CONSTRUCTION OF THE FIRST ROAD BRIDGES MADE OF ULTRA-HIGH-PERFORMANCE CONCRETE

Ziad HAJAR, PhD, Senior Project Manager, Eiffage Construction, France
Alain SIMON, Project Manager, Eiffage Construction, France
Daniel LECOINTRE, **Deputy Head of Large Bridges Division,** SETRA, France
Jérôme PETITJEAN, Senior Project Engineer, SETRA, France

ABSTRACT

The two bridges described in this paper represent a world first for road bridges. Their decks are an assembly of five π-shaped precast beams made of UHPC. The beams are longitudinally pretensioned with bonded strands and are jointed longitudinally and transversally with UHPC.
The concrete used has a compressive strength in excess of 24,700 psi (170 MPa) and a direct tensile strength of 1,200 psi (8 MPa). The fibre content is 3%. The steel fibres mean the concrete is not brittle and replace reinforcing steel, especially for transverse bending behaviour where only the FRC takes part.
Special experimental methods for characterizing FRC properties were also used to get pertinent design data. This application also required determining special calculation methods and design rules which are not currently covered by codes for the type of concrete employed.

Keywords: UHPC, Fibre, Prestressed Concrete, Precast Beam, Direct Tensile Strength

1 - BACKGROUND TO THE OPERATION

The bridges discussed in this article are overpasses on the Bourg-lès-Valence bypass in France's Drôme region (south-east). They are road bridges made from Ultra-High-Performance fibre-reinforced Concrete (UHPC), each with two spans of about 72 ft (20 m). These bridges are a world first, given that UHPC has previously been used only for footbridges and for beams in the cooling towers of two French nuclear power plants.

The Owner of the bridges is the French Government, represented by its Regional Department of Public Works for the Drôme district which also oversaw design and construction with the assistance of two more government agencies, the Service d'Etudes Techniques des Routes et Autoroutes (SETRA) and the Centre d'Etudes Techniques de l'Equipement (CETE) of Lyon. The bridges were built by contractor Eiffage Construction after a performance-based invitation to bid drawn up under an "Innovation Charter" signed by the National Roads Department (Direction des Routes) and the National Public Works Federation (Fédération Nationale des Travaux Publics). This charter aims to promote innovation in the field of bridges, cut-and-cover tunnels, and retaining walls.
The tender process stressed the aim of implementing solutions that take maximum advantage of precasting and/or industrialization of construction processes, and also required solutions that can be adapted to all traditional shapes of bridges and related structures.

2 - BRIDGE DESCRIPTION

In plan view the bridges (OA4 and OA6) are straight and unskewed. Their vertical alignments follow constant slopes of 0.75% (OA4) and 0.5 % (OA6). Their supports (pier and abutments) are of conventional design and are made from 4350 psi (C30) reinforced concrete. The foundations are shallow (Fig. 1).
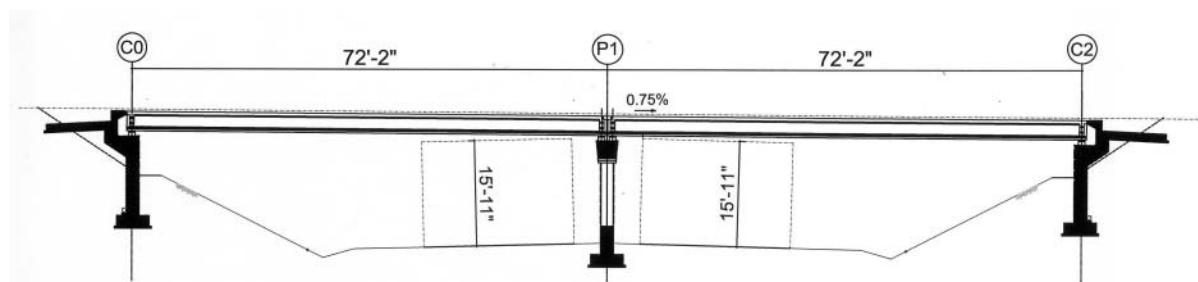

Fig. 1 - Longitudinal cross-section of OA4

Longitudinally the structure is based on a conventional overpass design using prestressed beams.
Each deck supports a 29.5 ft (9 m) wide road pavement with 3.3 ft (1 m) and 6.6 ft (2 m) wide sidewalks (figure 2). Transversally both decks are identical; they are made from an assembly of five π-shaped precast beams made from BSI "Béton Spécial Industriel" (special industrial concrete) concrete (Fig. 2 and 3). The beams are cross-braced at the supports only.

