STRESS-RIBBON PEDESTRIAN BRIDGES SUPPORTED OR SUSPENDED ON ARCHES

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Abstract: *A new structural system that combines arches with the stress-ribbon is described in terms of the architectural and structural solution, static and dynamic analyses, and the process of construction. The advantage of this structural system is demonstrated on several structures built in the Czech Republic and in Oregon, USA.*

1. INTRODUCTION

Stress-ribbon bridges consist of very slender concrete deck segments placed over bearing cables in the shape of a catenary. Prestressing the deck segments stiffens the structure, providing stability to the cables. These bridges are characterized by successive and complementary smooth curves. The curves blend into the environment and the curved shape, the most simple and basic of structural solutions, clearly articulates the flow of internal forces. The main advantage of these structures is their minimal environmental impact because they use very little material and can be erected without falsework or shoring, which could disturb the natural environment. Because there are no bearings or expansion joints, the bridges require only minimal long-term maintenance. The bridges built in the Czech Republic and in the USA were discussed in a paper presented at the International Bridge Conference in Pittsburgh in 1999 [1]. Problems connected with the design and construction of the stress-ribbon structures and a survey of such bridges built all over the world is presented in [2].

A disadvantage of the classical stress-ribbon type structure is the need to resist very large horizontal forces at the abutments, which determines the economy of that solution in many cases. For that reason, a new system that combines an arch with the stress-ribbon has been developed. The stress-ribbon is supported or is suspended on an arch. The structures form a self-anchoring system where the horizontal force from the stress-ribbon is transferred by inclined concrete struts to the foundation, where it is balanced against the horizontal component of the arch.

