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Cracking Control in the Concrete Slab of the Nevers Composite Bridge

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Summary

Experience has shown that just applying the French regulations was not sufficient to control transverse cracking in concrete deck slabs of composite bridges. Unacceptably wide cracks were observed. In 1995 a working group published Recommendations [1] to control this cracking and improve durability. The Nevers bridge was built between 1992 and 1995. The provisional version of the Recommendations [1] was taken into account during construction and its cracking was successfully controlled.

1. Presentation of the 1995 Recommendations

Composite structures have become frequent in France. They account for nearly 20% of the surface of bridges currently built as opposed to less than 5% in 1980.

Transverse cracking in the concrete slabs of composite bridges is accepted by the French design regulations. It requires a minimum reinforcement condition and a limit of the tensile stress in passive steel reinforcements of the slab in the zone of hogging moment.

Experience has shown that just applying the regulations was not sufficient to control transverse cracking in deck slabs. Excessively wide cracks were observed late in the 1980s in the zones of hogging moment, and even cracks in the sagging moment zones where the concrete is theoretically in compression. These unacceptably wide cracks are likely to affect the serviceability of these structures.

A working group, made up of the Administration and the contractors, analysed the causes of cracks observed in composite structures and in 1995 published Recommendations [1] to control this cracking and improve durability.

These Recommendations differentiate between two types of provisions:
- those intended to limit cracking intensity,
- those intended to limit crack width.

All the Recommendations will not be covered in detail here but it will be shown how they have been taken into account in the Nevers bridge.

2. Presentation of the Nevers Bridge

The bridge is situated on the Nevers bypass in the centre of France. It carries the National Road 7, the well-known route down to the south of France, over the River Loire east of the town.
It is composed of two composite box girders, each one 420 metres long (fig. 1 and fig 2).

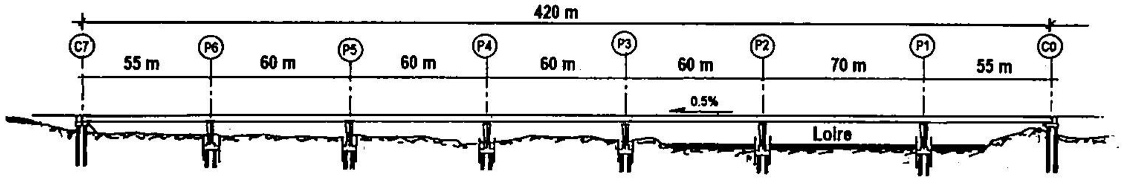

Fig. 1 Longitudinal section

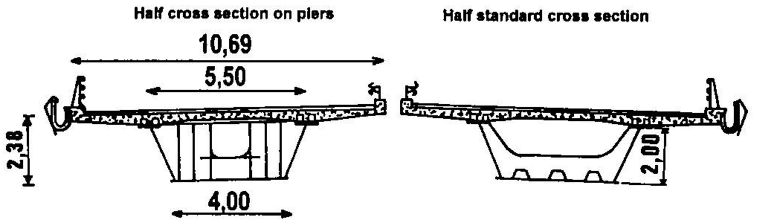

Fig. 2 Cross section

The bridge was built between 1992 and 1995.
As early as the design stage in 1991, special measures were imposed to limit cracking. These measures were supplemented in the course of work to take into account the provisional versions of Recommendations [1] as they progressively evolved, particularly the design rules.

3. Construction of the Nevers Bridge

3.1 Background data

3.1.1. Contract provisions
Table 1 below lists the main provisions of the 1991 contract and compares them with those of the final version of the Recommendations [1].

	1991 contract provisions	1995 Recommendations
pilgrim's steps concreting	yes	recommended
segment lengths	15 and 20 metres	> 8 metres
waterproofing	thick waterproofing layer	thick waterproofing layer
time before removal of formwork	not specified	24 hours minimum
resistance of concrete to removal of formwork	15 MPa	16 MPa
resistance of concrete at 28 days	35 MPa	> 30 MPa
concrete mix designing with respect to endogenous and thermal shrinkage	not specified	limit endogenous shrinkage and thermal shrinkage
curing + protection from weather	yes	recommended
lifting or lowering of supports	yes, 25 cm on one abutment n = 18 (80 % taken into account)	yes, within certain limits n = 18 (for d > 30 days)
design of construction stages	not specified	n = 6
design of long-term condition	n = 18	n = 18
green concrete shrinkage value	not specified	$\varepsilon_r < 1.5\ 10^{-4}$
long-term shrinkage value	$\varepsilon_r = 2.0\ 10^{-4}$	$\varepsilon_r = 2.0\ 10^{-4}$
minimum longitudinal reinforcement	1% in cracked zones	1% (for 20 mm deformed bars)

