Millau viaduct : Detailed design of concrete piers

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Summary

The Millau viaduct crossing the tarn valley is a multi cable-stayed spans, with a total length of 2 460 m.

The steel deck is supported by seven piers made of High Performance Concrete B60, the heights of which range from 78 m (P7) to 245 m (P2).

The aim of this paper is to describe the basic and detailed design concepts of the piers.

The wind loads were generally governing the design of the highest piers, and temperature effect the smallest ones. In addition to the specific studies under turbulent wind actions, and general calculation of longitudinal and transversal bending of piers, several structural analysis are developed using 3D finite element models analysis taking into account the elastoplastic and cracking effect of concrete.

Particular analysis are also developed to account for local stability phenomena, and distribution of concentrated loads at the top of piers, under service and construction loads especially temporary supports during launching of the steel deck.

Keywords : Cable-stayed bridge – Concrete piers – Wind action – Temperature Effect – Non linear analysis.

1. Introduction

The Millau viaduct is a 2460-metre-long, 8-spans cable-stayed bridge. Its six main spans are each 342 metres long, and its back spans 204 metres. But it is by its height that it really stands out: it holds the world record for the height of its piers, which range from 78 m (pier P7) to 245 m (pier P2).

From their base to a point 90 metres below the deck, the piers rise as a single hollow shaft, then they are divided into two separate parallel shafts. Since the shafts arrangement is longitudinal, each supporting two spherical bearings, they effectively contribute to the solidarization of the trapezoidal steel box-girder deck which is also stitched to the top of the pier shafts by prestressing cables (four 37T15s cables per bearing) anchored underneath the top part of the pier cap.

Above the deck, the piers are extended by 87–m-high inverted-Y steel pylons supporting a single-plane array of centrally-located stay cables. The feet of each pylon stand atop the twin pier shafts.

