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IV

General Report.

Generalreferat.

Rapport Général.

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Two of the many subjects which fall under the heading of reinforced concrete engineering — namely surface structures and long span bridges — have been selected for treatment at the fifth meeting, with a view to indicating the trend of their development. The justification for treating them together is the fact that both give rise to the same problems: — the need for a more penetrating analysis of the true conditions of stress so as to minimise the material required while still ensuring adequate safety, and, on the other hand, the need for constructional measures designed to ensure that the material is so stressed that its capacity for carrying load is utilised to the utmost.

The category "surface structures" may be subdivided into shells (or structures consisting of thin plates of uniform curvature) and folding structures (in which the thin curved plates are replaced by a polyhedron of facets).

The latter type has hitherto been applied mainly to bunkers and silos, to the chimney walls, of cooling towers and to roofs. Since the Paris Congress these applications have repeatedly been the subject of papers in the Publications of the I.A.B.S.E. The strict theory underlying them, taking account of bending stresses, has been trated by *Gruber*, *Grüning* and *Ohlig*. *Gruber*'s work is contained in Vols. 2 and 3 of the Publications, and in Vol. 3 *Craemer* has dealt with the stresses in continuous bunker walls due to dead load and to friction of the contained material.

Nothing arises here regarding applications of this type of structure, but, as *Dischinger* has pointed out in his paper, they are a less economical form of roofing than shell structures, in view of the higher bending stresses.

Under the heading of shells a distinction must be drawn between those spanning with simple curvature between supporting girders and those with compound curvature. In the first group, considerable increases in span are to be reported; thus *Valette* refers to a design by *Boussiron* for a roof of cylindrical shape in which the span both of the arches and of the girders is to be 51.5 m, and *Dischinger* gives an account of the construction of a roof in which the girders span a distance of 60 m and the arches 45 m; also of roofs over halls in which the spans of the arches are up to 100 m with the trusses at relatively short

[1] The author died shortly before the Congress and his report was read by *Mr. Bornemann*.

intervals. In France, besides cylindrical shells, skew shells have been developed, the conical shell being a special case, and applications have also been made to cantilever roofs, saw-toothed roofs, and roofs over halls extending up to 60 m span. These have been described by *Lafaille* in Vol. 3 and by *Fauconnier* in Vol. 2 of the Publications. Finally, under this heading of simply curved shells, there should be mentioned pipes simply supported between girders.

The most notable example of this form of construction as mentioned by *Valette* is a diffuser for the wind channel at Chalais-Meudon, conical in shape with an elliptical cross section, the axes of the ellipse at the open end being respectively 23 and 15 m in length, and the structure being supported at points 34 m apart. For large spans such shells are nearly always stiffened by means of ribs in order to increase their resistance to buckling. The recent tendency as regards the shape of the arch is to adopt a segment not of a circle but of an ellipse, with a higher camber than the corresponding catenary since in this way a more effective girder effect and more favourable distribution of stresses is obtained in the shell, and since, moreover, this type of arch with its increased height ensures a very considerable reduction in bending moment; also it has been shown by *Finsterwalder* in his contribution to the discussion that the elliptical cross section is less unfavourably affected by creep than a circular cross section.

In Vol. 1 of the Publications, *Finsterwalder* put forward his theory of cylindrical shell structures in which account is taken of the bending moment in the direction of the arch, but in which moments in the direction of the genatrix are neglected. An alternative theory, in which these bending moments also are taken into account, was published by *Dischinger* last year in *Beton und Eisen*, and in his present paper *Dischinger* observes that it has meanwhile been found possible to develop an accurate theory which holds good also of shells in which the height is considerably increased and in which the cross section is elliptical or similar; this theory is about to be published as a dissertation for the Doctorate.

In the case of a simply curved shell, in which the spans of the arches and girders are large, special attention must be paid to safety against buckling: this contingency must be examined both in the direction of the arch and in that of the genatrix at the same time, for the simultaneous action gives rise to considerably less favourable conditions than would be indicated by separate calculations for the two directions. The problem of a cylindrical shell which is continuous over several spans has been treated by *Dischinger* in Vol. 4 of the Publications.

