

Zeitschrift: IABSE reports = Rapports AIPC = IVBH Berichte
Band: 62 (1991)

Rubrik: Theme 4: Impact on future structures

Nutzungsbedingungen

Die ETH-Bibliothek ist die Anbieterin der digitalisierten Zeitschriften auf E-Periodica. Sie besitzt keine Urheberrechte an den Zeitschriften und ist nicht verantwortlich für deren Inhalte. Die Rechte liegen in der Regel bei den Herausgebern beziehungsweise den externen Rechteinhabern. Das Veröffentlichen von Bildern in Print- und Online-Publikationen sowie auf Social Media-Kanälen oder Webseiten ist nur mit vorheriger Genehmigung der Rechteinhaber erlaubt. [Mehr erfahren](#)

Conditions d'utilisation

L'ETH Library est le fournisseur des revues numérisées. Elle ne détient aucun droit d'auteur sur les revues et n'est pas responsable de leur contenu. En règle générale, les droits sont détenus par les éditeurs ou les détenteurs de droits externes. La reproduction d'images dans des publications imprimées ou en ligne ainsi que sur des canaux de médias sociaux ou des sites web n'est autorisée qu'avec l'accord préalable des détenteurs des droits. [En savoir plus](#)

Terms of use

The ETH Library is the provider of the digitised journals. It does not own any copyrights to the journals and is not responsible for their content. The rights usually lie with the publishers or the external rights holders. Publishing images in print and online publications, as well as on social media channels or websites, is only permitted with the prior consent of the rights holders. [Find out more](#)

Download PDF: 12.02.2026

ETH-Bibliothek Zürich, E-Periodica, <https://www.e-periodica.ch>

Theme 4
Impact on Future Structures

Leere Seite
Blank page
Page vide

Consistent Design and New Systems for Concrete Bridges

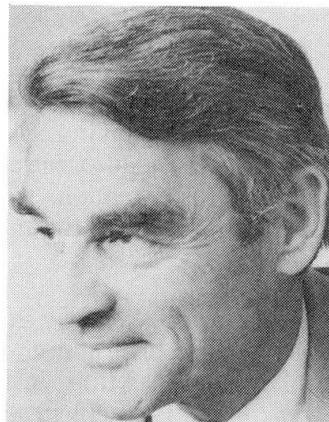
Systèmes traditionnels et innovatifs pour des ponts en béton

Traditionelle und neuartige Tragsysteme für Betonbrücken

Christian MENN

Dr. sc. techn.

Swiss Fed. Inst. of Technology
Zurich, Switzerland



Christian Menn was in private practice from 1957 to 1971. During this time, he was primarily active in bridge design. Since then, he has been Professor of Structural Engineering at the Swiss Federal Institute of Technology (ETH) in Zurich and has been active as consultant on many large bridge projects.

SUMMARY

The arrangement of reinforcement and dimensioning of prestressing steel have a major influence on quality and economy of concrete structures. This paper outlines criteria for traditional and new structural systems that are of fundamental importance in this regard.

RÉSUMÉ

La disposition des acier d'armature et le dimensionnement de l'acier de précontrainte ont une influence majeure sur la qualité et l'économie des constructions en béton. Le présent exposé présente des critères importants à cet égard pour des ouvrages traditionnels et innovatifs.

ZUSAMMENFASSUNG

Die Anordnung der Bewehrung und die Bemessung der Vorspannung haben einen entscheidenden Einfluss auf die Qualität und die Wirtschaftlichkeit von Betontragwerken. Im vorliegenden Beitrag wird aufgezeigt, welche Kriterien diesbezüglich bei traditionellen und neuartigen Tragsystemen grundsätzlich zu beachten sind.



1. RETROSPECTIVE LOOK

1.1 Introduction of Prestressed Concrete

By the late 1930s, a relatively high state of development had already been achieved in reinforced concrete construction. Girder bridges had been built with spans of nearly 70 m. The absence of a satisfactory means of controlling cracking and deflections, however, would prevent the construction of greater spans. Arch bridges, on the other hand, had been constructed with spans of over 100 m. Spans of arches were limited only by economy and technical problems in the construction of falsework. The introduction of prestressed concrete after World War II made possible a much wider range of spans for girder bridges.

Prestressed concrete originally consisted of two different concepts: (1) prestressing with steel bonded to concrete, developed in France, and (2) prestressing with external, unbonded steel, developed in Germany. The unsatisfactory behaviour of early externally prestressed bridges led to the dominance of the French concept of bonded prestressing in pre-tensioned and post-tensioned construction. As a result, prestressing wires or tendons were almost without exception located within the concrete section. A rapid development of optimal cross-section types for prestressed girder bridges took place soon after World War II.

In the 1950s and 1960s, developments occurred mainly in construction technology, including cast-in-place cantilever construction, span-by-span construction, incremental launching, precast girders with cast-in-place deck slab, cast-in-place construction on launching girders, and precast segmental cantilever construction. These new methods of construction resulted in cost savings much greater than improvements in design methods or higher material strengths. They enabled costs for falsework and formwork, which account for roughly 25 percent of total construction cost, to be reduced by approximately one third.

1.2 Fundamentals of Dimensioning

Freyssinet understood prestressed concrete as a fundamentally new method of construction, in which tensile stresses in concrete were completely prevented. Through this concept, known as *full prestressing*, he decisively influenced the dimensioning of prestressed concrete. It was thus common in the 1950s to dimension prestressed concrete girders to prevent tensile stresses at the extreme fibres of the cross-section. Experience with continuous T-girders and skew slab bridges soon revealed that full prestressing required a disproportionately high consumption of prestressing steel. As a consequence, *limited prestressing*, by which restricted tensile stresses were permitted, was soon introduced in several design standards.

Checks of safety at ultimate limit state, already common in the 1950s, revealed that the actual factor of safety of structures with full or limited prestressing is usually much higher than the values specified in standards. This led to the development of *partial prestressing*, by which the entire reinforcement, mild and prestressed, was dimensioned on the basis of safety at ultimate limit state. Partial prestressing resulted in additional economy and a unified concept of safety for reinforced and prestressed concrete.

The cost savings resulting from partial as opposed to full prestressing normally amount to only 1.5 percent of total construction cost. More important is the unified philosophy of safety for reinforced concrete and prestressed concrete that results from partial prestressing, which opens the entire range between classical reinforced concrete and fully prestressed concrete. This enables engineers to choose freely the area of prestressing steel based on criteria relating to quality, economy, and good detailing practise.

1.3 Experiences

Experience obtained from the vast number of prestressed concrete structures built thus far has proven that prestressed concrete has no fundamental deficiencies. Defects occur only due to problems with

materials, lack of care in design, detailing, and construction, or insufficient protection against unforeseen actions. (Deicing chemicals, for example, were not used in Switzerland prior to the 1960s. The reinforcement of many bridges designed before this time is inadequately protected against corrosion according to current standards.)

Experience has also shown that tensile stresses can occur in all types of structures, even fully prestressed, due to self-equilibrating states of stress. Such states of stress, which are difficult to quantify, can lead to the formation of cracks. Fortunately, these cracks are harmless provided sufficient mild reinforcement is provided to distribute them and to limit their width. Partially prestressed structures with sufficient mild reinforcement thus normally have better cracking behaviour as compared to fully prestressed structures with insufficient mild reinforcement.

2. NEW DESIGN CONCEPTS

2.1 Safety and Serviceability

Most new design concepts prescribe checks of safety and serviceability.

The check of safety normally serves as the basis of dimensioning. Forces and stresses due to actions, increased by appropriate load factors, are compared to structural resistance, decreased by a resistance factor. This simple convention has proven itself and is now firmly established. It nevertheless neglects many sources of risk, which are just as important as the statistical dispersion of actions and material strength and which must be considered in any comprehensive approach. These include, for example, changes in actions that occur over time and loss of structural capacity. The margin of safety must also account for small deficiencies in analysis, design, and detailing. Margins of safety should, moreover, be varied according to risk, economic importance, probability of unforeseen actions, and service life of structures. Engineers should be aware of these additional sources of risk and should not be tempted to make drastic reductions in factors when loads and resistances can be measured exactly (e.g. in existing structures). This way leaves little or no margin of safety for the previously mentioned additional risks.

The check of serviceability ensures durability, proper functioning, and undamaged appearance of structures. All requirements specific to the structure with regard to actions, structural system, and use should be compiled. The combination of material technology, detailing, and design that satisfies these requirements must then be selected.

2.2 Significance and Purpose of Prestressing

Prestressing technology enables the use of high-strength steel in concrete. This results in significant economic advantages, both direct (use of steel with high ratio of tensile strength to cost) and indirect (concrete sections are lighter and more slender).

An additional and even more important advantage of prestressing is its favourable effect on deformations and cracking. The prestressing force can be chosen according to economic criteria and performance requirements specific to the structure. (This is especially true for partially prestressed structures.) Deformations due to dead load can be compensated fully or to whatever extent is desirable and the cracking load (i.e., the load corresponding to the first formation of cracks) can be significantly increased. Sufficient mild reinforcement must be provided to distribute and limit the width of cracks resulting from restrained deformations. The quantity of mild reinforcement is normally independent of the magnitude of the cracking load. In addition, the cracking sectional force should be distributed over an extensive area when the cracking load is reached, to ensure cracks are well distributed. Sharp peaks in the cracking sectional force diagram (e.g. when prestressing tendons are coupled at construction joints) must be more strongly reinforced or avoided altogether.



