The Beaucaire-Tarascon cable-stayed Bridge over the Rhone River

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Summary

The Beaucaire-Tarascon Bridge, completed during summer 2000, crosses the Rhone river in the south of France, 20 km downstream from Avignon. It is a two-pylon cable-stayed bridge, 410 m long with a main span of 192.80 m. The concrete deck consists in two lateral ribs supporting a 22cm thick slab stiffened by concrete cross-beams every 3.64 m. The concrete pylons are lyre-shaped. The stay cables are anchored in a steel box encased in the top of the pylon and in the lateral ribs of the deck. The deck is built partly on scaffoldings and partly by the balanced cantilever method.

1. Introduction

Due to traffic congestion in Beaucaire and Tarascon, it was decided to bypass both cities. The new route crosses the River Rhone downstream of the two cities. The bridge crosses the river, about 300 m wide and the parallel roads, so that its total length is about 400 m. Due to environmental regulation, it was imposed that the piers located in the river have no effect on the water level upstream. That means that offset dredging is necessary. Four bridge projects were established by SETRA: a composite twin-girder bridge, with a main span of 118 m ; a prestressed concrete box-girder bridge, with a main span of 118 m ; a composite deck cable-stayed bridge with a main span of 193.80 m ; a concrete deck cable-stayed bridge with a main span of 192.80 m.

The successful bid by Léon Grosse and Bauland TP was a cable-stayed solution in prestressed concrete. The bridge itself is more expensive than the bridges without stay cables, but with only two slender piers in the river, the cost of the dredging is considerably lower.

2. Design

The bridge design was developed by M. Virlogeux, D. Dupieu, E. Conti, D. Le Faucheur and the architect C. Lavigne. The bridge is made of a slender prestressed concrete slab with longitudinal and transverse ribs, cable stayed from two towers. Intermediate piers in the side spans help to stiffen the structure. The bridge has a total length of 410 m, with five spans 25.55 + 81.80 + 192.80 + 81.80 + 25.55 m long (*Fig. 1*).

2.1 Deck

The deck is 12.10 m wide, accommodating two traffic lanes (*Fig. 2*). Because the bridge is rather far from the city centres, it was not necessary to provide sidewalks for pedestrians or specific lanes for cyclists. The structure is made of two lateral, almost rectangular ribs, which are very shallow (0.70 m) and rather wide (2.30 m). These ribs are joined via an upper slab 0.22 m deep, and via a series of floor beams 3.64 m apart. The floor beams are precast and prestressed with pretensioned strands. With a symmetrical transverse deck inclination of 2.5% —to facilitate drainage — the total depth of the structure is 0.81 m. At both bridge ends, the deck is transformed into a massive slab with the same depth, which acts as a counterweight.

