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IV

New Practices in Concrete Structures

IV a

New Trends in Design and Construction of Long Span Bridges and Viaducts (Skew, Flat Slabs, Torsion Box)

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1. Introduction

The object of this report is to review the present position of the art of designing, calculating and constructing concrete bridges and to indicate where there are problems that lend themselves to international treatment and where further research will be necessary to ensure development. In view of the vigorous developments and considerable achievements that have been accomplished in this field in various countries, the present Reporter cannot possibly be acquainted with all the important new features that have emerged. He accordingly requests forbearance and trusts that additional information will be forthcoming in the form of contributions to the discussions.

The term "long-span" used in the theme should be conceived in a *relative* sense, i.e., for a skew slab bridge a span of 20 m measured at right angles to the supports, and for a precast girder bridge a span of 40 m, will already be regarded as "long". Skew bridges are especially referred to because they are of increasingly frequent occurrence in densely built-up areas and are difficult to design and calculate.

Because of the great advantages offered by prestressed concrete, such long-span concrete bridges are nowadays no longer constructed of reinforced concrete, but of prestressed concrete, since this method of construction is structurally and economically advantageous and enables substantially longer spans to be built. In this connection, too, the favourable behaviour of prestressed concrete with regard to dynamic loading (cf. Chapter 14 in [1]) and its freedom from harmful cracks must be particularly emphasised. Both these properties make for good durability if the concrete is carefully made and of good quality. Accordingly, the following Report will confine itself to prestressed concrete bridges.

2. The appropriate degree of prestressing in relation to live load regulations

In the early days of prestressed concrete, "full" prestressing was invariably employed for bridges, i.e., no tensile stresses due to the bending moments were allowed in the extreme fibres, in accordance with E. Freyssinet's doctrine. For large live loads, especially with T-section girders, such "full" prestressing entailed very high initial compressive stresses in the tensile flange; these stresses acted constantly and therefore gave rise to considerable amounts of creep deformation. In many countries "full" prestressing was specified regardless of the magnitude of the specified live loads and safety factors. The regulations concerning loads and safety factors vary a great deal, however. In many countries these relate to the very heaviest special vehicular loads ("abnormal vehicles") or military vehicles with weights of 60 to 100 tons acting upon a relatively small area, whereas normal traffic, including heavy lorries (trucks), comprises loads which are merely between a quarter and a third of those very heavy loads. It so happens that prestressed concrete girders are very little affected by occasional rare overloading; even if the concrete in the tensile flange cracks, the cracks completely close up again, as a result of the compression developed by the prestressing force, immediately after the abnormal loading of short duration has ceased. Tests have always confirmed this great capacity for recovery of prestressed concrete girders after brief overloading. It is therefore not a reasonable procedure to keep the tensile flanges of girders permanently in a state of very high compression in order to obviate the occurrence of tensile stresses in the concrete in rare extreme cases of loading, the more so as one then has to put up with the above-mentioned creep deformations which alter the gradients.

It was due to the influence of U. FINSTERWALDER that so-called "limited" prestressing, in which tensile stresses of limited magnitude are permitted under full working load, was introduced into Germany at an early stage and applied in bridge construction. In the Recommendations of the FIP-CEB Joint Committee, which were issued on the occasion of the FIP Congress in Paris in 1966, three different classes with different degrees of prestressing are likewise introduced, and for bridges in Class II a limited amount of tensile strain is allowed. However, the procedure of laying down a limit for the tensile stress or tensile strain has the drawback that for rectangular or I-section girders tensile stresses are reached only at much higher percentages of the full live load than it is, for example, in the case of T-section girders. This is illogical.

The present Reporter considers that it would be more correct to let the degree of prestressing to employ depend upon the live load which can be expected to occur a million times. For bridges the prestress should be so chosen that, for this portion of the live load, no tensile stresses occur in the extreme fibre, while no limit should be laid down for the magnitude of tensile stress or tensile strain occurring under the rare abnormal loads envisaged in many regulations. This presupposes that the untensioned reinforcement is so designed that even in these extreme loading cases only invisible, finely distributed hair cracks will occur. This is easy to achieve, because under working load the tensile strain of the extreme fibres in prestressed concrete girders is, on account of the low position of the neutral axis as a result of the prestress, much smaller than in conventional reinforced concrete anyway. A further prerequisite is that the tendons and the untensioned reinforcement provided in the longitudinal direction should be so designed that the required safety against failure for the full live load is ensured. By adopting a somewhat lower permissible stress in the tendons for "limited" prestressing than for "full" prestressing it is possible to cater more readily for the needs of ultimate-load design.

Experience has furthermore shown that for short spans the live load moments which occur a million times attain a higher percentage of the calculated maximum moments than they do for long spans. Assuming that bridges have to be designed for specified vehicular loads of between 60 and 100 tons, the appropriate degree of prestressing could perhaps be so contrived that, for a bridge with a span of about 30 m, 50 to 60% of the maximum live load moment can be carried without producing flexural tensile stress, whereas for a span of 100 m this percentage could perhaps suitably be reduced to 40%. Measurements on a bridge over the Rhine, with a span of 186 m and carrying heavy traffic (trams and high proportion of goods vehicles), showed the peak values of the actual normal live load bending moments to be 14 to 16% of the calculated maximum moment (max M_p) according to the German Standard DIN 1072.

The appropriate choice of the degree of prestressing is not only of great importance to the economy of prestressed concrete bridges, but also to their behaviour under permanent load. Large creep deformations after construction of the bridge are certainly undesirable, because they alter the gradients and thereby—more particularly on short spans—impair the riding properties. Therefore with "limited" prestressing savings are effected and better behaviour under sustained load is obtained, without sacrifice of safety.

The design loads, dynamic coefficients and safety factors still exhibit considerable differences in the various countries (Figs. 1 to 3). Thus the Indian Road Congress requires loading trains of 75-ton lorries and very high dynamic coefficients, whereas the AASHO in the U.S.A. lays down a maximum vehicular load of only 32 tons. As a result of certain influences the French and German regulations introduced after the war likewise specify very high values. For a span of 40 m the French regulations give design moments three times as high as those given by the AASHO regulations. Objectively, such great differences are certainly not justified. More particularly in developing countries, which are being developed with considerable financial sacrifice, bridge construction should not be unnecessarily made more expensive by the application of extremely severe loading regulations.