Each line of bearings has 10 blocks receiving tapered shims beneath the flanges of the beams in order to adjust the level of the deck. Each bearing block has a jacking base on either side to enable the bearings to be replaced by jacking against the crossbeams to raise the deck.
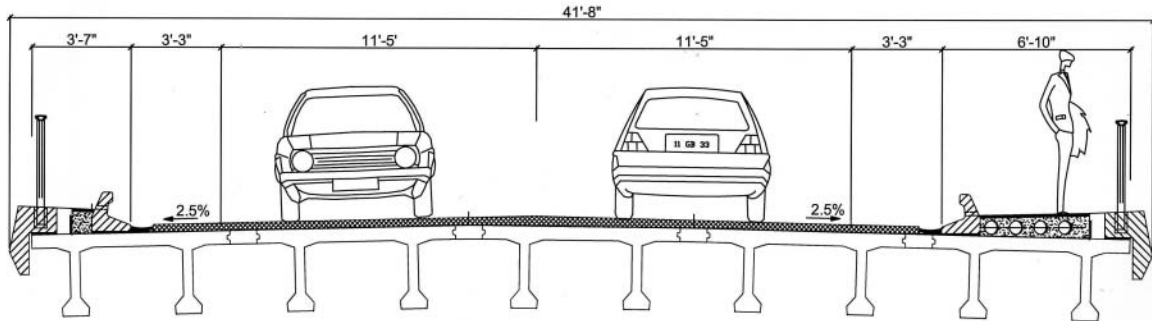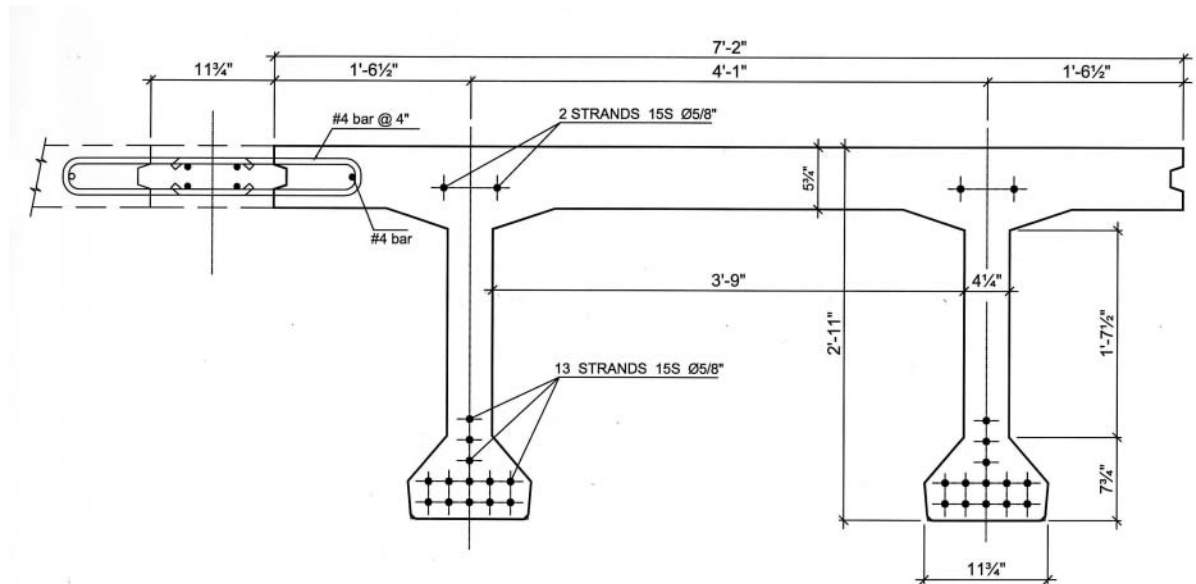

Fig. 2 - Typical cross-section

Fig. 3 - Cross-section of a beam for the OA4 bridge

The road deck was made continuous by placing in situ UHPC between the two simple spans, i.e. at the central pier. The π-shaped beams were also jointed together longitudinally with in situ UHPC.

All the beams are prestressed by pre-tension, using very low relaxation T15 Super strands of strength class 270,000 psi (1860 MPa). Each π-shaped beam of the OA6 bridge has twenty-six strands, with thirty in the beams of the OA4 bridge. There is no transverse prestress.

In addition, the BSI beams have no passive reinforcement, except where components are jointed together transversally or longitudinally and where equipment (pavement joints and safety barriers) is attached.

The equivalent thickness of the deck is 9.8 in (0.25 m), compared to 29.4 in (0.75 m) for a conventional prestressed slab bridge and 14.6 in (0.37 m) for HPC decks. Use of UHPFRC therefore divides the selfweight of the beams by about 3. This weight saving, which for this project was appreciated when the beams were being handled, could in future be a decisive factor for medium-span or long-span bridges.

3 - BSI CONCRETE

BSI concrete is an Ultra-High-Performance fibre-reinforced Concrete (UHPC) whose mix design is principally characterized by a high cement content, use of silica fume and small-diameter aggregate, and a low w/c ratio (Table 1).