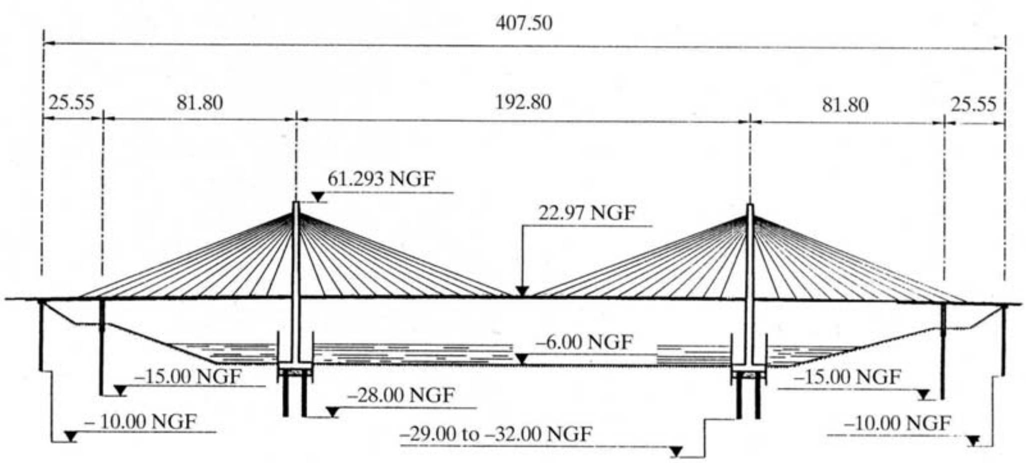

Fig. 1 General elevation

The deck is longitudinally prestressed by 19T15 internal tendons. The prestressing is designed to avoid every tensile stress in the deck under frequent traffic loads and to have almost centred compression under permanent loads. These requirements lead to place ten pairs of tendons in the end spans (*Fig. 3*). Half of these tendons run only on the massive slab, 33 m long. They are tensioned just after casting of this slab. The five other pairs of tendons extend on the side span and run from the abutment to the lower attach of each of the five first stays from the pylon. They are tensioned after closure of the side span. In the central span (*Fig. 4*), there are ten pairs of tendons, located in the lateral ribs and anchored on blisters at the intersection of the lateral ribs and the cross beams. These tendons are symmetrical about mid-span and their anchorages are placed at the attach point of the first ten stay cables from the pylons . The prestressing force is of course strongest in areas where the compressive force induced by the stays is the lowest. They are tensioned after closure of the main span. In addition to these tendons, there is one pair of tendons, 28 m long, symmetrical about each pylon, balancing the bending moments under dead load between the pylons and the first stays. These tendons are tensioned immediately after the construction of the first pair of segments. All the tendons are in straight line, except in the end spans and near the pylons.

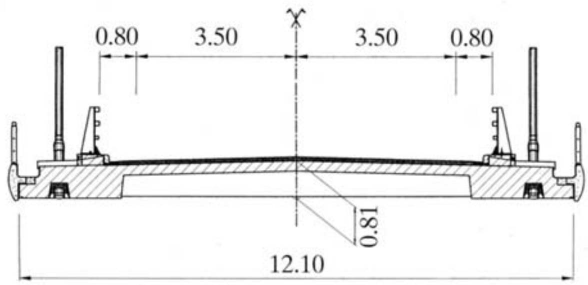

Fig. 2 Cross-section of the deck

2.2 Pylons

The pylons provide the necessary rigidity. They have the shape of a lyre in order to drive forces in a logical flow. The cable stays are installed in two vertical planes, one on each side of the road, and anchored in the upper parts of the pylon legs, which are vertical. The pylon legs diverge and turn around the deck, and converge again to limit the size of the cofferdam in the river. Cross beams join the pylon legs at the angles in order to balance force deviations. Below the deck, the cross section of the pylons legs is about 3.00 m x 3.00 m, with both dimensions decreasing above the deck.

