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Reducing the Risk of Cracks in Reinforced Concrete Structures.

Die Erhöhung der Rißsicherheit bei Eisenbetonbauten.

L'amélioration de la sécurité à la fissuration dans les ouvrages en béton armé.

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The difficulty of ensuring that reinforced concrete structures, however well reinforced, shall be entirely free from defects and cracks is known to every specialist. Fine hair cracks are in fact present in the majority of reinforced concrete beams, and are due to the tensile strength of the concrete having been exceeded by the bending stresses which result from the dead weight of the beam and the imposed loads in addition to temperature and shrinkage stresses, or, in most cases, to a combination of such stresses.

These fine hair cracks have no effect on the carrying capacity of the structure, as the tensile strength of the concrete has been left out of account in the statical calculations and all stresses on the tension side are carried by the steel embedded therein. The hair cracks may, however, afford a channel whereby in course of time the surrounding air may penetrate to the steel and cause rusting if it is damp or acidic. Twenty-five years ago this danger was the subject of lively discussion among engineers, but experience has since shown that in carefully executed reinforced concrete work it does not exist and that no fear need be entertained of the reinforcing steel being gradually destroyed through rust in this way.

In the last few years the question of the freedom of concrete from cracking has, however, again come to the fore in connection with the use of high tensile steels and with the execution of structures of ever increasing span. In accordance with the German regulations for reinforced concrete the type of commercial steel which has hitherto chiefly been used may be stressed up to 1200 kg/cm^2 in ordinary cases, and more recently a permissible stress of 1500 to 1800 kg/cm^2 has been authorised for St. 52. Generally the adoption of these higher stresses in the steel is associated with higher tensile stresses in the concrete, with the result that the margin against cracking becomes smaller.

During the past ten years the spans of girder bridges have continually been increasing. Thus the bridge across the Danube et Grossmehring, completed in 1930, has a span of 61.50 m over the central opening, and the bridge of the SA in Bernburg crosses the Saale by a span of 61.78 m. Both of these are girder bridges with a supended span in the central opening over the water. Large halls have been built as two-hinged frames of approximately 53.0 m span, and at the end of this paper reference will be made to a statically determinate roof structure carried on two supports at a clear distance of 50.0 m with a span of 50.80 m between centres of bearings.

It is to be anticipated that this process of development will continue, and that in the future even greater spans will be bridged by reinforced concrete structures subject to bending. In these large spans it becomes of cardinal importance to reduce the dead weight to a minimum, and the sections of the reinforced concrete members must, therefore, be made as light as possible. As a result, the portion of the cross section of concrete which is stressed in tension will be reduced in area; the tensile stress therein will be correspondigly greater and the safety against cracking will be smaller.

The direct tensile strength of concrete such as is used in reinforced concrete structures lies between 12 and 25 kg/cm^2 according to the quality of the work. The bending tensile strength, which as a rule affords a better standard of comparison, may be taken as 25 to 30 kg/cm^2, but it is to be noticed that the upper limit is reached only with the best possible workmanship, using aggregates of the highest possible quality and a correspondingly small water content.

The extensibility of concrete in tension lies between 0.1 and 0.2 mm per m; that is to say when this amount of elongation is exceeded the concrete begins to crack. In selected types of concrete it may be possible to increase the amount to 0,3 mm per m. In making this statement no account has been taken of plastic strain, the magnitude of which, in concrete under tension, has hitherto been little investigated, and the possible effect of which may be to increase the total elongation two or three times.

The shrinkage of concrete suitable for use in reinforced concrete structures is usually given as about 0.4 mm per m, a large proportion of the total shrinkage being attained by the end of a few months. The concrete continues, however, to shrink slowly for a further period, and does not reach its final dimensions till the end of about five years. The rate at which shrinkage proceeds is greatly influenced by the degree of dampness or dryness of the surrounding air. It is known that concrete will shrink very rapidly in warm dry rooms whereas under water it will not shrink at all, but on the contrary, will swell.

