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Objekttyp: **Article**

Zeitschrift: **IABSE reports = Rapports AIPC = IVBH Berichte**

Band (Jahr): **62 (1991)**

PDF erstellt am: **23.07.2024**

Persistenter Link: https://doi.org/10.5169/seals-47726

Comparison between Cast-in-Situ and Precast Segmental Construction

Comparaison entre voussoirs préfabriqués et voussoirs coulés en place

Konstruktionsbeton: Ein Vergleich von Ortbeton- und Segmentbauweise

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SUMMARY

This paper aims at giving the author's reaction to the notion of «structural concrete». Then a practical example is given with the comparison between the analysis of a cast-in-situ segmental bridge, and the analysis of a bridge made of precast segments. The last one is of course more detailed, with some specific analyses for the transfer of shear forces through segment joints.

RÉSUMÉ

L'auteur de cet article fait part tout d'abord de ses réactions à la notion de «Béton structurel». Puis il donne un exemple pratique avec la comparaison de l'analyse d'un pont construit au moyen de voussoirs coulés en place, et celle d'un pont construit au moyen de voussoirs préfabriqués. Cette dernière étude est bien évidemment plus détaillée, avec en particulier des modèles permettant l'étude du transfert de l'effort tranchant à travers les joints de voussoir.

ZUSAMMENFASSUNG

Dieser Beitrag liefert eine Interpretation des Begriffs Konstruktionsbeton. An einem praktischen Beispiel wird der Vergleich der Berechnung und Bemessung von abschnittsweise in Ortbeton mit in Segmentbauweise erbauten Brücken vorgeführt. Bei letzterer geht natürlich die Berechnung mehr ins Detail wegen der besonderen Nachweise für die Übertragung der Querkraft über die Segmentfugen.

This Colloquium fixed a very ambitious goal: structural concrete can mean many different things according to everybody's understanding, from his own formation and experience. We just shall try our best to explain what we personnally understand in this attempt to give a more unified approach of concrete structure analysis, modestly because the theme is vast and not yet clarified. Most of all, we hope not to repeat exactly what has already been said by other authors, as could happen in such very general discussions where all aspects are interconnected.

1. DURABILITY AND STRUCTURAL SAFETY. THE C.E.B. PHILOSOPHY

At the basis of all this approach is the design philosophy now widely developped by the work of C.E.B.:
- the distinction between Service Limit States, SLS, and Ultimate Limit States, ULS,
- the notion of partial safety factors.

1.1. Service Limit States

The principle is to check that "Service Limit States" are not overpassed in the structure normal situation, that is to say when the different loads are not far from their probable values. The aim of these verifications is mainly to guarantee the structure durability:
- to limit reinforcement corrosion by limiting the crack opening;
- to limit fatigue stresses...

Of course, many other requirements are to be fulfilled to obtain the desired durability, concerning mainly the choice of materials, the construction methods, the waterproofing equipment... But designers also fix, from their own experience, some design criteria which come in addition to specifications from the Codes, to reach an excellent durability and corrosion protection. As for an example, we generally try to balance as much as possible by prestressing forces the effects of permanent loads, bending moments and shear forces, following that way ideas developped some years ago by Renault Favre.
To our opinion, Service Limit States are of major importance for the building industry:
- 99% of the built structures are in the conditions of the SLS analyses, with loads not far from their probable values; and the more complex the structure is, the closer we are due to the quality of the designers and contractors selected for the design and construction;
- the experience of the building industry evidences that the greatest part of the "accidents", and the greatest part of the money involved, come from inadequate operation conditions; if we take the example of the French bridges built by the cantilever method between 1970 and 1979, roughly, we had widely open cracks, risks of fatigue in the prestressing tendons, but not a single collapse. Real collapses, involving Ultimate Limit State conditions, are fortunately very scarce.

1.2. Ultimate Limit States

The principle is to check that no collapse can occur when the different loads reach extremely improbable values (with a fractile which is normally of 5 per mille), and when the strengths of the constitutive materials – concrete and steel – are very low, reaching again extremely improbable values (with a fractile of the same order).
That means that the ULS analyses are done following the principle that:
- the structure has been built in the worse conditions regarding the material quality, the precise localisation of reinforcement and tendons, etc...
- and that loads have been widely underestimated for different reasons.

Such situations are of course – and fortunately – very scarce, with a very low probability. The definition of the partial safety factors, which lead to the definition of these situations, is evidently very difficult and is partially arbitrary.
Due to this very low probability, and to the arbitrary character of the partial safety factors, we don't consider that structures must be designed and that their dimensions must come from the ULS. To our opinion, the ULS analyses are only verifications of the structure safety, in very extreme conditions. The design must be more guided by SLS considerations, and, most of all, by the designer experience and art.
On the other hand, it is perfectly clear that the analysis of a possible collapse must be done with adapted models, which can by not means be elastic. This is the evident field of the plasticity theory, of struts and ties models, and of other models of the same inspiration.

1.3. First conclusion

Even if everybody now works on the basis of the C.E.B. philosophy, we considered necessary this preliminary statements.

We conclude that too much emphasis had perhaps been given to Ultimate Limit States:
- by reaction against pre-existent codes, only based on elastic models,
- and also due to the pro-eminence of the men who developped the plasticity theories to allow for analysing all types of collapse situations.

