The stability of web plates and its influence on the designing of plate girder bridges

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The Stability of Web Plates and its Influence on the Designing of Plate Girder Bridges.

Einfluß der Stabilität der Stegbleche auf die Gestaltung vollwandiger Balkenbrücken.

Influence de la stabilité des âmes sur la disposition des ponts à âme pleine.

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Members of the Association are familiar with the latest developments in the design and construction of solid web girder bridges. A review of these noteworthy structures was given by *Karner*[1] in 1932, while the present writer has outlined the most important fundamentals for the calculation of their stability for practical purposes[2].

The experience previously gained with small and medium span plate girders could not, however, be applied to the newer structures which were continually increasing in size. The thicknesses of the webs were not increased to correspond with the increased depth of the girders, but remained practically unaltered, in addition to which the new structural steels permitted of still higher permissible stresses.

The elastic similarity would seem to call for a uniform increase in all dimensions. Since, however, the webs in the new bridges are not nearly so thick in proportion to the increase in the depth of the girders, the webs can only be properly protected from bulging and warping by taking special precautions to increase stability.

It is true that the carrying capacity of plated web girder bridges is not yet limited by the bulging of isolated panels of the web. Where bulging takes place particularly in the elastic range, there is often a considerable margin between the bulging load and the carrying capacity. Where bulging takes place in the inelastic region, however, it frequently happens that no large reserves of strength are any longer available. In this case, the big deformations in the steel which take place at its yield point may result in the structure collapsing, or in its being useless on account of considerable permanent bending deformations.

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[1] *Karner:* Weitgespannte vollwandige Balkenbrücken in Stahl (Wide-span plate girder bridges in steel), Publications of the Int. Assocn. for Bridge and Struct. Engrg., 1 (1932), p. 297.

[2] *Schleicher:* Stabilitätsprobleme vollwandiger Stahltragwerke, Übersicht und Ausblick (Stability problems of plated girder constructions; a survey and outlook), Bauingenieur, 15 (1934), p. 505.

Any bulging of the web which may occur cannot easily be restricted to small values. A visible amount of bulge would unsettle the users of the bridge, although it would not necessarily jeopardise the existence of the structure. The transpositions of the stresses set up by bulging are difficult to calculate, but they may easily lead to local overstressing and considerable damage. As far as I am aware, they have never previously been allowed for in bridge building, even when the web-plate was not designed to be safe against bulging and the stiffeners were dimensioned as compression members for the shear force.

In bridge building, the only possible attitude to take is that the web-plates of plated girders should be stiffened so as entirely to obviate bulging. The advantage of doing this is that the distribution of the forces then approximates, for all loads, to the calculated state. The true aim in any investigation of stability is therefore this: the several panels of the web between the transverse girders should be stiffened by intermediate vertical stiffeners or by longitudinal stiffeners, or by both, or else by oblique stiffeners, so that the plate may be relied upon to remain flat under all loads, and does not bulge. At the same time, the designer can be satisfied with a comparatively small actual bulging safety factor for the most unfavourable conditions of loading.

The design of large plate girder bridges depends very largely on the bulging stresses. It is no exaggeration to say that their proper design is largely a matter of investigating the stability.[3] The difficulties which arise have been overcome in many different ways. The following remarks are a survey of a few of the points of view applicable in the assessment of safety against bulging, and they should also afford a guide to designs which are satisfactory from the aesthetic aspect. To avoid misunderstandings, it is well to note that these observations should not in any case be regarded as an attempt to give directions suitable for every condition. The idea is to draw attention to a few points which are not always duly and fully observed.

The following remarks refer exclusively to the stability of the individual bays of the web. It is assumed that the stiffeners present are sufficient to compel the formation of nodal lines in the bulged surfaces at the points where they are applied.

The plate girder bridges Figs. 1 and 2 do not exhibit any web stiffeners beyond the verticals present at the transverse girders. The panels of the web are approximately square, or slightly oblong, but the projecting roadways or footpaths have a considerable effect on the appearance of the structure.

Fig. 3 shows an example of the narrowly spaced verticals and tall and narrow web panels frequently adopted in welded girders, while Fig. 4 is a road under-bridge in which the web is divided up into oblong panels. In the over-bridge, Fig. 5, the square arrangement of the smoot surfaces of the web, and the absence of any projection, are among the reasons for the less satisfactory effect as compared to Figs. 1 to 4.