Large quantities of steel fibres are used (3% by volume for the Bourg-lès-Valence bridges) in order to give the concrete good ductility under tensile stress and, in most cases, to dispense with the need for passive reinforcement.

Cement	1,877 lbs *(1,114 kg)*
Silica fume	285 lbs *(169 kg)*
0 - 6 mm Aggregate	1,806 lbs *(1,072 kg)*
Fibres	394 lbs *(234 kg)*
Superplasticizer	67 lbs *(40 kg)*
Water	352 lbs *(209 kg)*
W/C ratio	0.19

Table 1 – Mix design per yd^3 *(per m^3)*   Photo 1 – Steel fibres

The fibres used are straight (0.8 in (20 mm) long, 12 mils (0.3 mm) diameter) and work by bonding with the cement matrix. They are made from high yield steel : σ_E = 174,000 psi (1200 MPa).
The rheological behaviour of fresh BSI is rather special: it is a viscous fluid and is self-levelling, with slump flow of 25 in (63 to 64 cm) with the DIN cone and no vibration of the table. This means no vibration is necessary to work the concrete into the forms.

Photo 2 – DIN cone slump flow

Its practical working life can be adapted to suit the requirements of the works. For example, the BSI used for jointing the Bourg-lès-Valence bridges had to have a working life of at least one hour to cover the time for transport and placement.

After 28 days, without heat treatment, the BSI for this project had the following characteristics:

28-day characteristic compressive strength (f_{ck})	25,380 psi (175 MPa)
28-day characteristic direct tensile strength of the matrix (f_{tk})	1,160 psi (8 MPa)
28-day characteristic post-cracking direct tensile strength (σ_{bt})	1,320 psi (9.1 MPa)
Modulus of elasticity	9,280 ksi (64 GPa)
Density	178 lbs/ft^3 (2.8 t/m^3)

Table 2 – Characteristics of BSI

This data was complemented by tests to characterize post-cracking tensile behaviour by measuring crack widths (direct tensile-strength test on notched cylinders or centre-point flexural tensile strength test with notched prisms) in order to determine a complete law for structural design purposes (Fig. 4).
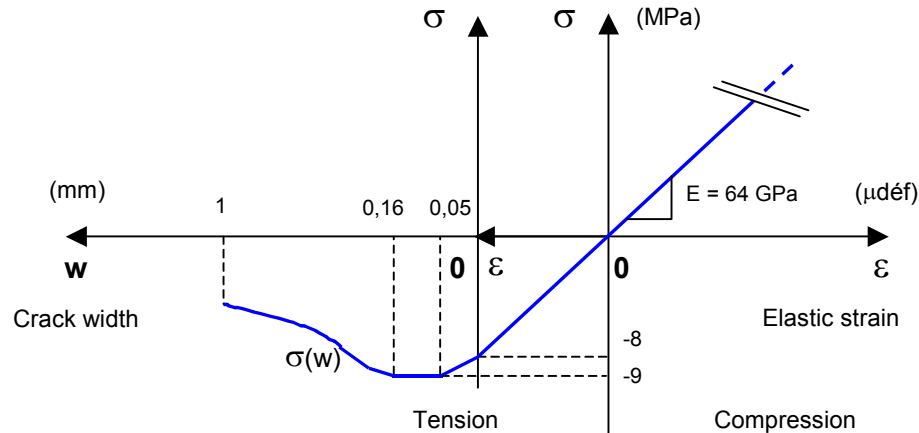
Fig. 4 - BSI constitutive law

4 - DESIGN VERIFICATIONS

4.1 – LONGITUDINAL BENDING

For Serviceability Limit States (SLS) the verifications under normal stresses are similar to the BPEL specifications[1], with the following allowable stresses:

- Compression limited to $0.60f_{cj}$ during construction, $0.50f_{ck}$ under permanent loads, and $0.60f_{ck}$ for rare combinations.
- Tension at any place limited to f_{tk} for rare SLS. No tension was allowed for frequent combinations. During construction, tension is limited to $0.70f_{tj}$.

where :
- f_{ck} and f_{cj} are the 28-day and j-day characteristic compressive strengths (Table 2).
- f_{tk} and f_{tj} are the 28-day and j-day characteristic direct tensile strengths of the matrix.

For Ultimate Limit States (ULS) the ultimate resistant moment of cross-sections was calculated in accordance with the BPEL rules, with a conventional program for design of cross-sections, taking a perfect elastoplastic law for compression and overlooking the concrete's tensile strength.

Overlooking the tensile strength of UHPC puts things on the safe side, but is not unreasonable for design verifications for prestressed concrete in which tensile forces are for the most part taken by overtension in the prestressing tendons.

