Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 2 (1936)

Artikel: Elimination of bending tensile stresses in R.C. bridges

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DOI: https://doi.org/10.5169/seals-3197

IVb 2

Elimination of Bending Tensile Stresses in R. C. Bridges[1].

Ausschaltung der Biegezugspannungen bei Balken- und Stabbogenbrücken.

Compensation des efforts de traction engendrés par la flexion.

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I. Historical Survey.

Since the beginning of the present century numerous scientists have devoted their attention to the subject of pre-stressing the reinforcing bars in reinforced concrete. The object of this idea was to attain

a) a reduction or the entire elimination of tensile stresses in concrete and therewith a reduction or the elimination of hair cracks detrimental to the life of the concrete. A reinforced concrete body subjected to compression only has, like natural stone, an almost unlimited life. This fact is proved to us by the old Roman constructions, in which both the stone itself and the mortar used as binder have withstood all the influence of time and weather up to the present day.

b) An increase in the permissible stresses. As is well known, the latter are very much less in the reinforcement of reinforced concrete than in all-steel constructions for the reason that reinforced concrete is a compound body in which the stresses have first to be transmitted to the steel by grip and shear. An increase of the stresses in the steel admissible for the combination of steel and concrete up to the limit of admissible stressing for all-steel constructions is not practicable owing to the excessive grip and also because of the numerous and rather large hair cracks that would thereby be caused. On the contrary, on this account the experienced reinforced concrete designer endeavours whenever possible to carry out his constructions in St. 37 instead of St. 52, the lesser stressing to which the former is subjected giving much greater safety against hair cracks.

The problem of a practical and economically satisfactory solution of this question is becoming increasingly important as the spans of our bridges and halls grow steadily greater, for the tensile stresses in the concrete become considerably higher in consequence of the rubbing of the tensile reinforcement. By pre-stressing the reinforcement compressive pre-stresses can be imparted

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[1] Howe and Toreign Patents applied for.

to the concrete so that the bending tensile stresses are compensated and none or only very few remain.

Koenen, one of the earliest reinforced concrete specialists, was the first to recognise the value of pre-stressing. He carried out a number of tests along these lines with reinforced concrete beams and realised that the pre-stressed reinforcement bars embedded in the concrete again lose a considerable portion of the pre-stresses imparted to them owing to shrinkage of the concrete and the contractions thereby induced. The difficulty arising from this fact caused the principle of pre-stressing to be abandoned as impracticable for quite a considerable time.

In another domain of reinforced concrete, hewever — the two-hinged arch with tie, which is extensively used both in bridge and hall building — pre-stressing has been adopted to a very considerable extent. The author's own process was first put into practice in 1928, in the construction of the 68 m span Saale Bridge at Alsleben by Messrs. Dyckerhoff & Widmann A. G.[2]. As is well known, very considerable additional bending moments are set up in these two-hinged arches with ties on account of the contraction of the arch and the elongation of the tie. In respect to the contraction of the arch we shall first examine that due to compressive forces and shrinkage. The plastic influence of creep will be dealt with later on. By pre-stressing the tie with the aid of hydraulic jacks it is possible to produce similarity between the axis of the pre-stressed system, loaded by its own dead weight, and the projected line of the system. In other words, that the span-rise ratio l/f is identical in both systems, so that the additional bending moments due to deformations are eliminated. The tie must be shortened in just the same manner as the arch, i. e. we must stretch the tie not only to the extent of the elongation it has undergone owing to tensile forces, but also to the extent that the arch has shortened. To enable us to do this, a joint of the width of the contraction of the arch must be left open in the reinforced concrete decking, and only closed when the tie is stretched. By applying this pre-stressing process and removing the tie from the cross section of the reinforced concrete decking, not only the additional bending moments of the arch, but also the tensile stresses of the reinforced concrete decking can be eliminated. After stretching the anchors can then be concreted, so that they now form, in conjunction with the reinforced concrete decking, a common tie in respect to horizontal thrust caused by live load. Of course this horizontal thrust and live load produce tensile stresses in the reinforced concrete decking, but these are slight and can be eliminated as well by increasing the straining force of the anchors on closing the joint to such an extent that the compresssive pre-stresses there by produced in the reinforced concrete decking compensate the subsequent tensile stresses due to the horizontal thrust caused by traffic.

The process described has been widely adopted in recent years in the construction of arched aircraft hangars, particularly those with spans of over 100 m.

[2] *Fr. Dischinger:* „Beseitigung der zusätzlichen Biegungsmomente im Zweiggelenkbogen mit Zugband" (Eliminating additional bending moments in double hinged arch with tie member). Volume I, "Publiactions" of the I. A. B. St. E. — Do. „Beton und Eisen", 1932. — Do. "Science et Industrie", 1932.

For this purpose the ties, constructed of thick round steel bars, were placed in a trench and after straining were concreted to protect them against rusting, The ties must be strained during decentering.

As the pre-stressed system must be similar to the original when the additional bending moments arising from contraction of the arch under compression and from the elongation of the tie are entirely eliminated, and as the contraction of the arch is given by the ratio of the stress to the modulus of elasticity, only very slight lowering at the spindles is necessary when decentering. For instance, a tensile stress of the arch amounting to $\sigma = 60$ kg/cm² and a modulus of elasticity of the concrete of $E_b = 210000$ kg/cm² only involve a lowering of the crown of 1/3500. Owin to the fact that decentering is effected solely by stretching and pre-stressing the tie, and only to a very insignificant degree by lowering the falsework, the considerable bending stresses set up by the usual procedure of decentering the arch by means of spindles are almost totally eliminated.

The lowering of the crown by 1/3500 mentioned above, naturally refers only to elastic lowering during decentering. In this connection the factor of preep must also be considered. Creeping sets in immediately after decentering, and we shall discuss its influence in detail at a later stage. It, too, causes substantial contractions in the axis of the arch and consequently lowering of the crown and deformation of the system. The effects of these contractions of the arch on the flow of forces are, however, fundamentally different from those of shrinkage or of the elastic contractions of the arch. To my knowledge Dr. *Mehmel* was the first to make mention of this fact when the effects of creep were being discussed in connection with the large-span aircraft hangars mentioned above. Creeping is a purely plastic process. In the case of an arch formed exactly on the funicular curve, when the influences of the tie elongation and the elastic contraction of the arch have been eliminated, the latter will also represent a funicular curve after plastic deformation. Plastic deformation thus does not set up any bending moments, and the system acts as if it had been concreted in this form. The lowering of the crown caused by creeping is now only percetible in a slight increase of arch thrust, corresponding to the reduced rise. From this we see that creeping exerts an influence quite different from that of elastic contraction of the arch. The latter produced a reduction of the rise and thus additional bending moments in the arch, which must be eliminated by stretching the tie, while creeping causes a slight increase of arch thrust without inducing additional bending moments. When stretching the tie, therefore, we do not need to make allowance for creep, nor must we eliminate its influence by stretching the tie a single time, as this one operation would be of an elastic nature and would consequently set up bending moments. We would further mention the fact that creeping can induce other and quite different influences if the arch is not formed in the funicular line. The unequal distribution of compressive stresses over the cross section thus caused would induce various degrees of creeping for the various fibres, thus setting up bending of the system and in consequence additional bending moments which are difficult to deduce.