The CEB and FIP are engaged in producing recommendations for uniform international design principles relating also to the degree of prestressing. Clearly, as a prerequisite for the application of such recommendations in bridge construction it will be necessary to interadjust and unify the loading regulations.

It would be desirable, at the Congress, to set up a Committee for dealing with these problems.

3. Design principles

In many countries there is a trend towards basing the design of structures solely on ultimate load methods. Experience has shown, however, that with prestressed concrete bridges it is absolutely necessary to take into account, and therefore calculate, the stresses under working load conditions and the deformations due to dead weight plus prestress. Also for the analysis of shear, torsion, transmission of force into the structure, moment redistribution due to creep, and limitation of cracking it will still long continue to be necessary to base oneself on the elastic theory, assuming uncracked homogeneous sections, the more so as these classic principles of design calculation yield results which, particularly for prestressed concrete, are quite realistic. Hitherto only the analysis for flexural load capacity, with due regard to the actual work diagrams of the materials, has been satisfactorily solved. The associated ultimate load methods for taking account of the moment redistribution in statically indeterminate systems can sometimes advantageously be used for checking the structural safety. For shear and torsion there is as yet no recognised ultimate load analysis, although the design for the internal forces must be based on the required ultimate load. Hence designers in actual practice are right in preferring to apply the elastic theory in this field, and there is still a great deal of work to be done by the researchers before the ultimate load methods can at least be extended to cover the whole range of design.

4. Skew bridges

4.1. Skew slab bridges

In skew slab bridges the principal internal forces, and therefore the design, are affected by numerous parameters. It is most essential to confine oneself to the main parameters. The magnitude and direction of the principal moments are affected by the intersection angle φ, the ratio of the "right" span l to the width b, the support conditions, and the positions of the loads. Theoretical treatment with the aid of the theory of plates is being applied with increasing success with the aid of electronic computers. This was done in an exemplary manner at the University of Bratislava by J. BALAŠ and A. HANUŠKA in 1964 [2, 3, 4]. The early work of the Danish investigator N. NIELSEN (Copenhagen, 1944) also deserves to be mentioned [5]. As far as the English-speaking part of the world is concerned, attention shall be drawn on the results reported by K. E. ROBINSON [6].

Balaš und Hanuška investigated among other things the influence of Poisson's ratio μ. They found that the maximum moments and maximum deflections augment with increasing Poisson's ratio (from $\mu=0$ up to $\mu=0.33$) in particular along the free edges. The results of model analysis investigations using models made of materials for which the ratio μ is higher than that of concrete (e.g. $\mu=0.33$) will therefore be on the safe side.

In view of the theoretical difficulties, model analysis was applied already at an early time in solving the problem. Since the positions occupied by the design vehicles are of considerable influence on the envelope of the maximum moment diagrams determining the design of sections, the structural design of skew slabs is almost invariably based on influence surfaces. For this purpose W. ANDRÄ and F. LEONHARDT have developed a method whereby the influence lines for the bending moments and bearing reactions with lateral strain taken into account can be directly drawn up [7, 8]. Further development of the method in view of using an electronic recording instrument presents no difficulties. By this means the required influence surfaces for a skew bridge can now be drawn and exploited for the specified loads in only a few days. For each point, however, it is necessary to determine three moment influence surfaces in order to be able to find the principal moments m_1 and m_2 from m_x, m_y and m_{xy}.

With the aid of model analysis some valuable numerical tables have been worked out and published. In this behalf reference shall be made of the book entitled "Schiefe Stäbe und Platten" ("Skew bars and slabs") [9] by H. HOMBERG and W. R. MARX (slabs with $b=l$ and φ between 20° to 90°) and especially of the set of tables by H. RÜSCH and A. HERGENRÖDER, "Einflussfelder der Momente schiefwinkliger Platten" ("Influence diagrams for the moments in skew slabs") [10], dealing with skew slabs with various b/l ratios and angles up to $\varphi=30°$ (Fig. 4).

The investigations carried out up to the present time have shown that the designer can usually confine himself to considering three significant points, which are the centre m of the slab, the point r on the free edge, and the obtuse corner s (Fig. 5). Figure 6 shows the principal moments and their directions as a function of the angle of intersection φ for $b=l$ and uniformly distributed load on the entire bridge surface (e.g. dead weight). It is apparent from this that the principal moment in point m in the direction of the span—i.e., m_1—

increases considerably from approximately $\varphi = 60°$ onwards with increasing acuteness of the angle, while the moment m_2 in the transverse direction decreases and even becomes negative when the intersection angle φ becomes very acute. The moment on the free edge in point r increases in the same way, but does not become considerably greater than m_1 in the centre of the slab which is of favourable effect on the necessary over-all depth of the structure. It should be borne in mind, in this regard, that the maximum positive moment on the edge will, with decreasing values of φ, shift away from the middle and move towards the obtuse corner, approximately up to the point $0.25l$, information about this is given in [3], [9] and [10].

Particular attention must be paid to the restraint moments of the unsupported edge occurring at the obtuse-angled corner, giving rise to a large negative moment which acts approximately in the direction of the support and diminishes rapidly, depending on the support conditions, while at right angles thereto a positive principal moment occurs which is due, in a sense, to the torsional resistance of the slab. The restraint of the unsupported edge at the obtuse-angled corner of course also brings about a considerable increase in the magnitude of the bearing reactions at that corner, and these, too, are dependent upon the support conditions of the slab.

How the magnitude of the principal moments for a bridge with $\varphi = 30°$ and $b = l$ is modified in relation to the corresponding values for a simple "right" bridge is exemplified in Fig. 7. In this case a span $l = 20$ m, a uniformly distributed loading $q = 3$ tons/m^2, and a concentrated live load $P = 30$ tons, was assumed.

The compilations of tables referred to above base themselves on linear pivotable bearings. Already in 1960 W. Andrä and F. Leonhardt [8] found that the maximum bearing reaction at the obtuse-angled corner, and therefore also the bending moments, can be favourably modified by using bearings consisting of points spaced fairly far apart instead of linear bearings. Fig. 8, which has been taken from their publication [8], shows that the largest influence ordinate of the bearing reaction A_1 can thus be reduced from 1.8 to 1.1 (for example). The negative moment at the obtuse-angled corner, which presents difficulties in the structural arrangements needed to cope with it, is correspondingly reduced.