A general theory for skew shells has been explained by *Laffaille* in Vol. 3 of the Publications and has been confirmed by experiment. In Vol. 2 *Fauconnier* has described tests to destruction carried out in France to check the validity of an approximate solution as applied to conical shells. The special case of the parabolic hyperboloid has been dealt with by *Aimond* in Vol. 4.

Whereas, in Germany, the mathematical treatment of the design of shell structures has been pursued to an ever increasing degree of refinement with the object of arriving at as exact as possible an understanding of the true conditions of stress, in France greater emphasis has been laid on liberating the designer as much as possible from the restraint of difficult calculations. The reconciliation of these two points of view will be mentioned again later in this report.

In the matter of shell structures with compound curvature no special advances since the Paris Congress call for mention, but the theory underlying these forms has been carried a great deal further with the aid of differential equations. The spherical shells for the large market hall at Dresden, mentioned by *Petry* in his paper at that time, have unfortunately not been carried out. Some notable applications of compound curved shells are mentioned by *Dischinger* in his paper. The reports by *Aimond* and *Granholm* are concerned with theory; *Aimond* deals with the geometrical significance of the general conditions of equilibrium in shells which are free to bend and hence deduces those marginal conditions which imply definite and stable conditions of equilibrium with different shapes of shell structure; *Granholm* aims at obtaining as simple a solution as possible, in the place of badly convergent infinite series; he considers the cupola as an elastic network, and treats the meridian strip as being throughout elastically supported. In this way the investigation of cupolas with varying thickness of walls may be carried out in a simple manner, and good agreement is obtained with a more rigorous solution.

The paper by *Parvopassu* deserves special mention as forming a transition between the two parts of this meeting. In that paper a brief historical account is followed by a cross section (as it were) through the whole field of reinforced concrete construction as applied to buildings and bridges, and the close connection that exists between research and effective engineering is demonstrated, each of these activities both stimulating the other and requiring its aid. It is, indeed, a conspicuous fact at the beginning of any discussion on long span bridges that spans admit of increase only if the principles of calculation, design and construction are susceptible to check and only if the assumptions — which must inevitably be made — are found to hold good when extended to cases out of the ordinary. We need to decide, therefore, the directions in which these assumptions require to be clarified and their validity guaranteed.

In the papers grouped under the heading of "long span bridges" those which refer to arched forms of construction predominate, because arch bridges are those which offer the greatest promise of notable increases in span, whether from an economical or a technical point of view. Here the term "arch bridges" is taken to include all curved structures which are stressed mainly in compression, regardless of whether the thrust of the arch is resisted by the abutments or is relieved by tie bars.

There are two conditions to be satisfied on which all others depend; namely that of minimising the dead weight of the arch and floor structure, and that of minimising the cost of falsework. The former condition requires a decision as to the permissible stresses to be adopted, which will affect the working stresses and the sizes of cross section. Opinions differ considerably on the subject of permissible stresses, according to the strength of the concrete which may be relied upon with certainty on the job; the point can be decided only on the merits of each particular case, depending on what methods are deemed to be economical for producing the concrete and, above all, for ensuring its density. In the case of the Traneberg bridge at Stockholm, as *Kasarnowsky* explains, a stress of 120 kg/cm² was allowed under the least favourable conditions of loading. *Boussiron* says that under normal conditions 100 kg/cm² represents a maximum which should not be exceeded unless the concrete is hooped, but if this is done he has no hesitation in doubling the permissible stress. In such a case the section of the concrete is made up of numerous hooped lamellae, the bond between which must be ensured by the provision of transverse reinforcement.

For the present these limits of stress may be high enough, for it appears from the papers of *Boussiron* and *Gaede* that an increase in the permissible stresses becomes advantageous only in the case of spans still regarded as exceptional for reinforced concrete bridges. For every span there is a limit of stress beyond which an increase is attended by hardly any reduction in weight. If, then, the limits here given are accepted, the problem becomes that of so designing the arch that the maximum stress is kept low and occurs in as large a portion of the arch as possible; at the same time, the range of variation in stress between the maximum and the minimum must be kept as small as possible over the cross section, having regard to fatigue effects and to the elimination of tensile stresses; that is to say, the absolute values of the moments should be as nearly equal as possible in every cross section, and they should differ as little as possible over the length of the arch; apart from this their maximum values should be such that the line of thrust remains within the core of the arch.

