New Po River Bridge at Piacenza, Italy: The Construction Process

Alessandro Contin, Partner, E2B s.r.l., Padova, Italy; Enrico Viviani, Partner; E2B s.r.l., Padova, Italy and Marco Viviani, Prof. Dr.; HEIG-VD-EC+G, Yverdon Les Bains, Switzerland. Contact: marco.viviani@heig-vd.ch
DOI: 10.2749/101686611X13131377725523

Abstract
Bridges play an important role in a region’s economy and in the lives of its inhabitants. The Po River Bridge at Piacenza (Italy) SS9 “Via Emilia” (state road 9) is one of the most important infrastructures in Piacenza’s economy logistics (traffic across the Po River in this area is estimated as 18 000–20 000 vehicles/day and about 80 buses/day, 3500 workers crossing the bridge in the Lombardy direction). A replacement to the old bridge, partially collapsed during a flood, had to be provided as soon as possible.

Keywords: incremental launching method; reconstruction; innovative pier cap; construction management; building process; deck truss.

Introduction
The Po River Bridge at Piacenza on SS9 “Via Emilia” (state road 9) is one of the most important infrastructures of the region.1 Spanning 1.1 km, the new bridge (see Fig. 1) had to repose on the ancient piers, and some portion of the century-old bridge, classified as cultural heritage, was just refurbished and strengthened. The driving principles for the initial design of the 800 m long main viaduct were to maintain the architecture of the old bridge, to construct a new structure and to aim for reasonable reconstruction times. The choice of a deck truss allowed for the desired architecture, appropriate load-carrying capacity and speedy erection. The span to depth ratio is in the range of 17, a typical value for modern deck trusses. An important issue was raised before beginning the industrialization of the construction process. The truss being spatial, multiple tubes (web members) converge in the connections from different directions and thus the local stresses and deformations had to be carefully studied in order to ensure safety and serviceability and to optimize the design of the tube flanges, the welding process and the non-destructive testing (NDT) quality control. Finally, the industrialization processes could begin and led to the organization of two independent fields and to an incremental launching method for which all the usual equipment had to be modified.

Connections and Splice Design
Connections and chord splices of spatial trusses are often vulnerable; therefore they had to be carefully designed, welded and controlled. Owing to the tight schedule and the dimension of the bridge, the connections were partially prefabricated off-site and transported to the floodplain site; two teams were engaged to finalize the connections, weld the flanges to the tubes and pass them over to the two truss construction teams.

Special mock-ups and masks were prepared in order to facilitate the construction and ensure the correct geometry of the connections. Throughout the process a visual control and an ultrasonic NDT ensured the overall quality of the welds. Several welding types were tested and finally a rutile-cored welding wire was chosen. The use of this type of wires permitted to reduce inclusion and defects on welds, an important property especially for the tubes’ end-flanges close to the connections. A three-dimensional finite element method (FEM) model was developed to study the connections. This model permitted to efficiently evolve the geometry of the connections and obtain a configuration easier to prefabricate (off-site) and to weld (in situ). Local stresses and strains were controlled for all connections and fatigue effects were checked as well.

Deck, Chords and Webs
Deck, chords and webs had to be constructed using readily available steel elements. The bottom of the truss is made of two closed profiles connected by tubes welded perpendicularly to the main bridge axis (see Fig. 2). In-plane stiffness, is provided by steel “X” braces. The bottom chord profiles were prefabricated off-site by welding commercial steel plates, while the truss bottom was entirely assembled at the floodplain site.

The top of the truss is composed of three chords; all of them open-steel profiles of different cross sections, connected by welded T-profiles, notched on their webs to accommodate the
steel ribs of the deck (see Fig. 2). In-plane stiffness of the truss top is provided by the orthotropic steel deck slab. Webs of the truss are all made of flanged tubes and are welded to the top and bottom chords (see Fig. 2).