3.1.2. Problems caused by deadlines and work procedure
The time imposed by the Project Owner for completion of the first deck was 18 months. Furthermore the contract proposed pilgrim's steps concreting to reduce tensile stresses – and consequently cracking – in the slab near the piers. The segments were 20 metres long for the regular spans, or 15 metres long for the side spans and the main span. Then after concreting, it was planned to lift the supports by 25 centimetres on the right bank abutment in order to reduce tensile stress in the slab concrete on pier P1 (fig. 3 and photo 1).

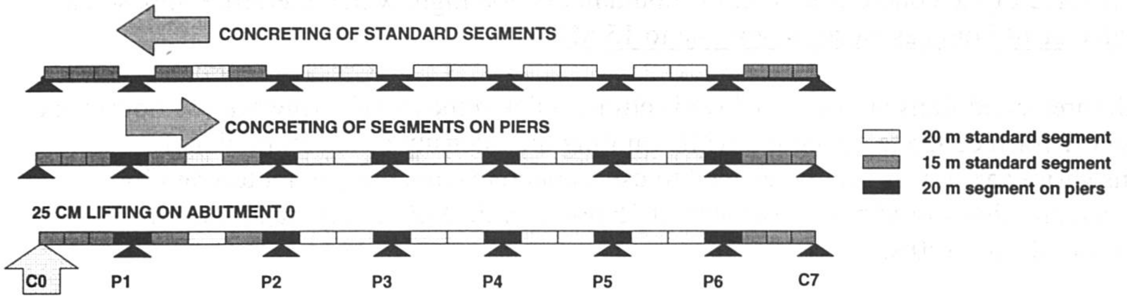

Fig. 3 Construction procedure by pilgrim's steps concreting

The contracting company used exactly the same general concreting procedure as that proposed in the contract. To comply with the tight schedule, it chose to concrete two segments per week (40 metres), using a 78 metric ton crane which travelled directly over the bridge webs for handling purposes (photo 1).
To meet the deadlines, the contractor had to remove the formwork at 8 a.m. for concreting that had been completed at about 6 p.m. the previous day. The last concrete casting had therefore been hardening for 14 hours when the formwork was removed. But the minimum off-form strength requirement to limit deformations had been fixed at 15 MPa.

In addition to the foregoing procedures – pilgrim's steps concreting, lifting the support on one abutment – other steps therefore had to be taken so as not to jeopardize the contractor's time schedule, while ensuring that cracking in the slab was of reasonable intensity and with controlled crack widths.

3.2 Steps taken to limit cracking intensity

3.2.1. Restrictions on crane travel
To prevent the green concrete being stressed by the 78-ton travelling crane or having to tailor the design of the longitudinal passive steel to this crane, severe conditions were imposed on the crane movements. At the end of the concreting, the crane had to be brought to a position vertically above a support, and naturally the concrete must not have begun to set before the crane was moved, which would be approximately four hours after concreting was started.
For the second deck, handling was performed from a crane travelling on the first deck, which enabled the technical constraints to be reduced.

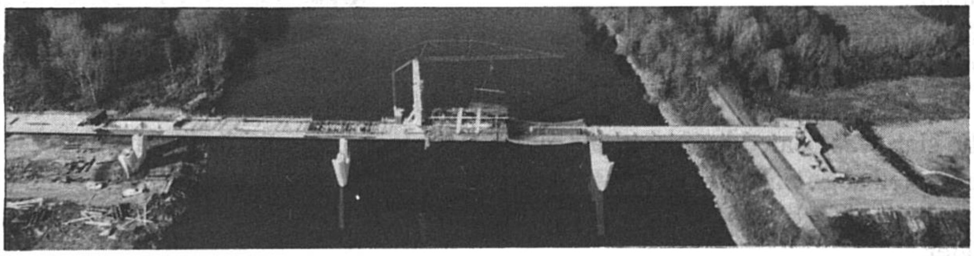

Photo 1
Crane on the bridge

3.2.2. Concrete mix design and installation

The concrete had to fulfil conditions that could not easily be made consistent with each other. It was not to begin setting for four hours and had to attain 15 MPa in fourteen hours.