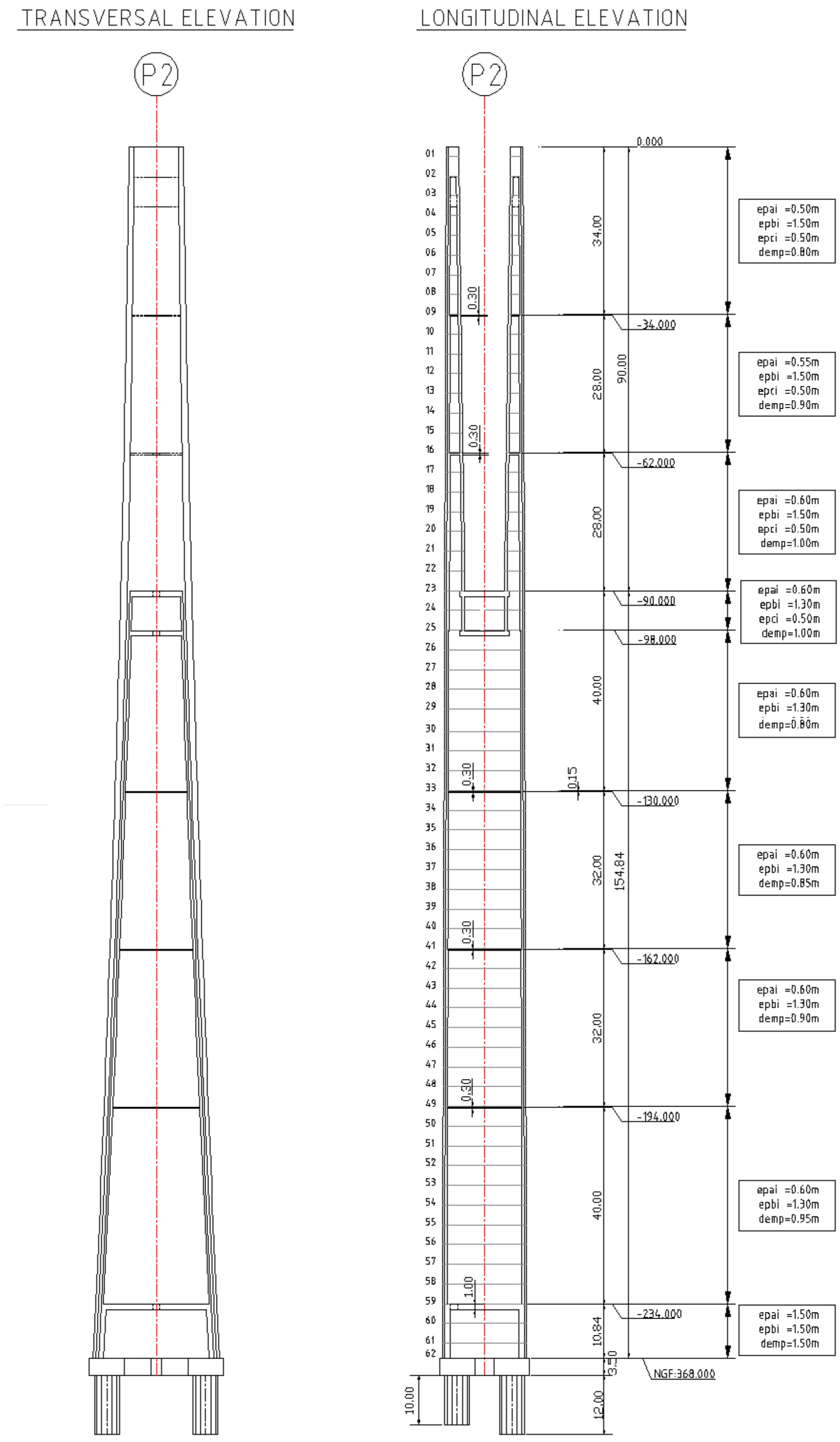

Figure 1: Pier P2 – Transversal and longitudinal elevations

2. Piers

Pier heights vary in accordance with site topography:

Pier	P1	P2	P3	P4	P5	P6	P7
Height (metres)	94.20	244.97	221.06	144.23	136.42	111.94	77.56

Transversally, i.e. in the direction most severely affected by wind, the piers taper parabolically from about 25 metres at the base of the tallest pier (P2) to 11 m. Longitudinally — the direction in which action effects are appreciably less — horizontal dimensions vary relatively little (15.50 m at the top, 17 m at the base), though the two end piers are exceptions.