**Field Organization**

The floodplain site, located about 8 m lower than the launching platform, was secured against floods by sheet piles (the entire area was flooded a few weeks before the field installation and during the construction of the bridge the water arrived again to the sheet piles). Each of the 11 bridge sections was assembled on the floodplain site, translated to a ramp, pulled up to the level of the launching ramp and then translated to the line of the bridge axis (see blue arrows in Fig. 3).

Each element, after undergoing a geometry and weight control once placed on the launching ramp, was welded to the precedent section (welding lot is marked in green in Fig. 3), controlled for splice quality, metallized, painted (painting lot is marked in yellow in Fig. 3) and finally launched. Metallization, a special in situ zinc-based coating of the bridge, increases the durability of the structure.

In order to have an appropriate launching sequence it was necessary to organize four main assembly lines at the floodplain (marked in red and labelled with a number in Fig. 3). The first and the second lines were in charge of assembling the connections and the webs (see Fig. 3), the third and fourth were used alternately to assemble the webs with the top and bottom chords to form 15 m sections of the bridge and to weld these 15-m sections to form the 76-m-long sections. These 76-m sections were then translated on the launching ramp. The assembly lines three and four (see Fig. 3) were fed with raw material (deck sections, connections and tubes) prefabricated off-site and in situ on lines one and two (see Fig. 3). About 100 welders and five teams of quality control worked during the bridge construction; each quality control team was equipped with an ultrasonic machine for non-destructive testing of welds. Pier caps were prefabricated off-site.

While the above-described field was organized and the operations started, a second field was already open and operated independently to check the bearing capacity of the ancient piers and of the soil. Piers of the Po River Bridge are typical constructions of the beginning of the past century: massive masonry piers with some stonework (in this case granite) on the top and on the sides (see Fig. 4).

Most of these massive masonry piers are now both internally and externally cracked and thus do not allow for a consistent bearing capacity. Furthermore, the vibration and loads induced by the bridge launching could seriously affect them. An extensive campaign of testing allowed determination of their overall conditions and a strengthening strategy was established. Strengthening of piers included the construction of a diaphragm wall around the ancient foundation (diaphragms were built shortly after the flooding by a specialized contractor) besides a further 500 mm reinforced concrete wall encircling the piers.

Finally, some of the pier tops were not at the right level (the new bridge depth being half that of the previous one) and special steel pier caps and bearings had to be designed in order to have the truss in the right position during and after the launching (see Fig. 3).

Loads transmitted to the piers during launching were carefully estimated by FEM analyses and then measured in situ by means of load cells. Loads on the nodes and inclinations of piers were monitored in real-time in situ by means of strain gauges and digital inclinometers, positioned respectively on the box girders and on the piers.

**Launching Equipment and Substructures**

Incremental launching of bridges has been refined through years of trials.
Pier Caps

The pier caps adopted are one of the core features of the construction system (see Fig. 6). The new deck truss is 4 m deep; half that of the old bridge depth. The bridge deck is 14 m large. New pier caps had to be designed according to multiple design criteria, many not negotiable. The first design criterion for an incrementally launched bridge is to ensure that the loads transferred to the substructures (due to friction) do not exceed the capacity. In this case, considering the increased capacity of the piers following the strengthening undertaken, even a common PTFE-coated pad would have transmitted excessive loads to the piers (this type of sliding pads transfers about 5% of the vertical load). For this reason another system had to be found.

Discussions with the contractors, FEM models, the innovative shape of the pier caps and past experiences permitted to create a new system capable of reducing friction forces to less than 2% of the vertical load. This type of launching technique is used in concrete structures as well. Designers and contractor engineers using this method of construction have to take into careful account wind loads and others forces occurring during the launching. The incremental launching method produces longitudinal, transversal and vertical loads on the substructures. Longitudinal forces are generated by the friction of the rollers (sliding pads) on which the bridge moves, transversal forces are eventually generated by the guide system of the truss and finally the vertical reaction on the bearings is generated by the dead loads of the bridge. All of these forces were taken into account to design the pier strengthening and several solutions were found to minimize them.