The shrinkage value of 0.4 mm per m as stated above can only be taken as a laboratory figure. In massive structures, and indeed in all-work out of doors, the shrinkage is less, being reduced by the natural dampness of the surrounding air. If the amount of shrinkage be reckoned at 0.15 to 0.20 mm per m a considerable part of the extensibility available in the concrete will already have been utilised even if it be assumed that plastic deformation has operated to relieve the load. It has been shown that reinforced concrete structures inside closed buildings and exposed to rapid drying may show fine hair cracks from shrinkage alone.

In the case of reinforced concrete girders of long span the tensile stress imposed on the concrete by shrinkage is usually smaller than the stresses caused by the external loads, particularly those due to dead weight and live load, and possibly the temperature stresses. Elongations of 0.2 to 0.4 mm per m must be reckoned with if the construction is to be economically feasible, and in most cases this in itself implies that the extensibility of the concrete is exceeded, with the result that the fine cracks on the tension side will become more pronounced and be apparent even to the unpracticed eye.

It is easy to understand the desire that these cracks should, as far as possible be eliminated, even though in most cases they are merely defects of appearance. The most effective solution to this problem would be for the cement industry to produce a cement capable of conferring a higher tensile strength on the concrete, or alternatively one which would reduce the modulus of elasticity E for concrete in tension, thereby increasing the elongation.

It is obvious that comparison with natural stones affords no great hope of this, for the latter all possess much higher compressive than tensile strengths. Another warning against exaggerated expectations may be found in the fact that during the past few decades scarcely any advance has been made in the matter of increasing the tensile strength of concrete. If, however, it became possible merely to increase the *bending* tensile strength of a good concrete, which may to-day perhaps be put at 40 kg/cm², by some 50% to 60 kg/cm², a great step forward would have been made, and many forms of structure would become possible which at present are excluded by the risk of cracking. Having regard to the low value of the tensile strength of concrete as such, a 50% increase within a reasonable time may not perhaps, be outside the bounds of possibility.

A further measure that might be generally adopted would be so to regulate the final distribution of stress, by the use of pre-stressing, that the tension in the concrete would be limited to an acceptable amount. Such pre-stressing might be applied either to the concrete or to the steel, but in the former alternative it would be necessary to use a type of cement which expands instead of contracting as does the cement now in use. Whether the cement industry is in a position to produce a cement of this kind is an open question: according to an article by *Henry Lossier*, «Les Fissures du Beton Armé»,[1] the French industry appears likely to do so in the near future.

Attempts to pre-impose a stress in the concrete through the medium of the steel are almost as old as reinforced concrete construction itself, having been suggested by *Koenen* in a paper entitled „Verfahrung zur Erzeugung einer Anfangsdruckspannung in Zuggurtbeton von Eisenbetonbalken“ which appeared in the Zentralblatt der Bauverwaltung in 1907.

To ensure that no cracks shall appear in the concrete, which implies that the tensile strength of the latter must never be exceeded, it is necessary to calculate and dimension the structure in such a way that the bending tensile stress σ_{bz} is kept within reasonable limits. According to the present practice of reinforced concrete design it is quite rightly customary to take no account of the bending tensile stress, for this affords no proper basis for calculation, and is affected by shrinkage, which in turn depends on the arrangement and cross section of the steel. The regulations for reinforced concrete are so framed that if they are carefully and correctly followed there is no chance whatever of hair cracks arising. According to DIN 1075, „Berechnungsgrundlagen für massive Brücken“ an

[1] Le Génie Civil, 1936, pages 182 following.

estimation of the bending tensile stress is necessary in reinforced concrete girder bridges of more than 20 m span, and the matter is governed by the requirement that σ_{bz} must not be greater than one fifth of the calculated compressive strength of the concrete, failing which special precautions are to be taken against dangerous cracking. This regulation is important, and might well be applied in a similar way to roof trusses of large span or to other such structures in reinforced concrete building work.

