NET GAIN

An unconventional launching procedure was chosen for erection of the largest network arch bridge in the world. Aleksei Surovtsev, Yuri Devichinsky and Sergei Azanov report

The finished bridge with its 380m span

As Russia's third largest city, Novosibirsk is a major hub of industry, science and transport in the Siberian region. The total population of the city is about 1.6 million and it is located at the centre of transport infrastructure of the region and acts as a major transit point for the Trans-Siberia Eurasian international transport corridor.

In response to its strategic importance, the city drew up a masterplan for its development up to 2030, including construction of new bridge crossings over the Ob River. The Bugrinsky Bridge, which has recently been completed, was highlighted as a priority by the masterplan, which also specified its alignment and boundaries. The 2.1km-long bridge, which carries six traffic lanes and has 1.5m-wide footpaths along each side, is notable as the largest network arch bridge in the world.

At the bridge location the river is 550m wide and the ground conditions have three different geological formations; Devonian slate and sandstone broken by inclusions of Upper Paleozoic granitoids. Moreover, the river bed has a seismic fault zone.

In this zone it was not possible to find any soil with satisfactory strength and deformation properties with the top at -90m, and at the edges of the fault zone, the solid top layer is inclined at up to 50° to the horizontal plane.

The safest and most reliable solution was deemed to be that of locating the bridge foundations outside the steep drop in the fault zone, creating a bridge with a main span of 380m. Hence the size of the main span was directly determined by the geology. Additionally, a 160m-wide and 15m-high navigation clearance had to be provided.

Two options were considered for the main span – either an arch or a cable-stayed bridge. The former required a 380m-long through arch with composite deck approach spans approximately 105m long; the latter would have been a cable-stayed bridge with a 380m-long main span and 170m-long back spans.

Again, the approaches to the cable-stayed structure would be composite deck structures with spans of approximately 105m. Ultimately the 380m-long arch bridge option was selected as the most cost-effective of the two.

Despite the considerable dimensions of the deck, the arch span is a good match for the scenery and scale of the Ob River. Arches are also a common architectural motif for the city of Novosibirsk, where many of the existing bridges have arch spans; in fact the city's coat of arms features an arch bridge over the Ob River. Thus, the accepted solution also has a symbolic meaning in relation to the city's history.

This particular bridge span was proposed as a 380m-long network arch, rather than the traditional type of arch with vertical suspenders; the most common type of through arch spans. The traditional through-arch design is extremely sensitive to non-uniform and asymmetrical live loads; such loading creates high bending moments in the elements of the upper arch rib and the tie, while the force difference in the suspenders can provoke fatigue problems.

These factors mean that the structural elements must have larger cross-sections, which in turn increases the quantity of material in the deck and hence the weight of the elements to be erected. Installation of inclined suspenders can mitigate these problems; the most efficient solution is to use inclined criss-crossed suspenders, and such an arrangement is known as a network arch.

The construction height of the bridge arch is 72.7m, which corresponds to a span/depth ratio of about 1/5. It has a total of 156 cables, and the arch legs are inclined inwards by about 12° towards the longitudinal axis of the deck. The total width of the deck is 36.9m which accommodates two carriageways each 13.75m wide and with three 3.75m-wide lanes in each direction, two emergency lanes of 1.5m and 1m at the side, central safety barriers, and two footways each 1.5m wide.

The bridge deck is formed of a steel stiffening girder with an orthotropic plate. The cross-section of the stiffening girder comprises two box girders and two I-beams and the webs of the box girders have the same inclination of 12° which enables them to be connected to the arch legs. The top and bottom flanges of the main beams have welded joints while the webs are connected with high-strength bolts.

The main load-bearing elements of the carriageway are transverse I-beams installed with a spacing of 15m. These beams have a web depth which varies from 1.85m at the connection to the box girder to 2.54m at the deck centre-line. Transverse beams with 680mm-high webs are installed between them at 3m spacings. The longitudinal stiffeners have a box cross-section of 180mm high, made of 8mm-thick steel.

At the piers either side of the main span, transverse box beams are installed. The arch leg cross-section is a box of 2m wide and 3.9m high and the webs of the arch legs range in thickness from 32mm to 40mm at the connections to the tie. The steel for the arch ribs themselves is 32mm thick.

The arch segments are connected with bolts; the web joints are flange connections while the bolts installed on the web are evenly spaced and were only required to contribute to the structural behaviour during the launching of the arch.