2. UNIFIED APPROACH OF STRUCTURAL CONCRETE

2.1. Some general remarks

It is perfectly clear that some existing codes consider concrete in a quite curious way, and that something had to be done to give an end to such a situation. To take the French Code for an example, we have:
- a code for reinforced concrete,
- a code for prestressed concrete,
- and a code for composite bridges.

That means that the concrete characteristics – strength, modulus of elasticity, shrinkage and so on – are given three times. And of course not exactly in the same way. The partial safety factors for loads are not exactly the same in the three codes...

This is ridiculous. And we are obliged to follow Jorg Schlaich and John Breen – among others – when they say that concrete behaviour is not a question of code, but varies gradually with the ratio of reinforcement and of prestressing, generally with no clear limit between reinforced and prestressed concrete.

But we must be clear: structural behaviour is only one aspect of the design problem. We must not forget durability aspects, which can lead to a minimum value of the prestressing forces, depending of the structure type, cost and maintenance conditions. The – generally – continuous behaviour of reinforced-prestressed concrete is not a key to open a new religious war on partial prestressing or on the prestressing level in structures.

Looking for durability, only forces and stresses produced by permanent loads – or permanent and frequent loads – are really important, of course evaluated in the SLS conditions with elastic models. Forces and stresses produced by extreme live loads – if they are highly improbable, like for bridge live loads – are generally not interesting. It is not important, to our opinion, if these extreme loads are balanced by reinforcement steel or by prestressing tendons (when a continuous reinforcement is possible, what excludes the case of precast segments); on condition that forces and stresses under permanent loads are convenient.

Considering forces and stresses produced by extreme live loads in SLS conditions, as it is specified in the French Code, is only an extremely convenient way to design structures with unfactored loads. This is of course easier and more accessible to practical engineers.

2.2. Our example

Our personnal duty, in this Colloquium, is to present a practical example of "structural concrete" application.

We thought that the best possible example, to evidence the relations between SLS and ULS and to show the importance of modelization, could be an analysis of the construction by the cantilever method of bridges:
- made of precast segments on one side,
- and of cast in situ segments on the other.

2.3. Some models

Before beginning the detailed analysis of our example, and the comparison between cast in situ and precast segmental constructions, we can evoke some of the models which can be used for the global analysis of our bridges.

The so-called "elastic model" gives the distribution of forces in the beam supposing that it is linearly elastic, whatever the use of the resulting forces in each section will be. This is the logical model for SLS conditions. But it is also used – according to the specifications of many codes – to evaluate the "ultimate" forces considered for the section ULS analyses, by just multiplying "elastic" forces.

Quite on the opposite side, plastic models can be built to analyse bending forces, by just placing plastic hinges according to the principles of the limit analysis.

Such an analysis must be extended to shear forces, though the notion of shear forces in ultimate conditions is even questionned by some authors (except as a calculation tool). Only more refined models can be then used, more or less inspired from the Ritter-Mörsch model, as for an example Jorg Schlaich's struts and ties models also developped by Peter Marti. Of course they don't exactly apply to precast segments and must be adapted to them as we shall see.

But these plastic analyses don't give precise evaluations of deformations; they are only built to give an estimation of the structural capacity. In some cases they are not adapted to the problem complexity:
- when second order effects become important; a precise evaluation of deformations is then evidently necessary that

the plastic models cannot give;

- and when the distribution of forces in the structure depends rather much on deformations; this is the case, for an example, with external prestressing: in ultimate conditions, the structure deformations produce tension variations in external tendons which cannot be analysed cross-section by cross-section.

Non-linearly elastic models must then be built considering representative strain-stress relations for concrete and steel, and also tension stiffening when it is necessary or easy. These models are now very well known for structures with continuous reinforcement – based on the principle of a continuous repartition of cracks –, but they had to be adapted to precast segments. This was recently done by Paulo C. de Rezende Martins and Bernard Fouré.

Of course, adapted models must be built also for the analysis of local problems, as we shall see for shear keys. This is the clear domain of struts and ties models which have no equivalent there.

3. SEGMENTAL CAST-IN SITU BRIDGES

We shall only consider bridges built by the cantilever method (or in fact by other segmental methods) where enough continuous reinforcement has been longitudinally placed to give to the structure the necessary ductility. The – often pathological – case of bridges without longitudinal reinforcement needs special analyses, not far from those corresponding to precast segments, due to the concentration of cracks in some very open ones.

3.1. Ultimate Limit State analyses

There is not much to say about Ultimate Limit States:

- the models corresponding to the application of the plasticity theory are quite well known, with three hinges in a continuous span and with the ductility requirements;
- the Ritter-Mörsch model truss is known for almost a century, which can also deal with shear forces;
- the struts and ties models presented by Jorg Schlaich, Peter Marti and others are not far from the Ritter-Mörsch truss, and can very well represent the ultimate conditions of these structures.

And, as explained by their authors, the notion of shear force itself looses much interest, and there is no question in how are balanced forces, by reinforcement or prestressing steel.

As we already explained, in some cases more complex analyses are necessary – or just interesting – when a precise evaluation of deformations is needed. For an example for flexible bridges with external tendons; a non-linearly elastic analysis is necessary to evaluate the structure deformations, and from there the tension variations in external tendons, considering representative strain-stress relations for the different materials.