4.2 – TRANSVERSE BENDING

The verifications for transverse bending are somewhat different, given that in this case there is no passive or active reinforcement other than the steel fibres.

Tension in the concrete is limited to f_{tk} for rare SLS combinations, and to $0.80f_{tk}$ for frequent combinations. These provisions are intended to limit the risks of cracking of the structure which can be subject to significant cyclic loading.

At Ultimate Limit States the resistance moment is calculated in accordance with the AFREM recommendations[2], taking account of the constitutive law of the cracked concrete. This resistance moment of the cracked section is calculated with a special program using a discretized approximation of the constitutive law for the actual tensile behaviour of the material and reflecting the equilibrium of the cracked section (Fig. 5).

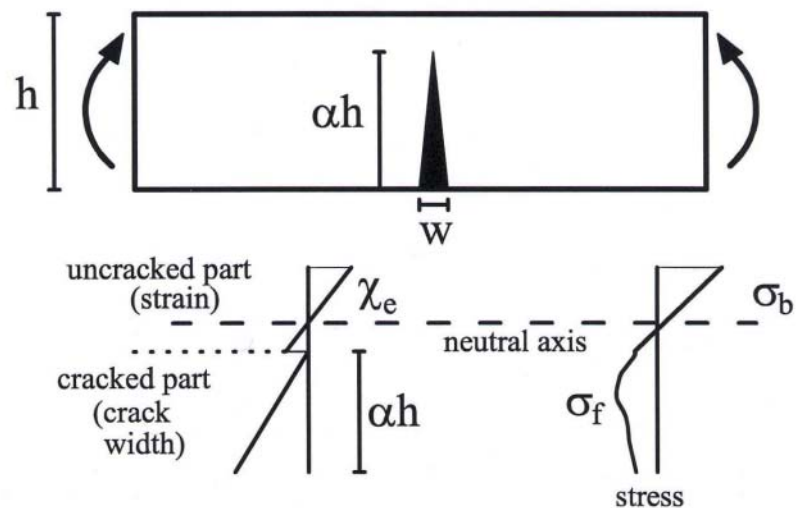The ULS verifications used a partial safety factor of 1.2 for the resistance moment.

Fig. 5 - ULS behaviour under transverse bending

4.3 – VERIFICATIONS FOR TANGENT LOADING

For the longitudinal behaviour of the prestressed beams, the verifications of SLS shear stresses were carried out in accordance with the BPEL rules.

For the transverse and longitudinal behaviour of the bridges, the verifications of ULS shear force were carried out with, in addition to the BPEL rules, the strength added by the fibres, calculated as per the AFREM recommendations. The fibre strength contribution depends on the concomitant opening of cracks w_u and therefore requires that bending/shear force interaction be taken into account (Fig. 6). The strength added by fibres V_f was sufficient to eliminate conventional shear reinforcement.
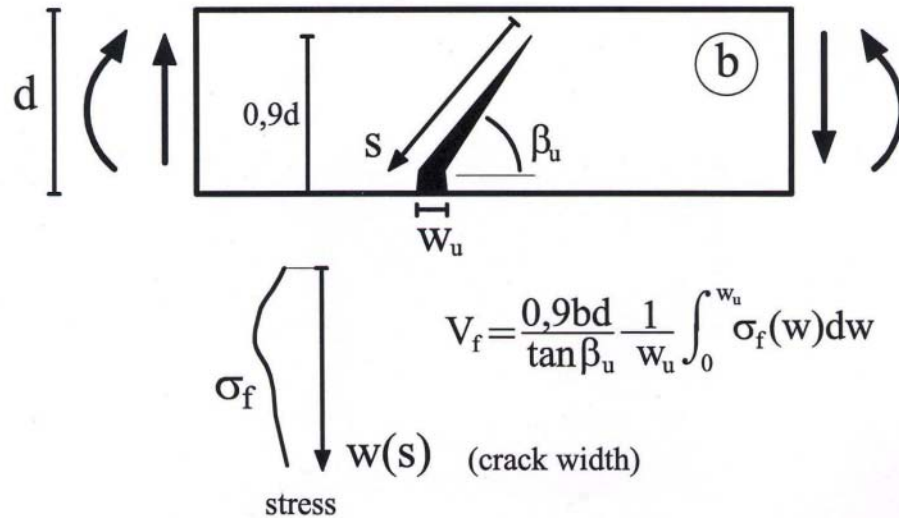
$$V_f = \frac{0,9bd}{\tan\beta_u} \frac{1}{w_u} \int_0^{w_u} \sigma_f(w)\,dw$$

Fig. 6 - Verifications under tangent loading

4.4 - NON-BRITTLENESS

The non-brittleness check consists in verifying that the fibre content is sufficient to take the tensile stresses should cracking occur, so that cracking does not reduce the resistant moment of the cross-section.