There are three possible ways of approaching this objective: namely by suitable choice of shape for the axis of the arch, the flow of the moments of inertia between the springing and the crown of the arch, and by influencing the position of the line of thrust when striking the falsework. As regards the shape of the arch the rise is the governing factor and the choice of the rise is usually confined within narrow limits determined by the span, by the constructional depth available (in cases where the roadway is above the arch) and by aesthetic considerations (in the case of a roadway suspended below the arch).

Hawranek, in his paper, discusses the possibilities of influencing the moment by choosing a line of thrust which deviates from the axis of the arch, and he shows that it is not possible in this way to obtain complete equalisation of the absolute values of the moments at all cross sections in the case of a built-in arch. *Boussiron* makes an exhaustive investigation into the variation of the moments of inertia, and refers to the papers by *Chalos* and *Valette* which are printed in Vol. 2 of the Publications. He shows that in this way a considerable degree of equalisation of moments along the axis of the arch may be obtained. in *Bauingenieur*, 1935, *Dischinger* has made suggestions as to how the extreme fibre stresses might be equalised by the use of temporary hinges or jacks placed eccentrically. The proposals made by *Fritz* in *Schweizerische Bauzeitung*, 1935, are similar. A diagrammatic representation of the amount of material to be used has been found very useful in deciding upon those values which allow for a free choice.

The urge towards reduction in weight and towards large radii of gyration has universally tended towards the adoption of hollow sections. On the question of hinged versus built-in arches, the bending moments arising in wide spans are weighing the balance ever more in favour of the built-in type. This, however, entails an increase in the degree of uncertainty, because no absolutely rigid fixation is obtainable even in rocky ground, and the lack of absolute rigidity increases with the magnitude of the load and of the moments at the springing.

None of the methods mentioned above for designing the arch can attain its objective unless the moment and thrust are correctly estimated. In the case of long span bridges it is no longer sufficient to calculate the stresses in the arch by reference to its undeformed axis, hoping that the arrangements for striking the centering will ensure that the axis of the arch lies in the intended direction; it is necessary to go further and check the stresses that may arise both in the elastic system and in the permanently deformed system that will result from creep and shrinkage, and to take these effects into account when designing the cross sections. With this object *Hawranek*, in his paper, has developed methods of calculation which take account of varying moduli of elasticity of the concrete according to its age, and also of the elastic deformation undergone by the axis of the arch. The same problem has been dealt with by *Freudenthal* in Vol. 4 of the Publications. *Kasarnowsky* explains how the secondary stresses brought into play by deformation were taken into account in the design of the Traneberg bridge.

Calculations of this kind nevertheless remain unsatisfactory unless it is possible to ascertain the true relationship between modulus of elasticity, age of concrete, and constructional procedure adopted. Knowledge of this modulus is also necessary for determining the resistance of the arch to buckling, and it is generally acknowledged that in the case of long span bridges safety against buckling is a factor which requires to be checked. Measurements which have been carried out on finished bridges more and more frequently in recent years with a view to determining the modulus of elasticity have brought no clarification of the problem, because they have been concerned only with local conditions and with a limited period of time. It is now established that deformations due to creep represent much the largest proportion of the total deformation arising, unless special measures are taken to reduce the creep, which is not always possible. *Dischinger* explains that the detrimental effects of creep can be eliminated if the arch is so shaped that no bending moments arise under dead load, because under that condition no additional moments can result from creep. This requirement can never be strictly fulfilled in the case of a built-in arch, for creep develops over a long period of time, during which seasonal variations of temperature may cause moments to persist for a long period although the arch axis coincided exactly with the funicular line for dead weight immediately after striking the centres. The best that can be done is to ensure that at points where creep strains may give rise to heavy additional moments, the permanent moments and the thermal moments persisting over long periods are kept as low as possible. These difficulties must not, however, be allowed to engender any temptation to undervalue exact calculations. The latter desideratum is countered by the urge to simplify the work of calculation, and both these tendencies are reflected in the paper by Mörsch dealing with the effect of braking loads on solid bridges, in which it is shown how braking effects may beconsidered simultaneously with the vertical loads in the resulting influence lines. The paper by Valette in Vol. 2 of the Publications, relating to the validity of calculations for arched *types* when applied on a different scale, is relevant here.

