami jsou osazeny vícelamelové mostní závěry. Celková šířka mostu včetně říms je 12,6 m. Volná šířka mostu pro 1 + 1 jízdní pruh je 9,5 m, veřejný chodník na římse vpravo má šířku 1,5 m. Izolace mostovky je celoplošná z asfaltových izolačních pásů. Vozovka na mostě je asfaltová dvouvrstvá, tl. 100 mm.

## POSTUP VÝSTAVBY

Zahájení prací bylo podmíněno zajištěním ochrany přezimujících živočichů v této lokalitě. Do doby realizace nám tak vstoupilo hned dvakrát zimní období s nepříznivými klimatickými podmínkami. Postup výstavby tak bylo nutné přizpůsobit stanoveným podmínkám od samotné demolice stávajícího mostu, přes postupné betonáže spodní stavby a nosné konstrukce až po finální betonáž posledního spojujícího pole nad řekou Jizerou, bez nutnosti extrémních opatření pro zajištění kvalitativních požadavků.

V každé z jednotlivých etap byly vybetonovány opěry a pilíře spodní stavby a osazena ložiska. Následně byly v průběhu přípravy bednění, vyvázání betonářské oceli a instalaci předpínací výztuže nosné konstrukce zahájeny práce na spodní stavbě další etapy tak, aby se celý proces maximálně zkrátil a zvolenou etapizací výstavby se dosáhlo maximální časové úspory.

Celkový objem betonu nosné konstrukce mostu byl 2215,73 m$^3$. Bylo tak nutné rozdělit betonáže stavby do sedmi etap, které byly postupně samostatně betonovány, následně předpínány a zároveň spínány s předchozí etapou v jednu spolupůsobící konstrukci. Betonáže postupovaly směrem od opěry OP1 k pilíři P7 na břehu řeky. Další etapy probíhaly v opačném směru od opěry OP11 k pilíři P9. Poslední etapou byla betonáž mezi pilíři P7 a P9 spojující 60m pole nad řekou. Na závěr byly na celém mostě vybetonovány železobetonové římsy.

Nosná konstrukce mostu byla zhotovena z betonu C 35/45 – XF2, XD1, XC4 – Cl 0,10 – Dmax 22 – S4 s požadavkem na stanovení skutečného statického modulu pružnosti pro předpínání a po 28 dnech pro přesný statický výpočet do realizačního projektu stavby. Vzhledem k nízké výšce pilířů byla zvolena betonáž nosné konstrukce na prostorové skruži. Pouze ve spojujícím poli nad řekou mezi P7 a P9 bylo využito základu ze zrušeného pilíře P8 uprostřed řeky pro podepření provizorní konstrukce z inventárních ocelových nosníků ŽBM 30, pomocí které byla překlenuta řeka v obou směrech. Skruž pak u pilířů P7 a P9 byla zavěšena na již vybetonované, konzolovitě vyložené části nosné konstrukce.

Fig. 7 Falsework of the main span
Obr. 7 Skruž hlavního pole

Fig. 8 Completed structure – left bank
Obr. 8 Dokončená konstrukce – levý břeh

## ZÁVĚR

Postup výstavby při provádění prací v prostoru říčního koryta si vyžádal nasazení říčních plavidel i speciálních podvodních technologií ovládaných potápěči. I přes tyto složité podmínky byl dopad na okolí stavby minimální a nebyla nikterak dotčena přírodní památka Skalní sruby Jizery, nacházející se v těsné blízkosti stavby. Most byl citlivě zakomponován do krajiny a díky štíhlosti konstrukce i přes malou výšku nepůsobí rušivým dojmem. Dílo bylo předáno objednateli s několikatýdenním předstihem oproti termínu ze smlouvy o dílo. Po úspěšném ověření statických předpokladů a kvality, provedením statické zatěžovací zkoušky dne 6. 11. 2024, byl slavnostně spuštěn provoz v polovině listopadu 2024.

Fig. 9 Integration of the bridge into the landscape
Obr. 9 Začlenění mostu do krajiny

## SPOTŘEBA MATERIÁLU (NOSNÁ KONSTRUKCE)

| | CELKEM | NA 1M$^2$ |
|---|---|---|
| BETON C35/40 | 2215,73 m$^3$ | 0,568 m$^3$ |
| PŘEDPÍNACÍ VÝZTUŽ | 113,41 t | 29,1 kg |
| BETONÁŘSKÁ VÝZTUŽ | 377,81 t | 96,9 kg |

# BRIDGE OVER THE SÁZAVA RIVER ON THE HAVLÍČKŮV BROD BYPASS
# MOST PŘES SÁZAVU NA OBCHVATU HAVLÍČKOVA BRODU

08

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**The bridge over the Sázava River and two railway lines is located on the newly built southeastern bypass of Havlíčkův Brod. The bridge structure is 275.2 m long and consists of 8 spans with a typical span of 37 m. It is a prestressed concrete beam, which is placed on high piers on a pair of bearings. With its simple shapes, the structure fits elegantly into the landscape. The construction process of the bridge was very specific, with the 7th span being constructed in an elevated position 2 m above the vertical aligment. Subsequently, this span was lowered to its final position and the subsequent spans were concreted.**

**Most přes řeku Sázavu a dvě kolejové tratě se nachází na nově vybudovaném jihovýchodním obchvatu města Havlíčkův Brod. Mostní konstrukce je dlouhá 275,2 m a je tvořena osmi poli o typickém rozpětí 37 m. Jedná se o předepnutý betonový trám, který je uložen na vysokých pilířích na dvojici ložisek. Konstrukce svými jednoduchými tvary elegantně zapadá do krajiny. Postup výstavby mostu byl velmi specifický, kdy sedmé pole bylo vybetonováno ve zvýšené poloze 2 m nad niveletou. Následně toto pole bylo spuštěno do definitivní polohy a proběhlo dobetonování krajního pole. Poté postupnou betonáží proběhla výstavba až k opěře 1.**

Fig. 1 View of the completed structure
Obr. 1 Pohled na dokončenou konstrukci

The new bridge on road I//38 is the first structure of the new southeast bypass of Havlíčkův Brod and directly connects to the urban development.

The bridge, with a total length of 275.2 m, crosses the non-electrified Havlíčkův Brod – Pardubice railway line, the Sázava River, the edge of the Termesivy asphalt plant, and the main Brno – Havlíčkův Brod railway line with three electrified tracks. The obstacles to be bridged provided a clear basis for the placement of the supports.

## FOUNDATIONS

Piers P3 – P6 are founded on piles with a diameter of 0.9 m and a length of approx. 5.0 m, taking into account the rock subsoil conditions. The shorter piers P7 and P8 along the electrified line to Brno are founded on a flat foundation. The foundation of pier P2 was very complicated, mainly due to its location. Pier P2 is situated in a very steep slope between the non-electrified railway line and the Sázava River. After conducting exploratory work and assessing the access, technology of a flat foundation in combination with a deep foundation was chosen. Due to the steep slope of the rock subsoil, part of the foundation was constructed as a concrete plug connected to the rock environment by steel micropiles with 108/16 mm pipe reinforcement.

## SUBSTRUCTURE

The piers are octagonal reinforced concrete columns with an extension at the top for the placement of a pair of bearings. The column head is formed by flat surfaces that protrude directly from the octagonal shape. The piers reach a height of up to 15 m at the inundation area of the Sázava River. The same octagonal shape with a circumference of 2.2 m has been chosen for all piers. The reinforced concrete abutments are founded on a flat surface.

## SUPERSTRUCTURE

The superstructure is designed as a continuous eight-span beam, a monolithic structure prestressed in the

Fig. 2 Longitudinal section
Obr. 2 Podélný řez

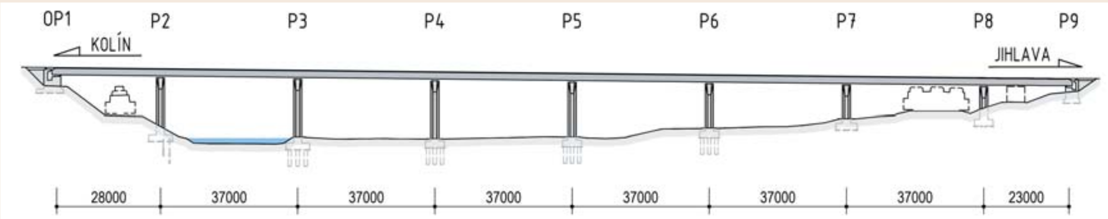
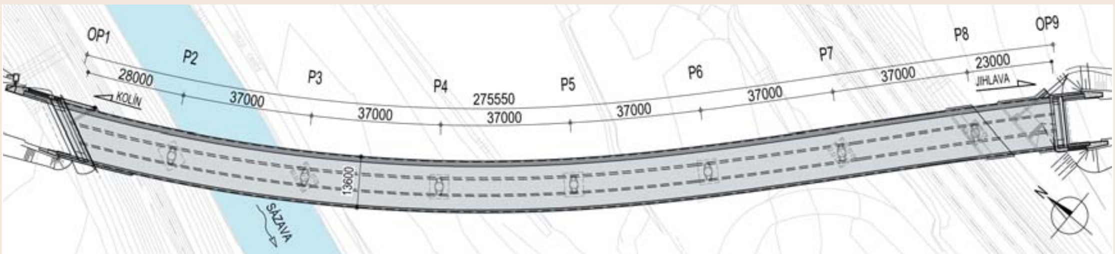



Fig. 3 Plan of the bridge
Obr. 3 Půdorys mostu



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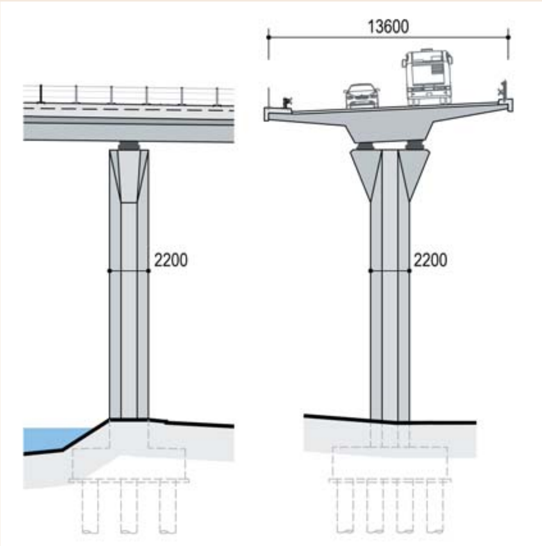


Fig. 4 Bridge piers
Obr. 4 Mostní pilíře

longitudinal direction and acting as reinforced concrete in the transverse direction. The cross-section is a constant height of 1.8 m. The cantilevers are 4.26 m and 4.34 m long, and the thickness of the cantilevers at the beam connection is 800 mm. The total width of the supporting structure is 14.10 m. The top surface of the superstructure and the bottom surface of the beam are aligned with the transverse slope of the roadway, typically 4.6%; from the end of the fifth span, the surface follows the transition of the roadway to a right-hand transverse slope of 5.0%.

The construction of the superstructure began at the second-to-last span, which was above an electrified railway line. Due to the limited space available in this area, it was necessary to concrete and prestress the superstructure in an elevated position +2.0 m. The length of the first phase was 53.0 m, with a span of 37.0 m.

The bridge span was lowered using hydraulic cylinders, which were placed on braced columns mounted on bearing blocks. The weight of the structure being lowered was 1,600 tons.

Subsequently, the entire structure was lowered into its final position. Then the outer span 8 was concreted. The construction of the other spans was carried out using a standard system on a fixed falsework.

Due to the unconventional construction method for a simple span to both sides, the arrangement of the prestressing cables had to be modified.

The prestressing of the lowered span was provided by straight cables, and only after the adjacent parts had been concreted was the structure prestressed with elevated cables. The following stages were prestressed in the standard way – 50% of the cables in one phase, the remaining 50% of the cables in the next phase.

The bridge accessories consist of standard bridge guardrails and railings. Girder grid expansion joints are located on the abutments. The spatial arrangement of the expansion joints was particularly complex on the OP1 abutment, as they had to take into account the large skew (55 g) supplemented by the spatial movement of the superstructure due to the plan arrangement in the radius and the stiffness of the relatively high piers.

As part of a pilot project by the Road and Motorway Directorate, the bridge is equipped with a comprehensive system for monitoring the behavior of the structure in relation to traffic loads. Optical strain sensors supplemented by conventional string strain gauges are installed in selected sections. All wiring is routed in protective ducts in a wide cornice and is led through modified bridge expansion joints to the enclosed abutment area. A control box for all electrical equipment is located at the inspection area of the abutment.

In addition to integrated response monitoring points, a camera surveillance system for vehicle identification and speed measurement is located in front of the bridge.

## SUMMARY

Despite all the technological challenges, the bridge structure was created with the aim of safely transferring traffic over obstacles, but also with the intention of disturbing the surrounding landscape as little as possible.

Fig. 5 Elevated structure of 7th span before lowering
Obr. 5 Zvýšená konstrukce sedmého pole před spuštěním

Fig. 6 View of the completed structure
Obr. 6 Pohled na dokončenou konstrukci

# ARCH OVERPASSES ON SECTION 0311 OF THE D3 MOTORWAY, TŘEBONÍN–KAPLICE-NÁDRAŽÍ

## OBLOUKOVÉ NADJEZDY NA ÚSEKU D3 0311 TŘEBONÍN–KAPLICE-NÁDRAŽÍ

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**This article presents a pair of reinforced concrete arch overpasses constructed on Section 0311 Třebonín–Kaplice nádraží of the D3 motorway. The structures are based on similar structural principles but differ in the degree of integral action: Bridge No. 209 is designed as a fully integral bridge, while Bridge No. 212 uses a semi-integral structural system. The article focuses primarily on the structural design and construction of these concrete bridges, including the overall structural arrangement, foundations, substructure, and superstructure, as well as essential aspects of the construction process. Emphasis is placed on the configuration of the load-bearing arch system and the detailing and arrangement of concrete hinges, which play a key role in the structural behaviour of both bridges.**

**Příspěvek se zabývá dvojicí železobetonových obloukových nadjezdů realizovaných na úseku D3 0311 Třebonín–Kaplice nádraží. Obě konstrukce vycházejí z obdobných konstrukčních principů, liší se však mírou integrálního uspořádání. Mostní objekt č. 209 je navržen jako plně integrovaný most, zatímco mostní objekt č. 212 je řešen jako semiintegrovaný. Článek je zaměřen především na návrh a realizaci těchto betonových mostů, včetně celkového konstrukčního uspořádání, založení, spodní stavby a nosné konstrukce, a na vybrané aspekty technologického postupu výstavby. Zvláštní pozornost je věnována uspořádání nosného obloukového systému a vrubových kloubů, které hrají klíčovou roli ve statickém působení obou konstrukcí.**

Fig. 5 Completed Bridge No. 212 before the opening for traffic
Obr. 5 Dokončený most č. 212 před uvedením do provozu

The pair of arch overpasses on the D3 Třebonín–Kaplice Railway Station section were unified in the implementation documentation stage into nearly identical structures, differing only in end abutment arrangement and geometry determined by road alignment, including longitudinal and transverse slope and bridge height over the D3 motorway – see Fig. 1–3. Both bridges are 6.6 m wide, with a clear width of 5.0 m to accommodate local roads over the motorway. The bridges are designed in a straight alignment with perpendicular arrangement. The structures consist of a shallow founded reinforced concrete arch, vertical piers, deck, and end abutments, described individually for each bridge in the following sections. Both bridges include 0.8 m wide reinforced concrete parapet with safety barriers on both sides and a two-layer pavement with full-area torch-on asphalt sheet waterproofing. Drainage is ensured by a combination of transverse and longitudinal slope, along with seven small stainless-steel pipes for waterproofing subsurface drainage. Additional bridge drains and longitudinal drainage pipes were not required according to the hydrotechnical calculation, which is beneficial for future bridge maintenance.

Integral Bridge No. 209. The total length of this bridge near the village of Netřebice is 67.55 m, with a constant longitudinal slope of 4.4 %. The arch span is 42.0 m with a rise of 5.52 m (L / 7.6). The span arrangement of the deck is 5.8 + 6.4 + 7.2 + 15.7 (connection with arch) + 8.4 + 8.2 + 9.3 m. It is a fully integral structure without bearings or expansion joints – see Fig. 4. The arch is shallow founded on 6.6 m wide reinforced concrete blocks, with an inclined foundation interface embedded into gneiss bedrock with varying degrees of weathering. The abutments are founded on a single row of piles with a narrowed shaft allowing expansion movement of the bridge ends without excessive pile stress. The piles are embedded on the length of 1.0 m into bedrock, while the remaining 4.5 m of shaft was reduced using 90 mm thick polystyrene lost formwork inserted into the borehole

Dvojice obloukových nadjezdů na úseku dálnice D3 Třebonín–Kaplice nádraží byla v rámci realizační dokumentace sjednocena na tvarově téměř shodné konstrukce, které se liší konstrukčním uspořádáním opěr a geometrií danou silničním řešením, jako je podélný a příčný sklon a výška mostu nad dálnicí D3 – viz Obr. 1–3. Šířka mostů je shodně 6,6 m a volná šířka 5,0 m zajištuje převedení polních cest přes dálnici. Mosty jsou navrženy směrově v přímém směru s kolmým uspořádáním. Konstrukce tvoří železobetonový, plošně založený oblouk, svislé stojky, mostovka a krajní opěry, jejichž řešení bude popsáno u jednotlivých mostů v následujících podkapitolách. Součástí obou mostů jsou železobetonové římsy šířky 0,8 m se zábradelním svodidlem po obou stranách a dvouvrstvá vozovka s celoplošnou izolací z asfaltových pásů na pečetící vrstvě. Odvodnění je zajištěno kombinací příčného a podélného sklonu společně se sedmi trubičkami pro odvodnění izolace. Mostní odvodňovače a podélný svod odvodnění dle hydrotechnického výpočtu nebyly vyžadovány, což je vhodné opatření pro budoucí správu a údržbu mostu.

Integrovaný most č. 209. Celková délka mostu poblíž obce Netřebice je 67,55 m a podélný sklon je konstantní 4,4 %. Rozpětí oblouku činí 42,0 m při vzepětí 5,52 m (L / 7,6). Rozpětí jednotlivých polí mostovky je 5,8 + 6,4 + 7,2 + 15,7 (spojení s obloukem) + 8,4 + 8,2 + 9,3 m. Jedná se o zcela integrovanou konstrukci bez mostních ložisek a závěrů – viz Obr. 4. Oblouk je založen plošně na masivních železobetonových blocích šířky 6,6 m s nakloněnou základovou spárou uloženou do skalního podloží tvořeného rulami různé míry zvětrání. Opěry spočívají na řadě pilot se zúženým dříkem, což umožňuje dilatační

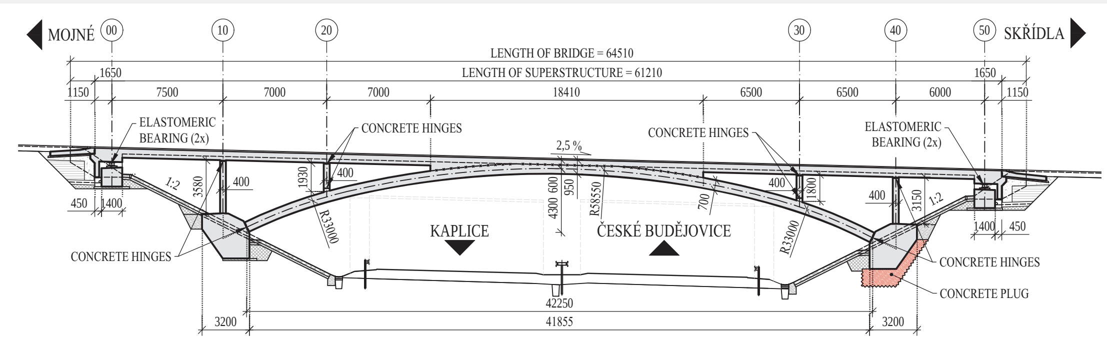


Fig. 1 Longitudinal section of Bridge No. 209
Obr. 1 Podélný řez mostu č. 209

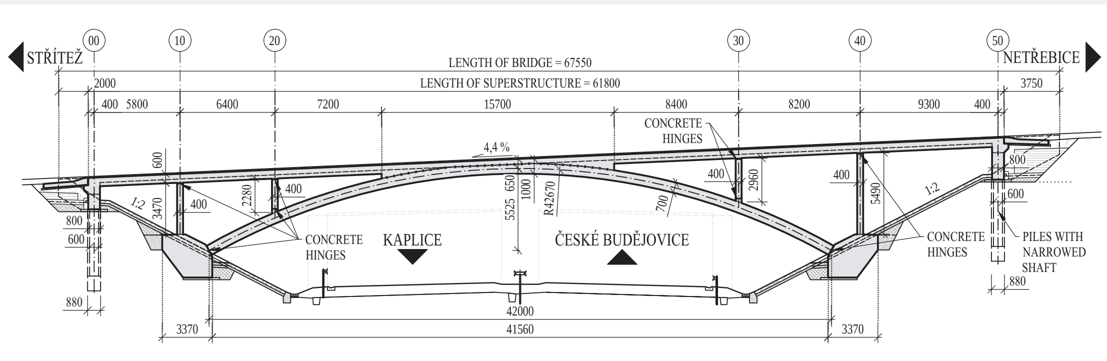


Fig. 2 Longitudinal section of Bridge No. 212
Obr. 2 Podélný řez mostu č. 212

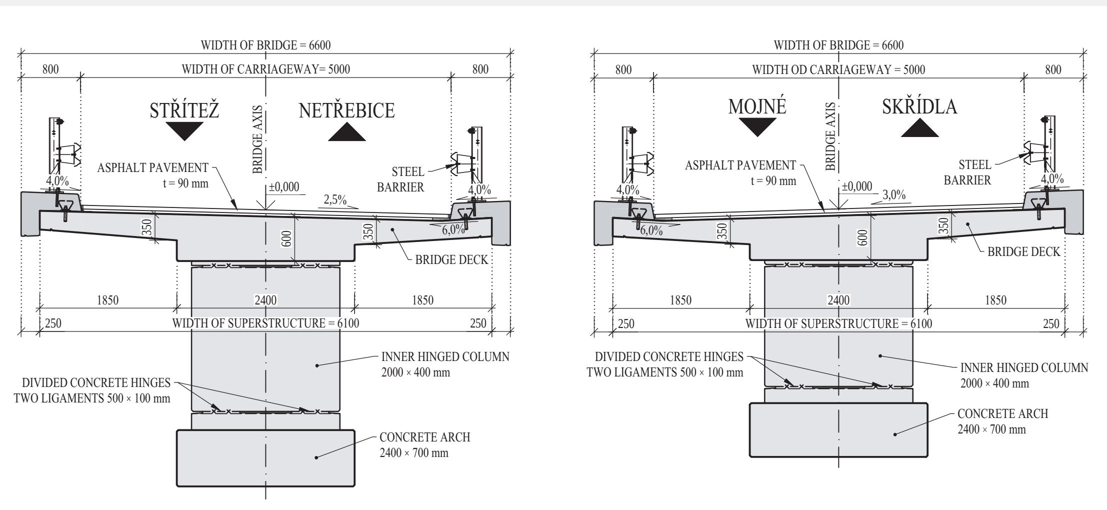


Fig. 3 Cross-Sections – left Bridge No. 209, right Bridge No. 212
Obr. 3 Příčné řezy – vlevo most č. 209, vpravo most č. 212

together with reinforcement. Due to the pressure of fresh concrete, the polystyrene was reinforced with steel straps. The 110 mm void between the polystyrene and the temporary casing was filled with hand-placed loose sand.

The substructure also includes a pair of outer piers embedded into the arch foundations. The pier cross-section is rectangular, 2.0 × 0.4 m. At the deck connection, concrete hinges were designed to reduce horizontal stiffness. The area of the joint was minimized due to low normal loads and designed as a pair of ligaments (0.5 × 0.1 m) separated by a polystyrene-filled gap. The superstructure consists of a two-hinged arch with concrete hinges at the abutments. The arch cross-section is a constant rectangle with dimensions 0.7 × 2.4 m. After evaluating several alternatives for the arch axis geometry, a circular shape with a radius of 42.67 m was chosen, as the differences in internal forces were negligible with respect to maintaining the clearance profile. The inner piers with a 0.4 × 2.0 m cross-section, are designed with concrete hinges at both ends. The deck is a reinforced concrete beam 0.6 m high and 2.4 m wide with 1.85 m cantilevers. Total deck width is 6.1 m. All structural components are made of C40/50 concrete.

Semi-integral Bridge No. 212. The bridge is located near Dolní Třebonín village at the beginning of section D3 0311. Its total length is 64.51 m, and the arch span is 42.25 m with a rise of 4.3 m (L / 9.8). The longitudinal slope is constant at 2.5 %. The bridge deck is divided into spans of 7.5 + 7.0 + 7.0 + 18.41 + 6.5 + 6.5 + 6.0 m. The structure is designed as semi-integral, with bearings at the end abutments and no expansion joints – see Fig. 5. The arch foundation is also shallow founded on massive reinforced concrete blocks. At Pier No. 40, the subgrade predicted by the engineering-geological survey was not encountered, and considering the required bearing capacity, it was necessary to rehabilitate the subgrade by removing the weathered rock. This space was subsequently filled with a concrete plug. Unlike Bridge No. 209, the end abutments could not be founded on narrowed shaft piles due to higher hard rock levels which would be unexcavatable. Therefore, a semi-integral abutment was chosen, consisting of an end crossbeam supporting wing walls, retaining walls, and transition slab, resting on two elastomeric bearings on a shallow founded abutment.

The arch cross-section is identical at 0.7 × 2.4 m. However, the arch axis geometry was adjusted due to the significantly lower bridge height. The shape consists of multiple radii: 58.5 m in the mid span, reducing to 33.0 m beyond the inner hinged piers. This configuration increases the rise and reduces internal forces in critical sections while maintaining the clearance profile for the motorway– see Fig. 6. Both inner and outer piers have a rectangular 0.4 × 2.0 m cross-section with the same concrete hinge arrangement as Bridge No. 209. The deck cross-section is identical, differing only by a larger transverse slope of 3.0 %.

The superstructure was cast in three stages on stationary falsework – see Fig 7. Construction started in March 2023 and reached completion in December 2024. The bridge behaviour was assessed by static load tests, demonstrating compliance with the design assumptions and the quality of execution. Since December 2024, the bridges have been open to traffic and have been operating without any difficulties.

### MATERIAL USAGE (BRIDGE NO. 209 – ARCH, BRIDGE DECK, INNER PIERS)

| | TOTAL | PER 1M² |
|---|---|---|
| CONCRETE C40/50 | 225.9 m³ | 0.599 m³ |
| REINFORCING STEEL | 37.7 t | 100.1 kg |

Fig. 4 Completed Bridge No. 209 before opening to traffic
Obr. 4 Dokončený most č. 209 před uvedením do provozu

Fig. 6 Detail of the arch load-bearing structure of Bridge No. 212
Obr. 6 Detail obloukové nosné konstrukce mostu č. 212

pohyb konců mostu bez nadměrného namáhání pilot. Piloty jsou vetknuty do skalního podloží v délce 1,0 m, zatímco zbývajících 4,5 m dříku je zúženo pomocí polystyrenového ztraceného bednění vloženého do vrtu společně s armokošem. Kvůli tlaku čerstvého betonu musel být polystyren ztužen ocelovými pásky.

Spodní stavbu dále tvoří dvojice vnějších stojek vetknutých do základů oblouku. Průřez stojek je obdélník o rozměrech 2,0 × 0,4 m. U napojení na mostovku byly pro snížení vodorovné tuhosti navrženy vrubové klouby. Plocha vrubového kloubu je s ohledem na malé přitížení redukována a kloub je navržen jako členěný z dvojice krčků o rozměrech 0,5 × 0,1 m s mezerou vyplněnou polystyrenem. Nosnou konstrukci tvoří dvoukloubový oblouk s vrubovými klouby v patách. Jeho průřez je konstantní obdélníkový o rozměrech 0,7 × 2,4 m. Po posouzení několika variant tvaru střednice byl zvolen tvar kružnice s poloměrem 42,67 m, neboť rozdíly vnitřních sil byly s ohledem na dodržení průjezdného profilu zanedbatelné. Vnitřní stojky o průřezu 0,4 × 2,0 m jsou navrženy jako kyvné s vrubovými klouby na obou koncích. Mostovka je tvořena železobetonovým trámem výšky 0,6 m, šířky 2,4 m s konzolami délky 1,85 m. Celková šířka mostovky činí 6,1 m. Veškeré části nosné konstrukce jsou z betonu C40/50.

Semiintegrovaný most č. 212. Most se nachází u obce Dolní Třebonín na začátku úseku D3 0311. Délka mostu je 64,51 m a rozpětí oblouku je 42,25 m při vzepětí 4,3 m (L / 9,8) Podélný sklon je konstantní 2,5 %. Mostovka je rozdělena na jednotlivá pole o rozpětí 7,5 + 7,0 + 7,0 + 18,41 (spojení s obloukem) + 6,5 + 6,5 + 6,0 m. Konstrukce je navržena jako semiintegrovaná s ložisky na krajních opěrách bez mostních závěrů – viz Obr. 5. Založení oblouku je taktéž plošné pomocí masivních železobetonových základů. U pilíře č. 40 musela být v tomto případě s ohledem na nezastižení předpokládaných hornin v základové spáře provedena náhrada zvětralé horniny plombou z prostého betonu. Hlavním rozdílem oproti mostu č. 209 je řešení krajních opěr, které s ohledem na vyšší úroveň skalního podloží nemohly být založeny na vrtaných pilotách se zúženým dříkem. Proto bylo zvoleno semiintegrované zakončení mostu, které tvoří koncový příčník, nesoucí křídla, plentovací zídky a přechodová deska. Tento koncový příčník je na nízké, plošně založené opěře uložen na dvojici elastomerových ložisek.

Průřez oblouku je shodně 0,7 × 2,4 m, avšak střednice byla v tomto případě s ohledem na výrazně nižší výšku mostu upravena. Střednice oblouku má tvar kružnice složené z více poloměrů – ve střední části je poloměr 58,5 m a za kyvnými stojkami se zmenšuje na 33,0 m. Toto řešení umožňuje zvýšit vzepětí a snížit vnitřní síly v rozhodujících průřezech při zachování dostatečného průjezdného profilu – viz Obr. 6. Vnitřní i vnější stojky mají obdélníkový průřez 0,4 × 2,0 m a mají shodné uspořádání vrubových kloubů jako most SO 209. Mostovka je taktéž konstrukčně totožná, liší se pouze větším příčným sklonem 3,0 %.

Nosná konstrukce byla betonována ve třech fázích na pevné prostorové skruži – viz Obr. 7. Výstavba byla zahájena v březnu roku 2023 s dokončením v prosinci roku 2024. Chování mostu bylo ověřeno statickou zatěžovací zkouškou, která prokázala shodu s předpoklady návrhu a kvalitu provedení. Od prosince roku 2024 jsou mosty uvedeny do provozu a dosud se nevyskytly žádné provozní problémy.

### SPOTŘEBA MATERIÁLU (MOST Č. 209 – OBLOUK, MOSTOVKA, VNITŘNÍ STOJKY)

| | CELKEM | NA 1M$^2$ |
|---|---|---|
| BETON C40/50 | 225,9 m$^3$ | 0,599 m$^3$ |
| BETONÁŘSKÁ VÝZTUŽ | 37,7 t | 100,1 kg |

Fig. 7 Bridge No. 209 during construction of the arch
Obr. 7 Most č. 209 během betonáže oblouku

# ECODUCT OVER THE D5509 NEAR BZENEC
# EKODUKT PŘES D5509 U BZENCE

10


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**The D5509 motorway passes through pine forests near the South Moravian town of Bzenec. In the flat landscape, it became necessary to build an ecoduct to allow wildlife to cross and to carry a forest road over the motorway, with a minimum overfill height of 2.1 m. A two-span reinforced-concrete frame structure with a broken shape was chosen for the motorway overpass. In the portal sections, the structure is designed so that no wing walls or retaining walls are needed to support the embankment. The result is a prototype ecoduct with a flat structure that does not require significant horizontal forces to be transferred into the subsoil. The design keeps the surface of the embankment at the lowest possible height, blending it naturally into the surrounding landscape.**

**Dálnice D5509 prochází borovicovými lesy v okolí jihomoravského města Bzenec. V rovinaté krajině vznikla potřeba vybudovat ekodukt pro přechod zvěře a převedení lesní cesty přes dálnici, a to s nadnásypem o minimální výšce 2,1 m. Pro přemostění dálnice byla zvolena dvoupolová železobetonová rámová konstrukce s lomeným tvarem. V portálových částech je konstrukce navržena tak, aby nebylo nutné doplňovat křídla či opěrné zdi pro zachycení nadnásypu. Byl tak vytvořen prototyp ekoduktu s plochou konstrukcí a bez nároků na přenesení větších vodorovných sil do podzákladí. Konstrukce drží povrch nadnásypu v nejnižší možné výšce a navazuje na okolní krajinu.**

## BASIC PROJECT DATA

| | |
|---|---|
| TYPE OF CONSTRUCTION | Reinforced concrete frame |
| SPANS | 19 + 19 m |
| WIDTH | 54,0 – 67,2 m |
| INVESTOR | Ředitelství silnic a dálnic ČR |
| BRIDGE DESIGNER | Link projekt s.r.o. |
| BRIDGE CONTRACTOR | SKANSKA a.s. |
| CONSTRUCTION TIME | 2023–2024 |

## DESCRIPTION OF THE BRIDGE

A location within a local biocorridor at the edge of the forest northeast of the town of Bzenec was selected for a wildlife crossing over the motorway. The biocorridor was defined as having a width of 40 metres at the crest and 80 metres at the base of the embankment. Ecologists required a minimum overfill height of 2.1 m to allow for vegetation planting. Outside the ecoduct itself, the embankment of the biocorridor was designed with a slope of 25% and supplemented with a relocated forest road connecting the divided pine woodland.

From the outset, the concept of a flat frame structure was considered – one that would not necessitate an increase in the biocorridor's crest height, especially given the flat landscape and the imperative to minimise the footprint on woodland. The aim was to find a structural type that would not require the transfer of significant horizontal forces into the subsoil, which consists of Quaternary aeolian sands approximately 10 m thick, underlain by Neogene sandy clays.

The final design of the ecoduct is a reinforced concrete frame cast in place, consisting of a kinked slab with two spans of 19.0 m each. The angled sections at the edges of the structure ensure that it meets the motorway clearance requirements while reducing stresses in the horizontal part. The cross-section comprises a 0.6-metre-thick slab, strengthened above the central support and in the angled sections by a haunch that increases its depth to 0.9 metres.

The width of the superstructure ranges from 54.0 metres at the top to 67.2 metres at the base. The structure is divided

Fig. 1 General view
Obr. 1 Celkový pohled

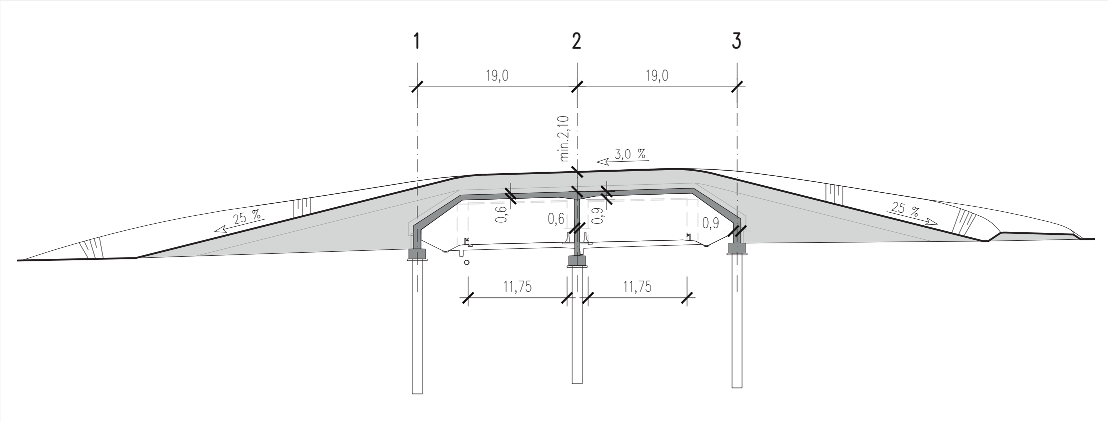


Fig. 2 Longitudinal section
Obr. 2 Podélný řez

## ZÁKLADNÍ DATA PROJEKTU

| | |
|---|---|
| TYP KONSTRUKCE | Železobetonová rámová konstrukce |
| ROZPĚTÍ | 19 + 19 m |
| ŠÍŘKA | 54,0 – 67,2 m |
| INVESTOR | Ředitelství silnic a dálnic ČR |
| PROJEKTANT MOSTU | Link projekt s.r.o. |
| ZHOTOVITEL MOSTU | SKANSKA a.s. |
| DOBA VÝSTAVBY | 2023–2024 |

## POPIS MOSTU

Na okraji lesa severovýchodně od města Bzenec bylo v rámci lokálního biokoridoru vybráno místo pro přechod zvěře přes dálnici. Prostor biokoridoru byl definován volnou šířkou 40 m v koruně a 80 m v patě násypu. Pro osázení zelení bylo dle požadavků ekologů nutné počítat s nadnásypem minimálně 2,1 m nad vodorovnou částí nosné konstrukce. Těleso násypu biokoridoru bylo mimo vlastní ekodukt navrženo se sklonem 25 % a doplněno o přeložku lesní cesty, která propojila rozdělený borovicový les.

Od počátku se uvažovalo s koncepcí ploché rámové konstrukce, která nebude vyžadovat zvýšení vrcholu biokoridoru, a to zejména s ohledem na rovinatý charakter krajiny a potřebu minimalizovat zábor lesních pozemků. Hledal se typ konstrukce bez nároků na přenesení větších vodorovných sil do podzákladí, které je v celé lokalitě tvořeno kvartérními eolickými písky mocnosti cca. 10 m s podložím z neogenních písčitých hlín.

Finální návrh ekoduktu představuje železobetonový monolitický rám se zalomenou horní příčlí o dvou polích s rozpětími 2 × 19,0 m. Zalomené části tvarují konstrukci v jejích krajích tak, aby respektovaly průjezdný profil dálnice a zároveň redukovaly namáhání vodorovné části. Příčný řez tvoří deska výšky 0,6 m, která se nad vnitřní podpěrou i v zalomených částech zesiluje náběhem do výšky 0,9 m.

Šířka nosné konstrukce se mění od 54,0 m ve vrcholu až po 67,2 m v patě. V příčném směru je konstrukce rozdělena dilatačními spárami do pěti dilatačních celků. Vnitřní tři

Fig. 3 View from the nottheast
Obr. 3 Pohled ze severovýchodu

Fig. 4 Bird's-eye view
Obr. 4 Pohled z ptačí perspektivy

transversely into five expansion units. The three central units are 10.8 m wide.

In the portal areas, the superstructure is shaped so that the cornices follow the 1:2 embankment slope along their entire length. The angled footings with walls support the appended parts of the structure. This angular geometry follows the spatial inclination of the structure's edge. This configuration creates a unified structural geometry, eliminating the need for additional wall or wing structures to support the embankment. The entire ecoduct structure is compact and monolithic, free from concrete hinges and bearings.

The motorway beneath the ecoduct has a transverse crossfall of 2.5%. The slope of the ‚horizontal' frame part was adjusted accordingly and set at 3.0%.

