IVa

Walled structures.

Flächentragwerke.

Surfaces auto-portantes.

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IVa 1

Cylindrical Shell Structures.

Zylindrisches Schalengewölbe.

Les voiles cylindriques.

Dr. Ing. U. Finsterwalder,
Berlin.

During the last few years the construction of shell roofs has been influenced by the following two factors:

1) Recognition of the plastic properties of concrete.
2) Endeavours to eliminate tension zones from the concrete of tensile members as far as possible.

Ever since shell roofs were first introducet it has been sought so to improve the underlying theory that the design (of what are often very bold constructions) might be placed on the safest possible basis. As Prof. *Dischinger* has indicated in his paper, it has been found possible to describe the conditions of stress very completely, but an important pre-condition for this continues to be the assumption of an elastic material with characteristics corresponding to those assumed in the calculations.

In this respect reliance has been placed mainly on the official regulations which prescribe a value of $E = 210000 \text{ kg/cm}^2$ as a basis for the calculation. On this value being checked by means of measurements when the shuttering was struck it was found that the deflections actually observed were smaller than those calculated, and from this circumstance it was deduced that a higher modulus of elasticity might be assumed, and with it a greater degree of safety against buckling than had been taken in the calculation.

The fact is however, that in course of time the deformations considerably increase and may often reach several times their original values. This increase in the deformation does not always follow immediately upon the construction, but more usually it occurs in the warmer seasons of the year when the concrete is exposed to the sun's rays and is caused to dry out; in winter time, on the other hand, only a small amount of movement can usually be observed. Despite the drop in the modulus for deformation as regards dead loading, an increase in the stiffness against temporary loading is observed, so that the actual modulus of elasticity has in fact become greater. If the dead load were to be removed this deformation would vanish only in accordance with the actual modulus of elasticity and the greater part of it would remain; it follows, therefore, that the initial state must have undergone a change.

This phenomenon is important in shell construction, because if the span is notably large and the thickness is small in proportion to the curvature, a limiting condition may be reached in which the initial shape undergoes considerable alteration through bending, this being true of shell construction in much the same way as of slabs subject to large deflections. Once a certain ratio between the curvature and the thickness (governed, among other factors, by the magnitude of the bending stresses) is exceeded, the shell requires to be stiffened by ribs, just as T-beams are used instead of flat slabs.

The bending stresses in a cylindrical shell can be reduced by the well known expedient of adopting an increased curvature, made up of sectors of circles, in the neighbourhood of the springing. This gives small radii of curvature near the springing where bending stresses mainly occur but where they exert no important effect, while at the same time the crown portions, where the radius of curvature is a maximum, are free from bending.

According to its conditions of loading, the shell may be stiffened by cross ribs in the direction of the arch or in the case of still flatter curvatures by both transverse and longitudinal ribs. The spacing of these ribs will depend on the danger of buckling in the intermediate portion of the shell, which tends to buckle into short waves. Since the camber of the shell is very small in proportion to the length of such a buckling wave, this arch effect is neglected, and buckling is calculated as for a flat slab which is stressed in its own plane and fixed along the edges. The depth of the rib is so chosen that dangerous bending deformations due to creep are eliminated, and that an adequate guarantee against buckling of the ribbed shell between the frames is assured.

On these principles very large arched roofs have been built over aircraft hangars, covering up to 60 m free span, for the girders with an arch span of 45 m and very flat curvature. Such a structure is shown in Fig. 1 in the characteristic one-sided form which offers the simplest possible solution to the problem in question: The two-storeyed building for workshops and offices was required to be open on the side of the aerodrome, and to have as flat a roof as possible, while allowing a free doorway opening of 50 m. Contrary to the usual arrangement the axis of the arch is placed parallel to the doorway so that the shell girder with its large moment of inertia spans across the door opening; at the back it is carried down to the level of the out-buildings and is supported on a row of pivoting columns, the auxiliary buildings at the back not receiving any horizontal thrust from the arch. In the end walls and in the central frame a heavy horizontal thrust from the arch is taken up, and to provide for this the edge of the arch along the end walls is strengthened to form a tension boom and is carried on the columns of the wall framing. The central girder is designed as a two-hinged frame with a pre-stressed tie in the floor, its loading is equal to the width of the door in one panel, or in the present case 50 m; it is designed as a twin frame to allow of possible dilatation. The shell is stiffened by thin ribs at 3.60 m centres which also serve the purpose of transferring point loads from the crane runways into the arch. In hangars already constructed these point loads amount to as much as 35 tonnes, and the ribs are thickened accordingly.

The calculation of stresses due to the point loads was made by the strict theory of shell construction, with approximation by means of *Fourier* series, and by measuring the deformations on completed structures it was possible to confirm the results of the calculations.