The effect of the spacing of the bearings and also the effect of the yielding (flexibility) of the bearings have moreover been dealt with by A. Mehmel [11] and by Homberg and others [12] in comprehensive investigations. Clearly, the bending moment pattern of such a skew slab will be significantly modified even by quite small differences in the yielding (the amount of "give") of the bearing points.

Hence it can be recommended that skew slabs be supported on individual bearings spaced at distances of 0.15 to $0.25l$, while it is ensured by means of sufficiently rigid and firm abutments that differences in the amount of yielding of these bearings are avoided. This latter requirement must, of course, also be strictly fulfilled in model tests.

At the acute-angled corner negative bearing reactions may occur which should preferably be obviated because, moreover, a movable bearing is needed there, which makes anchorage more difficult. By suitable choice of the bearing spacing and the prestress it can be ensured that only positive reactions will occur.

Calculation of the moments and shear forces due to the prestress can as yet be done only with the aid of influence surfaces. In this procedure the "radial" forces due to curvature of the tendons are applied to the influence lines along the tendons, the horizontal component of the prestressing force being additionally considered as a concentric (axial) or eccentric longitudinal force. T. Y. LIN calls this method, which has long been applied in Europe, the "load-balancing method" [13].

The question as to the most favourable kind of prestressing for skew slabs has not yet been sufficiently elucidated. It is certain that the desired effect can be produced only with curved tendons the "radial" forces of which counteract the loads. For this purpose the central zone of the slab will preferably be prestressed with tendons of single curvature, e.g., parabolic tendons (1), whereas the edge zone should be provided with tendons (2) with curvature in alternate directions in order to cope with the restraint moments at the obtuse-angled corner (Fig. 9). These restraint moments due to dead weight are, on the other hand, reduced by the "radial" forces developed by the prestress.

In general, the tendons will be made to radiate fan-wise from the obtuse-angled corner and comprise an angle of about 70° to 80° in the central zone. To deal with the restraint moments at that corner it is necessary to provide some short powerful tendons (3) parallel to the support. Fig. 9 shows an example of such a tendon arrangement.

It is, however, desirable to seek ways and means of simplifying the prestress. For example, in a number of cases a skew slab has been prestressed by means of parabolically curved tendons disposed parallel to the unsupported edges. With such an arrangement the effect of skew can be cancelled for dead weight, if the "radial" (upward) forces developed by the curvature of the tendons are equal to the dead weight. A solution of this kind cannot be considered to constitute an optimum arrangement, however, since it does not suitably deal with the live load moments.

Although the shear stresses and principal tensile stresses in solid prestressed slabs are of very small magnitude, it is nevertheless considered necessary to install stirrups (preferably of the closed type) at the edges and supports and more particularly in the vicinity of the obtuse-angled corner, the more so as there are, adjacent to the unsupported edges, torsional moments whereby the change of direction of the principal moments is effected.

Of course, the prestress can develop its action only if the shortening of the concrete is not hindered by the mode of support of the slab, i.e., it is necessary to ensure freedom of horizontal movement not only for changes of length due to temperature and shrinkage, but also for shortening due to prestress and creep. The fixed bearing can advantageously be located at an obtuse-angled corner. Roller bearings are not suitable as movable bearings because they cannot be set at right angles to the tangent of the plane of bending. For this reason it is preferable to employ rubber bearings, especially rubber pot bearings provided with a Teflon (poly-tetrafluore ethylene) sliding layer [14] or, for structures with larger dimensions, rocker bearings capable of movement in all directions.

The interconnection of abutment walls and skew slabs to form portal-type structures gives rise to a complex pattern of bending moments and makes the structural design more difficult. Such bridges have been carefully investigated and constructed in Switzerland [15]. This form of construction has advantages only if it is essential to have an unusually small construction depth.

Generally speaking, solid construction will be adopted for skew slabs, even for fairly long spans up to about 25 m. Embedding tubes in the slab in order to reduce its weight encounters difficulties with regard to the pattern of flow of force within the structure and the tendon arrangement thereby necessitated. When skew slabs of longer span have to be constructed, it is advisable to form rectangular cavities with the aid of permanent (non-recoverable) formwork, and in that case the ribs can be made to radiate fan-wise from the obtuse-angled corner, so as to enable the prestressing tendons to be arranged to suit the force pattern.

A skew bridge of this kind, with $l\varphi = 42.6$ m, $\varphi = 44°$ and a construction depth of only 1.50 to 1.15 m was, for example, designed by F. LEONHARDT in 1955 [16] (Fig. 10).

In 1966 a very large two-span skew slab bridge, with a skew span length of 2×40 m, $\varphi = 23°$ and a support length of 99 m, was constructed, without joints, for the Midsommerkransen traffic facilities at Stockholm [22].

For the construction of multi-span skew slab bridges it is advisable to determine the bending moments and bearing reactions by means of model tests. An example is afforded by the Bleichinsel Bridge at Heilbronn, which had to be designed not only as a skew bridge, but also had to be curved and of variable width (Fig. 11) [7]. A three-span skew underpass with an intersection angle of only 19° is described in [16], page 81.

C. SCHLEICHER [17] has latterly prepared computer programmes for multi-span skew slabs of variable thickness whereby the design problems can also be solved by calculation for simple boundary conditions.

In any case it can be said that the present-day bridge design engineer can fulfil all traffic requirements with regard to skew bridges with the aid of model analysis or numerical design tables and that the favourable structural action of slabs enables very low construction depths and also a simple and pleasing aesthetic treatment to be achieved. For the sake of further development it is desirable to investigate the safety against failure of such skew slabs with the object of simplifying and probably also reducing the prestress. The yield-line theory is not suitable for this because the requisite safety against large cracks, which is necessary in bridges, does not allow of any major deviations from the moments as determined by the elastic theory.