The central frame pier comprises ten trapezoidal shafts, each with a thickness of 0.6 m and a width ranging from 1.7 to 2.7 m. All three supports are founded on a single row of bored piles, each with a diameter of 1.2 m.

## CONSTRUCTION

Before construction began, consolidation embankments were placed in the area of the ecoduct to prevent excessive settlement during and after completion of the bridge. The consolidation process was monitored at two profiles equipped with horizontal inclinometers as part of the project's geotechnical monitoring. Once the embankments had been excavated and the deformations had stabilised, piling works commenced.

The superstructure was cast in situ on fixed formwork. Backfilling and compaction of the structure were carried out under enhanced supervision in a symmetrical arrangement and under geodetic control.

The superstructure was completed in October 2023, and the entire motorway section opened to traffic in a half profile in December 2024. The full profile was subsequently put into operation in July 2025.

## CONCLUSION

A prototype motorway ecoduct featuring a flat structural form that does not require the transfer of significant horizontal forces into the subsoil was constructed near the town of Bzenec. Despite the substantial height of the overfill, a slender, compact reinforced concrete structure could be designed that maintains the embankment surface at the lowest possible elevation, blending naturally into the surrounding landscape.

Fig. 5 Superstructure
Obr. 5 Nosná konstrukce

Fig. 6 Completed bridge
Obr. 6 Dokončený most

dilatační celky mají šířku 10,8 m. Krajní portálové celky mají ve vrcholu šířku 10,8 m s rozšířením k patám.

V portálových částech je nosná konstrukce vytvarována tak, aby po celé délce říms respektovala tvar nadnásypu se sklonem 1 : 2. Základy se stěnami jsou v této části zalomené a vynášejí apendixy nosné konstrukce. Zalomení sleduje prostorový sklon lemu nosné konstrukce. Tímto prostorovým vzorcem tak vzniká celistvá geometrie konstrukce bez potřeby dodatečných oddilatovaných stěn nebo křídel pro zachycení nadnásypu. Celá konstrukce ekoduktu je kompaktní, monolitická, bez vrubových kloubů a ložisek.

Dálnice je pod ekoduktem vedena v příčném jednostranném sklonu 2,5 %. Tomu byl přizpůsoben i sklon „vodorovné" rámové části a zvolen s ohledem na odtok podpovrchové vody jednostranně 3,0 %. V podélném směru dálnice je nosná rámová konstrukce vodorovná.

Středová rámová stojka je členěná. Je tvořena deseti dříky lichoběžníkového tvaru o tloušťce 0,6 m a šířce 1,7–2,7 m. Všechny tři podpěry jsou založeny vždy na jedné řadě vrtaných pilot o průměru 1 200 mm.

## VÝSTAVBA

Před zahájením výstavby byly v prostoru ekoduktu provedeny konsolidační násypy, jejichž cílem bylo eliminovat nadměrná sedání během a po dokončení stavby mostu. Vliv konsolidace byl sledován na dvou profilech s instalovanými horizontálními inklinometry a průběžně vyhodnocován v rámci geotechnického monitoringu stavby. Po ustálení deformací byly konsolidační násypy odtěženy a následně byly zahájeny práce na vrtání pilot.

Nosná konstrukce byla betonována na pevné skruži postupně po jednotlivých dilatačních celcích. Zásyp konstrukce a jeho hutnění probíhaly za zvýšeného dozoru, v symetrickém uspořádání a pod geodetickou kontrolou.

Nosná konstrukce byla dokončena v říjnu 2023 a celý úsek dálnice byl uveden do provozu v polovičním profilu v prosinci 2024. Plný profil byl následně zprovozněn v červenci 2025.

## ZÁVĚR

U města Bzenec byl zrealizován prototyp dálničního ekoduktu s plochou konstrukcí a bez nároků na přenesení větších vodorovných sil do podzákladí. I při vysokém nadnásypu se podařilo navrhnout subtilní a kompaktní železobetonovou konstrukci, která drží povrch nadnásypu v nejnižší možné výšce a navazuje na okolní krajinu.

Fig. 7 Portal section
Obr. 7 Portálová část

Fig. 8 Inside view
Obr. 8 Vnitřní pohled

# VIADUCT KRIVÁŇ–MÝTNA ON THE EXPRESSWAY R2, SLOVAKIA

# VIADUKT KRIVÁŇ–MÝTNA NA RYCHLOSTNÍ KOMUNIKACI R2 NA SLOVENSKU





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**The viaduct of a total length of 4.36 km is situated in an environmentally sensitive mountainous landscape between the towns of Kriváň and Mýtna. It runs along the slopes of the beautiful valley of the Kriváňský creek, which it crosses several times. The viaduct carries both directions of the 24.5 m wide R2 expressway on a single 27.50 wide bridge structure. The span lengths are from 60 to 150 m, the height of the piers is up to 35 m. Although the viaduct was built using three different technologies, it has a uniform architectural and structural solution along its entire length. The viaduct, which forms a semi-integral structural system, was built as a Design & Build project.**

**Viadukt celkové délky 4,36 km je situován v environmentálně citlivé horské krajině mezi městy Kriváň a Mýtna. Je veden na svazích krásného údolí Kriváňského potoka, který několikrát kříží. Viadukt převádí oba směry 24,5 m široké rychlostní silnice R2 na jedné mostní konstrukci široké 27,50 m. Rozpětí polí je od 60 do 150 m, výška podpěr je až 35 m. Ačkoliv viadukt byl stavěn třemi rozdílnými technologiemi, má po celé délce jednotné architektonické a konstrukční řešení. Viadukt, který tvoří semi-integrální konstrukční systém, byl postaven jako projekt Design & Build.**

Fig. 1 Bridge in the treetops – Viaduct Kriváň–Mýtna
Obr. 1 Most v korunách stromů – Viadukt Kriváň–Mýtna

## DEVELOPMENT OF THE STRUCTURAL TYPE

While at both ends the viaduct is led on the mountain slopes, the central portion crosses the existing highway and creek several times – see Fig. 1. That is why the span lengths of the viaduct's side parts are from 60 to 70 m, while the span lengths of middle part, due to the skew crossings of the creek and the highway, is from 70 to 150 m. It was evident that the viaduct's side parts can be cast in a movable scaffolding or can be incrementally launched, while the central part requires balanced cantilever construction.

From the beginning of the design, it was obvious that the one structure formed by a spine box girder with

Fig. 2 Viaduct structure
Obr. 2 Konstrukce viaduktu

Fig. 3 Progressive construction of the deck
Obr. 3 Postupná výstavba mostovky


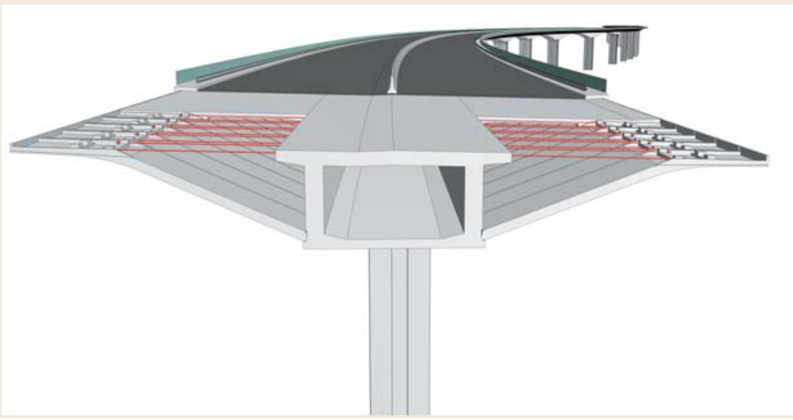

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Fig. 4 Spine girder & precast struts
Obr. 4 Páteřní nosník a prefabrikované vzpěry

large overhangs supported by narrow piers represents an optimum solution – see Fig. 2. This structure requires minimum excavation at the steep slopes and creates a clear and clean pier order, minimally disturbing the beautiful countryside. However, the designer had to prove that in the case of repair of one carriageway, all traffic can be transferred to the another one. Along the whole viaduct length, the width of the box girder's bottom slab is 6.50 m.

To reduce the weight of the construction equipment, it was decided to construct the bridge deck incrementally.

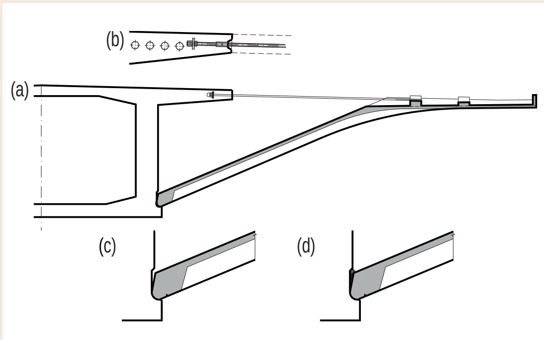



Fig. 5. Suspension of the precast struts
Obr. 5 Zavěšení prefabrikovaných vzpěr

Fig. 6 Suspension of the precast struts
Obr. 6 Zavěšení prefabrikovaných vzpěr

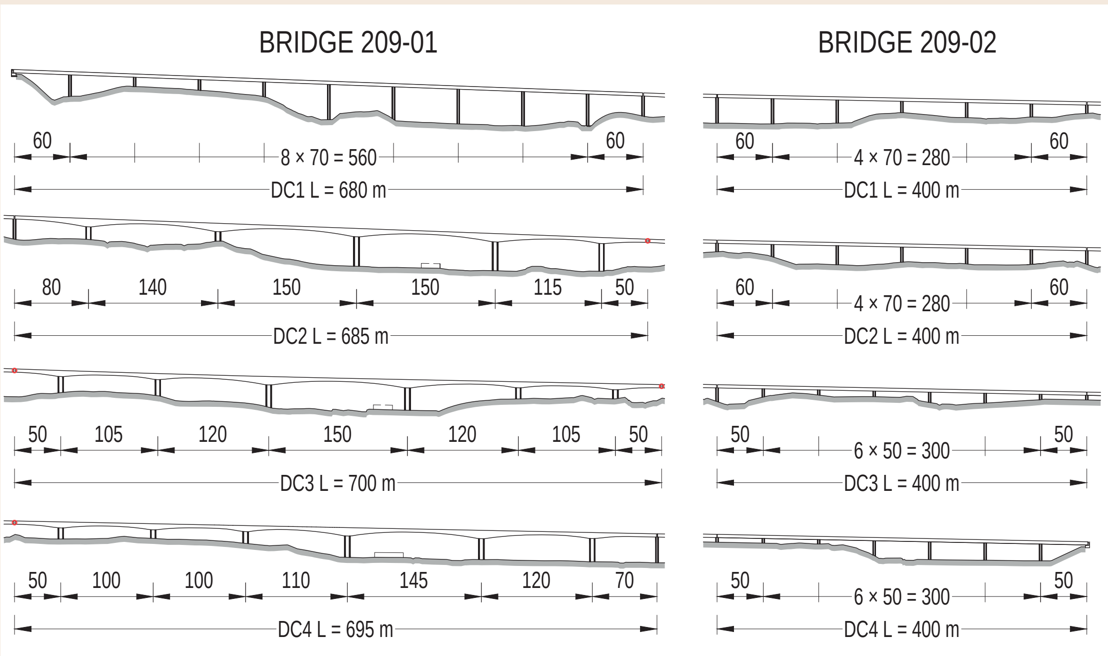



Fig. 7 Bridges´ elevations
Obr. 7 Podélné řezy mostů

At first, the spine box girder was cast and longitudinally prestressed, then precast struts were suspended on the spine girder, and the deck slab was progressively cast in simple formworks supported by these struts – see Fig. 3. The 2.50 m wide struts have a slab section stiffened by ribs at their edges. The ribs are situated outside; the inner surface is smooth. This not only simplifies production

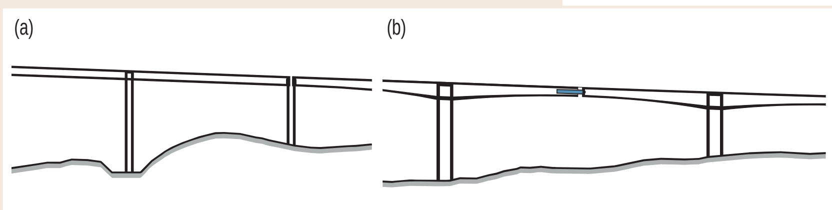



Fig. 8 Expansion joints: (a) typical, (b) at mid-span
Obr. 8 Dilatační spáry: (a) typická, (b) uprostřed rozpětí

Fig. 9 Typical expansion joint
Obr. 9 Typická dilatační spára

Fig. 10 Expansion joint at mid-span
Obr. 10 Dilatační spára uprostřed rozpětí

Fig. 11 Expansion beams: (a) erection, (b) service
Obr. 11 Dilatační nosník: (a) montáž, (b) provoz

but also increases the safety of workers moving on their smooth surface. In addition, this solution contributes to increasing the aesthetic effect of the viaduct. The combination of the smooth surface of the piers and the girder's bottom slab with the statically necessary ribbing of the outer struts creates a play of shadows that lightens the structure – see Fig. 4. The struts were placed on short corbels of the spine girder; their position was secured by two prestressing rods anchored in the top slab – see Figs. 5 and 6.

After casting and transverse prestressing of the deck slab, the space between the struts and the girder's webs was filled with a waterproof material – see Fig. 6d. This corresponds to the different static action of the struts during construction and service. During construction, the struts are hinge supported, while during service the struts are fixed into the box girder.

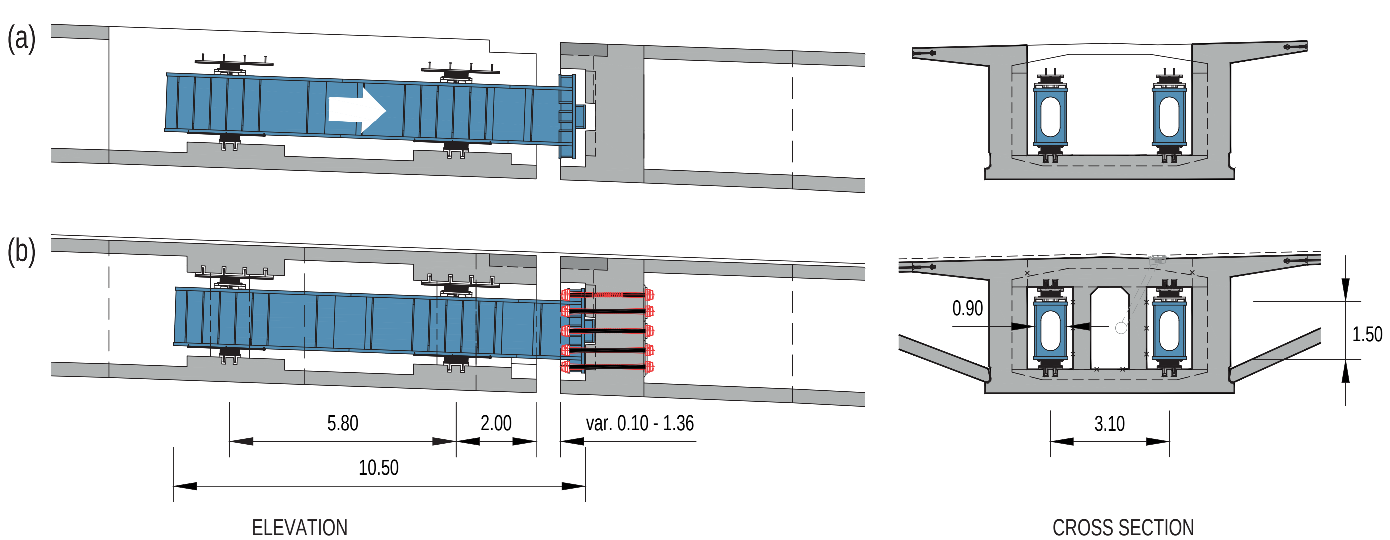
To simplify structural details and eliminate bearings, the structure was designed as an integral structure. Since piers formed by twin slender walls guarantee the stability of the cantilever structures during construction and at the same time allow large longitudinal movement of the completed multi-span structure, they were used not only for cantilever structures, but also for the viaduct's side parts. Here the twin walls support an advance constructed pier tables supporting the front legs of the overhead movable scaffoldings.

Administratively, the viaduct is divided into two bridges marked Bridge 209-01 and Bridge 209-02 – Fig. 7. The first bridge consists of four expansion units DC1 to DC4 with lengths from 680 to 700 m. The second bridge consists of four expansion units DC1 to DC4 with lengths of 400 m each. The piers situated in the middle of the expansion units are framed connected with the deck and footings, distant piers are –according to position and height of the piers – hinge or frame connected with the spine girder and footings. The hinges at footing were immobilized during the deck's construction.

The foundation of the bridge structure reflects the very different geological conditions along the length of the route, where high-quality bedrock alternates with lower-quality bedrock. For the foundation of the viaduct mostly micro-piles were designed, however, some supports are also founded on large-diameter piles or on spread footings. Also, jet grouting improves the poor-quality bedrock of two piers situated close to the creek.

Apart from the expansion joints between cantilevered and launched bridges, all other expansion joints are situated in the gap between the units' end diaphragms supported by slender walls – see Fig. 8a and 9.

The expansion joints of the cantilever structures are situated at midspans of the shortest spans – see Fig. 8b and 10. Deformations of the adjacent cantilevers are reduced by steel beams inserted into the spine box. The



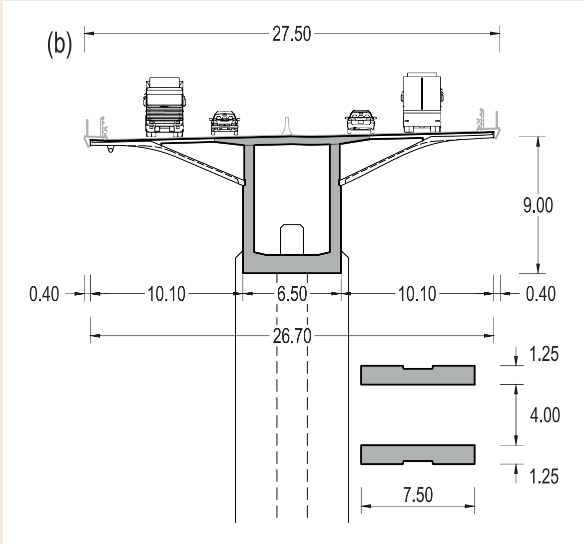



Fig. 12 Cantilever bridge
Obr. 12 Letmo betonovaný most

Fig. 13 Cross sections of the cantilever bridge at piers
Obr. 13 Příčný řez letmo betonovaného mostu u pilíře

beams are fixed into the midspan diaphragm of one cantilever and inserted into the box's cell of the adjacent cantilever where the beams are supported by neoprene bearings placed on the span diaphragms – see Fig. 11. The time-dependent analysis has proved that the mid-span deformations of these expansion spans are one half of the deformations of the structures with mid-span hinges.

## CANTILEVER BRIDGES

Cantilever bridges of a total length of 2,080 m (see Figs. 12 and 13) were progressively cast in balanced cantilevers. The depth of the girder in the middle of all spans is 3.50 m. The depth of the girder at the supports is 6.50 m for spans up to 110.00 m, for larger spans the girder's depth is 9.00 m. The haunch has the shape of the second-degree parabola. The length of the pier table was 12.5 m, the length of the segments was from 2.50 to 5.00 m, the length of the closure was 2.50 m.

The deck is prestressed by four cable systems. During the cantilever construction cantilever tendons were anchored in each segment. After the closure was cast, the span tendons and internal continuity tendons were installed and tensioned. After casting and transverse prestressing of the overhangs, the external continuity cables were installed and tensioned.

The expansion units were constructed in symmetrical cantilevers progressively built from their centers towards the expansion joints – see Fig. 14. Before casting the closures, the adjacent cantilevers were connected by erection steel beams and subsequently vertically adjusted. Before casting the mid-span joints of the outer spans, the adjacent cantilevers were jacked apart. The piers of the outer cantilevers were deflected outwards – in the direction opposite to their movement caused by creep and shrinkage of the concrete of the deck. In the case of pinned piers, the deflection has increased the rotational capacity

## SPAN-BY-SPAN CONSTRUCTED BRIDGES

The spine box girder of the expansion unit DC1 of bridge 209-01 with a length of 680 m and of the units DC1 and DC2 of the bridge 209-02 of a total length of 800 m (see Figs. 15 and 16) were cast span-by-span with overhanging cantilevers in a formwork suspended on an overhead movable scaffolding, which was formed by a tied arch with so called organic prestressing – see Fig. 17.

Fig. 14 Construction of cantilever bridges
Obr. 14 Výstavba letmo betonovaných mostů

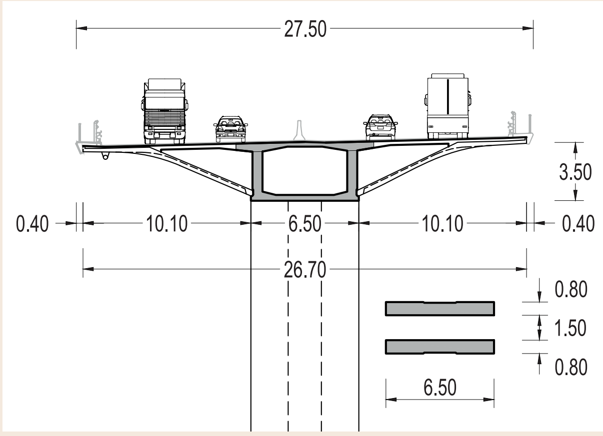



Fig. 15 Span-by-span constructed bridge
Obr. 15 Most stavěný po polích

Fig. 16 Cross sections of the span-by-span constructed bridge at piers
Obr. 16 Příčný řez mostu stavěného po polích u pilíře

Fig. 17 Span-by-span construction
Obr. 17 Výstavba po polích

Fig. 18 Progressive construction of the deck
Obr. 18 Postupná výstavba mostovky

The spine girder is prestressed by internal coupled tendons situated in the webs and non-continuous, support tendons, led at the top slab. Two spans behind the casting spans, the precast struts were gradually erected, and the overhangs were cast into a formwork supported by these struts in 50 m long sections – see Fig.18. After their transverse prestressing, external continuity cables were installed and tensioned.

## INCREMENTALLY LAUNCHED BRIDGES

The units DC3 and DC4 of the lengths of 2 × 400 m of the bridge 209-02 (see Fig. 19 and 20) were incrementally launched. The units were launched from a common support situated at the point of curvature change. Since the entire cross-section of the bridge can be relatively easily cast and subsequently launched, it was not necessary to create the deck incrementally. Therefore, the cross-section of the structure was modified. The outer slab struts were replaced by single bars located in the place of the struts' stiffening ribs. Thus, the character and unified architectural impression of the structure were preserved.

During launching, the deck was incrementally prestressed by coupled internal tendons situated in the deck's cross section. After the structure was launched, external continuity cables situated inside the box were installed and tensioned.

The deck was cast in segments of 30-35 m in length in the casting plant located at the location of the first span of the DC4 unit. The launching bearings were placed only on one pier's wall always closer to the launching jacks, the other wall support was free and served for placing the hydraulic jacks. The wall piers were temporarily reinforced

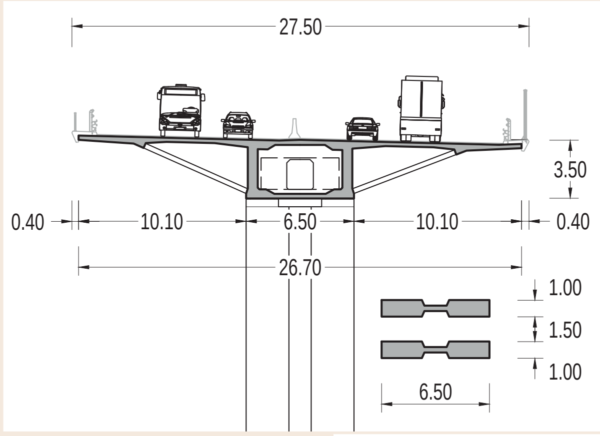



Fig. 19 Incrementally launched bridge
Obr. 19 Postupně vysouvaný most

Fig. 20 Cross sections of the Incrementally launched bridge at piers
Obr. 20 Příčný řez postupně vysouvaného mostu u pilíře

Fig. 21 Bridge launching
Obr. 21 Vysouvání mostu

by a steel truss. After the launching of the DC3 unit, the launching device and the steel nose were turned and the unit DC4 was launched in the opposite direction – see Fig. 21.

## CONCLUSIONS

The construction of the viaduct which proceeded without any significant technical problems began in 2022; and was completed in 2025. The viaduct, which was constructed in a beautiful valley in an environmentally sensitive mountainous area, had a minimum impact on the countryside both during its construction and in service. The architecture of the structure was developed from the true structural solution; the structure creates an economical semi-integral structural system which requires minimum maintenance. Although different construction methods have been utilized, the viaduct has a uniform architectural and structural arrangement along its whole length. Even though the viaduct is a considerable size, it does not overwhelm the beautiful landscape but rather complements it – see Fig. 22.

The viaduct was constructed as a Design Build Project, the client is Národná diaľničná spoločnosť (Road Construction Company), Bratislava. The expressway design is work of the firm Dopravoprojekt Bratislava. The viaduct was designed by the firm Stráský, Hustý a partneři, Brno, Czech Republic, the Project Manager was Libor Hrdina. The Viaduct was constructed by a Združenie (Joint Venture) R2 Kriváň – Mýtná formed by the firms Doprastav, Bratislava; STRABAG, Bratislava; EUROVIA SK, Košice, Slovakia and EUROVIA CS, Praha, Czech Republic.

Fig. 22 Cantilever bridge with 150 m long span
Obr. 22 Letmo betonovaný most s rozpětím 150 m

# VIADUCT OF THE SLOVAK MOTORWAY D1 NEAR THE VILLAGE OF TURIE – BRIDGE 202

# VIADUKT SLOVENSKÉ DÁLNICE D1 U OBCE TURIE – MOST 202

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**The 614.0 m long viaduct consists of two parallel bridges with 14 spans with a typical span of 43.5 m. Both bridges form a semi-integral structural system. The superstructures are made of box girders which were cast span-by-span in formwork supported by underslung movable scaffolding. The foundations of the bridges were influenced by the fact that the viaduct is situated in a landslide area.**

**Viadukt dlouhý 614,0 m se skládá ze dvou souběžných mostů o 14 polích s typickými rozpětími 43,5 m. Oba mosty jsou navrženy jako semi integrální konstrukční systém. Nosné konstrukce jsou tvořeny komorovými nosníky, které byly betonovány po polích do bednění podpíraného spodní výsuvnou skruží. Založení mostů bylo ovlivněno skutečností, že viadukt je situován v sesuvném území.**

Fig. 1 Bridge 202
Obr. 1 Most 202

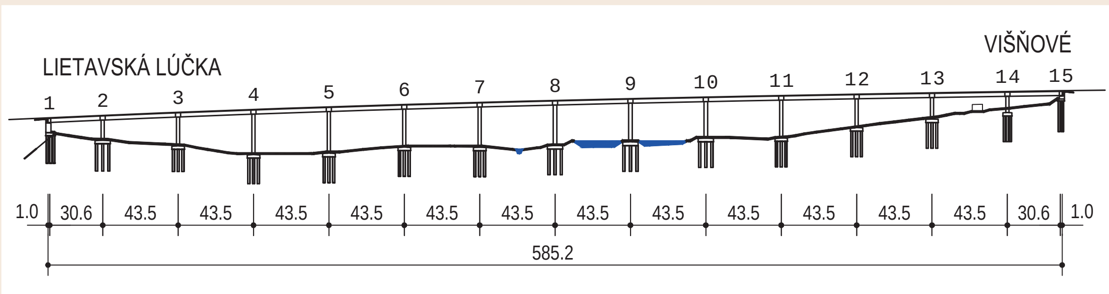
The Slovak motorway D1 near the city of Žilina is situated in the beautiful landscape of the Malá Fatra Mountains. In the Lietavská Lúčka – Višňové – Dubná Skala section, it runs on several long bridges – see Fig. 1. The construction of the motorway, which was carried out under the FIDIC Yellow Book regime, was started by the Joint Venture ´Združenie SALINI IMPREGILO – DÚHA'. The Venture strictly required that all bridges be designed as prefabricated beam structures.

Motorway construction started in 2015. The contractor carried out foundations, substructures, erected precast girders and cast composite slabs of some spans for a few bridges. In 2019, the Joint Venture decided to withdraw from the construction. In a new competition the Joint Venture ´Združenie ´SKANSKA – Višňové´ won the contract. Since construction of this viaduct had not started, the contactor has decided to abandon precast girder design and utilize their experience with construction of bridges using a movable scaffolding.

## ARCHITECTURAL AND STRUCTURAL DESIGN

The 614.0 m long viaduct carries the motorway over a long valley; retention ponds, a local road and a creek – see Fig. 2. The viaduct is formed by two parallel bridges with 14 bays of a typical span length of 43.5 m. The viaduct's axis is in a plan circle with a radius of 1,500 m and in a variable longitudinal slope from 4.4 to 1.5%.

Fig. 2 Elevation
Obr. 2 Podélný řez



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The prestressed concrete deck of both bridges consists of 14-span continuous box girder with a depth of 2.40 m. The width of the left bridge is from 13.55 to 15.05 m; the width of the right bridge 14.80 m is constant along its entire length. Above supports the girder is strengthened by 3 m long pier diaphragms with manholes, which with heights of 1.5 m was sufficient for easy transport of the inner cell's formwork of the box girder during construction.

The box girder from C35/45 concrete is longitudinally prestressed by four systems of Dywidag tendons of12-0.6" strands: (1) continuous tendons led at girder's webs that are coupled in construction joints, (2) continuous tendons led at the girder's webs that overlap above the piers and are anchored at opposite sides of the diaphragms, (3) short tendons situated at the tops slab above the piers and (4) short tendons situated at the bottom slab with anchors situated at the bottom corners of the box girder.

The piers are formed with slender columns of the octagonal cross section that transfer into column caps – see Fig. 3. On piers 8, 9 and 10 the deck is hinge connected with the caps, on the remaining piers and end abutments the deck is supported by couples of pot bearings.

The design of the viaduct's foundations addresses difficult geotechnical conditions of the bridge site in which the mountain's slopes are prone to landslide movements. Based on extensive geotechnical monitoring it was found that the rate of slope displacements is closely linked to fluctuations in the groundwater level. Therefore, drainage via 120 m long boreholes was implemented in this area, which decreases the groundwater level and a subsequent slowdown or cessation of the slope's landslides. However, the active landslide zones with ¼ movement remain at abutment 1 and piers 5, 6 and 7.

That is why abutment 1 is founded on 16 m long drilled piles and it is anchored by 25 m long rock anchors. Also, a provision for additional installation of more rock anchors is prepared. All piers are founded on 18 m long drilled piles and the footings of piers 2, 3, 5, 6 and 7 are protected by walls. These walls are formed by 16 m long drilled piles anchored by rock anchors of length from 26 to 30 m.

The viaduct's economical design was made possible by its detailed static and dynamic analysis. The bridge, which was analyzed by the MIDAS software system, was modeled as a 3D structure assembled from beam elements. A detailed time-dependent analysis of the gradually erected structure was performed for the designed and actual construction process.

## VIADUCT CONSTRUCTION

The bridge deck was incrementally cast span-by-span with overhanging cantilevers in formwork supported by Strukturas underslung movable scaffolding – see Fig. 4. At first the right bridge in direction from abutment 15 to abutment 1 was constructed, then the scaffolding was turned and used for construction of the left bridge in the opposite direction. The box girder was cast in two stages, at first, the bottom slab and webs, then the top slab. As soon as coupled continuous tendons and short pier tendons were post-tensioned, the scaffolding moved into the next span. Remaining tendons were post-tensioned when the neighboring span was completed.

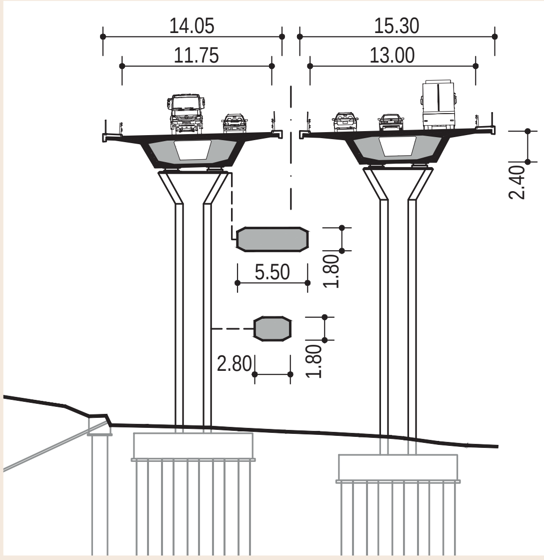


Fig. 3 Cross section at pier 5
Obr. 3 Příčný řez u pilíře 5

## CONCLUSIONS

The viaduct forms a structurally efficient and architecturally pleasing structure. Since the viaduct is composed of slender structural members, it creates a light and transparent structure that has a minimal impact on the environment.

The viaduct was constructed as a Design Build Project, the client is Národná diaľničná spoločnosť (Road Construction Company), Bratislava. The viaduct was designed by the firm Stráský, Hustý a partneři, Brno, Czech Republic, and was constructed by Skanska SK, a.s., Bratislava, Slovakia.

Fig. 4 Movable scaffolding
Obr. 4 Výsuvná skruž

# VIADUCT OF THE SLOVAK MOTORWAY D1 OVER A LANDSLIDE AREA – BRIDGE 204

# VIADUKT SLOVENSKÉ DÁLNICE D1 NAD SVÁŽNÝM ÚZEMÍM – MOST 204



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**The viaduct, which consists of two parallel bridges 254.0 m long, carries the motorway over a landslide area, a creek and a rural road. The six-span bridges with spans ranging from 34.50 to 100.18 m are made up of two different structures. The first three spans, which are formed by precast beams and a composite deck slab, are connected to a three-span cast-in-place box girder structure. The landslides within the site influenced the arrangement of the viaduct.**

**Viadukt, který je tvořen dvěma souběžnými mosty dlouhými 254,0 m, převádí dálnici přes svážné území, potok a polní cestu. Mosty o šesti polích s rozpětími od 34,50 do 100,0 m mají dvě konstrukčně rozdílné části. První tři pole, která jsou sestavena z prefabrikovaných nosníků a spřažené desky, spojitě navazují na letmo betonovanou komorovou konstrukci o třech polích. Uspořádání mostů bylo ovlivněno skutečností, že viadukt je situován ve svážném území.**

Fig. 1 Bridge 204
Obr. 1 Most 204

The Slovak motorway D1 near the city of Žilina passes through the beautiful Malá Fatra Mountains. In the Lietavská Lúčka – Višňové – Dubná Skala section, it runs on several long bridges – see Fig. 1. The construction of the motorway started in 2015. The contract, which was carried out under the FIDIC Yellow Book regime, was initially undertaken by the Joint Venture (JV) ´Združenie SALINI IMPREGILO – DÚHA´. The Venture strictly stipulated that all bridges must be designed as prefabricated structures. Therefore, the viaduct was originally designed as a 9-span precast beam structure with spans from 34.5 to 41.0 m long. However, the geotechnical survey performed upon the start of the motorway's construction indicated the presence of several active and potential landslides within the site. After numerous options were considered and worked out, it was decided to step over the most dangerous zones by using long spans.

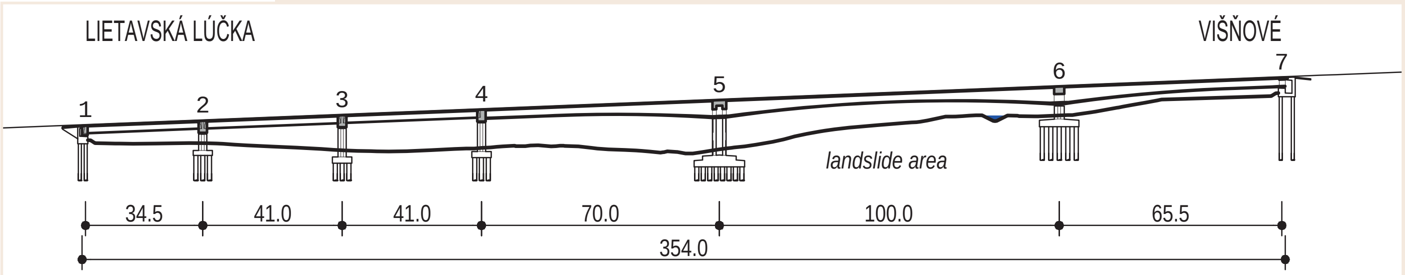
Meanwhile, the contractor carried out foundations, abutments, and piers, some precast girders were erected, and portions of the composite deck were cast for several bridges. In 2019, the Joint Venture decided to withdraw from the job. In a new bid, the Joint Venture ´Združenie ´SKANSKA – Višňové´ won the contract. It was agreed that the bridge 204 will be constructed according to the project developed under the previous JV.

## ARCHITECTURAL AND STRUCTURAL DESIGN

The 354 m long viaduct carries the motorway over a long valley with hazardous landslide areas, a creek and a local road – see Fig. 2. Each of the two bridges have dedicated horizontal and vertical alignments, as the motorway morphs from a circle with a radius of 2,000 m into two separate tunnel tubes. For the most part following a transitional curve, the horizontal distance between the left and right grows from about 16 m to over 23.5 m at the end. The width of the right bridge is 15.30 m, the width of the left bridge is 14.35 m.

The first three spans of both bridges (35.0 + 2 × 41 m long) are assembled of 2.0 m deep precast beams and

Fig. 2 Elevation
Obr. 2 Podélný řez



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a composite deck slab 0.23 m thick – see Fig. 3a. The following three spans (70.0 + 100.0 + 66.0 m long) were built by the balanced cantilever method – see Fig. 3b. The precast beams rest on transverse girders shaped as an inverted letter T. At the ends of the precast section an inverted T naturally reduces to L. The transverse girders are supported by two pot bearings each. Due to the different bridge width, the right bridge has 9 beams, and left bridge has 8 beams in a cross section. Each precast beam is assembled of three segments, which are mutually connected by prestressing; the joints are filled with epoxy.

The cantilever structures are made of single-cell box girders of a variable depth ranging from 2.70 m at midspan and at supports 4 and 7 to 5.20 m at piers 5 and 6. At supports the girders are stiffened by internal diaphragms. Above pier 5, where the superstructure is monolithically connected to the substructure, two diaphragms 1.15 m thick are provided above the walls of the split pier. Above pier 6 a single diaphragm 3.0 m thick transfers the stress from webs into two pot bearings. A similar situation exists above abutment 7. Furthermore, 0.70 m thick web stiffeners placed at approximately one third the span length transfer the radial forces from the external continuity cables to the webs.

The deck is prestressed by three cable systems of a classical arrangement. During construction cantilever tendons were installed and anchored in each segment. After the closures were cast, the span tendons and external continuity cables were installed and tensioned. The external continuity tendons, which are situated inside the box girders, are deviated at pier and span diaphragms.

At support 4 longitudinal rebars reinforcing the top slab of the box girder protrude into the composite slab of the beam structure. This reinforcement, together with additional reinforcement of the pier diaphragms, guarantee the monolithic connection of the precast spans with the cantilever structures.

Pier 5, which are monolithically connected with the decks of both bridges, serve as bridge's fix points – see Fig. 4. Longitudinally movable bearings support the deck on all other piers. All foundations rest on drilled piles. Protecting walls formed by anchored drilled piles are situated upslope of (i.e. to the right of) each support except abutment 1 and pier 2, to arrest the landslide movement.