With a web thickness of 32mm, making a full-strength joint with twin cover plates would require more than 40,000 additional bolts and about 100t of cover plates.

However, it was not possible to execute erection joints using just flange connections; during the construction of the arch, the bending moments were so high that cover plates for connection of the top and bottom arch ribs had to be installed. In view of this, the erection joints of the arch ribs were made with twin cover plates.

The two legs of the arch are connected to one another by means of longitudinal box braces. The portal criss-cross braces have a cross-section of 1m by 1.2m while the other braces have a cross-section of 800mm square; the flanges and webs of the braces are 20mm thick. The braces have both bolted and welded joints: the flanges are welded while the webs are connected with bolts.

The arch is shaped as a circular curve with a radius of curvature of 300m and formed of segments which are 10m long. The anchorages which connect the arch to the deck are at 10m centres and the cables are inclined at an angle of 60°. This angle was the result of iterative calculations based on a single criterion: to minimise bending moments in the arch for any live load combination.

The fact that the angle at which the cable is connected to the arch is constant enabled anchorage elements to be fabricated to a uniform design. Moreover, some 85% of the arch segments share an identical internal arrangement; where there are differences, they are mainly related to the need for erection joints to incorporate wind braces.

The anchors at the stiffening girder are passive while those on the arch are active; this arrangement was selected in order to reduce the size of the anchors at deck level. These deck-level anchors are fixed by means of hinges, a measure which was required because the rotation of the anchorages during the cables installation exceeded the allowable values of $\pm 0.3°$.

With installation of the suspenders in design position, the deformation both of the arch and the tie takes place. A series of calculations and analysis was carried out ahead of the bridge construction, the majority of it relating to the erection of the structure. These included launching analysis of the stiffening girder; wind tunnel tests of a carriageway segment, and of the whole arch; arch span launching analysis; analysis of the arch connection to the tie and the subsequent arch lowering using temporary piers; cable system installation analysis; structural analysis of special purpose auxiliary structures and service stage analysis.

The main advantage of network arches is the reduction in material consumption, but any arch creates challenges in terms of its erection, and network arches are particularly difficult to build compared with a traditional arch structure. In this case the arch was assembled and erected on the site. First the deck girders, which form the arch tie, were launched separately using temporary piers and then the arches were erected from the ends of the tie by launching towards the mid-span.

The erection of the stiffening girder was designed to be carried out by longitudinal launching of a continuous arch section of the main span and the five preceding approach spans.

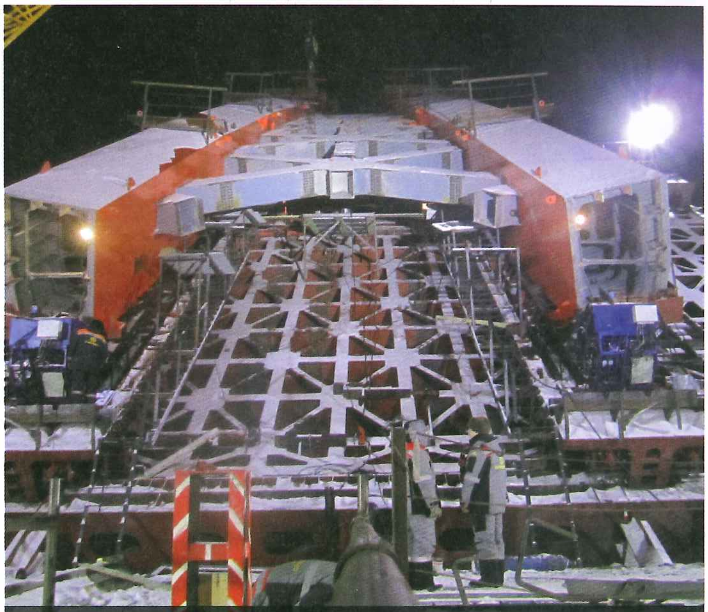
Several alternatives were considered for the arch erection: to deliver the arch elements by barge and then install them in their final position, to build the arch using a special truss-lifting crane, to launch the arch or to erect the arch using temporary towers installed on the two main piers.

The construction time and the quantity of materials to be used for temporary auxiliary structures were considered for each of these options and after comparison, the decision was taken to choose the vertical and radial launching process. Vertical and radial launching was carried out on a vertical curve with a constant 290m radius of curvature – believed to be the first time this has been done in the world.

The choice of launching was based on the engineering and cost analysis – it needed the minimum of auxiliary steel structures and enabled the shortest construction time.