In Figs. 1 to 5, the external appearance of the girders is uniform, and the distance between the stiffeners is the same throughout. This uniformity is not quite preserved in the next bridges. The diagonal stiffeners (Fig. 6) frequently adopted previously, disturb the harmony of the structure. The following remarks seem to be called for

[3] *Schleicher:* Fünfzehn Jahre deutscher Stahlbrückenbau (15 years of German bridge building in steel), Bauing. 16 (1935), p. 171. Cf. also Figs. 2 and 5.

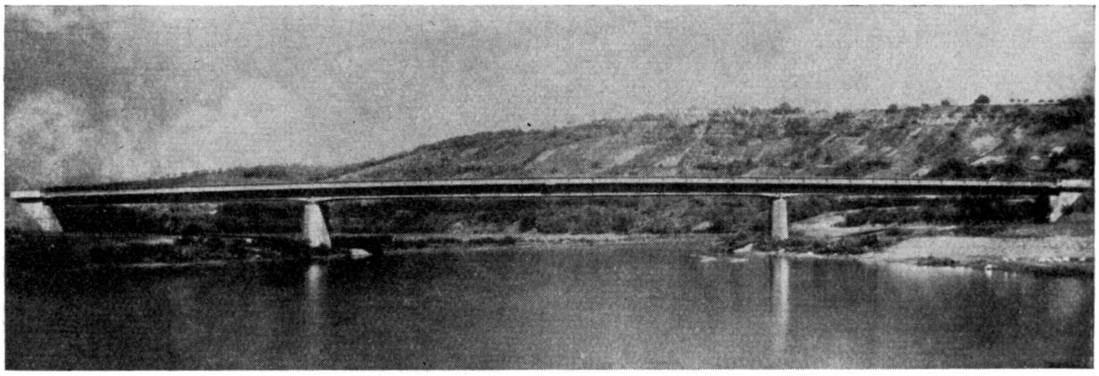

Fig. 1.
Neckar Bridge at Obrigheim-Diedesheim (central span of 90 m, 1936).

as regards the *modus operandi* of these stiffeners. From calculations made by my collaborator Burchard, the critical shearing stress of a square plate panel with diagonal stiffener (assuming that the stiffener, when it bulges, can compel the formation of a nodal line along the diagonal) is increased to about 4.6 times the

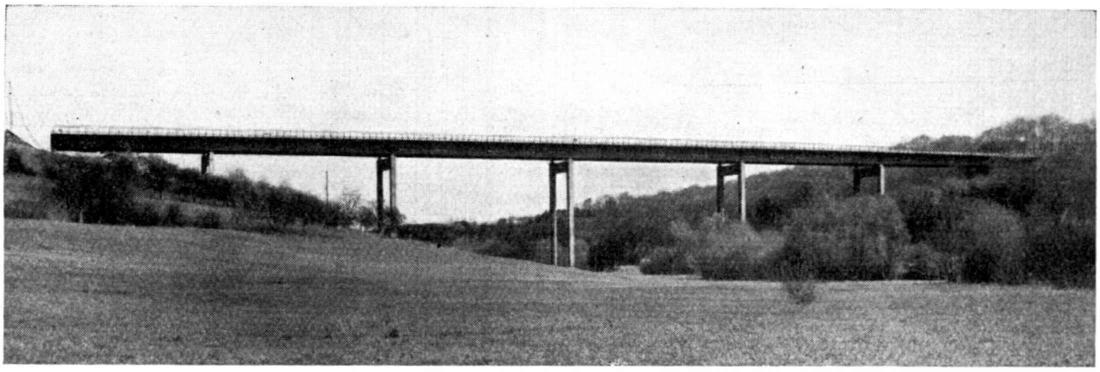

Fig. 2.
Sulzbach Viaduct near Denkendorf (largest span 63,8 m, 1936).

value of the unstiffened plate when the stiffener is located in the direction of compression, but only about 1.6 times when it comes in the direction of tension. This means that, in Fig. 6, the critical shearing stresses differ considerably in contiguous bays of the web.