The viaduct's economical design was made possible by its detailed static and dynamic analysis. The bridges, which were analyzed by the MIDAS FEM software, were modeled as 3D structures assembled from beam elements. As the bridges are progressively constructed, the static system changes, and the concrete of different structural members ages differently, therefore detailed time-dependent analyses of the various construction stages were performed for the designed and actual construction process.

## VIADUCT CONSTRUCTION

The construction of precast and cast-in-place sections proceeded simultaneously – see Fig. 5. After construction

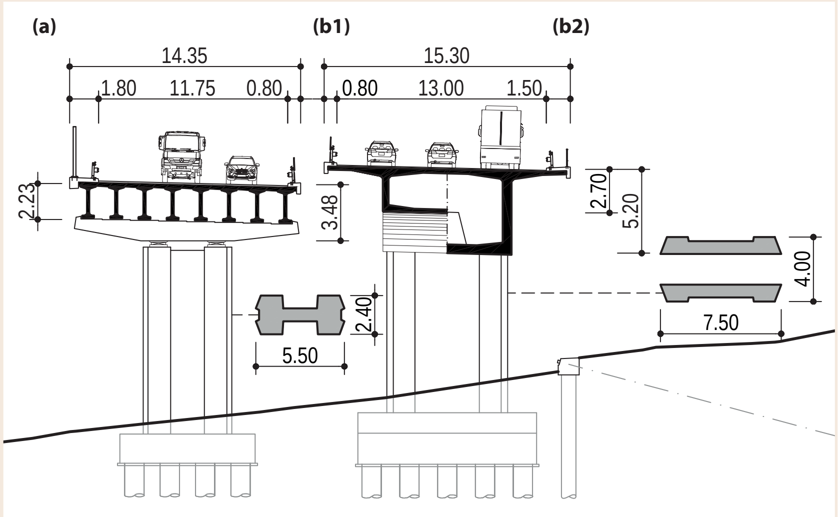


Fig. 3 Cross section:
(a) precast spans,
(b) cantilevered spans,
1 – at midspan, 2 – at pier 5
Obr. 3 Příčný řez:
(a) prefabrikovaná pole,
(b) letmo betonovaná pole,
1 – uprostřed rozpětí,
2 – u podpěry 5

of the piers, the transverse girders of the piers 1 through 4 were cast. Then the precast beams were erected, and composite slabs were progressively cast. At the same time 15 m long pier tables were constructed, travellers were erected and then 2 × 7 deck segments were progressively cast and prestressed in symmetrical balanced cantilevers. While the stability of the erected cantilevers above the pier 5 was given by the rigid connection to the (twin-walled) pier, the stability of the cantilevers above the pier 6 was guaranteed by temporary columns supporting the pier tables. After casting the 4.5 m long mid-span closures and 3.75 m long closures at pier 4, the decks were prestressed by span tendons and external continuity cables. After that, composite slabs above pier 4 were cast.

## CONCLUSIONS

The viaduct forms a structurally efficient structure bridging a dangerous landslide areas. Although it consists of two different structural systems, it forms a continuous structure requiring minimum maintenance – see Fig. 6.

The viaduct was constructed as a Design Build Project, the client is Národná diaľničná spoločnosť (Road Construction Company), Bratislava. The viaduct was designed by the firm Stráský, Hustý a partneři, Brno, Czech Republic, and was constructed by PORR s.r.o, Bratislava in JV with SKANSKA SK a.s., Bratislava.

Fig. 4 Bridge structure
Obr. 4 Konstrukce mostu

# VIADUCT OF THE SLOVAK MOTORWAY D1 NEAR THE VILLAGE OF VIŠŇOVÉ – BRIDGE 203

# VIADUKT SLOVENSKÉ DÁLNICE D1 U OBCE VIŠŇOVÉ – MOST 203

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**The 960.3 m long viaduct consisting of two parallel bridges with 24 bays and 40 m spans is described in terms of architectural and structural design and construction technology. Both bridges, which are led up to 42 m above terrain, form integral structural systems composed of three expansion units. The supporting structures, which are made of prefabricated beams and composite concrete slabs, are supported by slender piers made of twin wall columns.**

**Viadukt dlouhý 960,3 m, který se skládá ze dvou souběžných mostů o 24 polích s rozpětími 40 m, je popsán z hlediska architektonického a konstrukčního řešení a technologie výstavby. Oba mosty, které jsou vedeny až 42 m nad terénem, jsou navrženy jako integrální konstrukční systémy sestavené ze tří dilatačních celků. Nosné konstrukce, které jsou tvořeny prefabrikovanými nosníky a spřaženými betonovými deskami, jsou podepřeny štíhlými pilíři tvořenými dvojitými stěnovými sloupy.**

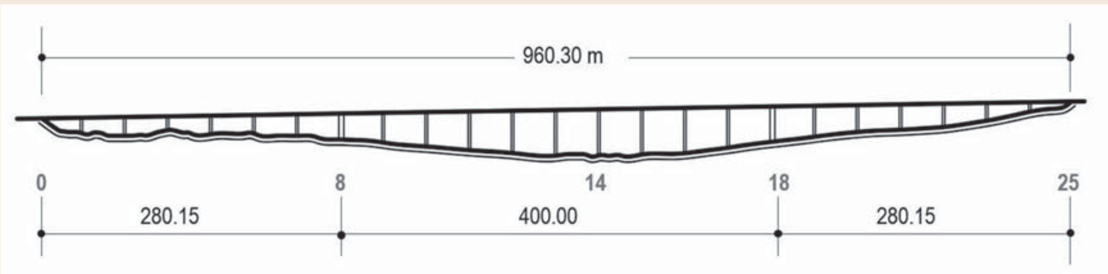
Fig. 1 Bridge 203
Obr. 1 Most 203

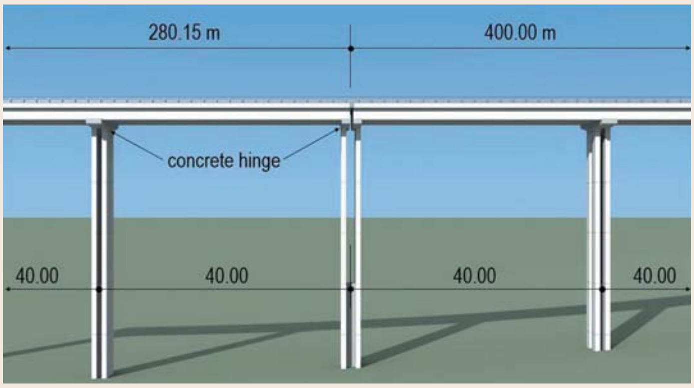
The Slovak motorway D1 near the city of Žilina is situated in the beautiful landscape of the Malá Fatra Mountains. In the Lietavská Lúčka – Višňové – Dubná Skala section, it runs on several long bridges – see Fig. 1. The construction of the motorway, which was carried out under the FIDIC Yellow Book regime, was started by the Joint Venture ´Združenie SALINI IMPREGILO – DÚHA. The Venture strictly required that all bridges be designed as prefabricated beam structures.

Motorway construction started in 2015. The contractor carried out foundations, substructures, erected precast girders and cast composite slabs of some spans for a few bridges. In 2019 the Joint Venture decided to withdraw from the construction. In a new competition the Joint Venture ´Združenie ´SKANSKA – Višňové´ won the contract. It was agreed that the structures under construction, including bridge 203, will be completed according to the existing projects.

The 960.3 m long viaduct carries the motorway over a deep and long valley, several local roads and a creek – see Fig. 2. The viaduct is formed by two parallel bridges with 24 spans of length of 40 m each divided into three expansion units. The fact that the deck is up to 42 m above ground level, and the supports are up to 39.70 m high made it possible to create an integrated structural system with bearings located only on the end abutments.

Fig. 2 Elevation
Obr. 2 Podélný řez





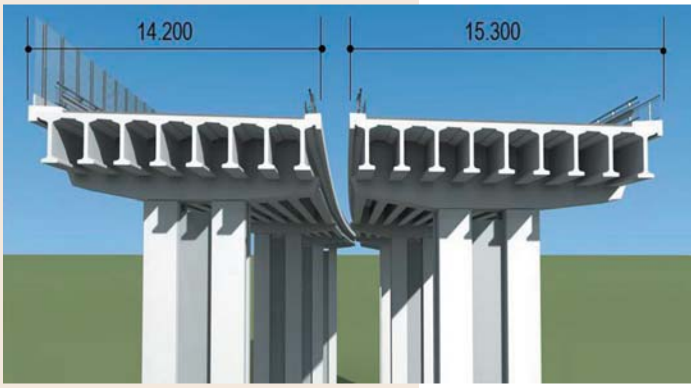




Fig. 3 Partial elevation with expansion pier
Obr. 3 Částečný podélný řez s dilatačním pilířem

Fig. 4 Cross section
Obr. 4 Příčný řez

The superstructure consists of prefabricated beams 2.00 m deep and a cast-in-place composite slab 0.225 m thick – see Figs. 3 and 4. Due to the layout of the motorway, the width of the left and right bridge is different. The left bridge with a total width of 14.20 m is composed of eight beams, the right bridge with a total width of 15.30 m is composed of nine beams. The beams are assembled of three reinforced concrete elements, which were mutually connected by prestressing on site.

The beams are placed on reinforced concrete pier caps, which are supported by twin wall columns. The caps have the shape of an inverted letter T (see Fig. 5). The reinforcement of the caps is supplemented by vertical bars anchored in the bottom flange. Since this reinforcement overlaps with the reinforcement of the composite slab, stiff crossbeams were created there.

All columns are hinge connected with the pier caps. They have the same shape; their width is 5.50 m, and their thickness is 0.80 m. The higher wall columns are interconnected at the foundations by longitudinal stiffening walls, the low piers situated close to the end abutments have concrete hinges at their foundations. The gap between the wall columns of 0.80 m is increased to 1.5 m for expansion piers.

The pier caps of the expansion piers, which were created by dividing typical caps, are L-shaped. During assembly, concrete blocks were inserted between them and were subsequently connected by prestressing. After casting the deck slab the temporary connection was removed. The abutments and most of the piers are founded on drilled piles.

The viaduct's economical design was made possible by its detailed static and dynamic analysis. The bridge, which was analyzed by the MIDAS software system, was modeled as a 3D structure assembled from beam elements. A detailed time-dependent analysis of the gradually erected structure was performed for the designed and actual construction process.

## VIADUCT CONSTRUCTION

The construction of the deck of both bridges was carried out in three stages. First, the outer expansion units were built with the process of erection of precast beams and the casting the composite deck in the direction from the abutment to the expansion piers. When the pier caps of the expansion piers were temporarily connected, the side spans of the central expansion units were erected. Subsequently, the beams of the central units were erected, and composite deck slabs were progressively cast. The beams were erected by a pair of cranes moving along the temporary road located along the bridge.

## CONCLUSIONS

The viaduct forms a structurally efficient and architecturally pleasing structure. Its realization clearly demonstrates that an economic integral structure requiring low maintenance can be easily created from precast beams. Since the viaduct is composed of slender structural members, it creates a light and transparent structure that has a minimal impact on the environment – see Fig. 1.

The viaduct was constructed as a Design Build Project, the client is Národná diaľničná spoločnosť (Road Construction Company), Bratislava. The viaduct was designed by edit: the firm Stráský, Hustý a partneři, Brno, Czech Republic, and was constructed by Doprastav, a.s., Bratislava, Slovakia.

Fig. 5 Beams and pier caps
Obr. 5 Nosníky a hlavice pilířů

# EXPANSION OF THE D1-233 MOTORWAY BRIDGE
# ROZŠÍŘENÍ DÁLNIČNÍHO MOSTU D1-233



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**The D1 motorway near Brno is the busiest section of the Czech Republic's motorway network. This section of the highway did not have sufficient capacity to handle the current traffic volume in its original four-lane configuration. A decision was therefore made to widen this section to six lanes. The first stage was section D1 01191.C, between the Brno-Centrum and Brno-Jih interchanges. Within this section the D1-233 bridge carries the D1 motorway via two independent structures, which were constructed while maintaining traffic flow on the motorway. A key phase in the construction of the bridge was the erection of the superstructure without the use of falsework. This technology significantly accelerated the work, contributing to an earlier commissioning of the entire section. The project was delivered under FIDIC Yellow Book (Design and Build) conditions, enabling the design and construction phases to be closely integrated.**

**Dálnice D1 je v úseku kolem Brna nejvytíženější částí dálniční sítě v České republice, přičemž kapacita stávajícího čtyřpruhového uspořádání je téměř vyčerpána. Proto bylo rozhodnuto o jeho rozšíření na uspořádání šestipruhové. První etapou se stal úsek D1 01191.C mezi mimoúrovňovými křižovatkami Brno-centrum a Brno-jih. Most D1-233 v tomto úseku převádí dálnici D1 pomocí dvou samostatných konstrukcí, které byly realizovány za plného dálničního provozu. Klíčovou fází byla výstavba nosné konstrukce bez použití podpěrné skruže. Tato technologie přinesla značné urychlení prací a přispěla k dřívějšímu zprovoznění celého úseku. Projekt byl realizován podle podmínek FIDIC Yellow Book (Design and Build), což umožnilo úzké propojení projekčních a realizačních fází.**

Fig. 1 View of the bridge
Obr. 1 Celkový pohled

## BASIC PROJECT DATA

| | |
|---|---|
| TYPE OF CONSTRUCTION: | Continuous composite prestressed concrete girder structure |
| DECK LENGTH: | 284.40 m left bridge (LB),<br>277.90 m right bridge (RB) |
| SPAN LENGTHS: | 18.5–32.5 m (11 spans LB, 10 spans RB) |
| BRIDGE WIDTH: | 40.1 m (20.05 m LB, 19.85 m RB) |
| INVESTOR: | Road and Motorway Directorate of the Czech Republic |
| DESIGNER: | Link projekt s.r.o., Brno |
| CONTRACTOR: | MI Roads a.s. |
| CONSTRUCTION TIME: | 08/2023–04/2025 |

## BRIDGE DESIGN PRINCIPLES

Due to heavy traffic and the poor condition of the existing structure, it was proposed that the bridge be completely reconstructed without reusing the old structure.

The new D1-233 bridge carries the D1 motorway (category D34.5/130) over the following: Railway Line No. 250 (Brno Main Station – Břeclav); the siding to Terminal Brno, a.s.; the planned north–south and west–south high-speed rail branches; a service road; and the planned relocation of the Leskava stream.

The D1 motorway alignment on the bridge includes a transition curve and a circular curve with a radius of 2,800 m. The vertical alignment follows a crest curve with longitudinal gradients ranging from +0.73% to –1.18%, and the roadway transverse slope is 2.5%.

## STRUCTURAL DESIGN

The bridge is supported by deep foundations comprising large-diameter bored piles (Ø1200 mm). The internal piers near the railway line are founded on two rows of piles (2 × 5), while the remaining piers are founded on a single row (1 × 6). This arrangement enabled the bridge to be constructed as an integral structure, eliminating the need for bearings and expansion joints on all the piers. The pile lengths for the internal piers vary between 13 and 20 metres, and the abutment piles are between 15 and 22 metres long. A static load test was performed on a preliminary test pile to verify the geotechnical conditions.

The substructure comprises massive end abutments and ten (LB) or nine (RB) intermediate piers. These piers comprise a pair of rectangular columns measuring 2.65 ×

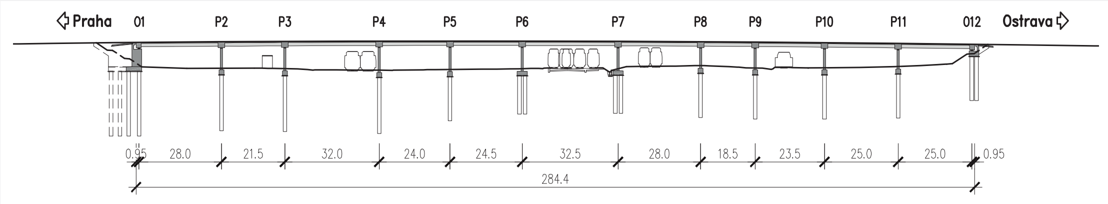



Fig. 2 Left bridge longitudinal section
Obr. 2 Podélný řez levého mostu

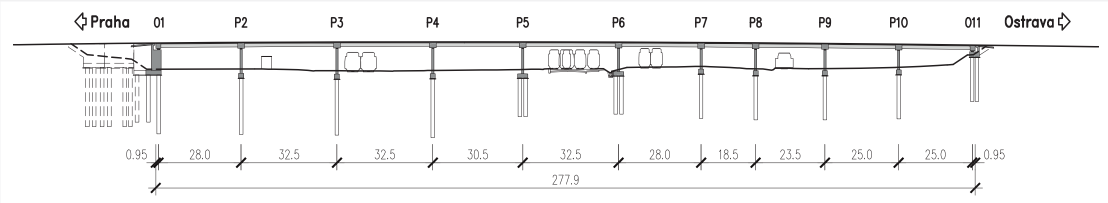



Fig. 3 Right bridge longitudinal section
Obr. 3 Podélný řez pravého mostu

## ZÁKLADNÍ DATA PROJEKTU

| | |
|---|---|
| TYP KONSTRUKCE: | Spojitá spřažená předpjatá betonová nosníková konstrukce |
| DÉLKA NK: | 284,40 m levý most (LM), 277,90 m pravý most (PM) |
| ROZPĚTÍ POLÍ: | 18,5–32,5 m (11 polí LM, 10 polí PM) |
| ŠÍŘKA MOSTU: | 40,1 m (20,05 m LM, 19,85 m PM) |
| INVESTOR: | Ředitelství silnic a dálnic s. p. |
| PROJEKTANT: | Link projekt s.r.o., Brno |
| ZHOTOVITEL MOSTU: | MI Roads a.s. |
| DOBA VÝSTAVBY: | 08/2023–04/2025 |

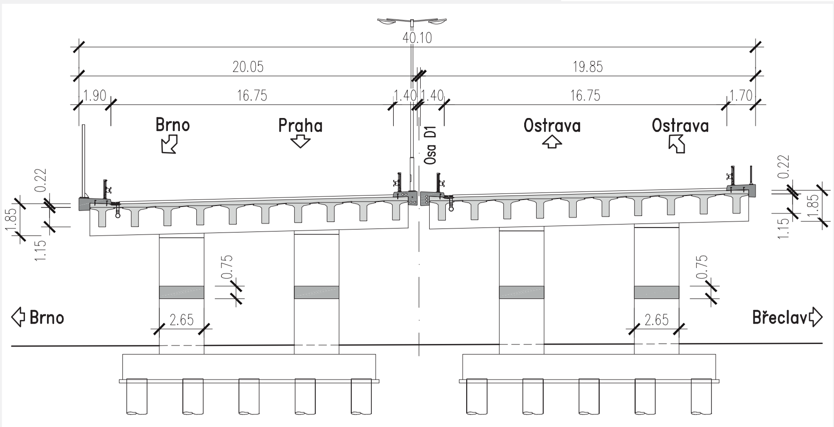



Fig. 4 Cross-section
Obr. 4 Příčný řez

## ZÁSADY NÁVRHU MOSTU

Vzhledem k vysokému dopravnímu zatížení a technickému stavu stávající konstrukce byla navržena kompletní rekonstrukce mostu bez využití konstrukce mostu stávajícího.

Nově navržený most D1-233 převádí dálnici D1 – komunikaci kategorie D34,5/130 přes železniční trať č. 250 Brno Hlavní nádraží – Břeclav, přes vlečku do areálu Terminal Brno, a.s., plánovanou severojižní a západojižní větev VRT, účelovou komunikaci a plánovanou přeložku potoku Leskava.

Směrově je na mostě osa dálnice D1 tvořena přechodnicí a kružnicovým obloukem o poloměru 2800 m. Niveleta je na mostě vedena ve vrcholovém zakružovacím oblouku, podélný spád je proměnný +0,73 % až –1,18 %. Příčný sklon vozovky na mostě je 2,5 %.

## KONSTRUKČNÍ ŘEŠENÍ

Založení mostu je hlubinné na vrtaných velkoprůměrových pilotách Ø1200 mm. Vnitřní podpěry u železniční trati jsou založeny na dvou řadách pilot (2 × 5), zbývající vnitřní podpěry na jedné řadě pilot (1 × 6). Toto uspořádání umožnilo navrhnout konstrukci bez nutnosti osazovat ložiska a dilatační závěry na všech mezilehlých podpěrách. Délka pilot vnitřních podpěr je proměnná v rozmezí 13–20 m. Opěry jsou založeny na pilotách délek 15–22 m. Pro ověření základových poměrů byla provedena statická zatěžovací zkouška nesystémové piloty.

Spodní stavbu mostu tvoří krajní masivní opěry a deset (LM) / devět (PM) mezilehlých podpěr, které se skládají z dvojice pilířů obdélníkového průřezu o rozměrech 2,65 × 0,75 m z betonu C40/50. Nosná konstrukce je na opěrách uložena na dvě všesměrná hrncová ložiska a jedno vodící jednosměrné ložisko. Vnitřní podpěry jsou k nosné konstrukci připojeny kloubově vrubovými klouby. Ty byly aktivovány prořezáním až poté, co byla nosná konstrukce plně zmonolitněna se spodní stavbou.

Nosnou konstrukci v příčném směru tvoří 2 × 10 ks prefabrikovaných dodatečně předpjatých nosníků T výšky 1,15 m z betonu C60/75 spřažených s monolitickou deskou průměrné tloušťky 0,22 m z betonu C35/45. Nosníky byly předepnuty ve dvou fázích jedním nebo dvěma 10 až 19lanovými kabely z lan Y1860 S7-15,7 v závislosti na jejich délce. Nosná konstrukce je v podélném směru zmonolitněna pomocí železobetonových příčníků. Vnitřní příčníky šířky 2,3 m a výšky 1,85 m byly zhotoveny ve dvou fázích – zárodek příčníku ve tvaru obráceného T z betonu C40/50 a dobetonávka příčníku z betonu C35/45.

Rozpětí polí mostu je proměnné v délkách od 18,5 do 32,5 m. Levý most tvoří celkem 11 polí při délce nosné konstrukce 284,40 m, pravý most 10 polí při délce nosné konstrukce 277,90 m. Celková šířka mostu je 40,10 m.

Pro všechny části konstrukce mostu byla uplatněna opatření pro ochranu mostu proti vlivu bludných proudů ve stupni č. 5 a pro ochranu před atmosférickými vlivy.

Fig. 5 Left bridge demolition
Obr. 5 Demolice stávajícího levého mostu

0.75 m and made of C40/50 concrete. At the abutments, the superstructure is supported by two multi-directional pot bearings and one guide bearing. The internal piers are connected to the superstructure by concrete hinges. These were activated (by cutting) only once the superstructure had been fully integrated with the substructure.

The superstructure's cross-section comprises ten precast post-tensioned T-beams (1.15 m deep) made from C60/75 concrete. These beams act compositely with a cast-in-place slab, which has an average thickness of 0.22 m and is made from C35/45 concrete. The beams were prestressed in two stages using one or two tendons containing 10 to 19 strands (Y1860 S7-15.7) depending on their length. The superstructure is made continuous above piers by means of the reinforced concrete cross-beams (diaphragms). The internal cross-beams are 2.3 m wide and 1.85 m deep. They were constructed in two phases: first, an inverted T-shaped starter section made of C40/50 concrete supporting the precast beams, followed by a subsequent concrete pour made of C35/45 concrete.

The bridge spans range from 18.5 to 32.5 metres, with the left bridge having 11 spans totalling 284.4 metres and the right bridge having 10 spans totalling 277.9 metres, giving a total bridge width of 40.1 metres.

Protective measures against stray currents (protection level 5) and against atmospheric effects were applied to all parts of the bridge structure.

## CONSTRUCTION OF THE BRIDGE

To maintain uninterrupted flow of traffic on the motorway, construction of the bridge was divided into several phases: temporary widening and shoring of the existing right bridge to accommodate a temporary four-lane arrangement; demolition of the left bridge; construction of the new left bridge; transfer of temporary lanes; demolition of the right bridge; and construction of the new right bridge.

A key phase was implementing the superstructure without temporary falsework. First, the full-height pier columns were built, followed by the inverted T-shaped cross-beam starter sections. These cross beams were then monolithically connected to the piers, with the connections being stressed by vertical prestressing bars. This detail was crucial for the possible installation of precast beams on the cross beams without the need for any shoring. This enabled the full span to be erected without the need for parallel installation of the neighbouring span. The prestressing bars ensured that the pier section of the future hinge remained in compression throughout the construction process. The cast-in-place deck slab was poured first in the spans, then over the internal piers including the final concrete pour of the cross-beams. Once the deck and cross-beams were completed, the concrete hinges at the pier heads were activated by precisely cutting the concrete. Finally, the prestressing bars were released and the ducts were grouted.

The reconstruction of the bridge began in August 2023 with the temporary widening of the right bridge. The left bridge opened in August 2024, followed by the right bridge in April 2025. Prior to opening, a static load test was performed on both bridges.

MI Roads a.s. was the bridge contractor, and Doprastav a.s. supplied the precast beams.

Fig. 6 Left bridge internal cross-beam starters
Obr. 6 Zárodky vnitřních příčníků levého mostu
Fig. 7 Left bridge beam placement
Obr. 7 Osazení nosníků levého mostu

## STATIC ANALYSIS

To analyse the bridge structure during construction and operation, 3D frame and shell/plate finite element models were developed using the MIDAS Civil software programme. These models represented the actual geometry and loading of the bridge, including the construction sequence, the profiles of the prestressing tendons, the time-dependent properties of the concrete and the interaction between the soil and the structure. The analysis covered incremental construction stages, such as the sequential placement of beams on cross-beam starter sections, the phased casting of the deck slab and the sequential casting of support cross-beams. This was followed by the formation of concrete hinges through the cutting of pier columns.

## CONCLUSION

The new D1-233 bridge carries a six-lane section of the first stage of widening the D1 motorway near Brno. Through close coordination between the designer and the contractor and by utilising the Design and Build contract model, the construction sequence was optimised. Despite the demanding engineering challenges, the chosen method significantly accelerated the works and contributed to the earlier commissioning of this critically busy section.

## MATERIAL USAGE (SUPERSTRUCTURE)

| | TOTAL | PER 1 M$^2$ |
|---|---|---|
| CONCRETE C60/75 | 3 405 m$^3$ | 0.304 m$^3$ |
| CONCRETE C35/45 + C40/50 | 3 885 m$^3$ | 0.346 m$^3$ |
| PRESTRESSING STEEL | 228 t | 20.3 kg |
| REINFORCING STEEL | 1923 t | 171.4 kg |

## VÝSTAVBA

Výstavba mostu byla vzhledem k nutnosti zachování plného dálničního provozu rozdělena do několika etap – dočasné rozšíření a podepření stávajícího pravého mostu, demolice stávajícího levého mostu, výstavba nového levého mostu, demolice stávajícího pravého mostu, výstavba nového pravého mostu.

Klíčovou fází realizace mostu byla výstavba nosné konstrukce bez použití podpěrné skruže. Nejdříve byly zhotoveny dříky vnitřních podpěr v plném průřezu a na nich zárodky příčníků ve tvaru obráceného T. Zárodky příčníků byly s dříky podpěr spojeny monoliticky a sepnuty svislými předpínacími tyčemi. Tento detail byl zásadní pro možnost osazení prefabrikovaných nosníků na zárodky příčníků bez nutnosti jakéhokoliv podepření. To umožnilo montáž celého pole bez potřeby souběžné instalace sousedního pole. Předpínací tyče zajistily, že průřez dříku podpěry v místě budoucího vrubového kloubu zůstane po celou dobu výstavby v tlaku. Betonáž monolitické desky probíhala nejdříve v polích, poté u vnitřních podpěr včetně dobetonávek příčníků. Po betonáži desky a příčníků byly přesným prořezáním betonu aktivovány vrubové klouby v hlavách podpěr. Na závěr byly uvolněny předpínací tyče a zainjektovány chráničky.

Rekonstrukce mostu byla zahájena v srpnu 2023 dočasným rozšířením pravého mostu. Do provozu byl levý most uveden v srpnu 2024, pravý most v dubnu 2025. Před uvedením mostu do provozu byla provedena statická zatěžovací zkouška.

Zhotovitelem mostu byla společnost MI Roads a.s., dodavatelem prefabrikovaných nosníků společnost Doprastav, a.s.

## STATICKÁ ANALÝZA

Pro analýzu konstrukce mostu v montážních i provozních stavech byly vytvořeny prostorové prutové a deskostěnové výpočetní modely v programovém prostředí MIDAS Civil. Výpočetní modely vystihovaly reálnou geometrii mostu a zatížení, včetně postupu výstavby mostu, vedení předpínacích kabelů, reologických vlastností betonu v čase a interakce konstrukce s podložím. Analyzována byla postupná výstavba konstrukce, včetně postupného osazování nosníků na zárodky příčníků, postupné betonáže desky nosné konstrukce a postupné betonáže nadpodporových příčníků s následným vznikem vrubových kloubů prořezáváním dříků podpěr.

Fig. 8 Right bridge superstructure construction (source: MI Roads)
Obr. 8 Výstavba nosné konstrukce pravého mostu (zdroj: MI Roads)

Fig. 9 Superstructure soffit
Obr. 9 Podhled nosné konstrukce

## ZÁVĚR

Nový most D1-233 v šestipruhovém uspořádání převádí část první rozšiřované etapy dálnice D1 v úseku kolem Brna. Díky využití smluvního modelu Design and Build a úzké součinnosti projektanta se zhotovitelem se podařilo optimalizovat technologický postup tak, že i přes náročné inženýrské výzvy přinesla zvolená metoda výstavby značné urychlení prací a přispěla k dřívějšímu zprovoznění tohoto kriticky vytíženého úseku.

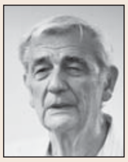
## SPOTŘEBA MATERIÁLU (NOSNÁ KONSTRUKCE)

| | CELKEM | NA 1 M$^2$ |
|---|---|---|
| BETON C60/75 | 3 405 m$^3$ | 0,304 m$^3$ |
| BETON C35/45 + C40/50 | 3 885 m$^3$ | 0,346 m$^3$ |
| PŘEDPÍNACÍ VÝZTUŽ | 228 t | 20,3 kg |
| BETONÁŘSKÁ VÝZTUŽ | 1923 t | 171,4 kg |

Fig. 10 Bridge before completion (source: MI Roads)
Obr. 10 Most před dokončením (zdroj: MI Roads)

# GENERAL RECONSTRUCTION OF BARRANDOV BRIDGE IN PRAGUE

# CELKOVÁ REKONSTRUKCE BARRANDOVSKÉHO MOSTU V PRAZE



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**Barrandov bridge in Prague is one of the busiest bridges in the Prague transport system. It transfers traffic from the City ring road across the Vltava River for a distance of 350 m. The bridge was put into operation in 1988 and from that time it was used mainly for heavy truck transport. After 30 years of operation, severe damage appeared and it was necessary to make its general reconstruction in the period from 2020 to 2024. Except for replacement of the bridge equipment (pavement, expansion joints, bearings, drainage etc.), the bridge deck was strengthened with an additional layer of UHPC and additional external prestressing tendons were installed inside the bridge deck. The load-bearing capacity of the bridge was increased and its remaining service life was extended.**

**Barrandovský most je nejzatíženějším mostem v dopravním systému Prahy. Převádí silniční dopravu městského okruhu přes Vltavu v délce 350 m. Most byl uveden do provozu v roce 1988 a od té doby sloužil převážně těžké kamionové dopravě. Po 30 letech provozu vykazoval poruchy takového rozsahu, že bylo třeba přistoupit v letech 2020 až 2024 k jeho celkové rekonstrukci. Kromě výměny mostního svršku a vybavení mostu bylo provedeno také zesílení jeho nosných konstrukcí. Mostovka byla zesílena vyrovnávací vrstvou z UHPC a do nosných konstrukcí bylo instalováno podélné přídavné předpětí formou volných kabelů. Most má nyní po rekonstrukci vyšší zatížitelnost a prodloužila se také jeho životnost.**

## PARTICIPANTS IN CONSTRUCTION

| | |
|---|---|
| CLIENT | Technical administration of roads of Prague a.s |
| MAIN CONTRACTOR | PORR a.s. |
| DETAILED DESIGN | Valbek, s.r.o. and Pontex, s.r.o. |
| AUTHOR'S SUPERVISION | Novák a partner, s.r.o. and Pontex, s.r.o. |
| EXTERNAL CONSULTANT | Faculty of Civil Engineering, Czech Technical University in Prague |
| CONSTRUCTION TIME | 2020–2024 |

## ÚČASTNÍCI VÝSTAVBY

| | |
|---|---|
| INVESTOR | Technická správa komunikací, a.s. |
| GENERÁLNÍ DODAVATEL | PORR a.s. |
| PROJEKTANT RDS | Valbek, s.r.o., a Pontex, s.r.o. |
| AUTORSKÝ DOZOR | Novák a partner, s.r.o., a Pontex, s.r.o. |
| EXTERNÍ KONZULTANT | Stavební fakulta ČVUT v Praze |
| DOBA OPRAVY | 2020–2024 |

## INTRODUCTION

Barrandov bridge was built in the 1980s as a part of the basic road system in Prague. The bridge is composed of two independent continuous prestressed concrete structures with 6 spans of lengths 34.66 + 61 + 71 + 72 + 66 + 45.99 m (Fig. 1).

The width of the bridge is variable according to the alignment of individual traffic lanes. In the narrowest place, there are four traffic lanes in each direction and side walks at both sides of the bridge. The width of the bridge deck is about 40 m. The depth of the bridge deck is 3 m in the longest spans, and it is reduced towards the bridge ends to 1.6 m. The two independent bridge decks have box girders with 3 cells and vertical webs (Fig. 2).

The shape of the main piers in the river and on the bank is exceptional. The piers are shared by both bridges. The bottom wall part supports a prestressed transversal beam; both parts are located parallel to the flow of the river, which allows for a smooth flow of water even at high water levels. Short walls with bearings are placed on each transversal beam. They have an aesthetical function and also the arrangement of bearings allows for a reduction of the skewness of supports of the superstructure. The architectural shaping of piers is a determining factor also for aesthetics of other piers and abutments. The piers are founded on underground walls, only two piers in the river are founded on micropiles.

Fig. 1 Barrandov bridge – longitudinal section
Obr. 1 Barrandovský most – podélný řez

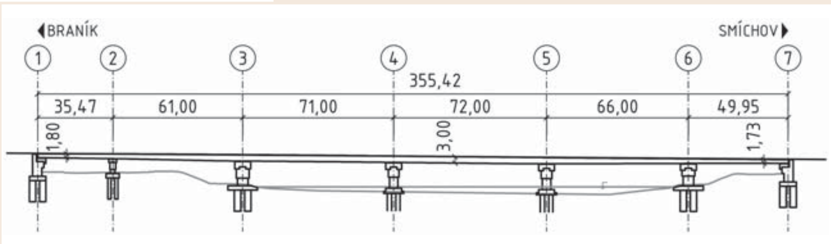



Fig. 2 Cross-section of the bridge above the pier in the river
Obr. 2 Příčný řez mostem nad návodním pilířem

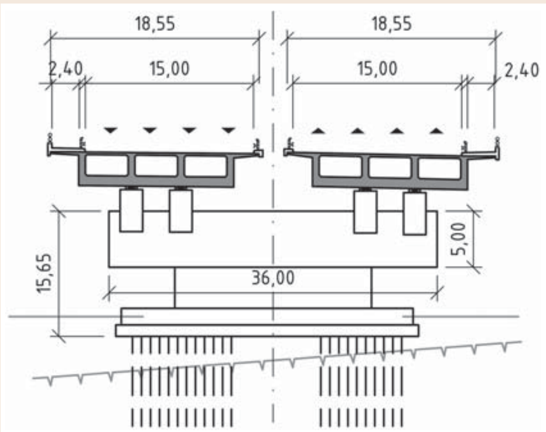
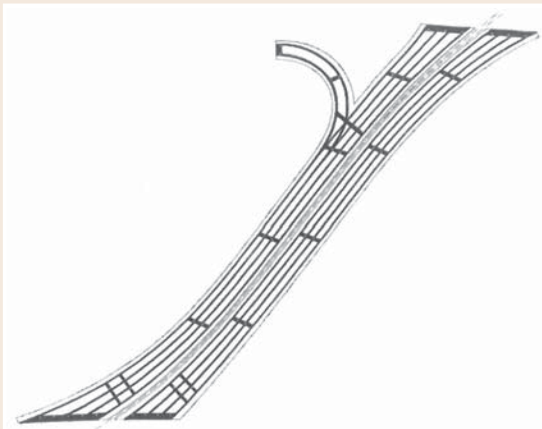



Fig. 3 Shape of the bridge in plan
Obr. 3 Tvar mostu v půdorysu



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The superstructure has an irregular shape in plan as it is shown in Fig. 3. It is made of prestressed concrete of the strength class originally designated B400, which roughly corresponds to the actual strength class C30/37. The original longitudinal prestressing is composed of tendons made of 12 strands Ø15.5 mm with a nominal load-bearing capacity of 2000 kN. 12 tendons are located in each web of the cross-section. The entire bridge was cast in sections on fixed scaffolding. Each section had the length of approximately one span, the working joints were placed in sections with low bending moments. The longitudinal tendons were connected in the working joints. No prestressing was designed in the transversal direction, where only mild reinforcement was used.

## DEGRADATION OF THE BRIDGE

The degradation of the bridges developed over a period of years. The bridge was heavily loaded by everyday traffic; nobody was encouraged for years to close the bridge and make a repair. Only in 2019, a detailed diagnostic survey was carried out, which discovered damage to the waterproofing, local degradation of concrete, reinforcing and prestressing steel in the working joints (Fig. 4). In addition, the service life of bearings and expansion joints had been exceeded. The owner required to extend the service life and also to increase the load-bearing capacity of the bridge, because of the importance of the bridge for road traffic in the city. Based on the diagnostic survey a plan of the general reconstruction was prepared. It comprised exchange of all the bridge equipment (pavement, side walks, drainage, bearings, expansion joints, etc.) and also a significant strengthening of the bridge. In 2020 and 2021, the substructure was reconstructed and it was planned to reconstruct the superstructure in 4 stages from 2022 to 2025.

## REPAIR OF THE SUBSTRUCTURE

Repair of the substructure started in 2020 – piers 2 and 3 and continued in 2021 – piers 4 and 5 in the river and piers 6 and 9. During the repair, no limitation of traffic was necessary. The surfaces were repaired; the drainage of water from horizontal surfaces was ensured. The ends of the transversal deep beams were checked because of the anchors of the pier prestressing. The state was found to be rather good; only locally, grouting under the anchors was required and protection against corrosion of the anchors was needed. No severe damage was found on the substructure. Fig. 5 shows one of the main piers after the repair.

Fig. 4 Corrosion of prestressing steel
Obr. 4 Koroze předpínací výztuže

## REPAIR AND STRENGTHENING OF THE SUPERSTRUCTURE

The bridge carries more than 140,000 vehicles daily. It was not possible to close the bridge. The client required to keep the traffic in 6 lanes from 8 lanes. Therefore, the reconstruction was planned in 4 stages always during the summer period, when there is less traffic and when only 2 lanes could be closed. The individual superstructures were strengthened by external tendons installed in the cells of the bridge (Fig. 6). 6 tendons located at the webs were composed of 7 strands Ø15.5 mm.