The above process, described in its application to an arch bridge constituting the special case of a strut frame, can of course be adopted for any other form of strut frame with curved compression boom and straight tension boom. In this connection the further development of the idea has been greatly facilitated by *U. Finsterwalder* who, when projects were invited for the Drei-Rosen-Brücke, a bridge in Basle (design Dyckerhoff & Widmann A.G.), Proposed a pre-stressed slab bridge. His project, as will be seen in Fig. 1, comprised arches cantilevering on both sides of the piers, which were encastered in the bedrock. The arch thrust of these overhung arches was to be taken up by pre-stressea straight-line cables connecting crown to crown. These high-tensile steel cables were to be pre-stressed by means of hydraulic jacks while the flasework was being released. This form of bridge asproposed by *U. Finsterwalder* is also a strut frame system with straight tension boom; the cantilever arch can be regarded as the inversion of a two-hinge arch bridge. The panels stiffening the arch extend to the level of the decking, i. e. of the tie, so that these strut frame

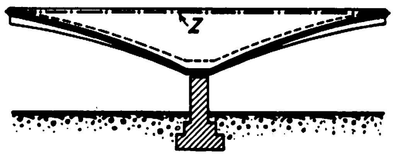

Fig. 1.

systems may also be regarded as beams cantilevering on both sides of the supports. Besides this project for the Drei-Rosen Bridge in Basle, *U. Finsterwalder* has also completed two other designs which have not yet become known in engineering circles, but which propagate the application of the above principle of pre-stressed strut frames to girder bridges.

II. Pre-stressed Girder Bridges.

Although in pre-stressed arches plastic deformations of the concrete due to creeping do not affect the pre-stressing, such is by no means the case with regard to girders. Creeping causes the mass of concrete to contract longitudinally just as do shrinking and compressive stresses, so that the stress drop in the pre-stressed bars is considerably increased by the effect of creep. For this reason *Freyssinet* proposed that for pre-stressing bars special steels of very high yield limits with correspondingly high stressing should be employed. In this manner he succeeds in greatly reducing the drop in stressing in relation to the original pre-stressing. *Freyssinet,* in the same manner as *Koenen,* uses straight bars for pre-stressing and the shuttering as abutments; the concreting is only done after the bars have been stressed. When the concrete has hardened the tensile force is transferred from the formwork to the hardened concrete. *Freyssinet* further provides for the pre-stressing of the stirrups, thus obtaining a concrete that is under compressive pre-stressing acting in all directions and can easily take up the bending stresses and shear occurring in the system. *Freyssinet's* process is certainly of extreme importance for factory-made products, especially in conjunction with his incidental proposals for improving the concrete by reducing creep[3].

[3] *Freyssinet:* Une revolution dans les techniques de Beton, Paris Libraire de l'enseignement technique Leon, Eyrelles, Editeur, 1936.

In large-span bridges and halls I find the application of the scheme involves certain difficulties — on the one hand because the pre-stressing forces are then so great that extremely strong formwork (of steel, for instance) would be necessary to act as abutment; and on the other hand, because straight pre-stressing causes equal moments to be exerted on the beams for each cross section, while the dead weight moments flow in curves. Difficulties are also to be expected at first when steels of very high yield limits are employed.

For this reason the author has worked along another line as regards the pre-stressing of large-span bridges, elaborating the ideas of the German patent DRP 535 440. The object of his endeavours is to effect pre-stressing even with normal St. 52 with elimination of the drop in stressing due to creep and shrinkage. In this manner freely supported reinforced concrete girder bridges of up to 80 m span, and through beams of up to 150 m span can be constructed in accordance with present-day stressing regulations and eliminating bending tensile stresses.

Before proceeding to describe these pre-stressed constructions, I shall touch upon the interesting subject of the sizes of spans achieved up to the present time. Today the maximum practical span for freely supported reinforced concrete girder bridges is about 30 m. Above this figure the dead weight of the structure increases extremely rapidly with the span, firstly because of the increasing depth of the beams, and secondly owing to the greater thickness of the web necessary to accomodate the large number of reinforcement bars. This rapid increase in dead weight quickly brings the structure to the limit above which reinforced concrete can no longer compete with steel, owing to the relative lightness of the latter in bridge construction. Owing to the greatly decreasing bending moments occuring in through beams or Gerber bridges, substantially greater spans can be attained with these types. The longest through beam in existence at the present time, the bridge over the Rio de Peixe in Brazil, has a span of 68 m. From a constructional and statical point of view it is of course possible to attain spans of up to 100 m. In this connection I would mention Prof. *Mörsch*'s[4] project for the Drei-Rosen Bridge in Basle, with spans of 56 — 106 — 56 m and my own unpublished project, submitted by the firms Dyckerhoff & Widmann, Wayss & Freytag, and Christiani & Nielsen, for a road bridge over the Süderelbe at Hamburg with spans of 64.5 — 103 — 102 — 103 — 64.5 m. The costs of these large-span reinforced concrete girder bridges were not very much higher than those of the steel bridges, but still the difference was sufficient to turn the scales in favour of steel.

Reinforced concrete girder bridges can only be improved in a competitive sense by pre-stressing; i. e. by the possibility of erecting pre-stressed structures whose dead weight is greatly reduced. The pre-stressing itself must meet the following demands:

a) The drop in the stressing of the above-mentioned anchors in consequence of the subsequent contraction of the concrete owing to shrinking and creep must be eliminated as completely as possible.

b) The possibility must be created of measuring the stresses in these pre-

[4] See „Beton und Eisen“, 1931, Issue 13, p. 14.

stressed anchors at any time and adjusting them if necessary by means of a suitable stressing device.

c) It must be possible to carry out the pre-stressing operation with very simple resources and, with a view to attaining cheapness and simple, rapid construction, with round bars.

d) The tensile stresses in the concrete must, if possible, be entirely eliminated or at least reduced to such an extent that hair cracks cannot occur.

e) The pre-stressed bars must be of such a shape that they transmit the greater portion of the dead weight to the supports, thus relieving the concrete of its high shear and enabling web thicknesses of 30—40 cm to be used even for the largest spans.

f) Pre-stressing must be effected in such a manner that the cross sections of the reinforced concrete are utilised as much as possible on both edges and right up to the admissible maximum degree of stressing for dead and live loads.

g) A still more complete solution than that given in f) is obtained if we succeed in pre-stressing bridges in such a manner that only centric compression forces are exerted in the beams by dead weight, so that the girder system functions as a centrically loaded member. This would have the advantage that under dead weight only elastic but no plastic contractions occur in the beams, so that under dead weight such a bridge would not undergo deflection. Deflections would thus only occur under live load, and then only of an elastic nature, since the live loads would not be permanent.

To fulfil the requirements a) and b) we must remove the main reinforcing bars from the cross section of the concrete, as in the case of German patent DRP 535440, for only by doing so are we in a position to eliminate the drop in stressing to a degree corresponding to the tension or thrust set up, these are eliminated by measuring the stressing in the anchoring and adjusting same.

To fulfil the requirement c), pre-stressing must be effected with thick round bars and not with riveted steel sections. Unfortunately, these round bars are not available in the necessary lengths, but in resistance welding we possess today an absolutely reliable means of manufacturing, independently and at site, any desired lengths of round bars by welding shorter bars together. Resistance welding is certainly the most reliable method, and if, after welding, a fair length of the round bar on each side of the joint is made red hot, the joint can be subjected to a certain amount of upsetting and the cross section of the weld thereby enlarged. In this way, too, any self-stresses that may have been set up by the heavy drop in temperature can be obviated at the same time.

To fulfil the requirements from d) to g), the anchors must be constructed as hanging trusses. The form of these trusses must be suited as nearly as possible to the line of dead weight moments, i. e. the distances of the suspension boom from the neutral axis must be proportional to the magnitudes of the dead weight moments. If at the same time the cross section of the reinforced concrete is cleverly designed we can succeed in obtaining a construction in which, under live load corresponding to the requirement f), similar extreme fibre stresses are set up without bending tensile stresses, or in which, in accordance with the requirement g), the dead weight moments and the shear forces caused by dead weight are almost completely eliminated, so that the pre-stressed beam acts under dead weight loading as a centrically loaded column. Let us now examine two girder bridges of very large span in which these ideas have been put into practice.