One solution consisted in selecting a high-strength concrete that gained strength very quickly. But because of the thermal and endogenous shrinkage which is constrained by the bridge frame, cracking was liable to occur in the slab. For this reason, the Recommendations [1] advised that the strength value of the concrete at 28 days should not be too high, which therefore imposed a strength value at 14 hours as close as possible to 15 MPa.

To meet all three conditions at once – delayed setting, quick removal of formwork (14 hours) and a minimum off-form concrete strength – while limiting the intensity of the thermal and endogenous shrinkage, it was finally decided to use a concrete containing a not too rapidly-hardening cement, that would have a strength at 15 hours of 22 MPa under standardized temperature conditions (20°C).

The selected concrete mix design and the thermal behaviour of a slab segment were modelled by a finite element programme (L.C.P.C TEXO) to determine the heating conditions strictly necessary to obtain an off-form strength of 15 MPa. Bearing in mind that the two slabs were to be concreted in winter, these calculations enabled two important thresholds to be fixed :
- the minimum external temperature T1 requiring heating of the slab
- the minimum external temperature T2 requiring the use of a hot concrete.

The transverse distribution of stresses due to thermal and endogenous shrinkage was also studied using a finite element programme (L.C.P.C. MEXO). The analysis showed that these phenomena were liable to generate tensile stresses of around 1.5 MPa in some parts of the cross section.

As the limit conditions had been determined by calculation, the decision to remove the formwork could not depend solely on the results of the informative samples. For this reason, in order to make the lapse of time before removal of formwork as short as possible, the strengthening of the concrete was monitored by a maturity meter. Based on a previous laboratory measurement characterizing the change in the concrete strength under known conditions, this instrument is able to predict the resistance of the concrete to compression at any time by continuously measuring the actual temperatures in the concrete. This maturity meter enabled the formwork to be removed at the most appropriate time and considerably helped to reconcile the various constraints.

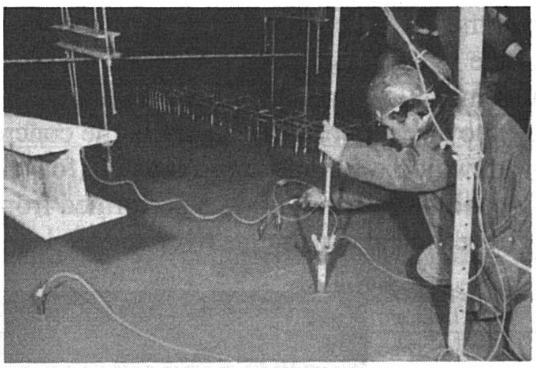

Photo 2
Positioning the temperature probes of the maturity meter

The slab was heated by forced-air oil heaters placed directly inside the girder under the newly-concreted segment. The girder structure lends itself to such heating, the ends of the heated zone simply have to be closed by a tarpaulin. The hot concrete was obtained by heating the mixing water.

3.3 Steps taken to limit crack widths

3.3.1. Calculations of forces in the slab

Table 2 below lists the main design assumptions specified in the contract or adopted in the course of construction and the assumptions in the Recommendations [1].

Assumptions	Nevers Bridge	1995 Recommendations
lifting of supports	n = 18 80 % of the effect for forces,	n = 18 (for d > 30 days)
design of construction phases	n = 6	n = 6
design of long-term condition	n = 18	n = 18
thermal and endogenous shrinkage value taken into account during construction	not taken into account	$\varepsilon_r < 1.5\ 10^{-4}$
long-term shrinkage value	$\varepsilon_r = 2.0\ 10^{-4}$	$\varepsilon_r = 2.0\ 10^{-4}$
crack width	. calculations according to Eurocodes 2 and 4 for cracks of 3/10 max.	calculations according to Eurocodes 2 and 4 for cracks of 3/10 max.
diameter of longitudinal reinforcements	e / 12	e / 12 max.
minimum longitudinal reinforcement	. 1% adopted throughout	1% (for high bond 20)

During construction : Full calculation was made of the concreting phases. This particularly highlighted the fact that one zone is far more stressed than the rest of the structure (-5.5 MPa compared with -3.5 MPa in the other spans). This zone is the second central segment of span 3 at the time of concreting the third segment of span 2 (longest span: 70m, fig. 4).