3.2. Service Limit States

The situation is quite different with Service Limit States. The real question is: what is the goal of the design ?

From what we already said, we conclude:

- that the distribution of stresses in the completed bridge is determinant for its durability;
- but, on the other hand, the distribution of stresses under the effect of "extreme" service loads has no fundamental interest.

Though many engineers object that concrete has anyway to suffer many cracks – due to the transversal behaviour of the box-girder, or to thermal effects in the hours which follow the concrete pouring –, we consider that a concrete structure will have a greater durability if concrete is subjected to longitudinal compressive stresses under permanent loads. We are absolutely convinced that is it unacceptable to have – still under permanent loads – tensile stresses in the top fiber, were water can run if the waterproofing is not perfect (which waterproofing could be considered as perfect ?).

With the increasing use of de-icing salt in Winter, concrete top slabs are subjected to severe aggression, and compressive longitudinal stresses – in addition to a very compact concrete – are favourable. We are more willing to place some transversal tendons – just corrosion-protected monostrands – to produce compressive stresses transversally, than to accept longitudinal tensile stresses; in some recent bridges we introduced such a transversal prestressing, dimensionned to avoid tensile stresses under the effect of permanent loads only, or under the effect of permanent loads and of a part of frequent loads.

In addition, creep deformations will develop, depending very much on the values of the bending moments in the structure. As the creep deformations cannot be very precisely evaluated, the greater the bending moments will be during construction – and in the bridge in operation –, the greater the uncertainties on the structure geometry and on time depending deformations will be. This is the reason why, following Renault Favre, we tend to design bridges in such a way that the bending moments produced by permanent loads are almost balanced by prestressing forces. In fact, the bending moments produced by prestressing forces generally balance 60 to 70% of the bending moments produced by permanent loads.

We can take as an example bridges built span by span with external tendons anchored from pier to pier. We note p the

– almost constant – lineic density of permanent loads. The total prestressing force, P, creates with an undulation Δe (figure 1) an isostatic bending moment (P . Δe), practically equivalent to an uplift lineic density of load:

$$q = \frac{8\,P\,\Delta e}{\ell^2}$$

The ratio ($\frac{q}{p}$) gives an excellent indication of the part of permanent loads balanced by prestressing tendons as regards bending forces.
Of course, such prestressing forces also balance shear forces produced by permanent loads.

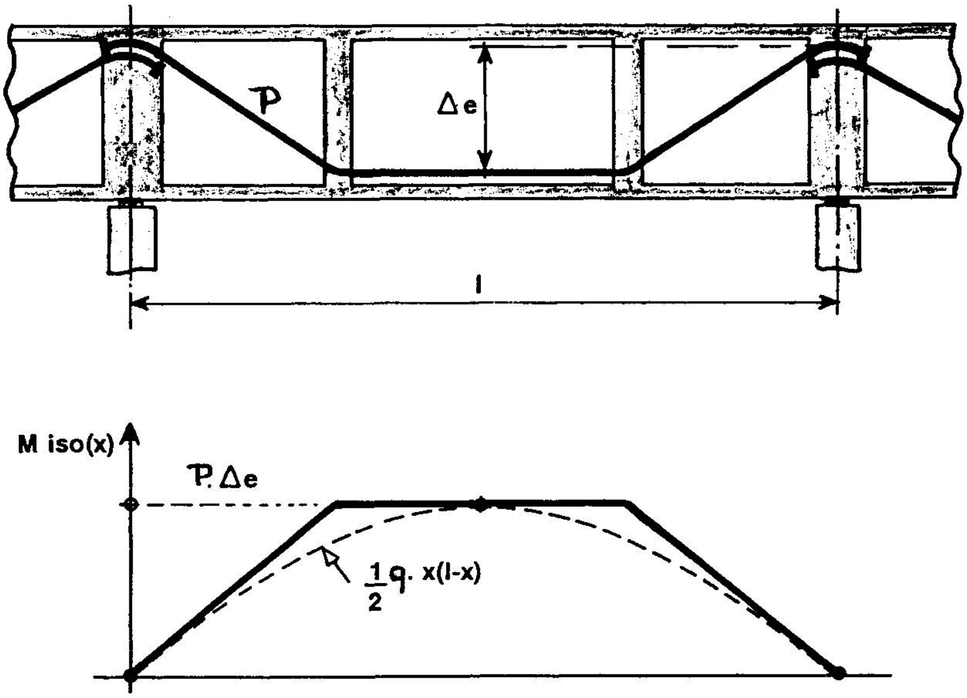


Fig. 1: Definition of an external tendon undulation, Δe, and the produced bending forces

This approach can be considered as non economic, because such criteria can lead to some additional tendons, or to more sophisticated tendon organizations. But their additionnal cost is extremely limited, and our experience – as civil servant since more than 20 years now – is that such costs are negligible as compared to the cost of bad durability and of design errors.
Finally, we must not forget the construction problems and the necessity of a precise control of the geometry, already evoked.
The analysis of deformations, following step by step the construction sequence, can be reasonably done only from the probable values of loads – that is in SLS conditions –, considering also concrete creep and shrinkage effects as well as steel relaxation in tendons.

