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Autor: Lebet, Jean-Paul
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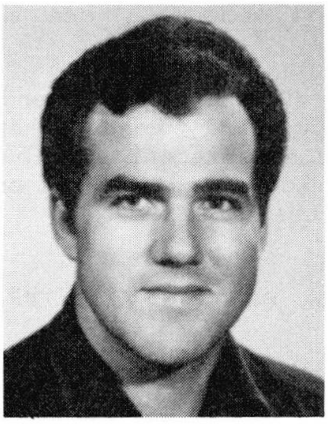
Composite Bridges

Ponts mixtes

Verbundbrücken

Jean-Paul LEBET
Dr. Eng.
Swiss Fed. Inst. Technology
Lausanne, Switzerland



Jean-Paul Lebet, born in 1950, gained his diploma in Civil Engineering at the Swiss Federal Institute of Technology in Lausanne. Working at the same place within ICOM (Steel Structures), he then studied for his Ph. D. He has been working in the field of research and testing of steel-concrete composite structures for 15 years, specialising in bridges.

SUMMARY

An overview of the most important types of composite bridges and their evolution is presented. Common construction procedures for the steel structure and concrete decks are discussed. Design requirements are reviewed. The behaviour of composite bridges and analysis of stresses under service conditions and design considerations affecting safety at the ultimate limit state are presented. Finally questions concerning durability and the use of pre-stressing are addressed. The opinions of the author reflect experiences and economic policies in Switzerland. These may differ from those in other countries.

RÉSUMÉ

La présente contribution donne une vue d'ensemble des types de ponts mixtes les plus importants, ainsi que de leur évolution dans le temps. Elle fournit également une discussion sur les procédés habituels de construction des structures métalliques et des tabliers en béton. Elle passe ensuite en revue les prescriptions relatives au dimensionnement. Elle présente encore le comportement des ponts mixtes et les calculs de contraintes dans les conditions de service, ainsi que les considérations de calcul ayant un effet sur la sécurité à l'état limite ultime. Il y est finalement question de la durabilité de tels ouvrages et de l'utilisation de la précontrainte. L'auteur donne son point de vue sur les expériences réalisées et les principes économiques admis en Suisse, qui peuvent être différents de ceux appliqués dans d'autres pays.

ZUSAMMENFASSUNG

Der Aufsatz gibt einen Überblick über die wichtigsten Typen von Verbundbrücken und ihre Entwicklung sowie über übliche Bauverfahren für die Stahlkonstruktion und die Betonplatte. Auf eine Diskussion der Bemessungskriterien folgen eine Analyse des Verhaltens von Verbundbrücken, der Beanspruchung unter Gebrauchslasten und der Einflüsse konstruktiver Details auf die Tragfähigkeit. Zum Schluss werden Fragen der Dauerhaftigkeit und Vorspannung angesprochen.

1. INTRODUCTION

In chapter 1 an overview of the most important types of composite bridges and their evolution is presented. Common construction procedures for the steel structure and concrete decks are discussed in chapter 2. Chapter 3 is devoted to design requirements. The behaviour of composite bridges and analysis of stresses under service conditions and design considerations affecting safety at the ultimate limit state are presented. In chapter 4, questions concerning durability and the use of pre-stressing are addressed.

It should be noted that the opinions of the author reflect experiences and economic policies in Switzerland. These may differ from those in other countries.

Eurocode 4, Part 1; the new European Recommendations entitled "Composite Steel-Concrete Structures", is currently in preparation [1]. This document, however, concerns the use of composite structures in buildings only. A second EC 4, part 2, concerning composite bridges has not yet been written. As a result, limited references can be made to this document.

2. TYPES OF COMPOSITE BRIDGES AND THEIR EVOLUTION

The majority of composite bridges built during the last 25 years have continuous spans. It is common to find bridges with ten or more continuous spans and with expansion joints at between 500 and 900 metre spacings. Typical span lengths vary between 30 and 90 metres. Spans in excess of 90 metres have also been built. The advantages of providing continuity between spans are not only economic. Continuity offers improvements in structural durability and user comfort due to having fewer joints and reduced deflections.

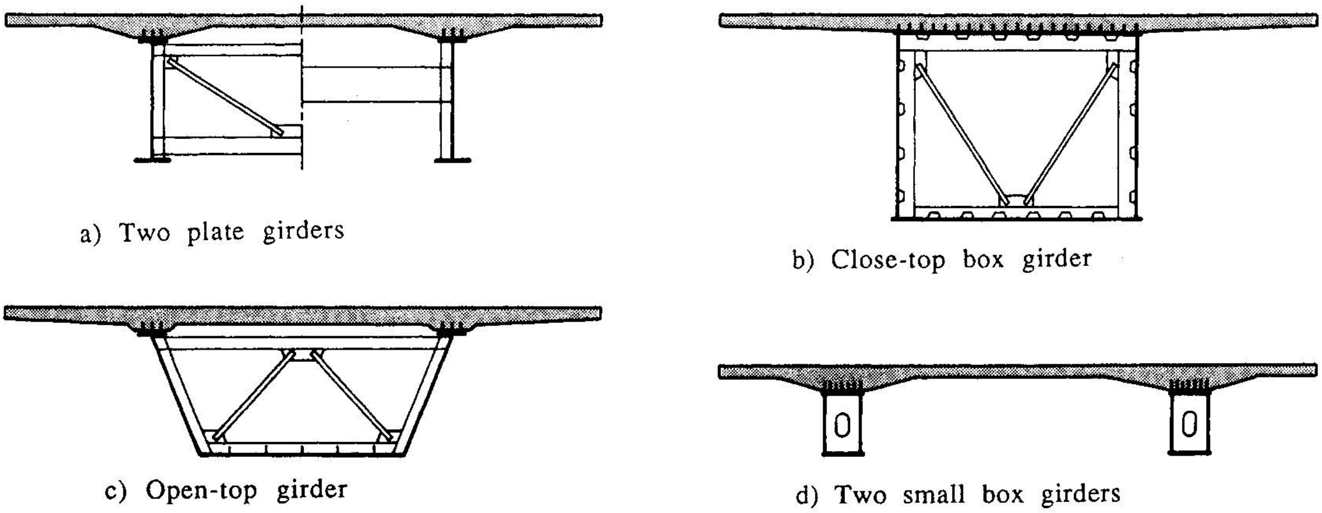
Two classical cross-sectional configurations are widely used. These are : a reinforced concrete slab with two or more plate girders, and a reinforced concrete slab with single or multiple box girders.

Plate girders (fig. 1a)

Plate girders are expensive when compared to rolled I-sections because of fabrication costs. This cost, however, is offset by their more efficient use of material. For example, flange and web size may be varied along the span and over supports to adapt to the bending moment and shear requirements. A very popular configuration, particularly in Switzerland, uses two plate girders to support a slab approximately 13 metres wide. Intermediate cross-members are usually provided, generally spaced at about 8.0 metres. The depth to span ratio of such bridges is usually between 1/20 and 1/25.

