

Structural steelwork for Střížkov underground station, Prague

A new underground station at Střížkov in Prague has been designed and built as a special structure with a high level of aesthetics. The hall structure is 160 m long, 42 m wide and 20 m high. Two crossing hingeless main arches spanning 160 m are fixed to concrete pylons. The roof structure is suspended from the main arches using a system of prestressed rods and supported by columns around the roof. Most of the members are curved with variable cross-sections (the majority of sections are welded boxes with a plate thickness of 10–35 mm except the columns, which are welded I-sections). The geometry of all parts made design, fabrication and erection very difficult. The whole structure was built under the direct supervision of the architect. The underground station was opened to the public in May 2008.

1 Main steel structure fashioned as whale's skeleton

1.1 Architect's vision

One of the most important transport projects in the northern part of Prague involves an extension to underground line C with three new stations. The whole new section was opened to the public in May 2008. Most interesting in terms of architecture and engineering is Střížkov station in Prague, in the Prosek section. This new underground station is the first of about 50 Prague underground stations to be built with a fully glazed envelope. The roof is about 5 m above ground level (top of structure is 20 m above ground level) and the tracks are 7 m below ground. The whole station area is more than 100 m long without

internal columns. The architect's idea is that the glass station hall will shine out at night into the open neighbourhood and during the day natural light will illuminate the platforms. The main steel structure fashioned as a whale's skeleton was conceived by the architectural practice of Patrik Kotas (the new tram line between Zličov and Barrandov in the difficult terrain of the southern part of Prague with "new age" solutions for stations and bridges or important reconstruction of Prague's main station were also designed by this practice) (Fig. 1).

1.2 Loadbearing steel structure

Architects conceive the shape of a structure to satisfy aesthetic needs in the main to jumbles up to difficultness of process realization since statically design with fabrication and erection to finality surface finish. Every partial task required an unconventional approach to reach a technical solution. All the curves of single parts were continuously checked by architect. Only the upper parts of transverse trusses are linear, all other edges are curved. Minor parts of edges were defined by mathematical formulas, major parts of edges drawn exactly by the architect (Figs. 2 and 3).

The roof to the underground station has an overall format of about 160 × 42 m, with a maximum height of 20 m above terrain. The two main arches have midlines like segments of a circle. Plane of arches are in slope at ap-