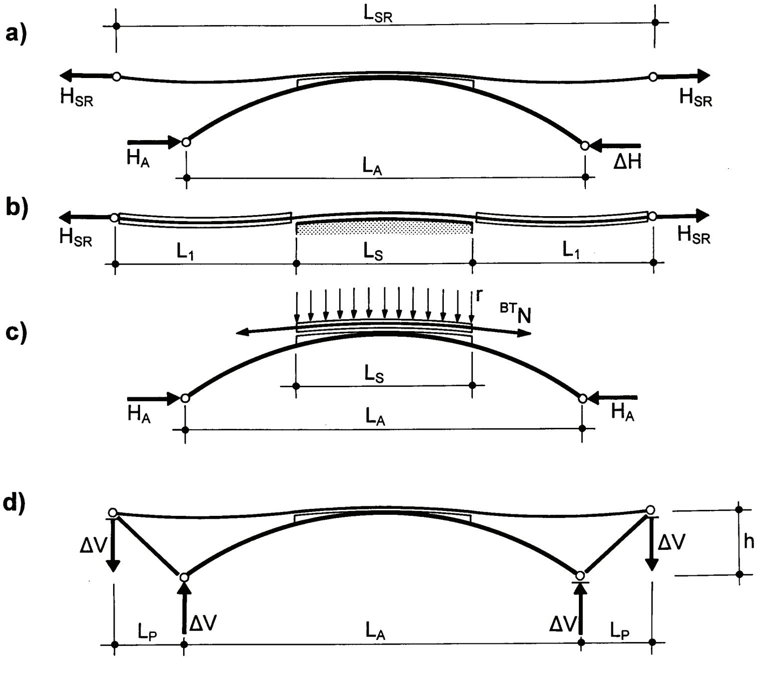


Figure 1: Stress ribbon supported by arch

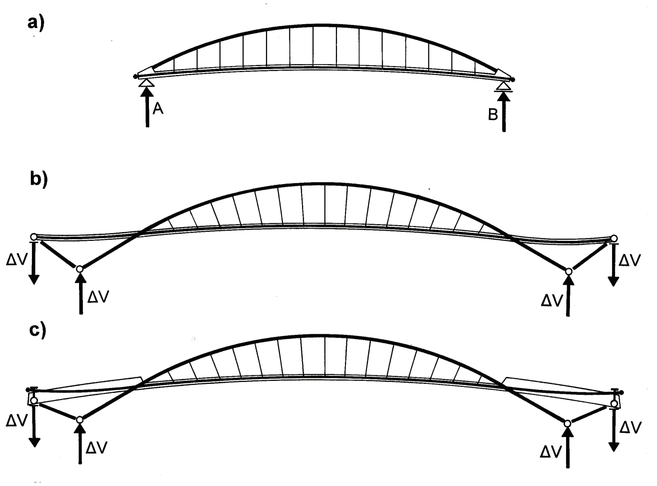


Figure 2: Stress ribbon suspended on arch

A stress-ribbon structure supported by the arch was carefully analyzed a tested at the Brno University of Technology, and the structural system that was developed was then applied to the design of four bridges that have been built in the Czech Republic. A stress-ribbon structure suspended on two inclined arches was applied in the design of the 92-meter-long McLoughlin Bridge built in Portland, Oregon.

2. STRUCTURAL SYSTEM

The development of the self-anchored stress-ribbon structure supported by an arch is evident from Fig. 1. It is clear that the intermediate support of a multi-span stress-ribbon can also have the shape of an arch (Figure 1a). The arch serves as a saddle from which the stress-ribbon spans can rise during post-tensioning and during temperature drop, and where the center "band" can rest during a temperature rise.

In the initial stage, the stress-ribbon behaves as a two-span cable supported by the saddle that is fixed to the end abutments (Figure 1b). The arch is loaded by its self weight, the weight of the saddle segments and the radial forces caused by the bearing tendons (Figure 1c). After post-tensioning the stress-ribbon with the prestressing tendons, the stress-ribbon and arch behave as one structure.

The shape and initial stresses in the stress-ribbon and in the arch can be chosen such that the horizontal forces in the stress-ribbon H_{SR} and in the arch H_A are the same. It is then possible to connect the stress-ribbon and arch footings with inclined compression struts that balance the horizontal forces. The moment created by horizontal forces $H_{SR}.h$ is then resisted by the $\Delta V.L_P$. In this way a self-anchored system with only vertical reactions is created (Figure 1d).

It is also obvious that the stress-ribbon can be suspended from the arch. It is then possible to develop several self-anchored systems. Figure 2 presents some concepts using such systems. Figure 2a shows an arch fixed at the anchor blocks of the slender prestressed concrete deck. The arch is loaded not only by its self weight and that of the stress-ribbon, but also with the radial forces of the prestressing tendons. Figure 2b shows a structure that has a similar static behavior as the structure presented in Figure 1d. To reduce the tension force at the stress-ribbon anchor blocks, it is possible to connect the stress-ribbon and arch footings by inclined compression struts that fully or partially balance the stress-ribbon horizontal forces. Figure 2c shows a similar structure in which the slender prestressed concrete band has increased bending stiffness in the portion of the structure not suspended from the arch.

3. MODEL TESTS

The authors believe that a structural system made up of a stress-ribbon supported by an arch increases the potential application of stress-ribbon structures. Several analyses were under taken to verify this. The structures were checked not only with detailed static and dynamic analysis, but also on static and full aeroelastic models. The tests verified the design assumptions and behavior of the structure under wind loading that determined the ultimate capacity of the full system.

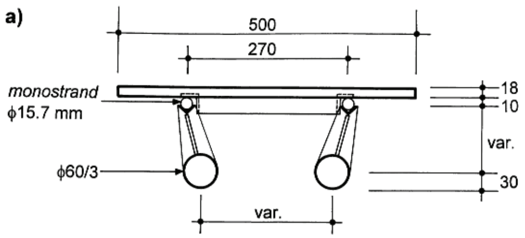


Figure 3: Static model – cross section

Figure 4: Static model

The model tests were done for a proposed pedestrian bridge across the Radbuza River in Plzen, Czech Republic. This structure was designed to combine a steel pipe arch having a span length of 77 m and the deck assembled of precast segments. The static physical model was done in a 1:10 scale. The shape is shown in Figures 3 and 4. Dimensions of the model and cross-section, loads, and prestressing forces were determined according to rules of

Figure 5: Static model – ultimate load

Figure 6: Wind tunnel test

similarity. The stress-ribbon was assembled with precast segments of 18 mm depth and the cast-in-place haunches were anchored in anchor blocks made with steel channel sections. The arch consisted of two steel pipes, and the end struts consisted of two steel boxes fabricated from channel sections. The saddle was made by two steel angles supported on longitudinal plates strengthened with vertical stiffeners. The footings common to the arch and inclined struts were assembled from steel boxes fabricated with two channel sections. They were supported by steel columns consisting of two I sections. The end ties consisted of four rectangular tubes. The steel columns and the ties were supported by a longitudinal steel beam that was anchored to the test floor.