Box girders (fig. 1a, 1b and 1c)

Due to their high torsional rigidity, box girders are used for large or curved spans. Two types of box girders are used: open-topped and closed. The upper flange of closed box girders may be used as permanent formwork for the slab. A valuable quantity of steel, however, is wasted in regions of zero bending moment. Shear connectors are typically placed across the entire width of the top flange. Open-topped box girders are more commonly used. Fewer shear connectors are needed, placed longitudinally on each top flange. For open-topped box girders, particular attention during erection is necessary. This is due to the inadequate lateral restraint of the top flanges prior to the slab being placed and connected. A trapezoidal box is often used to provide widely spaced supports for the slab and to make the bottom flange narrow and thick near internal supports to maximize the effective section. For bridges with large slab widths two small boxes are often used.



a) Two plate girders
b) Close-top box girder
c) Open-top girder
d) Two small box girders

FIGURE 1 Classical composite bridge cross-sections

The steel to concrete connection is generally provided by headed stud connectors. Such connectors are provided in both the positive and negative moment regions to ensure composite action along the entire length of the bridge. Minimum slab thickness is generally 250 mm between girders and 400 to 500 mm near girders. The thickness of the slab is usually increased to improve reinforcement cover.

For composite bridges Fe E 355 grade steel is normally used. For long spans, St 420 (f_y = 420 N/mm2) is often specified near intermediate supports. Where possible, weathering steel can be used to good effect. Weathering steel is not suitable in marine or coastal areas, industrial areas with high concentrations of chemical fumes, or where high humidity is a problem. Construction details must also be carefully designed to avoid water and dust concentrations.

New code provisions now consider the post-buckling resistance of plate girder webs. The ability of tension field action to enhance the capacity of slender webs with vertical stiffeners is recognized. Web capacity is a function of panel aspect ratio, the flexural capacity of adjacent members and web slenderness. When tension field action is used in vertically stiffened webs, the vertical stiffeners which act as compressive posts in the simulated truss action must be designed specifically for the ensuing compression. This requirement has considerably influenced the design of intermediate stiffeners. Whereas stiffeners were previously designed to a simple limiting stiffness criteria, they are now required to be designed to have adequate stiffness and strength. With new code provisions, the need for longitudinal stiffeners can often be reduced, and we prefer now a thicker web with fewer vertical stiffeners. The steelwork is thus simplified, therefore reducing fabrication costs. The results of recent developments in fatigue research, now incorporated into design recommendations, have also resulted in further cross-sectional modifications. Fig. 2 illustrates the evolution of composite highway bridge cross-sectional configurations during the last twenty years.

New types of composite bridges have recently been built. These include composite cable-stayed bridges, whose economic advantages can be of benefit in certain conditions, and composite truss bridges. Several composite truss bridges have recently been completed in Germany for the high speed railway,

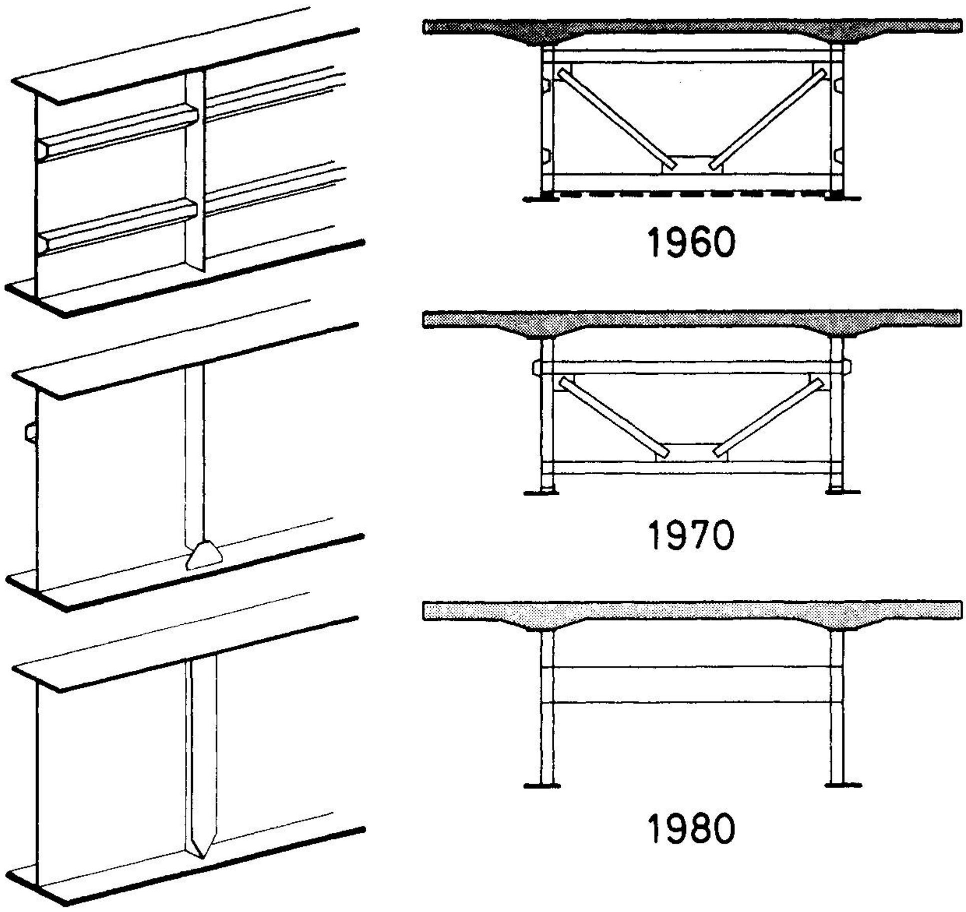


FIGURE 2 Evolution of composite bridge cross-sections

fig. 3. High serviceability requirements had to be satisfied to ensure passenger comfort. Another recent development is an innovative composite bridge with corrugated sheet steel webs, the "Viaduc de Maupré" in France, fig. 4. The web thickness is 8 mm for a height of 3 metres. Span length is about 53 metres. In addition, this viaduct is externally pre-stressed. The corrugated webs withstand shear forces without absorbing axial stresses due to pre-stressing, which is a very significant innovation. Significant weight savings were acheived.

In summary, the evolution of conventional composite bridges (plate and box-girders) may be stated as follows :
- The elimination of bottom flange lateral bracing.
- The replacement of "k" bracing with frames spaced further apart.
- The elimination of longitudinal stiffeners due to increased web thickness.
- Increased slab thicknesses.

In other words : the trends are towards the SIMPLIFICATION of the steel structure and construction details, and GENEROSITY in the design of individual elements (web thicknesses and slab thicknesses). Generosity and simplicity are economic necessities; labour is becoming ever more costly in relation to materials. In addition, liberal material use increases structural durability.

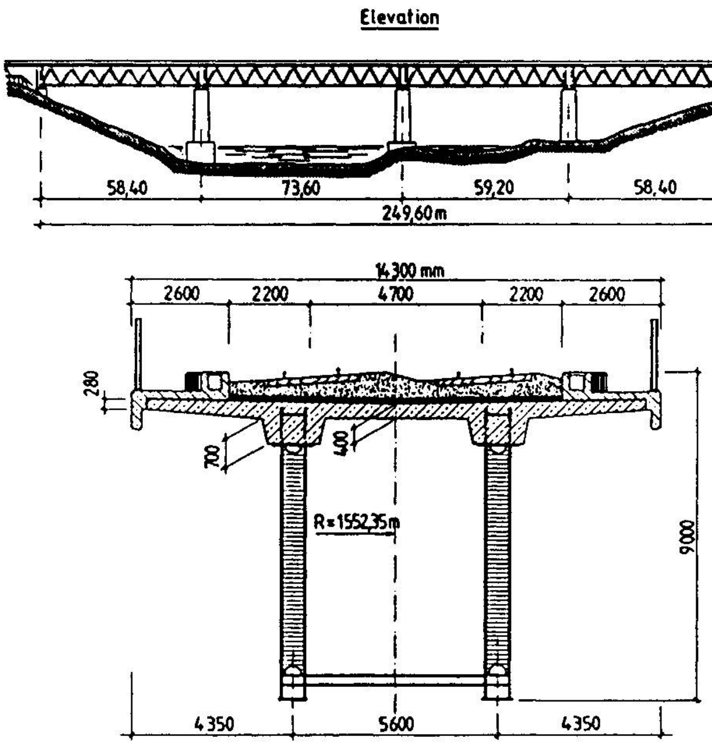


FIGURE 3 Kragenhöfer Bridge [2]