1) *Gerber girder bridge with spans of 98.5 — 110 — 125 — 110 — 98.5.*

The bridge is shown in Fig. 1. All the suspended girders have a span of 70.0 m. The cantilevering girders, however, have a span of 110 m with 27.5 m cantilevers, giving a central bay of 125 m. The depth of the girders is assumed to be 5 m throughout. The depth-span ratio of the girders is thus 1/25 in the central bay and 1/22 for the cantilevering girders. Even in comparison with steel bridges these are extremely small girder depths and in spite of their great slenderness the deflections undergone by them under live loads keep well within permissible limits. The structure was calculated as a first-class bridge with an 8.5 m wide decking and two footpaths each 2.0 m wide. The dead weigth of the suspension girders, including the necessary cross beams and the weight of the pre-stressing bars removed from the section of the concrete, amounts to 29.65 tons, while the cantilevering girder, which also had to be provited with a lower compression flange on account of the varying live load, comes to 35.10 tons.

The unvarying live load on the suspension girder totals, when calculated according to German regulations, $p_1 = 8.5\ (0.525 - 0.70) + 40.5 = 5.87$ t/m. To these must be added the points loads, composed of a steam roller of 24 tons and two lorries of 12 tons. From this, deducting the unvarying live load, is obtained a substitute load of $P_1 = 27.5$ tons.

For the cantilevering girder with 110 m span the corresponding values came to $p_2 = 5.52$ t/m, $P_2 = 29.7$ tons.

For the cantilever arms, for which the total span of 125 m is taken according to German loading prescriptions, the corresponding values are $p_3 = 5.4$ t/m and $P_3 = 30.0$ tons.

In order to simplify calculation we shall assume — against regulations — that when determining the negative moments of the cantilevering girder the point load P can act at both ends simultaneously. The dead weight and live load moments occurring for this loading are listed in Table 1 and entered in Fig. 3; the dead weight moments are given on the right and the live load moments and maximum moments M_{max} and M_{min} on the left.

Table 1.

	M_g	$+M_p$	$-M_p$	M_{max}	M_{min}
Middle Suspension Girder .	+18200	+4080	—	+18280	+18200 tm
Middle Cantilevering Girder	+ 9600	+9160	—8075	+18760	+ 1525 tm
Above Support	—45200	—	—8075	—45200	—53275 tm

Figs. 4 and 5 show the suspension and the cantilevering girders with the anchors removed from the concrete cross section and placed between the webs.

Fig. 2

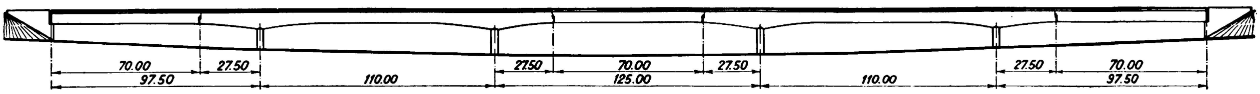

Fig. 3

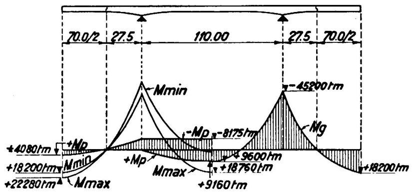

Fig. 4

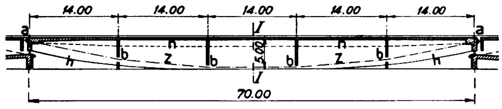

Fig. 7

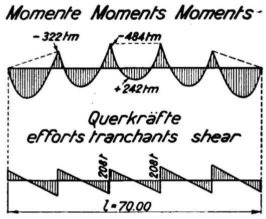

Fig. 5

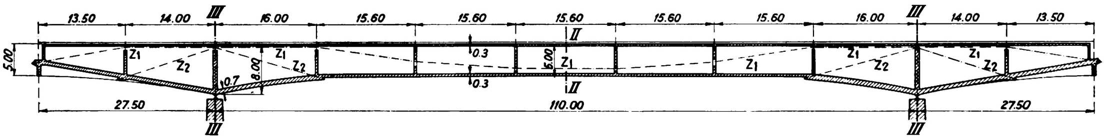

Fig. 6c III-III

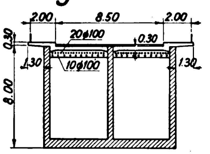

Fig. 6a I-I

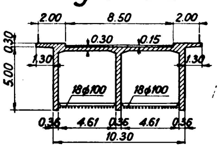

Fig. 6b II-II

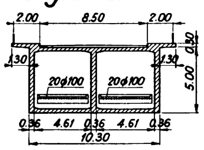

Fig. 10

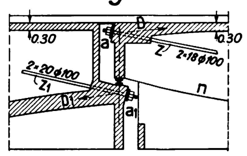

Fig. 2. Pre-stressed continuous hinged girder bridge.
Fig. 3. Bending moments due to dead weight and live loads.
Fig. 4. Longitudinal section of suspended girder.
Fig. 5. Longitudinal section of girder with cantilever arms.

Fig. 6. 6a. Cross section I—I through suspended girder.
6b. Cross section II—II through cantilever girder.
6c. Cross section III—III through cantilever girder.
Fig. 7. The moments and shear forces due to dead weight after pre-stressing.
Fig. 10. Detail of hinge.

Figs. 6a — 6c illustrate the shape of cross section in the middle of the suspension girder, in the middle of the cantilevering girder, and in the middle of the support. To begin with, we shall now discuss the effect of pre-stressing on the dead weight loading.

a) *Internal forces due to dead weight loading.*

The suspension boom of the suspended girder is composed of 36 bars of 100 mm $\varnothing = 2820\ \text{cm}^2$. The bars of the suspension boom are anchored at the ends of the suspended girder (a) stiffened decking slab. The ordinates of the suspension boom were so arranged that the total dead weight is transmitted by the boom to the supports, and for this purpose the reinforced concrete beam is supported on roller bearings or rockers at the bending-points of the suspension boom, so that longitudinal displacement can take place. Consequently the reinforced concrete beam is no longer continuously self-supporting for dead weight over a length of 70 m, but only over the distance between two cross beams. The latter are arranged at intervals of 14 m and in the form of through beams over five bays. In this manner the dead weight moments are reduced to about 1/40 and the shear forces drop to about one-fifth (see Fig. 7). The stretching of the suspension boom is effected by hydraulic jacks. The latter can be applied either at the anchor positions (a) to stretch the suspension bars longitudinally — for which reason, as already stated, they must be supported in a manner enabling them to be displaced longitudinally — or the jacks may also be applied at the bending-points of the suspension boom (b). In the latter case the stretching process is then effected downwards by the jacks to an extent corresponding to the elongation of the anchors caused thereby. This latter manner of stretching is only practicable as a general rule for suspended girders, i. e. for girders supported freely. Longitudinal stretching, however, is much more suitable for cantilevering girders. Fig. 8 shows the points of support, allowing longitudinal displacement, of the hanging truss in relation to the cross beams, Fig. 8a illustrating a roller bearing, Fig. 8b a rocker bearing and Fig. 8c a rocker bearing with interchangeable and longitudinally adjustable rocker, provided in case stretching is effected transversely to the direction of the tensile bars instead of longitudinally, by increasing the distances between the cross beams. The bends of the suspension bars are pulled down by the anchors (c) in Fig. 8c and also by means of hydraulic jacks, the latters being supported on the reinforced concrete beam with the assistance of auxiliary I-sections. In Fig. 9 the stretching device for the round anchors is shown in the form of hydraulic ring jacks.