3. FUNDAMENTALS OF ANALYSIS, DIMENSIONING, AND DETAILING

3.1 Design Procedure

In Switzerland, the design of bridges is normally divided into the following three phases:

1. Preliminary design
2. General design (for tender)
3. Final design (for construction)

In the preliminary design phase, several alternate systems are drawn at small scale and are checked or dimensioned as required with rough, simplified calculations. Reinforcement can be determined in this way to within 10 or 20 percent. It is essential that all critical cross-sections and details be worked out and drawn at large scale in this phase. Neglecting this important component of preliminary design can lead to compromises on details during later phases of design. This ultimately results in difficulties in construction and often in damage to the structure (fig. 1).

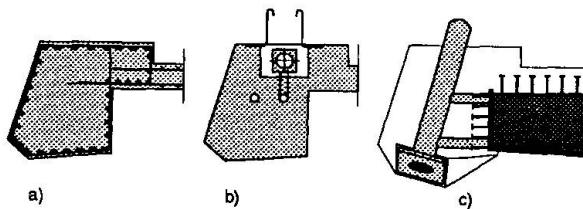


Fig. 1 Example of uncoordinated (unbuildable) detailing in general design: (a) Arrangement of mild reinforcement without consideration of splices; (b) Prestressing tendon; (c) Anchorage of stay cable

In the general design phase, analyses of greater precision are carried out to finalize cross-section dimensions and to determine reinforcement to within 5 percent. Simplified statical models are normally sufficient, however, even at this phase. All structural details, including splices of reinforcement, anchorage of tendons, and accessories (utility ducts, drainage, etc.) should be checked for constructability and finalized.

In the final design phase, the structural system and its components are checked using "exact" methods of analysis based on models that closely match actual conditions. Dimensions of cross-sections should not require modification at this stage. Provided details have been checked during the preliminary phase and finalized during the general phase, the constructability of the final design will be ensured and the prerequisites for high structural quality will have been satisfied.

3.2 Arrangement of Reinforcement

Reinforced concrete is a composite material in which tensile forces are resisted by steel reinforcement in practically all cases. The tensile strength of concrete may only be considered to a limited extent, and only when tensile stresses due to restrained deformations cannot occur.

The state of equilibrium of internal tension and compression elements is just as important as the external state of equilibrium of the entire system. The latter, however, normally receives much more analytical effort than the former. The effectiveness of reinforcement dimensioned on the basis of equilibrium of the entire system must be carefully checked using the state of equilibrium formed by the flow of internal forces.

The internal state of equilibrium can be easily and simply determined using truss models. These should correspond to some degree to the flow of forces in a homogeneous medium. The following points must be observed:

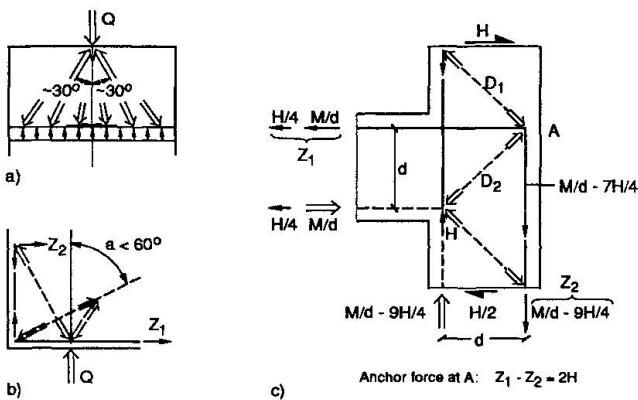


Fig. 2 Use of truss analogy to determine state of internal equilibrium

1. The angle at which concentrated forces spread out should be roughly 30 degrees on either side of the line of action of the force (fig. 2a)
2. The angle of deviation of force varies according to the presence of an active compressive force (e.g. due to prestressing) and the anchorage length of reinforcement. When no active compressive force is present, this angle should not be greater than 60 degrees (fig. 2b)
3. Connections of tension and compression elements should be detailed taking into account the location of anchor forces, local bearing stresses, and the corresponding spreading forces (fig. 2c)

The use of the truss analogy is not limited to the calculation of the flow of forces in corners of frames and other connections of members. It is also valid for forces within members, in particular for determining flexural development of reinforcement and for verification of splices in reinforcement. In box cross-sections, for example, the truss analogy leads to simple and clear calculation of flexural development length, anchorage length, and associated transverse reinforcement (fig. 3).

The truss analogy is a reliable tool in design practise for determining the flow of internal forces required for internal equilibrium. Figure 4 shows an example in which flexural development length (internal equilibrium) has not been considered. As a consequence, the flexural resistance of the system is insufficient to withstand the forces required for external equilibrium.

It is unnecessary to use methods of greater precision to minimize reinforcement, since arrangements of reinforcement that are clear, simple, and easy to place lead to better quality and lower construction costs. These factors outweigh any small savings in quantity of steel that may result from greater analytical precision.

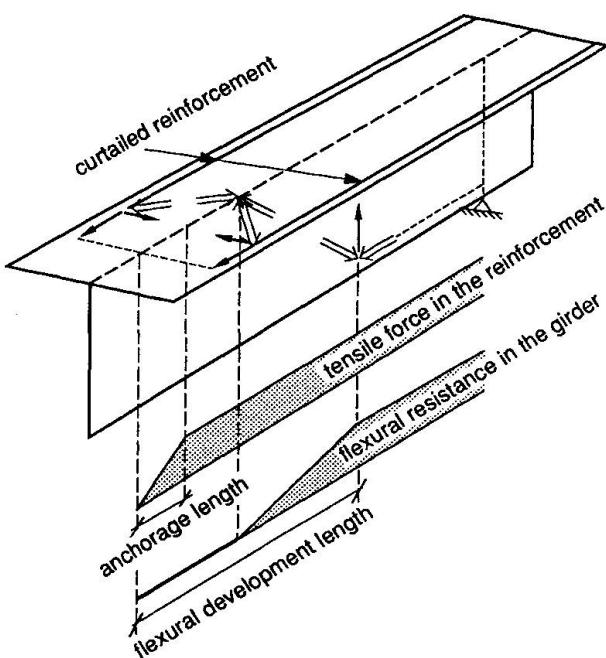


Fig. 3 Flexural development length

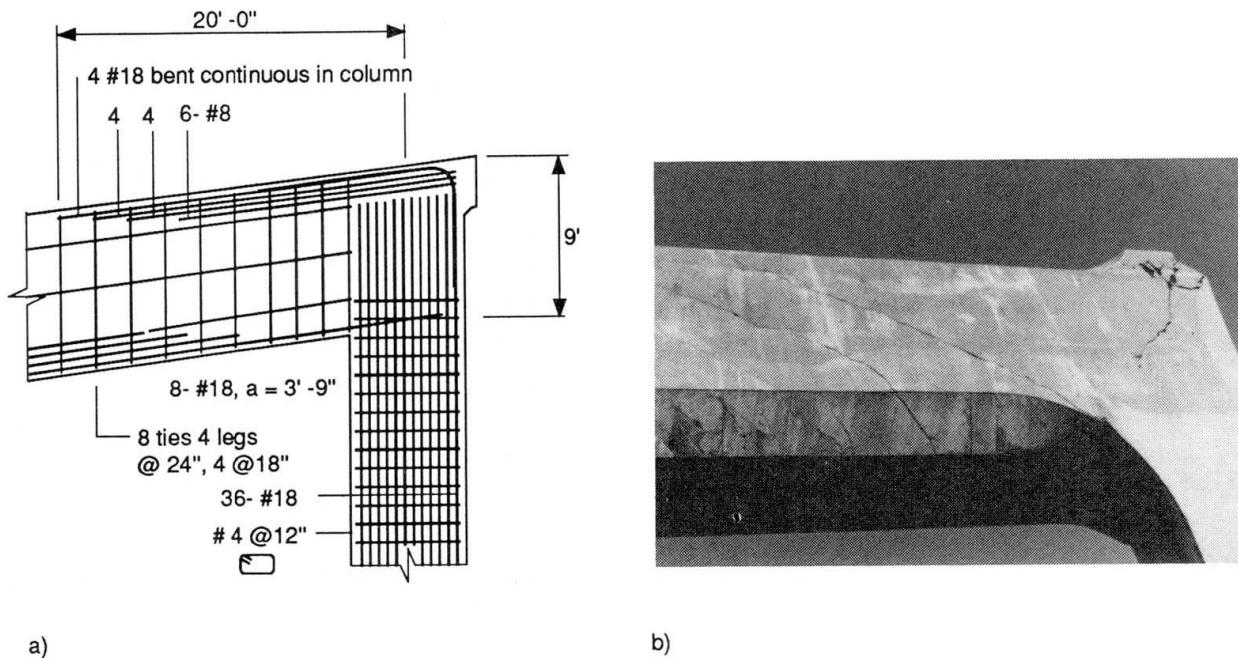


Fig. 4 Frame corner with insufficient flexural development length: (a) Reinforcement; (b) Pattern of cracks

4. NEW DEVELOPMENTS

With the exception of the first externally prestressed concrete bridges in Germany, tendons in girder bridges have almost always been located within the concrete cross-section. Fundamentally new concepts for the arrangement of prestressing appeared only with the construction of cable-stayed bridges, which are essentially none other than cantilevered girders with cables as tension chord and deck girder as compression chord. The stay cables, i.e. the main longitudinal reinforcement, are arranged outside the concrete cross-section and prestressed to produce the desired profile in the roadway. Due to the relative flexibility of the girder, stay cables cannot be prestressed as high as in classical prestressed concrete bridges. The prestressing force in cable-stayed bridges is roughly 30 percent of the yield stress.