Contractors familiar with this type of bearings, rollers and jacking equipment and a highly experienced launching team were enrolled for this project.

All the rollers were readapted in order to easily accommodate a rectangular rail that was welded to each of the bottom chords thus ensuring minimal friction and precise driving of the bridge truss during launching (see Fig. 6). To avoid transversal forces, a special pier cap was designed and equipped with a double roller (see Fig. 6). A FEM analysis of the stress generated by the bridge on the pier caps and piers during the launching phase was conducted. In case of minor misalignment of the truss during launching, these pier caps could be translated transversally (by means of hydraulic jacks) to avoid lateral forces. Launch force was generated using two large electro-hydraulic winches with a pulling capacity of 3100 kN (see Fig. 7); jacking equipment and transmission cables were directly integrated and anchored to the bridge abutment (Lodi side) that functioned as contrast block. Typical launching speed was of about 0.5 m/min with a pause every 25 m of advancement (used to check the launching direction). Typical length of each individual launching step was of about 75 m, corresponding to the span between the piers that was about 75 m for most of the 11 spans (see Fig. 1). The nose weight was 910 kN and it was 40 m long, about 60% of the typical launching span, while the relative nose/deck stiffness and weight were respectively of about 0.20 and 0.15.

Pier Caps

The pier caps adopted are one of the core features of the construction system (see Fig. 6). The new deck truss is 4 m deep; half that of the old bridge depth. The bridge deck is 14 m large. New pier caps had to be designed according to multiple design criteria, many not negotiable. The first design criterion for an incrementally launched bridge is to ensure that the loads transferred to the substructures (due to friction) do not exceed the capacity. In this case, considering the increased capacity of the piers following the strengthening undertaken, even a common PTFE-coated pad would have transmitted excessive loads to the piers (this type of sliding pads transfers about 5% of the vertical load). For this reason another system had to be found. Discussions with the contractors, FEM models, the innovative shape of the pier caps and past experiences permitted to create a new system capable of reducing friction forces to less than 2% of the vertical load (indeed the maximum force transferred to the piers was about 1.75% of the vertical load and
this also included the negative effect of the bridge curvature). The new system included: two compensated rollers per side, integrated in the pier caps (see Fig. 6), welding of a steel rail on to the bottom chords to reduce the contact surface and to efficiently guide the bridge (see Fig. 6), and two temporary semi-spherical mobile hinges between the pier caps and the piers. These hinges, permitting a longitudinal rotation to the pier cap, allowed accommodating of the nose of the bridge and, at the same time, allowed passing of the resultant of all forces always through the same point, at the centre of the pier (and of the hinges). This was why the magnitude of the effects on the pier caused by the launching forces was consistently lower than the design limits and all four rollers were always activated. After the launch completion, the pier caps were welded to the truss. Modelling and experience indicated that the technique adopted was appropriate; however, a system of real-time monitoring for beam stresses, horizontal forces and inclination of piers was installed. Never during the launching of the bridge the limits set for loads or inclinations were exceeded.

A system for truss centring was installed on some pier caps. It constituted two hydraulic jacks able to move transversally (to the main bridge axis) on the pier caps of about 80 mm. Although it was seldom necessary to use, this mechanism permitted to launch the bridge with the assurance that most of the lateral forces could be elided and that the centring of the nose and the bridge on the pier cap rollers was always optimal. Load tests confirmed the results of the numerical models and that the bridge could be inaugurated (see Fig. 8).

**Conclusion**

The new Po River Bridge at Piacenza (Italy) had to be reconstructed as fast as possible. The challenge was to replace a major bridge while refurbishing all the historical parts that had to be preserved. An innovative design and appropriate management of the construction process demonstrated that the various professional construction teams involved possessed the proper organizational skills to manage the construction of major infrastructures with precision within the given timeframe.

**References**

