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# Strengthening of an Externally Prestressed Bridge

## Renforcement d'un pont à précontrainte extérieure

## Verstärkung einer vorgespannten Brücke

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**SUMMARY**

Recently, on a bridge with totally external prestressing, the end cross-beams became severely cracked before all the prestressing tendons were tensioned. This paper presents the defects observed, their causes and the reinforcement technique applied. It also describes the calculation method, with a strut-and-tie model, developed for the justification calculations of this part of the bridge. Ideas for the design of new externally prestressed bridges are also given.

**RÉSUMÉ**

Récemment, les entretoises d'about d'un pont à précontrainte totalement extérieure se sont largement fissurées avant l'application de la totalité de la précontrainte. Le présent article présente les désordres constatés, leurs causes et la technique de renforcement qui fut utilisée. Il décrit également la méthode de calcul, basée sur un modèle bielles-tirants, qui a été développé pour le calcul justificatif de cette partie d'ouvrage. Des conseils sont également donnés pour la conception des ponts à précontrainte extérieure.

**ZUSAMMENFASSUNG**

Vor kurzem traten noch vor Beaufschlagung der gesamten Vorspannung breite Risse in den Endquerträgern einer komplett extern vorgespannten Brücke auf. Vorliegender Artikel erläutert die festgestellten Mängel, deren Ursachen sowie die zur Verstärkung eingesetzte Technik. Er beschreibt weiterhin die Berechnungsmethode auf Basis eines Zugstangenmodells, das zur Nachweisberechnung dieses Bauteils entwickelt wurde. Des weiteren werden Ratschläge zur Konzeption von Brücken mit externer Vorspannung erteilt.

## 1. INTRODUCTION

In France, external prestressing is now very widely used in large bridge design. Indeed, this technique has many advantages : structures can be lighter in weight, friction prestressing losses are low and, above all, prestressing reinforcements can be replaced if necessary.

The specificity of external prestressing is sometimes forgotten at the design stage, leading in some cases to regrettable design errors. One of its particular aspects, calling for close attention in bridge design, is the diffusion of anchoring loads or external tendon deviation loads through the structure.

In 1983, on the Aiguilly bridge, one of the first French externally prestressed bridges, the anchoring block, broke away several hours after tensioning. Since then, similar incidents have occurred on other externally prestressed bridges. Recently, on a bridge with totally external prestressing, the end cross-beams became severely cracked before all the prestressing tendons were tensioned.

This article presents the case of this bridge, the defects observed, their causes and the reinforcement technique applied. The article also describes the calculation method, with a strut-and-tie model, developed for the justification calculations of this part of the bridge. Advices for the design of new externally prestressed bridges are also given.

## 2. INITIAL CONSTRUCTION PROJECT AND DEFECTS OBSERVED

### 2.1 General bridge characteristics

The bridge is a launched structure comprising ten spans with an average length of 52 meters. The deck is a box girder measuring 10 m in width and 3.40 meters in height.

The bridge is entirely prestressed by means of super 19 T 15 external tendons running along one or two spans and which belong to the following families :

- undulating tendons, deviated one-third of the way along each span,
- temporary antagonistic undulating tendons,
- straight tendons running from one support to the next.

Deck prestressing was centered during launching and comprised four deviated undulating tendons, four antagonistic undulating tendons and four straight tendons. At the end of launching, the antagonistic tendons were detensioned and replaced by four normally deviated tendons. A pair of empty ducts was also provided for subsequent prestressing reinforcement.

The following quantities of tendons, anchored at the deck extremities, were thus provided :

- twelve tendons during the launching phase,
- twelve tendons after the launching phase,
- fourteen tendons in the event of tensioning of reinforcement tendons.

The prestressing load to be anchored was therefore very high: each tendon exerted a load of 330 tonnes before deferred losses, making a total load of 4600 tonnes at each end.

### 2.2 Characteristics of the end cross-beam and defects observed

In the construction project, the designer planned to anchor the extremities of all the tendons in abutment cross-beams with a thickness of no more than 1.70 m (photo 1).