It was necessary to drill holes in existing transversal beams where the ducts for external prestressing were installed. It was necessary to avoid the damage of steel reinforcement in the existing structure. The drilling was controlled by laser which defined exactly the direction of drilling. The holes had a diameter of 92 mm. Because of the length of the bridge, all external tendons were connected in the transversal beam over pier 4 – in the centre of the bridge – by overlapping. The tendons were also prestressed from this place, because of the lack of space at the ends

Fig. 5 Pier in the river after the repair
Obr. 5 Návodní pilíř po opravě

Fig. 6 External prestressing for strengthening of the superstructure
Obr. 6 Vnější předpětí zesilující nosnou konstrukci

Fig. 7 Anchors of external tendons in the end transversal beam
Obr. 7 Kotvy vnějšího předpětí v koncovém příčníku

Fig. 8 Temporary supports of the bridge during the exchange of bearings
Obr. 8 Dočasné podepření mostu při výměně ložisek

of the bridge. The anchorage of external tendons in end transversal beams is shown in Fig. 7. The anchors were supported by a cast in situ UHPC block. Additional reinforcement under the anchors could not be installed. All additional prestressing was delivered with protection PL3 (electrically isolated tendons).

The service life of original pot bearings is about 30 years. All bearings were removed and replaced by new spherical bearings with identical technical parameters (Fig. 8).

Also, the expansion joints at the ends of the bridge were worn out. New ones were installed. Some of them are up to 60 m long due to the skew of the bridge. Because the repair of the bridge was executed in stages, always only on one half of the bridge, the expansion joints also had to be assembled in halves. The installation of the new expansion joints is illustrated in Fig. 9.

After removing of surface layers from the bridge deck, its surface was carefully geodetically measured. It was concluded that the additional layer is necessary for levelling the irregularities in the top surface of the deck, because only a thin two-layer asphalt pavement should have been used on the bridge. The specialists of the external consultant of the client (Czech Technical University in Prague) proposed application of the UHPC layer. It was considered as an optimal solution, because this UHPC layer could significantly contribute to increasing of the load-bearing capacity of the bridge and also it could be used as waterproofing which contributed to extending of the service life of the waterproofing and to acceleration of the repair. It was a completely new technology. The experience from abroad was supplemented by the experience from research activities and from application of UHPC at new structures. Additional experiments were carried out, which proved the watertightness of working joints.

The principle of strengthening with UHPC lies in concreting a thin layer of UHPC (5 to 15 cm) on a treated surface of the bridge deck. Its surface must be clean, rough and wet, so that the bond of UHPC and existing concrete would be able to carry the shear forces at the interface without the necessity of additional connectors. The connectors were used in a limited scale for technological reasons and for elimination of deformation of the UHPC layer due to early shrinkage because of the non-uniform drying, which is inevitable in spite of the careful curing. In the areas where the UHPC layer is in compression, it naturally increases the load-bearing capacity of the bridge deck and mainly of the slab above the cells. In the areas where the UHPC is in tension, the reinforcement was designed; it also significantly contributes to the load-bearing capacity and to the stiffness of the section. This was especially appreciated on the cantilevers. Some of them are exposed directly to the traffic load, and therefore their strengthening was most welcome.

If the UHPC layer should also fulfill a function of waterproofing, it is necessary to avoid (or to significantly reduce) cracking. A high fibre content in UHPC could satisfy such a condition. A fibre content exceeding 3% (vol.) of fibres allows for a watertightness, however, the viscosity of the fresh UHPC is very demanding on the production and pouring of UHPC. The service life of standard waterproofing is about 30 years, but the service life of the UHPC layer is not shorter than the service life of the concrete structure. The working joints in the UHPC had to be executed as watertight which required their special arrangement. Application of UHPC also contributes to sustainability, because it avoids future closures of the bridge as of the necessity to change the waterproofing.

At the south bridge – where the repair of the superstructure started, there was no experience with UHPC, therefore only its strengthening function was used, and

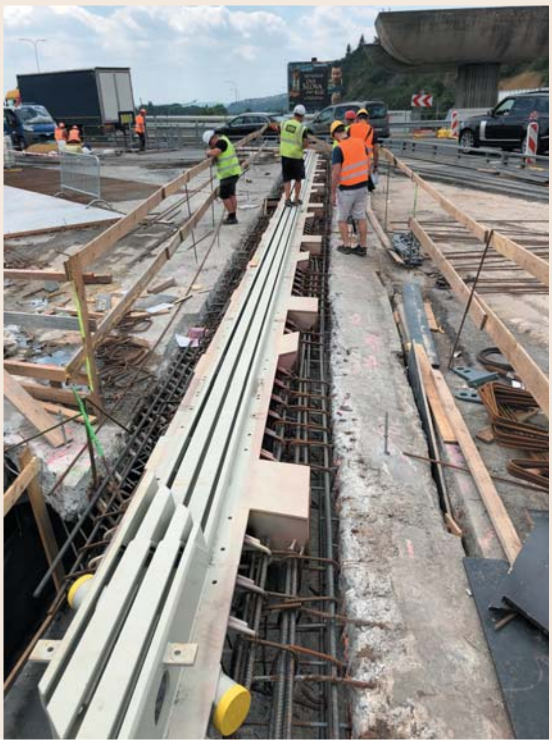

Fig. 9 Installation of the expansion joint
Obr. 9 Instalace mostního závěru

Fig. 10 Pouring of UHPC and its compaction with a vibration bar
Obr. 10 Pokládka UHPC a jeho hutnění vibrační lištou

Fig. 11 Side walk with higher railing and safety barrier
Obr. 11 Chodník s vyšším zábradlím a svodidlem

water proofing was made in a standard way with asphalt strips. During the two stages implemented in 2022 and 2023, some experience was gained and also experimental verification of watertight working joints was carried out. On the north bridge the full function (strengthening and waterproofing) was used. It made it possible to realize the two stages of reconstruction (planned for 2024 and 2025) in one extended season in 2024. The pouring of UHPC is shown in Fig. 10.

The application of UHPC showed its big advantages. It satisfied the function of the levelling, strengthening and waterproofing. The technology was developed during the four stages and it was concluded that the technology is applicable, advantageous and finally economical (especially from the point of view of life cycle costs). Because of the development of a number of details, it will be possible to use it for the strengthening of other bridges.

The drainage of the bridge was completely rearranged. The original system of external drainage gutters was replaced by curb drainage, which allows for a continuous flow of the water from the pavement surface. The new edge beams with the new safety barriers were made of concrete C30/37-XF4-XD3. The side walks also now transfer the lane for cyclists; the height of the railing had to be increased form 1.1 m to 1.3 m (Fig. 11).

## CONCLUSIONS

The heavily loaded Barrandov bridge was successfully reconstructed. The load-bearing capacity was increased, the durability was extended. New technologies were used, which allowed for improvement of the quality of the repair and for shortening of the construction time. The application of UHPC appeared as the most efficient solution especially if the damages discovered during the execution of the repair are considered. During the time of implementation, it was necessary to be flexible and to solve many unexpected issues. It was possible only because of the close cooperation of all participants of the construction and because of the support of the client for application of advanced technologies. The bridge after the repair is shown in Fig. 12.

## ACKNOWLEDGEMENT

During the repair the results of research project no. FV20472 supported by the Ministry of Industry and Trade of the Czech Republic were used. This support is gratefully acknowledged.

## MATERIAL USAGE (REPAIR OF THE SUPERSTRUCTURE)

| | TOTAL | PER 1 M² |
|---|---|---|
| UHPC | 1717 m³ | 0.113 m³ |
| PRESTRESSING STEEL | 146 t | 9.6 kg |

## SPOTŘEBA MATERIÁLU (OPRAVA NOSNÉ KONSTRUKCE)

| | CELKEM | NA 1 M² |
|---|---|---|
| UHPC | 1717 m³ | 0,113 m³ |
| PŘEDPÍNACÍ VÝZTUŽ | 146 t | 9,6 kg |

Fig. 12 Barrandov bridge after the repair
Obr. 12 Barrandovský most po opravě

# LIBLÍN BRIDGE OVER THE BEROUNKA RIVER: RECONSTRUCTION AS A HISTORICAL REPLICA

## LIBLÍNSKÝ MOST PŘES BEROUNKU: REKONSTRUKCE JAKO HISTORICKÁ REPLIKA

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**This article describes the complex reconstruction of the 1929 reinforced concrete arch bridge in Liblín. Due to the bridge's severe structural degradation and its status as a protected heritage monument, a historical replica approach was chosen instead of a conventional complete replacement. The paper presents the technical challenges of preserving the original load-bearing arches and foundations while replacing the deck and piers to meet modern safety and durability standards. This reconstruction serves as an example of maintaining visual authenticity and architectural integrity through replication within the context of 20th-century engineering heritage.**

**Článek se zabývá komplexní rekonstrukcí železobetonového obloukového mostu v Liblíně z roku 1929. Vzhledem k pokročilé degradaci konstrukce a zároveň jeho památkové ochraně byl namísto běžné kompletní výměny zvolen přístup formou historické repliky původního mostu. Text detailně popisuje technické výzvy spojené se zachováním původních nosných oblouků a základů při současné náhradě mostovky a pilířů tak, aby vyhovovaly moderním standardům bezpečnosti a trvanlivosti. Tato rekonstrukce je příkladem zachování vizuální autenticity a architektonické integrity v kontextu inženýrského dědictví 20. století díky vytvoření repliky mostu.**

Fig. 1 View of the original bridge
Obr. 1 Pohled na původní most

Reinforced concrete bridges from the interwar period represent an important part of Central European engineering heritage. The bridge in Liblín, located on the Berounka River north of Plzeň, is a notable example. Completed in 1929 based on a design by engineer Zdeněk Prošek, the structure consisted of two 50-metre reinforced concrete arches with a central pier founded in the riverbed. The deck, supported by a system of beams and thin wall supports in open spandrel, reflected construction practices of that era, including a large number of expansion joints.

Beyond its structural qualities, the bridge stood out for its architectural composition—particularly the articulation of the piers, the proportioning of the arches, and the distinctive railing design combining concrete and steel elements. Therefore, in 2019 it was listed as a cultural heritage monument.

After nearly ninety years of service, the bridge was found to be in a critical condition during an inspection in 2020. Severe deterioration of the concrete affected key structural elements, especially the cross beam above the central pier and the bases of the wall supports. Material diagnostics confirmed very low compressive strength, high moisture ingress, chloride contamination, and frost damage in most components. In contrast, the concrete of the main arches showed adequate strength and durability. This finding proved decisive for the selection of the reconstruction strategy.

Rather than a conventional replacement, the project was conceived as a historical replica, combining the preservation of essential load-bearing elements with a faithful restoration of the original architectural expression. The existing arches and foundations were retained, while the deck, piers, and all wall supports in open spandrel were completely rebuilt. This approach significantly reduced both construction complexity and environmental impact, as it minimized intervention in the river channel.

A key improvement was the redesign of the structural system into two expansion units, a significant reduction from the fourteen units in the original bridge. The excessive number of joints had been the primary cause of long-term deterioration due to water leakage. The new structure utilizes only three expansion joints, located at the abutments and at the central pier. These are designed as low-noise sinusoidal joints to minimize traffic-induced noise. Overall, the radical reduction in the number of joints, combined with their advanced technical design, significantly enhances both the durability of the structure and user comfort.

The bridge was widened to meet the current regulatory standards, including a single-sided pedestrian walkway. This widening was carried out asymmetrically to minimize torsional effects while preserving the visual balance of the structure. A specific design challenge concerned the twin-column piers at the arch springings: structurally, the deck

is now supported by a single column, while the second is executed as a non-load-bearing replica, maintaining the original appearance. This solution illustrates the principle of visual authenticity, where architectural integrity is preserved despite necessary changes in structural behaviour.

The careful restoration of architectural details was equally important. Heritage-defining features – such as the geometry of the piers and arches, the parapets, and the railing integrated into a broader cultural and historical context. By combining modern engineering requirements with a sensitive approach to heritage preservation, the project delivers a structure that meets contemporary standards while retaining the spatial and visual qualities of the original 1920s design. In an international perspective, it provides a valuable example of how reinforced concrete heritage can be preserved through informed and technically rigorous replication rather than mere conservation.

Fig. 2 Demolition of the bridge deck
Obr. 2 Demolice mostovky

Fig. 3 Construction of the new bridge
Obr. 3 Výstavba nového mostu

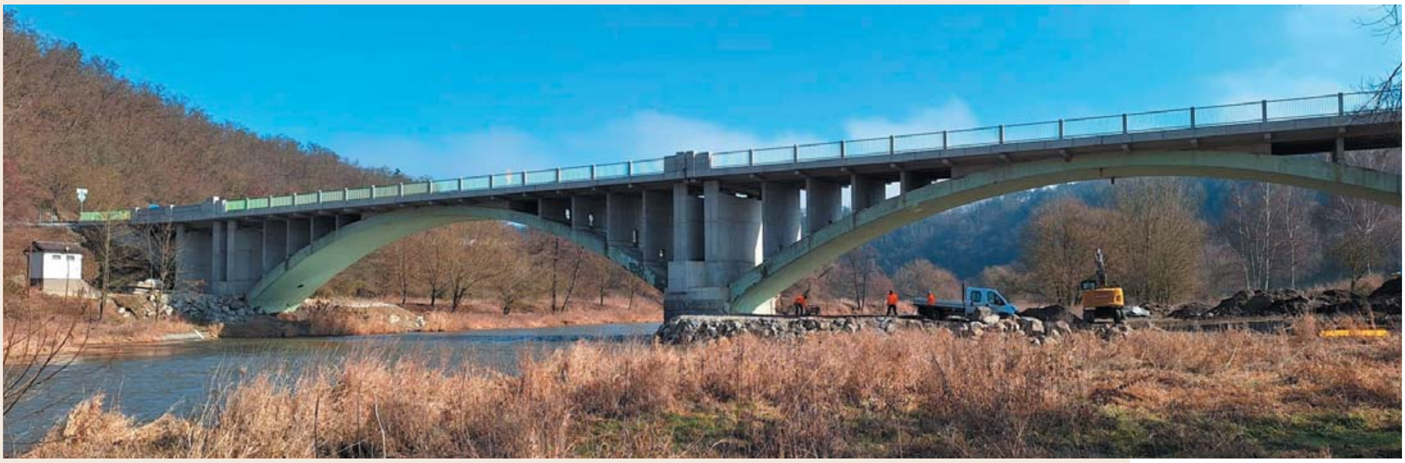

Fig. 4 View of the new bridge (prior to arch remediation)
Obr. 4 Pohled na nový most (před provedením sanace oblouků)

design – were reproduced as replicas. In some cases, adaptations were required to meet modern standards. For example, the original horizontal steel railing infill was replaced by vertical elements to comply with current safety regulations. Furthermore, the concrete railing elements were modified to accommodate the new layout of expansion units and the installation of expansion joints, all while maintaining the structure's overall visual character.

During the reconstruction itself, the existing arches were extensively used as a work platform, allowing demolition and subsequent construction to proceed without extensive temporary falsework in the river.

The Liblín bridge reconstruction demonstrates that even quite utilitarian concrete structures can be successfully

## MATERIAL USAGE: (SUPERSTRUCTURE)

| | TOTAL | PER 1 M$^2$ |
|---|---|---|
| STRUCTURAL STEEL S355 | 0 t | 0 kg |
| CONCRETE C30/37 | 448.5 m$^3$ | 0.354 m$^3$ |
| PRESTRESSING STEEL | 0 t | 0 kg |
| REINFORCING STEEL | 130.5 t | 103 kg |

## SPOTŘEBA MATERIÁLU (NOSNÁ KONSTRUKCE)

| | CELKEM | NA 1 M$^2$ |
|---|---|---|
| KONSTRUKČNÍ OCEL S355 | 0 t | 0 kg |
| BETON C30/37 | 448,5 m$^3$ | 0,354 m$^3$ |
| PŘEDPÍNACÍ VÝZTUŽ | 0 t | 0 kg |
| BETONÁŘSKÁ VÝZTUŽ | 130,5 t | 103 kg |

Fig. 5 Original railing
Obr. 5 Původní zábradlí

Fig. 6 Replica railing
Obr. 6 Replika zábradlí

# RIVIERA FOOTBRIDGE IN BRNO
# LÁVKA RIVIÉRA V BRNĚ

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**The footbridge was built in the city of Brno, Czechia, and connects the Riviera outdoor swimming pool and the adjacent Brno exhibition grounds, where the main obstacle is the newly built large city ring road. The total length of the footbridge is 284 m. The superstructure consists of a prestressed concrete spine beam with a height of 0.9 – 1.7 m, which is supplemented by transverse ribs as a bearing element in the transverse direction. The longest span is situated across the city ring road with a length of 47 m. The width of the superstructure is 6.7 m and the footbridge thus creates a good connection with the main Brno recreational area.**

**Nově postavená lávka v Brně spojuje venkovní koupaliště Riviéra a přilehlé veletržní prostory, které od sebe rozděluje nově postavený velký městský okruh. Celková délka lávky je 284 m. Nosná konstrukce je tvořena páteřním předpjatým betonovým nosníkem s výškou 0,9 – 1,7 m, který je doplněn o žebra v příčném směru. Nejdelší pole je umístěno nad městským okruhem a má délku 47 m. Šířka lávky je 6,7 m, a tvoří tak velkorysé spojení s brněnskou rekreační oblastí.**

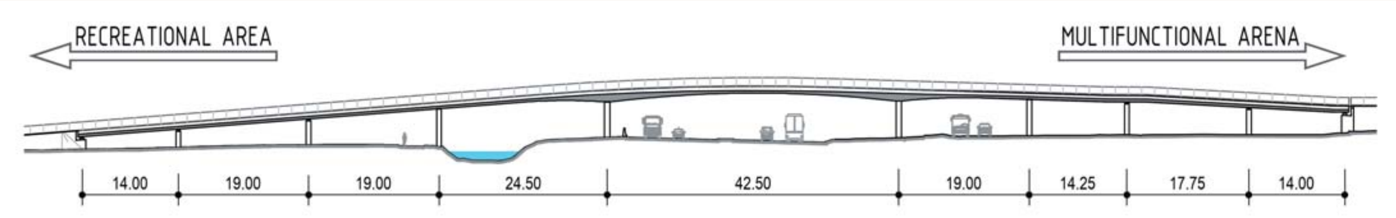



Fig. 1 Longitudinal section
Obr. 1 Podélný řez

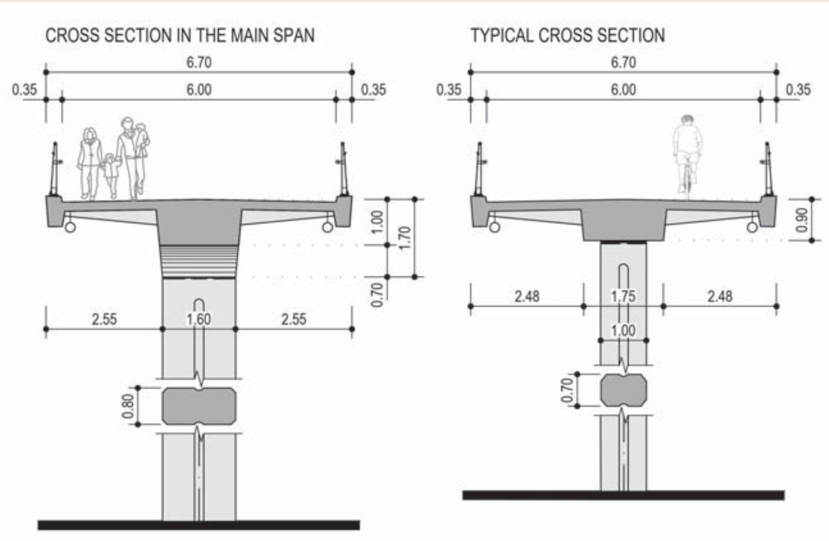
## OVERALL CONCEPT

The footbridge crosses the Svratecky creek towards the main obstacle in the form of the newly built city ring road. The main crossing obstacle is a two-way, three-lane road with the largest span in both directions.

The route of the bridge deck is S-shaped in terms of direction, starting with a right-hand directional curve with a radius of 70.0 m, followed by a straight length of 57.7 m and continuing in a left-hand directional curve with a radius of 17.0 m, see Fig. 1. In front of the first abutment, the footbridge connects to the existing footpath and the footpath from the Riviera swimming pool area. Abutment 10 is constructed by approach retaining walls. The cross slope on the footbridge switches from one-sided 2.0% to roof-centred and then to one-sided towards abutment 10.

The vertical alignment is designed according to obstacles and access level, therefore a combination of 8% and 5% slope is used along the footbridge.

Fig. 2 Cross section
Obr. 2 Příčný řez




## SUBSTRUCTURE

The foundation structures are designed as monolithic reinforced concrete supported by concrete piles Ø900 mm. The cross-sections of the pillars of the internal supports P02 – P04 and P07 – P09 are designed in dimensions of 0.7 × 1.0 m with chamfered corners, the cross-sections of the pillars P05 and P06 (in the place of the main span) are designed in dimensions of 0.8 × 1.6 m. The shafts of all piers are provided with fillets in the form of circular sections at both surfaces in their longitudinal axis, and the fillets are then provided at the top of the piers, smoothly transitioning into their full cross-section. The superstructure is supported on piers P04 to P07 by means of concrete hinges, and on the other piers by transverse fixed bearings.

The footings of the two end abutments are of analogous design, with two edge bearings and one in the middle for the longitudinal guidance of the superstructure.

## SUPERSTRUCTURE

The superstructure of the footbridge consists of a spine beam 0.90 m high and 1.75 m wide. Between supports P5 and P6, the height of the beam above the supports is increased to 1.7 m by a parabolic camber, with a spine beam width of 1.60 m, which follows the side inclination of the beam in a typical section. In the centre of the main bay, the height of the spine beam is 1.00 m. The typical width of the superstructure is 6.7 m with a clear width of 6.0 m, which is stretched from the spine girder by 0.2 m wide reinforced concrete ribs, thus making the structure lighter, see Fig. 2. The longitudinal axial distance of the ribs varies from 1.80 m to 1.80 m. The axial distance of the ribs is always bisected by the axis of the support. Two atypical ribs of 0.6 m width are designed in place of the P08 support, which will be used in the future to pin the additionally welded connection of the connecting footbridge between the cableway station and the bridge deck, that will carry the additional load from

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this connecting footbridge. To eliminate tensile reactions in the bearings at abutment 10 due to the torsion of the structure, the cross-section along the entire length of bay 9 (from abutment 9 to abutment 10) transitions to a constant cross-section corresponding to the typical cross-section at the ribs.

The superstructure has integrated cornices, which are formed by a soffit of 120 mm height. The width of the cornice on the upper surface is 350 mm, into which the railings are anchored.

The prestressing of the superstructure is designed along its entire length using a total of 4 bonded cables with 12 strands in a plastic duct. In the area of the main span, the prestressing is reinforced by an additional 4 pieces of 12-strand cables anchored with dead anchors in the area behind the P06 support.

## CONSTRUCTION

After the foundation and substructure were constructed, the space framework construction was realized for superstructure casting. In the first stage, the longest section of 90.6 m from abutment 10 was cast. Half of the continuous cables were anchored in the working joint and then coupled to the next stage.

This was followed by two phases of concrete pouring, which ended in December 2023. Once the superstructure was completely prestressed, the temporary support in the main span was removed.

A static load test was performed on the structure, where the load consisted of 4 fully loaded trucks. The test showed very good concurrence with the assumptions of the calculations.

A dynamic test of the structure was carried out using loads from pedestrian movement and also loads at selected points according to the actual shapes of the structure. The first bending natural frequency was determined by the test to be 1.679 Hz and the first torsional frequency was determined to be 2.287 Hz.

Fig. 3 View of completed structure during winter
Obr. 3 Pohled na dokončenou konstrukci v zimě

Fig. 4 Riviera footbridge
Obr. 4 Lávka Riviéra

## SUMMARY

The new footbridge has become an important link in this up-and-coming area of Brno, which will simplify movement and bring people closer to the recreational area directly from the centre. The footbridge is carefully inserted into the surrounding space. The designer of the all parts of documentation is Strasky, Husty and partners.

Fig. 5 Riviera footbridge with LED lighting
Obr. 5 Lávka Riviéra s LED osvětlením pohledu

# VELEDROM UHPFRC SEGMENTAL FOOTBRIDGE IN BRNO
# UHPFRC SEGMENTOVÁ LÁVKA VELODROM V BRNĚ




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**The new crossing of 7 road lanes is designed as a single-span parapet beam with overhung parts. The superstructure is composed of 7 UHPFRC segments that are prestressed by unbonded strands. The main span is 53.8 m with an overhang of 2.55 m at each side. The cross section in the main span is a 'U shape' with a recessed wall, the total height is 1.475 m. The overhung parts are "L shape" segments and they are connected to the U shape by a combination of prestressing bars and tendons.**

**Nové přemostění sedmi jízdních pruhů je navrženo jako parapetní nosník o jednom poli s převislými konci. Nosná konstrukce se skládá ze sedmi UHPFRC segmentů a je předepnuta nesoudržnými kabely. Rozpětí hlavního pole je 53,8 m a převislé konce jsou dlouhé 2,55 m. Příčný průřez hlavního pole je parapet ve tvaru U s prolisy ve stěnách a celkovou výškou 1,475 m. Převislé konce jsou ve tvaru L a jsou připnuty k hlavnímu poli tvaru U pomoí kombinace předpínacích kabelů a tyčí.**

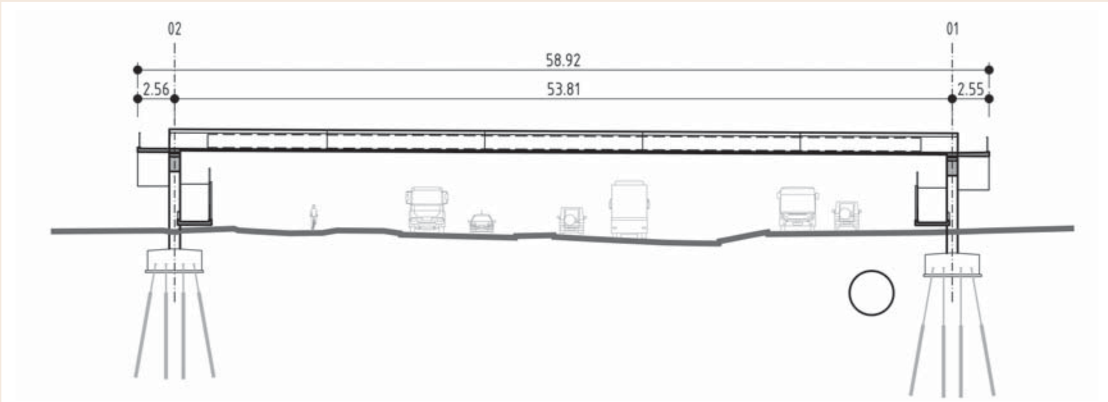



Fig. 1 Longitudinal section
Obr. 1 Podélný řez

## OVERALL CONCEPT

The footbridge is primarily planned to provide connectivity between the bus stop and the adjacent complexes. A future realignment of the urban road circuit is planned for this location. However, this layout requires a major update of the traffic design, which is not compatible with the location of the footbridge supports. For this reason, the footbridge is designed as a temporary bridge. It is also allowed that the footbridge superstructure may be dismantled and relocated during its lifetime. Nowadays, every newly designed bridge structure responds to a specific location on the site, and so it is difficult to relocate a given length of structure and use it in another location. Therefore, a design approach was chosen to allow adaptation of the designed footbridge to a different length arrangement.

The footbridge bridges the obstacle formed by the large city ring road itself with five lanes and the two-way Křížkovského Street. In view of the complex space, a solution without intermediate supports was chosen. The span of the main span reaches 53.8 m and is supplemented by overhanging cantilevers of 2.55 m, see Fig. 1.

Along the abutments there is an access walkway to the footbridge on both sides with a gradient of 7.8 %. The abutments are formed by a central concrete wall. This arrangement was intentionally chosen because of the spatial constraint of the main sewer. The abutments are supplemented with separate staircase constructions at the intermediate landing, which are aligned with the walkways. The clear width on the footbridge between the handrails is 2.0 m and the same width is used on the access walkways.

## FOUNDATION

Both abutments are depth based on a group of Ø108/16 tubular steel micropiles with a total of 28 pieces per abutment. At abutment OP1 the micropiles are realized in length of 9.0 m and at abutment OP2 the total length is 8.0 m. The micropiles are angled in a transverse direction to increase stiffness against horizontal forces.

## SUBSTRUCTURE

The abutments themselves consist of a central concrete wall, that is relieved by large openings. The structure is

Fig. 2 Abutment with prestressing tendons layout
Obr. 2 Opěra s uspořádáním předpínací výztuže

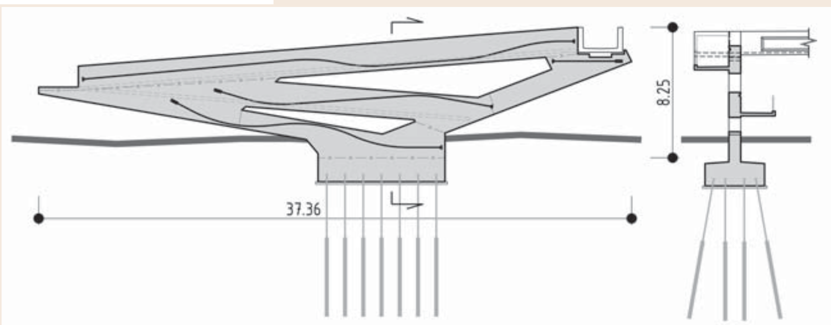
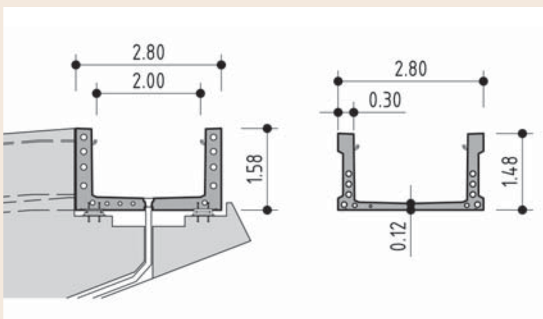



Fig. 3 Cross section
Obr. 3 Příčné řezy



thus at the borderline between beam and wall elements. The total length of the abutment is 37.4 m and the wall is 0.75 m thick. The abutment is supported for a length of 8.0 m on the foundation. The wall has an asymmetrical cantilever layout of 17.6 m on the stair side and 11.8 m on the superstructure side. The asymmetry is deliberately chosen to balance the mass of the structure. The entire abutment structure is prestressed with a system of 5 traced tendons per abutment, see Fig. 2. The tendon routes correspond to the stresses on the structure and the construction phase, which was carried out in four stages. At the same time, the tendons ensure that no stresses are reached above the tensile strength of the concrete under service loads, so that this exposed element is not prone to water ingress into the cracks.

Reinforced concrete slabs are cantilevered out from the central wall to form the supporting structure for the access walkways. The transverse slope is outward from the cantilevers to provide a connection detail to the bearing wall without water accumulation. Along the access walkways, a niche is formed in the central wall to accommodate a handrail.

## SUPERSTRUCTURE

The structural layout of the footbridge was chosen as a segmental one, with joints between the segments being filled with high-strength mortar. Regarding the versatility of use and long-term durability, a material solution made of UHPFRC was chosen. The main benefit is the high level of material savings, which is reflected in the weight of the individual segments (maximum 28 tonnes).

The superstructure is supported by a pair of elastomer bearings on the abutment. The combination of fixed and omnidirectional bearings on each support was chosen. Due to the plan layout, it is necessary to allow for deformation from temperature and rheological effects, which would not allow for any other arrangement due to the action of the abutment as a cantilever.

The typical shape in the main span is a 1.48 m high U-shaped parapet beam, where the parapet reaches a maximum width of 300 mm at the top and is relieved by recessing a thickness of 95 mm, see Fig. 3.

Four 12-strand prestressing tendons are designed in each parapet and two 7-strand tendons in the bottom slab.

The recess in the parapet is omitted at the bearing points for the purpose of anchoring the cables, and at the same time the bottom slab is thickened by 100 mm. Three Ø32 Y1050 prestressing bars are guided in the bottom slab from the last U-shaped segment to the overhanging L-shaped ends.

The prestressing is in protection level PL2 where plastic tendon ducts and segment joints are used. At the same time, however, it was necessary to use non-cohesive prestressing in the form of 'monostrands' to allow later de-anchoring and removal from the structure for possible relocation of the footbridge to another location.

During dynamic tests, the natural shapes of the footbridge and the response to a synchronous loading vertical impulse of multiple persons, free pedestrian movement and transverse impulse were monitored. The first bending natural frequency was determined by testing to be 1.172 Hz.

## SUMMARY

The footbridge appropriately complements the Brno Exhibition Centre, which has always been a showcase of the most modern construction offered in its time. The overall concept of the footbridge follows this tradition.

Fig. 4 Detail of superstructure and substructure connection
Obr. 4 Detail uložení nosné konstrukce s na spodní stavbu

Fig. 5 Wall abutment with openings
Obr 5 Stěnová opěra s otvory

Fig. 6 Velodrom footbridge
Obr. 6 Lávka pro pěší Velodrom

# ŠTVANICKÁ FOOTBRIDGE IN PRAGUE
# ŠTVANICKÁ LÁVKA V PRAZE

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**The footbridge spans the Vltava River in Prague, connecting the districts of Holešovice and Karlín with a ramp on Štvanice Island. Designed by the architectural studio AI Praha, and with Skanska a.s. as the general contractor, this segmental bridge is constructed from ultra-high performance fibre reinforced concrete – UHPFRC (hereinafter referred to as UHPC). The main specifications include the use of white concrete with $f_{ck}$=120 MPa. The main branch of the bridge is 300 m long and 5 m wide, while the ramp extends 75 m with a width of 4 m, comprising a total of six spans. The final span, which exceeds 40 m in length, is equipped with a pivot joint and a hydraulic lifting system to clear the Q2002+1 m flood level. The structure consists of 57 H-shaped segments arranged in both vertical and horizontal curves. The beams are joined using epoxy adhesive and four post-tensioned cables, each containing 19 strands, with a PL3 protection level.**

**Lávka překonává řeku Vltavu v Praze a spojuje Holešovice a Karlín, přičemž je doplněna rampou na ostrově Štvanice. Jedná se o segmentový most z ultra vysokohodnotného vláknem vyztuženého betonu (UHPFRC, dále jen UHPC), navržený architektonickým studiem AI Praha a realizovaný společností Skanska a.s. Hlavní specifikací konstrukce je bílý beton s charakteristickou pevností v tlaku $f_{ck}$ = 120 MPa. Délka hlavní větve činí 300 m při šířce 5 m, rampa má délku 75 m a šířku 4 m. Konstrukce je rozdělena do šesti polí, přičemž poslední pole o délce přes 40 m je opatřeno otočným kloubem a hydraulickým zdvihem nad povodňovou úroveň Q2002 + 1 m. Nosná konstrukce je tvořena segmenty tvaru „H" v celkovém počtu 57 kusů. Most je veden ve výškovém i směrovém oblouku. Spojení nosníků je provedeno pomocí epoxidového lepidla a čtyř předpínacích kabelů (19 lan) o odolnosti PL3.**

Fig. 1 The footbridge at dusk
Obr. 1 Lávka za soumraku

## 1. INTRODUCTION

The newest footbridge for pedestrians and cyclists in Prague across the Vltava River connects the developing districts of Holešovice and Karlín, providing access to the recreational area of Štvanice Island. An international competition was organized by the investor, the City of Prague, which was won by the design of Ing. arch. MgA. Petr Tej, Ph.D. and Ing. arch. Marek Blank. The project documentation (for building permit and tender) was prepared by the competition winner, AI Praha s.r.o., while the implementation documentation (issued for construction) was prepared for the general contractor SKANSKA a.s. by the design office TOP CON servis s.r.o.. The prefabricated UHPC components were supplied by KŠ PREFA s.r.o., Štětí plant.

The bridge structure is designed as a continuous parapet beam made of UHPC with six spans and a two-span descending ramp to Štvanice island. The width of the main bridge superstructure is 5.0 m (4.0 m clear width), and the width of the ramp superstructure is 4.0 m (3.0 m clear width). The substructure consists of massive, reinforced concrete, deep-founded, and white to match the superstructure pigment. The bridge's longitudinal profile was designed to follow the road levels on both riverbanks. The requirement to keep the bridge 1.0 m above the 2002 flood level is met by lifting a portion of the end span on the Holešovice side while the bridge is closed to traffic. The preparation of the IFC documentation and the bridge construction were carried out under close cooperation between the architect, the designer, and the contractor to ensure the architectural intent was fulfilled.

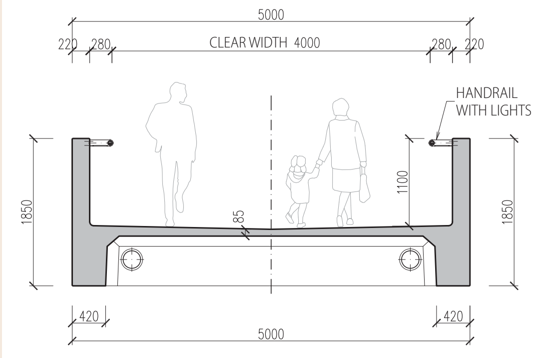


Fig. 2 Segment cross-section
Obr. 2 Příčný řez segmentem

## 2. TECHNICAL DESCRIPTION

### 2.1 Superstructure

#### 2.1.1 Segmental Superstructure

The bridge superstructure is designed as a continuous parapet beam with six spans (18.0 + 5×55.4) made of prestressed UHPC (class C120), with a two-span descending ramp (13.05 + 49.35). The structure is segmental (57 individually designed units), longitudinally post-tensioned, and monolithic reinforced concrete at the point of the ramp branching. In the end span on the Holešovice side, at a distance of 11.0 m from pier P50 (a point of zero moments), a joint is inserted into the superstructure allowing the end of the span to be lifted by 3.1 m, thus meeting the flood requirement.

The H-shaped cross-section (Fig. 2) consists of a pair of parapet beams 1.85 m high with variable widths (0.22 m above the deck and 0.42 m below the deck). In a typical segment, these beams are connected by two cross-beams (0.25 m wide) and a deck slab with a minimum thickness of 85 mm. The upper surface of the deck has a 2% cross-slope. The height of the parapet above the deck is 1.10 m. The width of the main segments is 5.00 m, while the ramp segments are 4.0 m wide. Typical segments are 5.54 m and 6.00 m long; ramp segments are 5.30 m. The units were manufactured using a short-line match-casting simulation, creating joints with shear locks that were bonded with epoxy during assembly. For most units, the bonded joint does not span the entire cross-section; a gap is left in the deck area to allow drainage directly beneath the bridge.

#### 2.1.2 In-situ Cast Superstructure

The branching point where the ramp separates from the main bridge is designed as a combination of prefabricated parapet beams and a monolithic UHPC slab (class C110) with a thickness of 0.75 m. The prefabricated parapet beams follow the shape and height (1.85 m) of the segmental units and are curved both vertically and horizontally with various radii, forming the external visible surfaces and acting as the "starter" for the monolithic deck slab. Longitudinal prestressing cables pass through these parapet beams.

#### 2.1.3 Prestressing

The superstructure is longitudinally post-tensioned by four cables, each with 19 strands (15.7 mm steel Y1860), running through each parapet beam. Given the segmental design, a PL3 protection level was used. Post-tensioning was carried out in five stages; cables with a maximum length of 120 m are overlapped and anchored in specific segments.