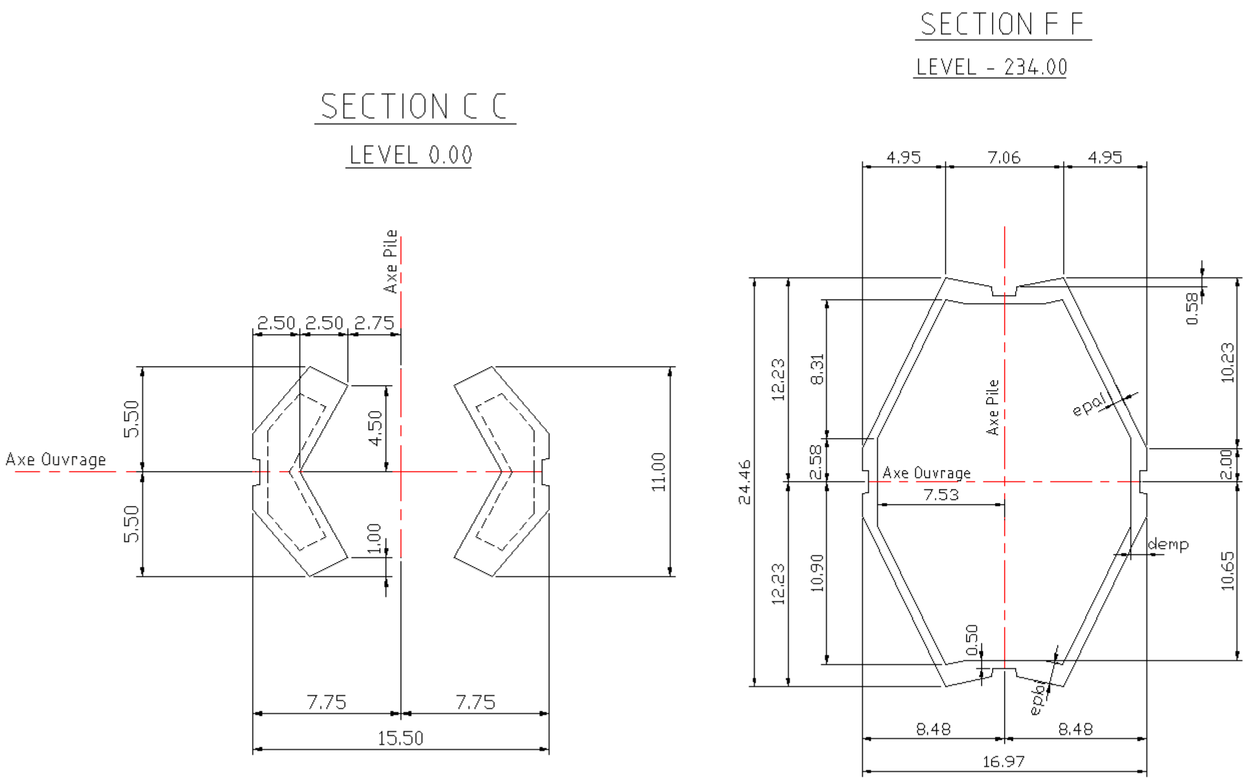

Figure 2: Typical pier cross-sections

Horizontal diaphragms in the hollow-box shafts of both the main pier shaft and twin ones serve as temporary work platforms and also stiffen the thin walls, thereby preventing localized buckling under high compressive stresses. They are routinely 30 cm-thick reinforced-concrete slabs. They are built with precast slabs or concrete beams serving as permanent formwork for cast-in-place topping.

The main shaft transitions into the double shaft over a height of 8 metres. This area is subject to intense stress concentrations, and is strengthened by two 1-m-thick stiffening floors and crosswalls between the two shaft sections.

Pier wall thicknesses vary from 1.50 m to 0.5 m in standard sections. In the bottom 10 metres from the foundation, they are even thicker since they distribute a large part of the pier load takedown into the foundations.

The pier shape is regular all the way to the deck soffit, i.e. there is no widening such as would be the case with traditional pier caps. However, the top of each pier does have a specially massive cap structure to a point about 18 metres below the deck. This exceptional height is due to the deck construction procedure: the top 5.50 metres or so of the pier cap transmit the reaction of bearings; there follows a hollow section serving as an access chamber for the deck stitching cable anchorages; the bottom part of the cap is about 10 m high, i.e. the height necessary for distribution of the reaction of the temporary bearings used during deck launching.

3. Foundations

The marl or limestone rockhead is reasonably close to the surface. The piers are founded on four 9 to 16-metre-deep concrete-filled circular shafts (4.50 m or 5 m diameter). In marl (piers P5 to P7), these are splayed at the base (elephant's feet) in order to reduce the depth of excavation required.

	P1	P2	P3	P4	P5	P6	P7
Pier foundation							
Shaft diameter (metres)	4.50	5.00	5.00	4.50	5.00	5.00	4.50
Base splay ∅(metres)	-	-	-	-	7.00	7.00	7.00
Depth (metres)	9 & 11	10 & 12	12 & 16	10.00	14.00	13.00	14.00
Footing							
Thickness (metres)	3.00	3.50	3.00	4.00	5.00	5.00	5.00
Horizontal dimensions (m)							
Longitudinally	17.50	18.00	18.00	17.50	19.00	18.00	17.50
Transversally	18.50	27.00	25.00	22.50	21.00	21.00	18.50

Table 1: Dimensions of shaft foundations and footings

The four foundation shafts are laid out beneath the pier walls to facilitate the most direct grounding out of vertical loads from the pier shaft. Consequently the pier walls are made thicker towards the base, making it possible to reduce the thickness of the footing.

4. Vertical prestress in piers

The twin pier shafts are each prestressed vertically by eight 19T15s cables running practically totally straight. The dead-end anchorages are at the bottom, and the cables are tensioned from the top of the pier.