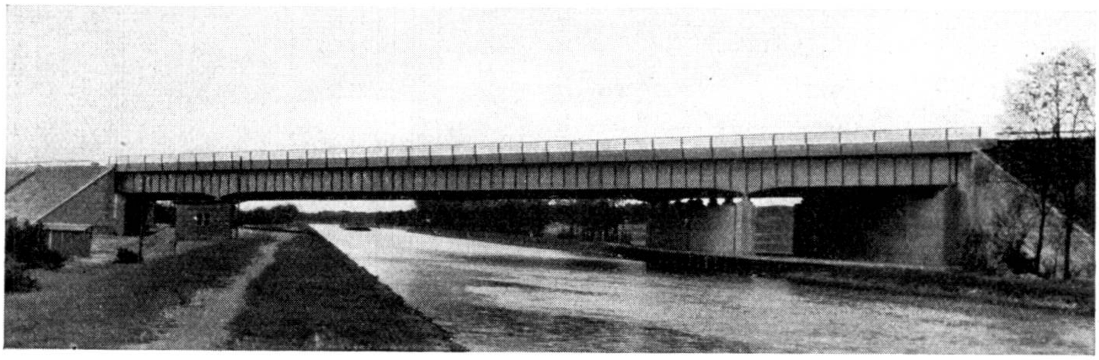

Fig. 3.
German State Arterial Road Bridge over the Hohenzollern Canal near Finowfurt.

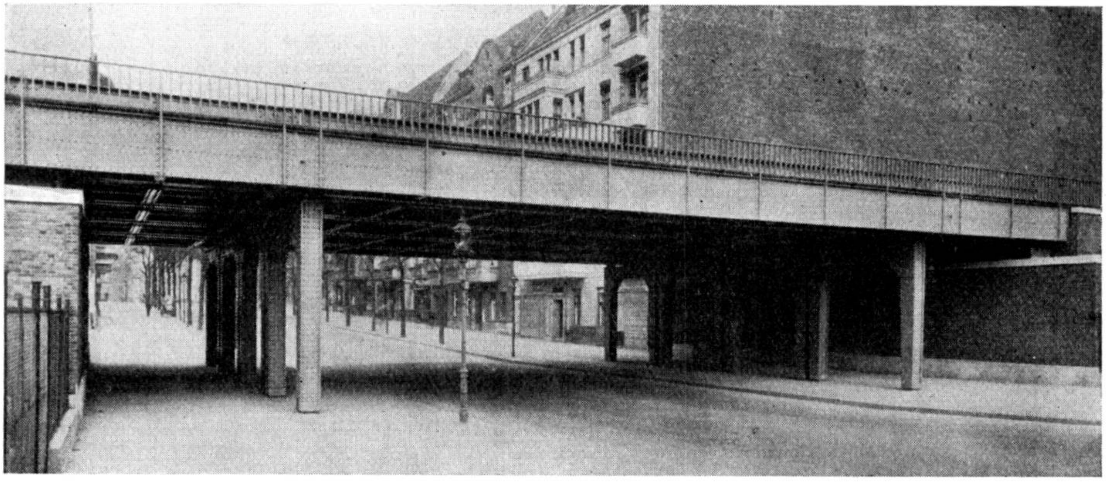

Fig. 4.
Schlüterstraße Overbridge Berlin.

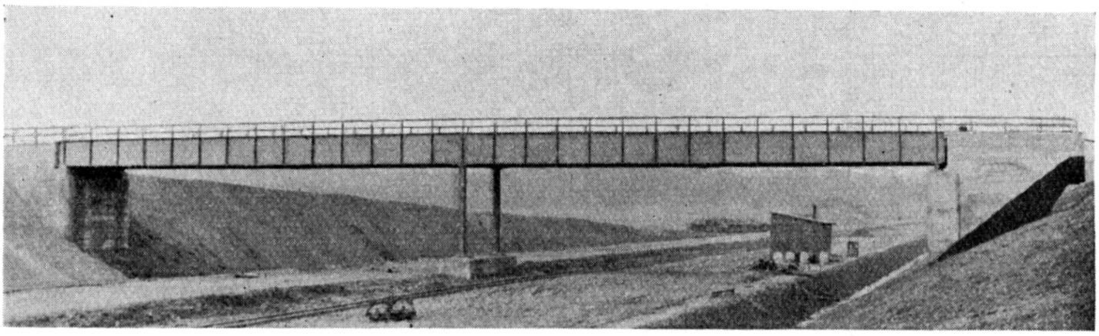

Fig. 5.
Overbridge, German State Arterial Roads.