In an article entitled „Die Donaubrücke Großmehring"[2] the author has calculated the maximum tensile bending stresses in a series of long span reinforced concrete girder bridges and obtained values between 37 and 47 kg/cm². In the bridge over the Saale near Bernburg, already mentioned, the corresponding

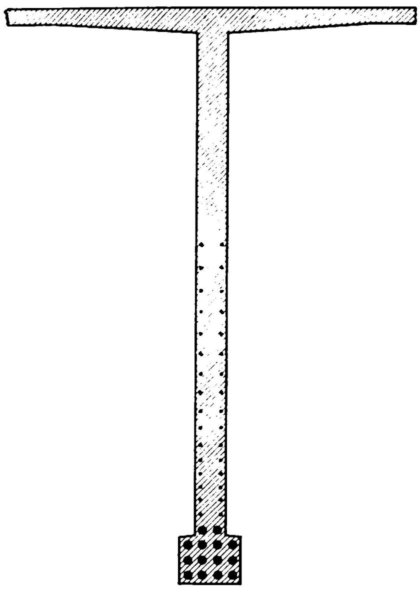

Fig. 1.

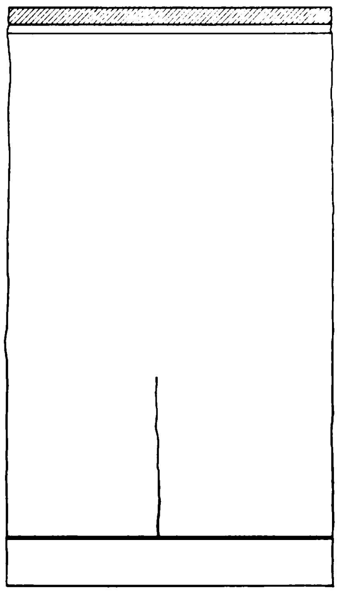

Fig. 2.

maximum value of σ_{bz} is 55 kg/cm². In the present state of concrete technique it would appear inadvisable to exceed this upper limit without quite special precautions.

In reinforced concrete structures of long spans the girders are even now being made of considerable height, for instance in the bridge over the Danube near Grossmehring which has already been mentioned several times the depth is 2.75 m at the middle of the span and 5.40 m over the bearings. When the cross section is of this magnitude the calculated tensile reinforcement placed close to the extreme fibre may properly be supplemented by adequate longitudinal reinforcement below the surface covering the whole of the tension zone, with a view to preventing the formation of cracks between the actual tensile reinforcement and the neutral axis, or at any rate to lessen their concentration. In the case of a section like that shown in Fig. 1, wherein the lower boom has been widened to accomodate the necessary tensile steel, it has been found that whereas the concrete shows no tendency to crack in the enlarged section at the bottom,

[2] Zentralblatt der Bauverwaltung, 1931, pages 123 following.

it is apt to do so in the relatively thin web portion above, and the purpose of the longitudinal bars indicated in Fig. 2 is to combat this tendencey. The presence of a large number of steel bars in the lower boom portion has the effect of increasing the extensibility of the concrete there, and this is supplemented by some measure of plastic deformation, which explains the absence of cracking.

Two mistakes that are frequently made may be mentioned at this juncture: the first is that of crowding many hooks into the same cross section, and the second is the occurrence of changes in the cross section due to openings or offsets. Wherever possible hooks should be avoided in the tension zone of the concrete, though this may not always be possible in cross sections subject to great variation in stress, as for instance at the corners of frames; in any case however, it is undesirable that a number of bars should terminate in hooks at the same place. The presence of the bent hooks is equivalent to a considerable

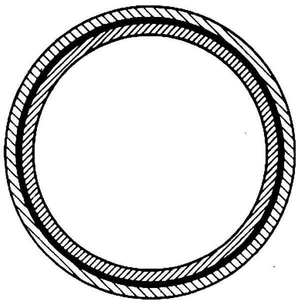

Fig. 3.

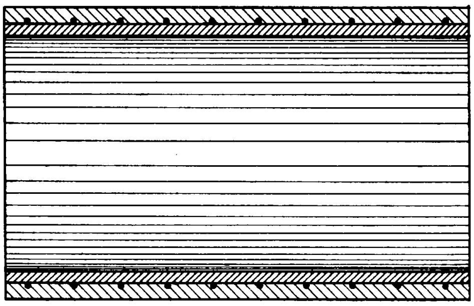

Fig. 4.

reduction in the cross section of concrete which may originate a crack at the point affected.