Fig. 1. Aerial view

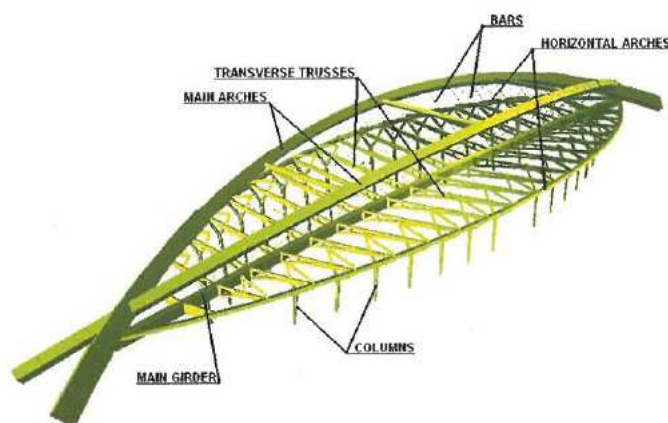


Fig. 2. Model of main arches

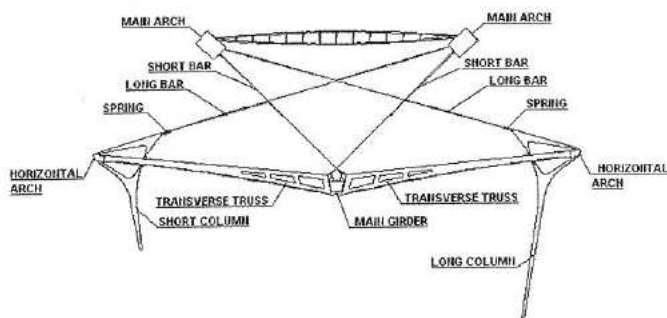


Fig. 3. Typical cross-section

prox. 45° to ground and arches are 2 m displaced each other. The crossing arches is near the supports. Respect to displacement of arches and their variable cross-section made fabrication very difficult. Inside the box sections there are diaphragms every 6 m. The arch members measure 1.5×3 m near the supports and 1.5×1.5 m at the apex, the boxes are welded from plate 20–35 mm thick (Fig. 4). The arch buckling lengths are determined with the positive influence of the main prestressed bars. Anchoring with each concrete pylon is with 26 M64 chemical anchors 1300 mm long and a base plate thickness of 60 mm with a system of stiffeners. The whole anchoring system is realized with a special non-conducting system. The extreme normal force at the supports is 17000 kN, the bending moment 12000 kNm.

The whole roof structure is suspended below the main arches by means of two systems of prestressed bars. Above the axis of the railway there is a 130 m long main girder supporting transverse trusses. The girder is a box section welded from plates 10–20 mm thick, measures 1.6×1.8 m and is in trapezoidal form (axis as part of circle in vertical plane). Around the periphery of the roof there is a “horizontal” arch (box section with variable cross-section measuring about 0.9×0.5 m is curved in vertical and horizontal planes). Transverse trusses every 6 m between horizontal arches and main girder are specially formed from a welded box section to the architect’s design. These trusses are 13–42 m long; the weight of longer beams is reduced by specially formed holes (Figs. 5 and 6).

Columns around the roof are shaped like a letter “Y”, about 7 m high on the south side of the station, 14 m on the north side. The columns are welded I-sections with va-



Fig. 4. Main arches, columns and horizontal arches



Fig. 5. Main girder and trusses during erection



Fig. 6. “White lines” of main girder and trusses

riable depth. Each of the 28 columns has its own geometry. All main structural items are fully welded, only the columns are connected to trusses and anchorage members by pins diameters 48 and 56 mm (anchorage members manufactured from stainless steel grade S480) (Fig. 7).

The influence of temperature change is very important for the main structure. The main arch span of 160 m would have less rigidity than the hanging system of main



Fig. 7. Columns during erection



Fig. 8. System of bars

girder plus trusses with peripheral columns. Lowering the temperature of structure would have evoked such strain of main arches, that bars would be functionless. Therefore, prestressed bars were necessary in order to maintain tension in the bars at the lowest temperature. Two systems of prestressed bars were designed using stainless steel grade S460. The first system of bars (M48, "short bars"), which supports main girder from main arches, performs the primary function. These bars are 7–15 m long and slope at 45°. The second system of bars (M30, "long bars") performs only a supplementary function (main reason for this system is aesthetic). These bars slope at about 30° and are 15–30 m long. Special springs with press capacity 180 kN and rigidity 1 MN/m (coiled from 55 mm dia. rod, weight of each spring 125 kg) are used to reduce the tension in the long bars to 30 %. Tension springs were necessary for a system of springs with rods, but for fabrications reasons compression springs had to be used and the tension force is converted to spring pressure by special stainless steel scissor cages. Maximum total forces inclusive of prestress in the M48 short bars are 670 kN and in the M30 long bars 160 kN (Fig. 8).

2 Fabrication of steel structure

For the design the whole structure was simulated as a 3D linear and non-linear model. The geometry with many curved members and the stability analysis of the main arches in interaction with systems of prestressed bars made the modelling very difficult. A precision 3D model was used for preparing production documentation, too.

With regard to shape and geometry, the production of the structure was very sophisticated. Connections of all main members were checked before delivery, but parts made by different fabricators were checked in the 3D model only (structure manufactured by three fabricators). The most difficult part was the crossing of the main arches and so a special fabrication process was prepared for assembling particular plates and the welding process. The production process had to be completed in a very short time. Production documentation started to be prepared in



Fig. 9. Assembly of crossing part of main arch

December 2005, fabrication began in January 2006 and erection in March 2006. The structure is made from steel grade S355JRG2, certain individual parts from stainless steel grade S240 or S480 (bars S460). The whole structure is schoop plating and finished with a top coat. Corrosion protection was included as part of the production schedule and in situ only welded areas repaired and the final white top coat applied. All butt welds were ground and cemented so that all lines and faces of the structure are clear and smooth.