In recent years, girder bridges prestressed with unbonded tendons have been built in increasing numbers. External tendons are normally placed inside box girders or between the webs of T-girders. Unbonded prestressing has the advantage that the force and condition of tendons can be continually monitored and, when necessary, tendons can be replaced or strengthened. The use of external tendons in precast segmental girders has the additional advantage of eliminating problems associated with leaking of grout at joints between segments. The disadvantage of unbonded prestressing is that at ultimate limit state, yield stress of the tendons is not always reached. A greater area of prestressing steel is thus required to ensure safety.

It is not necessary and normally not practical to use conventional box or T-cross-sections for externally prestressed bridge girders. These cross-sections, relatively difficult to build, were used with bonded tendons because of the protection they provided for prestressing steel. Their stiffness is generally much larger than required for user comfort with regard to deflections and vibrations, even when partially prestressed.

The following cross-section types can be economical and practical for externally prestressed bridge girders, depending on construction method:

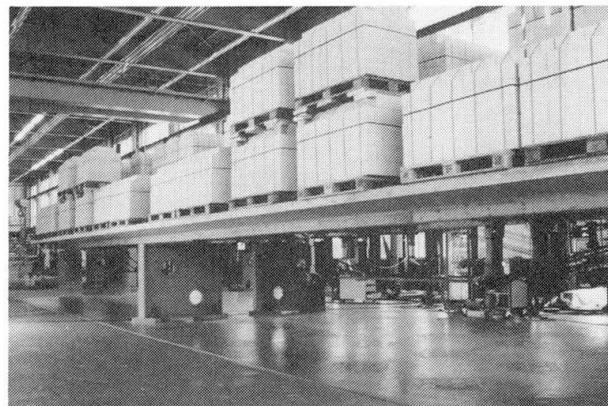


Fig. 5 Scale model of externally prestressed slab bridge

(a) *Girder constructed on conventional or mechanized falsework.* External tendons can be arranged beneath the cross-section, resulting in a total structural depth of roughly 1/15 of the span. For the concrete girder proper, a depth of only 1/40 of the span is necessary. The most appropriate type of cross-section is a compact T-girder with wide webs. Even though deflections and vibrations of such a system may be greater than for conventional girders, they are normally still within acceptable limits. Flexural stiffness can, if necessary, be increased (fig. 5). Girders with external tendons beneath the cross-section have the following advantages:

1. A simple shape for the cross-section which eliminates horizontal construction joints and thus improves the quality of the concrete.
2. Good cracking behaviour compared to externally prestressed box girders, since the cracking moment is reached only after relatively large deformations of the girder and increases in strain in the external tendons.
3. Ease of adjustment of roadway profile by subsequent stressing of external tendons to compensate for long-term deflections
4. Excellent ductility and behaviour at ultimate limit state. External tendons and internal reinforcement reach yield at roughly the same load.

(b) *Launched girders.* It is normally more economical to launch steel plate girders or steel trusses for the webs. Following this, a bottom slab can be cast near the supports, followed by the deck slab, both in concrete. External tendons are then put into place and anchored into the concrete. They are stressed corresponding to the progress of construction of the deck slab.

Cable-stayed bridges have normally been built with tall, slender towers to minimize the quantity of steel in the stay cables. This arrangement results in large bending moments in the towers due to partial live load, which are normally reduced through the use of backstay cables. A flexurally stiff girder is also required when cables are arranged in a harp pattern.

Stiffer, lower towers enable the use of the full range of effective depth of cross-section, from classical cantilever girders to classical cable-stayed bridges (fig. 6). In such systems, the tower acts as a cantilever and requires neither backstays nor a flexurally stiff girder. Since the tower is effectively prestressed by the axial force due to dead load of the girder, relatively little reinforcement is required to resist bending moments due to live load. This system enables the use of slender girders and side spans which can be greater than 50 percent of the main span length since backstay cables are no longer required. Although such systems require a greater consumption of steel in the stay cables, savings in tower costs are possible. For multiple-span cable-stayed bridges with tall piers, this system results in favourable structural behaviour and an aesthetically convincing form.

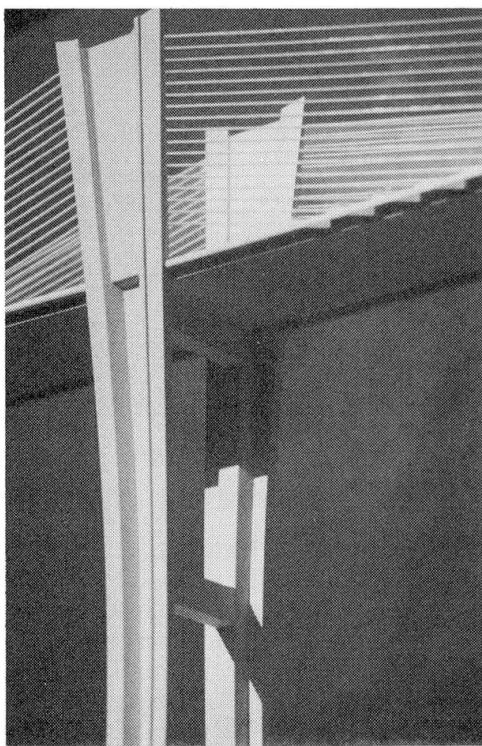


Fig. 6 Model of flexurally stiff, low tower for cable-stayed bridges (no backstay cables required)

Regardless of whether bonded or unbonded tendons are used, the area of prestressing steel is dimensioned together with any other reinforcement to ensure safety at ultimate limit state. Although this principle is also valid for cable-stayed bridges and bridges with external tendons beneath the cross-section, secondary effects such as curvature of the cable may also need to be considered. When unbonded tendons are used, the actual stress in the prestressing steel at ultimate limit state must be checked.

The initial stress in prestressing steel varies according to structural system. For box girders with bonded or unbonded tendons, the initial stress is taken as high as possible, normally 70 percent of yield stress. This ensures that the cracking load is as high as possible and the increase in strain required to reach yield at ultimate limit state is as small as possible. (Because of loss of prestress due to friction, relaxation, shrinkage, and creep, this increment of strain is normally significantly higher than the yield strain of mild reinforcement.) Undesirable deformations of the girder due to prestressing will normally not occur when the area of prestressing steel is dimensioned according to safety at ultimate limit state.

For cable-stayed bridges and bridges externally prestressed from beneath the cross-section, the initial stress in the prestressing steel is completely determined by the deformation of the girder, i.e. the prescribed roadway profile. For such systems, the initial stress normally ranges between 30 and 40 percent of yield stress. Provided the system is sufficiently ductile and properly detailed, it is also possible to reach yield in prestressing steel at ultimate limit state.

5. CONCLUDING REMARKS

The arrangement of reinforcement, anchorage lengths, and splice lengths can be simply and reliably determined with the help of the truss analogy of the internal state of equilibrium. The internal state of equilibrium, i.e. the state of equilibrium of the internal tension and compression elements, is just as important as the external equilibrium of the system. The latter is often given much more than its share of analytical attention, with negligible benefit.

Dimensioning of the area of prestressing steel must always be based on safety at ultimate state. The initial stress in prestressing steel should be chosen as high as possible to improve cracking behaviour, without exceeding prescribed limits on deformations due to permanent load.

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

Michel VIRLOGEUX

Ing. en Chef
SETRA
Bagneux, France

Michel Virlogeux, born 1946, graduated from École Polytechnique (1967) and École Nationale des Ponts et Chaussées (1970). He is Docteur Ingénieur of the Pierre et Marie Curie University (Paris 1973). He works as civil servant since 1970, in the SETRA (Bagneux) since 1974. Head of the Large Bridges Division since 1980. He is part-time Professor at the École Nationale des Ponts et Chaussées. He received the IABSE Prize in 1983.

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 personally 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 developed 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 developed 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 no 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 statement.

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 developed 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 personal 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 questioned 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 developed 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 loses 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 it is 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, dimensioned 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 ($P \cdot \Delta e$), practically equivalent to an uplift lineic density of load:

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

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.