4.2. Skew girder bridges

In the case of skew bridges with longer spans which cannot suitably be formed by solid or hollow slabs the width of the bridge is usually less than the span length, so that the effect of the skew upon the bending moment pattern is less. Experience has shown that it may then even be advantageous to eliminate the effect of skew upon the main girder system by not providing flexurally rigid cross-girders (diaphragms) at the supports but, instead, merely installing flexible transverse frames (Fig. 12) which produce no end restraint in the main girders. As a result, approximately equal moments are developed in the main girders disposed parallel to one another, and therefore these girders can all be of the same design. This is desirable in cases where the girders are to be precast. Load-distributing diaphragms can advantageously be arranged parallel to the supports, in order to interconnect points with equal deflection. Load-distributing diaphragms arranged at right angles to the main girders are efficient only if the beam grillage as a whole is narrow in relation to the span length (Fig. 13).

If the skew intersection involves building a bridge with several spans, the difficulties associated with a skew bridge can be avoided by using a torsionally rigid main girder, e. g., a box girder, which is supported only along its centre-line on individual columns, and by setting the right-angled abutments sufficiently far back (Fig. 14). This kind of bridge will be further dealt with in connection with elevated roads; it was first employed for reinforced concrete flyovers in California around 1954 (Garey Ave., Pomona, Calif., no publication available).

5. Elevated roads

For the routing of traffic through cities and built-up areas it is becoming increasingly necessary to build fairly long bridges, called elevated roads ("skyways"), the aesthetic treatment of which is of particular importance, especially in towns. Unsightly elevated roads may seriously impair the usability of their surroundings, whereas an elevated road structure of pleasing design has little effect upon the city's life, even under the bridge. A positive example in this sense is provided by the elevated road system at the Jan-Wellem-Platz at Düsseldorf (Fig. 15). Negative examples are, unfortunately, very numerous throughout the world.

On critically examining the various solutions for elevated roads and their effect upon the surroundings, it is realised that the following desirable features should be aimed at:

(1) slender structures presenting a restful appearance (i.e., free from "fussy" features) from below;

(2) slender columns, as few as possible, without clumsy transverse diaphragms at the supports;

(3) open railings which you can look through.

The cross-sections of elevated road bridges actually constructed, illustrated in Fig. 16, show what great differences may exist here. Undoubtedly the most elegant solution has been achieved in the elevated road at Düsseldorf because, as a result of the curved shape of the underside, the construction depth of the superstructure becomes, as it were, invisible. This produces an impression of extreme slenderness. The effect can be further enhanced by appropriate choice of colours, the underside being given a dark and the outer coping a light colour. The small dimensions of the forked supports are achieved by the use of high-tensile steel.

How greatly the slenderness of the supports matters in urban traffic is apparent from the elevated road in the Via Monteceneri at Milan (Fig. 16, Fig. 17 was not available), which has supports with the smallest possible dimensions attainable with concrete construction. The method of construction presupposes continuity of the structure over a number of spans.

For wider bridges it is also possible to obtain a pleasing result with two supporting columns, however, as is exemplified by the Fischerstrasse at Hanover (Fig. 18) and the approach bridge to the northern Rhine bridge at Düsseldorf (Fig. 19). In both cases the use of heavy diaphragms between the columns has been avoided. The latter are of slender design, so that the space under the bridge invites utilisation.

These aesthetic requirements have intentionally been stated first because for elevated roads, especially in cities, they are really important if the urban areas affected are to retain their amenity.

The satisfactory shape is obtained preferably by not installing the diaphragms at the supports below the main girders but more or less within the depth of the main girders (Fig. 20). In continuous structures this does not give rise to difficulties, because the design and structural arrangements for indirect support of the main girders on the diaphragms are sufficiently known, both if ordinary (untensioned) reinforcement or if prestressing is employed for establishing the connection [18]. Attention must be called to the need for providing so-called "suspension reinforcement". The diaphragms usually have a very low degree of slenderness, so that the amount of reinforcement for shear can be reduced [19, 20].

In the case of a solid or hollow slab the column will be installed directly under the slab (Fig. 21). The latter, when prestressed in both directions, has a very considerable strength with regard to punching shear, so that a moderate amount of stirrup reinforcement in the vicinity of the support will suffice. Tests for investigating the punching load of prestressed concrete slabs are desirable.

For precast single-span beams, too, a neat solution is provided by supporting the beams on the bottom flange of the crossgirder, or diaphragm (Fig. 22), as was already done in the construction of an elevated road in Moscow a good many years ago. A more recent good example is the Metro viaduct at Rotterdam [21], where the precast beams are supported on a flange on a table slab rigidly fixed to a column (Fig. 23). In this arrangement, however, firm fixity to the column is essential, and there are many joints.

In 1965, with the Hägersten viaduct at Stockholm, the Swedes showed a fine solution of the problem how, with precast beams and an in-situ concrete deck slab, a multi-span bridge can be given structural continuity without having to make the crossgirders protrude below the longitudinal girders [22] (Fig. 24).

In [23] LEONHARDT has dealt with the support problems presented by elevated roads and shown that even curved structures of this kind can be constructed as long continuous bridges which can be so contrived that they perform their changes of length only along the centre-line of the bridge. As a result it is possible even in curves to employ rocker supports with linear bearings at right angles to the centre-line of the bridge whereby the torsional moments in the main girders can be absorbed. In this way, for example, a more than 800 m long and, in part, curved elevated road bridge was constructed as a jointless continuous structure over 36 spans at Düsseldorf (Figs. 29 and 30 in [23]).

For the construction of curved elevated roads the box-section as main girder undoubtedly deserves preference because of its torsional rigidity, the trend of development being towards the choice of a single-compartment box girder with a deck slab cantilevered on both sides thereof (Fig. 25).

6. River and valley bridges

Construction methods are exercising an increasingly important influence on the design of big bridges. Both for in-situ concrete and for precast concrete construction the design engineer must have a clear conception of economical construction procedure. Conventional falsework is now only rarely employed. For long bridges veritable scaffolding and formwork "machines" have been developed, which are travelled from span to span and are usually supported on the final piers. For precast construction large steel erection girders are used. The cantilever method of construction has been further improved both for in-situ concrete construction and for construction with precast concrete members.

Keen competition particularly in this sphere makes for rapid changes and progress.

It is not the purpose of this report to describe this development, although it is of great importance to concrete bridge construction. Suffice to refer to some relevant publications [24 to 29].