3 Erection of main structure

Erection of the main steel structure started on the 15 March 2006 with the setting-up of the first steel part to reinforced concrete cote pylon and was finished on the 21 September 2006 by activation of bars tensions. The assembly of the structure was exacting not only with respect to the intricacy of the geometry, but also with respect to the great number of on-site welds. Single members for erection were up to 27 m long and 42 t in weight. Assembly proceeded from both sides with the help of a 300 t crawler-mounted crane. Priority of this crane was its ability of travel with weight. Members arrived continuously from three structure fabrication plants. Many members represented excess loads for transport and for the largest parts (main arch crossings – 4.6 m high, 6 m wide and 11 m long) it was necessary to close the highway.

The two main arches account for about 60 % of the total weight of the structure. The connections of all the parts of the arches were fully checked in the production plants before being sent for assembly. There was no chance to correct mistakes in members in situ. The geometries of all parts were surveyed in the production plants by a geodesist and a real 3D model prepared as an aid for erection. Individual arch members were temporarily supported by hydraulic presses (press capacity about 150 t) on temporary towers. The main structure is very sensitive to temperature. The geometry of the arches during erection was monitored continuously by a geodesist and the right vertical position was adjusted by presses. Differences between



Fig. 10. Erection of last part of main arch

night and day temperatures exceeded 30°C and the highest temperature of the white-painted members during sunshine was 40°C (for bright grey it was 55°C and for dark grey more than 60°C). Temperature was continuously monitored and the right geometry was promptly recalculated in situ. The most difficult step in erection was



Fig. 11. Assembly of first part of main girder under main arch crossing

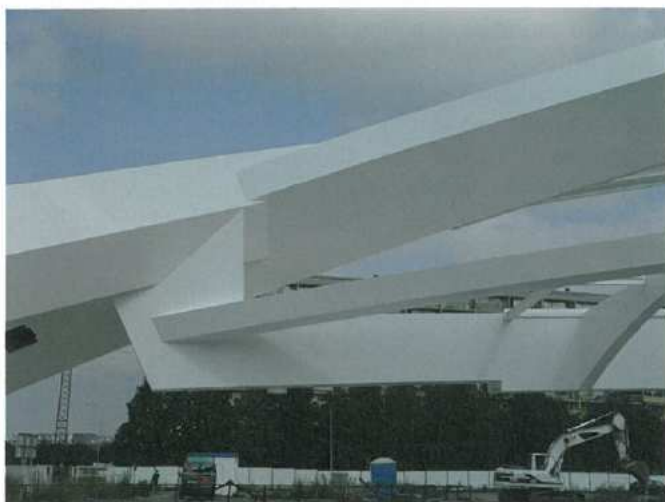


Fig. 12. "White lines" of arches

closing each main arch with the top member (each top member was 24 m long and weighed 30 t). All equipment was prepared for detail correction of geometry depending on change in temperature during final welding work. Fortunately, the weather during three critical days in June 2006 was very cold, with night-time temperature about 8°C and daytime temperature about 12°C without sunshine. Exactly after finishing the last weld of the main arches the hot weather returned. The structure warmed up and rose from its erection supports. From this moment on the main arches became self-supporting and the erection towers were dismantled (Figs. 9 and 10).

After finishing the erection of the main arches it was time to assemble the roof structure. All roof members (main girder, trusses, horizontal arches) were erected using erection supports under the main girder and columns as final supports of horizontal arches. The most difficult step in roof structure erection was placing the starter girder part under the main arch crossing. During roof structure erection, checking of geometry was carried out in the same way as for the main arches. The last step in the roof erection was connecting bars (Figs. 11 and 12).