The precast segments were made from micro-concrete of 50 MPa characteristic strength. The stress-ribbon was supported and post-tensioned by 2 monostrands situated outside the section. Their position was determined by two angles embedded in the segments. The loads, determined according to the rules of similarity, consisted of steel circular bars suspended on the transverse diaphragms and on the arch. The number of bars was modified according to desired load. The erection of the model corresponded to the erection of the actual structure. After the assembly of the arch and end struts, the monostrands were stranded.

Then the segments were placed on the monostrands and the loads were applied. Next, the joints between the segments and the haunches were cast. When the concrete reached the minimum prescribed strength, the monostrands were tensioned to the design force. Before erection of the segments, strain gauges were attached to the steel members and the initial stresses in the structure were measured. The strain gauges were attached at critical points of the stress-ribbon before casting of the joints. During erection of the segments, casting of the joints and post-tensioning of the structure, the deformations of the arch and the deck where carefully monitored and the forces in the monostrands were measured by dynamometers placed at their anchors (Figure 4).

The model was tested for the 5 positions of live load. At the end of the tests the ultimate capacity of the overall structure was determined. It was clear that the capacity of the structure was not given by the capacity of the stress-ribbon since, after the opening of the joints, the whole load would be resisted by the tension capacity of the monostrands. Since the capacity of the structure would be given by the buckling strength of the arch, the model was tested for a load situated on one side of the structure (Figure 5). The structure was tested for an increased dead load (1.3 G) applied using the additional suspended steel rods, and then for a gradually increasing live load P applied with force control using a hydraulic jack reacting against a loading frame.

The structure failed by buckling of the arch at a load 1.87 times higher than the required ultimate load Qu = 1.3 G + 2.2 P. The stress-ribbon itself was damaged only locally by cracks that closed after the overloads were removed. The structure also proved to be very stiff in the transverse direction. The buckling capacity of the structure was also calculated with a nonlinear analysis in which the structure was analyzed for a gradually increasing load. The failure of the structure was taken at the point when the analytic solution did not converge. Analysis was performed for the arch with and without fabrication imperfections. The imperfections were introduced as a sinus-shaped curve with nodes at arch springs and at the crown. Maximum agreement between the analytical solution and the model was achieved for the structure with a maximum value of imperfection of 10 mm. This value is very close to the fabrication tolerance. The test has proven that the analytical model can accurately describe the static function of the structure both at service and at ultimate load.

The dynamic behavior of the proposed structure was also verified by dynamic and wind tunnel testing (Figure 6) performed by Professor Miros Pirner at the Institute of Theoretical and Applied Mechanics, Academy of Sciences of the Czech Republic. The critical wind speed of the model was 11.07 m/s; the corresponding critical wind speed of bridge is 90.03 m/s.

4. PEDESTRIAN BRIDGE ACROSS THE EXPRESSWAY R35

The bridge crosses expressway R3508 that was built near a city of Olomouc, Czech Republic. The bridge is formed by a stress-ribbon of two spans that is supported by an arch (Figure 7). The stress-ribbon of the length of 76.50 m is assembled of precast segments 3.00 m long supported and prestressed by two external tendons (Figures 8 and 9).