For similar reasons, openings or sudden changes in section are undesirable and even small openings like those required for the passage of cables and the like should not, if possible, be placed in the tension zone. If such an arrangement is unavoidable additional steel should always be placed around them in order to prevent the formation of cracks.

The expedient of subjecting the steel to a preliminary stress as mentioned above has frequently been followed with success in the case of reinforced concrete members produced under factory conditions. An interesting example of this is provided by the Ruml pipe, in which the annular reinforcements are pre-stressed so as to avoid the occurrence of tension in the concrete under the tangential forces which result from heavy hydraulic pressure inside, and on account of the pre-stressing only compressive stress arises.[3] Fig. 3 shows a cross section and Fig. 4 a longitudinal section through such a pipe. The inner portion, as far as the steel, is concreted first in suitable shuttering, and when this has hardened the steel reinforcement heated in an oil furnace is wound tightly over the concrete core, the third stage of the process being the formation of the outer portion of the concrete. It is claimed that these pipes are completely water-

[3] See „Eisenbetonrohre R. T. System Ruml", by Dr. *F. Emperger*, Beton und Eisen, 1931.

tight even under pressures of ten atmospheres, and they have been widely used in Czechoslovakia and several other countries.

Where reinforced concrete structures are built on the site, pre-stressing has been used with success for purely tension members such as for instance the anchorage of a roof truss or an arch bridge. At the suggestion of *Dischinger*, tie-bars for taking up the horizontal thrust of reinforced concrete arch bridges have been pre-stressed by the use of hydraulic jacks. *Pujade-Renaud* in a paper entitled «Les hangars triples à hydravions de la base maritime de Karouba (Tunisie)»[4] has described French aeroplane hangars with roofs of arched construction wherein the horizontal thrust, if too great to be resisted by the ground itself, is withstood by circular steel bars extending from one bearing to another and embedded in the ground. In this instance the pre-existing stress has been obtained by forcing the bars apart at their centre.

Pre-stressed tie-bars of this kind have also been successfully applied in Germany for large covered hall structures built in the form of arches. Usually hydraulic jacks have been employed, by means of which the desired amount of load can be accurately imposed.

Figs. 5 and 6 show a wide span roof over a hall without intermediate supports consisting of relieved arches of 100 m span between the abutments. The arch ribs are placed at 5 m centres and bear at each end on continuous abutments, to which the anchorage of 40 m diameter bars is attached in the usual way by means of hooks embedded in the concrete. The tie-bars were interrupted at the centre of the hall in order to insert the hydraulic jacks used for pre-stressing, an arrangement which offers the advantage that the full half length of the tie-bars, measuring 53 m, could be obtained ready made from the rolling mills and did not therefore need to be welded on the site. Details of the joints in the tie-bars and the method of stressing are shown in Fig. 7: the bars connected to the left hand abutment are secured at their right hand ends in an anchoring beam to the right of the centre of the hall, and those which connect with the right hand abutment are similarly embedded at their left hand ends in a second anchoring beam placed to the left of the centre line. The respective tie-bars from either side thus overlap by about 3 m at the middle; those coming from the left pass through the left hand anchoring beam, and those from the right through the right hand anchoring beam, by the way of gas pipes. Hydraulic jacks of 50 tons lifting capacity were placed in the space between the two anchoring beams, enabling the latter to be forced apart and the required stress imposed in the tie-bars. This pressure was applied simultaneously with the removal of the scaffolding which was carried on screw jacks, and during the operations the relative positions of the abutments were read on Zeiss dials giving an accuracy of $\frac{1}{100}$ mm. A certain amount of tension was first applied to the tie-bars and the lowering of the scaffold was then begun: as soon as the Zeiss instrument on the abutments began to indicate a movement the pre-stress was increased and the scaffold was lowered by a further amount, and so on alternately until the calculated amount of pre-stress had been obtained in the

[4] "La technique des travaux", 1934, pages 85 following.