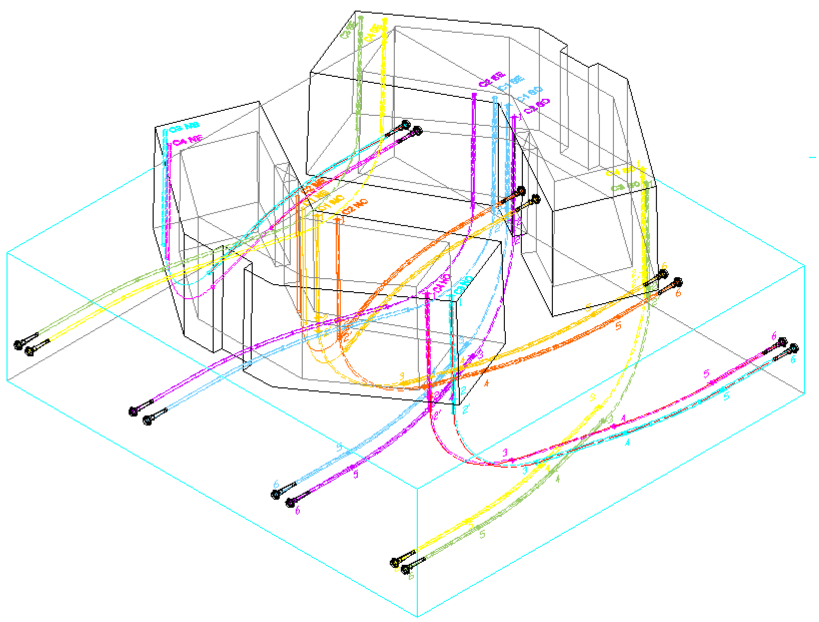

Figure 3 : Pier P7 – Vertical prestress

Prestressing was strongly recommended by the design experts as a means of ensuring greater project durability, by limiting cracking at the bottom of the end piers caused by frequent temperature-induced variations in deck length. By compensating the tensile forces induced by wind buffeting at the top of the twin pier shafts, for serviceability limit state conditions, it also gives the piers better wind resistance.

5. General design assumptions and methodology

The design rules and assumptions used in the structural calculations were based on French codes, complemented in many respects (wind effects, behavioural non-linearity, etc.) by the special technical specifications of the concession holder and by the recommendations of the board of experts set up by the Owner. With respect to durability, the concession contract demanded a design lifetime of one hundred and twenty years.

For verifications of pier design, calculation of reinforced-concrete elements took account of detrimental cracking under rare combinations and highly detrimental cracking under frequent combinations. For the prestressed twin pier shafts, verification was carried out to the Class III standard of the French BPEL limit-state prestressed-concrete design code.

The general methodology of calculations and verification of pier design was broken down into two main phases:

- the first phase consisted in establishing a preliminary design for the shaft flexural reinforcement, based on regulatory combinations of action effects determined from linear elastic analysis, except for rare and extreme temperature effects for which non-linear analysis with imposed displacement was carried out;
- in the second phase, the buckling resistance of the piers was analysed by means of 3D beam modelling (ANSYS) of the entire structure, taking account of material non-linearity (reinforced concrete) and geometrical non-linearity (pΔ effects).

6. Linear analysis – Design action effects

The general actions effects under service conditions were determined with a 3D beam model representing the entire structure and integrating foundation stiffness matrices. The FINELG program was used for detailed modelling of the different parts of the structure. The calculations at this stage considered linear elastic behaviour, and provided the force vector fields at any point of the piers for each elementary loading case (permanent loads, traffic loads, wind effects, temperature effects, etc.).

Assessment of wind effects

Because of the great height of the bridge and the severe wind turbulence encountered at the site, wind-generated forces are a major design consideration for the structure as a whole, and particularly for the piers.

The effects of buffeting were calculated with the spectral method which involves determining the modal response of the structure and then carrying out conventional quadratic combinations of the different modes. The extreme dynamic effects calculated in this way were then added to the static wind effect to determine the global effect of wind action. This method was used to analyse scalar phenomena (bearing reaction, etc.).

On the other hand, to determine the range of biaxial compound bending forces in the pier without having to simultaneously apply the non-concomitant extreme values of the three forces (normal force and two moments), two different approaches were adopted:

a) One approach, adopted by the design consortium, and based on the Leblond elliptical theory, provided 24 envelope action-effect vector fields for each section;

b) The second approach, proposed by the board of experts, involved determining the range of flexural stresses at a vertex of a pier section with two linear mode combinations. This approach has the major advantage of simplifying the definition of equivalent static loads necessary for non-linear analysis of the piers.