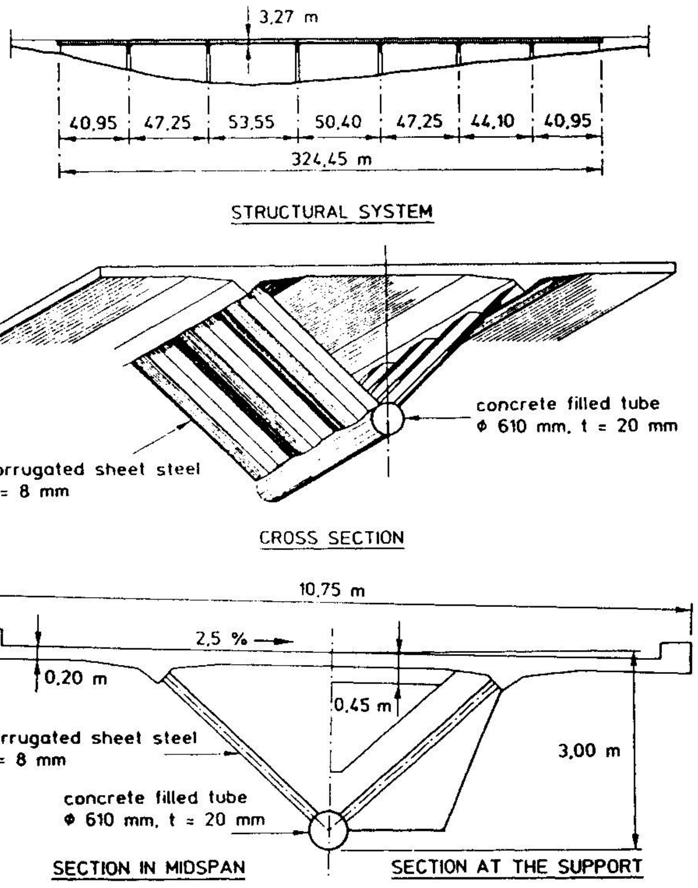
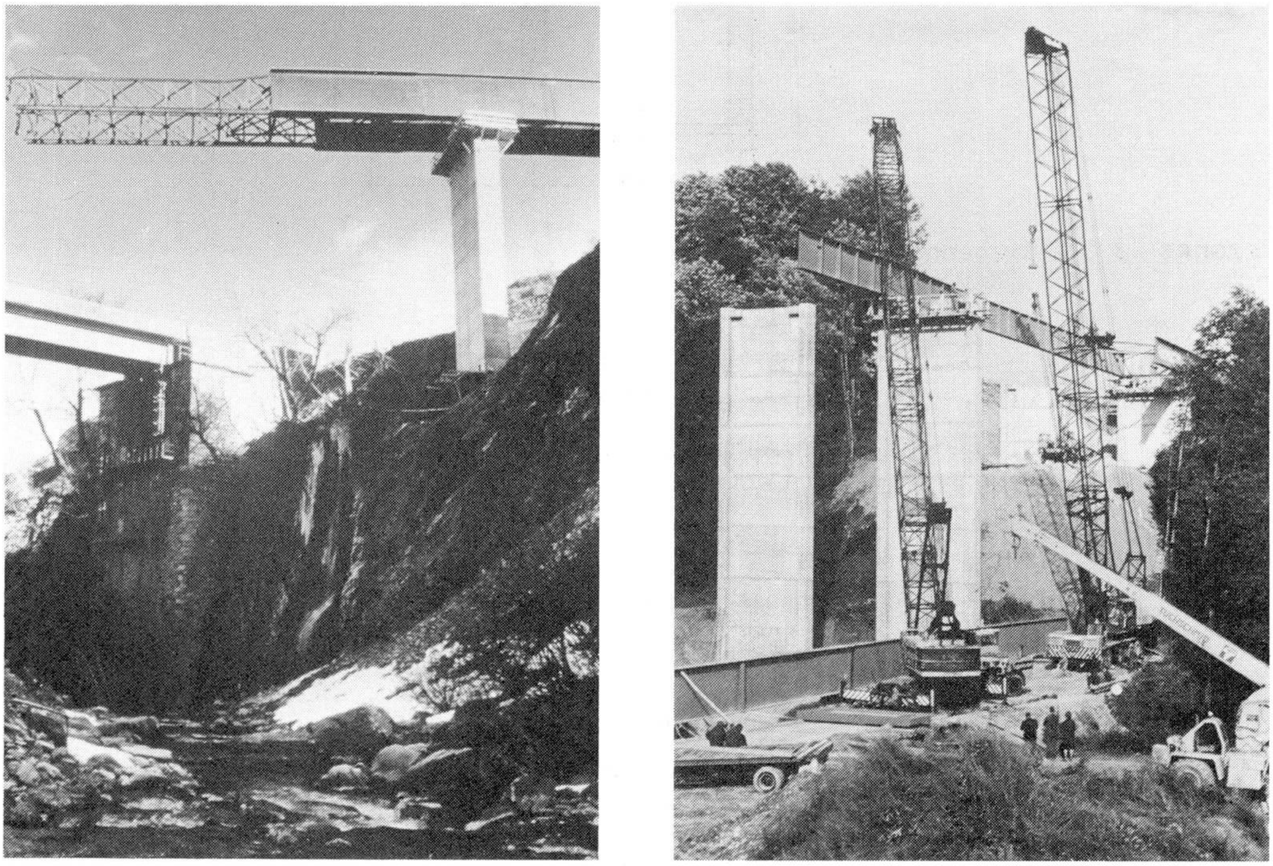


FIGURE 4 Structural system and cross section of the "Viaduc de Maupré" according to [3], [4]

3. ERECTION

3.1 Steel structure

Two erection methods are currently used : launching (fig. 5a) and lifting (fig. 5b). Lifting, using mobile cranes is, in general, the most economical method. Transportable units of plate or box girders are prefabricated and then site welded. Lifting, however, is not always possible, particularly in mountainous regions. In such cases the steelwork is assembled behind one abutment and the completed steel structure is launched. Launching bending moments are different from those at the final girder position. Bending moments during launching may often govern the design of the steel structure. To reduce these moments, a truss is added to the front of the steel structure.



a) launching b) lifting

FIGURE 5 Erection methods for the steel structure

3.2 Concrete slab

Three methods of construction are used : slip decking, precast and cast in situ. Each method is treated in detail in the following paragraphs. The steel structure, for each method of construction, is generally unpropped. The designer must ensure that lateral buckling of the upper flanges of the steel girders does not occur during the construction of the slab.

Slip decking

This method is also refered to as "sliding ribbon". It consists of repeatedly casting between 20 and 30 metres of slab on the steelwork near one end, or at the middle of the bridge. After each casting, the slab is jacked longitudinally : an adjacent slab is cast, then jacked again, and so on, fig. 6. These one week cycles continue until the entire deck is completed. Friction between the slab and the top flange of the steel girder is reduced by the method shown in fig. 7. Steel skates are placed in holes left to provide future shear connection between the slab and girder. The skates are lubricated during the jacking process with graphite. After completion of the jacking, stud shear connectors are welded in the holes, which are then filled with concrete. Consequently, shear connectors are grouped at one metre centres.