An increasing number of bridges is being built high above deep or wide valleys. In some cases the road may be as much as 150 m above the valley bottom (Europa Bridge near Innsbruck, Moselle Valley Bridge at Winningen, etc.). The piers of these valley bridges are major and important components, not only with regard to cost but also more particularly with regard to their appearance in the landscape. It has been found that even for wide bridges a very favourable solution consists in using narrow individual piers in the form of slender hollow sections or even tubular columns. Both in Italy and in Germany such columns have been provided with "mushroom heads" forming large tables, the remaining gaps between these being spanned by means of beams or slabs. This type of construction has been applied with exemplary success in the case of the Elztalbrücke (Elz Valley Bridge) (Figs. 26 and 27) with octagonal columns, flat pyramidal mushroom heads and solid connecting slabs. This bridge was constructed with the aid of a steel "machine" for installing and supporting the formwork. As a contrast, Fig. 28 shows a valley bridge with the wide piers that used to be normally employed.

For the bridging of wide valleys it is desirable to adopt longer spans, as in the case of the Siegtalbrücke (Sieg Valley Bridge) at Eiserfeld, which, despite its span length of 105 m, was constructed with continuous beams of constant depth, span by span, with the aid of a steel formwork "machine" (Figs. 29 and 30) [31].

For these large and high bridges the continuity of the superstructure over a large number of spans, and if possible over the entire length of the bridge, is a particularly favourable feature with regard to the possibility of securing the piers against buckling and to enabling the structure to resist wind load.

Among the special forms of construction developed for big bridges, mention must first of all be made of the "table bridges with V supports" (Fig. 31) as conceived by R. Morandi, Rome, and used more particularly for the Maracaibo bridge [32]. This system has been applied in the construction of the Columbia River bridge near Kinnaird in British Columbia, Canada.

Arch bridges are becoming rare because they are nowadays usually more expensive than girder bridges. Applied to arch bridges, prestressed concrete has resulted in longer spans for the longitudinal girders supporting the deck, so that forms of bridge emerge as characterised by the Glemstalbrücke (Glems Valley Bridge) near Stuttgart (Fig. 32) [33]. The bridge over the Arno near Incisa, in Italy, is similar in form, but with differences of detail which manifest themselves in the appearance (Fig. 33).

Consistent further development along these lines has led to the trussed frame, a form of construction favoured particularly in Switzerland (Figs. 34

and 35) [34]. With this system large bridges, including railway bridges, were constructed in prestressed concrete already at an early period (e.g., Horrem bridge with $l = 85$ m, see page 296 in [16]).

Finally, mention should be made of the portal-frame bridges with supports consisting of members in a triangular arrangement, as introduced by U. FINSTERWALDER (Fig. 36), which are particularly suitable for the construction of slender superstructures over a main span (page 287 in [16]).

For very long spans in flat country the suspension of beams by means of inclined cables often presents good possibilities (e.g., the Maracaibo Bridge). This very ancient system was revived from about 1946 onwards by F. DISCHINGER and was extensively used in steel construction (bridges over the Rhine in Germany). In the case of concrete bridges constructed on this principle the cables have to carry very large loads, which can be reduced by using lightweight concrete. In such applications it is essential to ensure reliable anchorage of the cables, as described in [35].

7. Continuity of girder bridges over several spans

In many countries throughout the world there are still misgivings about constructing multi-span continuous girder bridges, because adverse effects due to differential settlement of the supports are feared. Such fears are groundless more particularly with regard to prestressed concrete bridges. In Germany many continuous girder bridges have been built from as long ago as 1934 onwards, and favourable experience has been gained with them. Prestressed concrete bridges are generally slender structures and are, because of this, little affected by differential settlement. With the present-day knowledge of soil mechanics the probable settlements can be calculated with sufficient accuracy, so that they can be taken into account in the design. If large settlements are anticipated, the bearings of the superstructure can be made adjustable, so as to enable settlements to be subsequently compensated. Thus, for example, at Duisburg a long succession of bridge spans across a mining subsidence area was constructed with continuous prestressed concrete beams and adjustable bearings permitting movement in all directions [36], although sudden partial settlements of up to 80 cm and an anticipated total settlement of 2 m in course of time are expected to occur there.

An additional consideration is that the bending moments caused by differential settlement will, as a result of creep of the concrete, undergo a reduction of something like 40 to 70%, depending on the rate of settlement [37]. Furthermore, thanks to the high elastic limit of the prestressing steel, prestressed concrete possesses a high capacity for recovery in the event of the occurrence of unexpectedly large settlements which are compensated by adjustment of the bearings.

Nowadays continuous girder bridges are constructed span by span preferably with the aid of transportable steel falsework structures, the tendons being spliced at approximately the fifth-span points [38] (see Fig. 10.38 in [1].

The advantages of structural continuity are obvious: the bending moments in the main girders are greatly reduced and, consequently, the deformations —especially the creep deformations—are also reduced. Greater slenderness can be achieved, i.e., lower construction depths will suffice or, for a given depth, it is possible to construct longer spans than with single-span beams. Also, with continuity the need to provide two sets of bearings at each support is eliminated, and for long bridges rocker supports can be used to provide freedom of longitudinal movement. The main advantage is that the transverse joints over the supports are obviated, such joints being particularly undesirable for present-day fast traffic, besides continually requiring maintenance.

With the large number of continuous girder bridges that have been built more particularly in Europe the experience with regard to continuity has invariably been favourable, so that any still existing doubts as to the suitability of this form of construction can safely be abandoned.

7.1. The risk of cracking in the region of the supports of continuous girders

In some prestressed concrete bridges, especially with T-section girders, cracks have occurred at the undersides of the girders, beside the intermediate supports, at a distance ranging from 0.3 to $2.0h$ (Fig. 37), although calculations showed that for the loading case $g + v_0$ (dead weight + initial prestress) there were still compressive stresses in the section at the support. This phenomenon has been thoroughly investigated in K. H. WEBER's doctorate thesis [39] and the following causes have been established:

1. Temperature differences between the deck slab and the bottom face of the main girder.
2. Possible deviation of the position of the tendons over the support from the specified level in the section.
3. Temporary occurrence of excessively high prestressing force.
4. Curvature of the tendon axis too "flat" in the vicinity of the support.
5. Tensile stress due to pressure exerted by the bearing, especially with high pressures arising from incomplete hinge action (lead bearings, concrete hinges, etc.).