The carrying capacity of the shell for point loads is remarkably high, a fact which may be explained by the circumstance that the rib has the effect of distributing the load, by bending, over a large width, so that the lever arm with which the internal forces are transmitted to the trusses is a favourable one. The compressive stresses which arise in all directions due to such a load greatly outweigh the tensile bending stresses.

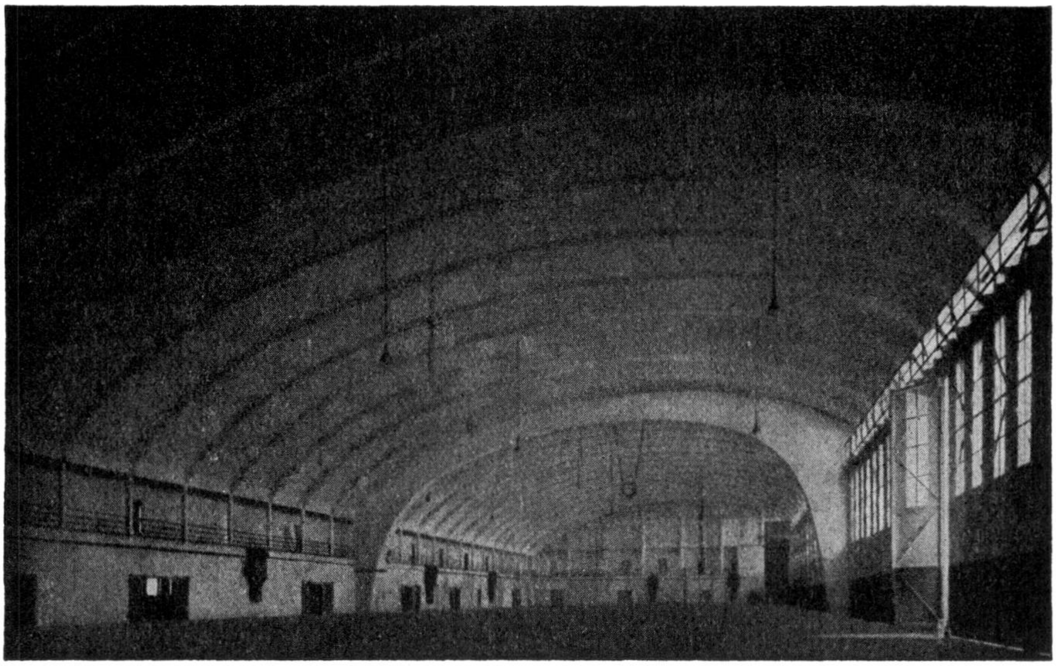

Fig. 1.

Aeroplane Hangar Area 35×100 m.

The effect of creep on the construction was also examined. Since the heavily reinforced parts of the structure along the edges are less influenced by creep than the remaining portions of the shell a not inconsiderable redistribution of stresses occurs, which may be estimated with close approximation by repeating the calculations with the modulus of elasticity reduced to 100000 kg/cm^2. It thus appears, both from theory and from practical observation, that the crown of the arch subsequently sinks further than the face girders which remain at an approximately constant level.

The construction of the face girder over the doorway is a problem of some difficulty and interest. In the first place the illumination of the hangar is effected entirely at the face girder; lighting from this high level on one side only ensures uniform illumination over the whole building. In the second place this face girder, in addition to carrying its vertical load, has to resist a tensile force of about a thousand tons due to its forming a tie bar for the shell, this force increasing parabolically from the supports towards the centre of the span.

In reference to lighting these hangars have furnished some particularly useful knowledge, indicating that even though the window area is considerably smaller the same lighting effect may be obtained as with the usual bands of light around the whole building. The principal reason for this lies in the smaller amount of glare, allowing the eye to make better use of the available light, and a second reason is to be found in the light coloured surfaces of the arch which — provided they lie at a lower level than the band of light — receive direct illumination and form a very effective additional source of light for the back portion of the hangar. Consequently, with 12 % clear lighting area in a hangar 45 m deep, fine mechanical work can be carried out even with the doors closed.