The suspension bars are stressed while the bridge is being decentered. As a matter of fact the process is the reverse, for the straining of the anchors transfers the dead weight from the falsework to the hanging truss, unloading the falsework. Lowering the falsework by means of spindles is only necessary to the extent to which the wood has expanded on being unloaded. To transmit the dead weigth of the bridge to the suspension boom, the anchors must be strained up to 1900 kg/cm², giving for the reinforced concrete beam compression pre-stressing of 5350 tons. As the suspension boom is anchored in the decking slab, which has been strengthened for this purpose, the neutral axis

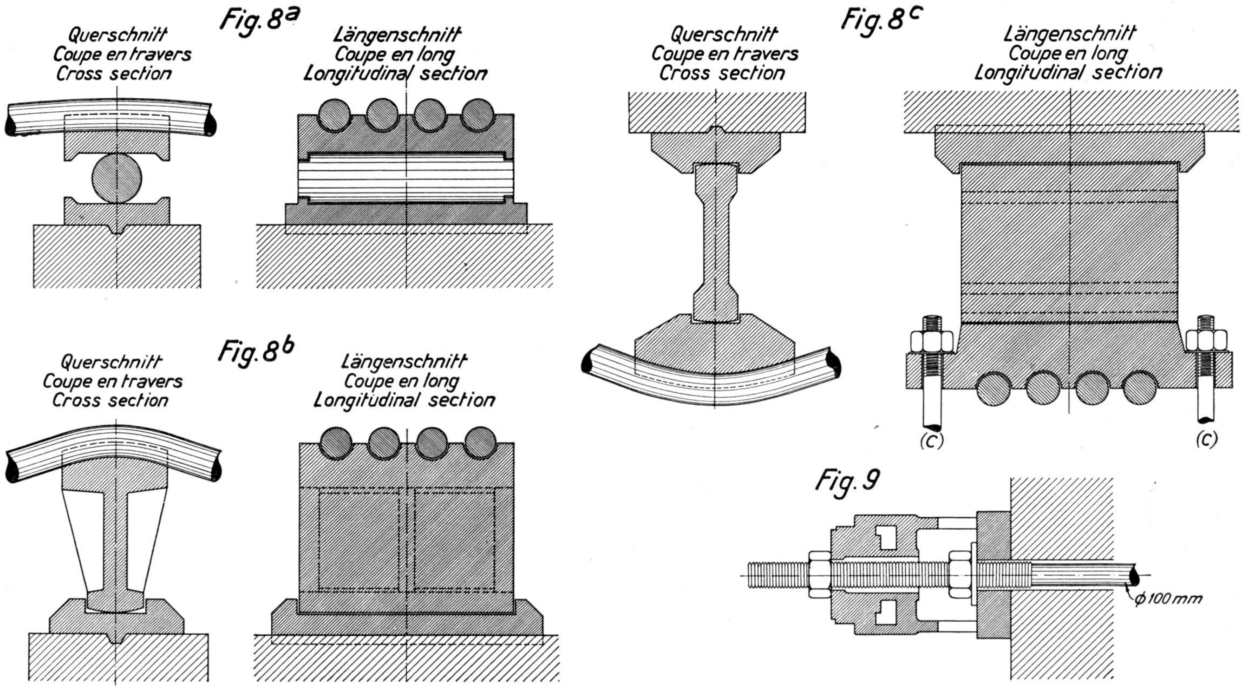

Fig. 8. 8a. Roller bearing for the support of ties.
8b. Rocker bearing for the support of ties.
8c. Rocker bearing for ties with removable rocker.
Fig. 9. Pre-stressing release arrangement.

must be very far up if a comparatively great degree of eccentricity of the compressive force is to be avoided. To do so, the lower part of the central rib is cut oïf, as shown in Fig. 4, and the lower portions of the outer webs prevented from contributing to the static action by means of transverse joints. The static action of the web surfaces below the line (h) in Fig. 4 is eliminated. But even if this should not be done, a portion of the webs would not be operative, because according to the strict theory of the disc, the law of proportionality cannot apply in the vicinity of the point of attack of the compressive force. Clearer static conditions, however, may be attained by eliminating the activity of these lower portions of the web. The suspended girder now assumes the static form of a Pauli girder. The advantage of this is that the beam under dead weight is only loaded by centric compressive forces. The position of the neutral axis (n) is shown in Fig. 4. From it we see that the distances between the suspension boom (Z) and the neutral axis are proportional to the dead weight moments, and consequently only centric compression forces act on the beam, as has already been stated. In order to obtain a greater lever arm of internal forces for the section of the support, with its very great negative dead weight moments, the co-operation of the decking slab was eliminated with the assistance of transverse joints. Thus the decking slab is not statically active; the fact that these portions of the cross section are not shaded (Fig. 5c) denotes this.

Fig. 10 shows the point of hinging. The dead weight of the suspended girder is transmitted to the supports of the cantilevering girder by the suspension boom (Z). This support reaction now resolves itself in its turn into a compressive and a tensile force. The compressive force is taken up by the 60 cm thick concrete compression slab of the cantilevering girder, the tensile force by the hanging truss (Z_1). The suspension boom (Z_1) consists of 40 bars of 100 mm $\varnothing = 3140$ cm² and extends over the whole length of the cantilevering girder, i. e. $110 + 2 \cdot 27.5 = 165$ m. But this boom alone is not strong enough to take up at its support the whole dead weight moment of 45200 t/m. For this reason it has been strengthened over a length of 30 m, at the places where the greatest negative moments occur, by the addition of an auxiliary boom (Z_2) consisting of 20 bars of 100 mm $\varnothing = 1570$ cm². The main boom (Z_1) only transmits the loads to the cross beams adjacent to the support, and these pass the loads on to the auxiliary boom (Z_2), which in turn transfers them to the main point of support. As the auxiliary boom is only half as thick as the main boom, it has been set at a greater incline, thus enabling it to take up the vertical loads of the boom (Z_1).

b) *The internal forces due to live load.*

As the pre-stressed reinforced concrete beam, exempt from bending tensile stresses, only undergoes deflections under live load, it is far more rigid than the hanging truss. Consequently a shear of no more than 411 tons is set up under live load in the boom (Z) of the suspended girder. Out of the total live load moment of 4080 t/m, therefore, only 1400 t/m is carried by hanging truss formed by the reinforced concrete beam and the tension boom, the remaining part, 2680 t/m, being carried by the beam alone. Thus the beam supports 65.5 % and the truss 34.5 %. In the cantilevering girders the portion carried by the truss is several times smaller. This is due to the fact that the truss cuts the neutral axis several times and is consequently only affected by the deflections of the beam to a very small extent. From these elucidations we see that on the whole the live loads must be transmitted by the pre-stressed beam. Reinforced concrete beams are very well suited to this purpose, since in consequence of the great compressive pre-stressing they are only able to take over the live load moments without undergoing bending tensile stresses by re-arrangement of the compressive pre-stresses. As the positive and negative moments in the middle of the bay are approximately of equal magnitude in the cantilevering girder, a lower compression slab had to be provided in this case also; this slab is of the same thickness as the decking slab. Thus the problem of reinforced concrete beam bridges without bending tensile stresses is solved by pre-stressing effected with hanging trusses.

c) *The effects of temperature changes.*

The influence of various changes of temperature on the reinforced concrete beam and hanging truss causes additional bending moments owing to the relative elongation or contraction of the hanging truss in relation to the beam. As the booms lie inside the beams, these temperature differences are very slight. German regulations provide for a temperature difference of $\pm 5^0$ C. The resultant influences on the stresses in concrete and steel are given in the following Table 2.

d) *The effects of creep and shrinkage.*

Pre-stressing of the hanging truss to enable it to take over the dead weight loads may produce compressive pre-stresses of over 50 kg/cm^2 in the reinforced concrete beams. Under the influence of these compressive pre-stresses the concrete undergoes an elastic contraction which is, however, without effect on the pre-stressing, since we could make allowance for them by shortening the anchors accordingly. On the other hand, special measures must be taken to counteract the contractions of the reinforced concrete beam caused by shrinkage and creep, since these influences only become apparent after decentering, i. e. after the anchors have been strained, and thus effect a considerable drop in stressing in the pre-stressed steel. These contractions of the concrete extend over a comparatively long period, particularly if poor mixtures — such with low fineness modulus of the aggregate — have been employed. The amount of creep is also to a great extent dependent upon the age of the concrete when first subjected to stressing, i. e. when the structure is decentered, and on the relative humidity of the air. This drop in stressing causes a partial re-arrangement of the dead weight loads of the hanging truss in its relation to the reinforced concrete beam. This detrimental effect can of course be reduced by keeping the creep of the concrete as low as possible. This can be done by employing rich mixtures, a good granular composition of the aggregate, by allowing the concrete to harden as long as possible before removing the formwork, by using high-quality cements and by thorough watering of the concrete (cf. also *Freyssinet*'s suggestions under 2) over a long period.