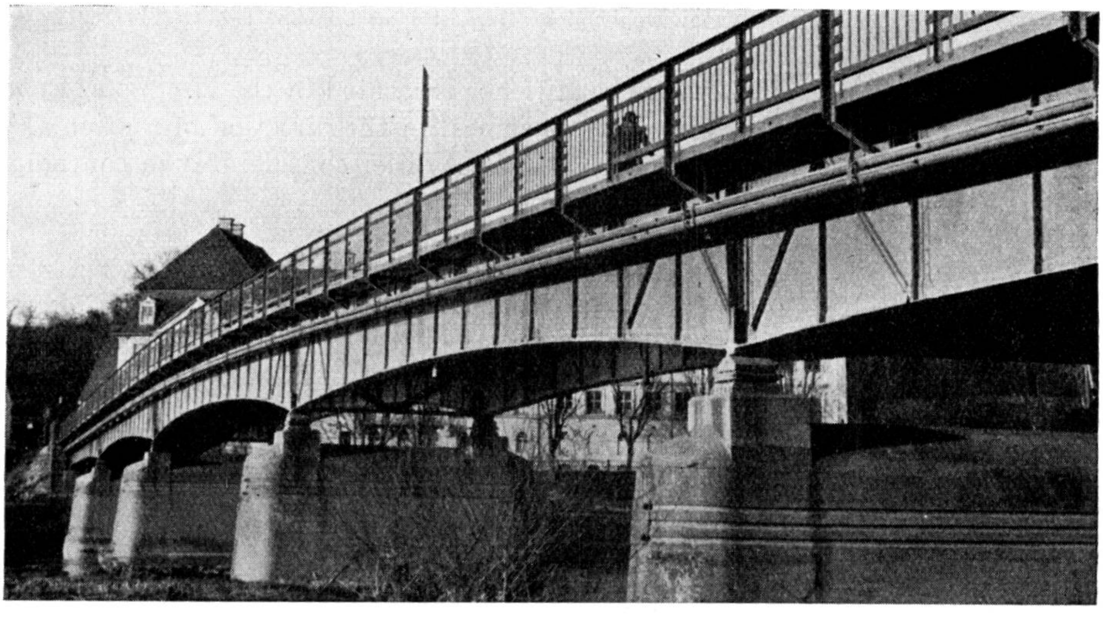

Fig. 6.
Road Bridge over the Danube near Donauwörth (largest span 32 m, 1876).

Irregularities in the spacing of the stiffeners may cause considerable disturbance under certain circumstances. In the railway bridge, Fig. 7, there were narrower spaced stiffeners in the vicinity of the welded web joints, and these are visible at a considerable distance. The further development of the welding method would enable these irregularities to be avoided to-day.

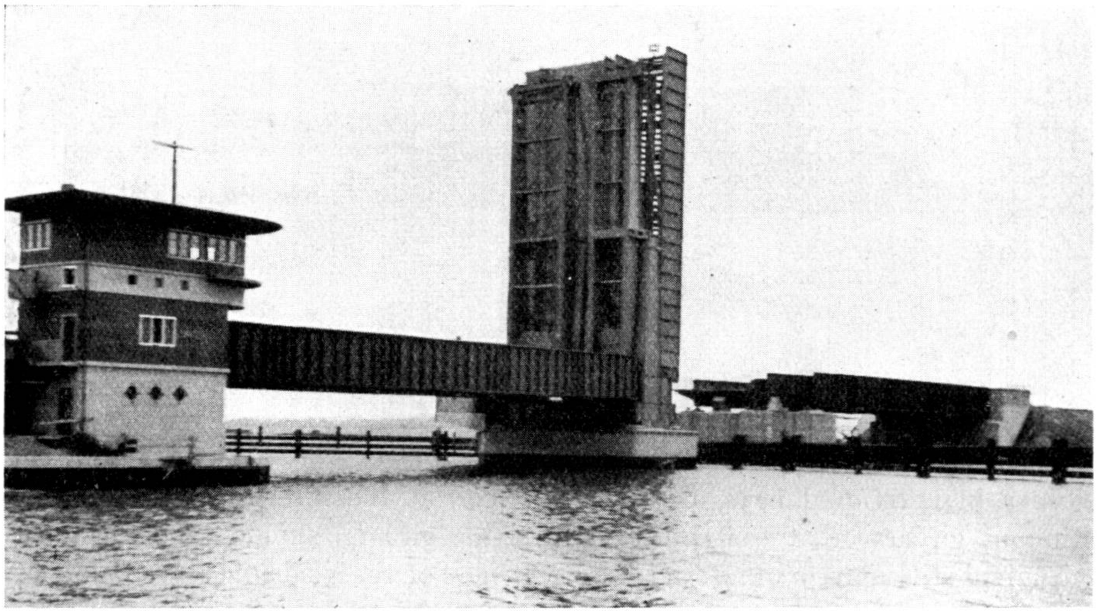

Fig. 7.
Ziegelgraben Bridge on the Rügen Dam.