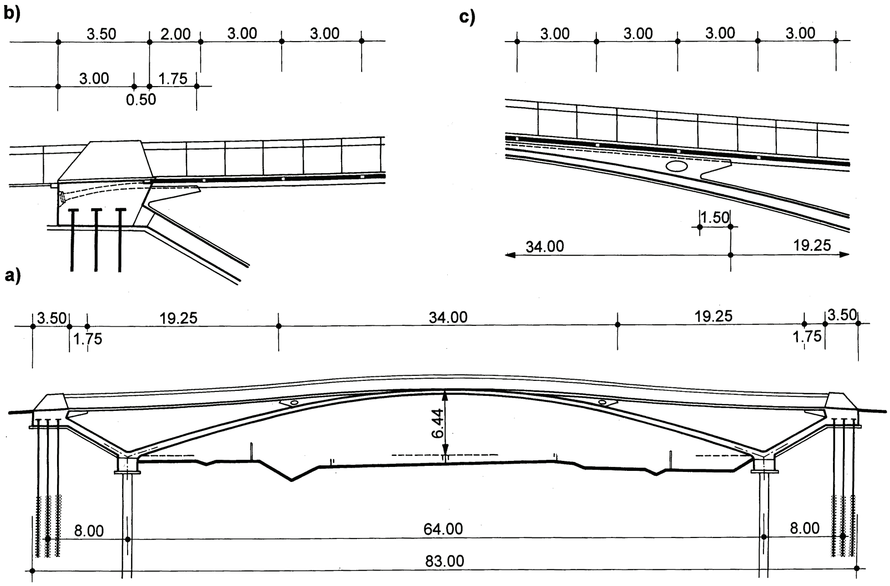


Figure 7: a) bridge elevation, b) partial elevation at the anchor block's saddle, c) partial elevation at the arch's saddle

The precast deck segments and precast end struts consist of high-strength concrete of a characteristic strength of 80 MPa. The cast-in-place arch consists of high-strength concrete of a characteristic strength of 70 MPa. The external cables are formed by two bundles of 31-0.6" diameter monostrands grouted inside stainless steel pipes. They are anchored at the end abutments and are deviated on saddles formed by the arch crown and short spandrel walls. The steel pipes are connected to the deck segments by bolts lo located in the joints between the segments. At the abutments, the tendons are supported by short saddles formed by cantilevers that protrude from the anchor blocks. The stress-ribbon and arch are mutually connected at the central band of the bridge. The arch footings are founded on drilled shafts and the anchor blocks on micro-piles. Due to unexpected local conditions the micro-piles failed. Therefore their function was substituted by an additionally cast ballast.

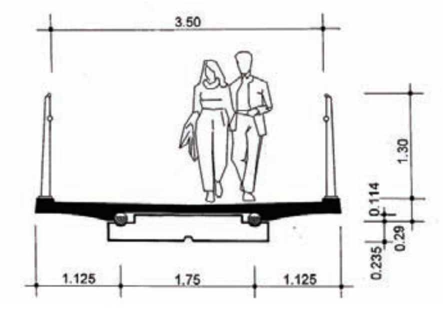


Figure 8: Cross section

Figure 9: Precast segments on external cables

The bridge was erected in several steps. After the piles were placed, the end struts were erected and the arch footings and end anchor blocks were then cast. The arch was cast in formwork supported by light scaffolding. When the concrete of the arch had sufficient strength, the external cables were assembled and tensioned. Then the precast segments were erected (Figure 10). After the forces in the external cables were adjusted, the joints between the segments were cast and subsequently the external tendons were tensioned up to the design stress. Since the cables are curved, the radia forces loaded the stress-ribbon and in this way the deck was prestressed.
The structural solution was developed on the basis of the described tests and a very detailed static and dynamic analysis. A great attention was also devoted to the analysis of the buckling of the arch. The stability analysis has proved that the structure has a sufficient margin of safety. The first bending frequency, $f_{(1)} = 1.53$ Hz, is close to walking frequency, $f_{(w)} = 2$ Hz. Therefore, a forced vibration was performed according to European standards [3]. The maximum acceleration, $a_{max} = 0.145$ m/s^2, is smaller than allowable acceleration $a_{lim} = 0.490$ m/s^2. Although the structure is extremely slender, the users do not have an unpleasant feeling when standing or walking on the bridge. The bridge was built in 2007.