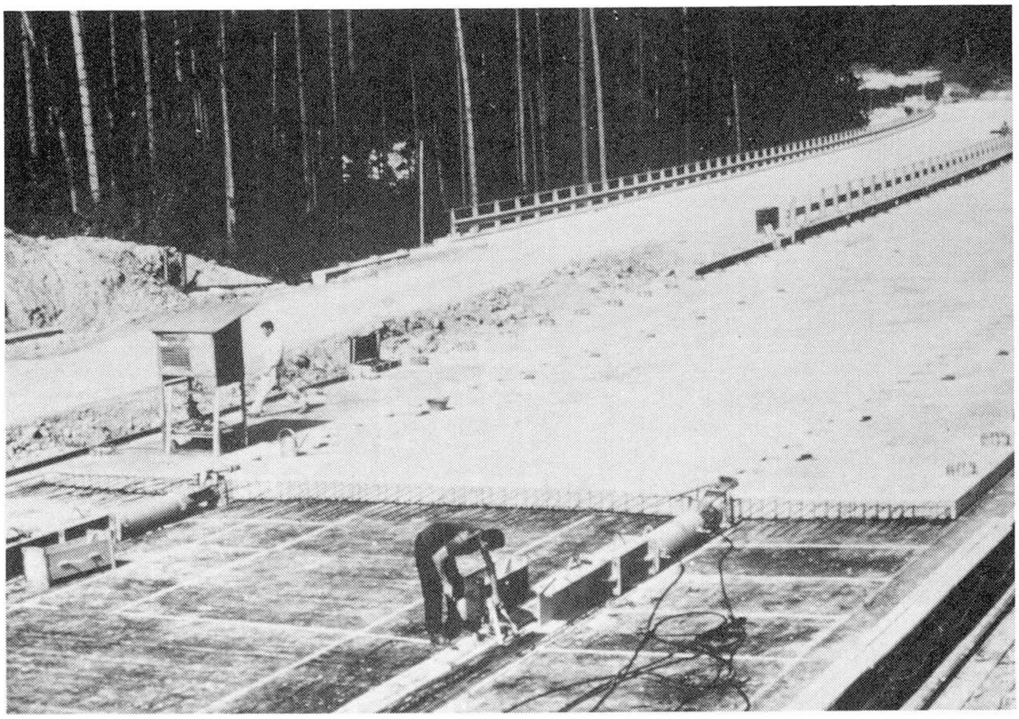


FIGURE 6 Slip decking of concrete slab

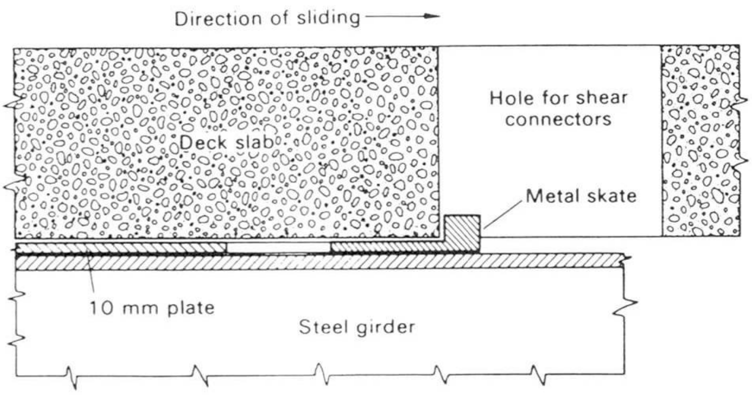


FIGURE 7 Detail of slip decking method of construction

The advantages of this method are two fold : the monolithic slab is free to shrink before being connected to the steel structure and the formwork need not be moved. This method is only suitable for straight bridges and for lengths not exceeding 400 metres. The number of connectors per group varies between 9 at mid-span and 12 or 16 near supports.

Precast slabs

This method consists of placing precast slabs, 2 to 3 metres in length, on the steel structure with a special crane, fig. 8. Groups of shear studs are already welded on the top flange, generally at one metre centres.

The advantage of this method is its speed. Disadvantages are, the numerous joints between the precast slabs, the precision with which slabs must be placed and differences in regularity between adjacent slabs. For all recent examples of this type of construction the precast slabs are pre-stressed before the slab to girder connection is provided. Pre-stressing closes the joints and this enhances the durability of the structure.

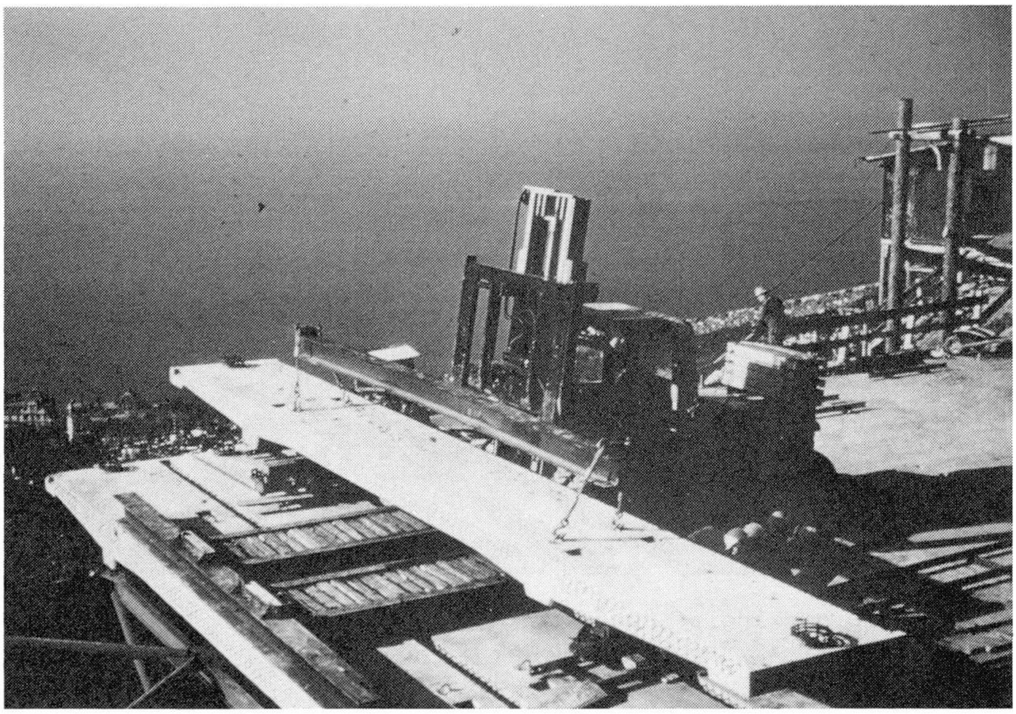


FIGURE 8 Precast slab construction

Cast in situ

This method consists of casting 30 to 40 metre lengths of slab on mobile formwork, supported by the already cast slab and by the steel structure, fig. 9. The formwork between the plate or box girders is displaced by sliding over the cross-bracings, if present. The slab can be transversely pre-stressed. Stud shear connectors are welded before casting so that composite action is immediate once the concrete hardens. As a consequence, the next casting induces stresses in the concrete of the previous step. This is a disadvantage because transverse cracks appear early in the slab. Additionally, crack size may be enlarged due to shrinkage. To enhance slab quality, stud connectors are welded in groups before casting and holes are left around them during casting. Part of the shrinkage may now occur without loading the rest of the structure. Longitudinal pre-stressing of the slab is also possible without introducing additional stresses in the steel structure. The holes in the slab around groups of connectors are then filled with concrete as late as possible.