#### 2.1.4 Hydraulic Lift

As mentioned, a stainless-steel joint (grade 1.4462) is integrated into the parapets of the end span near the Holešovice bank. This joint, combined with hydraulic jacks and a steel support structure located at the Holešovice abutment, allows the bridge end to be raised by 3.1 m to clear the flood level.

### 2.2 Foundations and Substructure

The substructure of the footbridge, with the exception of abutment OP 60, is founded on large-diameter bored piles (LDP) with a diameter of 880 mm and a maximum length of 15.50 m, installed under the protection of steel casing. Standard piers with expansion bearings are founded on 6 LDPs, while pier P20, which carries the fixed bearing, is founded on 8 LDPs. The lower end abutments OP00 and R20 are founded on a single-row pile bent consisting of a pair of LDPs. The pile toes are embedded into load-bearing pre-Quaternary rock consisting of clayey-silty shales at various stages of weathering (classes R5 – R3 according to ČSN 731001), which were encountered at shallow depths beneath the site, predominantly covered by varying thicknesses of anthropogenic fill (backfill).

Due to the restricted site conditions at Bubenské nábřeží, Abutment OP 60 is founded on a micropile capping slab (micropile raft) comprising steel tubes (108/16 mm, 10.0 m long) installed within 270 mm boreholes. The foundation works also involved extensive temporary geotechnical structures. The piers within the Vltava River and Pier P10 in its immediate vicinity were constructed inside sealed cofferdams. To facilitate the construction of Abutment OP 60, a soldier pile wall, anchored at three levels, was installed to support the Bubenské nábřeží roadway. Additionally, deep foundations were implemented for temporary structures within the Vltava riverbed

The footbridge substructure strictly follows the architectural design from the competition. The piers are massive structures, made of reinforced concrete (RC), with

Fig. 3 Detail of the decorative brass handrail
Obr. 3 Detail ozdobného mosazného madla

Fig. 4 Handrail with integrated LED light
Obr. 4 Zábradelní madlo s LED osvětlením

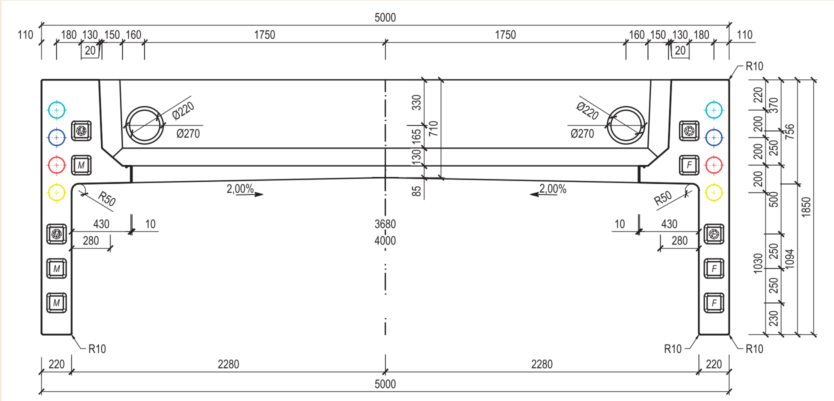


Fig. 5 Standard segment, head-on view in casting position

Obr. 5 Standardní segment, čelní pohled v licí poloze

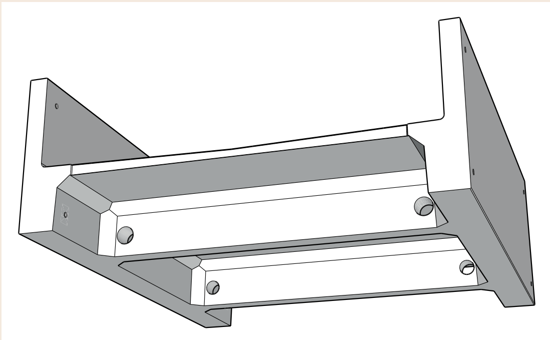

Fig. 6 3D view of standard segment

Obr. 6 3D pohled na standardní segment

a rectangular cross-section of 5.00 × 1.85 m, made of white concrete (class C30/37 XF4), matching the dimensions of the bridge segments. The height of the piers ranges from 7.65 m to 13.35 m.

The end abutments on the Karlín bank and Štvanice island are of low-height, recessed into the ground as much as possible, consisting of a bearing seat (sill beam) embedded into the LDPs and short suspended wing walls with parapets connected to the superstructure. The end abutment on the Holešovice bank is a monolithic RC box structure, providing space for the installation of hydraulic cylinders (piston cylinders) that allow the end span of the superstructure to be raised.

### 2.3 Equipment

The equipment reflects the material properties of UHPC (minimal absorption and high durability). The walkable surface of the segments requires no additional insulation or finishing; an anti-slip matrix was placed directly into the moulds. The in-situ cast branching area was leveled and unified with an epoxy-polyurethane coating. Drainage is achieved through transverse gaps between segments. Expansion joints are made of stainless steel with "finger" designs suitable for cyclists. The parapets serve as railings (1.10 m high) and are topped with a decorative brass handrail (Fig. 3) containing an integrated LED strip for public lighting (Fig. 4).

## 3. STATIC AND DYNAMIC ANALYSIS

The UHPC structure was designed and assessed in accordance with the current European and Czech technical standards. For the design and assessment of the superstructure, Methodologies 1, 2, and 3 developed by the Klokner Institute of CTU (Czech Technical University) in 2015, which were in force at the time of the tender design documentation, were applied.

The global static and dynamic analysis of the structures was performed using spatial (3D) models that account for all geometric and structural constraints of the individual components, including the bifurcation deck structure.

A decisive requirement for the prestressing design was the maintenance of a minimum compressive reserve of 1.0 MPa at the joints of the transversely segmented structure under the characteristic load combination.

For the detailed assessment of the UHPC components with tensile strength, numerous nonlinear local models of the superstructure were developed in the ATENA software in collaboration with the Klokner Institute. These models account for the tensile phase of the UHPC stress-strain diagram. Nonlinear calculations were primarily used to assess the bifurcation area where the descending ramp to Štvanice Island detaches; this section consists of a combination of precast parapets coupled with a monolithic slab via reinforcement. Given that the prestressing tendons are routed only within the parapets along a minimum radius, this section is subjected to a combination of vertical loads and radial tensile forces resulting from prestressing. Furthermore, nonlinear analyses were conducted to assess the thin, unreinforced deck slab under local loading, the anchorage zones of the steel hinges within the segments, the segment faces at the bonded joints, and many other details.

These local nonlinear calculations demonstrated that the crack widths are minimal, remaining well below the maximum allowable limit of 0.2 mm.

Based on the dynamic assessment, the installation of two tuned mass dampers (TMDs) was anticipated. The necessity and precise mass of these dampers were verified through dynamic testing following the completion of the footbridge. Subsequently, they were installed in the third span and within the descending ramp

## 4. UHPC MIX DESIGN FOR IN-SITU CASTING

The specific nature of this project lies in the combination of two entirely different structural components: precast segments, typical for most projects of this type, and a bifurcation section designed as a monolithic structure.

The bifurcation is located at the junction where the footbridge deviates from the main route towards Štvanice Island. Due to its dimensions, it could not be produced as a precast element. The total volume of concrete for the bifurcation is 125 m$^3$.

UHPC has a highly specific composition and production process. It was therefore necessary to develop a mix that could be easily produced like conventional concrete while achieving the required mechanical properties of

Fig. 7 Cast elements of the parapets, segments

Obr. 7 Vybetonované prvky parapetů, segmenty

Fig. 8 Cast segment with lifting and turning devices
Obr. 8 Vybetonovaný segment se zvedacím a otáčecím zařízením

Fig. 9 All-steel mould
Obr. 9 Celoocelová forma

Fig. 10 Placing of a segment for the 4th stage from a pontoon
Obr. 10 Osazování segmentu do čtvrté etapy z pontonu

Fig. 11 View of two mobile cranes operating on the Vltava River
Obr. 11 Pohled na dva mobilní jeřáby pracující na Vltavě

UHPC. From a series of laboratory trial batches, those meeting the short-term mechanical requirements were selected for further testing at a full-scale batching plant. Approximately 20% of all trial mixes were simultaneously tested using a production-scale mixer. After six months, the mix design was refined to meet both the design specifications and the technical capabilities of a standard concrete plant.

A second round of testing, evaluation, and optimization followed. According to the original schedule, casting was planned for the spring under ideal conditions. In collaboration with CTU, we measured temperature development in a 2 m$^3$ test block, which allowed colleagues from CTU to calculate the expected internal temperatures within the structure. The results were at the upper limit of acceptability. Once it became clear that the casting would be shifted to warmer months, a major revision of the mix design was required to extend its workability and reduce the core temperature of the structure. This was achieved through a water-cooling system. Its design was supported by Professor Vít Šmilauer, PhD, and its efficiency was subsequently verified during further trial pours.

The final mix design achieves a compressive strength of 120 MPa and a flexural tensile strength of 19 MPa. The mixing time per batch is 4 minutes, with a verified workability of 8 hours. Furthermore, the mix does not adhere to the drum of the transit mixers, and transport losses are negligible.

Fig. 12 Aerial view of the first 29 assembled segments
Obr. 12 Letecký pohled na prvních 29 sestavených segmentů

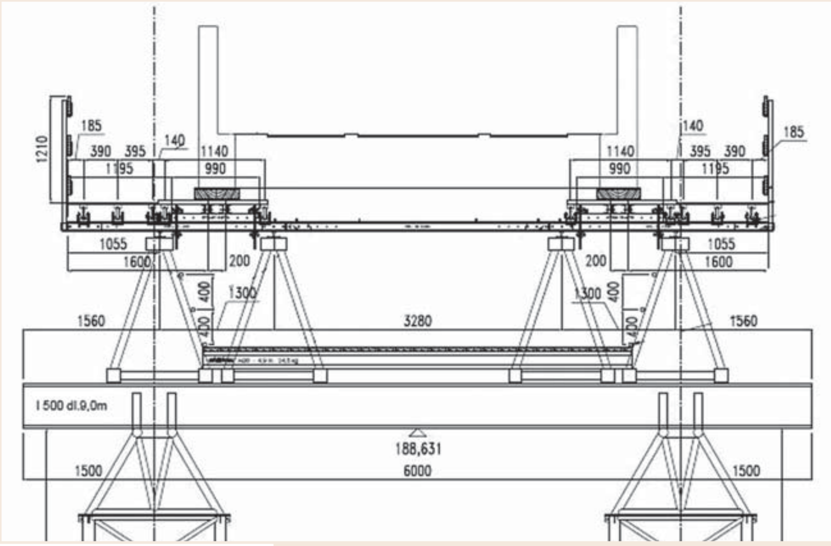


Fig. 13 Supporting system – cross-section
Obr. 13 Podpěrné konstrukce – příčný řez

## 5. PRECAST ELEMENTS

### 5.1 The segments and parapet beams

The superstructure was designed as a continuous parapet beam with six spans, including a two-span descending ramp to the island. It consists of 57 individually designed segments made of white UHPFRC 120, longitudinally post-tensioned together into a single structure, as described above.

One of the 55 typical segments and its cross section are shown in Fig. 5 and 6. The last produced and largest 2 segments with the embedded steel joint were longer and the thickness of their parapets were wider in the part with the joint.

The 7 precast parapet beams (Fig. 7), with 2 of them curved both vertically and horizontally, and 5 of them straight, altogether formed the outer shape and served as lost formwork for the in-situ cast sections. The entire superstructure is longitudinally prestressed with four cables of 19 tendons 15.7 mm Y1860 steel in each parapet beam. Due to the segmental solution, a prestressing system with PL3 protection and new-generation of Freyssinet couplers was used.

Fig. 14 View of the SuperStructure – cooling and spacer blocks
Obr. 14 Pohled na nosnou konstrukci – chlazení a distanční bloky

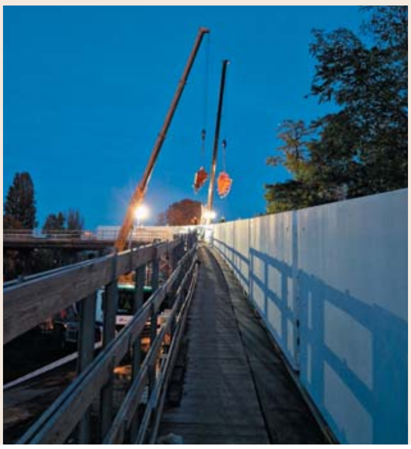

Fig. 15a, b Night concreting of SS – using cranes and buckets
Obr. 15a, b Noční betonáž nosné konstrukce – pomocí jeřábů a bádií

### 5.2 Precasting

The production of the segments described above involved overcoming several challenges. The first challenge was the geometry itself. The directional and elevational alignment of the bridge axis, along with the division into a polygonal segmental structure, resulted in unique shapes for each element. The angulation of the segment faces, perpendicular to the bridge axis, and the trajectory of the prestressing reinforcement, governed by a different required geometry than the bridge axis, led to varying positions for the connectors at the faces, with their angles sometimes differing slightly from the angle of the face. The second challenge emerged from the fresh UHPC material itself, which placed extreme demands on the moulds. The pressures induced by the fresh fine-grained (up to 2 mm) self-levelling mixture are hydrostatic, typically 50–70% higher compared to normal concrete. The required tightness and stiffness of the mould are far beyond what is needed for standard precast concrete. The third challenge was related to the use of the Reckli anti-slip imprint matrix on the walkable surface, as well as the production of the required design curves. To achieve this, the segments were cast in an inverted, upside-down position. Consequently, a method had to be devised to rotate the elements into their assembly position. This was accomplished using two cranes and a specially designed hoisting device (Fig. 8).

### 5.3 Mould

A special all-steel mould (Fig. 9) was designed and fabricated, hydraulically demouldable, with double faces to allow angular rotation of the contact surfaces. The adjustable faces enabled precise, variable fitting of inserts for shear locks and channel connectors for prestressing reinforcement, including angular deflections. During the mould design, the fabrication drawings of the actual segments were simultaneously modified to leverage the principle of anti-symmetry. Similar to production using imprints on a short match-cast line, the same steel faces were used for both the preceding and following segments, with precise angular rotation (0.01° increments, or 1 mm per 5 m in length). After each concrete casting, the produced segment was measured at a specified age, and the designer determined, based on the measured deviations from the ideal design, the so-called compensation-adjustments in the order of millimeters that corrected the alignment of the next face. Therefore, accurate geometry measurements were essential and applied throughout the process.

The total volume of precast elements exceeded 770 m$^3$. The largest precast segment, weighing over 50 tons and with a volume of nearly 20 m$^3$, containing a built-in stainless-steel joint, is the largest UHPC precast element in the Czech Republic. The total volume of more than 900 m$^3$ of UHPFRC used for the superstructure, combined with the white color, makes this bridge unique from a global perspective.

Fig. 16 The footbridge with tent and supporting system
Obr. 16 Lávka se stanem a podpěrnou konstrukcí

## 6. IMPLEMENTATION

### 6.1 Foundations and Substructure

As described above, nearly the entire structure is founded on large-diameter bored piles (880 mm). These are topped with a rectangular foundation made of standard grey concrete (class C25/30). The substructure is of greater interest, as it is constructed entirely of white concrete (class C30/37) to match the brightness of the superstructure in accordance with the architectural competition requirements. This color was achieved by using white cement (CEM I 52.5 R) and added pigments. However, this choice presented a minor challenge due to the rapid strength gain and shortened workability period. Combined with central Prague traffic and high summer temperatures, this necessitated meticulously planned casting operations. Notably, for piers P40 and P50 in the middle of the Vltava River, night-time casting was required, utilizing the largest available concrete pump in the Czech Republic and the installation of a water-cooling system.

### 6.2 Falsework and Bifurcation

A fixed falsework system consisting of MTP 100 towers was selected for the construction of the superstructure. In the Štvanice and Karlín sections, the system was supplemented with I-beams (I450 and I500), while the sections spanning the river were further reinforced with MJD SS and MJD S lattice girders. This setup ensured sufficient load-bearing capacity of the support structure for both the precast segments and the monolithic section (Fig. 13).

The bifurcation of the superstructure (heart is a unique part of the construction where the precast segments are integrated with a monolithically cast section. In this area, the falsework was supplemented with a double-deck system (double floor), which facilitated the precise positioning of the outer parapet beams while forming the underside of the bifurcation at the required gradient. The parapets were positioned using a 500-tonne crane with millimeter precision in both alignment and elevation. During the installation of the beams, it was essential to complete the final connection of all cable ducts, as any later adjustments or repairs would have been impossible. This was followed by the positional fixation of the segments, and the commencement of reinforcement works. During the reinforcement phase, two critical items were integrated into the slab: the cooling system and spacers (Fig. 14).

Following consultations with experts from CTU and considering the scheduled casting time, it was decided to install a cooling system using ¾" hoses. These were distributed throughout the slab in three layers, with a spacing of approximately 500 mm horizontally and 180 mm vertically, across six independent circuits totaling nearly 1,500 meters. The cooling was activated during the casting process and monitored regularly. It was deactivated approximately 60 hours after completion of the pour, once

Fig. 17 View of the underside of the branching area and the connected branches
Obr. 17 Pohled na spodní stranu rozpletu a připojená ramena

Fig. 18 Static load testing of the bridge
Obr. 18 Probíhající statická zatěžovací zkouška

Fig. 19 Štvanice ramp with the "River" statue
Obr. 19 Štvanická rampa se sochou „Řeka"

Fig. 20 Decorative handrail finial – rabbit
Obr. 20 Ozdobná hlavice na zábradlí – zajíc

its purpose was served; any further operation could have potentially harmed the curing concrete.

Due to the large surface area and the self-leveling properties of the concrete, the entire upper surface of the bifurcation had to be fitted with a top formwork lid. As the longitudinal gradient on the ramp exceeds 8%, it would have been impossible to maintain the required shape without this lid. Therefore, approximately 500 UHPC spacers were installed in a 500 × 500 mm grid, each featuring a through-duct for a tie rod (Schwupp rod) in the center. This allowed the lid to be firmly bolted to the bottom formwork, maintaining the required slab thickness of 750 mm (Fig. 15).

Once the reinforcement and formwork were complete, the casting followed. Due to the specific properties of UHPC–in this case, its non-pumpability–we were forced to use a casting method untypical for superstructures: using two cranes and concrete skips (buckets). The pour commenced at night to take advantage of better temperature and humidity conditions, as well as reduced traffic in central Prague (Fig. 12). After the casting was finished, the entire falsework was "wrapped" in heavy rubber tarps to prevent rapid heat loss, and both the concrete temperature and the cooling water temperature were continuously monitored.

## 6.3 Segment Assembly

The segments manufactured in Štětí were transported to the site as oversized loads by road. Following their arrival at night, they were installed the next day using cranes with sufficient lifting capacity (ranging from 200 t to 500 t, depending on the weight of the individual units and the placement radius). Due to the potential deflection and creep of the falsework, the segments are initially placed "loosely." They are only brought together and bonded once the entire span (stage) has been laid out. The falsework beneath the segments is equipped with double steel girders, allowing the individual segments to be slid into position using Teflon (PTFE) pads.

The process always begins with a dry fit (dry assembly) to verify the horizontal and vertical alignment. This is followed by any necessary adjustments to the wooden leveling shims. The segments are then backed off by a maximum of 10 cm to allow for the application of epoxy adhesive – specifically CarboResin in summer or winter formulations – before being pushed back to the contact joint, ensuring excess material is squeezed out. The joint was secured using a steel fixture on the side of the segment to prevent any opening during the curing process. This procedure was repeated until the entire

Fig. 21 Completed footbridge incl. landscaping
Obr. 21 Dokončená lávka vč. parkových úprav

Fig. 22 The footbridge between the trees
Obr. 22 Lávka mezi stromy

Fig. 23 The footbridge in use, reflected in the river
Obr. 23 Lávka v provozu, zrcadlící se v řece

stage was bonded. Since this process is highly sensitive to temperature and humidity, a tent structure was erected around the entire superstructure to maintain the required climatic conditions. (Fig. 16).

Once the adhesive has fully cured, the individual prestressing tendons are threaded and tensioned. The removal of falsework (stripping) occurs only after the tensioning of the subsequent stage, which completes the prestressing of all strands to 100%. This sequence is repeated for all five construction stages

## 7. CONCLUSION

The use of UHPC on such a massive scale was at that time a new phenomenon. Current Czech codes and standards did not fully account for the application of this material in such large-scale structures. However, standards evolved, the new Technical Rules of the Czech Concrete Society (2022) followed by the new Technical Rules of the Czech Ministry of Transport focused on UHPC (2024) now provide much more detailed and precise formal and practical guidelines for future projects.

Current completed UHPC structures serve as pioneers in the field. As such, it is not always possible to avoid errors that would not occur with materials and methods proven over decades. Nevertheless, the experience gained is invaluable for future applications. Given its unique characteristics, and despite certain challenges, UHPC will undoubtedly find its way not only into precast plants but also into in-situ construction. As a material of the future, it is destined to play a vital role in the industry.

## MATERIAL USAGE (SUPERSTRUCTURE)

| | TOTAL | PER 1M² |
|---|---|---|
| UHPC – PREFABRICATED SEGMENTS | 770 m³ | 0,475 m³ |
| UHPC – CAST | 125 m³ | 0.695 m³ |
| PRESTRESSING STEEL | 70 t | 38.8 kg |
| REINFORCING STEEL | 215 t | 119.4 kg |

## SPOTŘEBA MATERIÁLU (NOSNÁ KONSTRUKCE)

| | CELKEM | NA 1M² |
|---|---|---|
| UHPC – PREFA SEGMENTY | 770 m³ | 0,475 m³ |
| UHPC – BETONOVANÝ ROZPLET | 125 m³ | 0,695 m³ |
| PŘEDPÍNACÍ VÝZTUŽ | 70 t | 38,8 kg |
| BETONÁŘSKÁ VÝZTUŽ | 215 t | 119,4 kg |

Fig. 24 View of Holešovice and the footbridge from Štvanice
Obr. 24 Pohled na Holešovice a lávku směrem ze Štvanice

# FOOTBRIDGE OVER CHEB RAILWAY STATION
# LÁVKA PŘES KOLEJIŠTĚ NÁDRAŽÍ V CHEBU

21


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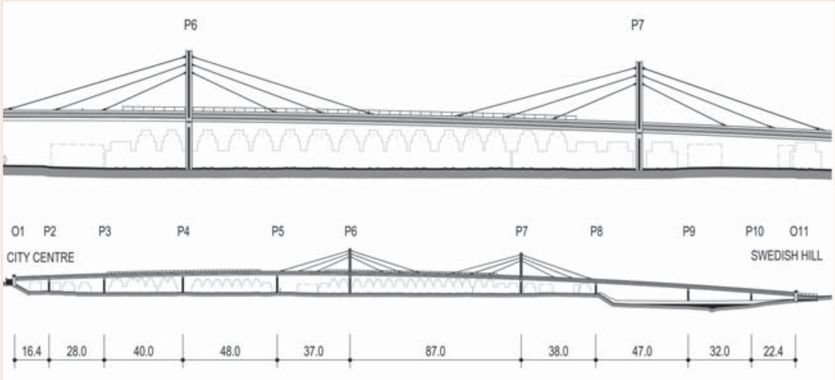
**The footbridge is designed as a prestressed concrete structure supplemented by a pair of steel pylons with semi-radial cables supporting the longest span of 87 m. The total length of the structure is 397.1 m with its 10 spans and it crosses 39 tracks in the area of Cheb railway station. The superstructure was produced in the casting yard and the individual segments were incrementally launched over the entire track together with the already installed pylons, which are an integral part of the superstructure.**

**Lávka je navržena jako předpjatá mostovka podepřená dvojící ocelových pylonů se semiradiálním uspořádáním kabelů v nejdelším poli o rozpětí 87 m. Celková délka lávky je 397,1 m je rozmístěna do 10 polí a překračuje celkem 39 železničních kolejí v místě kolejiště nádraží v Chebu. Nosná konstrukce byla zhotovena ve výrobně a jednotlivé lamely byly postupně vysouvány přes všechny koleje. Během výsunu byl na nosné konstrukci již osazen pylon.**



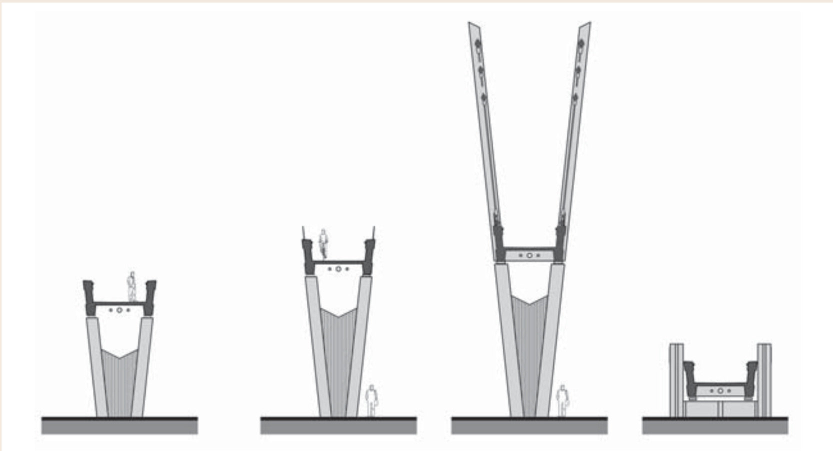



Fig. 1 Longitudinal section of the footbridge with detail of the suspended part of the structure
Obr. 1 Podélný řez lávkou s detailem zavěšené části konstrukce

Fig. 2 Cross section of the structure
Obr. 2 Příčný řez konstrukcí

Fig. 3 Incremental launching of the superstructure
Obr. 3 Výsun nosné konstrukce

## OVERALL CONCEPT

The footbridge crosses the railway station tracks in a straight line and connects the city centre with the emerging area in the west.

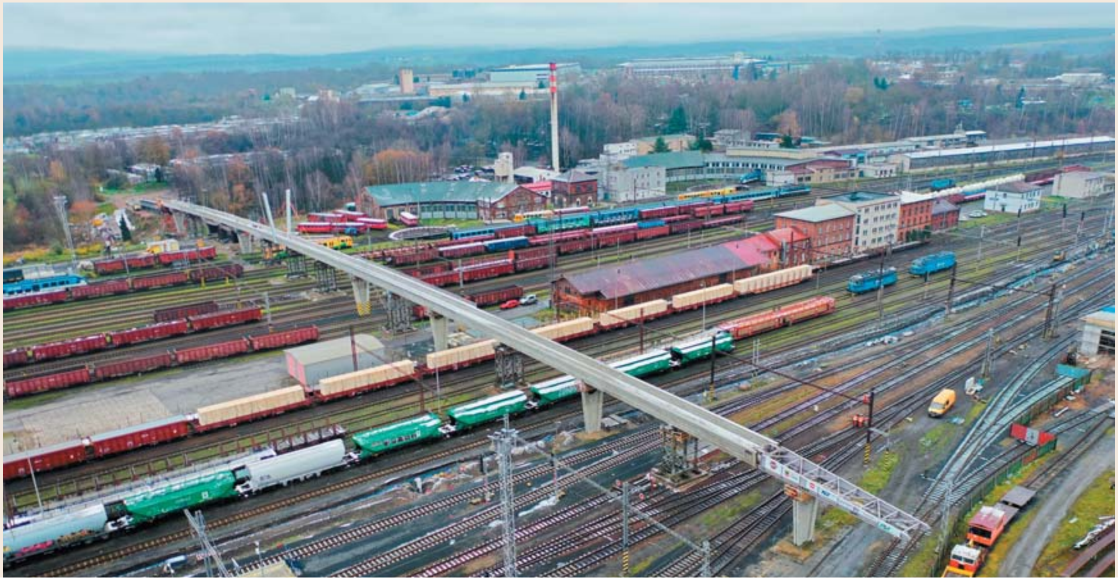
The total length of the footbridge structure is 397.1 m and consists of 10 spans, see Fig. 1. On the west side, the access to the footbridge is restricted by the adjacent local road, so it was not possible to design a seamless connection, and therefore a perpendicular staircase and a short perpendicular ramp were used to allow access to the footbridge. On the Svedsky vrch side, the footbridge transitions seamlessly into a curved ramp that follows the existing road structure. The clear width between handrails is 3.0 m.

## SUBSTRUCTURE

The piers are formed in cross-section by V-shaped columns which are connected by a wall up to a level of 1.8 m below the upper surface of the pier. Typical piers are 800 mm wide longitudinally and 1200 mm wide at the pylon locations (P6 and P7). The interconnecting wall is 400 mm and 600 mm thick in pylons, see Fig. 2. Grooves 150 mm deep are designed on the side of the pylons for visual relief. All the piers keep the same angle between columns.

## SUPERSTRUCTURE

The level of the footbridge is positioned in a constant atypical arc with a radius of 3030 m. Due to the constant



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curvature and the straight plan alignment, it was possible to use incremental launching technology advantageously due to the complicated arrangement of the obstacles.

The cross-section of the footbridge is formed by a symmetrical parapet cross-section of a H shape with a total height of 2.0 m. The H shape is an advantageous section for the incremental launching construction method. The parapets are angled outwards and their upper surface is at a height of 1250 mm above their starting level.. A recess of 60 mm depth is designed along the height of the parapet on both the outside and inside. The width of the parapet at the top is 500 mm, in the middle part it is 380 mm and the thickness of the intermediate bridge slab is 180 mm. The parapet is extended to 440 mm in the middle part of the parapet wall to accommodate the prestressing anchors near the abutments.

The pylons for suspension of the structure are located above the P6 and P7 piers and are an integral part of the superstructure.

The pylon cross-section is designed as a welded chamber with internal stiffeners. The pylon reaches a height of 13.75 m including the height of the parapet. Fully locked, coil-type cables are used. The cables are ended by a pin fork, and they are assembled with a turnbuckle for adjustment of inner force.

The prestressing of the structure is designed in two phases. For the first phase, the cables are straight cables located at the corners of the cross-section. In total there are four 12-strand cables in plastic ducts. These cables provide an almost central prestressing which is very advantageous for the displacement of the structure with regard to the need to resist for both positive and negative moments.

The second phase of prestressing is the continuity cables, which are traced in height adequately to the static actions. The prestressing contractor, VSL has installed the strands into the continuity ducts for the entire length of the 397 m structure. This fact enabled the primary direct prestressing to be carried out only inside the structure, without the

Fig. 4 View of completed pylons
Obr. 4 Pohled na dokončené pylony

need to use antagonistic assembly cables outside the footbridge.

## CONSTRUCTION

For space reasons, the location of the production plant on the side of Svedsky vrch was chosen.

The footbridge was divided into a total of 14 segments with a length of 22 – 30 m. A steel nose was used for the launching, it was pinned at the top face through a steel crossbeam to the supporting structure.

VSL provided the pull technology by means of a pulling system via a steel crossbeam placed on the concrete face of the sub-segment. After the segment was concreted in the factory, the Phase 1 cables were tensioned and pulled out to the projected position. Subsequently, a steel crossbeam was fitted to the face of the currently extended segment, over which the structure was secured, thus establishing a fixed point in the production plan.

During dynamic tests, the natural shapes of the footbridge and the response to a synchronous loading vertical impulse of multiple persons, free pedestrian movement and transverse impulse were monitored. The first bending natural frequency was determined by testing to be 1.099 Hz and the first torsional frequency was determined to be 2.45 Hz. The maximum acceleration of the superstructure is 0.09 m/s due to the induced force according to Eurocodes.

## SUMMARY

The footbridge was completed and opened to the public in September 2023, reconnecting two parts of the city of Cheb for pedestrians and cyclists. All stages of the project documentation were prepared by the company Strasky, Husty and partners.

Fig. 5 View of the completed structure
Obr. 5 Celkový pohled na lávku

# FOOTBRIDGE OVER THE ELBE RIVER IN HRADEC KRÁLOVÉ
# LÁVKA PŘES LABE V HRADCI KRÁLOVÉ

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**This footbridge over the Elbe River in Hradec Králové is an extraordinary structure made of modern materials and using innovative technologies. The proposed structural solution was a real challenge for architects, designers, and contractors, requiring close cooperation between them. The unique structural system is a combination of a subtle bridge deck made of precast UHPFRC panels, slender steel ribs of variable height, and main suspension cables. The design required the resolution of many complex details that often exceeded the scope of applicable standards and regulations. The authors of the architectural design are GEM.VISION, the designers are Valbek Ltd., implementation was carried out by a team of Stavby mostů, part of the VINCI Construction CS Group, the UHPC elements were manufactured by KŠ PREFA Ltd.**

**Lávku přes řeku Labe v Hradci Králové tvoří neobyčejná konstrukce zhotovená z moderních materiálů a s využitím inovativních technologií. Navržené konstrukční řešení bylo pro architekty, projektanty i dodavatele skutečnou výzvou a vyžadovalo jejich úzkou spolupráci. Unikátní konstrukční systém je kombinací subtilní mostovky z prefabrikovaných UHPFRC panelů, štíhlých ocelových žeber proměnné výšky a hlavních nosných kabelů. Návrh si vyžádal řešení mnoha složitých detailů, které mnohdy přesáhly rámec platných norem a předpisů. Autory architektonického návrhu je společnost GEM.VISION, projektanty společnost Valbek s.r.o., realizaci provedl tým společnosti Stavby mostů, součást skupiny VINCI Construction CS Group, prvky UHPC vyrobila společnost KŠ PREFA s.r.o.**

Fig. 1 Pedestrian Bridge over the Elbe River in Hradec Králové
Obr. 1 Lávka pro pěší přes Labe v Hradci Králové

## 1 INTRODUCTION

The pedestrian bridge in Hradec Králové connects the city centre with the developing area around the Aldis congress centre. The development of this area accelerated with the construction of the new ČSOB offices, and the bridge aims to support further growth.

Cycle paths run along both banks of the Elbe River and provide access to the area. The main purpose of the bridge is to redirect cyclists and pedestrians from the nearby ring-road bridge and facilitate movement between the two parts of the city. The bridge will also provide convenient access to the ČSOB offices.

The final design of the bridge structure is based strictly on the winning design from the 2014 architectural and structural competition. It is a very complex project involving not only the footbridge itself, but also a whole range of related structures. The overall solution is defined by several boundary conditions, particularly regarding the location of the bridge within the city, the parameters of the transported traffic, and the limitations arising from the nature and location of the obstacles to be crossed.

## 2 STRUCTURE

The bridge structure consists of two asymmetrical spans, with the main span across the river reaching almost 69 m. The main load-bearing elements are two fully-locked coil cables with a diameter of 130 mm installed in an optimized shape, ensuring structural stability. The bridge deck is made of lightened precast UHPFRC elements through which four unbonded prestressing cables pass. The deck and bearing cables are connected by slender steel crossbars. In the abutment area, UHPFRC elements are replaced with steel crossmembers. The substructure design reflects the modern appearance of the bridge deck.

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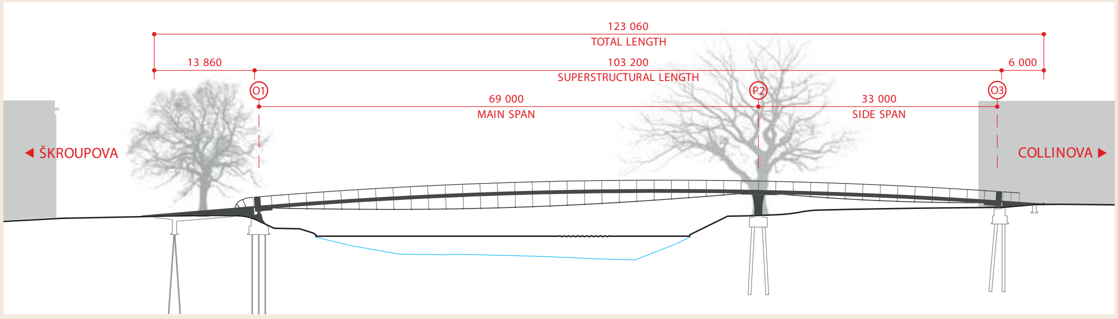


Fig. 2 Longitudinal section
Obr. 2 Podélný řez

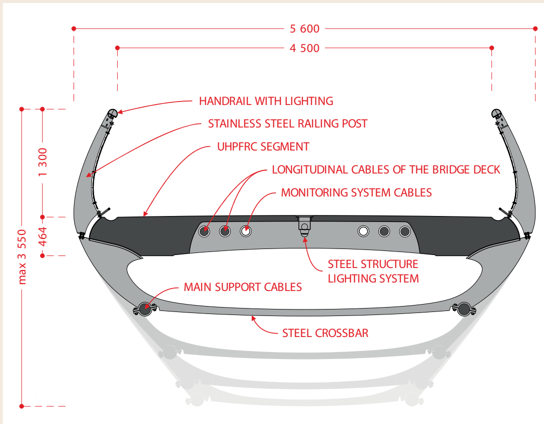


Fig. 3 Cross section
Obr. 3 Příčný řez

## 2.1 Design

The detailed design of the bridge and related structures is based on the tender design respecting the original concept proposed by the winning architectural competition team. The design is complex and relies on a global definition of geometry for all structural members. The final geometry results from an optimized distribution of internal forces and stiffness.

Several computational models were used for structural analysis to capture the complex behaviour of the bridge during construction and throughout its service life. For detailed components, special 3D finite-element models were applied. The results were verified by analytical hand calculations.

## 2.2 Foundations and substructure

The foundations consist of micropiles (implemented in situ) on the right river bank (abutment O1) and bored reinforced concrete piles on the opposite side (pillar P2 and abutment O3). Precast bored piles are uncommon in the Czech Republic, but their use brings advantages typical for prefabrication, particularly better quality control and time savings during on-site construction.

The substructure consists of two abutments and one pillar, all made of reinforced concrete. A common feature is their organic, curved geometry. The right-bank abutment (O1) includes the main abutment and a small "bridge" over utility lines supported by the abutment on one side and two columns on the other. This solution allows repairs of utility lines without demolishing the structure.

The pillar resembles a sculptural element rather than a conventional bridge pier, with surfaces curved in two directions. It is constructed from heavily reinforced C50/60 concrete and forms the fixed point of the bridge. The connection with the steel structure is ensured by massive shear steel threaded bars.

## 2.3 Prestressing system

The prestressing system consists of two groups of cables. The upper prestressing cables provide compression reserve in the structure, particularly in the joints between the UHPFRC elements, while the bottom cables act as the main load-bearing system of the bridge.

The main bearing elements are four fully-locked coil cables with a diameter of 130 mm anchored to the deck using a fixed and an adjustable cylindrical socket with a spherical nut and washer. To ensure long-term durability, individual wires are protected by a 95% Zn – 5% Al alloy coating applied by a hot-dip process under controlled factory conditions. Together with internal filling and zinc coating of inner wires, this protection ensures high durability even in

Fig. 4 Prestressing system
Obr. 4 Předpínací systém

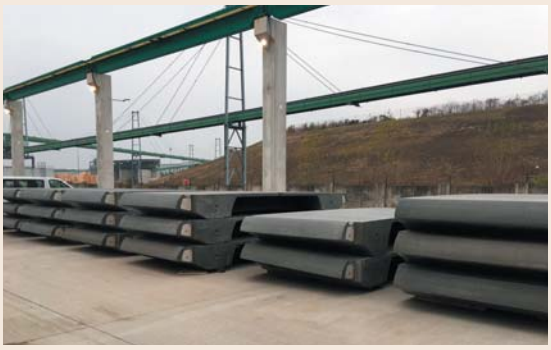

Fig. 5a, b, c UHPFRC segments
Fig. 5a, b, c Segmenty z UHPFRC

aggressive environments. Material conformity and testing followed international standards, verifying the modulus of elasticity and minimum breaking force.