3.3. Conclusions

We can then draw some conclusions from our analysis:

- Excellent models can be built for the ULS analyses, very well in the line of the notion of "structural concrete", with no basic difference between reinforcement and prestressing steel.
- The importance of the prestressing forces must be determined from durability specifications and criteria. As for us, we try to balance by prestressing forces a great part of the bending moments and shear forces produced by permanent loads, to also limit deformations.
- The analysis of the effects of "extreme loads" in SLS conditions, for the bridge in operation, can have only practical interest, but no philosophical important background.
- The elastic model is necessary, considering concrete creep and shrinkage as well as steel relaxation, for the step by step analysis of the construction sequence, specially for the control of geometry.

- The most critical situations of the construction sequence must be analysed in the ULS conditions also, for guarantying structural safety during construction, even if a normally good design might avoid any collapse.

4. BRIDGES BUILT WITH PRECAST SEGMENTS

We are going to repeat this analysis in the case of bridges built with precast segments.

4.1. Service Limit States

There are no great differences between cast in situ segmental bridges and bridges built with precast segments, as far as only Service Limit States are concerned. Three points only can be pointed out.

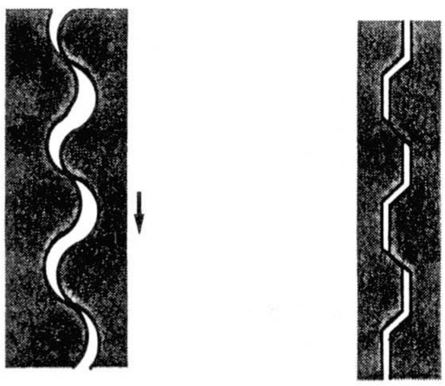
4.1.1. Reduction of creep effects
At first, we just can note that creep effects are generally more limited, because precast segments are generally much older when they are placed in the structure – and subjected to prestressing forces – than are cast in situ segments when the cantilever tendons are tensionned. On condition that prefabrication is well driven, the quality of geometry is generally better, and the redistribution of forces produced by concrete creep more limited. These effects are even more sensible with high strength concrete.

4.1.2. Stress limitations
The second point is more important: in the case of precast segments, the analysis – in the conditions of Service Limit States – of the effects of extreme loads – live loads and thermal forces – is of great importance. If tensile stresses tend to appear in some joints between segments, these joints will open. The connection between segments will be broken in the open joints, not in the glue itself, but at the concrete surface on one side[1].
We consider that this degradation must be avoided for a rather high level of loads, to maintain the bridge operation conditions and durability. Two consequences:

- a real need for a stress analysis under these extreme SLS loads, not necessary or only necessary for easing design in the case of cast in situ bridges;
- and a higher level of prestressing, since only prestressing forces can avoid tensile stresses in joints; in classical bridges at least.

4.1.3. Transfer of shear forces
The last point concerns the shear force equilibrium in joints.
Everybody knows that the most important problems appear when a new segment is placed: the glue has not yet hardened and acts as a lubricant, favourably for easing the segment ajustment to its exact position, but unfavourably for its static equilibrium when placed; and the prestressing forces which fix the segment against the already built part of the bridge are generally limited.
Shear keys in the segment webs are designed to transfer shear forces in the joint in this situation. They also make possible the precise adjustment of the new segment – vertically and transversally –, helped with other shear keys in the slabs which also transfer local shear forces in these slabs.
Though working in SLS conditions, with unfactored loads and in joints which cannot be open, the best approach for designing shear keys is a collapse analysis, based on a model inspired from the kinematic method of the theory of plasticity: we just consider a small vertical movement of the new segment.
We shall only analyse the case of multiple shear keys, which are now widely preferred to concentrated shear keys.
The model immediately shows that the shape of shear keys is determinant (figure 2):

- undulated shear keys would produce a dangerous concentration of forces on some bearing lines, which could provoke concrete splitting;
- on the contrary, polygonal shear keys appear extremely efficient, offering several small areas as supports.



Undulated shear keys — Adapted shape for shear keys

Fig. 2: Evidencing the shear-keys ultimate capacity as a function of their shape

1 We don't evoke here dry joints that we don't consider as a good solution in European climate (due to water circulation and freeze), and which have some important drawbacks anyhow.

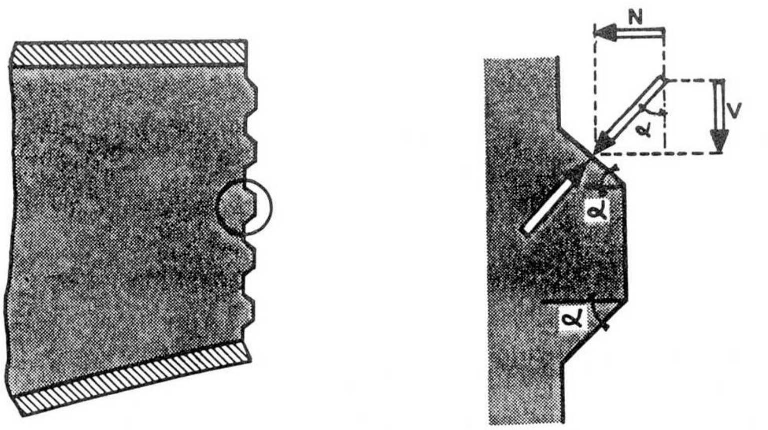


Fig. 3: Mechanical behaviour of multiple shear-keys

Considering then that there is no friction, and that all vertical shear forces must be balanced in the webs – not in the slabs which are too flexible transversally –, we can write that the total reaction on the inclined part of the keys is given by (figure 3):

$$R = \frac{V}{\cos \alpha}$$

and that the normal force transferred through this part of the keys is given by:

$$N_{sk} = V \operatorname{tg} \alpha$$

The rest of the normal force:

$$N_j = N - V \operatorname{tg} \alpha$$

produces compressive stresses on the vertical parts of the joint.