In the two-hinged-frame, Fig. 8, there are intermediate stiffeners in the outer panels of the web. In most cases it will be better to avoid this alternation between horizontal and vertical web panels. In the continuous double-span welded road

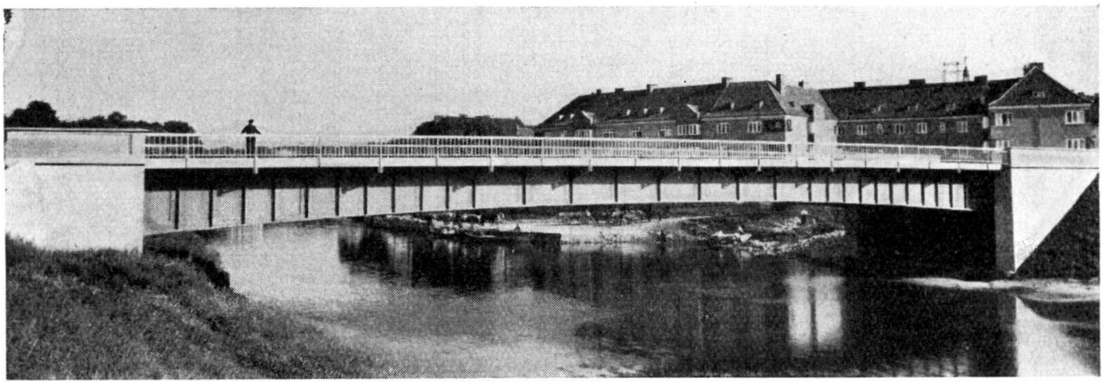

Fig. 8.
Angerapp Bridge, Insterburg (two-hinged frame, 1934).

bridge, Fig. 9, there is a closely spaced distribution of stiffeners in the region of the middle pier. Although these irregularities only show up slightly on the illustration, they can be avoided.

It must be left to each particular case to decide whether the web panels should preferably be elongated horizontally, or vertical. A very instructive comparison of both arrangements, as regards safety against bulging, is given in the table below. The writer is of opinion that endeavours should be made in every case to obtain

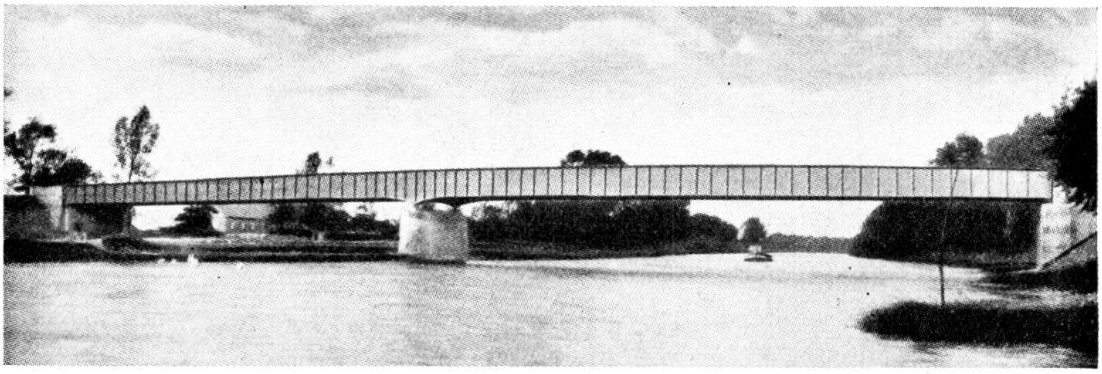

Fig. 9.
Bridge over the Ems near Steinbild (span 57,4 m, 1935).

a uniform external appearance, which means that the additional stiffeners necessary in individual bays of the web (e. g., at the intermediate supports of continuous girders or at the end bays of single girders) should take the form of longitudinal stiffeners, or of intermediate stiffeners on the inside of the web plate only.