To eliminate initial inelastic strains, the cables were prestretched to 50% of the minimum breaking force for at least five cycles. Cable lengths and socket positions were then precisely marked under specified loads and controlled temperature conditions.

The cables are installed in an optimized spatial shape defined by geometric requirements, including the necessary clearance for ship navigation, and by structural behaviour. They generate a virtual vertical force through transverse steel beams corresponding to the dead-load reaction at virtual supports and act as ties in a virtual Vierendeel beam under live load. The cables are connected to steel crossbars by clamps and are curved in both longitudinal and transverse directions. Separate cables are used for the main and side spans to avoid corrosion-sensitive saddle details and reduce friction losses.

The cables were assembled in their final position before installation of the steel crossbars and precast UHPFRC slabs. During prestressing, clamps allowed sliding to reduce friction losses and were fixed afterwards, integrating the cables into the final structural system. Prestressing gradually lifted the structure from the temporary support system until the final geometry and required cable stresses were achieved. Tensioning was performed using four systems operating simultaneously, each capable of pulling up to 535 tons, while the bridge geometry was checked by topographic survey.

Full compression of the bridge deck in the serviceability limit state is ensured by four unbonded prestressing cables passing through the UHPFRC slabs. These cables increase compression mainly in joints between segments, while part of the compression effect is also provided by the low-rise arch geometry of the superstructure. Each cable consists of 13 strands anchored in the steel abutment segment.

The prefabricated system enabled rapid installation and tensioning, reducing construction time, costs and on-site waste while ensuring high quality control throughout the system life cycle.

## 2.4 Steel structure

In areas above the abutments and pillar, UHPFRC segments are replaced by steel crossmembers filled with self-compacting concrete. These members provide rigid anchorage for the prestressing system.

Another part of the steel structure consists of slender crossbars connecting the concrete deck slabs with the bearing cables. During construction they are supported by braces that are removed after prestressing.

The crossbars continue geometrically into the railing columns. The handrail follows the structural grid and consists of elements of the same length as the deck segments. An LED strip integrated into the handrail illuminates the bridge deck.

## 2.5 UHPFRC segments

The bridge deck consists of a total of 39 precast segments made of UHPFRC (37 standard segments and 2 haunched segments located in front of and behind the pier). The segments consist of a pair of longitudinal edge beams and a thin slab connecting the beams. A pair of transverse ribs is designed to ensure sufficient rigidity of the segments and to place the external prestressing cables. The joint between the segments was filled with high-strength cement grout before the prestressing of the bridge deck and before the tensioning of the main load-bearing cables.

The architectural character of the bridge places high demands on surface quality. The walking surface contains a periodic anti-slip pattern created by a matrix in the mould. The remaining surfaces also meet requirements for fair-faced concrete with smooth texture, minimal pores and uniform colour.

The properties of fresh UHPFRC allow for perfect concreting of all details of variously shaped elements, and thanks to the mechanical parameters of UHPFRC, savings in construction materials are achieved. The physical properties of UHPFRC

Fig. 6 Railing
Fig. 6 Zábradlí

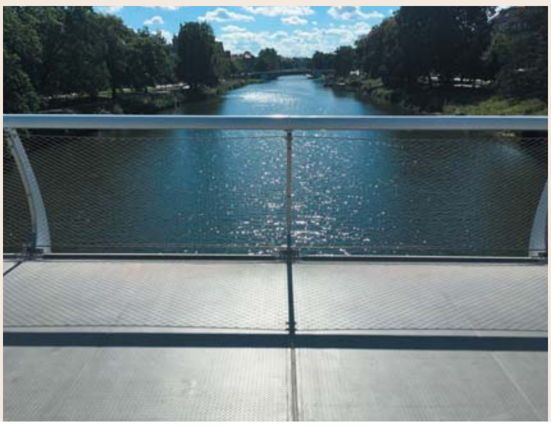

Fig. 7a, b Pedestrian Bridge over the Elbe River in Hradec Králové
Fig. 7a, b Lávka pro pěší přes Labe v Hradci Králové

guarantee minimal surface permeability and ensure a long service life for the structure. In addition to its aesthetic qualities, the walkable concrete surface of the bridge deck also eliminates the costs of constructing and maintaining conventional walkable surfaces that use various types of isolation, for example.

### 2.6 Long-term monitoring

The behavior of the structure depends on the interaction of all components and on the precise geometry and prestressing forces. It was therefore necessary to monitor the geometry and stresses in key elements during construction and throughout the bridge's service life.

The bridge is equipped with an automatic monitoring system that continuously records data and transmits it to the control center. The measured values are compared with theoretical predictions. Computational analysis defined the permissible limits for the monitored parameters. The system automatically evaluates the structure's response to external influences, such as loads and temperature changes, and sends reports to the control center.

## 3 CONSTRUCTION METHODS

Construction was complicated by limited space on the site, which prevented storage of materials or components. Deliveries therefore had to follow a strict just-in-time schedule. The site also contained 27 underground utility lines, which prevented the use of heavy machinery. The contractor addressed this issue by installing a gantry crane operating along the entire length of the bridge. Its runway was supported by the same temporary structure that carried the bridge components during assembly. The electric crane also provided environmental benefits compared with diesel-powered equipment.

The distinctive shape of the pillar required complex formwork. The contractor therefore used prefabricated formwork elements manufactured in a plant and assembled on site, which also helped achieve the required quality of fair-faced concrete.

During the assembly of the superstructure, the bridge was supported by a combination of heavy and light scaffolding. The heavy scaffolding foundations consisted of rammed HE300B steel profiles installed using a 30-ton excavator with a piling rig operating from a pontoon. In the longitudinal direction, cold-rolled steel beams supported both the bridge structure and the crane runway. The scaffolding deck supported the main cables and steel crossbars during installation.

The assembly process proceeded in several steps. First, the cables were installed, followed by the installation of the steel ribs. Subsequently, the UHPFRC segments were placed one by one using a gantry crane with a balance beam. The segments were delivered and stored on the left river bank. During installation, the geometry of the bridge was continuously adjusted. After all segments were installed, the joints were filled, followed by tensioning of the prestressing and bearing cables.

## 4 CONCLUSIONS

The pedestrian bridge and related structures were put into operation in March 2023. The bridge has thus become a significant landmark in this part of Hradec Králové and one of the largest applications of the innovative UHPFRC material in the country. We succeeded in creating a truly exceptional structure in every respect, one that exceeds the usual standards of construction practice in terms of its requirements for precision and quality. The entire implementation process was carried out in close cooperation between all partners involved (architects, designers, manufacturers, contractor). This was the only way to meet the theoretical requirements of the design and successfully translate them into the final form of the work.

We greatly appreciate the investor's support and courage in implementing this project. Together, we have achieved a milestone in the design and construction of bridge structures using current theoretical knowledge, technologies, and know-how.

# PEDESTRIAN AND CYCLE BRIDGE ACROSS THE BECVA RIVER

# MOST PRO PĚŠÍ A CYKLISTY PŘES ŘEKU BEČVU

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**This bridge across the Bečva River with a total length of 145.50 m is suspended on outwardly inclined suspension cables of three spans of lengths 15.75 + 105.00 + 15.75 m. The bridge deck, assembled of precast segments, is prestressed by unbonded cables; the suspension cables are made of locked coil strands, the suspenders are formed by steel rods. The function of the bridge was verified by detailed static and dynamic loading tests.**

**Most přes řeku Bečvu celkové délky 145,50 m je zavěšen na vně skloněných visutých kabelech o třech polích délek 15,75 + 105,00 + 15,75 m. Mostovka sestavená z prefabrikovaných segmentů je předepnuta nesoudržnými kabely, visuté kabely jsou z uzavřených lan, závěsy jsou tvořeny ocelovými tyčemi. Funkce mostu byla ověřena podrobnými statickými a dynamickými zatěžovacími zkouškami.**

Fig. 1 Bridge across the Bečva River
Obr. 1 Most přes řeku Bečvu

The Bečva River bridge, which is located between the villages of Ústí and Černotín, is part of the 'Bečva' cycle path connecting cities Velké Karlovice and Tovačov. The bridge is in a crest elevation with a radius of 1,705 m and tangents at the longitudinal slope of 5.26% – see Fig. 1. The width between the railings is 3.50 m.

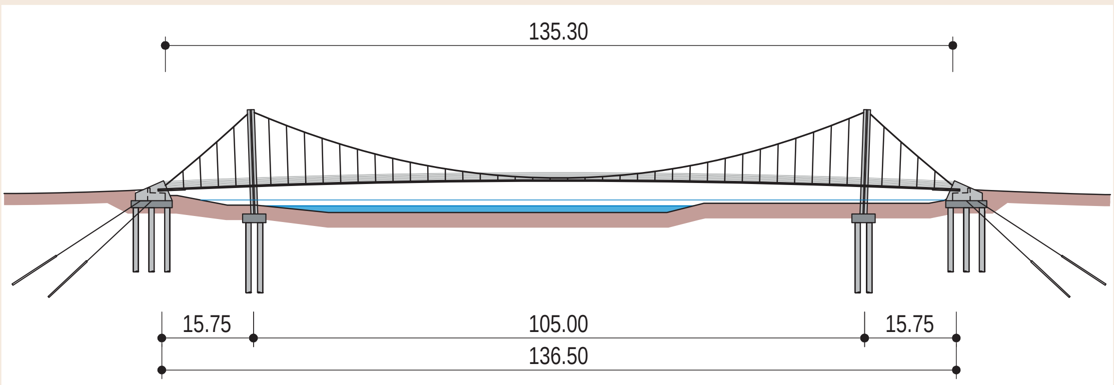
## STRUCTURAL AND ARCHITECTURAL DESIGN

The bridge forms a suspension structure with the main span of 105 m. The 137.70 m long deck is suspended at its outer edges on outwardly inclined suspension cables of three spans of lengths 15.75 + 105.00 + 15.75 m – see Fig. 2. The maximum vertical sag of the suspension cables in the middle of the main span is 12.00 m.

The bridge deck made of C70/85 concrete is composed of 45 3.00 m long precast segments and two 1.35 m long end cast-in-place diaphragms – see Figs. 3 and 4. The joints between the segments are made of self-compacting high-strength concrete. The 43 inner segments have a double-tee cross-section formed by edge 0.40 m deep beams and a deck slab, which are reinforced by end diaphragms. The two end segments are solid. The bridge deck is longitudinally prestressed by 2×2 cables formed by 22 monostrands led at the edge beams.

The suspension cables are made of locked coil strands. The cables are connected by forks to anchor plates anchored in the pylons and end abutments – see Fig. 5. The suspension cables are fix connected with the mid-span segment. During erection, the segments were mutually connected by steel joints, to which the suspenders are attached. To reduce the bending stresses of the deck, the deck's suspension at the pylon was

Fig. 2 Elevation
Obr. 2 Podélný řez



omitted. The deck is connected to the abutments by hydraulic dampers attached to anchor plates of the prestressing cables.

The 18.50 m high V-shaped pylons are made of C40/50 concrete. The pylon uprights with an inclination angle of 200 from the vertical have a constant pentagonal cross-section. In the lower part, the uprights are connected to each other up to a height of 3.00 m by a reinforcing wall 0.50 m thick. The pylons are fixed into foundation slabs supported by piles with a diameter of 900 mm.

The tensile force from the suspension cables is transferred to the end abutments, which are supported by drilled piles anchored by ground anchors. The bridge deck is supported on pairs of neoprene bearings at the end abutments. Vertical bearings situated between the deck edges and the pylons uprights ensure transfer of horizontal forces from the bridge deck to the pylon.

The structure was analyzed as a spatial structure assembled from either beam or shell elements. A modal analysis of the structure was performed to evaluate pedestrian comfort and wind effects. Aerodynamic stability was verified by checking the ratio of the corresponding torsional and bending frequencies, which was greater than the recommended value of 2.5. Pedestrian comfort was assessed according to fib Guidelines for the footbridge design. A dynamic load test confirmed that the structure is sufficiently stiff and that the movement of the structure induced by dynamic loads acting in both the vertical and horizontal directions does not cause discomfort to users.

## BRIDGE CONSTRUCTION

After construction of the abutments and pylons, the suspension cables were installed and tensioned: first the cables of the back spans, then the cables of the main span. Considering the shallow depth of the river, it was possible to erect the segments by cranes situated at the riverbed. Deck assembly began with the suspension of the middle segment. After its connection with the suspension cables, the other segments were gradually suspended in directions from the center of the bridge to both abutments with the maximum unbalanced weight of one segment. During assembly, the bridge deck gradually changed shape, first following the convex shape of the suspension cables, later the deck gradually changed into the designed concave shape. After the joints between the segments were cast, the structure was prestressed and the hydraulic dampers were installed.

## CONCLUSIONS

The bridge has a very slender deck; it is light and transparent. It has a minimal impact on the environment. Thanks to the appropriate arrangement of structural elements economically stressed by normal forces, it is sufficiently rigid and comfortable for users. The bridge was well accepted by inhabitants of connected villages and users of the cycle path.

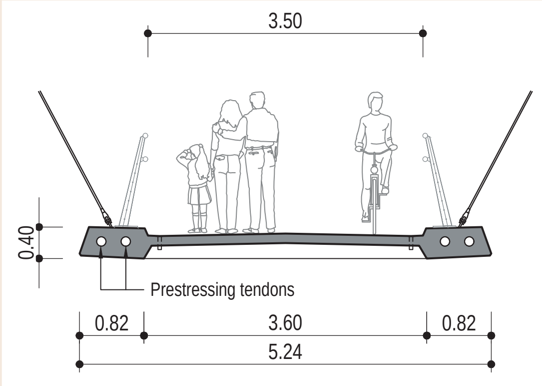



Fig. 3 Deck cross section
Obr. 3 Příčný řez mostovkou

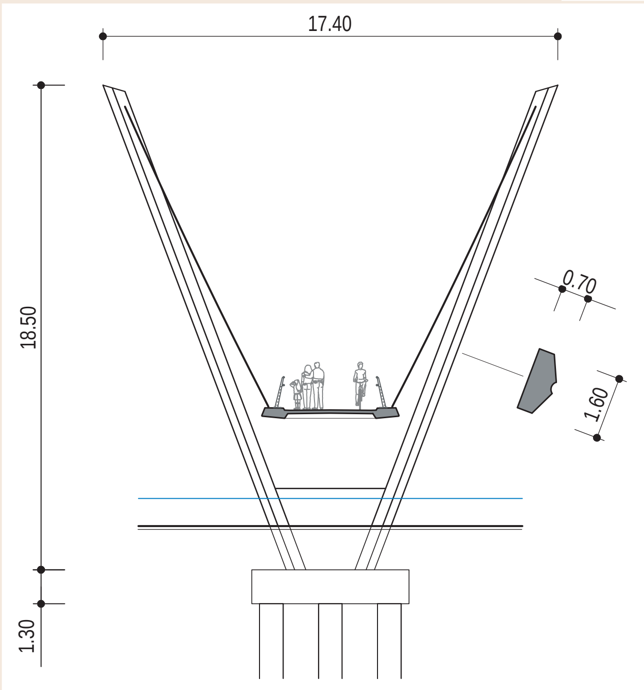



Fig. 4 Bridge cross section
Obr. 4 Příčný řez mostem

The construction of the footbridge began in the fall of 2020, and it was officially opened on June 29, 2023. The client of the bridge is the Microregion Hranicko together with the State Fund for Transport Infrastructure. The design of the bridge is the work of the firm Stráský, Hustý a partneři, s.r.o., Brno. The general contractor was a Joint venture of firms Eurovia CS, a.s. Praha and KKS, s.r.o., Zlín. Erection of the structure was performed by firm FIRESTA-Fišer, a.s., Brno.

Fig. 5 Bridge structure
Obr. 5 Konstrukce mostu

# FOOTBRIDGE ACROSS THE OPAVA RIVER
# LÁVKA PŘES ŘEKU OPAVU

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**This pedestrian and cycle bridge over the Opava River connects the Silver Lake recreational area with the municipal park in Opava. The construction of the footbridge was part of the revitalisation of the Silver Lake area. The footbridge's design is based on a concept developed by Link Projekt, who prepared the entire project from start to finish – from the initial study in 2010 to the final construction completed in 2023. Designed as a single-span integrated suspension structure, the footbridge has a total span length of 61.7 m and a slender, prestressed, cast-in-situ reinforced concrete deck with a variable width of 4.3–5.3 m and a thickness of 0.19 m. It is suspended from tension cables anchored to 6.0 m high inclined steel pylons on both sides.**

**Lávka pro pěší a cyklisty přes řeku Opavu propojuje rekreační oblast u Stříbrného jezera s Městskými sady v Opavě. Její výstavba byla součástí komplexní revitalizace areálu Stříbrného jezera, která zahrnovala také terénní a krajinářské úpravy včetně úprav břehů jezera. Podoba lávky vychází z návrhu společnosti Link projekt, která zpracovala celý projekt jako autorské dílo – od úvodní studie z roku 2010, přes jednotlivé stupně projektové dokumentace, až po vypracování realizační dokumentace v roce 2023. Lávka je navržena jako jednopolová integrovaná visutá konstrukce s délkou přemostění 61,7 m. Mostovka je tvořena štíhlou železobetonovou předpjatou monolitickou deskou tloušťky 0,19 m proměnné šířky 4,3–5,3 m s oboustranným zavěšením na visutých kabelech kotvených do skloněných ocelových pylonů výšky 6,0 m.**

Fig. 1 View of the bridge
Obr. 1 Celkový pohled

## BASIC PROJECT DATA

| | |
|---|---|
| TYPE OF STRUCTURE: | Fully integrated suspension structure |
| BRIDGE LENGTH: | 78.0 m |
| SPAN LENGTH: | 67.8 m, single span |
| WIDTH: | 4.3–5.3 m |
| CLIENT / INVESTOR: | Ministry of Finance with participation of the City of Opava |
| DESIGNER: | Link projekt s.r.o. |
| BRIDGE CONTRACTORS: | Metrostav a.s., Swietelsky stavební s.r.o. |
| CONSTRUCTION PERIOD: | 03/2021–03/2023 |

## ARCHITECTURAL AND STRUCTURAL DESIGN

The 78.0-metre-long footbridge is designed as a single-span suspension structure, emphasising clarity, simplicity and structural efficiency. The deck consists of a slender, 0.19 m thick, cast-in-situ reinforced concrete slab made of C40/50 concrete with edge stiffening ribs. Near the abutments, the slab's height gradually increases to reach the full height of the ribs. The deck is integrally connected to the abutments at both ends of the footbridge. Prior to connection with the abutments, the deck was post-tensioned using cables placed in flat plastic ducts.

The deck is suspended from a pair of REDAELLI FLC 80 'full locked cable' suspension cables. This solution ensures straightforward installation, as well as efficient long-term maintenance and inspection. With a span of 67.8 m and a sag of 4.2 m, the suspension cables form a catenary in the longitudinal direction. In the transverse direction, each cable plane is inclined outwards. The cables are fixed to the deck at its centre. At both ends, the cables are anchored into the pylon heads via CYN 76 anchorage clamps. System clamps with gusset plates (REDAELLI) are installed on the suspension cables to connect the hangers. The hangers are connected to the deck via steel gusset plates anchored in the deck concrete. The standard hangers are spaced at 4.50 m and are MACALLOY M24 bars made of S520 steel with turnbuckles for possible adjustment. The short hangers near the centre of the footbridge are atypical steel elements. The central cable clamp provides a fixed connection to the deck to limit deck deflections.

The suspension cables are anchored to slender steel pylons, each 6.0 metres high. The pylons are inclined longitudinally towards the abutments. In the transverse direction, each pylon forms a 'V' shape. They have a closed trapezoidal cross-section measuring 0.16–0.28 × 0.90 m and are fitted with passive back ties made of flat steel plates. The pylons

## ZÁKLADNÍ DATA PROJEKTU

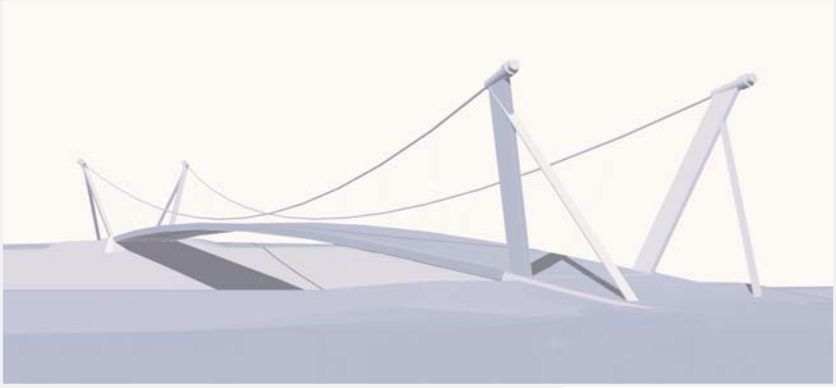
| | |
|---|---|
| TYP KONSTRUKCE: | Visutá integrovaná předpjatá monolitická konstrukce |
| DÉLKA LÁVKY: | 78,0 m |
| DÉLKA PŘEMOSTĚNÍ: | 67,8 m, 1 pole |
| ŠÍŘKA: | 4,3–5,3 m |
| INVESTOR: | Ministerstvo financí s účastí města Opavy. |
| PROJEKTANT: | Link projekt s.r.o. |
| ZHOTOVITEL MOSTU: | Metrostav a.s., Swietelsky stavební s.r.o. |
| DOBA VÝSTAVBY: | 03/2021–03/2023 |

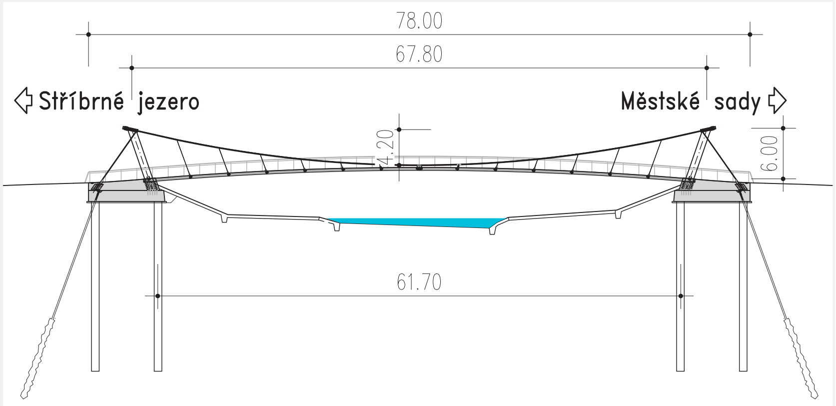
## ARCHITEKTONICKÝ A KONSTRUKČNÍ NÁVRH

Lávka celkové délky 78,0 m je navržena jako jednopolová visutá konstrukce s délkou přemostění 61,7 m. Při návrhu byl kladen důraz na čistou, jednoduchou a staticky efektivní konstrukci. Mostovka je tvořena štíhlou monolitickou železobetonovou deskou tl. 0,19 m z betonu C40/50 se zesílenými krajními žebry. Šířka mostovky je proměnná, u opěr je střední část desky postupně zesílena na plnou výšku žeber. Před spojením s opěrami pomocí petlicového styku byla mostovka dodatečně předepnuta pomocí kabelů vedených v plochých plastových trubkách

Mostovka je zavěšena na dvojici systémových visutých kabelů složených s ocelových lan s uzavřenou konstrukcí – „full locked cable" REDAELLI FLC 80, díky které je zajištěna jednoduchost samotné instalace i následná budoucí údržba a inspekce lan. Visuté kabely o rozpětí 67,8 m a vzepětí 4,2 m tvoří v podélném směru řetězovku, v příčném směru je rovina visutého kabelu vně odkloněna. Kabely jsou vedeny mimo obrys mostovky, ve střední části lávky jsou kabely pevně spojeny s mostovkou. Na koncích jsou nosné kabely zakotveny do hlav pylonů přes kotevní objímky typu CYN 76. Pro připojení závěsů jsou na visutém kabelu osazeny systémové svěrné objímky REDAELLI se styčníkovými plechy. K mostovce jsou závěsy připojeny styčníkovými plechy zakotvenými do betonové desky. Typické závěsy umístěné v rozestupech 4,50 m jsou tyčové MACALLOY M24 z oceli S520 se systémovými koncovkami doplněné o napínáky pro případnou rektifikaci závěsů. Krátké závěsy u středu lávky jsou řešené jako atypické zámečnické prvky. Středový závěs zajišťuje pevné spojení visutého kabelu s mostovkou a je tvořený styčníkovým plechem, který je přišroubován na svěrnou objímku visutého kabelu. Sousední krátký závěs je tvořen oboustrannými „očnicemi" z plechu, které jsou připojeny ke styčníkovým plechům.

Visuté kabely jsou na koncích lávky kotveny v ocelových štíhlých pylonech výšky 6,0 m, které jsou v podélném směru odkloněny k opěrám lávky, v příčném směru má pylon tvar písmene „V". Pylony uzavřeného lichoběžníkového průřezu rozměru 0,16–0,28 × 0,90 m se zadními pasivními táhly z ploché oceli jsou vetknuty do monolitické železobetonové opěry. Na horní části pylonu je navařena kotevní trubka visutého kabelu, která je na konci opatřena roznášecí ocelovou deskou s kruhovým otvorem pro průchod kotevní objímky visutého kabelu. Ocel pylonu je S355, ocel zadního táhla S420. Krajní opěry jsou tvořené masivními železobetonovými bloky z betonu C35/45. Založení lávky je hlubinné na vrtaných pilotách profilu 900 mm vetknutých do základu. Založení je doplněno na obou opěrách o mírně odkloněné trvalé zemní kotvy, které zabezpečují tlakové namáhání zadní řady pilot pro stálá zatížení.





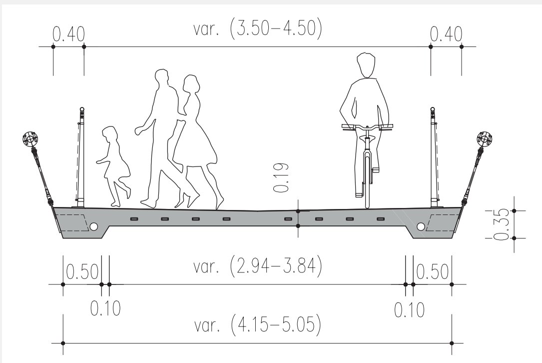




Fig. 2 Structural arrangement
Obr. 2 Konstrukční uspořádání
Fig. 3 Longitudinal section
Obr. 3 Podélný řez
Fig. 4 Cross section
Obr. 4 Příčný řez

Fig. 5 Steel pylons
Obr. 5 Ocelové pylony

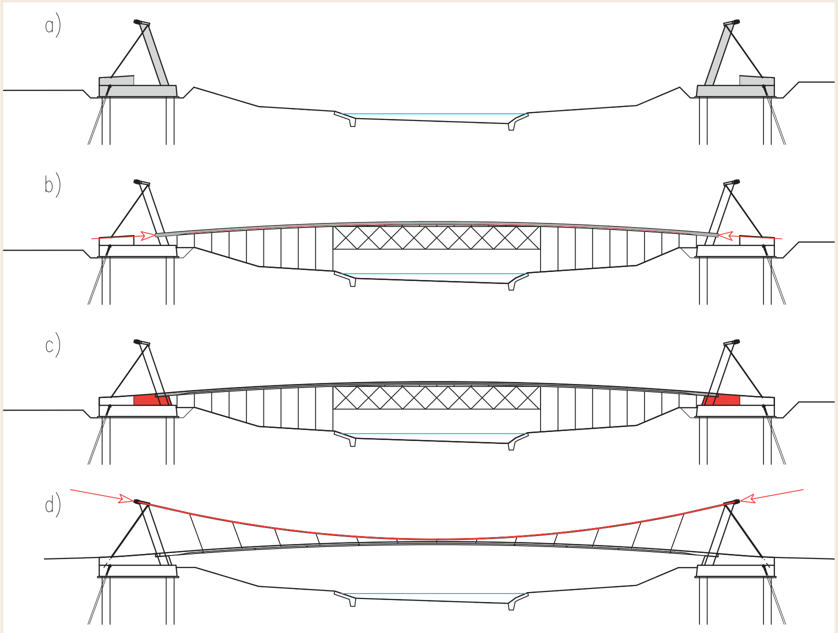



Fig. 6 Construction sequence
Obr. 6 Postup výstavby

are embedded in the reinforced concrete abutment. A steel tube for anchoring the suspension cables is welded to the top of each pylon and fitted with a distribution plate containing a circular opening for the cable anchorage clamp. The pylon steel grade is S355 and the back tie elements use S420 steel. The end abutments consist of massive reinforced concrete blocks made of C35/45 concrete. The structure is founded on 900 mm diameter bored piles embedded in the foundation block. These piles are supplemented by slightly inclined permanent ground anchors to ensure compression in the back row of piles under permanent loads.

## STATIC AND DYNAMIC ANALYSIS

Nonlinear static and dynamic analyses were performed using a global 3D frame model in Midas Civil. The computational model included all load-bearing elements and captured their geometric and material properties. This included a sensitivity analysis of subsoil stiffness through the horizontal elastic supports of the piles. A detailed analysis of construction stages and service conditions was carried out, incorporating the time-dependent behaviour of concrete. Optimising the geometry of the suspension cables minimised the bending effects on the deck caused by permanent loads.

A stability analysis was performed on the pylons and deck, which act as the primary compression elements. This was followed by a dynamic analysis. The latter included the determination of natural frequencies and vibration modes, as well as a serviceability assessment for pedestrian comfort under harmonic excitation. The maximum vertical acceleration induced by pedestrians moving on the bridge was av = 0.5 m/s$^2$, which met the relevant code limits.

Fig. 7 Tensioning of suspension cables
Obr. 7 Napínání visutých kabelů

Fig. 8 Falsework removal
Obr. 8 Odskružení mostovky

## CONSTRUCTION SEQUENCE AND LOAD TEST

The construction procedure was developed in close cooperation with the general contractor (Metrostav – Division 1) and the footbridge contractor (Swietelsky Stavební), based on the structural concept and static behaviour. Construction of the piles, foundations and ground anchors was followed by the installation and temporary fixing of the steel pylons. The deck was then cast on fixed falsework and subsequently post-tensioned. The second stage of abutment concrete was only poured after post-tensioning had been applied, integrating the deck with the abutments and pylons. While still on the falsework, the main cables and hangers were installed. The final structural shape was achieved by prestressing the suspension cables, which also enabled the falsework to be removed. During cable stressing, the forces of both the cables and the hangers were continuously monitored using strain gauges. Finally, the deck waterproofing, railings and lighting, which were integrated into the lower handrail and at the pylon bases, were completed.

Prior to being opened to the public, the footbridge underwent static and dynamic load tests. These tests confirmed the accuracy of the design analysis and the quality of the construction. The dynamic test verified favourable vibration behaviour that meets code requirements and ensures user comfort.

## CONCLUSION

The footbridge provides an important connection across the Opava River, leading to the recreational area at Silver Lake. Its design is based on clear structural principles and simple architectural forms, as verified by detailed static and dynamic analyses. Great attention was devoted to the architectural appearance and lighting design. Thanks to close collaboration between the designer and contractor, the result is an excellent structure that is expected to serve users safely, reliably and comfortably, while also becoming a popular local landmark and recreational destination.

## MATERIAL USAGE (SUPERSTRUCTURE)

| | TOTAL | PER 1 M$^2$ |
|---|---|---|
| STRUCTURAL STEEL S355 (PYLONS) | 41.3 t | – |
| CONCRETE SLAB C40/50 | 82.9 m$^3$ | 0.25 m$^3$ |
| PRESTRESSING STEEL | 2.6 t | 7.9 kg |
| REINFORCING STEEL | 21.1 t | 64.1 kg |

## STATICKÁ A DYNAMICKÁ ANALÝZA

Nelineární statická a dynamická analýza konstrukce byla provedena na globálním 3D prutovém modelu v programu Midas Civil. Výpočetní model obsahoval veškeré nosné prvky s vystižením jejich geometrických a materiálových vlastností včetně variantního simulování tuhosti podloží pomocí pružného podepření pilot. Byla provedena detailní analýza montážních a provozních stavů s podrobnou, časově závislou analýzou se zohledněním reologických vlastností betonu. Pečlivým návrhem geometrie visutých kabelů bylo dosaženo minimálního ohybového namáhání mostovky od stálých zatížení. Pro pylon a mostovku jako rozhodující tlačené prvky byla provedena stabilitní analýza. Na statickou analýzu navazovala dynamická analýza zahrnující určení vlastních tvarů a frekvencí konstrukce s následným posouzením pohody chodců na harmonické buzení pro kritické vlastní tvary s dosaženým maximálním vertikálním zrychlením $a_v = 0{,}5\ ms^{-2}$, které bezpečně splňuje dovolené normové zrychlení.

## POSTUP VÝSTAVBY, ZATĚŽOVACÍ ZKOUŠKA

Postup výstavby byl řešen v těsné spolupráci s generálním zhotovitelem stavby (společnost Metrostav – Divize 1) a přímým zhotovitelem lávky (společnost Swietelsky stavební) na základě konstrukčního řešení a statického působení konstrukce. Po zhotovení pilot, krajních opěr a napnutí zemních kotev byly osazeny ocelové pylony s montážním zakotvením do základů. Následně byla na pevné skruži vybetonována mostovka a bylo provedeno její předepnutí. Po zmonolitnění opěr s mostovkou byly osazeny visuté kabely se závěsy a jejich následným předepnutím získala konstrukce výsledný tvar a došlo k odskružení mostovky. Při napínání visutých kabelů byla kontrolována jak předpínací síla ve visutých kabelech, tak byly pomocí tenzometrů sledovány síly v tyčových závěsech. Následně byla dokončena izolace mostovky, osazeno zábradlí a zkompletováno osvětlení lávky umístěné v dolním madle zábradlí a při patách pylonů.

Lávka byla na závěr před uvedením do provozu ověřena statickou a dynamickou zatěžovací zkouškou, které potvrdily správnost předpokladů statického výpočtu, ale i kvalitu stavby samotné. Při dynamické zkoušce byla prokázána příznivá dynamická odezva konstrukce, která splňuje normové požadavky a zajišťuje komfort uživatelů.

Fig. 9 Suspension cables and hangers
Obr. 9 Visuté kabely a závěsy

## ZÁVĚR

Lávka zabezpečuje důležité dopravní spojení přes řeku Opavu s rekreační oblastí Stříbrného jezera. Při návrhu bylo užito čistých konstrukcí s jasným statickým a konstrukčním řešením, které bylo vyvinuto a ověřeno detailní statickou a dynamickou analýzou. Velká pozornost byla věnována architektonickému vyznění a osvětlení lávky. Díky úzké spolupráci projektanta a zhotovitele je výsledkem vynikající konstrukce a lze předpokládat, že lávka bude nejen bezpečně, spolehlivě a komfortně sloužit svým uživatelům, ale stane se také vyhledávanou spojnicí pro odpočinkové a rekreační aktivity.

## SPOTŘEBA MATERIÁLŮ (NOSNÁ KONSTRUKCE)

| | CELKEM | NA 1M$^2$ |
|---|---|---|
| KONSTRUKČNÍ OCEL S355 (PYLONY) | 41,3 t | – |
| BETON MOSTOVKY C40/50 | 82,9 m$^3$ | 0,25 m$^3$ |
| PŘEDPÍNACÍ VÝZTUŽ | 2,6 t | 7,9 kg |
| BETONÁŘSKÁ VÝZTUŽ | 21,1 t | 64,1 kg |

Fig. 10 Side view
Obr. 10 Boční pohled

# PEDESTRIAN BRIDGE IN THE TYRESTA NATIONAL PARK, SWEDEN

# LÁVKA V NÁRODNÍM PARKU TYRESTA, ŠVÉDSKO

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**The bridge forms a stress-ribbon of length of 74.40 m with a sag of 1.50 m. The deck is assembled of precast segments and cast-in-place end segments which are fixed into the end abutments. Due to limited access to the construction site the bridge was erected from the lake using construction equipment situated on pontoons. Structural arrangement was developed on the bases of detailed static and dynamic analyses.**

**Lávku tvoří předpjatý pás délky 74,40 m a průvěsu 1,50 m. Mostovka je sestavena z prefabrikovaných segmentů a koncových monolitických segmentů vetknutých do opěr. Vzhledem k omezenému přístupu na staveniště byl most postaven z jezera stavební technikou umístěnou na pontonech. Konstrukční uspořádání bylo navrženo na základě podrobných statických a dynamických analýz.**

Fig. 1 Tyresta Bridge
Obr. 1 Lávka Tyresta

The pedestrian bridge over Nyforsviken Lake connects the Alby nature reserve with the Tyresta national park. The parks are situated 25 km southeast of Stockholm. The presented stress-ribbon structure is a result of the architectural competition in which the first prize received the design of the architectural firm Gottlieb Paludan Architects, Denmark supported by the engineering firm Stráský, Hustý a partneři, Czech Republic.

## STRUCTURAL AND ARCHITECTURAL DESIGN

The 86.4 m long bridge consists of a stress-ribbon deck with a length of 74.40 m and a sag of 1.50 m. The stress-ribbon is fixed into 6.0 m long end abutments – see Fig. 2. The deck is assembled from 31 segments with a length of 2.40 m; 29 inner segments are formed by precast members; the end two segments are cast-in-place members in which reinforcing bars protruding both from the neighboring precast segments and the end abutments overlap with the segment reinforcement.

The 345 mm deep segments from C50/60 have a double tee cross-section formed by edge beams and a connecting slab. On the outer faces short overhangs supporting the railings are created. While the joints between segments are filled with concrete, the joints between overhangs remain free. The 50 mm wide joints were cast between the segment faces provided with low forming corbels – see Fig.3. All segments are supported by bearing cables formed by 19 monostrands grouted in stainless steel pipes. The steel pipes are connected to the deck by shear studs welded to the pipes and cast in the segment joints. The bridge deck is prestressed by 2 × 2 tendons led in the edge beams. These tendons are assembled from 13 monostrands grouted in PE pipes.

The end abutments form anchor blocks for anchoring both bearing and prestressing cables which overlap with 12 rock anchors formed by 19 strands – see Fig.4. The abutments are founded on rock.