An analysis of the bending moment in the joint could give the distribution of the reactions on the different keys.

But our model can also guide the selection of the angle, α . As the shear keys cannot be reinforced, the inclined reaction on each shear key must be directly transfered as a compressive force in the massive section, where stirrups can be placed: the angle α must not be too low (figure 4). Generally, it is equal to 45 degrees.

We can evidently conclude:

- that we have been able to determine the shear-keys shape;
- that we can have an idea of the necessary supporting area of the shear keys from the supported reaction:

$$R = \frac{V}{\cos \alpha}$$

- and that the prestressing force must be far over the minimum value:

$$P_{min} = V \operatorname{tg} \alpha$$

and placed in such a way that the total joint can be closed by compressive stresses.

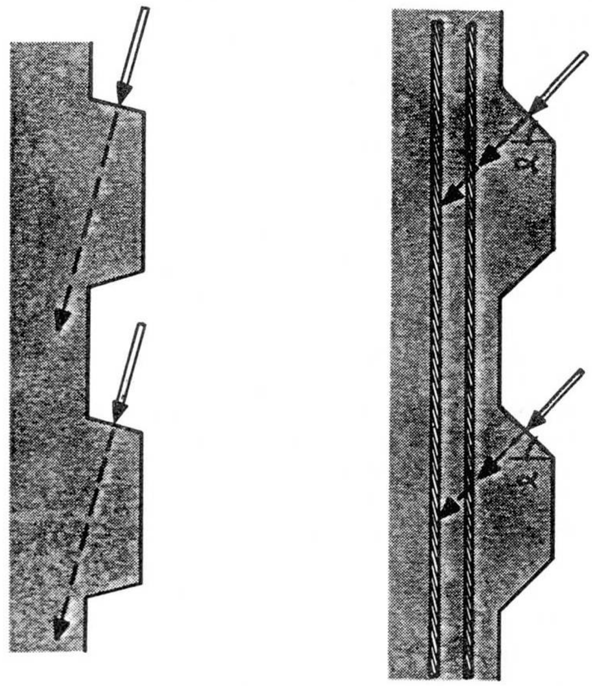


Fig. 4: Influence of the shear-keys shape on the transmission of forces

Of course, the situation must be analysed also at other stages of the construction, and for the bridge in operation. But due to glue hardening, such a model would be then extremely conservative.

Before passing to the ULS analysis, we just want to point out that the structural safety of bridges made of precast segments – the classical example of the so-called full prestressing – totally relies on the stability of shear-keys, that is on purely plain concrete elements. This statement will certainly please the tenants of the "structural concrete", as a good example of the necessary continuity of the concrete analysis.

4.2. Ultimate Limit States

The ultimate analysis of bridges made of precast segments needs some amendments of the classical models, and some preliminary statements.

4.2.1. The collapse model

The collapse mechanism evidently corresponds to the constitution of three "hinges" in a continuous span. But the deformation is mostly concentrated in the opening of some joints, not in continuously distributed cracks as in cast in situ bridges. Due to the relative importance of the joint openings, we often can consider that all deformations are concentrated there and that the different segments behave rigidly, except locally in open joints.

The consideration of the distribution of bending moments in the span gives some additional informations (figure 5):

- near supports, the bending moment variation is generally very rapid, whatever the organization of the prestressing tendons can be; in this situation, only the first joint can be widely open; the next joint, at a greater distance from the support, has to support a much smaller bending moment; except if many tendons are anchored in the first segment, before this second joint, it cannot be so widely open; it is generally totally subjected to compressive forces, or very slightly open at the top slab level;