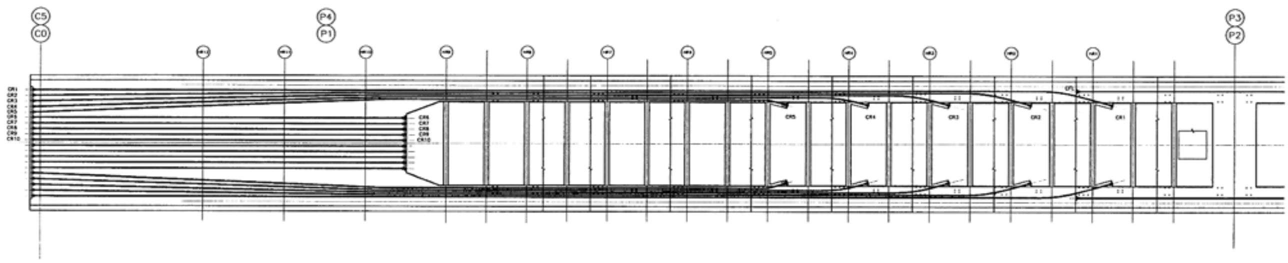

Fig. 3 Arrangement of prestressing tendons in side spans

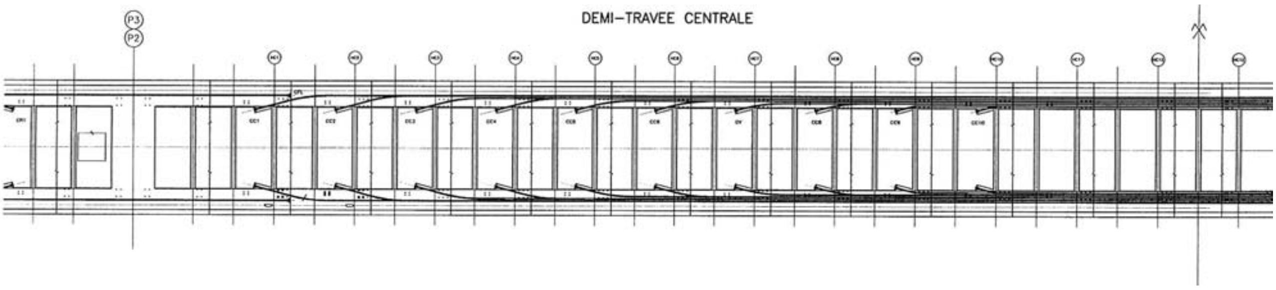

Fig. 4 Arrangement of prestressing tendons in main span

The lower cross beams of the pylons are prestressed by eight 12T15 straight tendons which balance the tension due to the compressive force in the legs and the bending moments induced by the bearings of the deck. The upper cross beam is always under compression under dead load and traffic loads. Passive reinforcement is needed to resist the bending moments induced by lateral wind.

2.3 Piers and bearings

The intermediate piers consist in two almost cylindrical shafts. The bearings are unidirectional pot bearings equipped with devices against upward displacement of the deck. The detailed design did point out that the traffic loads could create slight upward forces at the ultimate limit-state.
On the pylons, the elastomeric bearings provide longitudinal flexibility. They are equipped with lateral stoppers to avoid the lateral movements under wind effect.

Fig. 5 HDPE duct with helical fillet for stays

2.4 Foundations

Each pylon is founded on six piles, 2.00 m in diameter, with a circular arrangement selected to ease the construction of the cofferdam deep in the river. After the construction of the circular cofferdam (15 m in diameter and 23 m in height), the piles were bored and concreted inside steel casings driven through the alluvial deposits. No casing was needed in the bedrock (a hard marl layer) below.
The excavation inside the cofferdam reached 13.5 m below the average level of the river. A concrete plug, 2.75 m deep and connected to the casings, allowed water to be pumped out and the pile cap to be concreted. The pile cap extends up to 8.0 m below the river level and is also 2.75 m deep.

2.5 Stay cables

The deck is suspended by 48 pairs of Freyssinet cable stays, 12 pairs for each cantilever. The stays are made of parallel auto-protected strands (10 strands for the shortest stays, 19 strands for the longest stays). In classical Freyssinet strands, the wires are hot-dip galvanised and redrawn. The voids between the wires are filled with oil wax after the thermal treatment that follows stranding, and each strand is protected with extruded high-density polyethylene (HDPE), the quality of HDPE being of major importance for the strand durability.
For the Beaucaire Bridge, the wires were coated with a zinc-aluminium alloy (termed galfan) rather than with zinc. This has the major advantage of increasing the time during which the strand resists the French corrosion test by a factor of three, demonstrating that the strands have much greater durability. Consequently, the fabrication process had to be adapted by Tréfileurope in order to maintain the high fatigue resistance.
The cable-stay strands were installed in a HDPE duct, which was raised with the first strand. The duct has a central core of black HDPE, whereas the HDPE envelope is white to enhance the bridge's elegance, as was performed for the Vasco de Gama Bridge, Portugal. The white envelope is well adapted to the brilliant light of Southeast France. The duct is aerodynamically shaped: two

thin helical fillets wind round the duct in the same direction with a pitch length of 60 cm each. As demonstrated for the Normandie Bridge, these fillets eliminate, or at least reduce, rain- and wind-induced vibrations (*Fig. 5 and 6*).
Cable stays are anchored to the deck in notches. The width of the lateral ribs easily allows for this, and this solution is more elegant than creating blisters below the deck (*Fig. 7*). Cables are anchored to the pylons in steel anchorage boxes, which has been carried out frequently since the development of this technique by R. Greisch. Each cable is supported by a pair of shutter slats and passes within a steel pipe fixed properly in place (*Fig. 8*).
The stay cables are tensioned from the lower end. The lower anchorages are adjustable. This allows for compensating the creep deformation in the deck and the pylons after some years if needed. The adjustment will be made with a special jack.

3. Construction

3.1 Construction steps and erection techniques

The extreme parts of the side spans were built on scaffolding on each bank at the same time as balanced cantilevers were erected from the pylons. Seven pairs of segments were built before closure, with each element erected on scaffolding on the side span. The main cantilever was completed in the central span with five more segments. Construction began on the left bank, and after completion of the eastern side of the bridge, the two mobile carriages were transferred to the right bank in order to erect the western side.