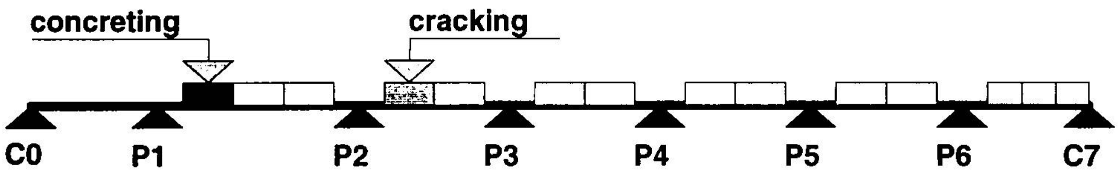

Fig 4 Cracking in span 3 during concreting of span 2

These tensile stresses exceed the tensile strength of a green concrete, which can be estimated at -2 MPa. A phenomenon often observed on composite structure sites is thus found by calculation. **When the concrete of a span is poured, cracking occurs at the end of the hardened concrete in the previous span.**

In service : The normal theoretical stresses in the concrete slab in service were calculated with a steel/concrete coefficient of equivalence of 18 for permanent loads and 6 for live loads. This calculation took into account a shortening effect (shrinkage + temperature) of 2.5 10^{-4}.

Heavy tensile stresses occur on the support and close to the segment end zones previously mentioned (up to -6 MPa).

3.3.2. Longitudinal reinforcements

The foregoing calculations show that in the case of the Nevers Bridge, cracking in the slab was inevitable, both during construction and in service, whatever the strength of the concrete.

However, in order to achieve reasonable crack widths, S.E.T.R.A. applied the rules of the provisional versions of the Recommendations [1] in compliance with Eurocodes 2 and 4.
Only two types of longitudinal reinforcements were used :
- segments on piers and the 2nd segment of span 3 are 1.35% reinforced (zones where the tensile

stress in the slab exceeded either 4 MPa during construction or 5.5 MPa in service),
- the other zones are 1% reinforced to take into account the effects of green concrete shrinkage.

3.4 Results

Cracking in the slab was recorded for both decks after the loading tests. The zones effectively cracked were shown to correspond to those foreseen by the calculations (these zones are the on-pier segments and the last central segment of each span).

The spaces between cracks were approximately 30 centimetres and the crack widths were as follows (photo 3) :

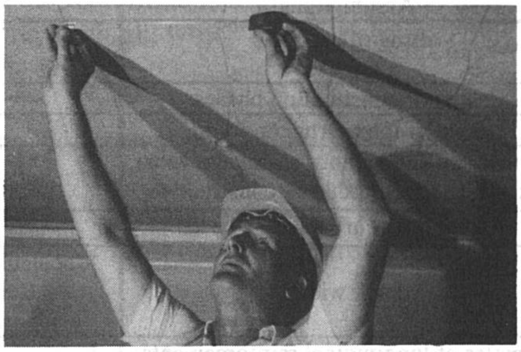

Photo 3
Transverse underlined cracking in slab

	% of cracks with widths < 2/10 mm	% of cracks with widths = 2/10 mm
downstream deck (first constructed)	95%	5%
upstream deck	70%	30%

This jobsite was thus successful as regards cracking control but it should be possible to do better by reducing the extent of cracked areas occurring during the construction stage.

The reinforcements used led to an increase in the steel ratio of around 15 kg/m^3, which corresponds to a 0.4% price increase in the contract.

4. Conclusion

The 1995 Recommendations [1] differentiate between two types of provisions – those aimed at limiting cracking intensity and those aimed at limiting crack widths.
It was possible to take into account most of the provisions aimed at limiting cracking intensity and virtually all the provisions aimed at limiting crack widths without jeopardizing the project's cost effectiveness.

The extra cost, which was less than 1%, seems most reasonable bearing in mind the improvement in durability to be expected as a result of the cracking control.
From this example, it will be seen that applying the Recommendations [1] does not penalize composite structures to any significant extent.

If the contract were to be drawn up today, the designers would impose removal of formwork after 24 hours minimum, in accordance with the Recommendations [1], which would enable a concrete with an even lower heat of hydration to be used. If it proved necessary to use a second travelling formwork, its effect on project time schedules or cost would have to be analysed.

References

[1] Composite bridges : Recommendations to control cracking in slabs - September 1995.
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