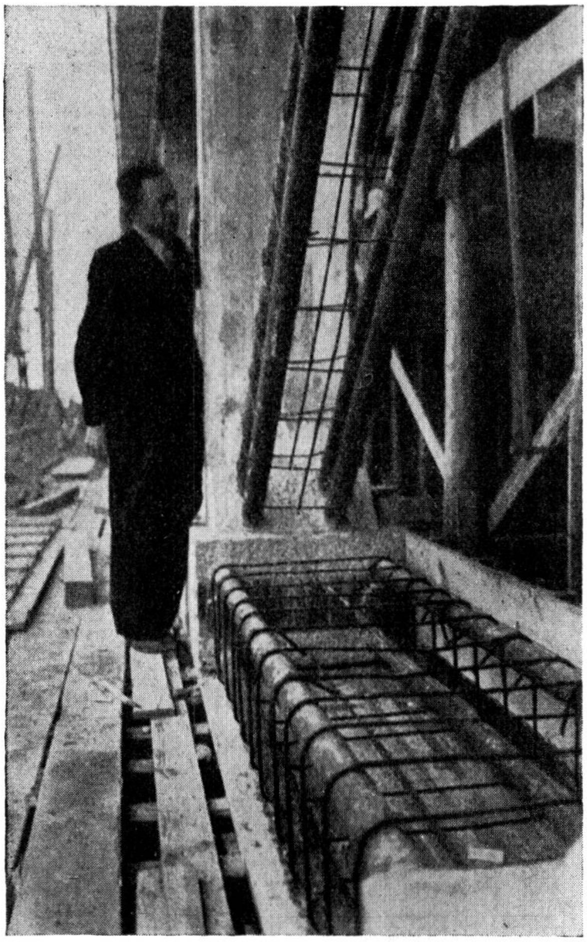

Fig. 2.
Trussed girder ever entrance door, shuttering removed.

The problem of designing the face girders was solved by the adoption of a reinforced concrete lattice girder of special construction. Such lattice girders have often been used abroad, secondary stresses being ignored, just as in steel construction, but in Germany this would not be possible as all secondary stresses have to be included within the scope of the permissible stress, and moreover the anchoring of the steel in the tensile zone is not permitted. These more onerous conditions were satisfied by not covering the bars in the tensile members of the girder until after the removal of the formwork (Fig. 2). Since the tensile members are made up of bundles of round rods, they can freely participate in the necessary angular deformations. In this way, apart from the secondary stresses, it was possible considerably to reduce the tensile stresses in the concrete, though high tensile steel (2100 kg/cm^2) was used in the tensile members. The system of truss adopted comprised tension diagonals and compression verticals, merging suitably into the stiffening ribs of the shell, so as to perform the additional function of transferring wind pressure from front of the door on to the arch. The tensile bars consist of a limited number of large round steel rods which are anchored into the concrete intersection point by means of nuts and plates. At each intersection the whole force of the diagonals is anchored, after being carried over the intersection in a flat bend; the counter force of the lower boom is likewise carried through the intersection point and is anchored on the other side. In this way the intersection point is subject to a heavy compressive stress and is enabled to equalise the forces of various members. The forces in the thin diagonal being anchored inside the intersection, and those in the wider lower boom outside the intersection, the reinforcing steel can be placed in position without difficulty. In order to avoid undesirable eccentricity, steel erection plates are built in, enabling the work to be done with great accuracy, and this being the case the verticals could be made so slender that they receive no tensile bending stresses due to secondary stresses. The economy of the trussed design lies in the fact that use is made of the qualities of the high tensile steel, thus saving considerable weight whilst at the same time increasing the stiffness in every direction. Through the introduction of the truss the possible span of a shell girder is considerably increased.

This particular example has been selected from among the many shell roofs carried out during the last few years, because it illustrates a particularly notable form and magnitude of the cylindrical type of shell developed in Germany. Every increase in the size and difficulty of structural problems must be accompanied by an extension of our knowledge of the theoretical principles and of methods of working — and last, but not least, of constructional materials. It is particularly in respect of materials that we have received such valuable stimuli from abroad: one may hope that this effect will be mutual, and will contribute to the enrichment of the engineer's art.

IVa 2

Experiments on Models to Determine the Most Rational Type of Reinforcement.

Modellversuche zur Bestimmung der zweckmäßigen Anordnung der Bewehrungen.

Recherches expérimentales des systèmes d'armatures rationnelles.

Dr. Ing. V. Tesař,
Paris.

Compliance with the elementary condition that the reinforcing bars must be placed in the direction of the maximum tensile stresses is essential for good work in reinforced concrete construction. Obvious as this condition may appear, it is too often not satisfied in practice. In the design of thin walled structures especially, certain assumptions are made which are not entirely justified, and the results of the calculations can then be only an imperfect representation of the true conditions of stress. If experimental tests are made, either on actual work or on models, wherein the reinforcing bars have been arranged in accordance with such calculations, the only result will be to show whether the reinforcement so provided has in fact been adequate: but it is impossible to infer from such experiments whether the system of reinforcement is a rational one.