Consideration of temperature effects

Given the length of the deck and the fact that it has no intermediate joints, an important feature of its design considerations is the behaviour of the end piers under the effects of varying deck length induced by temperature variations (expansion and contraction). The end piles (P1 and P7) are the most severely affected by this:

Rare temperature	longitudinal displacement at head of piers (cm)	
	Pier P1	**Pier P7**
RT1 = +35°	36	26
RT2 = -40°	41	31

These longitudinal displacement values are practically independent of the non-linear behaviour of the piers, for cracking of the piers has only a slight effect on the position of the deck's thermal fixed point. On the contrary, the action effects induced in the piers are directly related to their stiffness.

In the first phase of calculations the analysis considered a single pier modelled with SETRA's PYLOSTAB program, and takes account of geometrical second-order effects and of the behaviour of cracked reinforced concrete.

Figure 4 shows the evolution of horizontal forces against imposed displacement at the top of the northern shaft of pier P7:

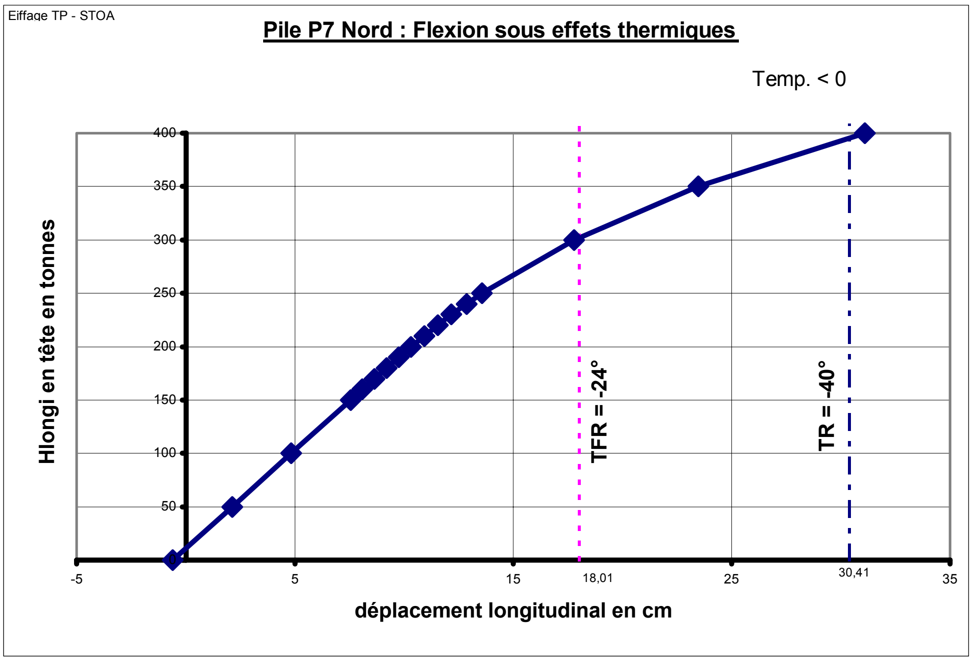

Figure 4 : Pier P7 - behaviour under temperature effects

It shows the different ranges of pier behaviour:

- linear range: linear behaviour up to a certain temperature variation (i.e. imposed displacement),
- non-linear range: post-cracking behaviour of pier, which becomes more flexible.

The end of the linear range corresponds to a temperature variation of more than 15°C. The start of cracking is delayed by the effect of the prestress which also increases the secant stiffness after cracking.

7. Non-linear calculations

The non-linear calculations [see paper enclosed] carried out in the second phase of studies served to analyse the non-linear behaviour of the piers, and particularly to verify pier design with action effects adjusted to take account of the different non-linearities in the overall behaviour of the viaduct.

The method used takes account of the material non-linearity of reinforced and prestressed concrete and the geometrical non-linearity of large displacements.

All the verifications carried out with action effects derived from non-linear analysis showed that the design adopted in Phase 1 of the studies was amply on the safe side, particularly with respect to the risk of buckling of the slender piers.