In spite of these precautions, however, there will always be sufficient shrinkage and creep to cause a very substantial drop in the stressing of the hanging trusses. Creeping and shrinking of, for instance, $40 \cdot 10^{-5}$ after decentering produces a drop in stress of 430 kg/cm^2 in the anchoring of the suspension girders and of about 700 kg/cm^2 in that of the cantilevering girders. For pre-stressing of 2000 kg/cm^2 in the hanging trusses this would mean that between one-third and one-fifth of the total dead weigth loads would be transmitted from the hanging trusses to the reinforced concrete beam. The pre-stressing might now be increased correspondingly when decentering is effected. But this would create opposed moments in the reinforced concrete beams. So these measures do not achieve the desired end. Even if the reinforced concrete beam could withstand such loading, the girder would undergo plastic deformations in consequence of these great moments. The right way is to employ straining devices of such a kind that they can be put into operation again to counteract any contraction that the beam has undergone owing to shrinkage and creep, the object being to obviate the drop in stressing that has already taken place and thus to raise the pre-stressing once more to the height originally calculated. The pre-stressing forces can be measured in various ways. We can measure them

1) with gauged hydraulic jacks,
2) with tensometers, applied directly at anchoring,
3) by measuring the deflection of the free-hanging, cantilevering truss between the points of bending.

As we have provided in the above project, by the form of the hanging truss and the shape of cross section, that only centric compressive forces are set up in the beam by dead weight load, so that the beam does not undergo deflection after decentering, we have still another way of regulating the amount of strain in hanging trusses.

4) As soon as the reinforced concrete beam shows a deflection of, for instance, 1 cm owing to shrinkage and creep, this is the sign of a certain drop in stressing which has a connection with the bending moments in the beam. We can eliminate these deflections by putting the hydraulic jacks in operation once more, now, however, we raise the beam not only as far as the neutral axis but beyond it, till it shows a negative deflection of 1 cm. In the course of time the beam will sink again in consequence of shrinkage and creep, and we repeat this subsequent stressing at ever-increasing intervals until creeping and shrinking have entirely disappeared. The constant variation of the deflection round about the neutral axis, serves the purpose of subjecting the reinforced concrete beam to plastic contraction only, but not to plastic bending, so that from the deflection of the reinforced concrete beam itself we can always deduce the magnitude of the forces in the hanging trusses. At the position of the neutral axis the actual stresses in the steel therefore correspond to those obtained by calculation.

As regards the cantilevering girders, the hydraulic jacks can be allowed to remain at the straining points (a_1) even after the bridge has been put into service, so that the drop in stressing caused subsequently by creep and shrinkage can be eliminated at any time. In the suspension girders, however, the straining points (a) are too high, and for this reason both initial and subsequent straining must in this case be effected at the bending-points of the hanging truss (b) by enlarging the distances with regard to the cross beams. This is effected by lengthening the interchangeable rocker in Fig. 8c. In the following Example 2 of a through girder, another method of re-stressing is shown, enabling re-stressing to be effected by longitudinal stretching without its being necessary to block traffic. In Table 2 appended the stresses resulting in the concrete have been given for cross sections I, II and III, in which the greatest bending moments occur.

Table 2.	Section I		Section II		Section III	
	σ_o	σ_u	σ_o	σ_u	σ_o	σ_u
Dead weight	— 51,3	— 51,0	— 46,7	— 46,7	— 53,5	— 53,5
Travic $+ M_p$	— 19,0	+ 26,7	— 42,3	+ 43,6	— 2,4	+ 0,3
$- M_p$	—	—	+ 37,2	— 39,8	+ 36,4	— 16,3
Temperature	± 1,3	± 7,4	± 0,1	± 3,6	± 10,1	± 1,6
σ_{max} for simple live load . . .	— 50,0	— 16,9	— 9,4	+ 0,5	— 7,0	— 51,6
σ_{min} for simple live load . . .	— 71,6	— 58,4	— 89,1	— 90,1	— 66,0	— 71,4
σ_{max} for double live load . . .	— 50,0	+ 9,8	+ 27,8	+ 44,1	+ 29,4	— 51,3
σ_{min} for double live load . . .	— 90,6	— 58,4	— 131,4	— 129,9	— 68,4	— 88,7

For the simple live load there are no tensile stresses set up in the concrete, with the exception of the insignificant value of + 0.5 kg/cm² in Section II. It is only under double live load that such bending tensile stresses appear in the reinforced concrete beams as make the formation of hair cracks a possibility. The reinforcement of the actual reinforced concrete cross sections is so dimensioned that the reinforcing bars can take up the tensile forces emanating from the double live load. This, however, by no means exhausts the carrying capacity of the reinforced concrete structure. For as hair cracks appear a substantial reduction of the moments of inertia and the modulus of elasticity take place, and consequently the live loads are now taken up to an ever-increasing extent by the hanging truss construction, whilst the reinforced concrete beam is relieved of its load. Rupture will occur when both the steel of the hanging truss and, in the same manner, the reinforcing bars of the beam have reached the yield limit, i. e. the rupture point. Proceeding from the yield limit, calculation shows that the suspension girder ruptures at eightfold, the cantilevering girder at five-fold live load. The safety of pre-stressed bridges is therefore extremely high, and for the following reasons:

a) because the anchoring is only stressed up to 2100 kg, though 2400 kg would be admissible, including additional forces. The latter, caused by wind and braking force, however, are taken up in the case of very low stresses by the cross sections of the reinforced concrete.
b) Though bending tensile stresses are not set up under single live load, yet for safety's sake strong reinforcing bars have been provided for the beams, thus raising the safety coefficient.

c) The dead weight of these solid bridges is considerably higher than that of steel bridges, and consequently they are much less sensitive to increased live loads.

It is also interesting to study the deflections set up in these very slender bridges by live loading. They are calculated for a span 1 = 70 m at 1/3160, for the cantilevering girder, which is substantially more slender (span 1 = 110 m) at 1/1100 and for the 125 m central bay at 1/1000.

It would have been more economical, and above all of advantage from a statical point of view, if the slenderness ratio of the cantilevering girder of 110 m unsupported length had been taken a little lower, say at $^1/_{20}$, corresponding to a girder depth of 5.5 m. This would have reduced the deflection of the cantilevering girder and the 125 m central span to app. 1/1500. At the same time, however, there would have been considerably smaller variations in the stresses caussed by live load for the respective cross sections in Table 2, so that under double live load the safety factor against hair cracks would still have been quite adequate.