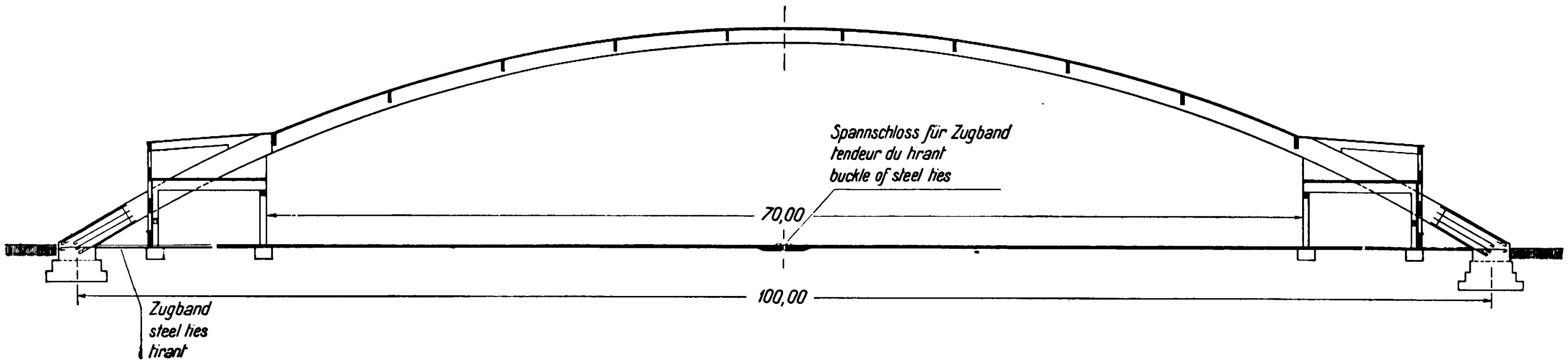

Fig. 5.

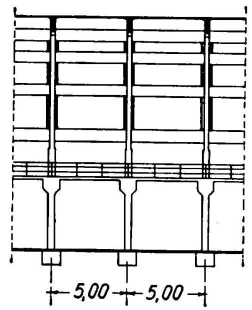

Fig. 6.

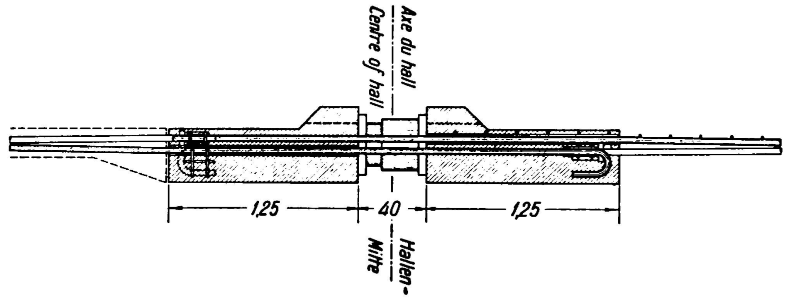

Fig. 7.

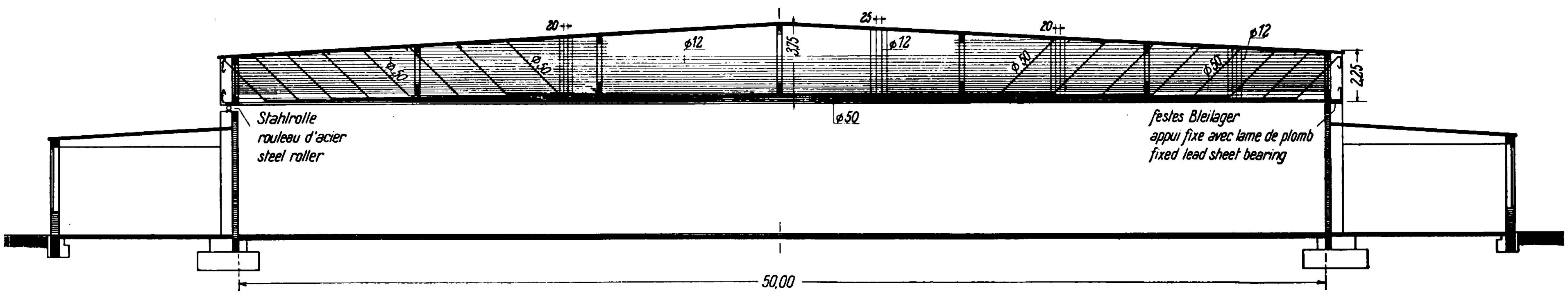

Fig. 8.

tie-bars and the scaffold had been completely struck. The displacement of the abutments during the operation amounted to less than 1 mm (an amount which was, of course, quite insignificant in a span of 100 m) and the extension of the tie-bars to 58 mm.