5. PEDESTRIAN BRIDGE ACROSS THE SVRATKA RIVER IN BRNO

Figure 10: Completed bridge

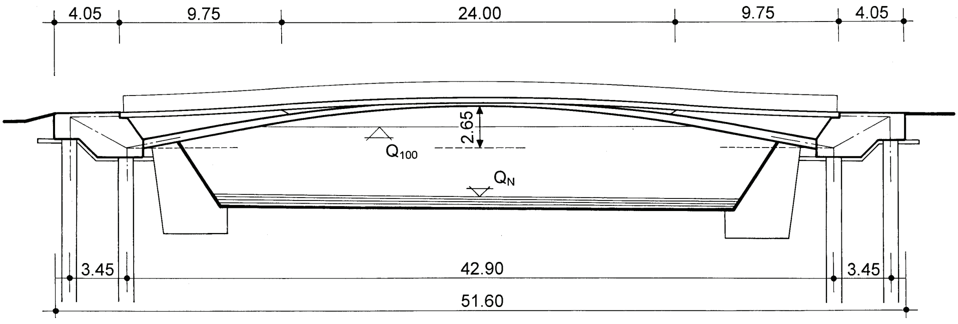
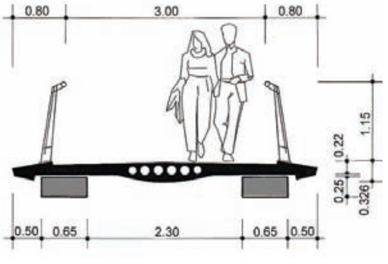


Figure 11: Bridge elevation

Another such bridge was built across the Svratka River in a city of Brno, Czech Republic. The bridge connects a newly developed area (Spielberk Office Centre) with an old city center. It is situated in the vicinity of a new international hotel and a prestige office area.



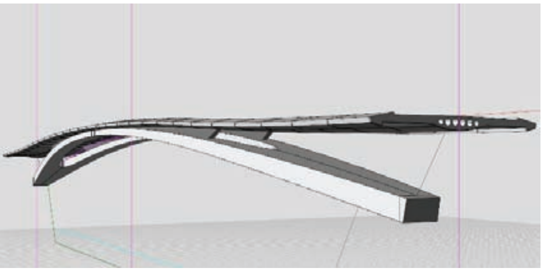


Figure 12: Cross section

Figure 13: Structural arrangement

Close to the bridge an old multi span arch bridge with piers in the river is situated. It was evident that a new bridge should also be formed by an arch structure, however, with bold span without piers in the river bed. Due to poor geotechnical conditions a traditional arch structure that requires resisting of a large horizontal force would be too expensive. Therefore, the self anchored stress ribbon & arch structure represents a logical solution. Also, smooth curves that are characteristic for stress ribbon structures allowed a soft connection of the bridge deck with both banks (Figure 10).

Since the riverbanks are formed by old stone walls, the end abutments are situated beyond these walls. The abutments are supported by pairs of drilled shafts. The rear shafts are stressed by tension forces, the front shafts are stressed by compression forces. This couple of forces balances a couple of tension and compression forces originating in the stress ribbon and arch. The arch span L = 42.90 m, its rise f = 2.65 m, rise to span ration f/L = 1/16.19. The arch is formed by two legs that have a variable mutual distance and merge at the arch springs. The 43.50 m long stress-ribbon is assembled of segments of length of 1.5 m. In the middle portion of the bridge the stress ribbon is supported by low spandrel walls (Figures 11 and 12). The stress ribbon is carried and prestressed by four internal tendons of 12 0.6” dia monostrands grouted in PE ducts. The segments have variable depth with a curved soffit. The stress-ribbon and the arch were made from high-strength concrete of the characteristic strength of 80 MPa.

a)

b)