The most simple manner of protecting the anchoring from rust is to give it an asphalt coating and then wrap sacking round the steel. The anchor bars may also be encased in concrete, the slab being only connected to the rest of the reinforced steel structure at the points of anchorage and hanging free, as did the suspension boom in the previous example, between the points of bending. Concreting is best done after the bridge has been previously loaded, thus preventing the setting up of tensile stresses in the reinforced concrete slab, due to shrinking. When creeping and shrinking are entirely accounted for, the anchor bars can be so concreted that the new reinforced concrete slab is connected to the webs. The whole then becomes a uniform, monolithic reinforced concrete structure acting under live loads in a manner different from that of hanging truss structures — a fact that must be taken into account in calculation.

2) *Continuous girder bridges with spans of 100 — 150 — 100 m.*

For calculation we take the same cross section of bridge as in Example 1, and for the central bay of 150 m span the same girder depth of 5.0 m. For the end bays of 100 m span, which are very unfavourably loaded, a girder depth of 6.25 m is provided. The considerably greater moment of inertia of the end bays causes a substantial unloading of the central span. For the ordinary through girder the dead weight and live load moments are shown in Fig. 12. In addition to the externally statically indeterminate self-stress moments now come the internally statically indeterminate forces of the hanging truss. In order to keep the calculation from becoming too involved, and furthermore to attain clear static action, we must proceed with as few hanging trusses as possible. From these and other considerations we see the necessity of a hanging truss extending over the whole length of the girder, anchored at the ends of the beams and so designed that it is able to transmit all the dead weight loads to the supports. Just as in Example 1, the hanging truss must be displaceable longitudinally in relation to the cross beams, i. e. borne on roller or rocker bearings. Its shape must be such that the distances between truss and neutral axis are proportional to the dead weight moments.

Owing to the great difference between the dead weight moments at the supports and those in the bay, the girder depths for the support cross sections would be several times greater (see Fig. 13a). Thus for practical reasons this principle of a simple hanging truss extending over the whole length of the beam must be discarded and auxiliary booms (Z_2) provided for the portions of greatest negative moments, as in Fig. 13b. The thickness of these booms should be such that $M_g = Z_1 f_1 + Z_2 f_2$, the object being that the total dead weight moments are carried by the suspension boom. As Fig. 13 shows, this type of pre-stressing forms a combination of hanging truss and strut frame, for not only is the suspension boom subjected to bending, but also the beam, which acts as a compression element, is bent in respect to its neutral axis. By the pre-stressing as shown in Figs. 13a and 13b we have now succeded in eliminating all but centric compressive forces from the through beam under dead weight, apart from the slight bending moments for the transference of the loads from one cross beam to the other. The construction of the continuous bridge and the form of the hanging truss in Fig. 14 were executed in this manner. The bridge cross sections at mid-span are depicted in Figs. 15a and 15c, and the cross section of the support in Fig. 15b.

Since the hanging truss Z_1 extends over the whole length of the beam, very long round bars are necessary. The latter can of course be made at site by welding shorter pieces together, but they are extremely difficult to lay on account of their great weight. It is therefore advisable, instead of splicing, to use steel rockers (see Fig. 16) to connect these long bars at one or more places. This form of connection does not affect the forces in the booms. As the connections are in the interior of the reinforced concrete beam, the hydraulic jacks can remain in their position of application until creep and shrinkage has entirely abated and thus the final re-stressing effected. The influence of live load on the stressing of the continuous suspension boom is also very slight in the case of the through bridge, because this suspension boom intersects the neutral axis a number of times. The same applies, however, for the auxiliary boom (Z_2), because over the short distance this boom can hardly undergo bending from the live load in the reinforced concrete beam, in particular owing to the strong haunches. For the suspension girder (Z_1) 40 bars of 100 mm $\varnothing$ are necessary, and for the auxiliary boom 20 bars of 100 mm $\varnothing$. In spite of the much longer spans under consideration in Example 2, therefore, the same cross sections of steel would be sufficient. This fact is due to the favourable influence of continuity. To lower the position of the neutral axis in the range of negative moments and thereby to obtain good lever arms for the suspension booms, particularly at the cross sections of the supports, the activity of the decking slab would have to be eliminated as in Example 1 by means of transverse joints; these would be situated in the region between cross beams 6 and 10. The decking slab in this region is designed as shown in Fig. 17. The transition to complete elimination of the decking slab action must of course be effected gradually. This can be achieved in quite a simple manner by graduating the transverse joints.

In contrast to the *Gerber* girders, in the through girder the auxiliary boom (Z_2) was not anchored in the lower compression slab, but in a separate, intermediary slab in the vicinity of the neutral axis (see Fig. 18), thereby eliminating all

Table III.

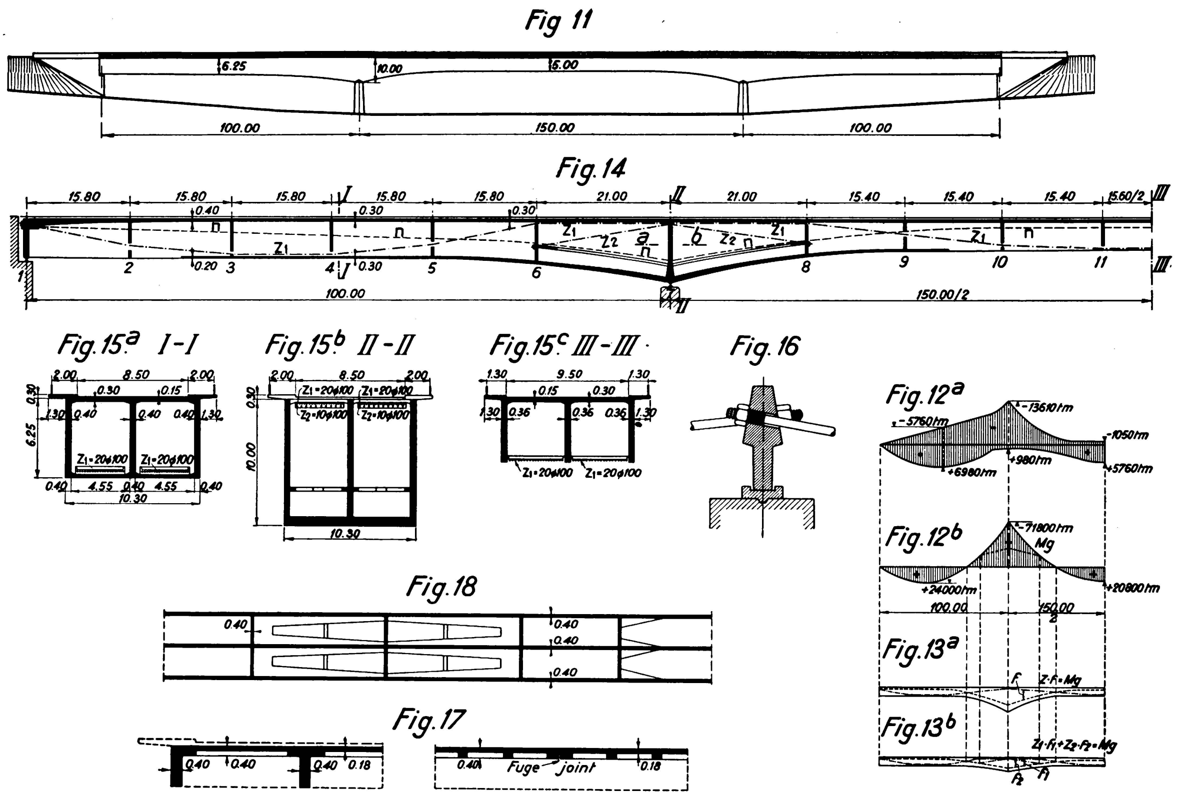

Fig. 11. Pre-stressed continuous girder bridge.
Fig. 12. 12a. Moments due to live load.
12b. Moments due to dead weight.
Fig. 14. Longitudinal section.