It was verified that the elastic moment M_{el} corresponding to f_{tj} being reached at the outer fibre was less than the resistance moment of the cracked section, M_{fis}, calculated when the tensile strength of the fibres is taken into account, but overlooking overtension in the prestressing tendons in order to be on the safe side.

4.5 - VERIFICATION OF PRESTRESSING TRANSFER LENGTH

In order to limit bursting and general stress distribution effects in the beam end blocks, and to avoid any cracking in the prestress transfer length, the ends of some of the strands were sheathed. Three sheath lengths 7.2 ft (2.2 m), 11.5 ft (3.5 m), and 18 ft (5.5 m) were used to allow a progressive transfer of prestressing forces.

The verifications were carried out in accordance with the provisions of the BPEL rules, by checking that the shear stresses in the transfer length are lower than the acceptable values. With UHPFRC, the strand anchorage length is considerably reduced, and to be on the safe side no account was taken of gradual introduction of prestress forces, in this case of pretensioned prestress. The general distribution stress thus obtained was 810 psi (5.57 MPa), which is less than the intrinsic strength of the BSI, even when cracked.

5 - VALIDATION BY TESTING

Before the bridges were built, test series were undertaken to validate the assumptions used for deck design and to verify the behaviour of the concrete at the scale of the actual structure. These test series comprised the following tests:

- Construction of a beam test component to check that there are no problems with prestress distribution and for new concrete characterization testing (flexural tests on sawn prisms recommended in reference[3]).

- Flexural tests on full-scale slabs in order to validate the transverse bending behaviour of the deck:
  - Flexural test on a monolithic slab,

  - Flexural test on a slab with a construction joint, to represent the longitudinal jointing between precast beams.

5.1 – TEST COMPONENT

A 16.4 ft (5 m) long test component (photo 3), a half-π-shaped beam, was made in the same way as the beams made for the bridges.

Photo 3 - Test component

Prestress distribution

Two arrangements for spreading out prestressing tendon anchorage were tested. At one end the arrangement was identical to that proposed for the beams, and at the other an additional strand in the bottom flange was sheathed. The test was conclusive, in so far as no stress-distribution cracking was observed at the ends of the element. In addition, the external appearance of the element was very satisfactory and the mechanical characteristics of the concrete were quite high : f_{ck} on cube ~ 31,000 psi (214 MPa).

Flexural tests on sawn prisms

To study the combined effect of the form and concrete placement on the distribution of fibres, test prisms were cut at different angles and at different locations in the web and upper flange of the test component. A 0.39 in (10 mm) wide notch was sawn in these 4.33x4.33x15.8 in (11x11x40 cm) prisms and they were tested in centre-point bending, notch downwards, with recording of mid-span crack width.

Fig. 7 compares the mean curve for the 20 tests carried out on prisms sawn from the flange of the beam with that given by theoretical calculation based on the constitutive law for the concrete, weighted by a coefficient reflecting the effect of sawing on the efficiency of fibres near the edges (the fibres used were 0.79 in (20 mm) long and a 0.39 in (10 mm) wide disturbance zone on each side of the section was considered).
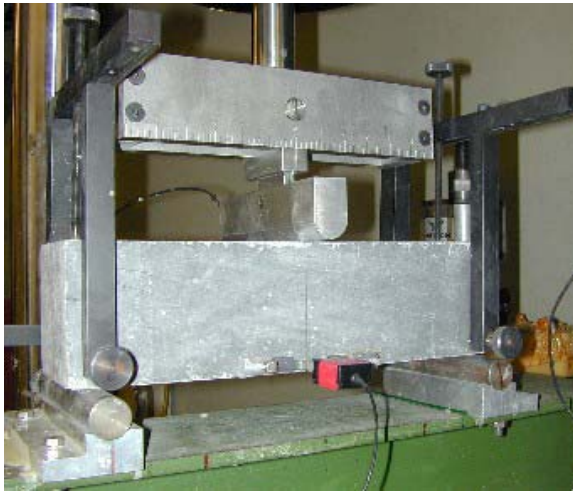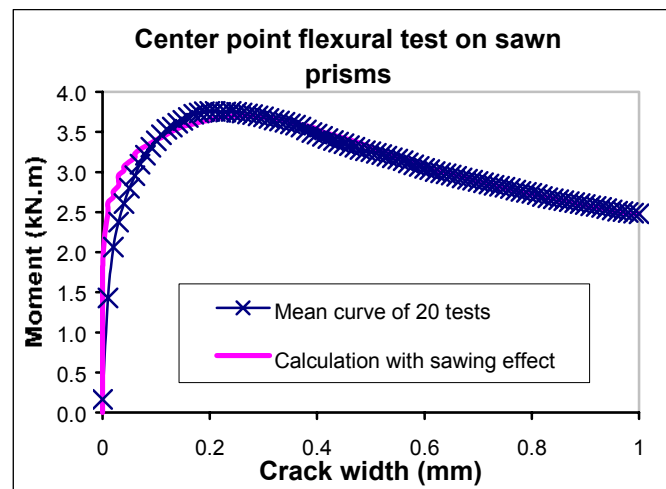

Photo 4 – Test device (ENTPE)   Figure 7 – Test results

The match is quite good for the crack-width range considered. The difference between the calculated resistance moment (M_{CAL} = 32,660 in lbs (3.69 kN.m)) and the tested resistance moment (M_R = 33,280 in lbs (3.76 kN.m)) is less than 2%.