Fig. 10 shows only the massive stiffeners present on the transverse girders. Between them are two inside, intermediate stiffeners, which can be recognised by the double rows of rivets. Fig. 11 shows the same type of stiffening applied to the structure illustrated in Fig. 2. On the front girder will be seen only the large web bays between the main verticals at the position of the transverse girders, while the additional inside intermediate stiffeners will be noticed on the rear girder.

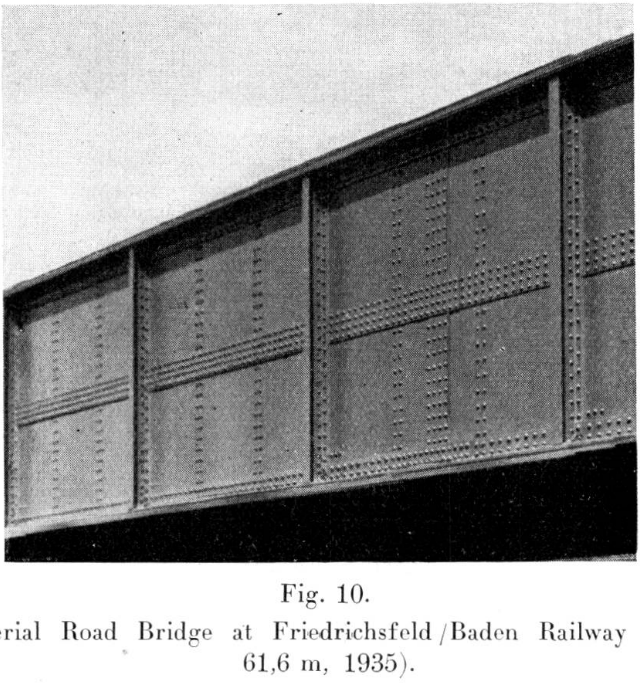

Fig. 10.
German State Arterial Road Bridge at Friedrichsfeld/Baden Railway Stn. (Middle span 61,6 m, 1935).

The appearance of the structure may be improved by dividing up large areas by an external longitudinal stiffener on the middle butt-joint of the web, and this applies particularly to very tall girders. In Fig. 12 there is an internal longitudinal stiffening in the compression zone, in addition to the longitudinal rib in the middle.

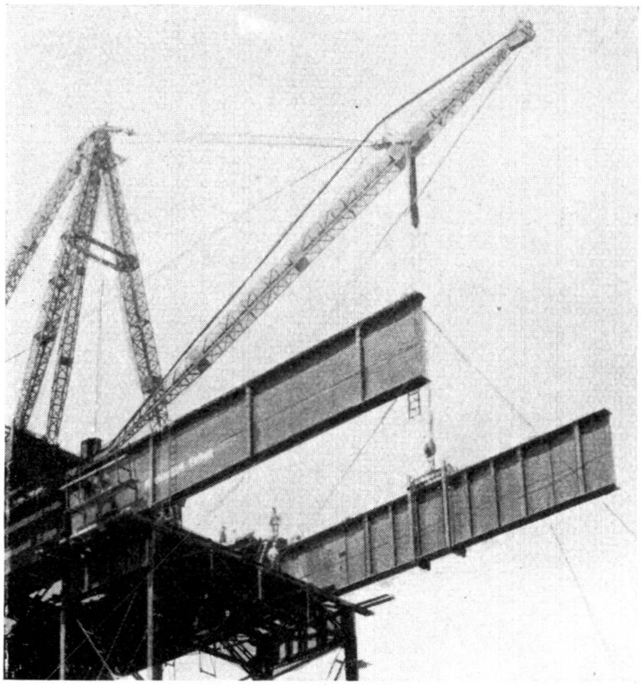

Fig. 11.
Erection of the Sulzbach Viaduct near Denkendorf (cf. Fig. 2).

There are no intermediate stiffeners. In Fig. 13 there are intermediate stiffeners on the inside, in addition to the verticals visible on the transverse girders. The longitudinal stiffeners located on the inside only will be recognised by the rows of rivets in the vicinity of the upper boom, and on either side of the central pier in the lower compression zone.