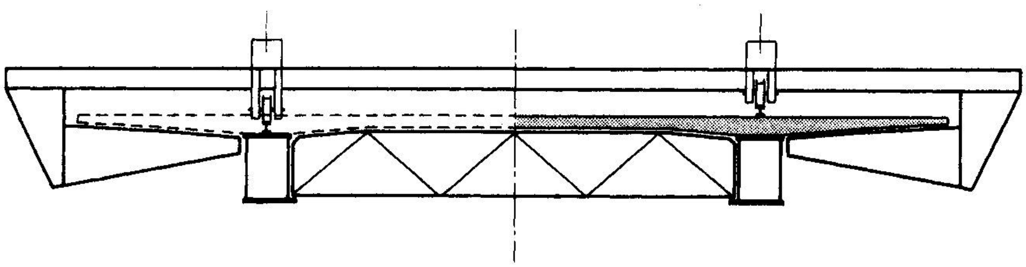


FIGURE 9 Concrete slab cast in situ on a mobile formwork

4. DESIGN REQUIREMENTS FOR COMPOSITE BRIDGES

The design of composite bridges must meet two requirements :
- Verification of ultimate limit states.
- Verification of serviceability limit states.

Actually, we probably pay too much attention to designing for the ultimate limit states and not enough to the serviceability limit states such as : corrosion, concrete carbonisation and exposure to chlorides. The reason for this is that service goals are often not well defined. For composite bridges, these goals are the following :
- The function of the structure in relation to the comfort of the users (vibrations).
- The durability of the structure in relation to corrosion, de-icing salts and chemical actions.
- The appearance of the structure, where the presence of water, cracking or deflections may, for example, be deemed unsatisfactory.

Adequate serviceability requirements may be influenced by economic considerations related to the cost of construction as well as the cost of inspection and maintenance. Serviceability is assured if the structure behaves within agreed or previously defined limits. These limits relate to :
- Cracking,
- Deformation,
- Vibration,
- The quality of construction materials.

Measures which ensure the serviceabiltiy of the structure are not always verifications by calculation. The choice of suitable construction materials, careful detailing of the design and good workmanship during construction considerably enhance structural serviceability.

4.1 Serviceability limit states

Concrete cracking

By itself the cracking of the slab is not a problem. The problem is that water can penetrate into cracks allowing de-icing agents to accelerate the corrosion process. It is generally agreed that wide cracks must be avoided because they may contribute to rupturing of waterproof membranes and thus accelerate the corrosion of reinforcement or pre-stressing tendons. Such corrosion damage is difficult to detect and repair. To limit crack width, the most widely used solution is closely spaced longitudinal reinforcement with an appropriate sequence of slab construction in the negative moment regions. In Switzerland a minimum reinforcement area of 0,8% at mid-span and 1% near supports is recommended (of gross cross sectional area of the slab).

Another method to reduce transverse cracking is to longitudinally pre-stress the slab. This induces compressive stresses in the slab under service conditions. A minimum area of longitudinal reinforcement is also recommended. This solution is being used more and more in Switzerland. Later, different longitudinal pre-stressing methods will be discussed.

To pre-stress the slab to prevent cracking under service conditions, the required compressive stress must be determined. In figs. 10 and 11, an analysis of the stresses in the slabs of continuous composite bridges with two plate girders under service conditions is illustrated. The support and midspan stresses in the concrete slab at the lower fibre, $\sigma_{b,inf}$, and at the upper fibre, $\sigma_{b,sup}$, are indicated for spans of 30, 60 and 90 metres. The slab is 13 metres wide. The following actions are considered :

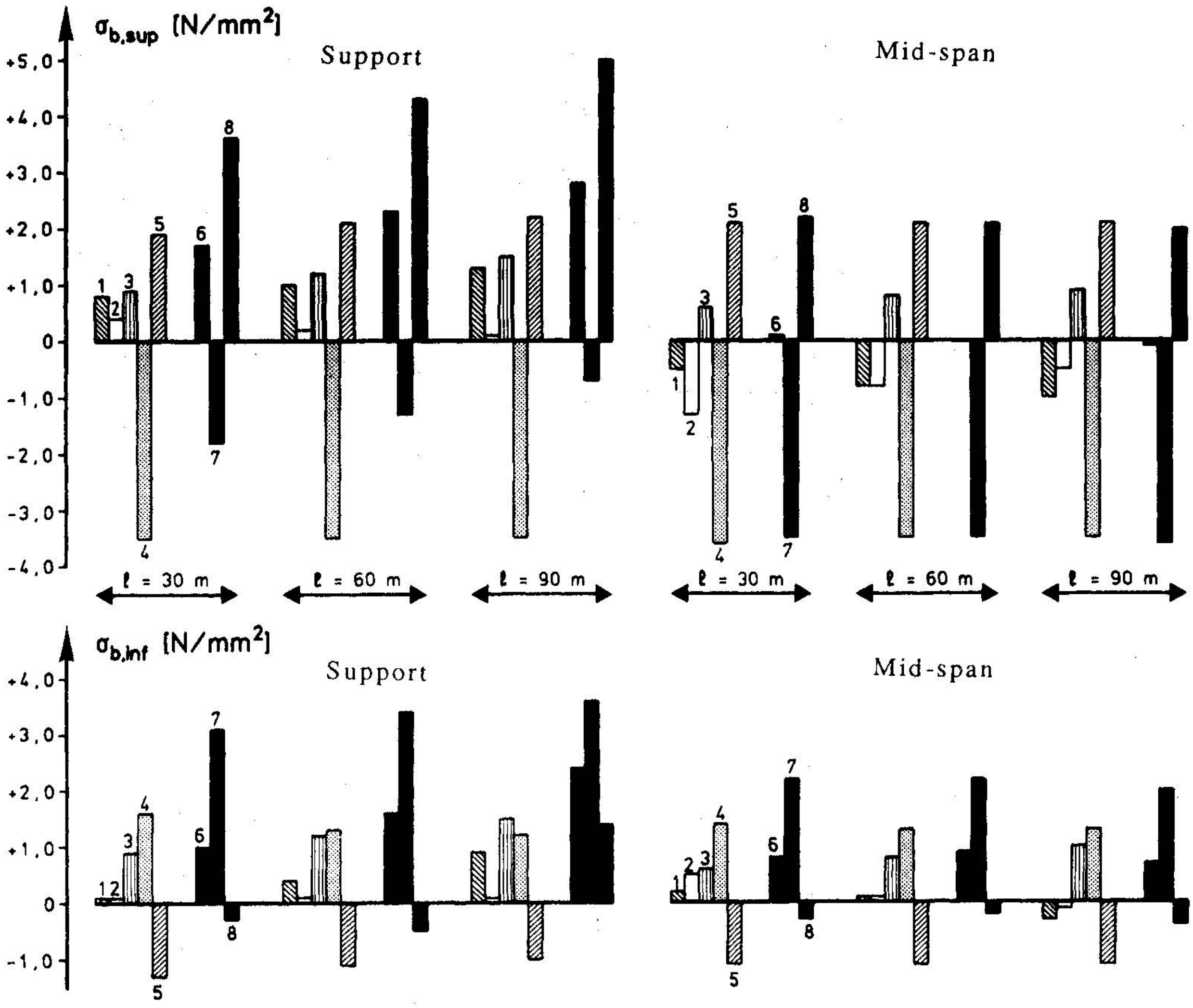


FIGURE 10 Stresses in the concrete slab under service conditions for different span lengths, l