Fig. 15. 15a. Cross section I—I.
15b. Cross section II—II at support.
15c. Cross section III—III.
Fig. 17. Construction of bridge decking in the zone of negative moments due to dead weight.
Fig. 18. Intermediate slab for anchoring of auxiliary tie Z_2 (section a—b)

secondary stresses caused by eccentric application of forces. This solution also has the advantage that the high webs are given intermediate stiffening above the supports. Towards the latter the force of this slab is gradually transmitted to the webs, so that as the supports are approached the slab can be narrowed down, as shown in Fig. 18. The highest stresses in the two suspension booms again amount to 2100 kg/cm². The resultant stresses in the concrete of the respective cross section 4, 7 and 12 are given in Table 3.

Table 3.	Section 4		Section 7		Section 12	
	σ_o	σ_u	σ_o	σ_u	σ_o	σ_u
Dead weight	$-43,5$	$-43,5$	$-62,0$	$-62,0$	$-57,0$	$-57,0$
Traffic $+M_p$	$-22,9$	$+23,4$	$+28,5$	$-18,0$	$-27,5$	$+47,5$
$-M_p$	$+19,9$	$-24,5$	$-5,2$	$+3,9$	$+6,7$	$-14,8$
Temperature	$\pm 0,7$	$\pm 4,0$	$\pm 4,3$	$\pm 1,7$	$\pm 2,1$	$\pm 10,1$
σ_{max}	$-22,9$	$-16,9$	$-29,2$	$-56,4$	$-48,5$	$+0,6$
σ_{min}	$-67,1$	$-72,0$	$-71,5$	$-81,7$	$-86,0$	$-81,9$

The distribution of stresses in these pre-stressed bridges is, as can be seen in Tables 2 and 3, the same as that with which we are familiar in arch bridges. In the latter the compressive stresses of arch action are superimposed by bending forces due to live load, temperature changes and shrinkage. In pre-stressed girder bridges the compressive force taking the place of arch action is artificially produced with hydraulic jacks by means of trusses. The only difference between the two systems is that the pre-stressed girder bridge possesses a greater degree of safety when the live load is increased. Arches with high piers have a safety coefficient of about $n = 2.5$ in respect to increase of live load. For flat bridges these values lie between $n = 3$ for solid arches and $n = 6$ for heavily reinforced hollow arches. The safety is higher for pre-stressed bridges because the suspension boom assists the beam under bending stress the more, the nearer we approach rupture. This also applies for the cross sections with varying live load moments, in which, for example, the suspension boom lies on the opposite side of the neutral axis for negative live load moments, for in the state of rupture the neutral axis is displaced to a considerable degree towards the edge, so that the suspension boom loses its lever arm in respect to the neutral axis and thus the negative moments set up by n-times the live load are counteracted by the dead weight moments, since the latter are no longer taken up by the suspension boom.

Comparison between the system of pre-stressed plate girder bridges and the arches discussed at the commencement of this article reveals an astonishing similarity in the action of the two. In the arch bridges pre-stressing eliminates bending moments under dead weight, which is due to the fact that by shortening the tie and suspension rods we have succeeded in creating similarity with the geometrical original when the system is under dead weight. It is only a little smaller, owing to contraction caused by compressive stresses. In the preceding paragraphs we have attained exactly the same end for girder bridges as well. Under dead weight the beam did not undergo bending, nor was it deformed after decentering because it was only loaded by centric compressive forces, just as in the case of the arch bridge. Only a shortening of the axis of the beam occurred, owing to its centric compressive forces and to shrinkage and creep.

Thus we have now found the basic principle for the hydraulic pre-stressing of reinforced concrete girder systems, and in the following paragraphs we shall outline the application of this principle for other types of bridges also. As there is not sufficient space available in this article, however, I shall confine myself to a very brief description and leave the details for another occasion.

In conclusion, the approximate dimensions of the two projects discussed may be quoted. For the suspended girder of the *Gerber* bridge about 0.9 m^3/m^2 is necessary; for this bridge as a whole an average of 1.23 m^3/m^2 and 370 kg/m^2 round bars. The through bridge with a span of 150 m needs the same quantities of concrete, but 400 kg/m^2 round bars; Prof. *Mörsch*'s project for the Drei-Rosen Bridge in Basle, with spans of 56—106—56 m required 1.63 m^3/m^2 concrete and 350 kg/m^2 round bars[5].

III. Pre-stressed Suspension Bridges and Bridges of the Bowstring Girder Type.

We shall first discuss suspension bridges with eliminated horizontal thrust, considering the stiffening girder under compression to be of reinforced concrete instead of steel and seeing what advantages and disadvantages a composite system of this kind possesses. As is well known, the deflections undergone by suspension bridges are very great. The maximum deflection depends on the ratio between cable tension due to live load, and the modulus of elasticity of the cable. For increasing dead weight of the stiffening girder the cable must be stronger, which causes the tension in it due to live load, and with it the deflection of the bridge, to be reduced. When the stiffening girder is executed in reinforced concrete double the dead weight of the bridge must be reckoned with, and consequently the deflections caused by live load are reduced by about one-half. Degree of rigidity is naturally not decisive in itself; the economic aspect of the bridge is of far greater importance. The cost of the cable will be doubled. This increase in cost, however, is offset by the saving that can be effected by using reinforced concrete, which is cheaper than steel, for the stiffening beam and the decking. For spans of up to 200 m I find that there is no doubt as to the greater economy of such composity-bridges. For longer spans, such as the 315 m span at Cologne-Mülheim, composite construction could only be considered for admissible stresses in the concrete of from 130 to 140 kg/cm^2. And as present-day high-grade cements attained strengths of about 600 kg/cm^2 there need be no hesitation whatever. Owing to the great moments of inertia of reinforced concrete hollow structures, slight girder depths for the stiffening beams are sufficient. One of these suspension bridges, having spans of

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[5] The first pre-stressed bridge to be built on this system is at present in course of construction.

60 — 200 — 60 m, is shown in Fig. 19. The stiffening girder has a depth of only 3 m in the middle bay, i. e. 1/67 of the span. At Cologne-Mülheim the depth of the stiffening girder is 6.0 m, or 1/52.5 of the span. In spite of the fact that the stiffening girder is much more slender, the deflection under live load for the suspension bridge in Fig. 19 is only 1/725, while in the Cologne-Mülheim bridge this value comes to 1/400. In this connection it should be remembered that at Cologne-Mülheim the stiffening beam is designed as a *Gerber* girder with two hinges and is therefore operative in transmitting live loads to the suppports. In the suspension bridge shown in Fig. 19, on the other hand, there are three hinges in the centre bay, so that the whole dead weight of this bay has to be carried by the suspension cable alone. In the end bays the depth of the girder is 4.0 m because, as Fig. 20 shows, here greater bending moments occur. The form of cross section is illustrated in Fig. 21. The dead weight of the bridge amounts to 52.5 t/m in the centre bay, 63.5 t/m in the end bays, including the high-grade steel cable. Life load was calculated as 8.5 t/m. For a rise-span ratio of 1/9 of the cable in the centre bay, the horizontal thrust due to dead weight amounts to 11800 tons and for live load to 1900 tons. The permissible cable tension of $\sigma_c = 5000$ kg/cm^2 is accordingly divided up into 4310 kg/cm^2 for dead weight and the very small figure of 690 kg/cm^2 for live load. This small live load stress has a direct bearing on the slight deflection mentioned above. The stresses in the concrete of the stiffening beam are the following:

	Centre Bay	End Bays
Due to Dead Weight	— 67.3	— 64.0
Due to Live Load	— 24.5	— 24.9
σ_{min}	— 91.8	— 88.9

Since the stresses emanating from live load as extremely small in relation to the compressive stresses due to dead weight, hair cracks need not be feared in the stiffening beam until the live load has been increased several times its original value.