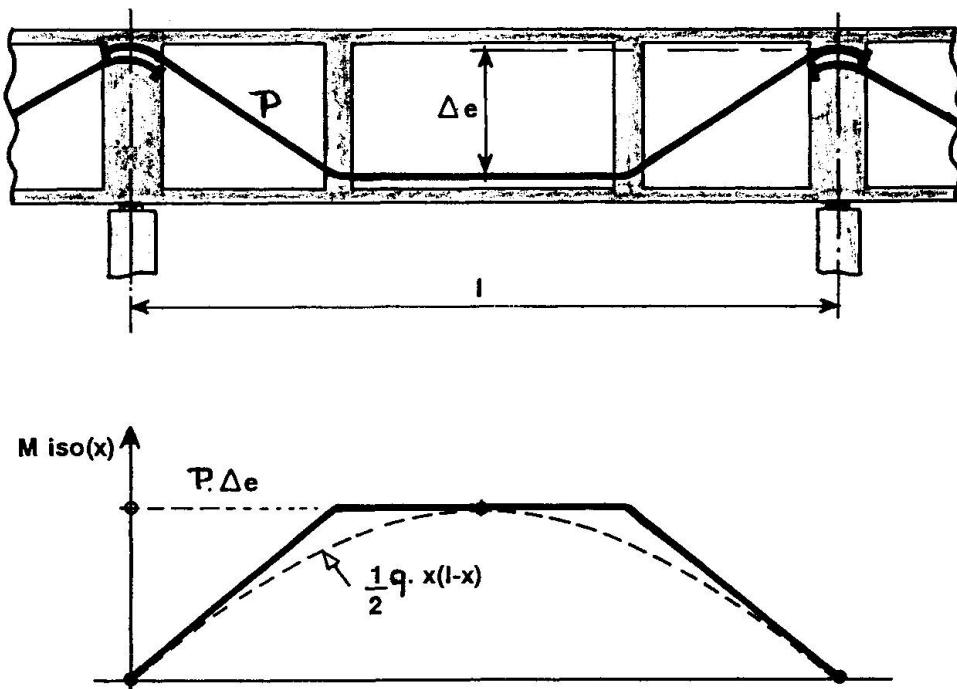


Fig. 1: Definition of an external tendon undulation, Δe , and the produced bending forces

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 additional 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 side¹.

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.

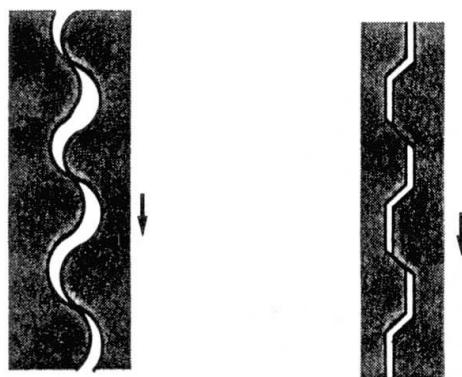
Everybody knows that the most important problems appear when a new segment is placed: the glue has not yet hardened and acts as a lubricant, favourably for easing the segment adjustment to its exact position, but unfavourably for its static equilibrium when placed; and the prestressing forces which fix 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.

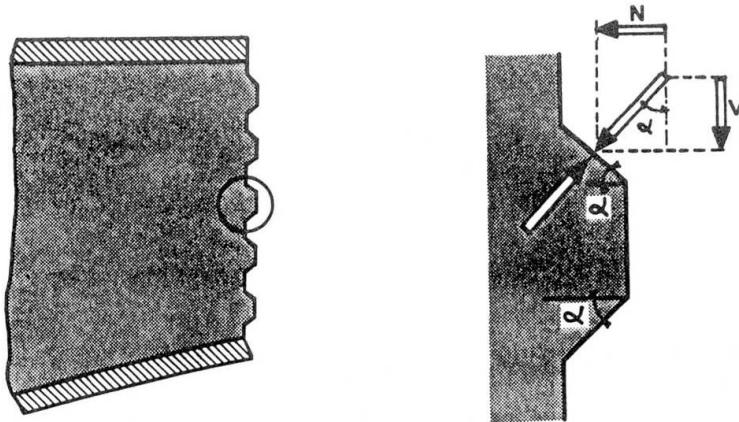


Fig. 3: Mechanical behaviour of multiple shear-keys

The rest of the normal force:

$$N_j = N - V \tan \alpha$$

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 transferred 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:

$$P_{\min} = V \tan \alpha$$

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

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.

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:

$$N_{sk} = V \tan \alpha$$

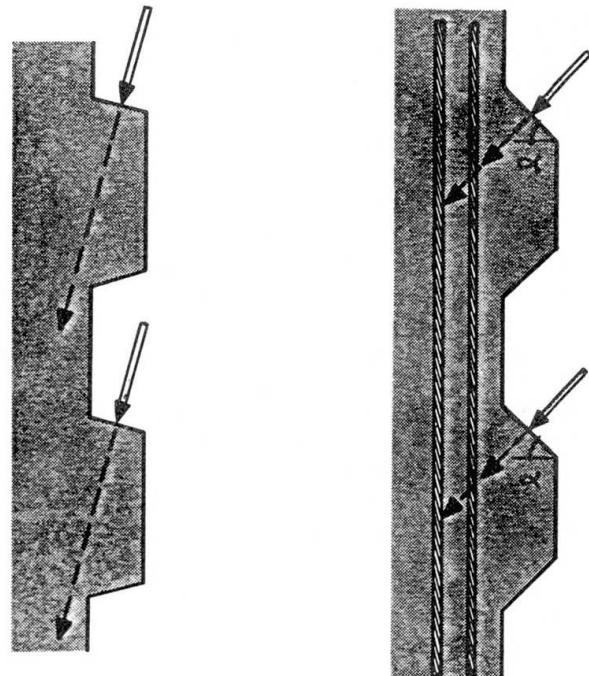


Fig. 4: Influence of the shear-keys shape on the transmission of forces



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;

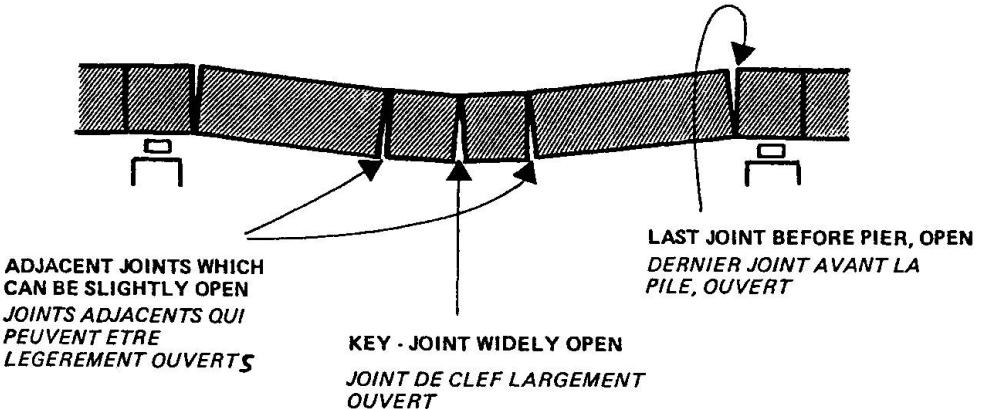


Fig. 5: Ultimate behaviour of a segmental beam

- 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?

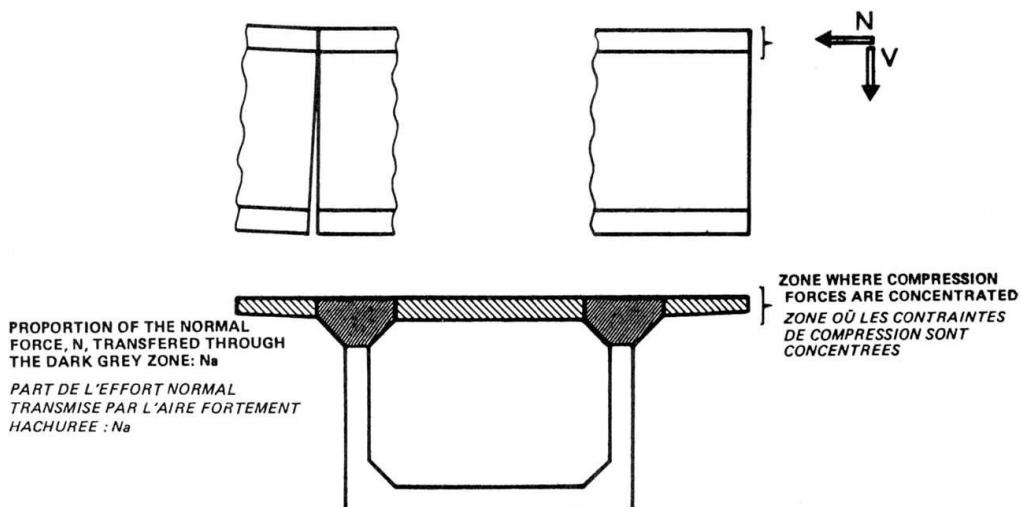


Fig. 6: Proportion of the compression stresses balancing shear forces in the Ultimate Limit State conditions, with a positive bending moment