In arch bridges wherein the roadway is suspended below the arch it is possible considerably to reduce the bending moments in the latter if the suspension bars between the arch and the floor are given the form of a triangular arrangement, by being made inclined, that only those hangers come to act which are in tension. In Vols. 1 and 4 of the Publications, *Nielsen* has dealt with various systems of this kind. *Boussiron* in his paper describes the largest structure with inclined suspension bars which has hitherto been built, namely the Castelmoron bridge, which has a span of 143 m.

Let us now turn to the second of the main requirements, namely that of minimising the cost of the falsework. This is dealt with in the papers by *Boussiron, Hawranek* and *Kasarnowsky*. A successful choice of construction for the falsework is in fact often decisive as regards the feasibility of a long span arch bridge, or as regards the competitive power of a solid arch against a steel arch. The maximum reduction in the cost of falsework in relation to the unit price of the superstructure is obtained when the falsework can be re-used several times, a notable example of this being the bridge at Plougastel which has three equal openings and also the Traneberg bridge in Stockholm which has two twin spans. But conditions as favourable as this seldom arise, and the possibility of limiting the span of the falsework by dividing it into a number of separately supported portions becomes less as the span of the bridge increases. The problem which generally arises is that of building the falsework in a single span which can be only slightly shorter than that of the completed bridge. When it is remembered that rigorous conditions have to be imposed upon the accuracy of the intrados of the arch, and upon deformability of the falsework while the load is being imposed, it will be clear that the design and construction of the latter entails difficulties no less formidable than those appertaining to the design and construction of the bridge itself.

A simple and reliable solution as regards the conditions of a satisfactory deformation is provided by the use of a steel falsework arch, as adopted for the Traneberg bridge. For still greater spans, however, this type would rapidly become too heavy and uneconomical, on account of the necessary resistance to buckling. *Hawranek* suggests that such an arch should be strengthened by a suspension cable, but as a rule a timber falsework will be more economical, especially if used in conjunction with a suspension cable, and indeed this often provides the only means of constructing the work. Apart from suspension cables, prestressed tension members may often be used with advantage. Arrangements whereby the desired shape of the centreing can be reproduced and maintained during the process of loading and striking may also be useful. Finally, forms of centreing have been proposed which remain in place as part of the finished structure, somewhat after the manner of the *Melan* system. In the pure form of the latter the rigid reinforcing frame constitutes the whole of the falsework, but for large spans this ceases to be economical.

With a view to reducing the amount of material required for the falsework it has often been suggested that the latter should be loaded with only a portion of the weight of the arch; in other words the arch should be built up in the form of rings, only the lowest of which would have to be carried by the falsework, the successive rings being supported on those already in position with or without the aid of the falsework. This very attractive idea is exposed to the disadvantage, however, that the conditions of stress in the separate rings during the process of construction cannot be accurately ascertained without difficulty, and that even if the desired shape of the finished arch can be ensured by this method uncertainties will still remain on account of variations in the modulus of elasticity of the different parts which have been concreted at different times, and which have been subjected to different amounts of pre-stress before being freed from support.

With the striking of the falsework for long span arch bridges the object pursued is usually that of conferring a definite shape of axis on the arch after elastic compression has taken place. This objective cannot be attained by merely sinking the falsework from below the arch; it is sometimes necessary to make use also of hydraulic jacks inserted at suitable places, so that such loads can be introduced into the arch as will give rise to the desired conditions of stress and strain. The object cannot, however, be realised unless it is possible accurately to calculate the state of deformation beforehand.

The technical limit of span for solid arch bridges is fixed by *Freyssinet* at 1000 m and by *Boussiron* at 800 m, the attainment of these spans depending on increased permissible stresses which at present generally lie beyond what is economically practicable. At the same time spans of between 200 and 300 m, such as come into question in practice, are already easily obtainable with the present permissible stresses.