1. The dead weight of the non-structural elements acting after the composite action is present.
2. A truck weighing 250 kN.
3. Concrete shrinkage with $\varepsilon_{cs} = 0{,}15$ ‰. This is a measured value, for a reinforced slab kept under the same atmospheric conditions as the bridge.
4. Positive temperature gradient, fig. 11; such gradients have been frequently measured. The gradient shown represents average daily temperature variations.
5. Negative temperature gradient fig. 11; the gradient shown represents reasonable values that can be expected during winter.
6. Sum of 1 + 3
7. Sum of 1 + 3 + 4
8. Sum of 1 + 3 + 5

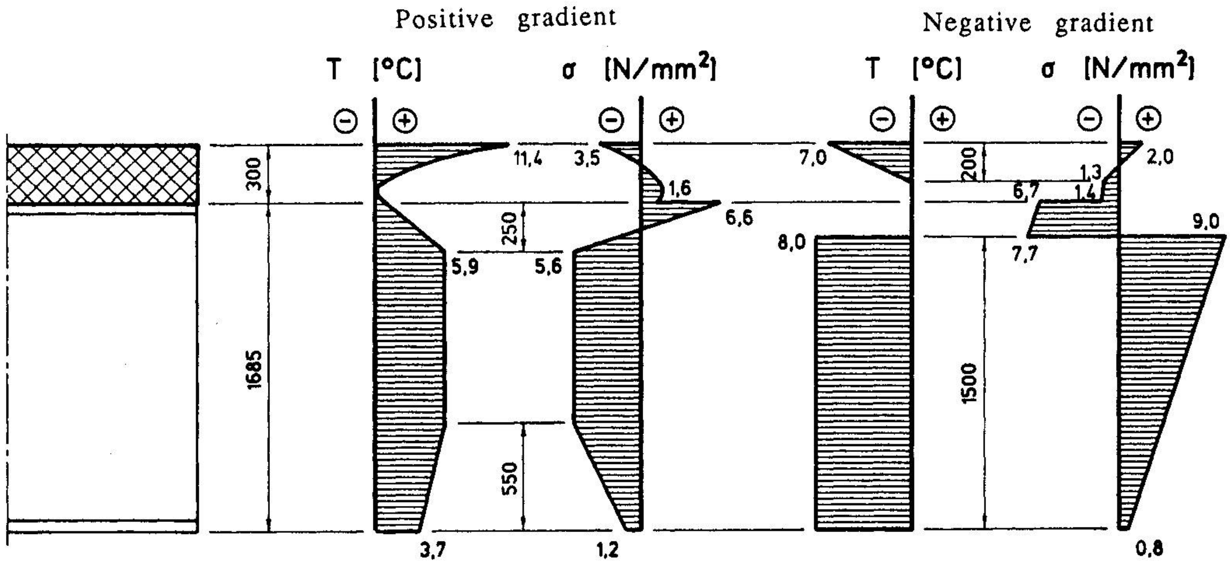


FIGURE 11 Residual temperature stresses for positive and negative gradients of temperature

Residual slab stresses corresponding to the above positive and negative temperature gradients are also indicated in fig. 11. Note that slab stresses are nearly constant along the span. Under positive temperature gradients, the lower part of the slab is in tension. Under negative temperature gradients, the upper part of the slab is in tension. Using these results and those shown in fig. 10, the following may be concluded :

- The largest positive and negative stresses in the slab are caused by temperature gradients. These stresses are about the same for each of the three spans considered.
- Under the combined effect of dead load and shrinkage, the slab is always in tension at the mid-span and over the supports; these effects can be considered as permanent.
- The worst combination of these permanent loads plus temperature gradient creates tensile stresses in the lower fibre of the slab of 3 N/mm^2 over the supports and 2 N/mm^2 at mid-span. In the upper fibre, tensile stresses of 5 N/mm^2 over the supports and 2 N/mm^2 at mid-span are noted.
- Truck loading has small effects in comparison with other loading cases.

Typical concrete tensile strength is about 2 N/mm2. This indicates that cracking may occur both near supports as well as at mid-span. Considering these results, a minimum compressive stress of 3 N/mm2, due to pre-stressing, should be introduced along the entire length of the bridge. This would be adequate to ensure that the slab is almost always in compression under service conditions.

Deformations

Deformations under service conditions must be checked for composite bridges. In the Swiss code "Actions on Structures" a maximum deflection of L/600, where L is the span length, is suggested to provide adequate rigidity and to maximise user comfort [5].

Normally, a camber is provided to compensate for deflection arising from self weight of the structure and other permanent loads. A compensation for the deflection arising from a portion of traffic loading may also be considered. This must be considered carefully, if too large, the appearance of the bridge may be unsatisfactory due to the permanent camber.

Longitudinal deformations of the bridge must be accounted for when designing expansion joints : normally located at both ends of the bridge. Longitudinal deformations must be calculated considering shrinkage, pre-stressing, long term deformations and temperature variations. In Switzerland, temperature variations for the design of expansion joints is considered to be +/- 37.5°C.

4.2 Ultimate limit state considerations

Generally, the plate or box girders of composite bridges are class 4 cross-sections, according to EC 4, part 1 [1]. This implies that elastic global analyses may be used taking into account local buckling by determining the effective cross-section.

Actions that must usually be considered are :
- Dead weight. This includes the steel structure, concrete slab and non-structural elements (such as pavements, barriers, etc.),
- Traffic,
- Wind,
- Shrinkage,
- Temperature.

Elementary beam theory, taking into account the favourable effect of the slab on the transverse distribution of load, is often used to analyse two plate girder composite bridges. A grillage method is more suitable for multi-plate and box girder composite bridges.

As an example, the transverse distribution of load for a two plate girder composite bridge with a concrete parapet connected to the slab is shown in fig. 12. Both measured and calculated values are indicated. For the example cross-section, the parapet may take as much as 20% of the applied load. This reduction could be taken into account when designing the girders.

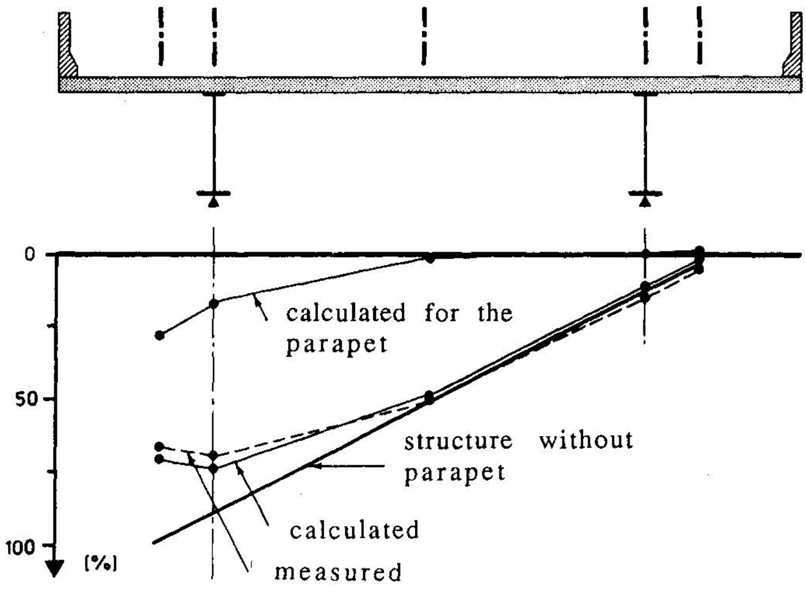


FIGURE 12 Exemple of transverse distribution of load with the influence of a parapet connected to the slab

The uncracked cross-sectional properties are used, over the entire length of the deck when carrying out the elastic analysis of continuous composite bridges. Slip between steel and concrete is neglected. Research has verified this assumption, even with groups of connectors at one metre centres [6]. The influence of concrete cracking near internal supports is taken into account by allowing for the redistribution of the bending moment. The maximum redistribution from the supports to adjacent mid-span regions is limited to 10%.