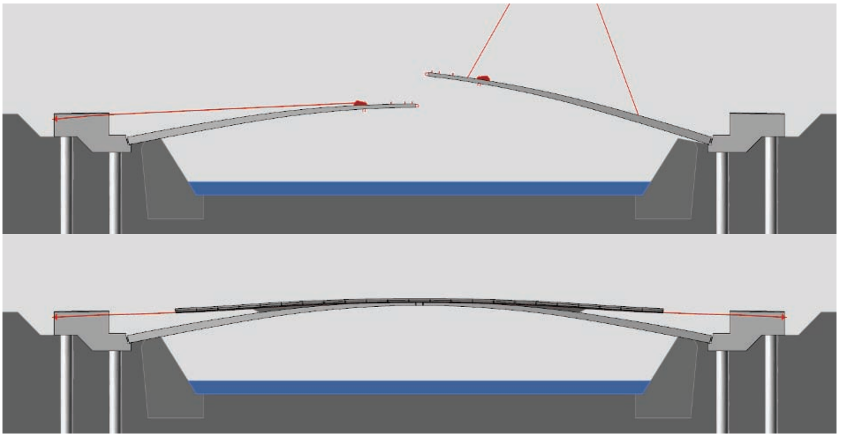


Figure 14: Erection of the bridge: a) arch segments, b) deck segments

The arch was assembled from two arch segments temporarily suspended on erection cables anchored at the end abutments (Figure 13a). Before a mid-span joint was cast, the length of the erection cables was adjusted. In this way the effects of deformations of the shafts were eliminated. After that the erection cables were replaced by external cables that tied the abutments. Then the segments were placed on the arch spandrel walls and on the external cables (see Fig. 13b). Subsequently, the internal tendons were pulled through the ducts and tensioned. Finally, the external tendons were removed. In this way, the required geometry of the deck was obtained. After casting the joints between the deck segments, the cables were tensioned up to the design stress and, as a result, the deck was prestressed.

The structural solution was developed on the basis of the very detailed static and dynamic analyses. The first bending frequency, f(1) = 1.912 Hz, is very close to the walking frequency, f(w) = 2 Hz. Therefore, a forced vibration was performed according to European standards [3]. The maximum acceleration, amax = 0.162 m/s2, is smaller than allowable acceleration, alim = 0.691 m/s2. The bridge is very stiff and the users do not have an unpleasant feeling when standing or walking on the bridge. The static function and quality of the workmanship were checked by loading test. The trucks were situated on the whole and half length of the deck. The construction of the bridge started in February and was completed in September 2007. New structure was well accepted by public.

6. MCLOUGHLIN BOULEVARD PEDESTRIAN BRIDGE, PORTLAND, OREGON, USA

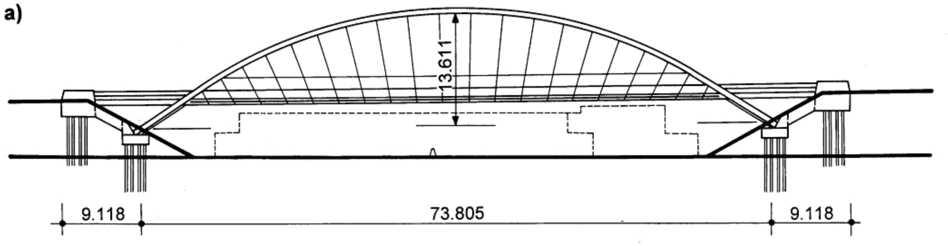


Figure 15: Elevation

The McLoughlin Boulevard Pedestrian Bridge (Figure 14) is a part of a regional mixed-use trail in the Portland, Oregon metropolitan area and is owned by the City's Parks & Recreation Department. The bridge is formed by a stress-ribbon deck that is suspended on two inclined arches (Figure 16b). Since the stress- ribbon anchor blocks are connected to the arch footings by struts, the structure forms a self-anchored system that loads the footing by vertical reactions only (Figure 14). The deck is suspended on arches via suspenders of a radial arrangement; therefore, the steel arches have a funicular/circular shape. The slender arches are formed by 18-inch-diameter pipes that are braced by two wall diaphragms – see Figure 15.
The stress-ribbon deck is assembled from precast segments and a composite deck slab (Figure 16a). The shape of the segments was developed from those used in the Rogue River Pedestrian Bridge, Grants Pass, Oregon. In side spans, the segments are strengthened by edge composite girders (Figure 16c). The deck tension due to dead load is resisted by bearing tendons. The tension due to live load is resisted by the stress-ribbon deck being prestressed by prestressing tendons. Both bearing and prestressing tendons are situated in the composite slab. The bearing tendons that were post-tensioned during the erection of the deck are formed by two bundles of 12 by 0.6" diameter strands that are protected by the cast-in-place slab; deck prestressing tendons are formed by six bundles of 10 by 0.6" diameter tendons that are grouted in ducts. Edge pipes and rod suspenders make up part of the simple and elegant hanger system. The suspenders connect to "flying" floor beams cantilevered from the deck panels to provide the required path clearance. The edge pipes contain a small tension rod that resists the lateral force from the inclined suspenders on the end of the floor beams. Grating is used to span the gap between the edge pipe and the deck panels. Protective fencing is placed in the plane of the suspenders to open up the deck area, and the rail is cantilevered from the suspenders.