5.2 - FLEXURAL TESTS ON SLABS

Two flexural tests were carried out on slabs by the CSTB. The dimensions of the slabs matched those of the flange of the π-beams: 5.9 in (15 cm) thick, 3.8 ft (1.15 m) long (distance between the webs of the π-beams). They were made 15.7 in (40 cm) long to limit edge effects on fibre distribution.
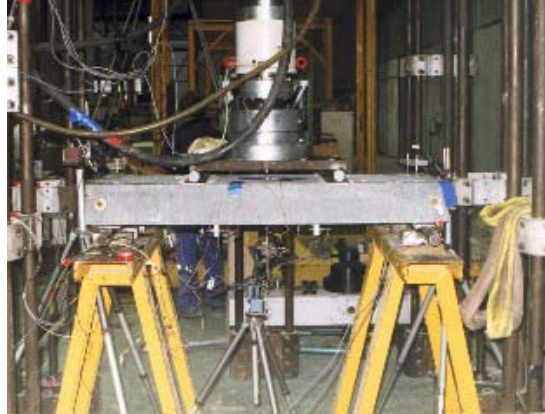
Photo 5 – Test device (CSTB)

Third-point bending test on monolithic slabs

The third-point bending test was displacement controlled (mid-span deflection) and taken to rupture of the test specimen.
The response of the slab was characterized by a preliminary phase of elastic behaviour followed by an elastoplastic phase (microcracks held closed by fibres, then initiation of a macrocrack), and finishing with a strain-softening phase, illustrating the reduction in resistance capacity as a macrocrack opens up (Fig. 8).
The following values are deduced from analysis of the curve obtained:

- Cracking moment: M_f = 135,420 in lbs (15.3 kN.m) i.e. equivalent elastic stress on bottom fibre : σ_e = 1480 psi (10.2 MPa)
- Instantaneous modulus of deformation: E = 9,240 kips (63.7 Gpa)
- Moment of rupture: M_r = 271,725 in lbs (30.7 kN.m)
- Ductility resulting from fibres: M_r / M_f = 2

Comparing these test results with the action effects taken into account in verifying the design of the bridges, it can be seen that there is a quite comfortable safety margin.
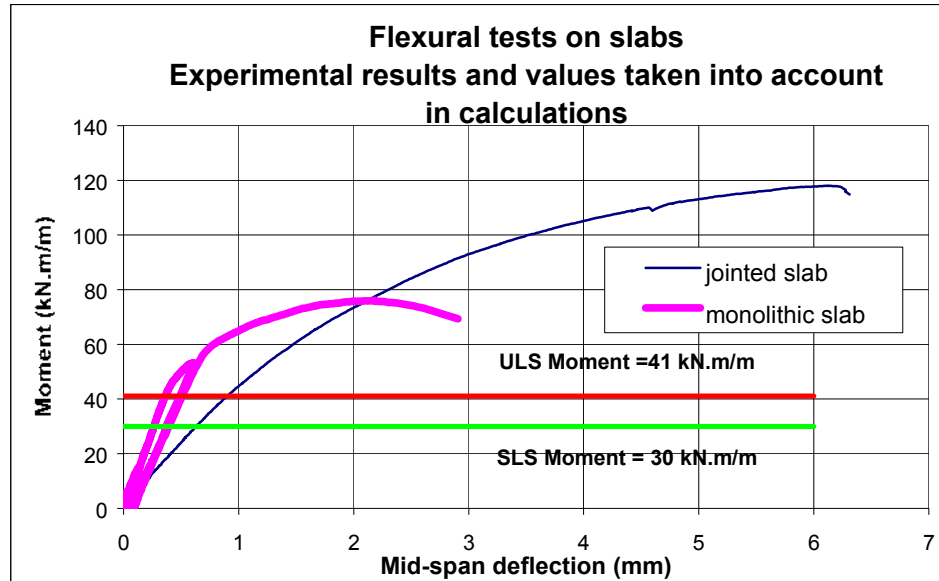
Fig. 8 – Test results