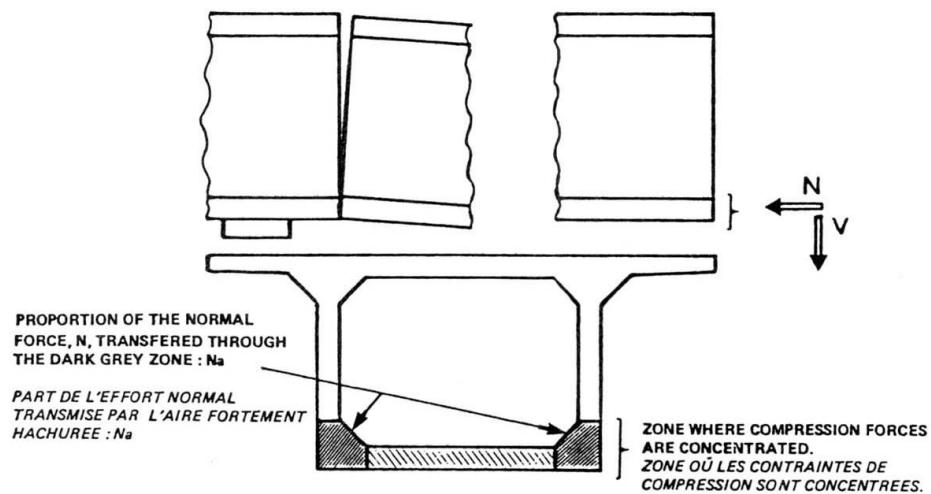
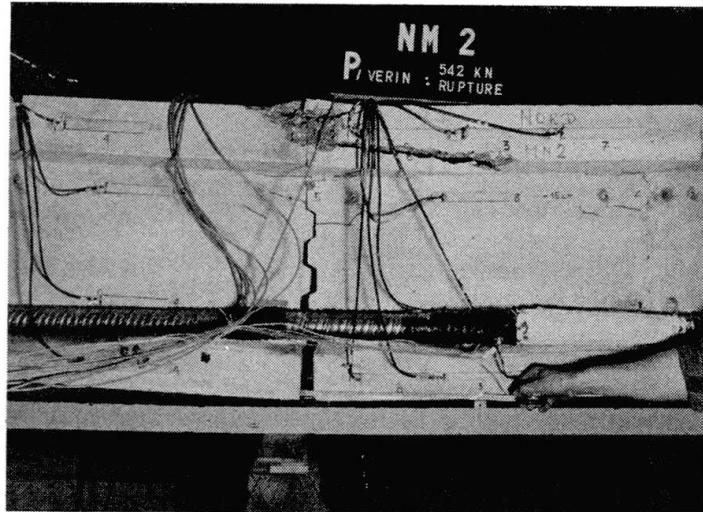


Fig. 7: Proportion of the compression stresses balancing shear-forces in the Ultimate Limit State conditions, with a negative bending moment

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.

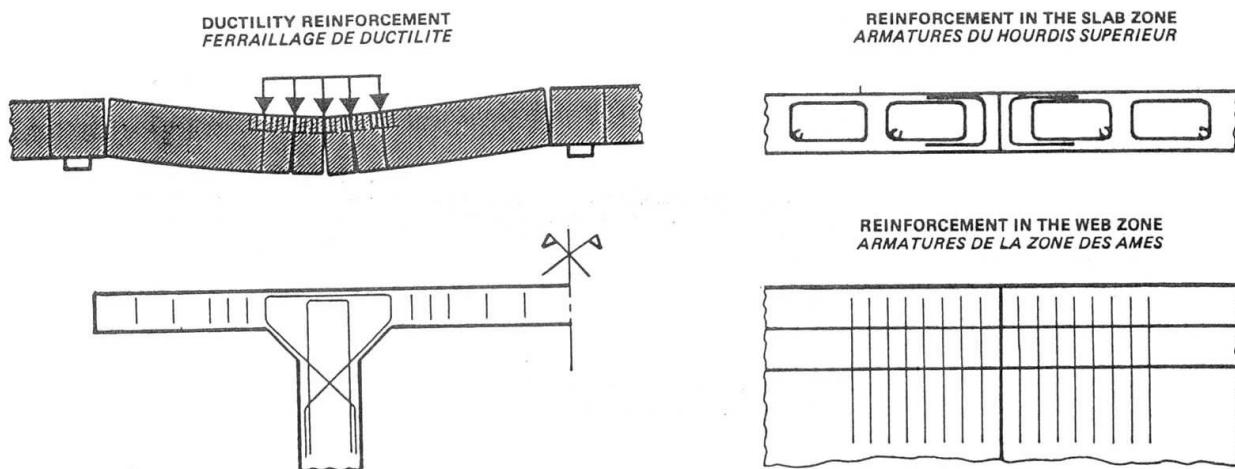


Fig. 9: Reinforcement arrangement to produce the joint ductility

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 reuse 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:

$$N_n > V \operatorname{tg} \alpha$$

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:

$$N_n > \frac{V}{\operatorname{tg} \phi}$$

If we accept for $\operatorname{tg} \phi$ 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 transferred to the support. Stirrups must be introduced to lift this shear force in the webs, from where it can be transferred 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 transferring 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 than 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 transferred if the distance is limited. But, anyhow, the problem must be seriously considered if the joint can be widely open in ultimate conditions.

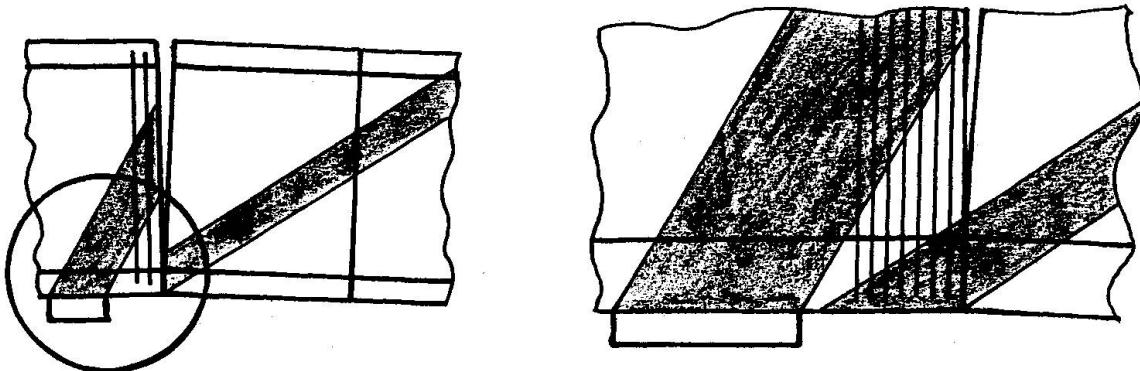


Fig. 10: Transfer of shear force from the open joint to the support, and the necessary reinforcement

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 developed 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.

REFERENCES

1. SCHLAICH, J., K. SCHÄFER and M. JENNEWEIN, Towards a consistent design of structural concrete. PCI Journal, May/June 1987, Vol. 32, No. 3, p. 74 to 150.
2. MARTI, P., Trussmodels in Detailing. Concrete International, December 1985, Vol. 7, No. 12, p. 66 to 73.



3. FOURÉ, B., and M. VIRLOGEUX, Précontrainte extérieure au béton: comportement jusqu'à rupture de poutres à voussoirs préfabriqués. Supplément au numéro de Travaux de Janvier 1986, Dixième Congrès de la FIP à New Delhi, A.F.P.C., p. 71 à 80.
4. VIRLOGEUX, M., Non-linear analysis of externally prestressed structures. FIP Symposium, Jerusalem, September 1988, p. 319 to 340.
5. P.C. de R. MARTINS, Modélisation du comportement jusqu'à la rupture en flexion de poutres en béton à précontrainte extérieure ou mixte. Thèse de doctorat de l'École Centrale de Paris, Septembre 1989.
6. VIELHABER, J., Vorspannung ohne Verbund im Segmentbrückenbau. Technische Universität, Berlin, Berichte aus dem Konstruktiven Ingenieurbau, Heft 8, D 83, 1989.
7. VIRLOGEUX, M., Non-linear analysis of externally prestressed structures. La Technique Française du Béton Précontraint, 11ème Congrès de la FIP, Hambourg, A.F.P.C., Juin 1990, p. 165 à 193.
8. FOURÉ, B., et P.C. de R. MARTINS, Flexural behaviour up to failure of externally prestressed concrete beams. La Technique Française du Béton Précontraint, 11ème Congrès de la FIP, Hambourg, A.F.P.C., Juin 1990, p. 195 à 215.
9. VIRLOGEUX, M., Shear strength of beams made of precast segments. La Technique Française du Béton Précontraint, 11ème Congrès de la FIP, Hambourg, A.F.P.C., Juin 1990, p. 217 à 236.

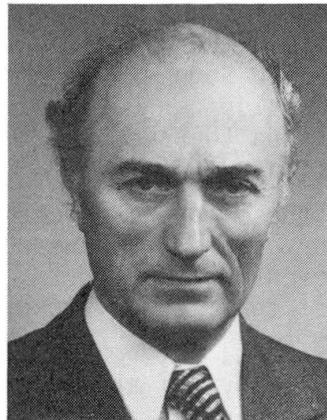
Impact of Rational Approaches on Design Practice

Impact d'une approche rationnelle sur la conception des ouvrages

Einwirkung einer rationalen Betrachtungsweise auf die Entwurfspraxis

Daniël VANDEPITTE

Emeritus Professor
Ghent University
Ghent, Belgium



Daniël Vandepitte, born in 1922, designed a number of bridges at the onset of the prestressed concrete era. He later taught and wrote a three volume book on structural analysis. He acted as a technical consultant, mainly to an agency assessing the safety of civil engineering and buildings structures.