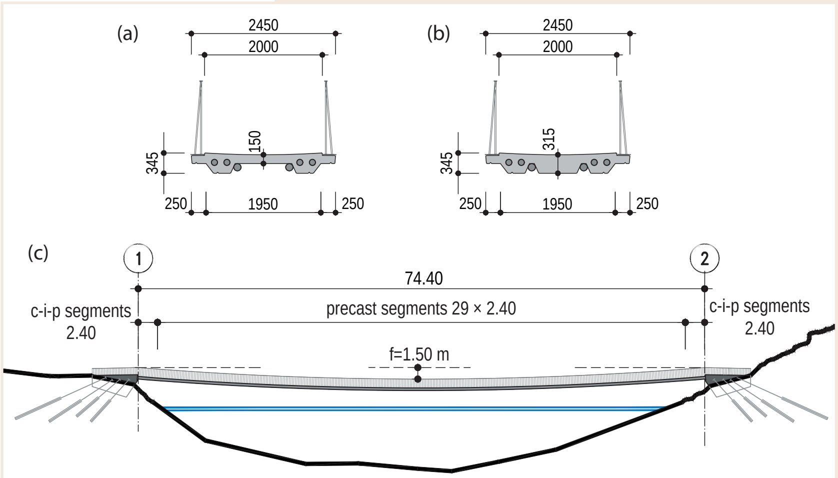


Fig. 2 Bridge:(a) typical cross section, (b) cross section at abutments, (c) elevation
Obr. 2 Lávka: (a) typický příčný řez, (b) příčný řez u opěr, (c) podélný řez

## BRIDGE CONSTRUCTION

Since the narrow approach hiking trails are formed from wooden walkways, there was no access for construction

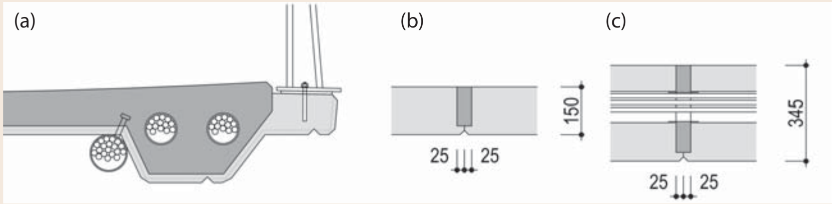


Fig. 3 Deck at joints: (a) cross section, (b) longitudinal section at slab, (c) longitudinal section at edge beam
Obr. 3 Mostovka ve spárách: (a) příčný řez, (b) podélný řez v desce, (c) podélný řez krajním trámem

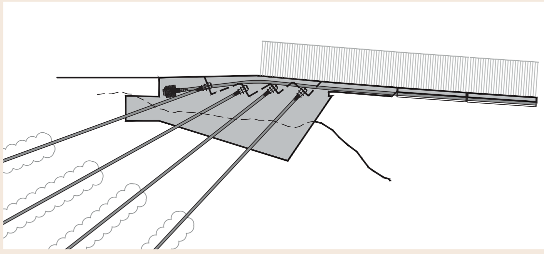

Fig. 4 Longitudinal section at the end abutments
Obr. 4 Podélný řez krajní opěrou opěrou

equipment from both banks. Therefore, all equipment and construction materials were transported to the bridge site on pontoons.

At first the abutments were cast and rock anchors were installed and tensioned. Then the erection strands carrying the steel pipes were erected and tensioned. After that the steel pipes were suspended on erection strands. After the pipes were mutually welded, the monostrands forming the bearing cables were pulled through the pipes and tensioned to the designed stress.

The precast segments were erected from the centre of the bridge to both abutments with the maximum unbalanced weight of one segment – see Fig. 5. When all segments were placed, the formworks of the end segments were suspended on the end precast segments and on the abutments, PE pipes in the joints were installed and monostrands of the prestressing tendons were pulled through the segments and abutments. After adjustment of the tension in bearing cables, the joints between the segments and the end segments were cast. When concrete of the joints had sufficient strength, the prestressing tendons were tensioned. Then the cables were grouted and railings were erected. The bridge construction started in 2021, and was completed in 2023 – see Fig. 6.

## STATIC AND DYNAMIC ANALYSES

The structure was analyzed as a spatial structure assembled from beam elements. The non-linear analysis of the erection and service states started from the chosen initial state in which the required geometry and stress state were defined. The structure was analyzed by using Midas Civil software.

The fact that the bridge deck is prestressed by un-bonded cables formed by mono-strands guarantees that with any deformation of the bridge deck, the tension in the cables increases along their entire length. When the structure is loaded by a live load, it deforms and corresponding radial forces in the cables stabilize the structure and limit its deformations. This creates a very stiff static system in which the response to the movement of the structure from pedestrian or wind loads is minimal.

A modal analysis of the structure was performed to evaluate pedestrian comfort and wind effects. Aerodynamic stability was verified by checking the ratio of the corresponding torsional and bending frequencies, which was greater than the recommended value of 2.5. Pedestrian comfort was assessed according to Sétra Guidelines for the design of footbridges. The structure is sufficiently stiff and the movement and acceleration of the structure induced by dynamic loads acting in both the vertical and horizontal directions does not cause discomfort to users.

## CONCLUSIONS

The bridge has a very slender deck; it is light and transparent. It had a minimal impact on the environment during the construction. The stress ribbon structure does not disturb the beautiful environment, but rather it complements it. Thanks to the appropriate arrangement of structural elements economically stressed predominantly by normal forces; it is sufficiently rigid and comfortable for users. Since it does not have bearings and expansion joints it requires minimum maintenance. The bridge was well-received by users.

The client of the bridge is: Tyresô kommun, Stockholm. The bridge was designed by AFRY, Stockholm for whom the firm Stráský, Hustý a partneři, s.r.o., Brno developed the structural design and worked out detailed static and dynamic analysis. The bridge was built by PEAB, SWE.

Fig. 5 Segments on bearing cables
Obr. 5 Segmenty na nosných kabelech

Fig. 6 Tyresta Bridge
Obr. 6 Lávka Tyresta

# STRUCTURAL CONCRETE IN TRAM TUNNEL AND ROAD GALLERY CONSTRUCTION UNDER CONSTRAINED URBAN CONDITIONS – I/42 VMO ŽABOVŘESKÁ PHASE II

# KONSTRUKČNÍ BETON PŘI VÝSTAVBĚ TRAMVAJOVÉHO TUNELU A SILNIČNÍ GALERIE V OMEZENÝCH MĚSTSKÝCH PODMÍNKÁCH – I/42 VMO ŽABOVŘESKÁ, ETAPA II

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**The construction of the I/42 VMO Žabovřeská, Phase II, seamlessly followed the works carried out within Phase I and thus became an integral part of the City Ring Road in Brno. The implementation of this project eliminated a long-standing bottleneck where the existing two-lane I/42 road had been constrained by the Svratka River and the steep rocky slope of Wilson Forest. At the same time, the original alignment of the I/42 road, which had run longitudinally parallel to the tram line, was removed. The completed project enabled the widening of the road into a directionally divided four-lane carriageway, including the construction of a road gallery. This also created new public space and a calm urban zone in the wider city centre of Brno. The tram line is now routed through a newly constructed 500 m long tunnel. The total length of the project is 920 m. Construction works commenced in December 2020, the road was opened to traffic in September 2024, and full completion was achieved in 2025. In the same year, the project received the "Construction of the Year 2025" award. The article itself focuses on the project from the perspective of the Technical Supervisor of the Investor (Assistant to The Engineer), particularly with regard to the supervision of compliance with the prescribed quality and technical parameters. Structural concrete was used in the construction of the tram tunnel, both in the section with a cut-and-cover lining and in the mined section. Reinforced concrete structures were also designed and implemented for the retaining walls, and the road gallery, as well as the control room and technical facilities.**

**Stavba I/42 VMO Žabovřeská, etapa II, plynule navázala na práce provedené v rámci etapy I a jako taková se stala součástí Velkého městského okruhu v Brně. Realizací této stavby bylo odstraněno úzké hrdlo, kde byla stávající dvoupruhová silnice I/42 omezena řekou Svratkou a strmým skalnatým svahem Wilsonova lesa. Současně byl odstraněn původní stav silnice I/42 vedené v podélném souběhu s tramvajovou tratí. Dokončená stavba umožnila rozšíření komunikace na směrově rozdělenou čtyřpruhovou silnici, včetně realizace silniční galerie, čímž došlo rovněž k vytvoření veřejného prostranství a klidové zóny v širším centru Brna. Tramvajová trať je nyní vedena v nově vybudovaném tunelu o délce 500 m. Celková délka stavby činí 920 m. Zahájení prací proběhlo v prosinci 2020, stavba byla uvedena do provozu v září 2024 a dokončena v roce 2025. V témže roce získala titul Stavba roku 2025. V samotném článku je kladen důraz na realizaci z pozice technického dozoru investora (asistenta správce stavby), zejména na kontrolu dodržování předepsaných kvalitativních a technických parametrů. V rámci stavby byly použity konstrukční betony jak při výstavbě tramvajového tunelu v části s přesýpaným ostěním, tak v části s raženým ostěním. Konstrukce ze železobetonu byly dále navrženy a použity při realizaci zárubních zdí, silniční galerie, jakož i prostor řídicího velínu a technického zázemí.**

Fig. 1 Tunnel section concrete work and formwork
Obr. 1 Betonáž a bednění části tunelu

Fig. 2 The implementation of waterproofing foil
Obr. 2 Instalace hydroizolační fólie

## SECURING THE SLOPE OF WILSON FOREST

Prior to commencing the main structural works, it was necessary to stabilise the rock mass forming the slope of Wilson Forest. The slope was located directly above the future portals of the tram tunnel and the road gallery.

As part of the removal of the disrupted section of the tram line, a closure of the section between the Pisárky and Bráfova tram stops was required. Replacement bus services were provided for passengers throughout the closure period.

Before construction works in the Wilson Forest area could begin, the slope had to be secured using protective mesh and rock anchors. The total area covered with mesh was approximately 10,000 m$^2$. Several types of anchors of varying lengths were installed, with approximately 3,000 anchors used in total to stabilise the slope

Due to the instability of the rock mass and its limited accessibility, the installation of these stabilisation measures had to be carried out using rope access techniques and specialised equipment.

## THE TRAM TUNNEL

The largest volume of construction activities during the first year of the project consisted of tunnelling works. The tunnel was excavated using the New Austrian Tunnelling Method (NATM), which involves staged excavation of the tunnel profile in partial sections, allowing an immediate response to adverse geological conditions encountered.

Excavation commenced in May 2021 and was completed with the breakthrough of the top heading in the second half of August 2021. The Contractor excavated the 332 m long tunnel in just over three months. Tunnelling progressed continuously, with two to three blasting cycles carried out per day. The advance length per round ranged from 1.5 to 2.5 m. In total, more than 24,000 m$^3$ of excavated material was removed.

Following completion of the excavation works, construction of the primary lining and subsequently the secondary lining proceeded without interruption. The mined tunnel section was designed as a double-shell structure with an intermediate umbrella waterproofing system consisting of a welded waterproofing membrane – see Fig. 2.

The total volume of concrete used for over-excavation backfilling and tunnel lining amounted to approximately 4,500 m$^3$, of which 2,300 m$^3$ constituted the secondary lining. Considering the situation in the Czech construction industry in 2021, when the sector was affected not only by significant price increases but also by material shortages, the Contractor implemented part of the primary lining using fibre-reinforced concrete (polypropylene fibres with the total amount used exceeding 6 tonnes), in which the fibres partially replaced conventional reinforcement.

The tram tunnel consisted of two sections. The first section, 162 m in length, was constructed as a cut-and-cover structure. The second section, 332 m long, was excavated in rock using the mining method.

The cut-and-cover tunnel structure was constructed of cast-in-place reinforced concrete class C30/37, with a wall thickness of 450 mm. Concreting was carried out using a steel travelling formwork system, including counter-formwork, with a segment length of 8 m. The formwork system comprised not only the shuttering itself but also equipment ensuring proper compaction of the concrete. Concreting of a single segment took approximately 4 hours.

The tram tunnel cross-section was designed to ensure safe two-way tram operation, including inspection walkways on both sides with a width of 1.0 m. The tunnel had a base width of 8.8 m and an internal height of 7.7 m.

The primary lining was constructed using sprayed concrete with a thickness of 100–200 mm, reinforced with lattice girders and welded wire mesh. Subsequently, a protection and drainage layer consisting of 500 g/m$^2$ geotextile and a 3 mm thick waterproofing membrane was installed. The secondary lining had a thickness of 300 mm and was constructed of C30/37 concrete.

Fig. 3 View of the track bed (up) and the track structure (down)
Obr. 3 Pohled na kolejové lože (nahoře) a konstrukci tramvajové trati (dole)

## TRAM TRACK STRUCTURE

The tram track structure consisted of a reinforced concrete slab made of C30/37 exposure class X1 concrete with a Fimafob surface treatment (75–125 g/m$^2$). The slab was reinforced with welded wire mesh 150/150/8 with a 50 mm concrete cover and was constructed in two stages over an underlying C16/20 exposure class X0 concrete base layer.

In the first stage, the lower reinforced slab was cast on an anti-vibration mat. Its thickness varied according to the transverse gradient defined by the track alignment. The rails, including fastening systems and adjustment sleepers, were positioned in the designed horizontal and vertical alignment using adjustment screws prior to casting the upper slab.

Fig. 4 Single-sided gallery (up) and double-sided gallery (down)

Obr. 4 Jednostranná galerie (nahoře) a oboustranná galerie (dole)

In the second stage, the track and fastening system were encased in concrete up to the level of the plastic baseplates (minimum embedment depth 8 mm). The upper slab had a constant thickness of approximately 290 mm.

The right and left tracks were concreted separately. Expansion joints corresponded to the tunnel lining construction joints at approximately 8.0 m intervals. Drainage was provided by a transverse gradient directing water into prefabricated concrete drainage channels connected to the sewer system – see Fig. 3.

Service walkways were constructed along both sides of the tunnel, supported either by concrete foundation strips or by the structural slab, and separated by vertical anti-vibration mats. Cable ducts and utility chambers for communication and power lines were integrated beneath and within the walkways. The total length of the walkways was 500 m, with barrier-free connections to the portal areas.

### Road Gallery

In 2021, the Contractor carried out works not only on the tram tunnel section but also on the stabilization of retaining walls along the future tram alignment and supported the construction of a pedestrian footbridge. The footbridge was designed to provide a safe crossing over Road I/42. Other major construction activities focused on the road gallery structure.

The designed gallery comprised two directly connected parts: a single-sided gallery on Branch B of Road I/42 (Žabovřeská), with a length of 247.7 m measured along Axis B, covering the carriageway in the direction from Pisárky to Královo Pole; and a double-sided gallery – an ecobridge spanning both carriageways of Road I/42 (Branches A and B) – see Fig. 4. The ecobridge had a length of 80.0 m measured along Axis A. Both parts were divided into individual expansion units.

The single-sided gallery consisted of a reinforced concrete frame structure with a continuous rear wall (on the right-hand side in the direction of chainage), transverse frame walls, and a front wall with light openings forming a colonnade between Branches A and B.

The columns were connected to the frame beams by rigid joints. At their base, the columns were monolithically connected to the central foundation rib walls. The front edge of the gallery roof slab was finished with a vertical parapet and railing at the front and on the left-hand side in the direction of chainage.

The foundation of the single-sided gallery was partly shallow and partly deep, comprising bored piles with diameters of up to 1,200 mm and variable lengths of up to 12 m. In approximately the first half of its length, the structure was extended on the right-hand side by an emergency lay-by with a width of 3.0 m. The total length of this extension, including the taper section, was approximately 52.5 m. The final appearance of the structure is shown in Fig. 5.

## TRAM AND GALLERY CONTROL SYSTEMS

The Operational Technology Facility (OTF) was located at the northern portal of the VMO and the tram tunnel, near the end of Bráfova Street, between the tram tunnel and the slope of Wilson Forest. The OTF was positioned adjacent to the outer lining of the tram tunnel and partially extended into the existing hillside.

Fig. 5 Completed structure opened to traffic
Obr. 5 Dokončená stavba již za provozu

As a pedestrian walkway was constructed above the OTF within the backfill above the northern portal of the VMO and the tram tunnel, the facility was predominantly located below the proposed finished ground level. Only the front façade with the main entrances and a portion of the roof—finished with washed aggregate paving (cobblestone)—remained accessible.

All control systems and technological equipment within the project were connected to the operations technology facility (described above), where switchboards, substations, and other necessary equipment were installed in dedicated rooms to ensure proper system functionality.

In the event of a failure or emergency, the dispatcher was able to control traffic in both the tram tunnel and the road gallery directly from the control centre located in the operations technology building.

The transport and technology control system was used to manage road and tram traffic as well as the technological equipment of the tram tunnel. It ensured the transmission of operational data to the central control rooms. Under normal operating conditions, the system monitored traffic and technological equipment in both the tunnel and the gallery. In emergency situations, it proposed traffic restrictions or tunnel/gallery closures to the dispatcher and automatically activated the required operational technologies according to predefined scenarios.

In the event of a tunnel closure, the dispatcher was able to control the traffic signals. The control system also integrated individual subsystems, such as air quality control, CCTV integration, and high-voltage equipment control.

## CONCLUSION

I/42 VMO Žabovřeská represented an exceptionally complex project combining a mined tunnel and a cut-and-cover tunnel, extensive reinforced concrete structures, rock slope stabilisation, and advanced technological systems within a constrained urban environment.

During the implementation phase, systematic control of compliance with the design documentation, technical standards, and the FIDIC Red Book contractual conditions was essential. Particular attention was paid to the quality of structural concrete, the execution of the primary and secondary tunnel linings, waterproofing systems, and the construction of load-bearing elements of the road gallery and retaining structures. Equally important was the supervision of geotechnical monitoring during tunnelling works and the coordination and commissioning of technological systems prior to opening the project to traffic.

Despite challenging geological conditions and instability in the construction materials market, the project was put into operation within the planned timeframe and fully completed in 2025. It can be concluded that rigorous technical supervision, continuous quality control, and effective communication among all construction stakeholders were key factors in the successful delivery of the project. The Author of the article acted as The Engineer's representative throughout the entire construction period. Ultimately, the project represents not only a significant improvement to transport infrastructure in the City of Brno, but also an example of well-coordinated delivery of a technically demanding project while maintaining the required quality and safety standards.

# NEW BUBNY RAILWAY STATION BUILDING – STRUCTURAL DESIGN

# NOVÁ BUDOVA NÁDRAŽÍ BUBNY – KONSTRUKČNÍ ČÁST



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**The new Praha–Bubny railway station is part of the project "Modernizace trati Praha-Bubny (vč.) – Praha-Výstaviště (vč.)" and is located in the centre of Prague. The project represents one stage in the development of a new rail connection between the city centre and Václav Havel Airport, while simultaneously increasing the capacity of the railway line between Prague and Kladno. The new station building is situated in close proximity to the Vltavská metro station and tram stops as well as the old Praha–Bubny station. It will serve as a major transport hub for passengers travelling towards the city centre from the directions of Kladno and Kralupy nad Vltavou. The structure has an area of approximately 250 × 50 m and a height of 17 m. All load-bearing structures are designed as reinforced concrete elements with high architectural and structural performance requirements.**

**Nové nádraží Praha-Bubny je součástí stavby „Modernizace trati Praha-Bubny (vč.) – Praha-Výstaviště (vč.)" a nachází se v centru Prahy v nedaleko původního nádraží Praha-Bubny. Stavba je jednou z etap budování nového železničního spojení mezi centrem Prahy a Letištěm Václava Havla a současně zkapacitněním spojení mezi Prahou a Kladnem. Budova nového vlakového nádraží se nachází na začátku modernizovaného úseku v těsné blízkosti stanice metra a tramvají Vltavská a původního nádraží Praha-Bubny a bude sloužit jako důležitá dopravní křižovatka pro cestující směřující do centra města ze směrů Kladno a Kralupy nad Vltavou. Nádražní budova má půdorys o rozměrech cca 250 × 50 m a výšku 17 m a veškeré nosné konstrukce jsou železobetonové s vysokými pohledovými i konstrukčními nároky.**

## GENERAL INFORMATION ABOUT THE PROJECT

The new Praha–Bubny railway station is part of the project "Modernizace trati Praha-Bubny (vč.) – Praha-Výstaviště (vč.)" and is located in the centre of Prague, close to the original Praha–Bubny station. The project represents one stage of the development of a new railway connection between the city centre and Václav Havel Airport, while simultaneously increasing the capacity of the railway line between Prague and Kladno.

The new station building is situated at the beginning of the modernised section near the northern end of the Negrelli Viaduct, in close proximity to the Vltavská metro station and tram stops as well as the original Praha-Bubny station. It is intended to serve as a major transport hub for passengers travelling towards the city centre from the directions of Kladno and Kralupy nad Vltavou. In the future, it will become the principal station on the railway line to Václav Havel Airport.

Construction started in spring 2023 and was finished in autumn 2025. The client is Správa železnic, s.o., and the general designer is Metroprojekt a.s. The construction works are carried out by a consortium consisting of Metrostav TBR, OHLA ŽS, and Elektrizace železnic Praha. The new station building itself is being implemented by Metrostav TBR and Metrostav CZ (Fig. 1).

## SO 01-61-01 – PRAHA–BUBNY RAILWAY STATION, SO 01-62-01 – PRAHA–BUBNY RAILWAY STATION – ROOFING STRUCTURE

The new station building has a plan dimension of approximately 250 × 50 m and a height of 17 m. Passenger concourses and operational facilities of the railway station are located on the ground floor. Train operation takes place on the first floor, which comprises a total of four tracks and three platforms. Passengers access the platforms by escalators and elevators. All platforms are covered by a reinforced concrete roof structure with circular skylights. In the southern part of the roof there is an accessible terrace with a view of the city centre. In the future, the client expects the utilization of the extensive roof area for construction of a multi-storey administrative building. The railway track of the station is connected on the southern side to the adjacent Bubny underpass structure and on the northern side to the adjoining viaducts, which form part of the modernisation of the entire railway section (Fig. 2).

All load-bearing structures are designed as reinforced concrete elements with high architectural and structural performance requirements. The building follows a strict architectural concept, and all visible reinforced concrete structures are designed in white fair-faced concrete. These include, in particular, the concourse columns, platform columns, and the columns and slab of the roof structure. Prior to the construction, concrete sampling and testing was carried out in cooperation with specialists from TBG METROSTAV s.r.o. and the Klokner Institute of the Czech Technical University in Prague. The selected concrete for the load-bearing exposed structures achieves its white colour through the addition of titanium white pigment.

## CONSTRUCTION PROCESS

The first construction works involved the preparation of a temporary railway track outside the station area in order to maintain uninterrupted train operation. Subsequently,


Fig. 1 Construction site overview
Obr. 1 Výřez ze situace stavby

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## OBECNÉ INFORMACE O STAVBĚ

Nové nádraží Praha-Bubny je součástí stavby „Modernizace trati Praha-Bubny (vč.) – Praha-Výstaviště (vč.)" a nachází se v centru Prahy v nedaleko původního nádraží Praha-Bubny. Stavba je jednou z etap budování nového železničního spojení mezi Prahou a Letištěm Václava Havla a současně zkapacitněním spojení mezi Prahou a Kladnem.

Budova nového vlakového nádraží se nachází na začátku modernizovaného úseku u severního konce Negrelliho viaduktu a zároveň v těsné blízkosti stanice metra a tramvají Vltavská a původního nádraží Praha-Bubny a má sloužit jako důležitá dopravní křižovatka pro cestující směřující do centra města ze směrů Kladno a Kralupy nad Vltavou. V budoucnu půjde o hlavní stanici na trati na Letiště Václava Havla.

Výstavba byla zahájena na jaře roku 2023 a dokončena byla na podzim roku 2025. Objednatelem stavby je Správa železnic, s. o., a jejím generálním projektantem Metroprojekt Praha a.s. Stavbu provádí sdružení firem Metrostav TBR, OHLA ŽS a Elektrizace železnic Praha. Objekt nové nádražní budovy realizovaly Metrostav TBR a Metrostav CZ (obr. 1).

## SO 01-61-01 – ŽST PRAHA-BUBNY, SO 01-62-01 – ŽST PRAHA-BUBNY – ZASTŘEŠENÍ

Nová nádražní budova má půdorys o rozměrech cca 250 × 50 m a výšku 17 m. V přízemním podlaží se nachází vestibuly pro cestující a zázemí železniční stanice. V prvním podlaží probíhá vlaková doprava. Jsou zde celkem čtyři koleje a tři nástupiště, na která se cestující dostanou pomocí eskalátorů a výtahů. Všechna nástupiště jsou zastřešena jednotnou betonovou konstrukcí s kruhovými světlíky. V jižní části zastřešení je navíc terasa s výhledem na centrum hlavního města. V budoucnu uvažuje investor o využití velké střešní plochy pro výstavbu několikapatrového administrativního objektu. Průběžné kolejiště na nádraží je napojené na jihu na sousední stavbu Podjezd Bubny a na severu na přilehlé estakády, které jsou součásti modernizace celého úseku (obr. 2).

Veškeré nosné konstrukce jsou železobetonové s vysokými pohledovými i konstrukčními nároky. Stavba podléhá architektonickému návrhu a všechny viditelné železobetonové konstrukce jsou navrženy z bílého pohledového betonu. Jedná se zejména o sloupy vestibulů, sloupy nástupišť a sloupy a desku zastřešení. Před započetím výstavby pohledových konstrukcí proběhlo vzorkování betonu ve spolupráci se specialisty z TBG Metrostav s.r.o. a Kloknerova ústavu ČVUT v Praze. Zvolená varianta pro nosné pohledové konstrukce je beton, jehož bílé barvy je dosaženo přidáním pigmentu z titanové běloby.

## PRŮBĚH VÝSTAVBY

Prvními pracemi na stavbě byla příprava provizorní koleje mimo prostor nádraží, aby byl zachován vlakový provoz. Následně proběhlo odstranění původního kolejiště v celém prostoru stavby a v květnu 2023 započaly zemní práce pro založení nového nádraží. Celý objekt je podélně členěn do pěti dilatačních celků DC1–DC5 o rozměrech zhruba 50 × 50 m, které určovaly postup výstavby. Nejprve probíhala výstavba dilatačních celků DC1, DC3 a DC5, kde jsou umístěné vestibuly a následně stavba pokračovala dilatačními celky DC2 a DC4, kde jsou spojovací chodby mezi vestibuly (obr. 3).

Fig. 2 Architect's visualization
Obr. 2 Vizualizace architekta

Objekt nádraží je založen celkem na 493 velkoprůměrových pilotách délky 4,5–9 m, na kterých je následně provedena základová železobetonová deska o mocnosti 0,8–2,5 m. Deska je členěna na mnoho stavebních celků jednak pro úsporu betonu a také pro vedení kanalizace a dalších technologií.

Sloupy vestibulů jsou provedeny z bílého pohledového betonu dle architektonického návrhu a pro optimální prostorové řešení výsledné stavby je zde použito několik různých průřezů sloupů. Jsou zde čtvercové a obdélníkové sloupy, dále sloupy kruhové o různých průměrech a také sloupy oválné. Pro dosažené požadované pohledové kvality byl navíc použit beton se zpomaleným nárůstem pevnosti (90denní). Výška sloupů je 4,7 m, a i když se jedná o nosnou konstrukci budovy, zatížení a dimenze sloupů odpovídají spíše mostním konstrukcím – sloupy nesou desku nádraží s nástupišti, provozovanými kolejemi, a navíc do nich přechází také zatížení ze zastřešení celého nádraží a budoucí výstavby na střeše nádraží (obr. 4).

V listopadu 2023 proběhla betonáž první stropní desky vestibulů DC1. Jedná se o železobetonovou desku tloušťky 0,8 m, která slouží jako nosná konstrukce nádraží. Během následujících dvou měsíců byly vybetonovány desky DC3 a DC5 a na zbývajících dilatačních celcích, kde nejdou vestibuly pod nástupišti, proběhla výstavba základových patek sloupů.

Zastřešení je také svými rozměry blíže k mostním konstrukcím než standardním pozemním stavbám. Sloupy jsou v rastru přibližně 9 m, jsou provedeny ze stejného betonu a podléhají stejným nárokům jako sloupy vestibulů,

Fig. 3 Foundation slab, insulation
Obr. 3 Podkladní beton a izolace pod členěnou základovou deskou

Fig. 4 First floor columns
Obr. 4 Sloupy a stěny vestibulů

the original track layout within the entire construction site was removed, and in May 2023, earthworks for the foundations of the new station commenced. The entire structure is longitudinally divided into five expansion units (DC1-DC5) with approximate dimensions of 50 × 50 m, which determined the construction sequence. Construction initially focused on expansion units DC1, DC3, and DC5, where the concourses are located, followed by units DC2 and DC4, which contain connecting corridors between the concourses (Fig. 3).

The station building is founded on a total of 493 large-diameter driven piles with lengths ranging from 4.5 m to 9.0 m. A reinforced concrete foundation slab with a thickness varying between 0.8 m and 2.5 m is constructed on top of the piles. The slab is divided into multiple construction sections, both to optimise concrete consumption and to allow for the installation of sewer lines and other technical services.

The concourse columns are executed in white fair-faced concrete in accordance with the architectural design. Several different column cross-sections are used to achieve an optimal spatial arrangement of the final structure, including square and rectangular columns, circular columns of various diameters, and oval columns. To achieve the required surface quality, concrete with delayed strength development (90-day) was used. The columns have a height of 4.7 m and, although they form part of a building structure, the applied loads and column dimensions correspond more closely to those typical for bridge structures. The columns support the station slab with platforms and operating railway tracks and also transfer loads from the roof structure and the planned future structures on top of the station roof (Fig. 4).

In November 2023, the casting of the first platform slab in unit DC1 was carried out. This reinforced concrete slab has a thickness of 0.8 m and serves as the primary load-bearing structure of the station. During the following two months, the slabs of units DC3 and DC5 were cast. At the same time in the remaining expansion units without concourses beneath the platforms, the construction of column footings took place.

Due to its dimensions, the roof structure is also closer in character to bridge structures than to standard building construction. The columns are arranged in a grid of approximately 9 m and are made of the same concrete and subject to the same requirements as the concourse columns. However, they are nearly 10 m tall and geometrically more complex. The columns are very slender and therefore heavily reinforced, and they also accommodate electrical installations and roof drainage systems. During construction, this resulted in a highly complex spatial arrangement of individual elements; nevertheless, thanks to close cooperation between the designer, structural engineer, and skilled craftsmen, all challenges were successfully overcome (Fig. 5).

After completion of the columns in the first expansion unit, construction of the roof slab commenced. The slab was cast on a fixed spatial falsework system in April 2024. The roof slab has a thickness of 0.8 m and includes circular openings with a diameter of 4.6 m for prefabricated skylights. The casting of the slab was carried out using two different concrete mixtures simultaneously: pigmented concrete with titanium white for the underside surface, slab edges, and visible edges of the skylight openings, and non-pigmented concrete for the remaining volume of the slab. The total volume of concrete in each roof expansion unit ranges from 1,200 to 1,400 m$^3$. Individual roof expansion units are structurally separated, and strip bearings are installed at their interfaces to transfer vertical loads (Fig. 6 and 7).

At the end of July 2024, the casting of the final roof slab was completed, and the installation of prefabricated skylights was finished in the autumn. At the same time, all remaining works were carried out in the service areas, concourses, railway tracks, and platforms. These included casting of staircases and escalator ramps, installation of elevators and escalators, construction of partitions, installation of all technical systems, application of waterproofing and thermal insulation layers, cladding, plastering, floor installation, and other necessary works required for the completion of the building.

## CONCLUSION

The resulting structure represents a railway station of unique dimensions within the Czech Republic, constructed using new methods and modern materials, achieving the highest standards of structural performance and architectural quality. The total volume of materials comes to 26,000 m$^3$ of concrete and 5,200 tonnes of reinforcing steel. The total duration of construction of the load-bearing structure was 15 months.

The project also includes the revitalisation of the entire surrounding area of the former railway brownfield in Bubny, with the aim of creating a modern and pleasant urban environment. The new station will offer excellent transport accessibility due to its proximity to metro, tram, and bus lines and will further contribute to the integration of public transport and suburban railway services (Fig. 8).

## REFERENCES

Project documentation „Modernizace trati Praha-Bubny (vč.) – Praha-Výstaviště (vč.)", projekt stavby, METROPROJEKT a.s., 08/2021

Map data – www.mapy.com, Seznam, a.s. 2024

Photographs – construction team of Metrostav TBR a.s.

s tím rozdílem, že jsou téměř 10 m vysoké a tvarově složitější. Sloupy jsou velice štíhlé, z pochopitelných důvodů silně vyztužené a prochází jimi elektroinstalace a odvodnění zastřešení. Během realizace jsme se tedy potýkali s komplikovaným prostorovým uspořádáním všech prvků, ale díky spolupráci s projektantem, statikem a šikovnými řemeslníky se zadařilo (obr. 5).

Po dokončení sloupů na prvním dilatačním celku proběhla výstavba desky zastřešení. Ta se betonovala na pevné prostorové skruži v dubnu 2024. Deska zastřešení má tloušťku 0,8 m, obsahuje otvory průměru 4,6 m pro střešní prefabrikované světlíky. Betonáž desky proběhla ze dvou betonových směsí současně. Z betonu s titanovou bělobou, ze kterého je provedený spodní povrch, boky desky a viditelné hrany otvorů pro světlíky. Současně s tím probíhala betonáž z betonu bez pigmentu, který je v nepohledovém objemu desky. Celkové množství betonu v každém dilatačním celku zastřešení je 1200–1400 $m^3$. Jednotlivé dilatační celky zastřešení jsou navzájem oddělené a pro přenos svislých zatížení jsou na rozmezí instalována pásová ložiska (obr. 6 a 7).

Fig. 5 Scaffolding and supports for the roof slab
Obr. 5 Příprava skruže pro desku zastřešení

Fig. 6 Reinforcement of the roof slab
Obr. 6 Výztuž desky zastřešení

Fig. 7 Roof without falsework
Obr. 7 Zastřešení po odskružení

Na konci července 2024 proběhla betonáž poslední desky zastřešení a na podzim také montáž prefabrikovaných světlíků. Zároveň proběhly veškeré práce v zázemí, ve vestibulech, na kolejišti a nástupištích. Jednalo se o betonáž schodišť a ramp pro eskalátory, montáž výtahů a eskalátorů, budování příček a instalaci veškerých technologií, provedení hydroizolací a tepelných izolací, realizaci obkladů, omítek, podlah a další nezbytné práce pro dokončení objektu.

## ZÁVĚR

Výslednou konstrukcí je nádraží jedinečných rozměrů v ČR, prováděné novými postupy, z moderních materiálů v nejvyšší konstrukční a pohledové kvalitě. Celkem bylo použito 26 000 $m^3$ betonu a 5200 tun betonářské výztuže. Celková doba realizace nosné konstrukce byla 15 měsíců.

Součástí stavby je také revitalizace celé přilehlé oblasti bývalého železničního areálu v Bubnech, která má za cíl vytvořit moderní a příjemný prostor. Nové nádraží bude mít také dobrou dopravní dostupnost, jelikož se nachází v blízkosti metra, tramvají a autobusových linek, a nadále přispěje k propojení MHD a příměstské vlakové dopravy (obr. 8).

## POUŽITÉ PODKLADY

Realizační dokumentace „Modernizace trati Praha-Bubny (vč.) – Praha-Výstaviště (vč.)", projekt stavby, Metroprojekt Praha a.s., 08/2021

Mapové podklady – www.mapy.com, Seznam, a.s., 2024

Fotografie – realizační tým Metrostav TBR a.s.

Fig 8 Finished roof structure
Obr. 8 Dokončená nosná konstrukce

# UNIVERSITY FACILITIES FOR SPORT AND BEHAVIORAL HEALTH – HYBRID LOAD-BEARING SYSTEM

# UNIVERZITNÍ ZÁZEMÍ SPORTU A BEHAVIORÁLNÍHO ZDRAVÍ – HYBRIDNÍ NOSNÁ KONSTRUKCE



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**The area of Černá louka in Ostrava on the left bank of the Ostravice River was for many years considered one of the less attractive parts of the city, with little reason for residents or visitors to visit. Two new buildings forming part of the Ostrava University Campus have radically transformed this area, and the surrounding land is now widely used by the public for active recreation and cultural activities. The facilities for sport and behavioral health, together with the Arts and Design Cluster and landscaping along the left bank of the Ostravice, have become a prominent new landmark in this part of the city.**

**Území Černé louky v Ostravě na levém břehu Ostravice dlouhé roky patřilo k těm nepříliš půvabným místům, které obyvatelé ani návštěvníci města neměli důvod navštěvovat. Dvě nové budovy, patřící do kampusu Ostravské univerzity, toto místo radikálně proměnily a celé přilehlé území je nyní využíváno širokou veřejností k aktivnímu odpočinku i kulturnímu vyžití. Zázemí sportu a behaviorálního zdraví i Klastr umění a designu společně s úpravami okolního terénu na levém břehu Ostravice se právem staly novou vyhledávanou dominantou této části města.**

Fig. 1 View of the completed building
Obr. 1 Pohled na hotovou budovu

The University Facilities for Sport and Behavioral Health are part of the newly developed Ostrava University campus on the left bank of the Ostravice River in Černá louka – see Fig. 1. They are adjacent to another new campus building, the Arts and Design Cluster.

The sports facility includes a central sports hall for ball and selected racket sports with a capacity of 550 spectators, halls for gymnastics, aerobics and fitness, a climbing wall, locker rooms, teaching spaces including a lecture hall and classrooms, as well as research and monitoring facilities, laboratories for sports science, magnetic resonance imaging, and staff offices – see Fig. 2. An indoor running track with four lanes is located up to the fourth above-ground floor. It is directly connected to the laboratories for movement biomechanics and behavioral health, providing the possibility of direct kinetic and kinematic measurements. The running track extends beyond the building envelope in curved sections, forming the main architectural feature of the facility – see Fig. 3, Fig. 4, complemented by a terraced façade with stepped seating for up to 1,000 spectators. A green roof is publicly accessible and includes a workout area and an outdoor running track. The basement houses parking for visitors and the general public.

The project was initiated in 2017 with documentation for a building permit, followed by detailed construction documentation in 2019. Construction proceeded from 2020 until autumn 2022, when the building was handed over for use.

The building has a hybrid load-bearing system comprising a reinforced concrete frame, steel elements for long-span floor and roof structures, and suspended arches supporting the running track. The building is divided by structural expansion joints into three parts, each with different functional layouts in the above-ground storeys.

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The building is founded on large-diameter bored piles acting together with a 300 mm-thick foundation slab, reinforced with strip footings beneath columns to a thickness of 700–1000 mm. Beneath the main piers, where four piles are designed, the foundation pit also serves as a pile tie.

The basement load-bearing structure is monolithic reinforced concrete. Vertical elements include perimeter walls, internal core walls, and columns. Horizontal elements consist of floor slabs, reinforced with beams or column capitals in critical areas. Column spacings are irregular, ranging from 4.5 m to 10.8 m. Column cross-sections are predominantly square, 400 × 400 mm, while the main massive piers supporting the steel roof trusses at the third floor have cross-sections of 1000 × 2000 mm – see Fig. 5. The floor slab above the first basement level is 300 mm thick, with the maximum beam depth

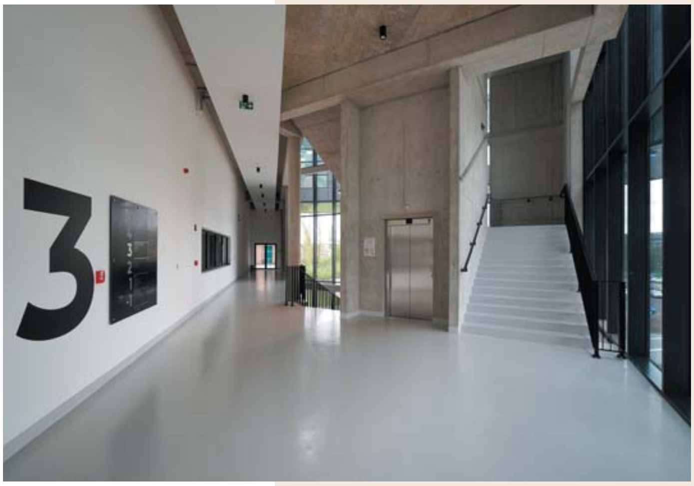
Fig. 2 Climbing wall
Obr. 2 Horolezecká stěna



Fig. 3 Running track
Obr. 3 Běžecký ovál

Fig. 4 Running track – interior
Obr. 4 Běžecký ovál – interiér

Fig. 5 Column with a brace – support of steel trusses
Obr. 5 Pilíř se vzpěrou – podepření ocelových vazníků



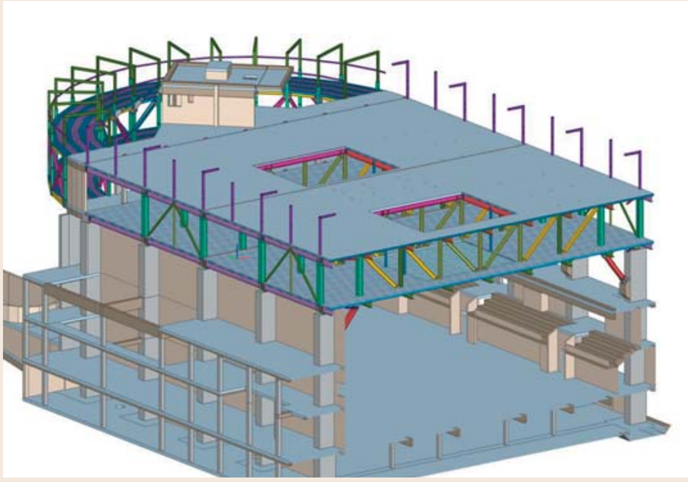


Fig. 6 Stairway tower
Obr. 6 Schodišťová věž

Fig. 7 3D model of the construction
Obr. 7 3D model konstrukce

Fig. 8 Tension rods of the steel structure of the running track
Obr. 8 Táhla ocelové konstrukce běžeckého oválu

Fig. 9 Detail of the connection of the tension rods
Obr. 9 Detail připojení táhel

including the slab reaching 700 mm. The substructure is provided with a full waterproofing system.