The process for launching the arch had a number of key features. In plan, the arch has a variable configuration, meaning that at the beginning of the launch process,

The deck was launched across the river from the banks



Incremental launching was also chosen for construction of the arch members

Close-up of the upper arch members

Load test of the finished bridge

the distance between the central lines of the arch boxes was 3.8m and at the end of the launching it was more than 26m. Secondly, the temporary piers used for the launch process were very high. Thirdly, the launching was carried out on a vertical circular curve with a 290m radius. The launching line had a 36° inclination to the plane of the tie. These features were the main requirements governing the construction procedure.

A 412m-long arch was launched from both ends of the main span girder towards the arch centre using 36m-long launching noses. The maximum length of a cantilevered section of the launched semi-arch reached 76m. The launching process comprised 19 steps and there were two scheduled stops during the operation: one to enable removal of the temporary end piers after the launching nose reached the intermediate temporary pier and the second one to enable the launching nose to be dismantled after the front arch segments reached the central temporary piers. The 19th step included the closure of the top arch section, after which the support blocks were removed and the arch was connected to the tie.

All arch assembly and launching operations were carried out on two service bridges constructed at the permanent piers, and the deck of these bridges was inclined to accommodate the curve of the arch. Braces were used on the service bridges to transfer temporary loads to the permanent piers while axial forces from the service bridge to the arch tie were transferred via the end stop. Guide frames with trolleys installed on them were used to support preassembled segments during the launching operation; these frames were located on the service bridge deck.

When unloaded, the guide frames could move across the bridge axis. At the launching, the guide frame position was secured in the longitudinal direction using the frame support. Forces from the frame support were transferred to the service bridge beams. Four jacks, each with a capacity of 300t, were used as pushing devices – two jacks at the axis of each semi-arch. When the erected part of the arch was moving, axial forces were taken by the jacks and the trolley supports. The maximum launching force at each axis of the semi-arch was 390t and the trolley moved along the bridge axis only; transverse movements of the trolley were prevented using lateral stops.

At the tie, temporary piers were assembled for the launching operation and these were fixed using cable back-stays. All piers were equipped with tracks for launching the variable geometry arch. On each pier, a set of variable height rollers was provided and these were equipped with sliding plates with a friction coefficient of 0.07.

The arch launching and erection procedure were carried out under constant geodetic control. The blocks of the launching nose were dismantled once the arch segments had passed over the tracks of the central temporary piers.

During the launch, the arch was secured using Dywidag rods; these rods were also used when reloading the jacks or making joints between the segments. The variable geometry of the arch elements did not permit the use of lateral stops to adjust the horizontal position of the launched structures on the temporary piers. The only way to ensure that the semi-arches were in the correct design position was by adjusting the forces in the jacks on the service bridge, and due to this, the jack forces were controlled by computer. The vertical position of the temporary piers was checked via geodetic surveying and the forces in the temporary piers and backstays were also monitored and controlled during the operation.

After the launching was completed, the removal of the temporary structures and installation of suspenders were carried out simultaneously. Since the presence of the temporary piers and the service bridges hindered the installation process, the suspenders were tensioned by groups.

The suspenders were split into those which required temporary piers for their installation; those which could only be installed once the central temporary piers were partially dismantled; and those which could only be installed once the central temporary piers and the service bridges had been completely removed.

Carrying out this process in parallel with the removal of the temporary structures significantly complicated the selection of an optimal installation sequence for the suspenders, as well as the analysis of the erection stages. When designing a cable-stayed structure, the choice of a practical and efficient installation sequence of the cable system is the most challenging task. Using the discrete optimisation method, engineers were able to identify a sequence that complied with all structural requirements of the cable-system supplier, as well as meeting the contractor's construction programme deadlines.

Once the installation of the cable system had been completed, the carriageway was paved with mastic asphalt. Then the calculation for the final, precise tensioning of the suspenders was carried out, to achieve the designed profile of the bridge deck and to bring the actual forces in the suspenders as close as possible to the design values. This analysis was carried out using non-linear mathematic programming. The cable installation subcontractor then performed the final tensioning of the suspenders in a very short time.

In parallel with the final tensioning, decorative elements intended to improve the appearance of the bridge were installed at the connection between the arches and the tie ■

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Temporary supports for the deck launch

Client: Road Construction Administration of the City of Novosibirsk
Main designer: Institute Stroyproekt
Main contractor: Sibmost