SUMMARY

A few examples of the potential impact of a rational design approach for structural concrete on design practice are discussed, and it is attempted to learn a lesson from rather arbitrary constraints on past design practice. The following topics are commented upon: unreinforced concrete, structural concrete reinforced with only steel fibres, the merits of limited prestressing, external prestressing, mechanical modelling, prestressed concrete suspension bridges, the concept of the prestressed concrete suspension bridge with a triangular system of suspenders, and reliance on the tensile strength of concrete.

RÉSUMÉ

L'impact potentiel d'une approche rationnelle du béton structurel sur la pratique de la conception et du calcul des ouvrages d'art et des bâtiments est illustré par quelques exemples. L'auteur essaie, en outre, de tirer une leçon de restrictions plutôt arbitraires qui entraînaient les projeteurs par le passé. Les sujets discutés sont les suivants: le béton non armé, le béton structurel armé seulement de fibres d'acier, les mérites de la précontrainte restreinte, la précontrainte extérieure, le modelage mécanique, les ponts suspendus en béton précontraint, le concept du pont suspendu à tablier en béton précontraint et à système triangulaire de suspentes, et la prise en compte de la résistance du béton à la traction.

ZUSAMMENFASSUNG

Einige Beispiele der möglichen Auswirkung einer rationalen Betrachtungsweise auf die Entwurfspraxis werden besprochen, und es wird zugleich versucht, eine Lehre aus früheren willkürlichen Einschränkungen zu ziehen. Die folgenden Themen kommen zur Sprache: unarmierter Beton, nur mit Stahlfasern armerter Konstruktionsbeton, beschränkt vorgespannter Beton, externe Vorspannung, mechanische Modellierung, vorgespannte Hängebrücken, das Konzept der vorgespannten Hängebrücke mit dreieckigem Hängersystem und Nutzung der Betonzugfestigkeit.



1. PRELIMINARY REMARKS

1.1 The writer of the present paper does not possess a reliable crystal ball and therefore does not purport to venture any predictions about prospective developments, but will pursue the less ambitious goal of discussing a few examples of more or less unrelated areas where a rational design approach for structural concrete may affect design practice in the future, while trying to learn a lesson from the vicissitudes of past design practice.

1.2 Codes, whether they are legally binding or not, exert a powerful influence on design practice, more so than research reports or symposium proceedings, although part of the content of these may eventually trickle into codes. Engineers who have been active as members of code writing committees know that a new code is generally copied to some extent from a previous edition. Once in a long while a given recommendation can be traced back to a 50 year old predecessor of the code and is found to have been reproduced, possibly verbatim, in a number of successive editions.

The committees which wrote these may have acted wisely in borrowing from older standards. They may know or assume that there were excellent reasons for introducing a particular requirement in the first place. Their decision to retain it may be justified by the fact that it has stood the test of time.

However, it is quite likely that, after a few decades, hardly anybody is aware of the original rationale, if any, of an old recommendation, and, if a number is involved, of the reason why a particular figure has been selected. Many codes were initially written to meet an urgent need and were therefore of limited scope. Re-editing them, essentially unchanged, accounts to some extent for the incompatibility of certain features of codes covering adjacent fields. It is necessary to reassess from time to time the arguments underlying known rules, to delete them or to place them on a more rational footing, if no credible justification can be found. The Colloquium on Structural Concrete sets out to do that systematically and that purpose is entirely commendable.

It should be emphasized, however, that even recommendations regarding structural concrete which are not at variance with any other recommendations regarding structural concrete should not be taken for granted.

1.3 Once in a while, one encounters the notion that a type of structure is not permissible if it is not allowed and codified explicitly in existing standards. That is a deplorable attitude because it stifles innovation.

2. UNREINFORCED CONCRETE

The Final Invitation to the Colloquium on Structural Concrete states that structural concrete represents a continuous spectrum from unreinforced concrete to the most involved combinations of concrete with steel reinforcement. Implicit in this definition is the assumption that present codes allow the use of unreinforced concrete. But do they really?

If unreinforced concrete should be acceptable at all, it would seem to be in structural elements that are subjected almost exclusively to compression by the external loads, i.e. in columns and walls. Yet, the First Draft of the CEB-FIP Model Code 1990 specifies a minimum longitudinal reinforcement of 0,8 % in columns [2, p. 10-5] and a minimum vertical reinforcement of 0,4 % and a minimum horizontal reinforcement of 0,12 % in walls [2, p. 10-6]. It is stated, moreover, that these minimum measures constitute obligatory prerequisites of the application of the Code [1, p. 4-2]. The CEB-FIP Model Code 1978 prescribes the same minimal reinforcement [3, p. 189-191], except that the requirement for horizontal reinforcement in "reinforced concrete walls" is at least 50 % (instead of 30 %) of the vertical reinforcement. The wording of the article regarding precast wall panels in the draft of the 1990 Model Code [2, p.14-9]

does imply, however, that unreinforced walls are permissible, provided that they are precast.

Of course, one can easily imagine reasons why a certain amount of steel in concrete columns and walls is beneficial. For one thing, it hampers cracking and the propagation of damage. For another, it serves the purpose of resisting bending induced in the vertical structural elements by beams connected monolithically with the columns and by slabs connected monolithically with the walls. Reinforcing steel also renders structural elements carrying vertical loads less vulnerable to accidental lateral loading, due, for example, to a horizontal bump against the column or wall or to an explosion. But one may well wonder whether and why exactly reinforcement is really necessary, especially in concrete walls, since concrete has been depicted in recent years as a fairly ductile material.

The minimum percentages, 0,8 % and 0,4 % and 0,12 %, do not appear exorbitant, but they may amount to a lot of steel in walls. Is there a rational basis for these particular figures ?

Asking these questions seems legitimate in view of the existence, in Switzerland and probably elsewhere, of 20 storey buildings whose brick masonry walls are load bearing and do not contain steel or reinforced concrete frames. Those masonry walls are not reinforced. Yet the tensile strength of brick masonry, especially that of its horizontal joints, is definitely lower than that of concrete, and reinforced concrete floor slabs framing into a masonry wall also generate bending.

Do concrete walls subjected to little or no bending and transverse shear need to be reinforced, while masonry walls do not ? Are designers of concrete walls too timid or are designers of masonry walls reckless ?

If a detached examination of these questions led to the conclusion that the designers of tall buildings with load carrying brick masonry walls have not been remiss, that outcome might stimulate the erection of buildings having considerably more than 20 storeys and unreinforced concrete bearing walls.

The "Introduction to the CEB-FIP/MC 90" [1, p. VI] announces the eventual addition of a chapter concerning plain concrete. The wording "plain concrete (such as, for example, mass concrete)" suggests that the chapter will not focus on unreinforced concrete bearing walls, although it may not exclude them.

3. STRUCTURAL CONCRETE REINFORCED ONLY WITH STEEL FIBRES

Promoters of steel fibre reinforced concrete would like to reinforce structural concrete beams and slabs (not resting directly on ground) and tension piles with steel fibres only. One may doubt the economic merit of such structural elements because their fibre reinforcement is located partly in regions where no tensile stresses exist, and in the regions where reinforcement is needed, many fibres are so orientated that they are inefficient.

However, the said promoters are not given the chance to prove that the doubters are wrong, because such applications are prohibited by codes of practice [4]. The ban is based on the possibility that the distribution of the fibres in the concrete may be non uniform and that critical regions may contain too few fibres. The proscription should be lifted if and when mixing methods are developed which ensure uniform dispersion of the fibres and a practical way to ascertain uniformity of dispersion is available.



4. FULL PRESTRESS VERSUS LIMITED PRESTRESS

At the time of the inception of prestressed concrete technology, 50 years ago, full prestress was generally considered as technically ideal. This was partly due to the then dominant view that reinforced concrete and prestressed concrete are fundamentally different materials. Many prestressed concrete beams manufactured in those days had little passive reinforcement or none at all.

Reference [5] describes tests on two prestressed concrete beams of 28,8 m span. They contained neither passive longitudinal reinforcement, nor stirrups. Except for a few unstressed rebars in the end blocks, they were reinforced only with post-tensioned 7mm wires. The beams were taken from a dismantled 30 year old bridge (it would not be easy to find recent beams of such large size and almost totally devoid of passive reinforcement). The girders exhibited a small number of wide cracks under increasing load. They deflected considerably before they failed, and in that sense they were certainly not brittle. Whether they were tough in the sense that a very gradual decrease in their loading would have accompanied still increasing deflection after the load had reached its ultimate value, is unlikely, but could not be ascertained because the tests were not deformation controlled. It is probable that few deformation controlled tests to failure have been carried out on large prestressed concrete beams containing virtually no passive reinforcement. Anyhow, the two beams discussed here collapsed suddenly and explosively by disintegrating into a fairly small number of large pieces of concrete, not by crushing of the compressed flange.