The longitudinal reinforcement of the piers was also validated. For piers P2 to P6, the mean percentage of reinforcement is 0.8 to 0.9% for the twin shafts and 0.8% for the single shaft. For the two end piers, the reinforcement ratio is substantially higher at the bottom of the twin shafts (1.75% of the sectional area of concrete).

8. Design of pier caps

Since the points where forces are applied in the construction and service phases are totally different, the pier caps had to be designed totally separately for each phase.

In both cases, the calculations were carried out with the strut-and-tie method and were complemented by verifications based on calculation of zones of concentrated and distributed forces (BAEL and BPEL limit-state reinforced and prestressed concrete design codes).

Construction phase study

In the construction phase, deck launching forces are transmitted to the piers by specially designed temporary structures.

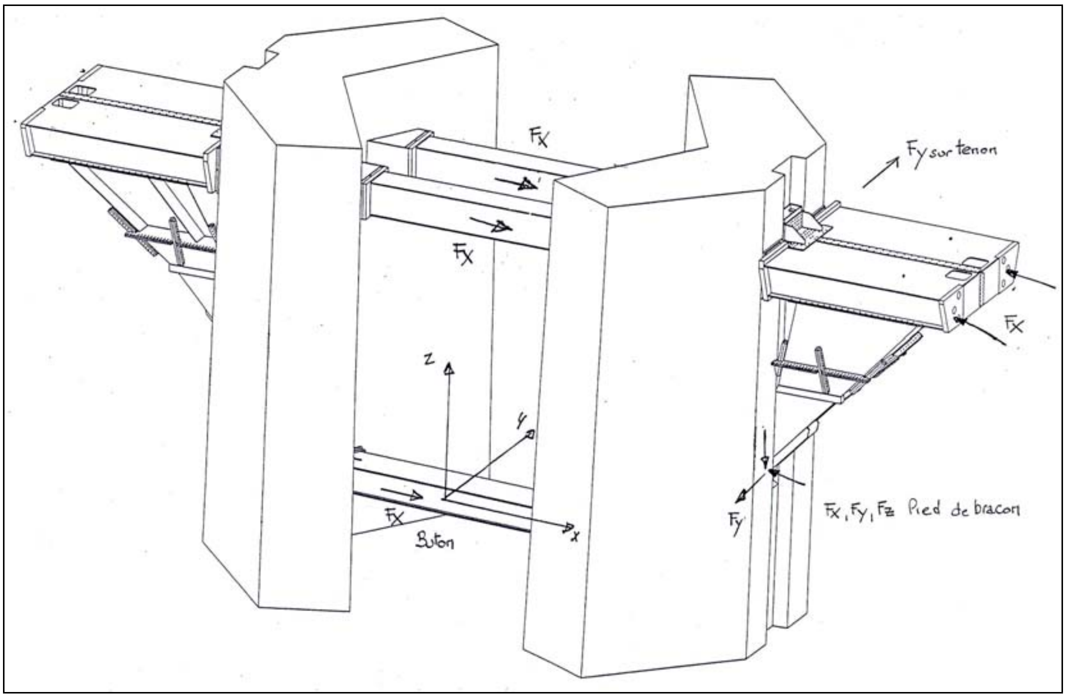

Figure 5 : Diagram of forces applied

Apart from local problems of hoop reinforcement and confinement of concrete beneath the feet of the temporary structure's raking struts, the main problems were the vertical distribution of force between the point of application beneath the foot of the strut and the walls in the standard pier section, and horizontal distribution of Fy forces (wind) by the foot of the strut and the tenon.

Vertical force in the pier cap is distributed in two stages:

- between the foot of the strut and the level of the horizontal brace between the twin shafts,
- between this level and the bottom of the pier cap.

In some phases and under certain wind conditions, horizontal forces perpendicular to the deck can reach very high levels: 15.06 MN ULS at the tenon, which is practically entirely opposed by a force at the foot of the strut (effect of lateral deck tilting under wind load).