The pre-stressing of the tensile reinforcement in a member subject to bending which is to be concreted on the site constitutes a more difficult problem, and in recent years many proposals in this direction have been made. *Freyssinet* in his recent book «Une révolution dans les techniques du béton» describes a reinforced concrete girder which was pre-stressed so as to place all the steel in tension by the amount necessary to take up the thrust.[5] The process is well conceived but takes up a good deal of time, and its economy does not appear to be established. Hitherto it has always been found that such proposals are difficult to carry out by the means available on the site, and it remains to be seen whether the future will bring a change in this respect.

Apart from the constructional devices described above, all necessary precautions must be observed to increase the tensile strength of the concrete and to make full provision against the formation of undesirable cracks. It is important to take all possible steps to secure a concrete of high tensile resistance, such as the choice of suitable cement and aggregates, careful concreting and after-treatment of the concrete.

No such wealth of experimental results is available for the tensile or bending tensile strength of concrete as for its compressive strength. For the present, therefore, we must be content with the broad generalisation that an increase in the tensile strength follows similar laws to that of compressive strength. In other words the stronger the concrete in tension the stronger it will be in compression, though the two strengths will not increase in the same proportion. It is not to be recommended that reinforced concrete structures of long span, which are exposed to bending stresses, should be made from concrete of the usual quality having an average compressive strength of perhaps 150 to 180 kg/cm^2, but if certainty against cracking is to be ensured the concrete must show a compressive strength of 250 to 300 kg/cm^2 or preferably more.

To obtain strengths of this order it is necessary to use cements of high quality (indeed of exceptionally high quality), giving the maximum possible tensile strength. They must also be so chosen as to cause a minimum amount of shrinkage, at any rate initially. Every technician knows that the liability of cement to shrinkage is very variable; there are some cements which give good results in this respect and others which have been found to shrink excessively. Unfortunately the use of this knowledge must remain a matter of hit and miss until more perfect methods of testing for shrinkage are developed and until cement manufacturers can be called upon to supply their customers with information not only of the fineness and as to the compressive strength that may be anticipated but also as to the shrinkage properties of their product.[6]

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[5] See also Volume 4 of the "Publications" of the I.A.B.St.E. and the "Preliminary Publication" of the Berlin Congress.

[6] In this connection see also the present author's paper „Entwicklungsrichtungen im Eisenbetonbau", Bautechnik, 1936, p. 141.

It should further be remembered that in accordance with the explanations given above the tensile bending stress in the concrete may be combined with the tensile stress arising from shrinkage, even though the latter may be reduced by the plastic deformation of the concrete. The amount of cement should, therefore, be liberally proportioned even at the risk of increasing the shrinkage, for the advantage of the greater strength of high quality cement outweighs the disadvantage of the increased shrinkage which may result from the larger cement content and from the greater fineness of the high quality cement.

The modulus of elasticity of concrete is subject to wide variations, depending mainly on the nature and proportions of the aggregates and binding material, the water content, the methods of working and after-treatment followed, as well as on the position and magnitude of the stress. Some detailed numerical data on this matter have been published by *Hummel*,[7] who obtained the following figures for the elongation due to tensile bending of concretes prepared from different aggregates with an admixture of 350 kg of high quality cement per m^3 and a slightly plastic consistency:

Aggregate	σ_{bz} kg/cm²	Breaking values		Compressive strength
		Specific elongation $1 \cdot 10^{-4}$	E kg/cm²	kg/cm²
Red quartz porphyry	48	2.94	163 000	479
Quarzite	49	2.89	169 000	483
Grey stone chippings	50	2.66	188 000	485
Broken stone	44	1.98	222 000	488
Basalt chippings	48	1.93	249 000	555