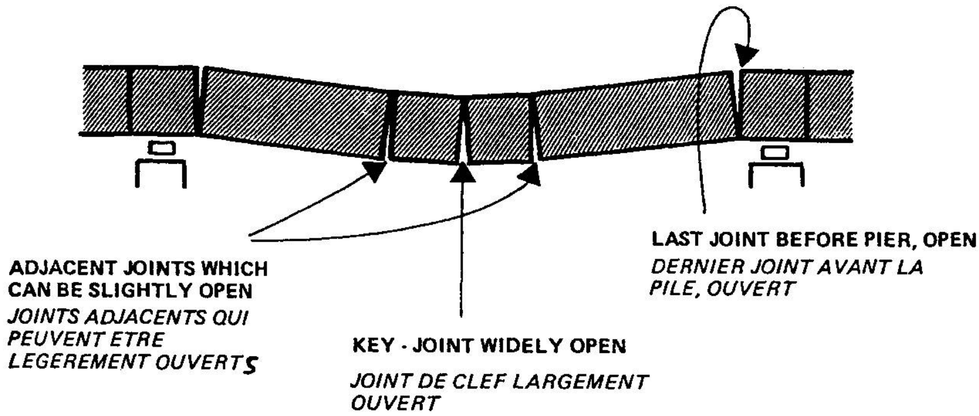


Fig. 5: Ultimate behaviour of a segmental beam

- in the mid-span zone, the bending moment variation is always very limited; several joints can be open; if there is one joint at mid-span, it can be open with the next one on each side; if there is no joint at mid-span, the two symmetrical joints – one on each side – will be open, and most probably the next one on each side;
- the figure fits with the distribution of forces: several joints open at mid-span, with practically the same forces in each of them, with an almost constant bending moment and a very limited shear force; one single joint really open near the support, with the "hinging" moment and an important shear force; this important shear force explains the rapid bending moment variation, and the "closure" of the next joint;
- this model is a very good representation of the bridge behaviour when it is prestressed with external tendons, the tension variation of which – constant in the span – depends of the bridge global deformation; deformations can be a bit more distributed near supports in the case of internal bonded tendons, due to the local tension variations in tendons passing through open joints: different tension variations can balance different bending moments in two successive open joints, if the moment variation is not too rapid;
- at last, we can state that with classical distributions of prestressing tendons, the joints at mid-span open before the joint near each support; we mean that the joints at mid-span open for lower loads than the joints near supports, that is before for an increasing load; this affirmation cannot be evidenced from our simple model, since no deformation can take place before the opening of the three hinges as far as we consider the segments as rigid; but the real deformation of the span can allow for joint openings in the mid-span zone before opening the joints near supports; this result comes from the hyperstatic effect of prestressing forces: in SLS conditions, compressive stresses are much more important in the top slab near supports than in the bottom slab at mid-span when cables are continuous in the span, specially for external tendons which produce higher hyperstatic moments.

There is no other mystery in the ultimate analysis of bending moments in bridges built with precast segments.

4.2.2. Evaluation of the tension variations in external tendons

Only the case of external tendons needs some additional words, when we want to consider the tension variation in external tendons to evaluate the real ultimate bearing capacity. As we already explained, we then need a model which can give a precise evaluation of the span deformation; a model equivalent to the non-linearly elastic model that we use in the case of cast in situ bridges, considering the crack distribution as continuous.

It is then necessary to build a model for the local deformation in open joints, giving a relation between the joint opening and the bending moment. This was done by Paulo C. de Rezende Martins and Bernard Fouré in a remarkable work which constitutes a major step for the understanding of segmental bridge behaviour.

4.2.3. Ductility and rotational capacity

But, what about ductility and rotational capacity?

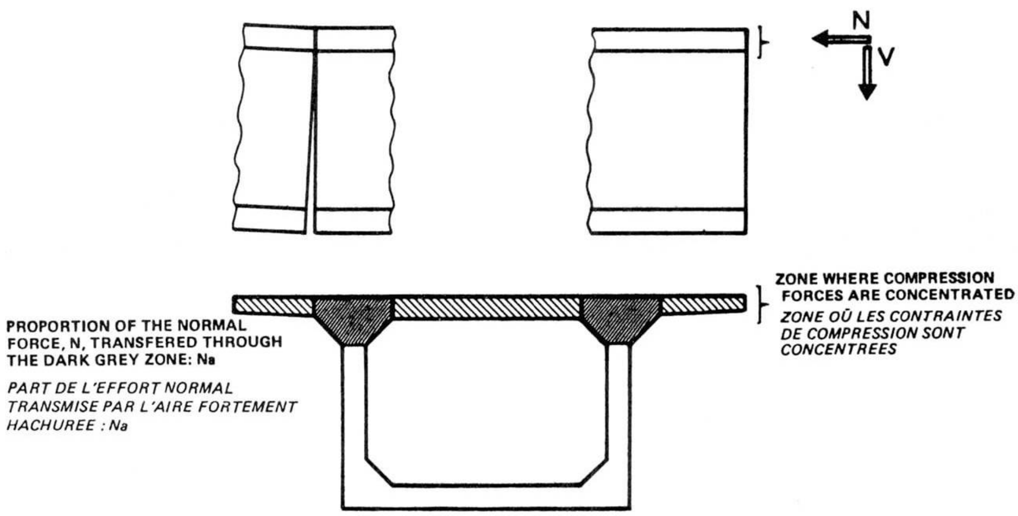


Fig. 6: Proportion of the compression stresses balancing shear forces in the Ultimate Limit State conditions, with a positive bending moment