The object of this contribution to the discussion is to recall attention to the possibility of devising rational systems of reinforcement through research by the method of photo-elasticity. This method has been known in principle for a hundred years, and was made available in engineering practice thrity-six years ago by the eminent French engineer and scientist *Mesnager*, since when, through the notable work of *Coker*, *Filon* and other investigators, the scope of photo-elasticity has been extended to all branches of construction. For the bibliography of the subject reference may be made to the author's recent article in the fourth volume of the Publications.

Three-dimensional problems, wheter plane or curved, are capable of resolution by means of suitable experimental methods, and this applies to both thin-walled and massive constructions.

In order to reduce the length of this contribution to a minimum, reference will be made straightaway to an actual example of experimental investigation. Fig. 1 is an elevation of the cantilever bridge at Bry-sur-Marne, belonging to a type several examples of which have already been built in France, in the neighbourhood of Paris. The cantilevered portion is 22.5 m long on each side, and the simply supported central portion is 22 m long, giving a clear span of 67 m. There are no horizontal reactions. The cantilever is rendered stable by the weight of the filling over a floor which forms an extension of the lower girder of the heavy framework in which the cantilever is embedded. This frame leave a free passage, 3.50 m high by 8.60 m wide, for the roadway which runs along the river bank.

The bridge is of recent construction, the contractors being the firm of *Schwartz*-Hautmont. It was considered desirable by the Service des Ponts et Chaussées of the Seine Department (under Messieurs *Levaillant,* inspecteur général, and *Gaspard* and *Peyronnet,* ingénieurs, of that service) that photo-

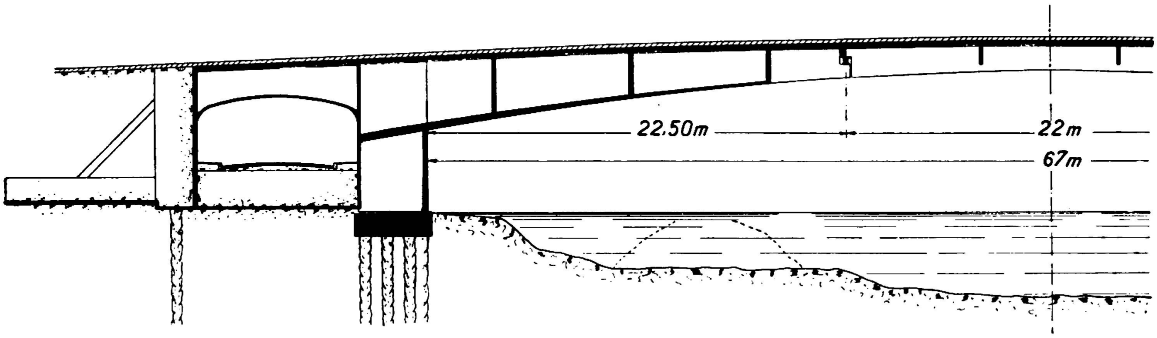

Fig. 1.

elastic experiments should be carried out on a small scale model in the laboratories of the Ecole Nationale des Ponts et Chaussées in Paris, with the object of checking whether the system of reinforcement as indicated by the usual form of calculations might not have been the cause of the cracking observal to have taken place in two instances of similar bridges previously constructed. The following is a brief description of the researches carried out. The model was made of xylonite, faithfully reproducing the dimensions of the bridge to a scale of 1:100 so as to ensure similarity between the conditions of stress in the actual work and those in the model. The model was strengthened, apart from the parts in compression, to correspond with the reinforcement as experimentally provided, wherever the proportion of the latter was in excess of 2 %. It had been found in earlier experiments that for ensuring similarity in the elastic phenomena there is in practice no need to take account of reinforcements if their percentages are less than 2 %, when using models of a homogeneous material such as glass, xylonite, bakelite, etc., as the homogeneous material in resisting tension automatically compensates for the absence of reinforcement in the model — always provided that the percentage of reinforcement, indicated above, is not exceeded.

Where, however, the reinforcement is heavier than this, it becomes essential to increase the strength of the tensile fibres in the model made of homogeneous material accordingly, and this may be accomplished either by increasing the thickness of the model or by attaching reinforcements to it to correspond with those in the concrete in excess of 2 % of the cross section.

In the present case, then, the additional thickness in the model served as its reinforcement, the cross sections so added being made equivalent to between ten and fifteen times the excess of reinforcement over 2 % in the tension zone of the concrete.