4 Prestressing of bars

4.1 Short bars – first phase

A system of short bars (M48) ensures the main load-carrying capacity and was prestressed in two phases. After finishing erection on assembly supports, the first phase started. For bringing prestressed force into bars was chosen installation of tensioning pieces on purposely distorted structure. Using hydraulic presses, the main girder was vertically deformed 30–130 mm on each erection support. The maximum vertical reaction at every erection support was about 1000 kN. Due to the irregularity of gradual lifting of the continuous structure at eight points, presses with 1500 kN capacity were used and precamber was performed step by step in the range 8–30 mm. After finishing this process, 34 short bars were installed. The structural calculations had determined a temperature of 10°C as sufficient for introducing the desired prestressing forces. Assembly of the bars was carried out in August with a minimum night-time temperature of about 15°C and maximum daily temperatures of 35°C . Activation of bars was done during one night between 2 and 5 a.m., when the temperature was lowest.

After finishing this activation, the structure heated up 30°C during the next morning and temperature deformations lifted the girder off the erection presses. Due to the higher temperature during prestressing, only 60 to 70 % of the prestressing force needed was achieved. A greater precamber (which would correspond to a minimum night-time temperature of 15°C) could not be realized. Mounting support were built in the area of the future track on the reinforced concrete basement ceiling. Carrying capacity of this ceiling has been exhausted by reactions assembly support 1000 kN. But precambering for 15°C would invoke a reaction of about 1600 kN. It was also not possible to postpone the process to a period with lower temperatures. Therefore, it was necessary to carry out the second stage of prestressing using special hydraulic equipment.

4.2 Long bars

After finishing the first phase of prestressing the short bars, the system of long bars (M30) was prestressed. The tension forces required were achieved by assembling compressed springs, which were mounted between the bars and a shaped plate on the truss. A prestressing force of 60 to 100 kN had been calculated for each of the long bars. Each spring was fitted into a special cage and pushed to the calculated length by means of four threaded rods. Shortening of up to 130 mm was needed. The compressed spring was fitted with bars and then the cage was dismantled. Expansion of the spring initiated a prestressing force in the bar. This method of prestressing has proven highly effective because the right forces are achieved without the need for further corrections (Figs. 13 and 14).



Fig. 13. Springs in cages prepared for mounting



Fig. 14. Spring with long bar

4.3 Short bars – second phase

The second phase of prestressing the short bars was the last activity in primary structure erection. It was needed to increase the tension forces in the bars. Special hydraulic machinery, a technotensioner from the bars supplier, was used for this phase. During prestressing with this equipment, forces in all bars were continually monitored. A technotensioner makes it possible to increase the force, but not reduce it. For regular forces in bars the whole system was continually recalculated for the actual structure temperature. Increasing temperature about 15 °C evoked increasing force about 20 %. The maximum force in the



Fig. 15. Interior of station



Fig. 16. Glass wall with "Y" columns

bars during prestressing was 480 kN, the maximum force during service will be 700 kN.

Individual prestressing of each bar invoked the interaction of adjacent bars. Significant influence was measured especially for the next three bars on either side of the bar being tensioned. This interaction was dependent on the magnitude of the force in the individual bar before tightening and its position in the structure. Based on their location in the structure, the bars are from 7 to 15 m long. The shortest bars are at both ends, where the structure is more rigid. The longest bars are in the middle, where rigidity of the structure is the lowest. For final prestressing in the second phase it was necessary to increase the tension in each bar by about 100 kN. Due to interaction of adjacent bars, tensioning had to be carried out in two steps. For each step, the middle bars were tensioned first and then the work proceeded to the shorter bars at the ends. Very important factors for determining the right prestressing force were temperature of structure and time of implementation. This work was carried out in September. The increase in the temperature of the structure between sunrise (14–15 °C) and afternoon (28–30 °C) changed the force in the bar from 20 to 50 kN due to its position. Therefore, the prestressing procedure was corrected so that the largest forces in the middle bars were applied in the morning. After the second prestressing step, there was good agreement between the computational 3D model and the actual structure.