Third-point bending test on jointed slab

The test slab was made in two phases, and otherwise in exactly the same way as proposed for the bridge (jointing details are given in Fig. 3). The test device was identical to that for the monolithic slab.
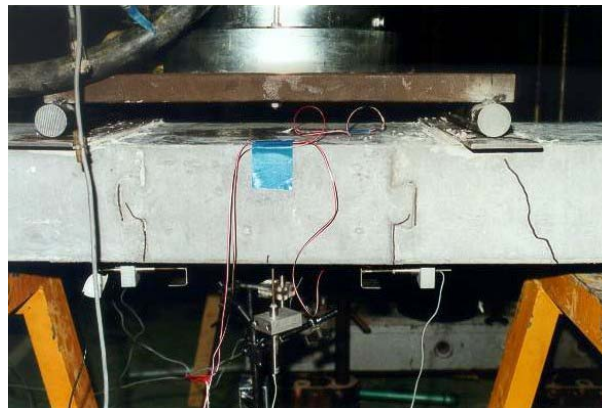

Photo 6 – Jointing

Its behaviour under load was characterized by an initial phase of separation at the construction joint, followed by a phase of cracking controlled by the passive reinforcement. The slab failed when it cracked outside the jointing area, beyond the area with constant moment, at the very end of the reinforced area. The moment of rupture of the slab was about 416,000 in lbs (47 kN.m), compared to 374,375 in lbs (31 kN.m) obtained with the monolithic slab.

The bearing capacity of the jointing zones is therefore more than adequate, and their behaviour is perfectly satisfactory.

6 -PRECASTING BEAMS

The beams were cast at the plant of Dutch precaster Hürks Beton whose workforce had already built up sound experience with BSI concrete when making the beams for the Cattenom and Civaux power plants.

6.1 – FORMS

The steel forms were of conventional design, but with some special features made necessary by the rheological characteristics of BSI and the absence of passive reinforcement:

- Stiffeners to limit deformation of the walls due to the thrust of the BSI (viscous fluid of greater density than conventional concrete),
- Special attention to leaktightness of joints between form panels, in order to contain the self-compacting concrete and ensure a perfectly smooth finish,
- Retractable form parts at crossbeams to avoid any cracking of the beam due to restrained shrinkage.

Photo 7 – Form

6.2 - BSI CONCRETE MANUFACTURE

BSI concrete is simply made by adding water, superplasticizer, and fibres to premixed dry ingredients, as follows:
  - premix delivered, bags opened and weight checked, fed into double planetary mixer,
  - mixed briefly, dry, to deflocculate fines,
  - water and superplasticizer added, more mixing,
  - fibres separated ("de-clumped"), weighed, and added by vibrating conveyor belt,
  - more mixing until the criterion laid down for mixer power absorption is attained,
  - mixer emptied into a concreting hopper,

- BSI quality checked by systematic sampling (rheology, temperature, air content).

Photo 8 – Mixing in progress

Photo 9 – Slump-flow measurement

This manufacturing cycle takes 15 minutes on average for 1.3 yd^3 (1 m^3) batches.

6.3 - BEAMS MANUFACTURE

The concrete placement method and means of checking set had been validated beforehand on a 16.4 ft (5 m) long test beam (Cf. 5.1). From this were determined a placement method and a procedure for monitoring BSI set by maturity measurement.
These precautions meant that the level of reproducibility for the different beams was good; this was subsequently demonstrated by the fact that there was no significant deviation of mid-span deflection (Δ < 0.2 in (5 mm)).
The concrete placement steps for making the beams were as follows:

- Concrete placed in the webs in layers, with continuous longitudinal displacement of the hopper and controlled drop height and discharge,
- Concrete placed in the flange in successive layers, with continuous to-and-from movement across the beam; the reason for this procedure was to ensure random distribution of fibres by limiting concrete flow,
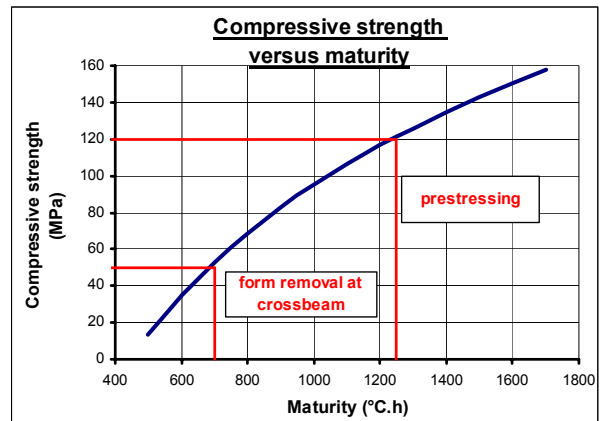- Surface trowelled and curing compound applied.