The model constructed in accordance with these principles (Fig. 2) was subjected to a system of forces to correspond with full load on the cantilever with minimum load over the remainder of the structure. Fig. 3 represents the first experimental stage, the plotting of *isoclines,* which are the geometrical loci of those points where the principal stresses are oriented along the plane

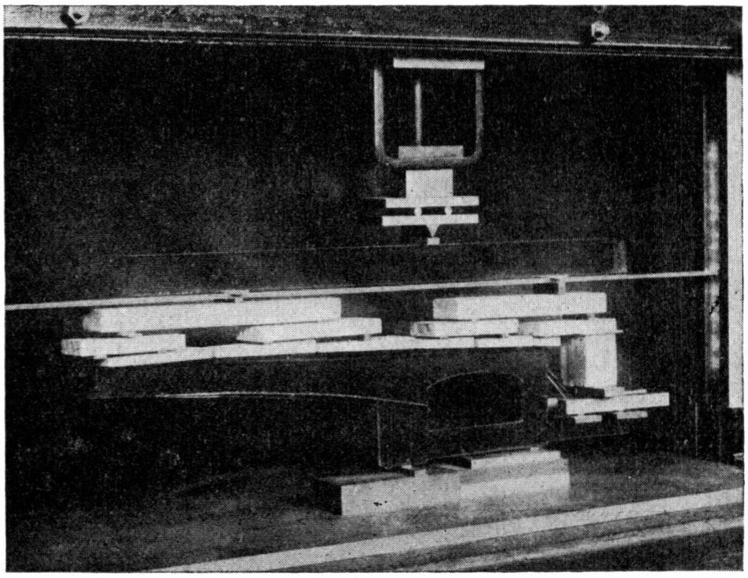

Fig. 2.

of polarisation, the arrangement of which is shown in the right bottom corner of the same figure. From this the corresponding *isostatic* lines Fig. 4 were derived. The stresses parallel to the isostatic lines, which are indicated respectively by broken and full lines, will be designated as v_1 and v_2.

The quantitative solution to the problem is furnished by Fig. 5 which gives the curves of compression (or tension) from which may be determined the moments, the normal forces and the shears at any desired section. Figs 6 and 7 show the isoclines and isostats for the right hand pier and for the girders embedded therein to an enlarged scale, together with certain details which could not be included in Figs. 3 and 4. It will be noticed that in these diagrams the isostats in the zones corresponding to the compression slabs are omitted, for as the present experiments were confined to observations in a single horizontal direction normal to the median plane of the model, they did not enable a detailed investigation to be made of the phenomena occuring in the compression slab, where, in addition to the stresses v_1 and v_2, there is a further stress v_3, the effect of which combined with that of the non-uniform distribution across the thickness of the slab, is to cause perturbations incapable of being analysed by observation in a single direction. An experimental solution of the stresses in the compression slabs would necessitate the use of a glass model made to a larger scale and the making of observations in several directions.

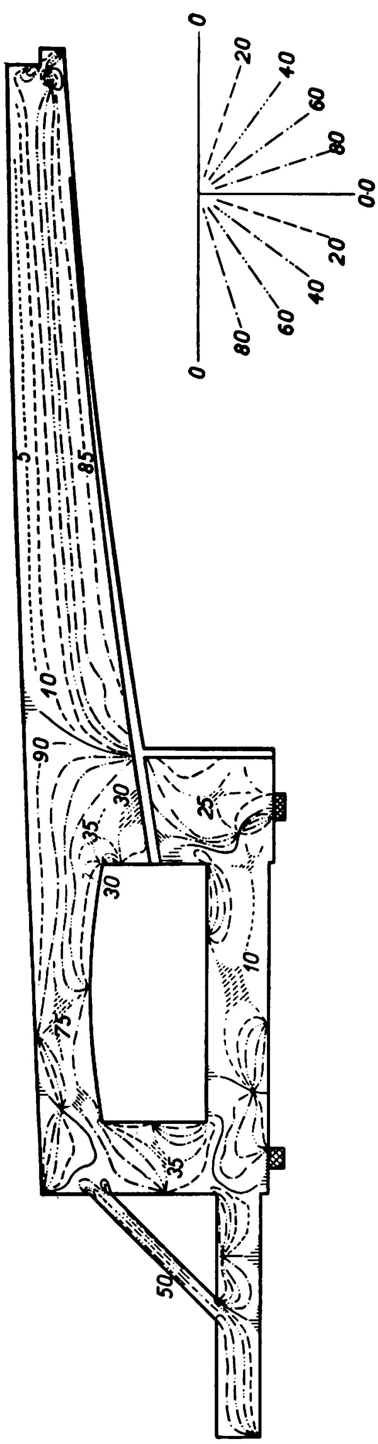

Fig. 3.