Figure 16: Arches

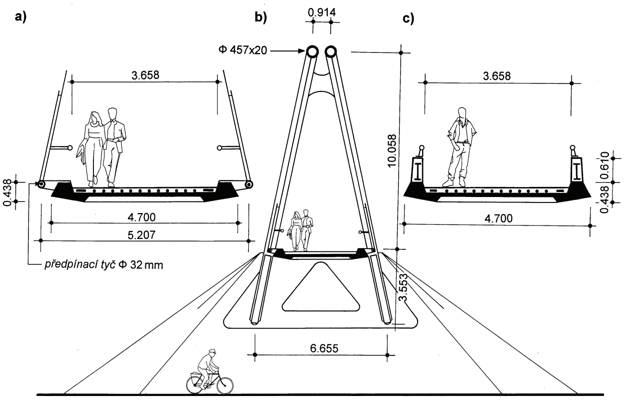


Figure 17: Cross section: a) main span, b) bridge, c) side span

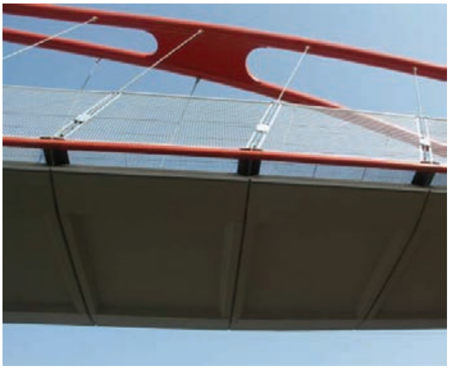


Figure 18: Suspension of the deck on the arches

Figure 19: Completed bridge

The structural solution was developed on the basis of very detailed static and dynamic analyses. Great attention was also devoted to analysis of the buckling of the arch and the dynamic analyses. The non-linear stability analysis has proved that the completed structure (Figures 17 and 18) has a very large margin of safety. However, during construction when the load is resisted by the arches only, the margin of safety was relatively low. Therefore, the erected structure was stiffened by a mid-span erection tower that loaded the arch by a controllable force.

The multi-mode analysis completed for a given response spectrum has proved the structure has satisfactory response to seismic load. Although the first bending frequency, $f_{(1)} = 1.021$ Hz, is below the walking frequency, $f_{(w)} = 2$ Hz, the forced vibration performed according to European standards has proved that maximum acceleration, $a_{max} = 0.104$ m/s^2, is smaller than allowable acceleration, $a_{lim} = 0.505$ m/s^2. Pedestrians do not have an unpleasant feeling when standing or walking on the bridge.

Arches were fabricated in halves and one half set in place on each of two consecutive nights. A center tower supported the arches during erection and was then lowered to allow the arch action and prevent the arches acting as girders. The segments were erected in sets of five, arranged symmetrically from the mid-span. After erection of all segments, the deck slab was cast and post-tensioned. The edge girders of the side spans were then cast and protective fencing was mounted on suspenders.

Construction of the bridge commenced in March, 2005, and was completed in September, 2006, at a cost of $1.4 million USD. The structural system proved that a signature bridge of this type, which acts as a gateway to both the City of Portland and the City of Milwaukie, can be very cost-effective.

7. CREDITS

The research work of the stress ribbon structure supported by arches was performed at Brno University of Technology. The pedestrian bridge across the expressway R35 was designed by Strasky, Husty and Partners, Brno and was built by Bögl a Krysl, Plzen. Project Engineer was Libor Hrdina. Pedestrian bridge across the Svratka River in Brno was designed by Strasky, Husty and Partners, Brno and was built by SKANSKA DS, Brno. Project engineer was Petr Stefan. The McLoughlin Bridge in Portland, Oregon, was designed by OBEC Consulting Engineers, Eugene, Oregon, with collaboration of Jiri Strasky, Consulting Engineer, Greenbrae, California. The Project Engineer was Gary Rayor. The bridge was built by Mowat Construction Company, Vancouver, Washington. The static and dynamic analyses were performed by Dr. Radim Necas and Richard Novak.

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