With the same bending tensile strength of $\sigma_{bz} = 48$ kg/cm² the specific elongation at fracture of concrete made with red quartz porphyry is 2.94×10^{-4} and that of concrete made with basalt chippings only 1.93×10^{-4}. It is understandable that concrete with a larger value of ε and a smaller value of E may be less susceptible to cracking than a concrete with a smaller ε and a larger E, and it is important that the implications of these experiments should be applied in the selection of aggregates.

The arrangement and sections of the reinforcing bars also are important from the point of view of crack formation. The bars should be placed as far as possible apart, though this rule is difficult to follow in the case of wide span structures wherein the section of concrete has to be reduced to a minimum and where, therefore, a smaller number of larger bars must be used. In Germany it is customary to use round bars almost exclusively. A few years ago the Isteg steel was placed on the market, and it is possible that bars of this kind, or bars with protrusions, may contribute to a finer distribution of the cracks even if they do not increase the carrying capacity.

In roof trusses of long span it will frequently not be possible to dispense with construction joints, having regard to the working conditions in concreting and

[7] „Beeinflussung der Betonelastizität" by Dr. Ing. *A. Hummel*, Zement, 1935, pages 665 following.

to the availability of suitable plant, the erection of scaffolding, etc. So far as possible such construction joints should be placed only in the compression zone of the concrete and should run at right angles to the direction of pressure. If it is impracticable to avoid their presence in the tension zone they will entail a reduction in the available concrete section and it is essential that this should be made up by "knitting" the concrete together across the gap by a large number of additional bars.

The concrete increases in strength during the first few weeks and months, so that the later it receives its full stress the better its quality and the greater the guarantee against the formation of cracks. It is important, in this connection, that the concrete should be protected from premature drying immediately after it has set, by covering it over and keeping it damp. A concrete which is permanently kept damp shrinks only a small amount or may even swell, and by this means the additional stresses due to shrinkage may be avoided during the period in question and for some time afterwards, the final shrinkage stresses being

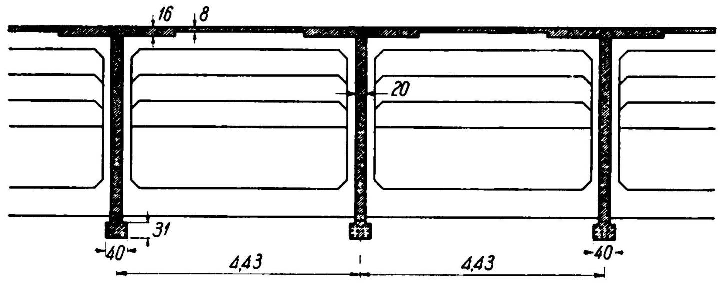

Fig. 9.

thus considerably reduced. It is also important to delay the removal of shuttering as long as possible, or if this cannot be done to provide temporary supports capable of carrying the whole weight of the structure.

In conclusion a description may be given of one more roof girder of large span which was built with due regard to the considerations mentioned above and has proved entirely successful.

The roof system shown in Figs. 8 and 9 covers a clear space of 50.0 m and consists of statically determinate girders on two supports with spans of 50.80 m. So far as the author knows, this is the first occasion that a structure of the type in question has been employed over so great a span. The spacing of the girder is 4.43 m and the roof covering is a reinforced concrete slab 8 cm thick. The latter is thickened to 16 cm over the girders in order to provide an adequate compression boom at these places. The cross bracing of the girders is provided by framed purlins at 8.40 m centres. The depth of the girder at the centre is 3.75 m and tapers to give a suitable fall to the roof being 2.25 m at the end. The thickness of the web is only 20 cm and in order to accomodate the tensile reinforcement the lower edge is thickened to 40 cm with a height of 31 cm. The bearing at the fixed end is on lead and that at the movable end on steel rollers.