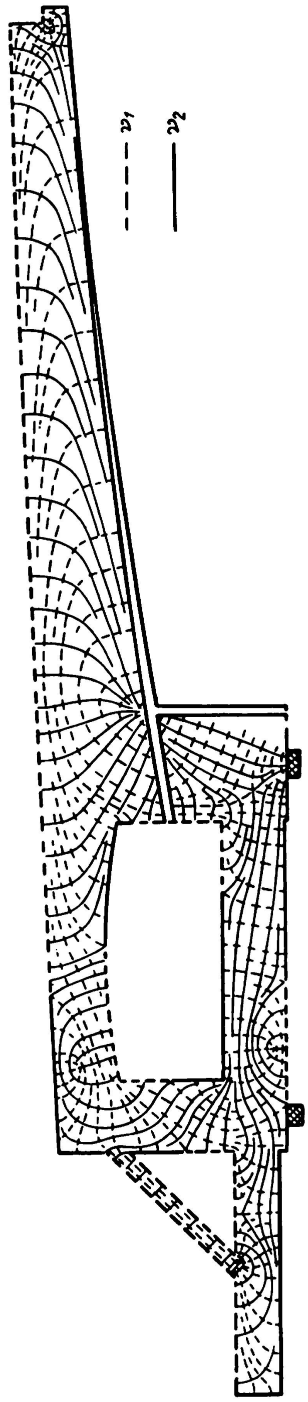

Fig. 4.

Fig. 9 represents the lines of equal stress v_1. The numerical values marked in Figs. 8 and 9 are expressed in tons per linear metre, and provide a means of obtaining the average values of the stresses across the thickness in kg/cm^2. This is done by dividing the numerical value v_1 or v_2, as read off the diagram by 10e, where e denotes the thickness in metres at the corresponding point in the actual structure.

The lines of equal stress v_1 have been indicated as chain lines where they represent tensions and as full lines where they represent compressions. It will be noticed that v_1 is always a tension except in the four cross-hatched zones, where it is a compression, but in Fig. 9 on the contrary, the stresses v_2 are all compressive, with the exception of one zone which is shown hatched to indicate tension.