Stresses are calculated using elastic theory and an effective cross-section taking into account :
- The effective width of the concrete flange,
- A cracked composite section in negative moment regions neglecting concrete in tension but including reinforcement.

The elastic section properties of composite girders may be expressed as those of an equivalent steel section by dividing the contribution of the concrete components by a modular ratio, $n = E_S/E_C$. Two different modular ratios, due to concrete properties, should be used to distinguish between short term and long term effects. Shrinkage effects, with an allowance for creep, may be accounted for by defining a further modular ratio. In general, these modular ratios may be taken as :
- 6, for short term effects,
- 12, for shrinkage modified by creep,
- 18, for long term effects.

Composite properties should be calculated for each loading condition at each cross-section. Adequate resistance is assumed if the sums of the stresses acting on the different cross-sections do not exceed the material resistances. This control is schematically presented in fig. 13 for two cases : propped and unpropped during concreting.

Stresses in the reinforcement near internal supports may be 50% higher than those calculated using the preceding method. This is due to the stiffening effect of the cracked concrete.

The required shear connection is determined using elastic theory and the VAy/I formula. As a conservative approach, the slab is considered to be uncracked in negative moment regions. Connectors are placed in such a way as to cover the design envelope of vertical shear, which is proportional to longitudinal shear. The shear connection must also meet fatigue requirements.

For slender members (members for which buckling stress is well below yield stress), the first and secondary order effects due to temperature and shrinkage must be considered. The calculations for continuous members are carried out assuming the slab to be uncracked and unreinforced : it is impractical to consider in detail the interaction between the concrete cracking and internal forces due to temperature and shrinkage. The design longitudinal force in the slab is transferred across the interface between slab and girder at the ends of a continuous length of deck using supplementary connectors. This force is introduced into the steel structure over a transfer length which is assumed equal to about half the effective width of the concrete slab. The force due to the primary effects of shrinkage can be neglected for these connectors because it acts in the opposite direction to all other loads.

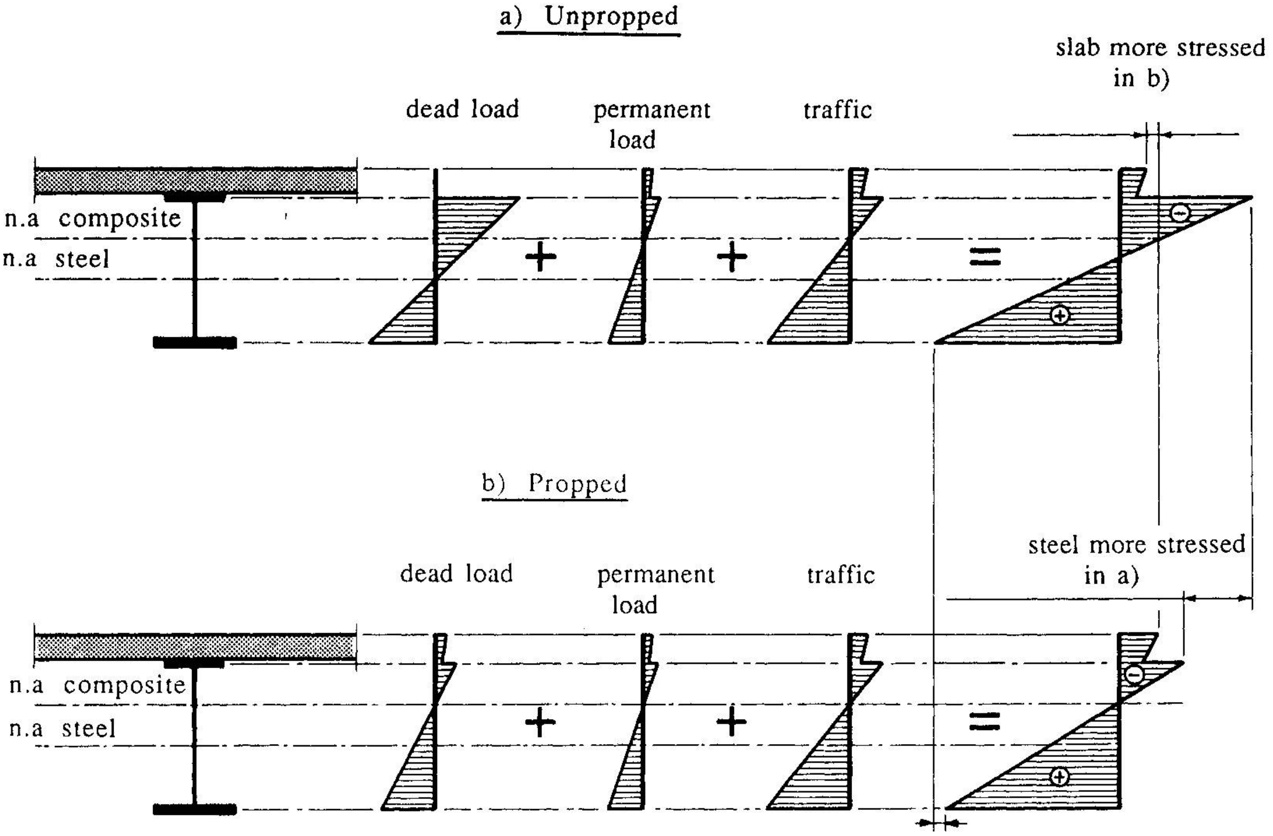


FIGURE 13 Summation of stresses for unpropped and propped composite bridges

5. USE OF PRE-STRESSING IN COMPOSITE BRIDGES

The question of whether or not to use pre-stressing in composite bridges is not yet definitively answered. Above all else pre-stressing is used to increase structural durability by preventing cracking of the concrete slab under service conditions. Other measures can also be used to increase slab durability :

- Careful choice of the concrete composition.
- Careful placement of the concrete with protective measures to ensure adequate humitidy during curing.
- Choice of an appropriate construction sequence for the slab.
- Use of closely spaced longitudinal reinforcement to control crack widths.
- Providing adequate cover to reinforcement
- Careful installation of a waterproof membrane.
- Adequate control and maintenance of the structure.

Durability is a long-term problem and experiences to date have shown that deck deterioration occurs more rapidly than expected. It is not yet clear which combination of the above mentioned measures with pre-stressing will provide the best durability improvement. Pre-stressing is expensive, but it is hoped that the initial cost of the structure can be justified by decreased maintainance costs due to improved durability.

Transversely, the use of pre-stressing depends on the spacing of supports for the slab and the length of the slab cantilever. Usually with two plate girders and a cantilever length of about 4 metres or more, transverse pre-stressing is used also for the design resistance.

Longitudinally, due to the significant axial and flexural stiffnesses of the steel section of steel-concrete composite structures, longitudinal pre-stressing losses in the concrete slab may be high due to concrete creep and shrinkage. Economic solutions must be found to minimize these losses.