An increase in the spans of girder bridges beyond what has hitherto been obtained is scarcely possible, by the methods hitherto in use, merely by increasing the permissible stresses in the concrete and steel and by improving the design of cross sections, because crack formation in the concrete (a tendency which grows with every increase in the stress) tends to impair the life of the structure, and because the own weight of the structure increases so rapidly that very soon the economic limit is passed. These disadvantages may be overcome if pre-stressing of the reinforcement is applied in such a way that either the concrete is entirely relieved of tensile stress, or the stress is so reduced that no cracks can be formed. At the same time a greatly improved utilisation of the cross section of concrete is secured in this way, which leads to a reduction in the dead load; moreover, by sufficiently heavy pre-stressing, the deflections may be greatly diminished. *Dischinger* in his paper lays down the condition that the cross section should be so designed, and the pre-stressed reinforcement should be so arranged, that none but concentric compressive forces will occur in a girder bridge under dead load. If this is secured, bending effects will arise only under live load and will be purely elastic, so that the life of the structure will be a maximum.

In order to attain this object *Dischinger* proposes to separate the principal steel reinforcement from the cross section of the concrete, and to arrange them inside the box shaped girder as a suspended funicular tensile polygone, so shaped that the distances from the neutral axis of the concrete will be proportional to the dead weight bending moment. The reinforced concrete girder is to be supported from the suspended reinforcing boom at the points of intersection, in such a way as to allow the girder to move longitudinally. The reinforced concrete section will then act under dead load only as a continuous girder carried on these internal points of support, and will have to cover much smaller spans than the external span of the girder. In this way the dead load moments of the reinforced concrete structure will be reduced to an extraordinary extent, the dead load stresses becoming relatively small by comparison with the pre-stressing imposed by the suspended reinforcing boom. The latter is to be stressed by means of hydraulic jacks until it transfers the whole of the dead load of the reinforced concrete girder on to the bearings. The reinforced concrete construction may with advantage be divided up by joints in such a way that only certain specified portions of the cross sections co-operate statically, and so that the neutral axis assumes the most favourable position.

As a result of creep in the concrete the intended condition of stress in the girder would disappear in course of time. In order to avoid this *Dischinger* proposes that the suspended reinforcing boom should occasionally be re-stressed until a stable condition is attained. Besides *Gerber* girders, and continuous girders with spans up to 150 m, *Dischinger* refers to the design of a suspension bridge with a reinforced concrete stiffening girder. At the present time a girder bridge of 70 m span is being constructed in accordance with his suggestion. The same object is being pursued, though in a somewhat different way as regards details, by *Freyssinet*. It is to be hoped that by the adoption of solutions of this kind the spans of reinforced concrete girder bridges may, within the limits of the economically feasible, still be considerably increased.

Reviewing the conclusions that emerge from this discussion, a number of general results will be evident. The greater the demands made upon reinforced concrete construction, whether in the form of shells or in that of bridges, with a view to increasing the span while minimising the amount of material, the more imperative does it become to check the assumptions on which our theories are based, and to confirm whether these continue to correspond with the more exacting requirements made. As responsible and conscientious engineers we ought to adopt new and bold methods only when their safety is guaranteed to us by calculation. We are compelled to make assumptions in order to be able to calculate at all, and in every case, therefore, we idealise the properties of the material and the mechanical conditions obtaining in the structures. There can, therefore, strictly speaking be no accurate theories, and what are known as approximate solutions differ from others mainly in the degree of idealisation applied to their basis. Does this mean that what are called rigorous solutions are to be ignored? Not at all; for these provide the only means whereby we can assess the degree of approximation of the solutions used in practice, and the more rigorous they are the better will they enable us to do so. Moreover they provide the essential basis for all experiments on finished structures, for work in the experimental laboratory, for observing what is essential, and for correctly interpreting observation.

Conversely, it must be insisted that anonyme who develops rigorous solutions should be particularly careful to check his assumptions, in order that what appears to be increased rigour may not merely be more difficult mathematical treatment. But even mathematical proof of adequate safety is not enough, unless care is taken, in constructing the work, to ensure that its condition shall correspond as far as is possible with the assumptions underlying the calculations. To stress this inter-relation between all concerned — theorists and practical men, testers of materials, staticists, designing and constructing engineers — is one of the most important objects of this Congress. It is a need which the elucidation of the theme of the present paper throws into special relief.