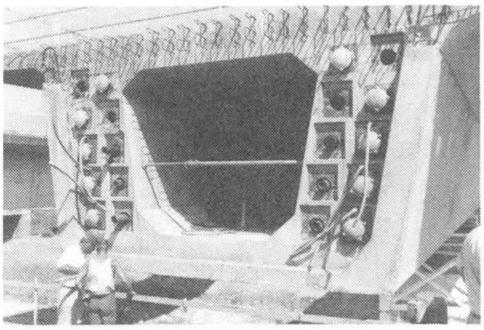


photo 1

The bridge was built according to this design and after casting the formwork was left on the cross-beams to simplify work-site operations. Substantial cracking, localized in two zones (figure 1) was discovered when the formwork was removed :

- on the cross-beam itself, on the lateral faces and on the face opposite of the cross-beam anchorages,
- on the outside of the box girder, along the entire height of the web.

The cracks were mainly horizontal and the largest of them had an opening of 0.2 mm. There was around one crack per prestressing anchorage level. Considering that a reinforced concrete part is liable to rupture when crack opening exceeds 0.4 mm, there was little remaining margin. When these defects were discovered, the final tendons had not yet been tensioned, an action that was bound to further aggravate the situation. Work was therefore suspended and a reinforcement project was devised.

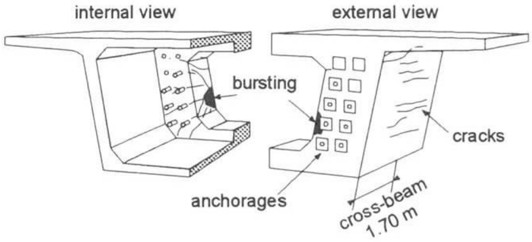



figure 1

### 2.3 Criticism of initial design

Clearly, the cross-beam on which the tendons were to be anchored was too thin. A length of 1.70 m, corresponding to half the box girder height, is not sufficient to transmit anchoring loads effectively to the box girder. We think that a thickness in the order of magnitude of the deck height (3.40 m) would have been more realistic.

Other factors also had a negative influence. The first problem was the large number of tendons: in general, three to four external tendon pairs are anchored on an end cross-beam. Here there were twice this number.

Secondly, the anchoring points were poorly distributed over the cross-beam: the tendons must be anchored, whenever possible, all around the box girder, as close as possible to the webs and slabs to ensure direct transmission of anchoring loads to the box girder. In this case, the tendons were anchored in two lines arranged at the center of the cross-beam uprights. Some of the anchoring points were thus located almost 90 cm from the box girder walls.

A further aggravating factor was the absence of transverse prestressing - both horizontal and vertical - which would have offset diffusion loads. It is often judicious to use additional prestressing to offset the tensile loads produced by the diffusion of prestressing loads so as to avoid excessive loads on the cross-beam and its subsequent cracking.

Lastly, the quantity of reinforcement required was substantially underestimated, as we will see below.

We wish to point out to designers the importance of taking into account, at the design stage, all these aspects associated with the diffusion of prestressing loads. A number of articles have already discussed this problem [1].

2.4 Criticism of the cross-beam justification in the initial design

In order to design and justify the cross-beam, the designer used the rules given in Appendix 4 of the French prestressed concrete rules [3] concerning "zones of application of concentrated forces". In fact, these rules only cover the case of end members with a cross-section identical to the standard deck cross-section, as is the case for internally prestressed decks. In the present case, the standard deck cross-section is very different from the end cross-section and these calculation models cannot be applied.

By applying Appendix 4, as well as including correct quantities of steel surface and rupture reinforcement immediately behind the anchoring points, the designer provided for "general equilibrium" reinforcement comprising (figure 2) :

- distributed vertical reinforcements: $A_v = 285\ cm^2$
- distributed horizontal reinforcements: $A_t = 345\ cm^2$ (per cross beam)

We will see below the serious inadequacy of this reinforcement.

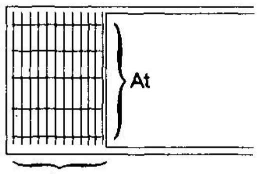


figure 2

## 3. RECALCULATION OF THE INITIAL DESIGN

In this paragraph, we will examine the various calculation methods used for expert assessment of the defects observed.