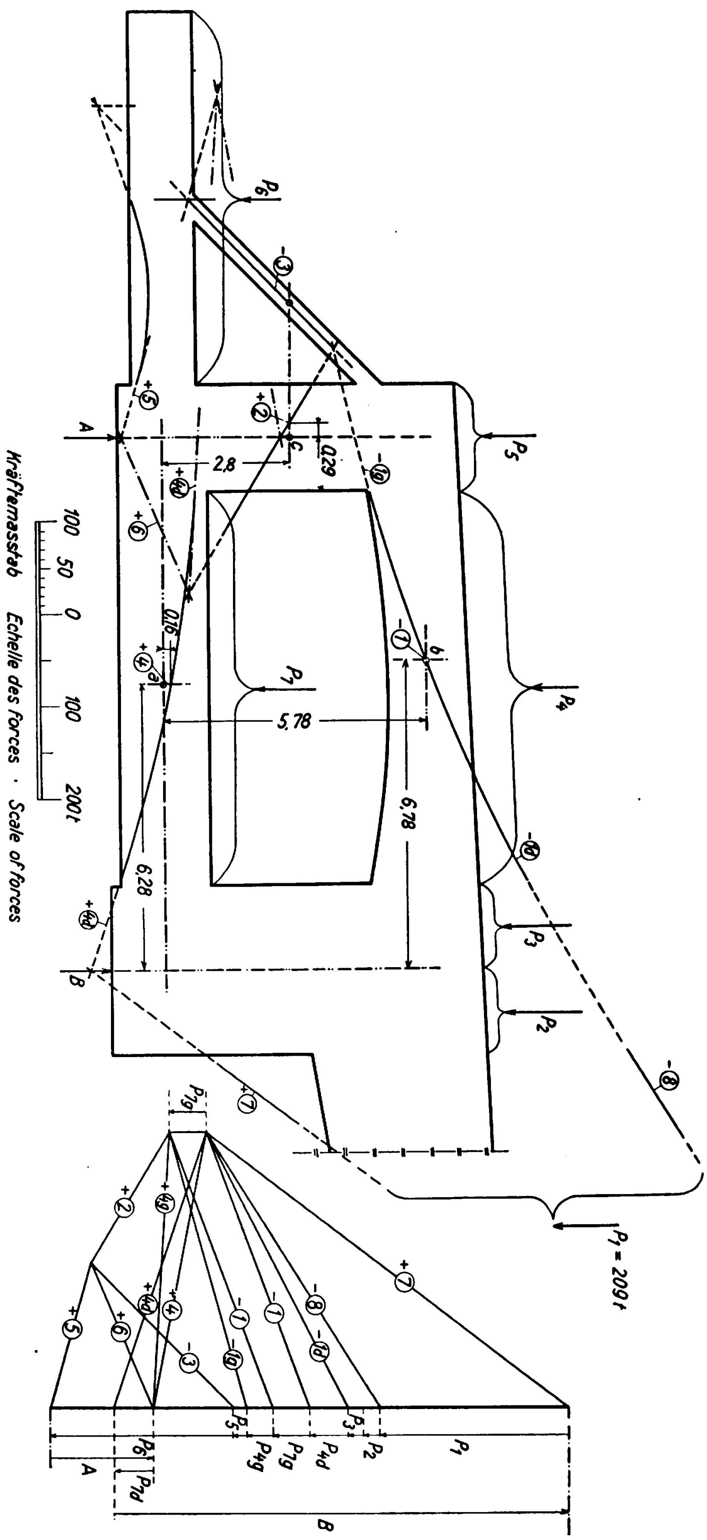

Fig. 5.

The practical upshot of this is that the isostatic lines of Figs. 5 and 8 fail to confirm the propriety of the system of reinforcement derived from calculations made in the accepted way. *Moreover, the experimental study discloses the existence of tensile stresses in the concrete, which are not negligible, in those zones where the usual calculation assumes the absence of any such tensile effect, the supposition being that all tensile forces are taken by the reinforcement close to the tensile surface.*

In order to overcome the risk of cracking — or at any rate to ensure that any cracks that do occur will be so minimised as to be practically imperceptible — there is every reason for arranging the reinforcements along the direction of the tensile isostats. Apart from the principal reinforcing bars close to the tensile edge, which are required by the usual form of calculation, it is of value to add other bars in the region separating the compression concrete from these principal

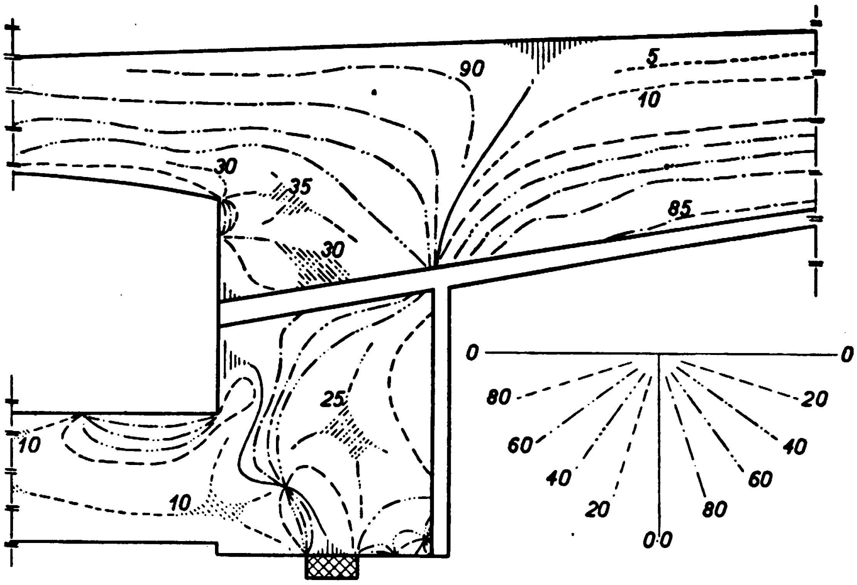

Fig. 6.

bars, wherever the amounts of tension as determined from Figs. 6 and 7 are in excess of the permissible tensile stresses in reinforced concrete.

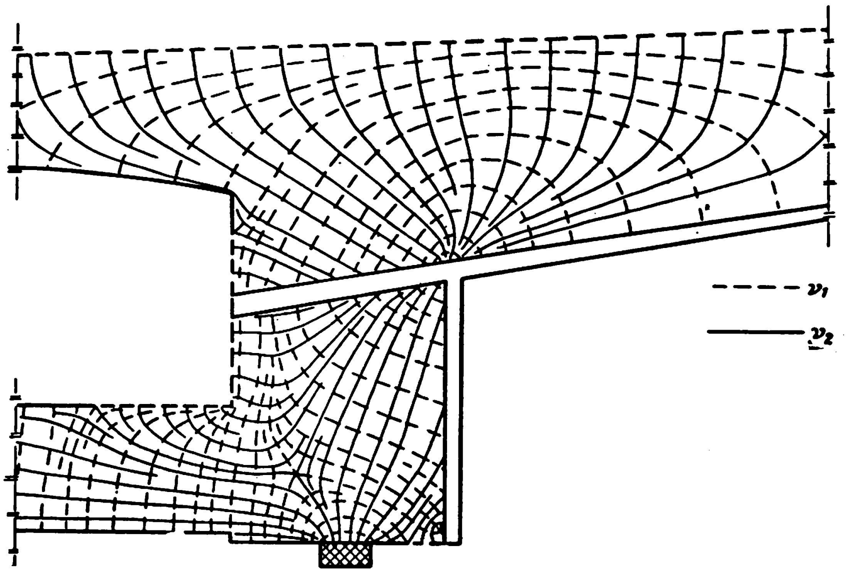

Fig. 7.