That is a reason for believing that it is in many cases technically better to withstand the tensile forces in structural elements subjected to bending and to shear partially with active reinforcement and partially with passive reinforcement, rather than with active reinforcement alone. Limited prestressing has other well-known mainly economical advantages : less prestressing steel (partially offset by more passive rebars), less room needed in the cross-section for prestressing tendons and ducts, a smaller prestressing force in the precompressed concrete flange and therefore a flange of smaller size, less creep deformation. The reverse side of the coin : the increased importance of the effects of fatigue (which, in the case of railway bridges, may justify the requirement : no tensile stresses in the concrete under maximum service loading), and the increased risk of cracking.

It is highly laudable to strive, as the Colloquium on Structural Concrete does, for methods of design and analysis that allow for a consistent and continuous spectrum of designs corresponding to a wide interval of degrees of prestress. That is quite the opposite of dogmatically denouncing limited prestressing, as one school of thought was prone to do in the past. In the writer's opinion, engineers should, in the future, resort more often to limited prestressing of beams and, in many cases, will thus design structures which are better, technically and economically.

5. EXTERNAL PRESTRESSING

External prestressing has been used in many structures for more than 45 years, practically from the onset of prestressing technology. Prestressing cables may be placed on either side of the web of a concrete plate girder or inside a box girder. References [6, p. 108 and 190] , [7] , [8] and [9, p.6 and 313] are just four among numerous, decades old publications describing structures with external prestressing. There have been mishaps, due to sloppy grouting of the prestressing tendons and to their subsequent corrosion. For that matter, internal tendons have been known to sustain corrosion too. There are, on the other hand, structures with external prestressing that are decades old and in excellent shape.

There were no grounds for rejecting external prestressing on principle. Yet, the concept used to be opposed in some quarters. The 1978 CEB-FIP Model Code [3, p. 63] mentioned it, but otherwise ignored it. External prestressing has been rediscovered in the last few years and advocated in a flurry of fairly recent papers which quite rightly emphasize the considerable advantages that it has had all along. The technology described in those papers hardly differs from practice as it had evolved in previous decades. The essential requirement for durability was and remains care in protecting the prestressing tendons from corrosion.

To be sure, in some cases there may be (almost) no bond between external tendons and the concrete, and the corollaries of that situation should be allowed for in the analysis. The effects of the prestressing force, considered as artificially created external loads [10] and including both a longitudinal compressive load and transverse loads in sections where tendons change direction, decrease gradually as a result of shrinkage, creep and relaxation, but they may increase under live load because the tendons are forced to deflect together with the concrete beam. Bond is seldom lacking completely, though, since the friction generated where the slope of the tendons changes is similar to bond. Furthermore, it is often possible to achieve a more rigid longitudinal connection at such points, if so desired, and the prestressing force then varies discontinuously from one part of the tendons between two successive points of deviation to an adjacent part.

Provided that those features of external prestressing are taken into account in the analysis and construction, it is and always has been just as sound a pre-stressing method as prestressing with internal bonded tendons. The satisfying circumstance that external prestressing is now recognized in the draft of the 1990 CEB-FIP Model Code [1, p. 1-9] enhances the concordance between the code and the facts, and it should encourage designers of concrete structures to take greater advantage of the substantial benefits of that prestressing mode in the future.

6. MECHANICAL MODELLING

The introduction and the acceptance of strut-and-tie models and of B regions and D regions [2, p. 6-1 and 6-16] [11] [12] represent an important progress, even if implementing these concepts may still require much thought and discussion. For example, allowing an angle as low as $\text{arc cot } 3 = 18,4^\circ$ between the imaginary compression diagonals in the web and the chords of a beam subjected to shear [2, p. 6-17] does seem somewhat excessive and its safety should be checked before it is allowed to stand. The strut-and-tie model has great merits :

- It goes back to basics, the lower bound theorem of plasticity theory, and even much further back, to a fundamental principle of statics, the theorem of the triangle of forces enunciated in 1586 by the Flemish engineer and scientist Simon Stevin [13, p. 90].
- It provides insight into the interplay of forces within a structure.
- It is very versatile and enables an engineer exercising his intuition wisely to devise sagacious solutions, even in cases of unusual structural arrangements, which are hardly amenable to treatment by a cut-and-dried method of analysis.

Strut-and-tie models and the concept of B regions and D regions supply designers with powerful design tools.



7. PRESTRESSED CONCRETE SUSPENSION BRIDGES

The first self-anchored suspension bridge whose concrete stiffening girders are prestressed by the main suspension cables was built in 1954 [14]. Figure 1 is an outline of the structural system, reduced to its bare essentials and drawn without the towers, with far too few suspenders and with an exaggerated camber of the stiffening girder. At the time, different design rules applied to reinforced concrete and to prestressed concrete in Belgium. But is the suspended structure reinforced or is it prestressed ? Both views can be argued :

- The stiffening beams do contain a fair amount of passive reinforcement, but there are no prestressing tendons within the depth of the suspended structure. Consequently, it is reinforced concrete.
- Although the prestressing cables are located above, even far above, the concrete floor system, the latter is prestressed by them. Hence it is prestressed concrete.

Incidentally, "prestressed concrete", taken at face value, is a widely applicable general phrase : a concrete arch, for example, is prestressed by its own weight. So is a masonry arch.

What is depicted in figure 1 is an obvious example of a structural system which may be analysed efficiently by conceiving of the prestressing effects as external loads acting upon the concrete component and all proportional (except for second order effects) to the horizontal component of the tensile force in the main suspension cables. The prestressing effects are (fig. 2) : the compressive forces acting upon both ends of the stiffening girder and the forces exerted upwards by the suspenders upon the girder (and including the effect of the camber if a perfectly straight beam is substituted for the actual cambered beam, as has been done in figure 2).

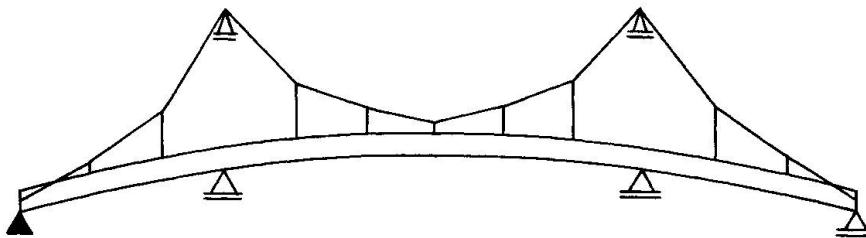


Fig. 1 - Outline of a prestressed concrete suspension bridge

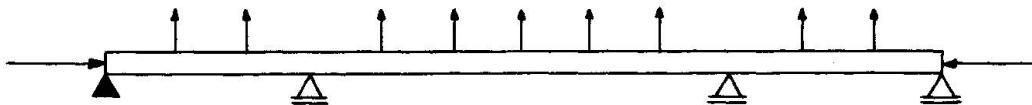


Fig. 2 - Prestressing effects on the stiffening girder of a prestressed concrete suspension bridge

The analysis of the bridge and of similar bridges was based on the simple and transparent model in figure 2. The analysis is expounded in some detail in reference [15]. As in other prestressed structures, the prestressing effects decrease as time elapses. The system could be so designed that the upward forces acting on the floor structure balance the permanent loads at a certain stage, for example immediately after the prestressing operation. A more efficient

design requires, not amazingly, that the upward forces exceed the permanent loads, even after the upward forces have decreased as a result of the gradual loss of prestress.

The combinations of factored loads, which the bridge should be able to withstand before reaching an ultimate limit state, should be the usual combinations and should include i.a. the dead load multiplied by the usual corresponding partial safety factor, say 1,35. Thus, the dead load multiplied by 0,35 and other factored loads should be superimposed on the situation comprising the dead load itself and the (factored) prestressing forces. If, instead of that, the designer applied the factor 1,35 to the sectional forces existing in the cables and in the stiffening girders after the prestressing operation, he would not, the writer believes, be complying with the general spirit of the concept of partial safety factors, since he would fail to take account of the way in which the interaction between the components of the structural system influences the sectional forces in those components at loading stages exceeding the service loads.

8. CONCEPT OF THE PRESTRESSED CONCRETE SUSPENSION BRIDGE WITH A TRIANGULAR SYSTEM OF SUSPENDERS [16]

The main difference between the type of bridge discussed in section 7 and the bridge system outlined in figure 3 lies in the fact that the latter has slanting instead of vertical suspenders. The suspension bridge is self-anchored in both cases and consequently it does not need two external anchorages capable of resisting large horizontal forces. If, given the live load intensity, the spans of the bridge sketched schematically in figure 3 are long enough, the dead load produces so much tension in the inclined suspenders that it is never exceeded by compressive stresses superimposed on it later and generated by the live load. Therefore, the suspenders are never called upon to withstand compression and they may be designed as cables or ropes.