The above-ground storeys feature a combination of internal and perimeter columns together with core walls acting as lateral stiffening elements. Floor slabs are predominantly monolithic reinforced concrete, locally provided with column capitals and perimeter stiffening beams.

In the southern end, the roof of smaller halls is formed by prefabricated prestressed girders with a span of 16.5 m and a height of 1.55 m, supporting 150 mm-thick prestressed hollow-core floor slabs. At this level, a setback storey is located, with a steel load-bearing structure.

The central section includes a two-storey multipurpose sports hall at the first-floor level. Stair towers projecting in front of the building on both shorter sides contain monolithic staircases, service shafts, and an elevator shaft – see Fig. 6. The tower walls are 300 mm thick. Surrounding the hall are circulation floor slabs 250 mm thick, edged with perimeter beams of total height 400 mm. Monolithic spectator stands are provided along the longer side at the lowest level.

Project design and construction required meticulous coordination among all parties. Structural drawings

Fig. 10 Night view of the completed building
Obr. 10 Noční pohled na dokončenou stavbu

for both concrete and steel elements were developed in 3D models to minimize clashes and address construction details at the design stage – see Fig. 7. In the reinforcement detailing, particular attention was given to the placement of rebar at nodes connecting steel structures and fabricated steel elements – see Fig. 8, Fig. 9, which was time-intensive but ensured that the complex structure was completed without major issues.

The documentation was developed within the company Recoc Ltd.

Fig. 11 View of the roof of the completed building
Obr. 11 Pohled na střechu dokončené budovy

Fig. 12 Multipurpose sports hall
Obr. 12 Víceúčelová hala

Fig. 13 Gymnastics hall
Obr. 13 Gymnastický sál

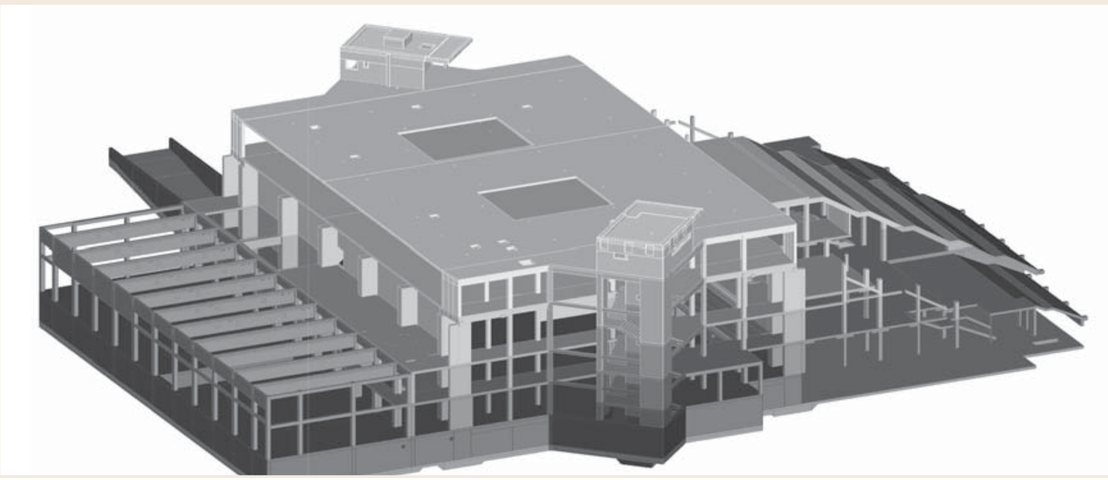


Fig. 14 3D model of the construction
Obr. 14 3D model konstrukce

# TRAM STOP 'VÝSTAVIŠTĚ' IN PRAGUE HOLEŠOVICE
# TRAMVAJOVÁ ZASTÁVKA VÝSTAVIŠTĚ V PRAZE HOLEŠOVICÍCH

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**The tram stop consists of structural members made of printed UHPC. The roof structure is formed by a plane grid of mutually connected curved ribs. The grid is connected to the capitals supported by three columns. The architectural design amplified the results of the analysis of the 'form finding'. The stop´s structure has an economical organic shape given by a pure static action.**

**Tramvajovou zastávku tvoří konstrukční prvky z tištěného UHPC. Střešní konstrukce je tvořena rovinným roštem, který je sestaven ze vzájemně propojených zakřivených žeber. Rošt je uložen na hlavice podepřené třemi sloupy. Architektonický návrh umocnil výsledky analýzy „hledání tvaru". Konstrukce zastávky má ekonomický organický tvar daný čistě statickým působením.**

Fig. 1 Tram stop 'Výstaviště'
Obr. 1 Tramvajová zastávka Výstaviště

## INTRODUCTION

On the edge of Prague's Stromovka park, the first tram stop from printed UHPC concrete was built – see Fig. 1. The roofing superstructure was produced by a special robotic arm of the Czech company So Concrete (www.so-concrete.com) using Robotic Additive Manufacturing technology. The stop includes a Plexiglas roof, a Plexiglas side protective wall, an information panel and a bench, as well as integrated LED lighting. Both the roof and the side wall have polished graphic patterns enhancing the structural solution of the roof. The surface of the concrete floor panels was similarly treated.

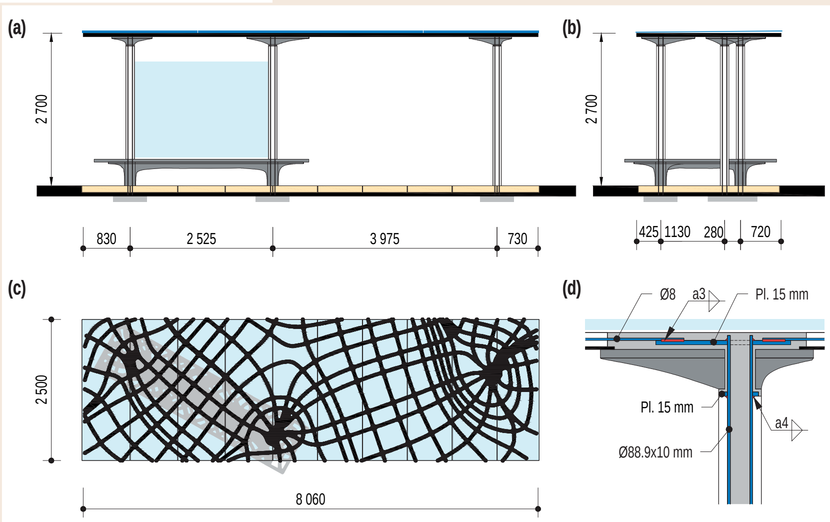


Fig. 2 Structural arrangement: (a) front elevation, (b) side elevation, (c) plan, (d) detail of the connection of the roof with the column,
Obr. 2 Konstrukční uspořádání: (a) čelní pohled, (b) boční pohled, (c) půdorys, (d) detail spojení střechy se sloupem

The architectural design of the tram stop was performed by Federico Diaz, Dimitri Nikitin and Záviš Unzeitig, who amplified the results of the analysis of the 'form finding' carried out by the design office Stráský, Hustý a partneři, Brno.

## STRUCTURAL DESIGN AND STATIC ANALYSIS

The shelter with a floor plan of 8.06 × 2.50 m has a roof structure consisting of a plane grid made of printed UHPC – see Fig. 2. The grid, which is formed by a system of mutually connected planarly curved ribs, is connected to the capitals supported by three columns.

The shape and arrangement of the ribs resulted from a concept developed by the famous Italian engineer Piero Luigi Nervi, who developed 'isostatic' floor structures formed of concrete slabs reinforced with ribs following the course of the slab's principal moments. The procedure for designing the shape and arrangement of the roof's ribs is clear – see Fig. 3, on which the magnitudes and directions of the principal moments of an isotropic slab supported at three points are plotted. From these directions, the numbers and trajectories of the ribs were iteratively derived so that their stresses were as uniform as possible. The ribs were modeled by beam elements that were connected to shell elements describing the function of the column capitals. The structure was designed for Eurocode's loads; the structural members were verified according to the Czech recommendation TP07 ČBS.

The bench, which diagonally connects two columns, is formed by a beam with overhangs. Its span is 2.90 m. It has a solid cross section; however it is lightened by openings at its edges in the central, supporting part.

The columns consist of a steel pipe 88.9x10 mm filled with UHPC to increase their rigidity. The steel pipes are covered by white UHPC. The pipes are welded to circular foot plates anchored by six bolts. The columns support prefabricated capitals supporting the ribs of the roof structure from UHPC C110/130. During the assembly of the structure, the columns were protected by a PVC pipe, which also served as a formwork for casting of their cover.

All ribs have a constant depth and width of 60 mm – see Fig. 4. At supports, the ribs are composite with the capitals of depth from 40 to 140 mm. In order to reinforce the ribs, they were created step by step – see Fig. 5. First, a U-section was created, into which the reinforcement was inserted and with the help of the special robotic arm the free space was concreted. Even the actual U cross-section was created incrementally. The side webs were printed using Robotic Additive Manufacturing technology,

Fig. 3 Slab principal movement trajectories and derived rib geometry
Obr. 3 Deskové hlavní momenty a odvozená geometrie žeber

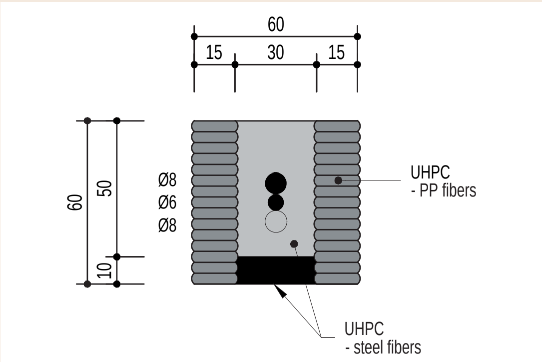


Fig. 4 Rib cross section
Obr. 4 Příčný řez žebrem

Fig. 5 Reinforcing steel inserted into the U section
Obr. 5 Betonářská výztuž vložená do U průřezu

between which the bottom flange was subsequently cast using the same technology. Above the heads, the ribs were formed only by the side walls, the space between them was filled after their erection.

## TRAM STOP CONSTRUCTION

During construction, the roof structure was divided into six assembly parts. In the place of the joints, the filling in the profiles was omitted and the reinforcing bars were extended. After fitting the individual parts, the protruding rebars were welded and the space between the side walls of the ribs was filled with UHPC.

The capitals were concreted into formwork made of extruded polystyrene using the same technology; the shape of the heads was digitally milled in polystyrene. The roof ribs are connected with the capitals with epoxy resin and shear studs anchored in the capitals and in the rib in-fill concrete. During construction, the capitals were placed on steel rings welded to the columns' steel pipes. At the level of the rib bottom fibers steel plates were put on the steel pipes and welded to them. To guarantee the transfer of support bending moments, the protruding rebars of the ribs were welded with these steel plates. Subsequently, the space between the rib walls was filled with UHPC.

The bench is also composed of two interconnected elements – the lower, supporting part and the sitting slab part. The lower part was concreted into an extruded polystyrene formwork. The upper part was created in two steps. First, the outer contour and contours of the bench openings were printed on the base plate. Then concrete reinforcement was inserted between the holes and the space between the holes was filled with UHPC. Subsequently, the two parts were connected by epoxy resin and studs.

The tram stop was built in the fall of 2022. When the footings were completed, the composite columns were erected and the bench was positioned. This was followed by welding the columns steel rings and erection of the capitals. The members of the roof structure were then placed on a light stationary scaffolding and already erected capitals. After welding of the joint rebars and then welding of the rebars to the column connecting plate, the space between the walls of the ribs was concreted. This was followed by the removal of the scaffolding, the fitting of the plexiglass roof, LED lighting and finishing work – see Fig. 6.

## CONCLUSIONS

The stop has an economical organic shape given by a pure static action. It is elegant, interesting and unusual. Natural principles have created a structure that is in harmony with the surroundings, which does not overwhelm, but rather complements them. The tram stop construction is the result of an exemplary collaboration of artists, designers and technologists who respect each other.

The tram stop´s architectural design, production and construction were performed by the firm So Concrete a.s., Praha. The form finding and structural design were carried out by the design firm Stráský, Hustý a partneři, Brno.

Fig. 6 Roof structure
Obr. 6 Konstrukce střechy

# STRUCTURAL LIGHTWEIGHT CONCRETE LC 35/38 FOR PRECAST GRANDSTAND ELEMENTS OF THE BRNO MULTIPURPOSE ARENA

## LEHKÝ KONSTRUKČNÍ BETON LC 35/38 PRO PREFABRIKOVANÉ TRIBUNOVÉ DÍLCE MULTIFUNKČNÍ ARÉNY BRNO

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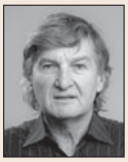
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**This article describes the production and installation of precast grandstand elements made of lightweight structural concrete for the Brno Multipurpose Arena. To reduce self-weight, LC35/38 D1.8 concrete was used, with a required static modulus of elasticity Ec ≈ 22 GPa and consistency of F5; approximately 1804 $m^3$ of lightweight concrete was produced. The article presents a verified production procedure (controlled pre--wetting, extended mixing, and dosing of Liapor lightweight aggregate) and verification of physical and mechanical properties, including static and dynamic testing of the finished elements.**

**Příspěvek popisuje realizaci prefabrikovaných tribunových dílců z lehkého konstrukčního betonu pro Multifunkční arénu Brno. Pro snížení vlastní hmotnosti byl použit beton LC35/38, D1,8 s požadavkem na Ec ≈ 22 GPa a konzistenci F5; vyrobeno bylo cca 1804 $m^3$ lehkého betonu. Uveden je ověřený výrobní postup (řízené předmáčení, prodloužené míchání, dávkování lehkého kameniva Liapor) a ověření fyzikálně mechanických vlastností včetně statických a dynamických zkoušek hotových dílců.**

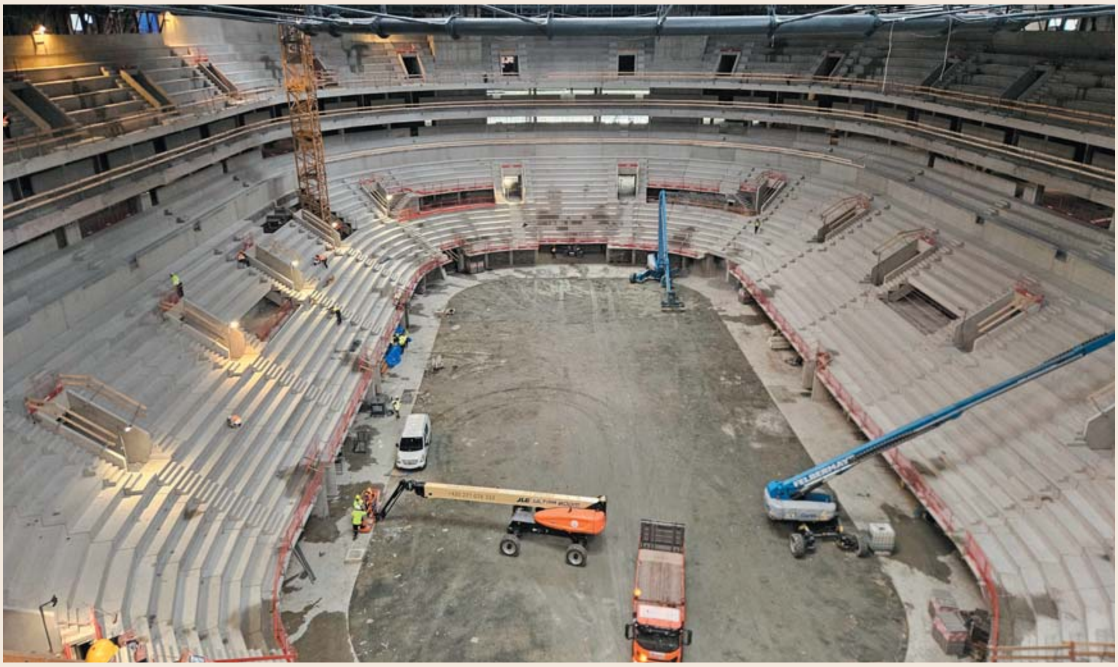


Fig. 1 Brno Multipurpose Arena: installation of precast grandstand elements made of structural lightweight concrete (LC35/38 D1.8) during construction
Obr. 1 Multifunkční aréna Brno: montáž prefabrikovaných tribunových prvků z lehkého betonu (LC35/38 D1,8) v průběhu výstavby

Structural lightweight concrete (LC) with ceramic porous aggregate based on expanded clay Liapor offers an efficient solution for precast grandstand elements, reducing self-weight while maintaining the required load-bearing capacity and stiffness. In practice, this enables rationalisation of the load-bearing structure, reduced foundation demands and faster erection through element standardisation. Critical parameters for stable production include moisture control and water absorption of the lightweight aggregate, workability stability over time, and the selected production concept (pre-wetting / wetting water, accurate and stable aggregate dosing, and mixing time).

The use of LC in grandstands has already been verified in several Czech projects. For the Eden Stadium (Prague), grandstand units were produced from LC35/38 D1.8, as lightweight self-compacting concrete (LWSCC) with requirements for fair-faced concrete quality. The project confirmed the benefit of reduced unit weight, but also the sensitivity of production to consistent workability and dosing accuracy under varying Liapor moisture conditions. At the Karlovy Vary multipurpose hall, LC was applied to selected seating elements and corridor walls; grandstand components such as stair treads and bench elements were produced from LC25/28 D1.8. The experience confirmed the importance of element standardisation and an appropriate structural concept.

The Brno Multipurpose Arena design was prepared by Arch.Design and A PLUS (designer / lead designer). The general contractor is HOCHTIEF CZ a.s.; the precast grandstand units were manufactured by Prefa Brno; the Liapor aggregate was supplied by Liapor s.r.o.; and site erection was carried out by AZ Prezip a.s. The grandstand precast units were designed using LC35/38 D1.8, with a minimum required static modulus of elasticity Ec = 22 GPa. The required workability of fresh concrete was specified as flow class F5. The scope comprised 987 L-shaped bench beams and 201 special elements, with a total of 1,804 $m^3$ of LC with Liapor aggregate (Fig. 1, 2).

The constituent materials were CEM I 42.5 R cement, Liapor aggregate 2/10 mm (bulk density approx. 650 $kg/m^3$), natural aggregate 0/4 and 4/8 mm, high-temperature fly ash and a PCE superplasticizer. Continuous pre-wetting of the lightweight aggregate was not applied. Workability stability was achieved by controlled pre-wetting using approximately 40% of the mixing water and by extended mixing (approx. 4 min), of

Lehký konstrukční beton (dále LC) s keramickým pórovitým kamenivem na bázi expandovaného jílu Liapor představuje pro prefabrikaci tribunových dílců účinnou cestu ke snížení vlastní hmotnosti při zachování požadované únosnosti a tuhosti. V praxi to umožňuje racionalizovat nosné konstrukce, omezit nároky na založení a zrychlit montáž díky unifikaci prvků. Kritickými parametry pro stabilní výrobu jsou řízení vlhkosti a nasákavosti lehkého kameniva, stabilita konzistence v čase a volba výrobního postupu (předmáčení / smáčecí voda, přesné a stabilní dávkování lehkého kameniva, doba míchání).

Použití LC pro tribuny bylo v ČR ověřeno již na více realizacích. Pro stadion Eden (Praha) byly vyráběny tribunové prvky z LC35/38, D1,8 z lehkého samozhutnitelného betonu (LWSCC) s požadavky na kvalitu pohledového betonu. Realizace potvrdila přínos snížení hmotnosti prefabrikátů, ale zároveň i citlivost a náročnost výrobních procesů na udržení stabilní konzistence a přesnost dávkování při různé vlhkosti kameniva Liapor. V multifunkční hale v Karlových Varech bylo použití LC aplikováno na vybrané prvky hlediště a stěny koridorů; tribunové prvky, např. schodišťové stupně a lavičkové prvky, byly realizovány z lehkého betonu LC25/28, D1,8; zkušenost potvrdila význam unifikace prvků a volby vhodného konstrukčního řešení.

Projekt Multifunkční arény Brno zpracovala společnost Arch.Design a A PLUS (autor projektu a generální projektant). Generálním zhotovitelem stavby je HOCHTIEF CZ a.s., prefabrikované tribunové prvky vyrobila Prefa Brno a lehké keramické kamenivo Liapor dodala společnost Liapor s.r.o., montáž na stavbě prováděla firma AZ Prezip a.s. Tribunové prefabrikáty byly navrženy z lehkého betonu třídy LC35/38, D1,8 s minimální požadovanou hodnotou statického modulu pružnosti Ec = 22 GPa. Požadovaná konzistence čerstvého betonu byla stanovena v třídě F5. V rámci dodávky bylo vyrobeno 987 lavičkových nosníků tvaru L a 201 kusů atypických prvků ramen v celkovém objemu 1804 m³ lehkého betonu s kamenivem Liapor (Obr. 1, 2).

Použité vstupní suroviny byly cement CEM I 42,5 R, lehké kamenivo Liapor frakce 2/10 mm, se sypnou hmotností 650 kg/m³, přírodní kamenivo frakcí 0/4 a 4/8 mm, vysokoteplotní úletový popílek a superplastifikátor na bázi polykarboxylátů. Na lehkém kamenivu nebylo předem aplikováno kontinuální smáčení. Stabilita konzistence byla zajištěna řízeným předmáčením přibližně 40 % záměsové vody a prodlouženou dobou míchání (cca 4 min), z čehož cca 3,5 min připadalo na předmáčení lehkého kameniva.

Pro dosažení přesného objemu čerstvého betonu a stability výroby bylo nutné upravit způsob dávkování lehkého kameniva. Standardně používané hmotnostní dávkování je pro lehké kamenivo nevhodné, proto bylo nahrazeno dávkováním objemovým. Běžné kamenivo a ostatní suroviny byly dávkovány obvyklým způsobem.

Ověření fyzikálně mechanických vlastností bylo provedeno kombinací laboratorních zkoušek zkušebních těles a testováním statické únosnosti na vybraných prefabrikátech. Pevnost v tlaku byla stanovena na zkušebních krychlích o hraně 150 mm v stáří 7 a 28 dnů, současně byla sledována objemová hmotnost betonu v nevysušeném stavu a vysušeném stavu (Tab. 1–2). Naměřené hodnoty potvrdily dosažení požadované pevnostní třídy LC35/38 při odpovídající objemové hmotnosti. Požadavek na tuhost byl ověřen stanovením statického modulu pružnosti Ec na vývrtech o průměru 150 mm z hotových prefabrikovaných prvků (Tab. 3). Bylo prokázáno splnění minimální hodnoty Ec = 22 GPa přímo na reprezentativním prvku.

Pro posouzení reprodukovatelnosti a stability výroby byly vyhodnoceny provozní kontrolní zkoušky betonu LC35/38, D1,8 na pravidelně odebíraných zkušebních tělesech. Během výroby bylo provedeno celkem 95 měření objemové hmotnosti (nevysušený stav) a pevnosti v tlaku a 71 stanovení objemové hmotnosti ve vysušeném stavu. Cílem bylo zhodnotit běžnou variabilitu výroby a zachytit případné odchylky související zejména s vlhkostí lehkého kameniva, konzistencí směsi a nastavením dávkování a míchání. Souhrnné výsledky jsou uvedeny v Tab. 4. Kromě průměru a směrodatné odchylky je doplněn koeficient variability, který umožňuje rychle posoudit stabilitu výroby, a percentily P05–P95, vymezující rozmezí, ve kterém leží 90 % provozních hodnot. Časový

Fig. 2 Manufacturing of a precast grandstand unit: casting into the mould with installed reinforcement (LC35/38 D1.8; flow class F5)
Obr. 2 Výroba prefabrikovaného tribunového prvku: betonáž do formy s osazenou výztuží (LC35/38, D1,8; konzistence F5)

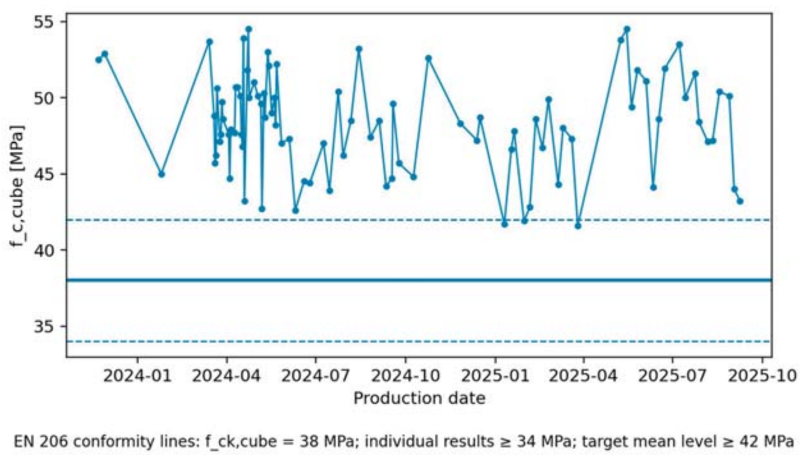



Fig. 3 LC35/38 D1.8 – compressive strength development over time with EN 206 conformity lines
Obr. 3 LC35/38 D1,8 – vývoj pevnosti v tlaku v čase s vyznačením mezí shody dle EN 206

which about 3.5 min was dedicated to pre-wetting the lightweight aggregate.

To achieve an accurate fresh-concrete yield and stable production, the dosing method for the lightweight aggregate had to be modified. The standard mass-based dosing was found unsuitable for lightweight aggregate and was therefore replaced by volumetric dosing. Normal-weight aggregate and the other constituents were dosed in the conventional manner.

Verification of physical and mechanical properties combined laboratory testing on specimens with static load testing on selected precast units. Compressive strength was determined on 150 mm cubes at 7 and 28 days; the concrete density was measured in both non-dried and dried states (Tables 1–2). The measured values confirmed achievement of the required strength class LC 35/38 at the corresponding density. The stiffness requirement was verified by determining the static modulus of elasticity Ec on 150 mm diameter cores taken from finished precast elements (Table 3). Compliance with the minimum value Ec = 22 GPa was demonstrated on a representative element.

To assess reproducibility and production stability, routine production-control test results for LC35/38 D1.8, were evaluated from regularly taken specimens. A total of 95 measurements of density (non-dried state) and compressive strength, and 71 determinations of density in the dried state, were available. The objective was to evaluate normal production variability and to capture deviations related primarily to lightweight aggregate moisture, mixture workability and the settings of dosing and mixing. Summary results are given in Table 4.

Tab. 1 Density and compressive strength after 7 days of curing (non-dried state)

| DENSITY [KG/M$^3$] | COMPRESSIVE STRENGTH $F_{C,CUBE}$ [MPA] |
|---|---|
| 1830 | 47.1 |
| 1840 | 46.9 |
| 1840 | 47.0 |

Tab. 2 Density and compressive strength after 28 days of curing (non-dried state)

| DENSITY [KG/M$^3$] | COMPRESSIVE STRENGTH $F_{C,CUBE}$ [MPA] |
|---|---|
| 1790 | 54.3 |
| 1780 | 55.2 |
| 1790 | 54.9 |
| 1790 | 54.8 |

Tab. 3 Static modulus of elasticity Ec – cores from finished elements

| CORE ID | D [MM] | L [MM] | D [KG/M$^3$] | Ec [MPA] |
|---|---|---|---|---|
| 22.11.-1 | 152.7 | 143.0 | 1820 | 25500 |
| 22.11.-2 | 153.9 | 142.8 | 1780 | 23900 |
| 22.11.-3 | 152.7 | 143.2 | 1800 | 23900 |

In addition to the mean and standard deviation, the coefficient of variation is included for a quick assessment of production stability, together with percentiles P05–P95 defining the range containing 90% of production values. The time evolution of compressive strength is shown in Fig. 3 with EN 206 conformity lines.

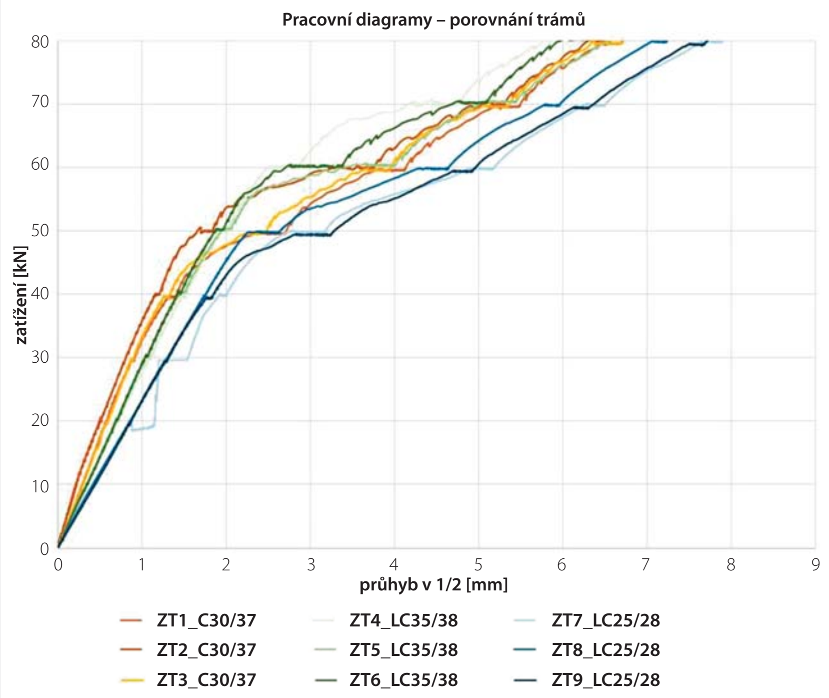
To verify the behaviour of lightweight concrete precast elements, experimental tests were carried out at the AdMaS research centre, Brno University of Technology, Faculty of Civil Engineering, under the supervision of the project's chief structural engineer Ing. Vladimír Dibelka, Ph.D. (PBK Chrudim a.s.). The programme addressed static resistance, deformation levels under the expected actions, and response under impact-type dynamic loading. The tests were designed as a comparison between a conventional reference concrete C30/37 and two lightweight concrete variants, LC25/28 D1.6 and LC35/38 D1.8. Static loading tests were performed on beams 180 × 400 × 4000 mm in a test frame. Results were evaluated using load–deflection diagrams (kN vs. mid-span deflection, mm). Three specimens were tested for each mix to compare scatter. The diagrams show that LC35/38 D1.8 exhibits a significantly more favourable response than LC25/28 D1.6 (higher load level at comparable deflection). Compared with C30/37, its behaviour is comparable even at higher deformation levels, confirming suitability for the required service actions.

In addition, an impact-type dynamic loading simulation was performed on a full-scale grandstand element (8580 × 510 × 940 mm) in a laboratory test frame using a controlled hydraulic loading system and a contact impact body. The loading history, deformations and structural response to dynamic impacts were recorded by instrumentation; documentation and evaluation are provided in the test photograph and the accompanying diagram (see Fig. 4 and 5). The above information is based on materials provided by Ing. Vladimír Dibelka, Ph.D.

The presented Czech applications, and in particular the Brno Multipurpose Arena, confirm that structural lightweight concrete LC35/38 D1.8, provides a technically and operationally reliable solution for precast grandstand elements. Routine production-control results and their statistical evaluation demonstrate stable production quality in terms of achieving the required density, compressive strength and static modulus of elasticity. A key prerequisite for reproducibility is monitoring and control of lightweight aggregate moisture and stabilisation of mixture rheology through controlled pre-wetting, the use of an effective superplasticizer and adequate mixing time. Experimental verification on finished elements, including static loading tests and dynamic impact loading, demonstrated stable element response and supported the suitability of the selected material and production concept for elements subjected to long-term service loading.

Tab. 4 Summary statistics of production results (LC35/38 D1.8)

| PARAMETER | N | MEAN | STANDARD DEVIATION | COEFFICIENT OF VARIATION (%) | P05 | P95 |
|---|---|---|---|---|---|---|
| DENSITY (NON-DRIED STATE) [KG/M$^3$] | 95 | 1793.68 | 42.73 | 2.38 | 1734 | 1843 |
| DENSITY (DRIED STATE) [KG/M$^3$] | 71 | 1630.99 | 48.64 | 2.98 | 1570 | 1680 |
| COMPRESSIVE STRENGTH $F_{C,CUBE}$ [MPA] | 95 | 47.86 | 3.67 | 7.67 | 41.24 | 53.56 |




průběh pevnosti v tlaku je znázorněn v Obr. 3 s vyznačením mezí shody dle EN 206.

Pro ověření chování prefabrikovaných prvků z lehkého betonu byly ve výzkumném centru AdMaS Brno, Fakulty stavební Vysokého technického učení v Brně, pod vedením hlavního statika projektu Ing. Vladimíra Dibelky, Ph.D. (firma PBK Chrudim), provedeny experimentální zkoušky zaměřené na statickou únosnost, hodnoty deformací při předpokládaném zatížení a na chování prvků při rázovém dynamickém zatížení. Testování bylo koncipováno jako porovnávací mezi tradičním referenčním betonem C30/37 a dvěma variantami lehkého betonu LC25/28 D1,6 a LC35/38 D1,8. Statické zatěžovací zkoušky byly provedeny na trámcích 180 × 400 × 4000 mm na rámových zkušebních lisech. Výsledky byly vyhodnoceny pomocí pracovních diagramů zatížení–průhyb (kN vs. průhyb v 1/2 rozpětí, mm). Pro každou směs byly testovány tři prefabrikáty pro porovnání rozptylu chování. Z pracovních diagramů vyplývá, že LC35/38 D1,8 vykazuje výrazně příznivější odezvu než LC25/28 D1,6 (vyšší úroveň zatížení při srovnatelném průhybu). Ve srovnání s tradičním betonem C30/37 je jeho chování srovnatelné i v oblasti vyšších deformací, což potvrzuje vhodnost volby na požadované provozní zatížení.

Současně byla provedena simulace rázového dynamického namáhání na full-scale tribunovém prvku o rozměrech 8580 × 510 × 940 mm v laboratorním zkušebním rámu s řízeným hydraulickým zatěžovacím systémem a kontaktním nárazovým tělesem. Průběh zatěžování, deformací a odezva konstrukce na dynamické rázy byly snímány měřicí aparaturou; dokumentace zkoušky a vyhodnocení jsou uvedeny na fotografii ze zkoušky a v připojeném grafu (viz Obr. 4 a 5). Výše uvedené informace jsou převzaty ze zdrojů Ing. Vladimíra Dibelky, Ph.D.

Předložené realizace v České republice a zejména realizace Multifunkční arény Brno potvrzují, že lehký konstrukční beton LC35/38, D1,8 představuje pro prefabrikaci tribunových prvků technicky i provozně spolehlivé řešení. Výsledky provedených provozních kontrolních zkoušek a jejich statistické vyhodnocení dokládají stabilní kvalitu výroby z hlediska dosažení požadované objemové hmotnosti, pevnosti v tlaku a statického modulu pružnosti. Klíčovým předpokladem reprodukovatelnosti je monitorování a řízení vlhkosti lehkého kameniva a stabilizace reologie směsi prostřednictvím řízeného předmáčení s využitím účinné superplastifikační přísady a adekvátní doby míchání. Experimentální ověření na hotových prvcích včetně statických zatěžovacích zkoušek a dynamického namáhání prokázalo stabilní odezvu prvků a podpořilo vhodnost zvoleného materiálového i technologického konceptu pro prvky vystavené dlouhodobému provoznímu namáhání.

Fig. 4 AdMaS Brno test set-up: full-scale grandstand element during loading test (measurement of deformations/deflection)
Obr. 4 Zkušební uspořádání v AdMaS Brno: full-scale tribunový prvek z lehkého betonu během zatěžovací zkoušky (měření deformací / průhybu)

Fig. 5 Load–deflection diagrams: comparison of beams made of C30/37, LC25/28 D1.6 and LC35/38 D1.8. Source: Ing. Vladimír Dibelka, Ph.D. (PBK Chrudim)
Obr. 5 Pracovní diagramy zatížení–průhyb: porovnání trámů z betonu C30/37, LC25/28 D1,6 a LC35/38 D1,8 (zatížení [kN] vs. průhyb v 1/2 rozpětí [mm]). Zdroj: Ing. Vladimír Dibelka, Ph.D. (PBK Chrudim)

Tab. 1 Objemová hmotnost a pevnost v tlaku po 7 dnech zrání (nevysušený stav)

| OBJEMOVÁ HMOTNOST [KG/M$^3$] | PEVNOST V TLAKU $F_{C,CUBE}$ [MPA] |
|---|---|
| 1830 | 47,1 |
| 1840 | 46,9 |
| 1840 | 47,0 |

Tab. 2 Objemová hmotnost a pevnost v tlaku po 28 dnech zrání (nevysušený stav)

| OBJEMOVÁ HMOTNOST [KG/M$^3$] | PEVNOST V TLAKU $F_{C,CUBE}$ [MPA] |
|---|---|
| 1790 | 54,3 |
| 1780 | 55,2 |
| 1790 | 54,9 |
| 1790 | 54,8 |

Tab. 3 Statický modul pružnosti Ec – vývrty z hotových prvků

| OZNAČ. VÝVRTU | D [MM] | L [MM] | D [KG/M$^3$] | Ec [MPA] |
|---|---|---|---|---|
| 22.11.-1 | 152,7 | 143,0 | 1820 | 25 500 |
| 22.11.-2 | 153,9 | 142,8 | 1780 | 23 900 |
| 22.11.-3 | 152,7 | 143,2 | 1800 | 23 900 |

Tab. 4 Souhrnná statistika provozních výsledků (LC35/38 D1,8)

| PARAMETR | N | PRŮMĚR | SMĚR.ODCH. | KOEFICIENT VARIABILITY (%) | P05 | P95 |
|---|---|---|---|---|---|---|
| OBJEMOVÁ HMOTNOST (NEVYSUŠENÝ STAV) [KG/M$^3$] | 95 | 1793,68 | 42,73 | 2,38 | 1734 | 1843 |
| OBJEMOVÁ HMOTNOST (VYSUŠENÝ STAV) [KG/M$^3$] | 71 | 1630,99 | 48,64 | 2,98 | 1570 | 1680 |
| PEVNOST V TLAKU $F_{C,CUBE}$ [MPA] | 95 | 47,86 | 3,67 | 7,67 | 41,24 | 53,56 |

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