The reinforcement of a girder is represented in Fig. 8 the tension bars being of 50 mm diameter and stressed up to a maximum of 1200 kg/cm^2. In addition to these and the vertical stirrups, which are 12 mm in diameter, longitudinal reinforcing bars also of 12 mm diameter were provided in both the faces of the web with a view to preventing the formation of cracks. In the compression zone at the middle of the span it was possible to omit these, but at the two ends they were provided over the whole depth in order to co-operate with the bent-up longitudinal bars and the vertical stirrups in carrying the shear forces. In girders of this depth, so limited in thickness, special care is needed to ensure neatness and accuracy in arranging the steel according to plan.

The cement used was a high grade type supplied by the Thysen works under the name of Novo, and the aggregate was made up of sand and quartz porphyry chippings. Test cubes made during the concreting operations showed compressive strengths of over 400 kg/cm^2 at 28 days. Special precautions were necessary in pouring the concrete in view of the large number of reinforcing bars and also because the Novo cement began to harden after a few hours in the warm summer temperature. These difficulties, however, were entirely over-

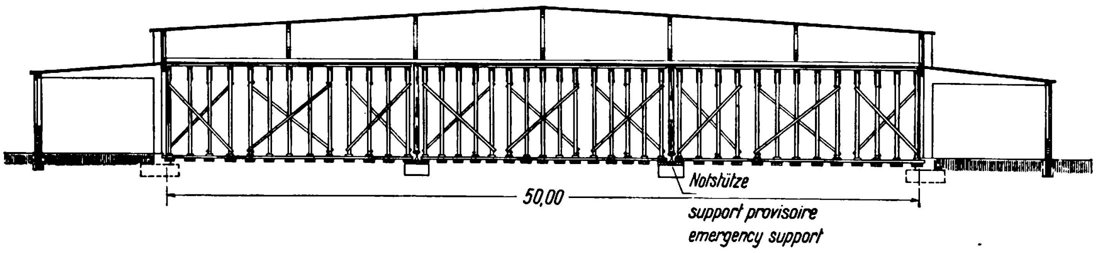

Fig. 10.

come with the result that all of the girders were concreted in such a way that on removal of the shuttering not a single cavity was found, nor did any bad place need to be patched.

To allow of the concrete being kept properly damp during the first few weeks provision was made for sprinkling it from water pipes close to either side of each girder, the pipes having small holes at 20 cm intervals through which the water was squirted against the sides of the girders. It was possible in this way to keep the concrete uniformly and continuously damp for about six weeks, after which the sprinkling had to be discontinued on account of the progress of other operations in the construction of the hall.

The shuttering for one of the girders is shown in Fig. 10, consisting of a simple framing of round scaffold poles arranged to transmit the load to the ground in the most direct way, but in view of the necessity for completing the floor as early as possible it was not feasible to allow these supports to remain in position as long as might have been wished. In order, therefore, to permit the shuttering to be struck from the girders as late as possible, temporary supports were provided in advance under the third points, each consisting of two heavy round timbers resting on screw jacks at the foot and capable of carrying the whole dead load of the girder. These temporary supports did not interfere with further building work of the hall — particularly of the floor — and they did not, therefore, have to be removed until about six weeks after concreting, whereas the other supports had had to be taken away after about three weeks.

The total deflection of the girders on the removal of the temporary supports amounted to about 5 cm at the middle, representing approximately $\frac{1}{1000}$ of the span, and in the course of years this deflection will increase to some extent on account of plasticity. To provide for this increase, and particularly with a view to good appearance, the lower boom of the girder has been given a camber of 24 cm on the centre line beforehand. All the work was carried out to perfection and none of the girders showed cracks of any kind.

Finally, it is proper to lay every emphasis on the fact that cracks which result merely from the tensile strength of the concrete having been exceeded, and which do not imply any structural defect such as insufficient steel reinforcement, are not dangerous to the work except in so far as they may allow rusting of the tensile reinforcing bars under load. Apart from this contingency such cracks have no significance, and in order to prevent the rusting of the bars it is sufficient that the cracks should be pressed out or covered over with a spray of cement grout, an elastic paint or an elastic compound after three or four years have elapsed and the shrinkage is approaching its final amount.