3.1 Calculation of the type "deflected wall resting on two supports"

One of the calculation methods applicable is that of Appendix E.5 of the French reinforced concrete rules [4]. This method stems from a strut-and-tie planar calculation model validated by testing.

Here, the cross-beam is considered as a diaphragm attached to the upper and lower slabs (figure 3). In fact, only a part of the prestressing loads are transmitted as shown in the diagram, the rest being diffused directly towards the web. This share of total load can easily be evaluated by assuming that the distribution of stresses leaving the cross-beam is more or less linear. Here, it is 50%, the equivalent of five times 330 tonnes.

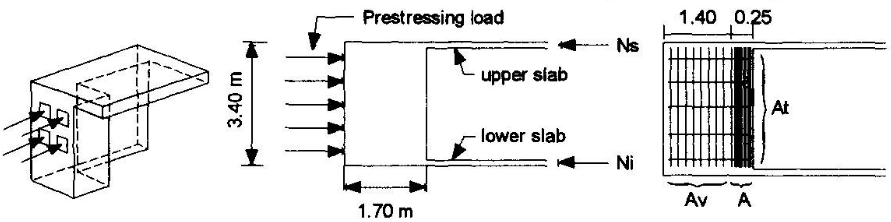


figure 3

The application of Appendix 5 of the BAEL [4] for diffusion in the vertical plane produces the following steel reinforcement requirements:

- principal reinforcement : $A = 235\ cm^2$
- secondary reinforcement : $A_v = 355\ cm^2$
- transverse reinforcement : $A_t = 230\ cm^2$

Comparison with existing steel reinforcement (see paragraph 2.4) reveals a severe lack of vertical reinforcement. Moreover, the vertical reinforcement should have been concentrated on the face opposite the anchoring points, where the cracks were observed.

The simplified calculation model that we have presented cannot be used to determine diffusion in the horizontal plane because the cross-beam is perforated. We therefore developed more representative calculation models to perform this task.

3.2 Finite-elements calculation of the cross-beam

We used a finite-elements method to gain a clearer picture of the cross-beam's mechanical behaviour. The calculation is imperfect, since it is elastic and does not take account of cracks in the concrete, but nevertheless gives a good idea of the loads exerted on the concrete.

Our spatial model concerned a transverse half-deck (symmetrical along the axis). It measured 5.10 meters in length (figure 4). Only the effects of prestressing were considered.

The graphical representation of stress isostatics inside the girder gives a good picture of behaviour (figure 5):

- behind the anchoring plate, loads spread through the cross-beam mass,
- a part of the load, mainly from the lines of anchoring points close to the web, goes directly to the web,
- the other part of the load rises to the upper slab or goes down to the lower slab via the upright.

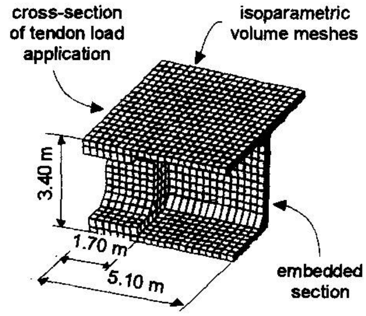
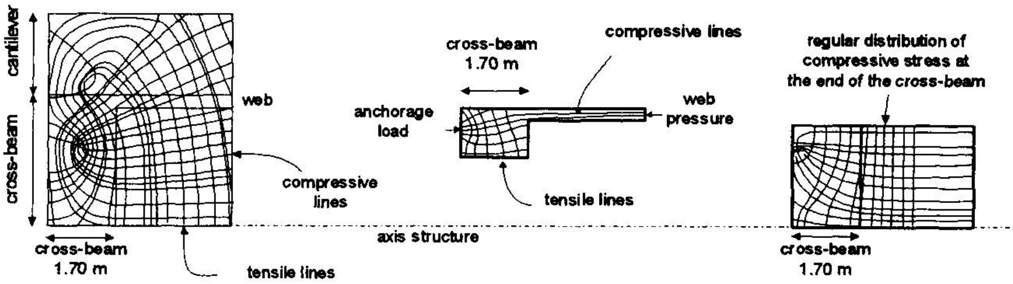


figure 4




*horizontal section of the upper slab* *horizontal section at mid-height of the box girder* *horizontal section of the lower slab*

figure 5

When the deformation of the part is visualized, the zones under greatest tension, exactly where the cracks occurred, are revealed. These areas are the back of the upright and the outside of the web (figure 6). We observe a "banana" shaped curvature of the uprights which explains why the web cracks occurred on the outside of the box girder and not the inside. Moreover, on these outer sides, more thermal and shrinkage effects are superimposed, resulting in more extensive cracking.