It must be emphasised, as previously indicated, that tensile stresses may exist in the slab near supports and also at midspan. Pre-stressing of the slab (a minimum of 3 to 4 N/mm^2) must be provided over the entire length of the structure. Solutions that provide pre-stressing only in negative moment regions are generally not suitable. Pre-stressing must be introduced as soon as possible after concreting. Experience indicates that transverse cracks can appear quickly.

Alternative methods of pre-stressing composite bridges are :

- Pre-stressing by jacking supports, fig. 14. In this method the steelwork is raised above its final level before the slab is cast. After curing of the slab the composite section is lowered to its final level. In continuous composite bridges with larger spans the required vertical displacement of the steel beams may be very large. With this method, pre-stressing losses in the concrete slab may be as high as 50%.

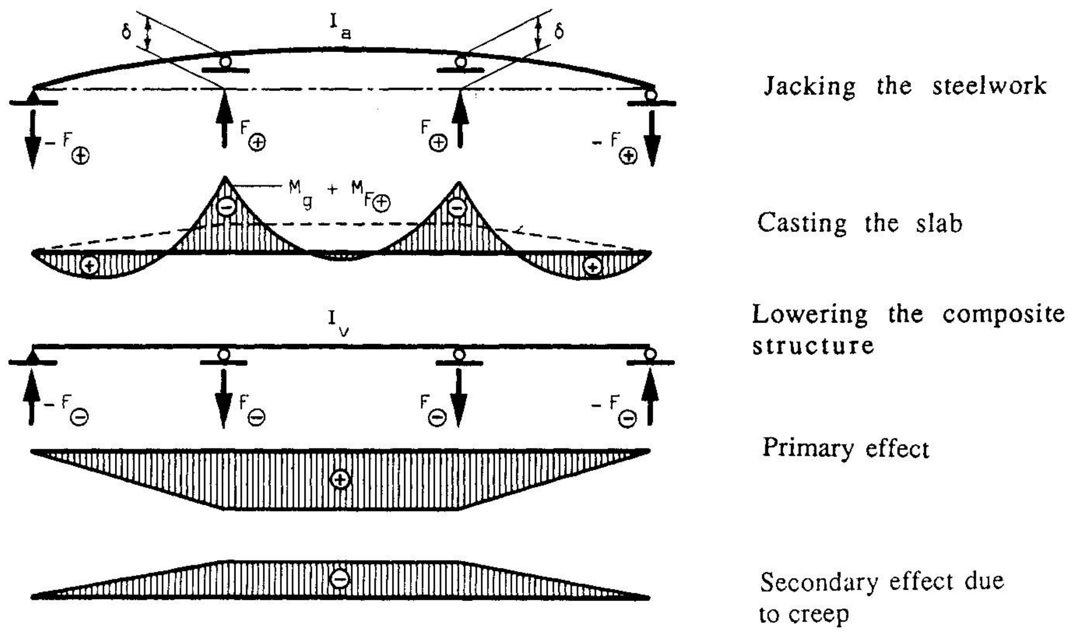


FIGURE 14 Pre-stressing by jacking supports

- Pre-stressing the slab and steel section (entire cross-section) using cables placed longitudinally in the concrete slab. This method was often used simply to pre-stress regions near internal supports. Large cracks can occur near anchorages if adequate reinforcement is not provided. As previously stated, this method does not properly increase slab durability. If the slab is pre-stressed over the entire bridge length losses may approach 50%.

- Pre-stressing the slab only, using cables placed longitudinally in the slab. In this method, the slab is pre-stressed before being connected with the steelwork. Pre-stressing forces and losses are smaller than for the entire composite section. This method is convenient when the slab is constructed by the slip decking method or with precast slabs. This method is also suitable when the slab is cast in situ if holes are left in the slab around groups of shear connectors during concreting. During pre-stressing, the slab slips on the upper flange of the steel structure. If sufficient time can be allowed before filling the holes with concrete, creating composite action, part of the shrinkage and creep is free to occur. Several Swiss composite bridges have been pre-stressed in this manner with good results.

- Pre-stressing with external cables using truss action. This method is sometimes used to strengthen existing bridges. The cables are situated between the plate girders or inside the box girders without surrounding them with concrete. Their position and their pre-stressing forces are chosen to assure that the slab is under compression for dead and permanent loads. This method is being used for a new Swiss composite bridge now under construction, fig. 15. The advantages of this system are :
  - It facilitates cable control and maintenance.
  - Cable replacement and addition of cables may be performed at later dates.
  - The loss of pre-stressing due to shrinkage and creep is minimized.
  - Cracking of the slab near cable anchorages is avoided.
  - Disadvantages are costly anchorages and cable supports.

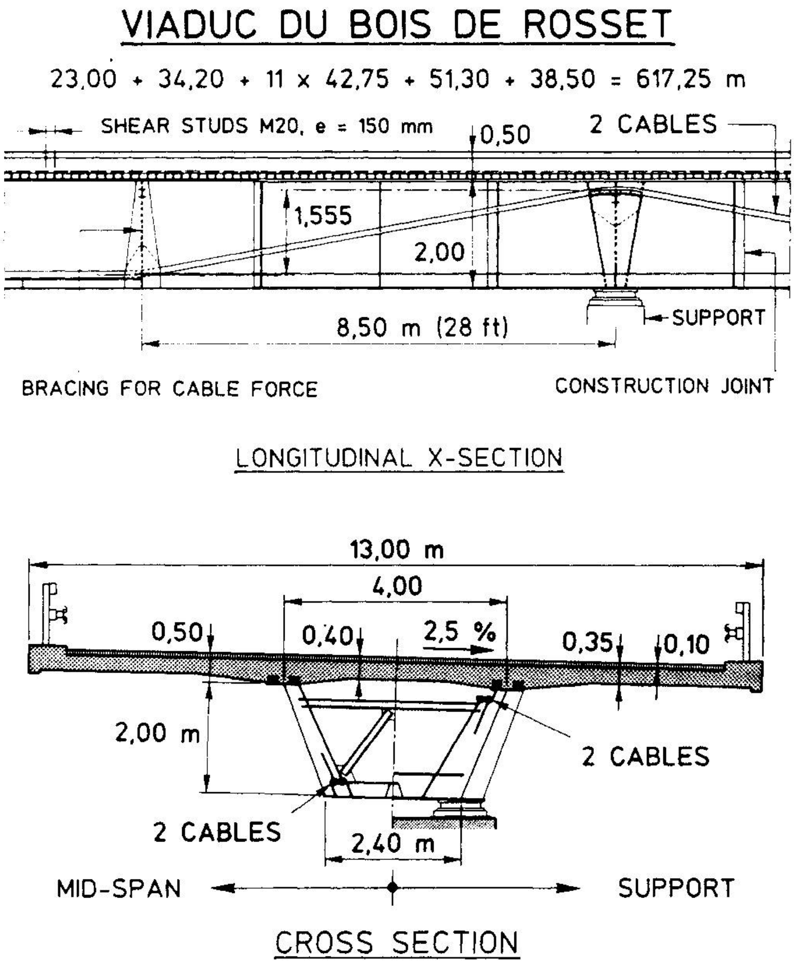


FIGURE 15 Pre-stressing with external cables

6. CONCLUSIONS

In this lecture, I have given you an overview of composite bridges with some highlights on particular aspects. What I would like to emphasise is that during design conception it is important to go for simplicity and for generosity in the dimensions of sections, and to concentrate on the durability aspects.

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