Fig. 6 Installation of the strands

Fig. 7 Anchorage of stay cables under the deck

Due to the deck flexibility, the mobile carriages were equipped with auxiliary cable stays. After completion of a segment (7.28 m long), the mobile carriage was moved forward for the erection of the next segment. The auxiliary cable stays were then tensioned, the reinforcement was installed and the segment concreted. The tension in the auxiliary cables was adjusted at each step during concreting in order to control the levels of the mobile carriage and the cantilever. The riverside segment was concreted on Thursday, and the bank-side segment the following day. Before closing the side span, stability cable stays were tensioned step-wise between the head of the pylon and the cast-in-place element on the scaffolding to balance the load of the riverside segment. These cable stays were detensioned stepwise when the bank-side segment was cast (*Fig. 9*).
The following Monday, when the concrete of the two segments had hardened, geometric surveys were carried out and the new pairs of cable stays, which suspend the new pairs of segments, were installed by the isotension technique. At the same time, the auxiliary cable stays were detensioned step-wise. The tension given to the new pairs of cable stays was equal to 80% of the desired one and was increased to 100% only after the transfer of the load of the mobile carriages onto the new segments in order to control tensile stresses. Then the carriages were launched forward for the next operation. When increased from 80% to 100%, i.e. on Tuesday, the tension was corrected to adapt to the results of more precise geometric surveys.
Following this construction process, the cable tensions almost perfectly balanced the loads at each construction step, reducing the bending moments to very low levels.

Fig. 8 Lifting of the steel anchorage box

The levels of the two cantilevers differed by about 3 cm, probably due to the influence of temperature on the geometry, despite the corrections that had been made. Some adjustments were therefore made before closing. After the installation of equipment, the lengths of the cable stays were adjusted to balance the effect of the equipment's weight and to compensate a large part of the predicted creep and shrinkage deformations. This adjustment was adapted in order to correctly distribute the cable tensions and to ensure the correct bridge geometry. The adjustments needed were determined on the basis of a final geometrical survey and measurement of the tensions in the cable stays.

3.2 Control of geometry

The control of geometry during construction aims at reaching both the correct geometry and the correct distribution of forces in the structure. That means that, at the end of construction, there is no bending moment in the pylons and bending moments as low as possible in the deck, under permanent loads. Of course, to have a chance to reach both requirements (i.e. geometry and forces) the fabrication geometry of the deck must be perfect.

The position of each new segment is determined in relative geometry by reference to the previous segment and not in fixed axes because there are many possible causes of uncertainty on the absolute position of the deck (temperature effects, difference of self-weight between the two arms of the cantilever erected from a flexible pier, etc.). Practically, benchmarks are placed at the anchorage points of the stay cables on the two last segments. Then, for the casting of the new segment, the position of the new anchorage point is adjusted in relation with the line joining the two benchmarks in order to reach the required camber, taking into account corrections due to thermal effects during the hardening of concrete. Following this method, the final geometry of the deck without self-weight and without stays is the required one. The geometrical adjustment of each new segment must be very accurate because, after concreting, it is no more possible to bring corrections to the fabrication geometry.

Fig. 9 Stability stays tensioned to balance the weight of the riverside segment

Two methods were considered to determine the adjustments of cables' length introduced in the new stays after launching forward the mobile carriages.

The first method consists in computing, by structural analysis, the deflections produced at the end of the last segment and at the top of the pylon, by a given deformation of the new stays. It is then easy to calculate the length adjustments (i.e. the slide of the tensioning jack) necessary to put the deck at its required absolute profile.

In the second method, the geometrical survey and the measurement of tension in the stays are used to compute the actual neutral length of the new stay (i.e. their length without tension). The length correction introduced aims at giving the new stays their estimated required neutral length (consistent with the structural analysis).

Of course, when the fabrication geometry is perfect and when the loads during construction do not differ from the estimated values, the two methods give exactly the same result. But there are always geometrical imperfections and the load are not perfectly known, so that each method has different advantages and drawbacks.