4.4 Monitoring of tension forces in bars

The process forces in the bars were continuously monitored during prestressing. Tensiometers were glued to all short bars and every alternate long bar. These were connected to a gauging station and monitoring and recalculation proceeded on-line in situ. Forces in short bars were also controlled by the technotensioner indicator. Forces in long bars were controlled by conversion of the measured size of the compression spring. Different methods of force checking proved to be in good agreement. Two months after finishing the erection of the structure, the forces in the bars were checked in other weather conditions (tensiometers were retained on bars from erection finalization) with results again corresponding with the structural 3D model.

5 Summary

The main steel structure of Střížkov underground station was very difficult for design, fabrication and assembly. Basic shapes were taken from the architect's visualization and all details were prepared with a high level of aesthetics. The whole process of design and construction was carried out

under the direct supervision of the architect. The self-weight of the structure is 950 t. Primary players in the realization process: client – Prague Public Transport Company; architect – Patrik Kotas practice; general designer – Metroprojekt Praha; designer of steel structure – Excon; fabrication and erection – Metrostav division 7 and Excon.

In 2009, the European Steel Design Awards of ECCS were given to outstanding projects in 18 European countries: the Střížkov underground station was one of the projects.

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Author:

Ing. Jaroslav Vácha, Excon a.s., Sokolovska 187/203, Prague 9, CZ, vacha@excon.cz

Book reviews

Burke Jr., M. P.: **Integral and Semi-Integral Bridges**. Oxford: John Wiley & Sons. 17 x 24 cm, Hardcover, 256 pages. ISBN 978-1-4051-9418-1 84,90 €

Worldwide, integral bridges are being used in greater number in lieu of jointed bridges. The main reason for this is they tend to be less expensive to build as well as easier to maintain and more economical over their life time. This is principally due to the non-existence of bearings and joints that are main sources of maintenance costs during life time. Contrary to most European countries where only few experiences with integral bridges have been gained so far, in the USA the construction of most moderate length bridge types is based on this concept.

The book at hand is not a kind of design guide; it does not take the place of a primer on the analysis, design and construction of continuous bridges and specific components. In fact it is based on 11 scientific papers, focusing on subjects that are significant for the design and construction of integral and semi-integral bridges. Problems are identified, suggestions for their avoidance are given and improved constructions and details are discussed. In detail, the following subjects are covered:

- Concise history of deck-type highway bridges in the USA from the jointed single-span bridges in the early 1930s

to the fully integral bridges in the late 1990s

- Description of the uncontrolled growth/pressure phenomenon, which is not taken into consideration by many designers in the USA yet. This effect is described in detail in an appendix, providing a brief description of the issue and containing documentation of some pavement and bridge damages associated with this phenomenon.
- Attributes and limitations of integral abutment bridges, including design provisions to account for most of the limitations named
- General considerations and simplifications for the design of integral abutment bridges, including the handling of creep, shrinkage, earth pressure, settlement, thermal gradients, buoyancy, earthquakes, etc.
- Early concerns and design decision resulting in the construction of the first fully integral deck-type highway bridge in 1938 (Teens Run Bridge of Gallia County, Ohio)
- Description of problems with integral abutment bridges (e. g. deck slab cracks, lateral rotation of semi-integral bridges, erosion etc.), including advices how to anticipate and prevent them
- Description of some problem-solving techniques used by bridge design engineers to achieve successful bridge designs

- Behaviour of skewed semi-integral bridges during expansion and simplified procedures for estimating the magnitude of forces involved
- Differences in design and construction of semi-integral bridges in certain US-states, detailed sketches and a summary of experiences
- Description and illustration of the structure movement systems approach to the design of highway bridges. Here integral and semi-integral bridge applications are conceptualized holistically as composite structures that are conceived to be composed of various types of structure movement systems.
- Discussion of design errors which occurred during a bridge replacement and rehabilitation project for the state of Ohio, resulting from a general lack of awareness.

Concluding it should be mentioned that the design recommendations and concepts should not be adopted wholesale but need to be adapted to European standards as well as philosophies. This becomes particularly apparent when looking at the willingness to abandon considerable benefits such as slender structure and saved middle support to achieve simplifications during the design process.

Daniel Pak, Aachen