The risk of such cracking is reduced by using "limited" prestressing, which automatically gives higher flexural compressive stresses at the bottom of the girder for the case $g + v_0$ than "full" prestressing does. Furthermore, the tendon centre-line should be given its curvature within a short length (0.7 to $1.0h$) over the support. **I**-sections and box-sections are more favourable than T-sections, as the centroidal axis is not located so high up as in the latter. As a safeguard it is recommended that this region of a continuous girder be provided with a sufficient length of untensioned reinforcement consisting of thin bars spaced at about 10 cm, so as to ensure that any cracks that occur will remain very fine hair cracks and thus be invisible and harmless.

8. Cross-sectional design of girder bridges

There exist two schools of thought concerning the cross-sectional design of girder bridges:

(1) The French school, which uses closely spaced main girders and a thin and only very lightly reinforced, transversely prestressed deck slab. The individual main girder webs are very thin (Fig. 38).

(2) The German school, which has for a good many years gone for widely spaced main girders (5 to 8 m apart) the webs of which and also the deck slab are made relatively thick (Fig. 39).

The first-mentioned type of cross-section is suitable for precast individual beams, whereas the second type is appropriate for in-situ concrete construction.

According to the French school of thought the arch action is taken into account in the design of the deck slab, as envisaged by Y. GUYON [40]: hardly any untensioned reinforcement is used, and the transverse tendons are located centrally. The economy of this form of construction resides in the design of the deck slab. In many other countries this method of slab design is not permitted, however.

In bridge design according to the German school of thought the deck slab is designed to resist bending in accordance with the theory of plates, taking due account of the restraint at the junctions with the main girders. H. RÜSCH [41] published a comprehensive set of tables for determining the bending moments in such deck slabs for the German standard loadings laid down in DIN 1072. K. HOMBERG [42] has latterly produced influence surfaces for deck slabs—cantilevered or fixed to the main girders—of varying thickness. With the aid of these influence surfaces it has been ascertained that the deck slab spans can be made even substantially larger without sacrifice of economy. In accordance with Homberg's proposals, some bridges are now being built with cross-sections as shown in Fig. 40, with cantilever widths of 8 m and spans of up to 16 m between the main girders. These deck slabs are, of course, transversely prestressed. Diaphragms (cross-girders) are dispensed with in girder bridges of this kind, so that the falsework with the formwork can be designed to be travelled longitudinally as construction proceeds (Fig. 41a). The work of constructing such bridges is thereby significantly simplified.

If there are more than two main girders, it is nowadays normal practice to calculate the transverse distribution of heavy vehicular loads over the individual girders with the aid of a beam grillage analysis. Such calculations have been applied in Germany since as far back as 1938 (LEONHARDT [43], HOMBERG

[44, 45]). A favourable method of analysis which takes torsion into account has been developed by Y. GUYON and C. MASSONNET [46, 47, 48]. More recently, in English, the books by P. B. MORICE and G. LITTLE [49], by A. E. HENDRY and L. G. JÄGER [50] and by R. E. ROWE [27] have become available on the subject. In many countries the beam grillage analyses have been programmed for electronic computers, so that taking account of transverse load distribution is no longer very time-consuming. It should merely be noted that, to ensure such load distribution, it is generally sufficient to provide only one diaphragm (at mid-span), and certainly not more than two or three, per span. It is, however, justifiable to provide three or more diaphragms per span if these also perform the function of keeping the degree of restraint of the deck slab approximately constant over the whole length of the bridge.

That even with two main girders the load distribution becomes more favourable than would be the case if the basic lever principle were applied is something that has long been known. This effect has, in the past, been investigated with the aid of the theory of folded plates. More recently, a suitable method for analysing this has become available in W. ANDRÄ's doctorate thesis [51]. Its accuracy has been checked and confirmed by means of model tests.

For particularly slender long-span bridges the box-section (Fig. 42) has come to be widely adopted. It can suitably be precast in successive segments, as was first done in the construction of the Ager bridge [52] and subsequently for many other bridges. Box girders must be designed for bending and torsion. Care must be taken not to make the bottom slab too thin if it is required to resist high longitudinal compressive stresses. Thin bottom slabs can advantageously be stiffened with some transverse ribs. If the main girder soffit is curved (variable depth of girder), the "radial" forces, due to curvature, exerted by the longitudinal compressive forces in the bottom slab must not be overlooked. With regard to torsional design the reader is referred to Section 9.

In order to reduce the span of the bottom slab and also to shorten the piers (in the transverse direction), the box girder webs are often sloped inwards towards the bottom, so that a trapezoidal cross-section is obtained (Fig. 42). Lately, too, box-sections have been designed which comprise triangular lateral cavities (Fig. 43) in order to form additional supporting brackets for the wide cantilevered deck slab. This solution can give the girder bridge a very slender appearance. However, it is essential to provide additional stirrups or tendons in the vertical webs, over the entire web depth, in order to resist the vertical component of the force developed by the inclined outer slab. It must also be pointed out that the weight of the bottom slab of the box girder should similarly be suspended by means of additional stirrups—something that has been overlooked in many a bridge design.

Another trend in cross-sectional design is towards reducing not only the number of webs but also the web thickness in relation to the width of the bridge and the length of the span. This is more particularly possible if the soffit of the girder is curved because then the shear force is partly absorbed by the slope of the force in the bottom of the girder. The steeply inclined principal tensile stresses in the webs can be cancelled by means of vertical or likewise steeply inclined prestressing bars. The criterion for the web thickness is constituted by the inclined principal compressive stresses due to shear force + torsion. In this case it is necessary to apply a lower limit to the compressive stresses than in normal compressive members, as will be further explained in Section 9. So far, the smallest relative web thickness $\bar{b}_0$ has been adopted in the construction of the bridge over the Rhine at Bendorf [53]. In this bridge, which has a span of 208 m and a width of 13.2 m, the webs are 8.0 m deep and 0.37 m thick at the supports, so that:

$$\bar{b}_0 = \frac{b_0 d}{b \cdot l} = \frac{0.37 \cdot 8.0}{13.2 \cdot 208.0} = 0.001 \, .$$

The world's largest prestressed concrete bridges have been built with these box girders, e.g., the Rhine bridge at Bendorf with a span of 208 m [53], the bridge over the Oosterschelde in Holland [54], the bridge to the island of Oléron in France [26], and many others. Box-sections are particularly suitable for the cantilever method of construction which was introduced with such great success for prestressed concrete by U. FINSTERWALDER and which has since been applied in a multitude of variants.