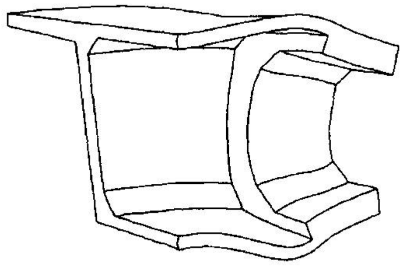


figure 6

The finite-elements model also gives an interesting indication of the tensile loads in the concrete. They reach 7 MPa, three times the tensile strength of concrete. Clearly, the concrete is bound to crack, whatever the quantity of reinforcement used. To prevent this cracking, tensile loads should have been offset by local transverse prestressing of the cross-beams.

As the finite-elements calculation we have just presented is not sufficient to determine stresses in the reinforcements, an additional strut-and-tie model was used.

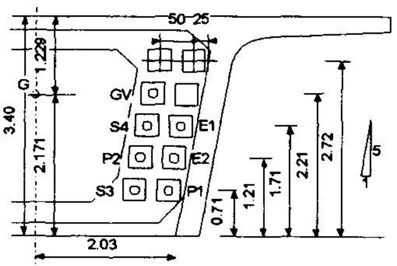



figure 7

### 3.3 Strut-and-tie model

We chose to build the strut-and-tie model to study the phase of greatest stress, when all the final tendons and the two reinforcement tendons are tensioned, making a total of 7 tendons pairs in all. (figure 7).

A network of principal struts represents the struts transmitting loads from the anchoring points to the box girder. We associated a strut with each anchoring point, and each of these struts goes, via the upright, towards a portion of the box girder whose surface area is chosen to offset exactly the load of an anchoring point in service, i.e. 330 t (figure 8). These struts are balanced transversally by two networks of struts and ties, one on the front face of the upright, on the box girder side, the other on the back face, on the anchoring point side. The organization of these struts stems from information obtained with the finite-elements model. Figure 9 gives a view of the cross-linked model obtained, which is isostatic.

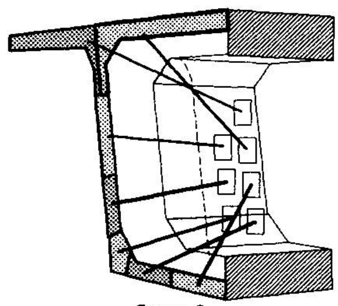


figure 8

The calculation allows to determine the intensity of the general equilibrium loads. The table below gives the quantities of reinforcements needed to ensure overall equilibrium in two situations: a 1.70 m cross-beam (as in the original project), and a 3.20 m cross-beam.

| | | 1.70 m cross-beam | 3.20 m cross-beam |
|---|---|---|---|
| Principal reinforcement | A | 246 $cm^2$ | 133 $cm^2$ |
| Transverse reinforcement | $A_t$ | 255 $cm^2$ | 138 $cm^2$ |

In both cases, stress in the steel reinforcements was limited to $2/3.f_e$, i.e. 267 MPa.

We found once more that the initial vertical reinforcement was greatly under-estimated and that a larger cross-beam would have substantially improved the situation.

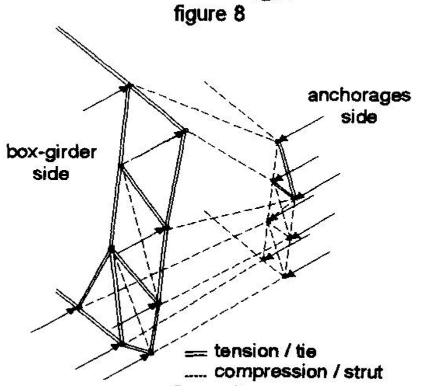



figure 9

## 4. REINFORCEMENT PROJECT

These calculations clearly indicated that the cross-beam, as originally constructed, would be unable to withstand the loads involved. Reinforcement was therefore necessary.