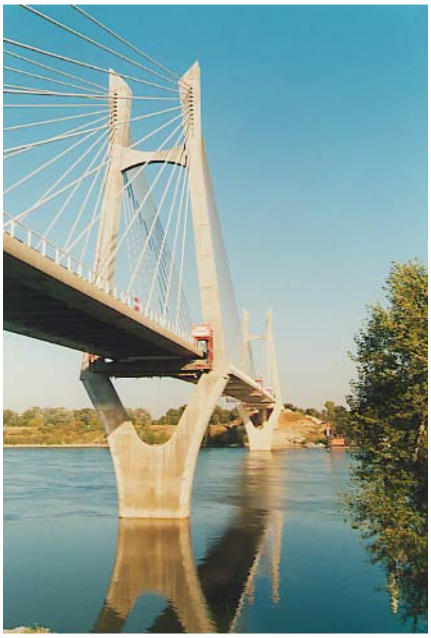

Fig. 10 The completed bridge

The first method allows to compensate differences of loads. So even if the self-weight of the deck is under or overestimated by a few percents, there is no need for additional adjustment of cables' length before closure of the spans. That is why the contractors prefer this method. As a counterpart, it compensates also the uncertainty on temporary loads (uncertainty on the weight of the mobile carriages, but also little construction devices, stocking of cables, anchorages, etc. which are not taken into account by the analysis). When these loads are removed, it can create strong bending moments and additional deflections in the deck. Furthermore, this method compensates the geometrical imperfections without controlling the bending moments in the deck and the tension forces in the stays.

The second method introduces practically no bending in the deck and the stay tensions are regularly distributed. It does not enclose in the deck the bending moments due to not well known temporary loads. But in case of difference of self-weight, it is necessary to adjust the cables' length before closure of the spans.

For the construction, the contractor chose the first method and the values were systematically checked with the second one.

4. Wind analyses

The Mistral wind that blows from the North in the Rhone valley can reach high velocities. Therefore, the reference wind velocity had to be evaluated at the site as an average value over 10 min, 10 m above water, and corresponding to a 50-year return period. It was calculated as 32 m/s.

A section model test gave the aerodynamic stationary coefficients of the profile. From these data, the dynamic response to turbulent wind was evaluated. The results of this analysis, performed before finalising the contract, led to a slight amendment of the construction steps in order to achieve earlier closure of the side spans.

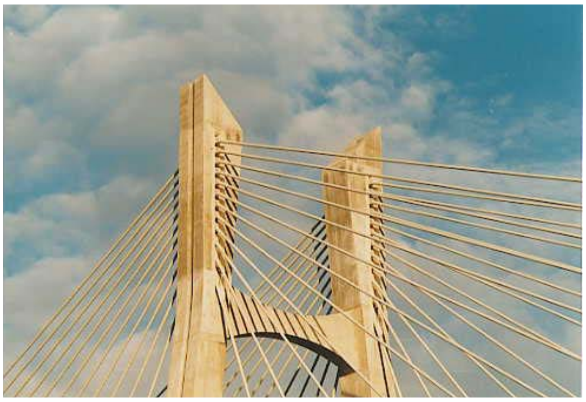

Fig. 11 Pylons' head

The most critical situations, with respect to the wind effects are the construction stages in balanced cantilever. The base of the pylon is high and slender and the vertical component of the wind speed, which is neither constant nor uniformly distributed along the deck, can excite vibration of the whole cantilever in the vertical plane of the structure.

These vibrations create important bending moments in the pylon, and the bending moment is all the more important that the cantilever is long. In order to limit the dynamic effect of the wind, the initial construction steps, planning to erect nine pairs of segments in balanced cantilever before closing the side spans, was modified in order to close the side spans after having built seven pairs of segments. In such a way, the wind effect is not more unfavourable than the accidental fall of a mobile carriage.

Wind analyses were performed during the detailed design using the Scanner computer program, under several construction situations (using a 10-year return period wind velocity) and in operation (50-year return period). These analyses confirmed that the deck construction situation the most unfavourable for the pylon was the seven segments balanced cantilever, just before closure with the cast in place back spans. It was also confirmed that the forces and moments induced by the wind in the deck were negligible compared with the effects of traffic loads. However, it was evidenced that, on the completed bridge, the wind could create important longitudinal displacements. For this reason, it was decided to increase the displacement capacity of the bearings and of the expansion joints.