9. Design for shear and torsion

The design of prestressed concrete girders to resist shear forces (*shear* design) has not yet been satisfactorily clarified. In most of the shear tests on prestressed concrete girders no shear reinforcement was provided or the investigators failed to deal systematically with the many variables involved. The design regulations applied in the various countries partly require too much and partly require too little shear reinforcement.

It has been established that the webs should contain a certain minimum reinforcement which, according to tests performed by HANSON and HULSBOS [55], should be as follows for vertical stirrups and concrete of class B 450 (i.e., with a specified 28-day strength of 450 kg/cm²):

$$\min \mu_S \beta_S = 6\% \text{ kg/mm}^2 \quad \text{where} \quad \mu_S = \frac{F_{e,\,stirrups}}{b_0 a}$$

and $F_{e,\,stirrups}$ = cross-sectional area of stirrup reinforcement
a = stirrup spacing
b_0 = thickness of web
β_S = yield point of stirrup reinforcement.

In many cases this minimum reinforcement is sufficient.

It has also been established that there is no point in limiting the (inclined) principal tensile stresses or the shear stresses with regard to the tensile strength of the concrete. It is not difficult to resist the principal tensile forces efficiently by means of reinforcement or by prestressing the web. On the other hand it must be realised that the (inclined) principal compressive stresses constitute an important limit for the shear stress conditions in the webs. Tests at Stuttgart have shown that the grouted tendon ducts in the webs so greatly reduce the compressive strength of the latter that, in designing for inclined compression, the thickness of the ducts should be deducted from the web thickness. Table I gives some of these test results. This need for deducting the duct thickness is avoided by installing the prestressing cables directly beside the web, instead of inside it, and connecting them—after tensioning—to the web by means of stirrup reinforcement and a concrete casing, so as to obtain a connection that will adequately transmit shear. This arrangement was proposed by F. LEONHARDT and W. BAUR in 1957 and has since actually been applied by them in a number of cases [56, 52]. U. GIFFORD [57] has likewise built long-span bridges with thin webs on this principle.

It should also be borne in mind that the compressive stresses which, at failure load, occur in the concrete "struts" between the shear cracks may become significantly greater than indicated by the conventional theory, this increase being due to additional flexural stresses and to other secondary effects. The limiting values must therefore be applied with caution. At the present time the FIP-CEB Joint Committee is discussing proposals with a view to limiting the principal compressive stresses and establishing the design of the shear reinforcement. However, it is desirable to carry out further tests on the subject.

In [58] B. THÜRLIMANN makes a notable proposal for the shear design of prestressed concrete beams, which provides a good basis for further discussion of this problem.

Torsion plays an important part more particularly in box girders. In order to investigate the behaviour of prestressed box girders subjected to torsion, two large-scale tests were performed at Stuttgart. These are reported in [59] and [60].

It was found that, for obtaining the necessary safety against failure, the shear stresses due to torsion should be fully resisted by reinforcement and that therefore no reduction in consequence of the longitudinal force exerted by the prestress is possible. It was furthermore established that the torsional reinforcement should be relatively closely spaced (approx. 10 cm) at the outer corners of the box-section so as to ensure that the concrete at the outermost edges cannot spall off as a result of the change of direction of the "struts". Also, in these tests a reinforcing network with bars parallel to the axes of the girder was found to be more suitable for resisting the torsional tensile stresses than networks with bars at 45° to the axes.

If the box girder is liable to undergo torsion in both directions, it is most certainly necessary to provide reinforcing networks with bars spaced 10 cm to at most 15 cm apart, because possible hair cracks that can develop in, for example, the bottom slab may intersect one another. In order as far as possible to obviate the occurrence of such hair cracks under working load, it is desirable to employ a relatively high degree of prestressing for such box girders.

At failure load (in the case of failure due to inclined compression in the web) the inclined compressive stresses were sometimes more than three times as high as they should theoretically have been, so that for torsion, too, the warning already uttered above is relevant. The behaviour of the prestressed box girders in "state I" (uncracked concrete) under working load showed good agreement with the values calculated with the aid of Bredt's formula.

10. Transmission of prestressing forces into beams

The magnitude and distribution of the tensile stresses occurring in the transmission zone (tendon anchorage zone) have been investigated in a number of research projects, so that the requisite reinforcement can now quite satisfactorily be designed. Unfortunately, the knowledge concerning this is not yet widely disseminated, and for this reason damage in these transmission zones is still of frequent occurrence. A summarising review of present knowledge on the subject is given in Chapter 9 of the book [1]. There, too, the results of tests for the end regions of beams are stated in which, besides the anchorage forces of the tendons, the bearing reaction—affecting the position and magnitude of the tensile stress—is also taken into account (Fig. 44).

It often occurs that tendon forces have to be resisted at the edges of plates or diaphragms. The tensile forces involved can be calculated without difficulty by means of the present methods of stress analysis for plates. In this connection reference should be made to the summarising report of the knowledge concerning deep beams ("girder walls") in [61] and to the publications by W. Schleeh mentioned therein.

In pre-tensioned prestressed concrete members the tendons must, in the anchorage zone where the prestressing force is transmitted by bond alone, definitely be provided with a binding of transverse reinforcement and be connected to the web by means of closely spaced stirrups. Particularly for this reinforcement the text-books ought to give the requisite information as to design and distribution and thereby make it available for application in practice.

In many prestressed concrete structures it occurs that the tendons have to be anchored within a member. For this it is necessary to provide either jacking pockets or even jacking apertures. If a tendon exerts its force upon a diaphragm in this way, tensile forces will be produced directly beside the anchorage in the spanning direction. These forces will likewise require local special reinforcement to resist them. The magnitude of the tensile forces in question has been determined by photo-elastic methods and reported in [62], so that for this case, too, appropriate design of the necessary reinforcement is possible (Fig. 45).