### 4.1 Reinforcement principle

A reinforcement block was added in front of the existing cross-beam to take up the loads exerted by the six tendons still to be tensioned.

In order to attach this reinforcement block to the existing box girder, it was necessary to provide:

- additional horizontal transverse prestressing and embedded seam reinforcements for the webs;
- seam reinforcements and four concrete keys in the upper slab;
- seam reinforcements over the entire interface of the lower slab.

Lastly, vertical prestressing was also added to the webs to prevent any extension or cracking.

### 4.2 Detailed description of the reinforcement block

The cross-beam reinforcement block comprises a U-shaped framework, measuring 1.5 meters in length, added to the inner side of the cross-beam (figure 10). Concreting was carried out from inside the box girder, but also through four openings made in the upper slab which served as a transmission key for loads after hardening of the concrete.

Transverse prestressing is provided by nine 7 T 15 internal tendons, 5 of which pass through the lower part of the reinforcement block and 4 through the upper part. These tendons are curved to create a thrust which partly offsets the thrust of the external longitudinal tendons (figure 10). They are anchored on concrete anchoring blocks outside the box girder.

The seam reinforcements used are as follows:

- 28 bars of 20 mm in diameter on the lower slab,
- 5 bars of 16 mm in diameter on the upper slab,
- 37 bars of 16 mm in diameter in the webs.

These reinforcements are embedded in holes drilled in various parts of the box girder. The depth of embedding is adapted to the steel diameter, in compliance with known test results [5].

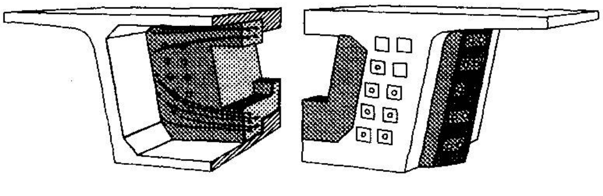


figure 10

### 4.3 Design of the reinforcement block

Here we will detail the two major points that were examined:

- the inherent strength of the cross-beam reinforcement block and its steel reinforcements;
- the attachment of the block, and the external anchoring blocks, to the box girder.

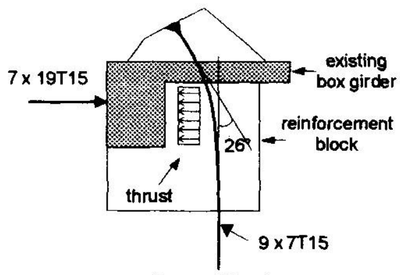



figure 11

#### 4.3.1 steel reinforcement of the cross-beam reinforcement block

We calculated the principal steel reinforcement of the block using a strut-and-tie model similar to the one described above.

To determine the thrust exerted on the block, the two following factors were taken into account:

- firstly, the fact that the longitudinal prestressing already in place before concreting of the block, comprising four 19T15 tendons per web, is already taken up by the existing cross-beam. We nevertheless estimated that 50% of this prestressing, i.e. two tendons per web, would, over the long term, diffuse to the cross-beam reinforcement block, as a result of creep and relaxation;
- secondly, that the thrust of the transverse reinforcement block tendons directly offsets the thrust of one 19 T 15 tendon (figure 11).

Finally, we considered that the reinforcement block supported only 57% of the total thrust of the seven longitudinal tendons per web, i.e. 1300 tonnes.

The quantities of steel reinforcement, positioned on the front face of the reinforcement block (box girder side), are 80 $cm^2$ per web.

#### 4.3.2 Reinforcement block/ box girder attachment

Loads are transmitted from the reinforcement block to the box girder by concrete-to-concrete friction and by a pin effect of the embedding reinforcements.

To obtain a good friction coefficient, the surfaces were roughened to obtain indentations of at least 5 mm.