In conclusion it may be apposite to recall the fact that photo-elastic methods of measurement offer a means for arriving at the proper amount of preimposed stress to be applied to reinforcing bars with the object of favourably pre-stressing the concrete, and of judging its effectiveness.

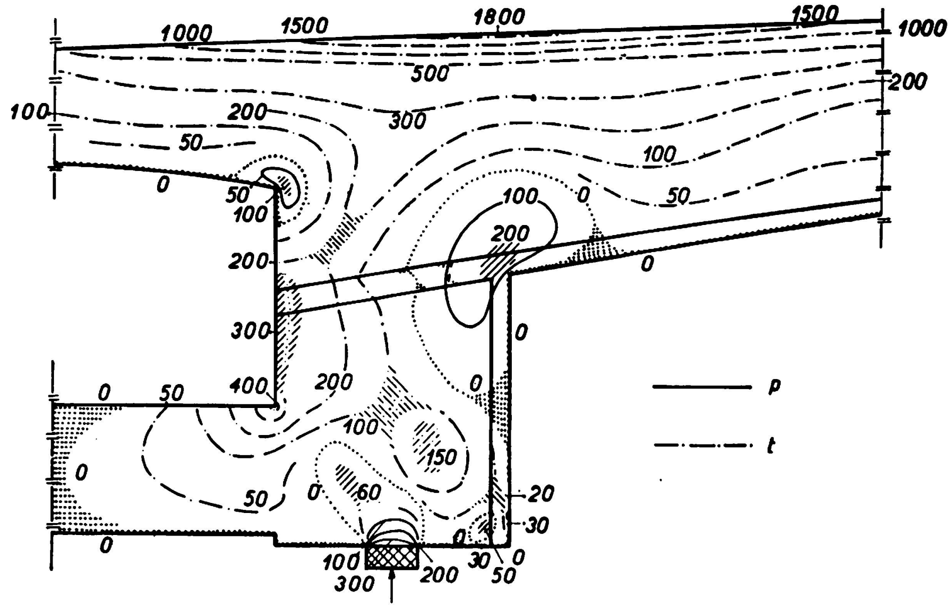

Fig. 8.

Lines of equal stress v_1 values in t/m.

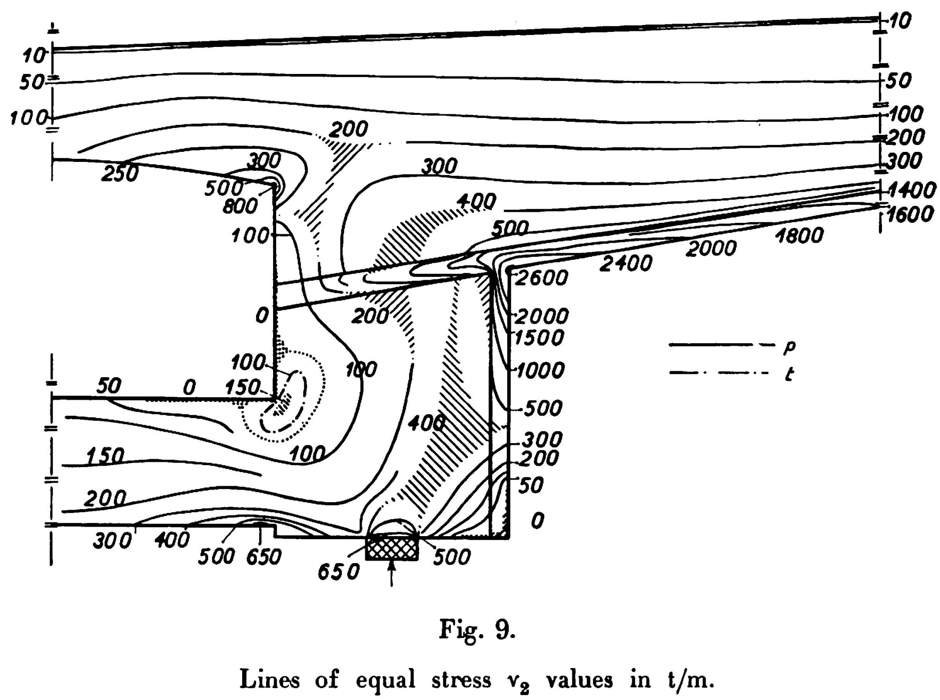

Fig. 9.

Lines of equal stress v_2 values in t/m.