In the first design arrangements, the force at the tenon was applied in the vertical groove in the piers and transmitted to the mass concrete of the pier by a short 40-cm-wide vertical 'corbel' (each of the two ridges forming the groove at the apex of the V-section). Because of reinforcement difficulties with this part of the structure and the risks associated with concreting and steel-bending tolerances, all parties involved preferred to transmit the forces to the concrete by means of a special steel component embedded in the concrete, called the "spider".

The same design arrangements were applied for the foot of the strut.

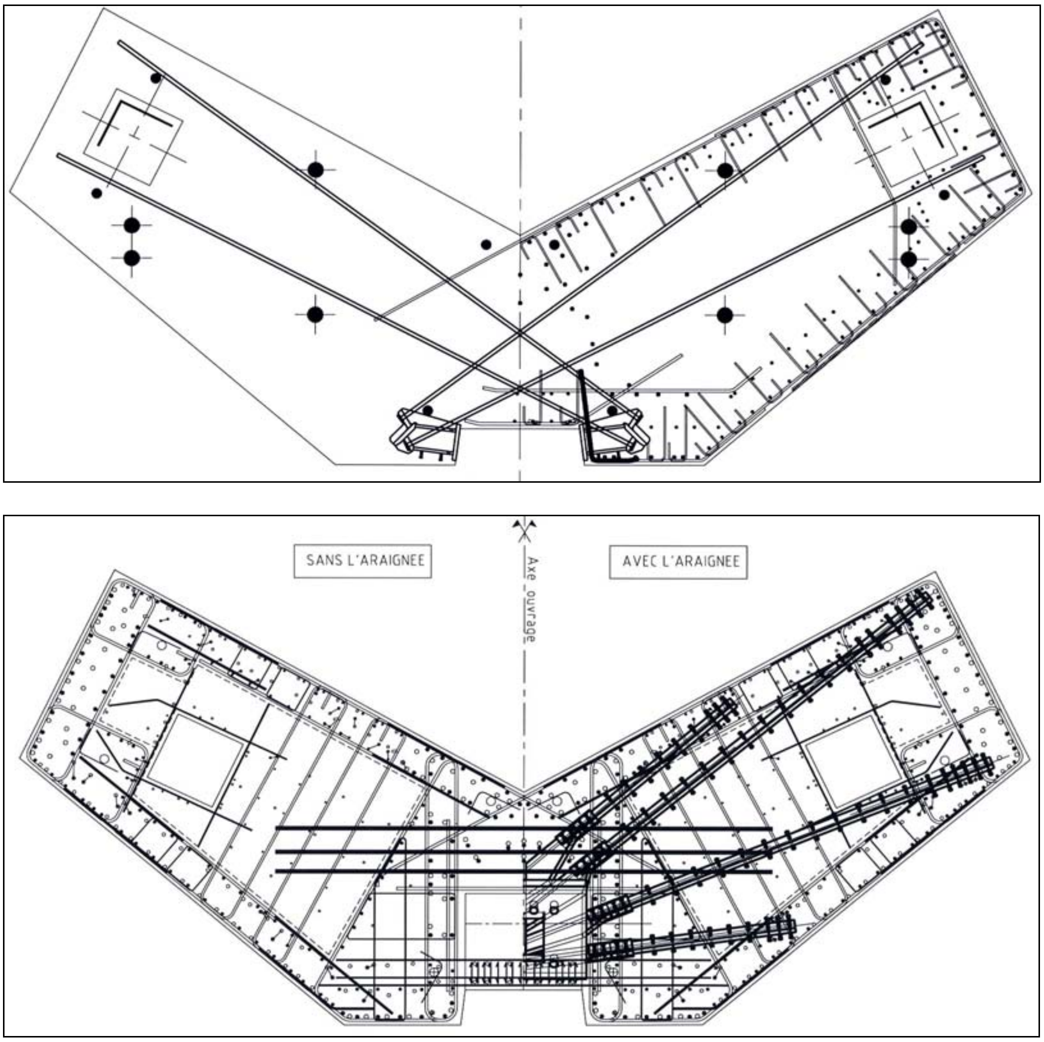

Figure 6 : Top of pier – Embedded steel

The ties are 40 and 50 mm diameter bars for the upper spider and steel flats with studs for the lower spider.