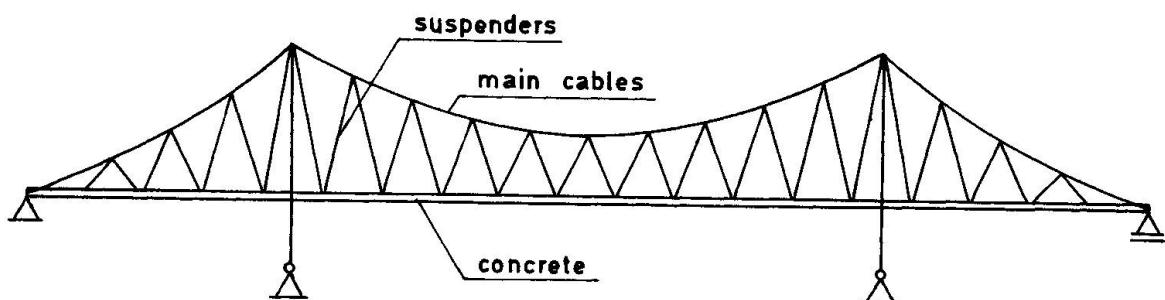


Fig. 3 - Outline of a prestressed concrete suspension bridge with a triangular system of suspenders

Under live load the truss action in the structural system as a whole is so dominant that the suspended concrete floor structure is subject to little longitudinal bending, definitely less than the bending induced in the stiffening girders of the type of bridge schematized in figure 1. In figure 3, the stiffening function is assigned to the truss, thereby obviating the necessity for the deck to possess any flexural rigidity of its own in excess of the rigidity it needs to resist local bending and buckling in between suspenders. As a result, the floor structure may be quite slender. Moreover, it is prestressed longitudinally by the horizontal component of the forces applied to its ends by the main cables,



so that what little longitudinal bending arises from the loads does not require very much passive reinforcement. For all these reasons, the structural system described seems quite economical and attractive (see also 8.1 Note).

However, dimensioning the passive (or active) reinforcement within the suspended concrete deck would not be a straightforward process, if some of the customary design methods were to be utilized. Horizontal prestressing is applied predominantly by external forces at both ends of the bridge and only to a minor degree at the suspension points, where it originates from the slanting suspenders as an outcome of the live load. As in the case of the bridges discussed in section 7, the prestressing cables are the main suspension cables and they are located far outside the depth of the bridge floor. An approach which does not draw a fundamental distinction between reinforced concrete and prestressed concrete, but treats both as variants of the same material, structural concrete, furnishes clear answers to the design questions raised by the types of structure discussed in sections 7 and 8.

8.1 Note

Of course, the structural system, as it is portrayed in figure 3, is able to carry itself only after it is completed. If the topography of the site is such that elevated bridge ramps are necessary, a construction procedure can be devised in which no scaffolding under the bridge floor is involved. In the case of figure 4, the main suspension cables can be temporarily fastened in C and D to two large concrete slabs which are cast simply on the ground on either side of the river and which resist sliding by friction developed by the weight of the earthen embankments placed on top of the slabs. Precast portions of the bridge floor are then suspended from the cables (the corresponding deflection of the cables is omitted from the sketch).

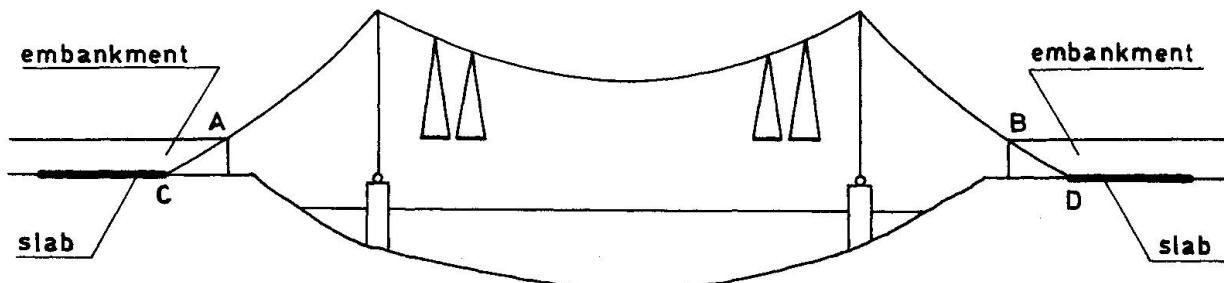


Fig. 4 - Construction procedure

When all the segments of the floor structure are erected, they are made continuous, the tensile forces in the cables are transferred to the suspended deck at A and B, and the parts AC and BD of the cables are deactivated.

9. RELIANCE ON CONCRETE TENSILE STRENGTH

Code writers, teachers and designers have often felt uneasy when confronted with and reflecting upon the inconsistency between the stated general design assumption that concrete does not resist tensile stresses and the realization that, in fact, we often rely and are forced to rely upon concrete tensile strength, mostly well-nigh unwittingly, but quite systematically when shear in slabs

comes up. In recent years admirable efforts have been made towards a better understanding of tensile behaviour, tensile strength, strain softening and fracture energy of concrete [17] [18]. It is to be hoped that these efforts will generally improve the coherence of structural concrete strength theories and, more specifically, that they will lead eventually to a scientifically based treatment of punching shear, a problem that has never been solved in a really satisfactory manner.

The effects of such a development could be far-reaching. To mention just two examples :

- It is conceivable that better insight into the reliability of concrete tensile strength may enable the rule "Linear members of minor importance such as lintels . . . , may be designed without web reinforcement" [2, p. 6-16] to be made applicable to more important linear members.
- Designers of flat slabs, in order to take care of punching shear around columns, often resort to more or less elaborate devices obtained by welding rolled steel sections and embedded in the concrete slab. A more coherent and efficient use of concrete tensile strength, resulting in a more constant safety factor, may diminish the need of sources of strength other than the own strength of the concrete and of rebars.

10. CONCLUSIONS

A comprehensive consistent approach to structural concrete design and analysis, valid without discrepancy for the whole spectrum between unreinforced concrete and reinforced and/or prestressed concrete with a wide range of degrees of prestress, is a boon to designers. It eliminates ambiguities and confusion. It enables engineers to find reliable solutions for unusual problems, such as those arising in connection with innovative structural concepts.

It is salutary to question the rationality of received rules and ideas from time to time, for example those regarding the minimum reinforcement in concrete walls and columns.

Engineers who have the creativity and daring needed to go off the beaten track should not be obstructed by dogmatic prohibitions. Codes should not ban any structural concepts or arrangements, unless there is a good reason, based on more than preconceived ideas, to cast serious doubts on their soundness.



REFERENCES

1. CEB-FIP Model Code 1990. First Draft. Chapters 1-5. Bulletin d'Information CEB N° 195, Mars 1990.
2. CEB-FIP Model Code 1990. First Draft. Chapters 6-14. Bulletin d'Information CEB N° 196, Mars 1990.
3. Code Modèle CEB-FIP pour les structures en béton. Bulletin d'Information CEB N° 124/125-F, Avril 1978.
4. Guide for Specifying, Mixing, Placing and Finishing Steel Fiber Reinforced Concrete. ACI Committee 544. ACI Journal, March-April 1984, p. 140.
5. RIESSAUW, F.G. and TAERWE, L., Loading and material tests on two 30 years old prestressed concrete beams from the De Smetstraat bridge in Ghent (in Dutch). Journal of the Belgian Ministry of Public Works, 1980, N° 1, p. 1-20.
6. MAGNEL, G., Prestressed Concrete. Concrete Publications Limited, London, 1948.
7. STORRER, E., The bridge across the Meuse river at Sclayn (in French). Journal of the Belgian Ministry of Public Works, 1950, N° 2 and N° 4.
8. VANDEPITTE, D., Two prestressed concrete bridges across the Ring Canal surrounding Ghent (in French). Précontrainte-Prestressing, 1954, N° 1, p. 1-11.
9. LEONHARDT, F., Prestressed concrete in practice (in German). Wilhelm Ernst & Sohn, Berlin, 1962.
10. BRUGGELING, A.S.G., An engineering model of structural concrete. Introductory report, IABSE Colloquium Structural Concrete, Stuttgart, 1991.
11. SCHLAICH, J., Modelling. Invited lecture, IABSE Colloquium Structural Concrete, Stuttgart, 1991.
12. MACGREGOR, J.G., Dimensioning and detailing. Invited lecture, IABSE Colloquium Structural Concrete, Stuttgart, 1991.
13. STRAUB, H., The history of the art of the civil engineer (in German). Birkhäuser Verlag, Basel und Stuttgart, 1964.
14. VANDEPITTE, D., Recent prestressed concrete construction in Belgium. Prestressed Concrete Development Group, London, 1956.
15. VANDEPITTE, D., Analysis of prestressed concrete suspension bridges and estimation of the loss of prestressing force with time (in Dutch). De Ingenieur, 1966, p. Bt. 11-16 and p. Bt. 23-29.
16. VANDEPITTE, D., Self-anchored prestressed concrete suspension bridges with parabolic cables. Symposium on Suspension Bridges, Lisbon, November 1966, p. 38.1-38.8.
17. HILLERBORG, A., Reliance upon concrete tensile strength. Invited lecture, IABSE Colloquium Structural Concrete, Stuttgart, 1991.
18. KONIG, G. and DUDA, H., Basic concept for using concrete tensile strength. Invited lecture, IABSE Colloquium Structural Concrete, Stuttgart, 1991.