11. Prestressing steels, tendons, anchorages

With regard to the individual wires or strands that go to make up the tendons there is a trend towards the adoption of larger cross-sections. Thus, for example, in Germany wires of steel grade St 125/140[1]) with a diameter of 12 mm have now come into common use, and the application of 16 mm wires is on the way. Starting from the U.S.A. and Britain, the 7-wire and 19-wire strand consisting of wires 3 to 4 mm in diameter is coming more and more into prominence.

So far as the properties of steel are concerned, it is being endeavoured to raise the limit of proportionality in order to reduce relaxation. Unfortunately, this is achieved partly at the expense of ductility, which is a very necessary property, e.g., for anchorages based on wedge action. It is questionable whether this trend of development is correct. Adequate ductility of the steel is most essential particularly with regard to anchorages. Reducing the relaxation would not be necessary if the permissible steel stresses at tensioning were not so high as, for example, in France, where a value equal to 85% of the tensile strength of the steel is allowed. Such a high stress, even if it occurs only temporarily at the time of tensioning the tendons, is at variance with all sound rules of engineering with regard to safety, which must be ensured also during construction, i.e., when tensioning is in progress. There is no loss of economy if the permissible stress is somewhat reduced and a lower degree of prestressing (as envisaged in Section 1 of this report) is adopted.

Also with regard to the tendons there is a tendency to increase the prestressing force per tendon. Whereas formerly designers contented themselves with forces of 25 to 50 tons per tendon, nowadays there are already a number of systems available with tendons of 200 tons and upwards. The anchorages and tensioning devices used for such tendons are indeed somewhat clumsy, but on the other hand the advantages from the point of view of structural design are undeniable. With the aid of the so-called concentrated tendons used in the Baur-Leonhardt prestressing system structures containing cables developing over 3000 tons of prestressing force per cable have meanwhile been constructed (Caroni Bridge) [63].

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[1]) The two values represent the 0.2% proof stress and the tensile strength of the steel respectively (in kg/mm^2).

Tendon anchorages by means of wedges are coming into increasingly widespread use. With such anchorages the unavoidable slip of the wedges must be duly considered. The reliability of wedge anchorages should, however, be very carefully investigated, so that the details—e.g., including the hardness of the wedges, etc.—can be accurately specified. In Germany, dynamic testing of the anchorages (not embedded in concrete for the purpose of the test) with 2 million pulsations (stress variations) with a lower stress limit equal to 50% of the tensile strength and with a stress variation range of at least 1200 kg/cm^2 is required. This severe requirement has revealed many a defect in anchorages, and those anchorages which have been tested in this way are thus known to possess a high degree of reliability.

In many cases spliceable tendons are used. These are now available in various prestressing systems. More particularly at splices it is essential that the anchorages be very reliable and have high ultimate strength. The splice joint itself is usually formed by means of screw-threaded connections, which should be amply designed, since ordinary screw threads do not possess any very great fatigue strength. This strength can, however, be increased by adopting a suitable shape for the screw thread.

12. Corrosion protection of prestressing steel

Unfortunately, some cases of harmful corrosion to prestressing steel have ocurred. In so far as the corrosion was not caused by clorides from additives or the like, it was ascertained in most cases that the grout had not perfectly surrounded the prestressing steel. It is therefore urgently necessary to achieve further improvement of the quality of the grout and of the grouting procedure. The FIP recommendations for grout unfortunately give no strict ruling as to the maximum water/cement ratio for grout; according to the German directives this must not exceed the value of 0.44. Also, a special additive with water-reducing and expanding action should be specified in order to obviate the formation of cavities. Furthermore, it is necessary to ensure, more than has hitherto been done, that the prestressing wires are somewhat uniformly distributed over the cross-section of the sheath or duct, so that the flow resistance encountered by the grout will not vary too much. The sheaths, too, require to be further improved in design so as to ensure that the wires are only over the shortest possible distances in contact with the walls of the sheaths.

It should also be investigated whether, instead of cement grout (which continues to be rather unsatisfactory), it is not possible to find another grouting medium with a more reliable corrosion protecting action and good strength for establishing bond.

Protection against corrosion, of course, imposes the further requirement that the concrete surrounding the tendons must be dense and free from cracks.

To be certain of this, the concrete cover should be related to the tendon sheath diameter; it should be at least equal to the diameter of the sheath or, for large cables, at least 8 cm.

13. Parapets and guard fences

It repeatedly occurs that vehicles meet with serious accidents on bridges or even fall off bridges. In both cases this is attributable to the solution that has hitherto usually been adopted to provide protection, namely, the arrangement whereby either a 60 to 70 cm high concrete guard member or a high kerb with an adjacent heavy bridge parapet is provided. Both protective devices are hard and unyielding, so that a vehicle which happens to stray from the carriageway receives a severe blow. If the centre of gravity of the vehicle is sufficiently low, it will be hurled back, badly damaged, on to the carriageway; if, on the other hand, the centre of gravity is higher, the vehicle will somersault over the parapet and fall off the bridge. The solutions hitherto employed have also been attacked by O. A. KERENSKY [65].

So it is being increasingly widely realised that this method of protection is wrong. It is at variance with the law of nature that kinetic energy is destroyed by energy of deformation (force × distance). If the protective device is unyielding, then there is no "give", i.e., no "distance", so that the force becomes tremendously large. In tests carried out in California and in Sweden it was shown already years ago that with yielding protective devices, ropes in particular, the vehicles can be caught and stopped in such a manner that only slight damage is caused and that, above all, generally no serious personal injuries occur.

In the last two years the German Road Research Association (Forschungsgesellschaft für das Strassenwesen) has conducted extensive further investigations, which showed that even heavy lorries (trucks) which collide with suitable protective devices at an angle of 20° and a speed of 80 km/h are "gently" deflected back on to the carriageway and sustain no damage worse than dented bodywork. More tests with yielding bridge parapets are planned. There is no doubt that bridge parapets can be so constructed that vehicles are safely stopped by them and are not hurled back on to the carriageway. For this it is necessary that the parapet structure should be able to deflect laterally a distance of 0.8 to 1.0 m. The method of fixing should be so contrived that no damage to the bridge deck itself is caused.

On the other hand, it should be possible to look through the parapet, so as to give the drivers of vehicles an unobstructed view of the river under the bridge. There is nothing so frustrating to the motorist as driving over a river and not even seeing the water.

In any case it is an important task for us, bridge engineers, to develop suitable parapets. International co-operation in this field would be of real value.