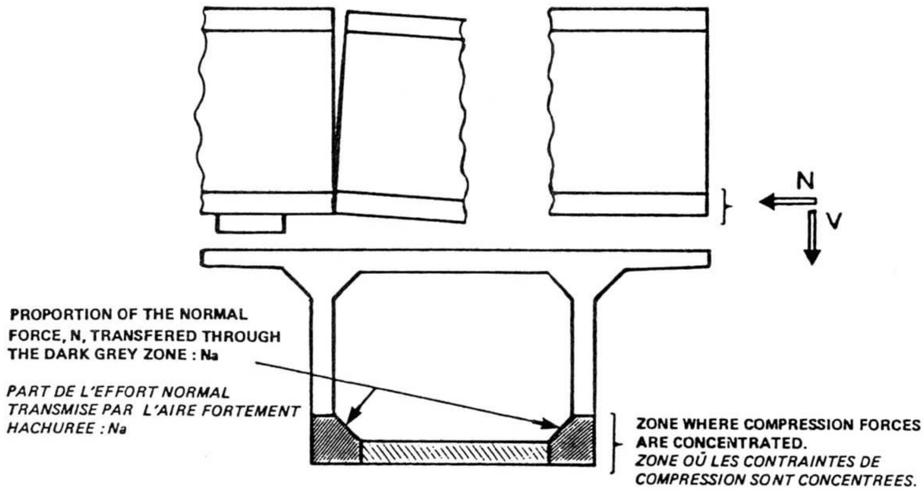


Fig. 7: Proportion of the compression stresses balancing shear-forces in the Ultimate Limit State conditions, with a negative bending moment

The question can be easily answered. The joint opening must be permitted by the deformation of the part of the cross-section which remains subjected to – high – compressive stresses: the lower nodes and slab near supports, and the upper nodes and slab at mid-span (figures 6 and 7). The segments on both sides of open joints must be extremely ductile in these areas, specially in the nodes, to avoid concrete splitting (figure 8). These areas

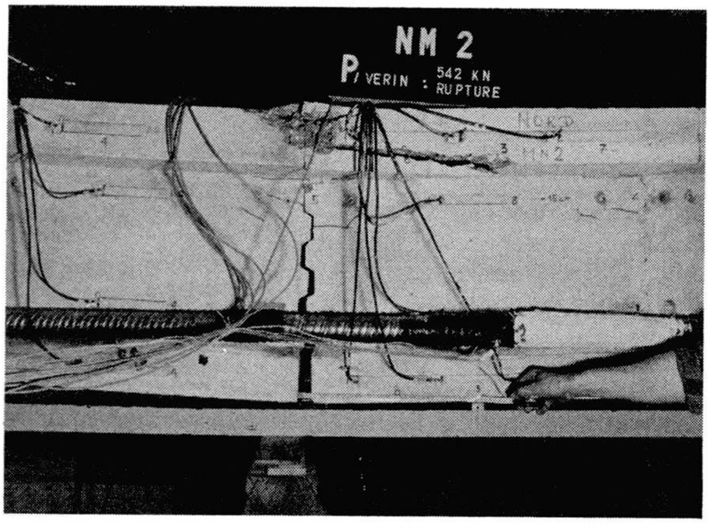


Fig. 8: Concrete splitting in the top slab of precast segments, in a widely open joint. View of a test at Saint-Rémy-lès-Chevreuse (photo C.E.B.T.P.)

must then be conveniently reinforced (figure 9), and the structure ductility and rotational capacity will come from the joint openings and from the ductile deformations of these zones.

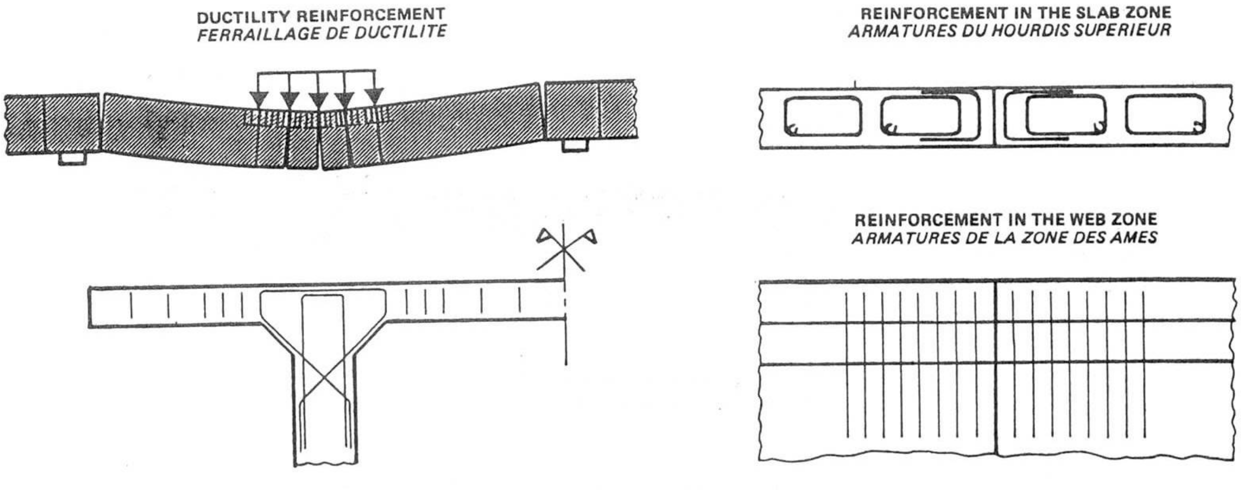


Fig. 9: Reinforcement arrangement to produce the joint ductility

4.2.4. Transfer of shear forces

We now only have to deal with the distribution of shear forces. The single problem is the transmission of shear forces in open joints. Of course, the problem is critical only near the supports, since shear forces are extremely limited at mid-span. We then shall only consider the case of an open joint near a support, but the same principles can apply in other areas.