Fig. 10 - R$_C$ versus maturity

Operations carried out after concrete placement were governed by maturity measurement:
- Partial form removal at crossbeams : R$_C$ = 7,250 psi (50 MPa),
- Full form removal and prestressing of the beam : R$_C$ = 17,400 psi (120 MPa).
Since prestress was applied when the BSI was about 40 hours old, two beams were produced per week.

| Photo 10 – Flange concrete placement | Photo 11 – Temporary storage of beams |

7 - BEAM ERECTION AND JOINTING

The 20 beams for the two bridges were transported to the site by rail and road.

7.1 - BEAM ERECTION

A truck-mounted crane was used to hoist the beams into place on top of the conventional concrete piers and abutments.

Placement of the 10 beams of the first bridge (OA4) took a little over 2 days whereas, benefiting from experience, those of the second bridge (OA6) were placed in just half a day.

The beams rest on concrete bearing blocks with neoprene bearing pads, on top of which are placed tapered shims to obtain the required crossfall.

Photo 12 – Erection of a precast beam

7.2 - JOINTING OF BEAMS

BSI joint slabs were cast between the main beams to make each deck as monolithic as possible.

7.3 – SITE MANUFACTURE OF BSI

A ready-mix batching plant was chosen for making the "in situ" BSI for a number of reasons, particularly:
- capacity and performance of the mixer,
- presence of a conveyor belt for feeding the fibres into the mixer,
- reliability and accuracy of production control tools,
- proximity to work site.

The complete procedure for batching at the plant and transporting to the site by truck mixer (about 15 minutes) was validated before the works started. A campaign of suitability tests served to adjust the production parameters and check that the BSI produced met the stipulated requirements.

The stages in site production of the BSI were identical to those for precasting (Cf. 6.2). Because of the parameters of the mixer at the ready-mix plant, each batch was only 26.5 ft^3 (750 litres) and the cycle took about 20 minutes.

The BSI was inspected (rheology, temperature, air content) and tracing samples taken before each truck mixer left for the site.

After verification of the conformity criteria, the concrete was transported to the site with the drum turning at a speed determined during the suitability tests. This was done to prevent segregation of the mix that might have been caused by too great a centrifugal effect.

7.4 - CONCRETE PLACEMENT AT JOINTS

The longitudinal joint slabs between the main beams were shuttered by panels suspended from the deck, which meant no scaffolding was necessary. Special attention was paid to the leaktightness of the forms.

Photo 13 – Joint slab   Photo 14 – BSI placed in wheelbarrows

Given the small quantities involved, the concrete placement technique used was one of the simplest imaginable: the BSI was poured into wheelbarrows for transport to the areas to be jointed. The forms were filled simply by the self-compacting nature of the concrete, without vibration.

CONCLUSION AND NEW APPLICATIONS

The innovative structures at Bourg-lès-Valence, the first road bridges built of UHPC, are already a reference allowing more general use of these new materials.

At the end of 2002, approximately two years after beams manufacture and one year after the opening to traffic, a detailed inspection of the bridges was carried out. No water seepage or cracks have been noticed on the beams.

Photo 15 – View of completed bridge

The publication of the Interim Recommendations on UHPFRC[3] gives future Operating Authorities a reference to use in establishing their specifications and engineering firms a starting point for their design calculations.

However, while these conventional bridges with small spans have served to validate the performance of the material, they have not yet revealed all the freedom and daring in design UHPCs make possible.

The toll-gate of the Millau Viaduct, currently under construction, which will have an elegant roof based on a thin Ceracem® (ex-BSI) shell, is the next step in the development of this new material.

This roof will look like an enormous twisted sheet of paper, 321 ft (98 m) long and 92 ft (28 m) wide, with a maximum thickness of 2.8 ft (85 cm) at centre (Photo 16). Its alveolate structure will be like an aircraft wing and will be made of match-cast prefabricated segments, 6.5 ft (2 m) wide, connected together by an internal longitudinal prestressing. In all, 1,308 yd³ (1,000 m³) of Ceracem® will be used, weighing a total of about 2,800 tons.

Photo 16 – View of Millau Viaduct toll-gate

REFERENCES

1. BPEL 91 révisé 99 "Règles techniques de conception et de calcul des ouvrages et constructions en béton précontraint suivante la méthode des états limites. Fasc. 62 (Titre premier, section 2 du CCTG) ", April 1999
2. AFREM – BFM "Recommandations sur les méthodes de dimensionnement, les essais de caractérisation, de convenance et de contrôle. Eléments de structures fonctionnant comme des poutres", December 1995
3. SETRA – AFGC "Ultra High Performance Fibre-Reinforced Concretes. Interim Recommendations", January 2002
4. Birelli G., Cadoret G., Dutalloir F. and Thibaux T., "A new, very high performance concrete,", *International symposium on high-performance and reactive powder concretes*, Sherbrooke (Canada), August 2000.