The influence of expansion in the cable and contraction (compression) in the concrete, as well as the estimated influence of creep and shrinkage, on deflection, is best obviated (see Fig. 22) by cambering the suspended girder in the centre bay in the manner customary in steel construction. The same applies to the erection of the cable. However, the effect of creep and shrinkage is difficult to estimate mathematically, besides which it goes on over a considerable period of time. In order to eliminate these influences with certainty, therefore, the suspension rods must be shortened with the assistance of hydraulic jacks (H) to an extent corresponding to the progression of creep. The exact amount of shortening necessary can be determined at any stage owing to the fact that under dead weight the stiffening girder may be taken as being free from bending. It is free from bending when no deflection whatever occurs in it. The arrangement of the hydraulic jacks (H) for shortening the suspension rods can be seen in Fig. 20. These jacks are left at their place of application even after the bridge has been put into service, and remain there until such

Fig. 19

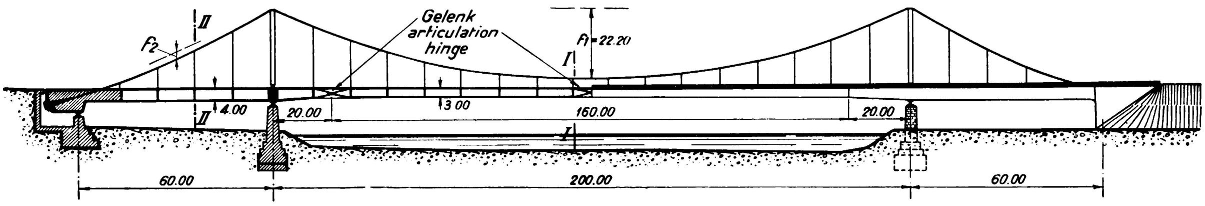

Fig. 20

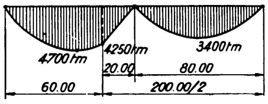

Fig. 21 a

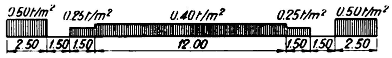

Fig. 21 b I-I

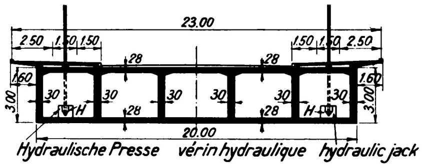

Fig. 21 c II-II

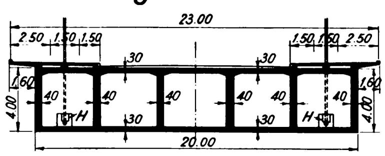

Fig. 22

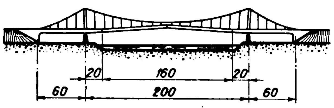

Fig. 23 a

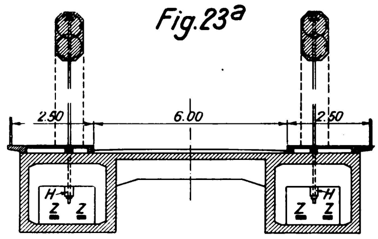

Fig. 23 b

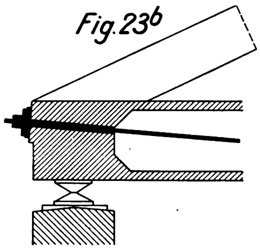

Fig. 19. Longitudinal section and elevation.
Fig. 20. Diagrams of moments due to live loads.
Fig. 21. Sections, arrangement of live loads.
Fig. 22. Erection scheme.
Fig. 23. Pre-stressed girder bridge with stiffening arch.

time as creep and shrinkage are completely over. By subsequently closing the centre joint the stiffening girder could also be made to transmit a portion of the live load, as is customarily done in steel construction, and the deflection of the bridge due to live load still further reduced. However, this is not necessary, for the deflections of these suspension bridges in composite steel and concrete are in themselves extremely slight.

In conclusion mention should be made of pre-stressed bowstring beams in reinforced concrete. Pre-stressing should be effected in such a manner that the stiffening girder under dead weight loading is again entirely free from bending moments. Consequently we must see to it that after decentering the system resembles the geometrical original projected. The bowstring girder under compression contracts by the amount $\frac{\sigma}{E}$. Now, however, we must also shorten the stiffening girder and suspension rods to the same extent by pre-stressing. For this purpose it is necessary first of all to leave an open joint in the stiffening beam, which is only closed when the tie is strained. Fig. 23 shows the cross section of a bowstring type bridge of 100 m span for a rise-span ratio of $b/1 = 1/7$. The bowstring is spirally reinforced, the object being to keep the depth of section as small as possible. The stiffening beam is composed of two hollow sections which have to take up the bending moments. The dead weight of the bridge is 27 tons, the live load 6.0 t/m. This yields a horizontal thrust of $H_g = 2380$ t, $H_{g+p} = 2910$ t. Accordingly, for the tie $2910/2.1 = 1380$ cm^2 is necessary. However, we go up to 2000 cm^2. By straining the tie we first close the joint, already mentioned, left open in the stiffening beam; the tie is then further stressed up to 2400 kg/cm^2, producing in the stiffening girder a compressive stress of $2000 \cdot 2.4 - 2380 = 2420$ tons. This compressive force in the stiffening girder corresponds to a compressive force of 38 kg/cm^2. When the live load is unfavourably placed a tensile stress of 25 kg/cm^2 is produced in the stiffening girder, so that the latter is also free from bending stresses under live load. Owing to this high compressive pre-stressing the stiffening girder is able to take over about $2^1/_2$ times the live load before hair cracks appear.

A brief account of the effect of creep and shrinkage must now be given. The stiffening girder is shortened thereby, and a drop in stressing, which can be measured exactly on the sag, takes place in the freely hanging tension boom, which runs the whole length of the bridge. We obviate this drop in stress by re-stressing with the hydraulic jacks, which are permanently in place. The effect of creep is very much greater in the bowstring girder, so that the arch is consequently flattened somewhat. The resulting deflection of the stiffening girder must be counteracted by shortening the suspension rods in the same manner as for the suspension bridge discussed above.

We have thus demonstrated that in nearly all reinforced concrete structures it is possible, by hydraulic pre-stressing of the tensile elements, to obtain similarity between the system under dead weight and the geometrical original concrete. Reinforced concrete beams and arch bridges possess an almost unlimited life when subjected to compression only. This is also true, though in a much less degree, of steel tensile bars under stressing that does not increase to any great extent. Tensile bars made of steel can easily be renewed in time to come; indeed, in girder bridges this operation can even be carried out without putting the bridge out of service.

The process of pre-stressing reinforced concrete girder systems, as described above, may also be applied to other types of structure, and particularly in hall construction. By employing it, halls of up to 100 m span can be constructed. I shall refer to this point again in a later publication.

Now that it is apparent how simply the drop in stress caused by creep and shrinkage can be eliminated, there need be no further doubts as to the advisibility of using cables of high-grade steel for pre-stressing instead of St. 35. These cables have the advantage that fulfil requirements with much less weight and smaller cross sections. The anti-rusting property of the cables is extremely good, and they can be stretched before erection.

Summary.

Proceeding from systems of pre-stressed arched and strut frames with pre-stressed ties, it is shown that in the case of girder bridges, suspension bridges and those of the bowstring type, it is also possible to eliminate bending stresses in the reinforced concrete and furthermore to exclude all but centric compression forces occurring even in girder bridges under dead weight. The main point of the problem is so to choose the method of pre-stressing that, after decentering, the system loaded only with its dead weight is similar to the geometrical original projected; in other words, that in the system under dead weight loading no deformations of any importance are caused, although contractions due to compression stresses do occur.