Sliding resistance was verified with respect to ultimate limit states using the following generalized seam rule [6]:

$$1{,}35\,P_m \le \left(0{,}85\,N_P + A_{st}\frac{f_e}{\gamma_s}\right)\frac{\varphi}{1{,}2}$$

where $P_m$ is the sliding load due to longitudinal prestressing, $N_p$ is the normal prestressing load at the slide surface, $A_{st}$ is the total surface area of seam reinforcements, $f_e$ is their elastic limit, $\gamma_s$ equals 1.15 and $\varphi$ is the concrete/concrete friction coefficient after setting, fixed at 1.2 in view of the indentations.

It should also be noted that for the upper slab, the shear strength of the keys partly offsets the sliding load.

For verification of external anchoring block bonding, the loads $P_m$ (destabilizing) and $N_p$ (stabilizing) both come from the transverse prestressing. The resistance condition is therefore written:

$$P_m \le \left(N_P + A_{st}\frac{f_e}{\gamma_s}\right)\frac{\varphi}{1{,}2}$$

4.4 Vertical web prestressing

Of course, the ideal solution would have been to use vertical prestressing, similar to the transverse prestressing, to fix the reinforcement block to the box girder, but this was not possible due to the execution problems involved. Vertical web prestressing was nevertheless placed at the end of the cracked zone in order to halt the propagation of the horizontal web cracks which would have inevitably occurred as a result of shrinkage, thermal effects and tensioning of the new longitudinal tendons.

Five solutions were considered (see figure 12).

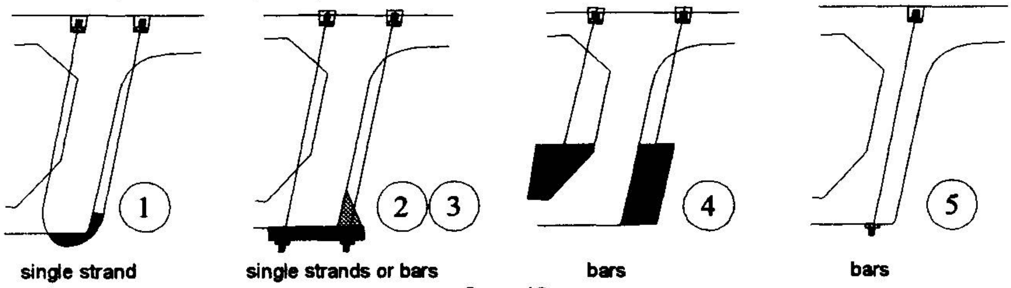



figure 12

Finally, the contractor installed three Ø 36 bars inside the webs, spread longitudinally over a distance of 1.5 m (figure 13). These bars were designed to counteract a stress equal to $f_{tj}$. To limit the risk of crack propagation during drilling of the web, drilling started at the point farthest from the abutment - above the uncracked zone - and then progressed towards the abutment. Moreover, each bar was tensioned before the next hole was drilled.

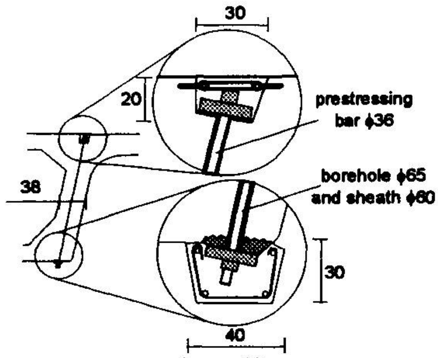



figure 13

## 5. CONCLUSION

The repair of this bridge cost two million francs (US $ 400,000). It also prolonged construction time by two months. From an aesthetic point of view, the consequences were less dramatic : the external anchoring points were decorated with colored paint and only the twelve covers of the vertical prestressing bars mar the appearance of the deck underside.

It is nevertheless clear that this incident could have been avoided at the project design stage if the designer had respected the following basic rules, specific to external prestressing tendon anchoring zones:

- massive parts should be used to ensure progressive distribution of stresses;
- loads should be transmitted as directly as possible from the anchoring points to the deck;
- diffusion loads should be offset by means of a simple set of struts and ties, avoiding unwanted bending and torsion loads;
- tensile stresses in the concrete should be limited by means of transverse prestressing to minimize the cracking which is inevitable if only ordinary reinforcements are used. Indeed, it is difficult to control cracking in a complex part, designed on the basis of imperfect models, especially with superimposed thermal effects and shrinkage.

## ACKNOWLEDGEMENTS

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