Service phase study

Under service conditions, the top part of the pier cap distributes concentrated forces transmitted by:

- the spherical bearings beneath the deck,
- the 37T15s deck stitching cables,
- and the eight 19T15s vertical prestress cables of the twin shafts, tensioned from the top.

Passive reinforcement was designed using the strut method in conjunction with regulatory requirements concerning zones of distribution of concentrated forces.

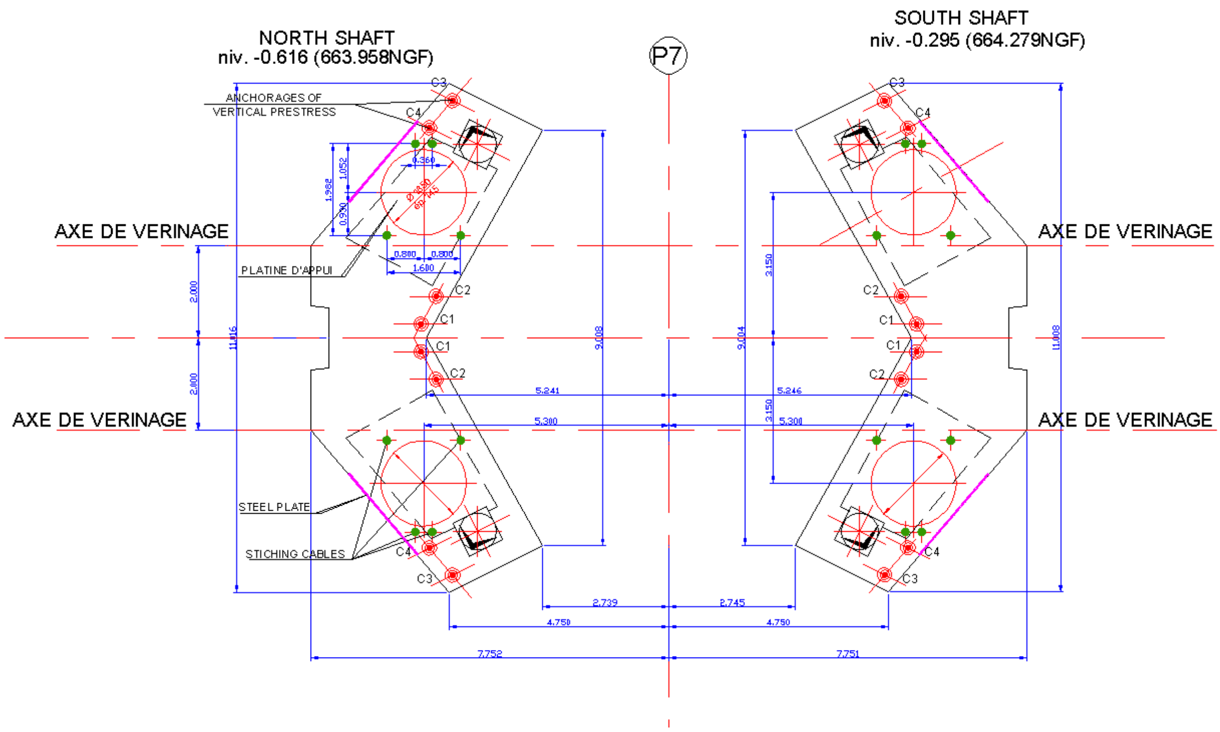

Figure 7 : Layout at top of piers

The maximum vertical reaction of the superstructure is about 9000 tonnes per bearing at the serviceability limit state and 11,500 tonnes at the ultimate limit state. The bottom plate of the bearings is a 150-mm-thick, 1.85-m-diameter steel plate. Shear studs over the entire bottom surface transmit horizontal forces into the pier-cap concrete. It is embedded in a boxout in the last concrete lift.

Given the short distance between the bearing plate and the edges of the pier, plus the intensity of the pressure applied beneath the plate, the outside of the pier is belted with steel (sacrificial formwork). The 30 mm thick, 1.25 m wide steel belt is anchored to the concrete by several layers of horizontal 20-mm-diameter deformed reinforcement bars carefully laid amongst the hoop reinforcement and bursting reinforcement. The concrete subject to high localized pressures is thus perfectly confined and hooped, and the level of safety with respect to distribution of bearing reactions is considerably improved.