At first, we must check that the shear force can pass in the joint. In fact, the importance of the compressive force solves the problem as we can show by two different approaches:

— Though glue must have hardened in practically all situations where we have to make an ULS analysis, we can re-use the model that we used for the SLS analysis. We can then consider that the normal force in the nodes of the compressed part of the joint – and not in the bottom slab except if it is very thick, due to its limited transversal rigidity –, noted N_n, must at least balance the shear force:

$$N_n > V \, \mathrm{tg}\, \alpha$$

where α still is the shear-key angle. We just have to check that the supporting areas of the shear keys in the nodes are not out of scale with the reaction that they must receive:

$$R = \frac{V}{\cos \alpha}$$

— We can also consider that, due to glue hardening, we have a classical friction problem, with an angle φ. The shear force has then to be balanced by the normal force in the nodes:

$$N_n > \frac{V}{\mathrm{tg}\, \varphi}$$

If we accept for tg φ the classical value of 1,0, the two approaches lead to the same conclusion. But the first one gives an idea of necessary area of the shear keys.

Practically, prestressing forces are such as compared to shear forces that there is no real question of shear force transmission in open joints.

But we have not finished our analysis: the shear force is now located in the lower nodes on the support side (figure 10), and has to be transfered to the support. Stirrups must be introduced to lift this shear force in the webs, from where it can be transfered to the support, in a strut-and-tie system. The stirrups have to be concentrated in a distance which must be related to the height of the compressed zone in the joint. Their length depends on the distance between the open joint and the support, and also from the desired inclination of the inclined concrete struts transfering forces to the support.

This point can be extremely critical, because it can be problematic to place the necessary area of stirrups. This can lead to a desired limitation of the joint opening. Fortunately, we already said that joints near supports are generally in safer conditions that at mid-span, and are later open. This can save the problem.

At last, different systems can be adopted to transfer this shear force from the open joint to the support. A part of the shear force can be directly transfered if the distance is limited. But, anyhow, the problem must be seriously considered if the joint can be widely open in ultimate conditions.

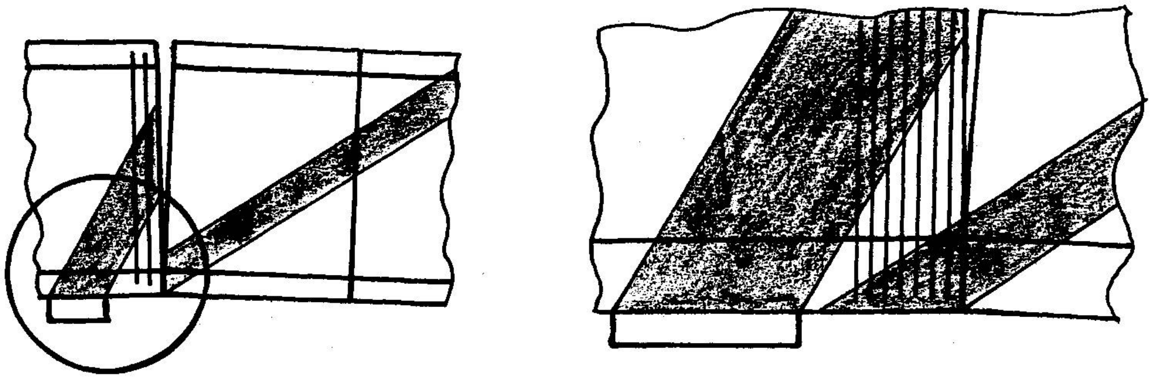


Fig. 10: Transfer of shear force from the open joint to the support, and the necessary reinforcement

4.3. Conclusion

Some aspects of our presentation can look surprising, or at least unusual. But, even if practical analyses have never been done that way, engineers who developped the segmental construction had an excellent foreview of the real problems, as the evolution of the design of shear-keys – mainly due to Pierre Thivans – clearly evidences it.
And finally, there is no reason to be specially anxious: bridges built according to good design criteria will easily fulfill the specifications of such analyses. If prestressing forces balance a great part of the effects produced by permanent loads – both bending moments and shear forces –, if the prestressing forces produce the necessary compression to balance extreme SLS loads, and if the distribution of tendons is satisfactory, not far from corresponding to continuous tendons on the whole span, there is little chance that great problems could occur: shear keys will easily transfer shear-forces, and the joints near supports will be almost closed – if not completely under compression – in ultimate conditions, under ULS loads, so that the shear force transfer from joint to support will not need impracticable reinforcement.

5. CONCLUSION

Theories are not religions. Codes are not bibles. Theories and models are just tools for a good design and a great safety. Codes are just laws to avoid unskillful use of theories and models, and unprudent evaluation of loads.
Such a Colloquium has as a goal to help engineers for a better understanding of structural concrete behaviour. To propose them better models for the analysis of the different problems that they have to solve: different tools.
Each engineer will choose the most adapted for him, according to his experience and formation, and of course to the nature of the questions.
But these models and theories, once again, are nothing but tools, used for the verification of the invented structure.
What is the most important is the invention process, and the criteria for the evaluation of dimensions, reinforcement and prestressing forces that each engineer builds for himself. The efficiency and the simplicity of the structural concept is the best key to structural safety.

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