The design of girders at the supports is a particularly important point. A good effect can be achieved even with a slightly curved lower boom line (Fig. 14). Figs. 15 and 16 show that the points of support may, without objection, be more emphasized than the ordinary stiffeners at places of transverse girder. But they should not be emphasized too strongly (cf. Fig. 17). The multiple stiffeners at the supports, adopted in several new bridges, have a somewhat disturbing effect, by showing that the designer could only overcome his difficulties by special additional structural components which have the appearance of "expedients". Moreover, these multiple stiffeners can be avoided, even for maximum loads, as is proved by the example of the road bridge over the South Elbe at Harburg-Wilhelmsburg.

As the last illustrations show, it is important to observe a certain amount of precaution when additional stiffeners are provided for on the inside of the web only. The girder in Fig. 18 is stiffened by longitudinal ribs on the inside of the

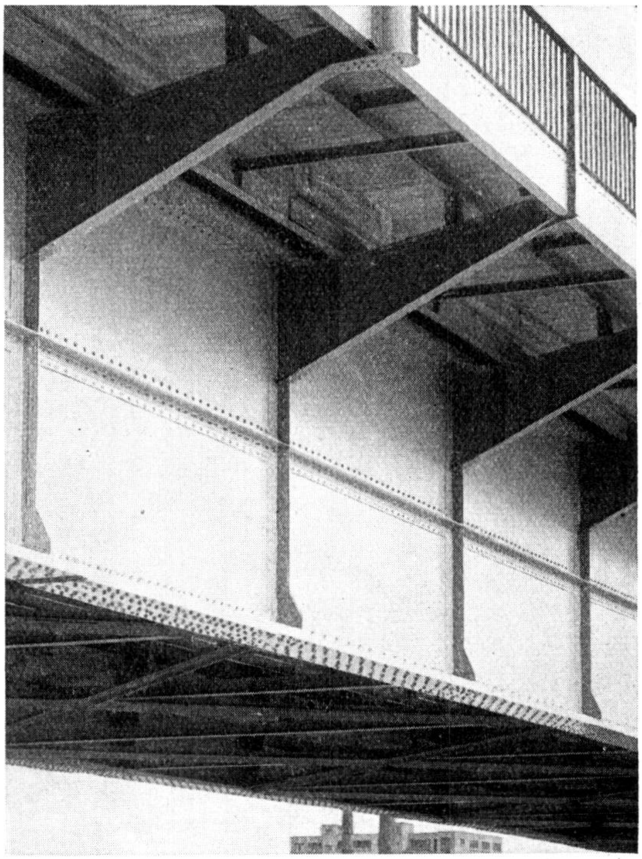

Fig. 12.
"Three Roses" Bridge at Basle (middle span 105 m, 1934).

web at the river pier. These stiffeners are staggered on account of the very variable height of the girder. The particular rows of rivets are very noticeable. This is more noticeable still in Fig. 19, where the internal stiffenings follow the line of the lower

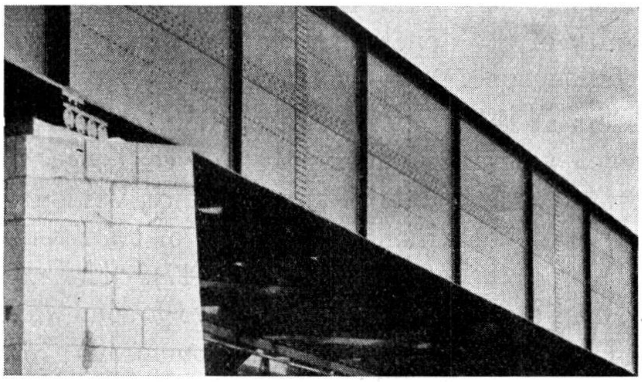

Fig. 13.
German State Arterial Road Bridge over the Main near Frankfort-Griesheim (largest span 70 m, 1934).

boom in the region of the negative moments, or in Fig. 20, where the stiffening ribs following the line of the lower boom come on the outside. This considerably emphasizes the point of support, but the appearance of the bridge would have been improved without them. In larger girder bridges, the web usually has to have a longitudinal butt joint. It is often an advantage to apply longitudinal stiffeners on the middle-joint, either on one side (Fig. 10) or both sides (Fig. 12). Special attention should be given to the lines of the longitudinal joints when the girder is provided with haunches (Figs. 14 and 15). The vertical joints of the web usually recur at distances of 10 to 15 m. In the structure as Figs. 12 and 16, the web-plate joints are located 15 m apart on each third cross girder. The connection between the cross girder and the footway bracket coincides with the cover plates

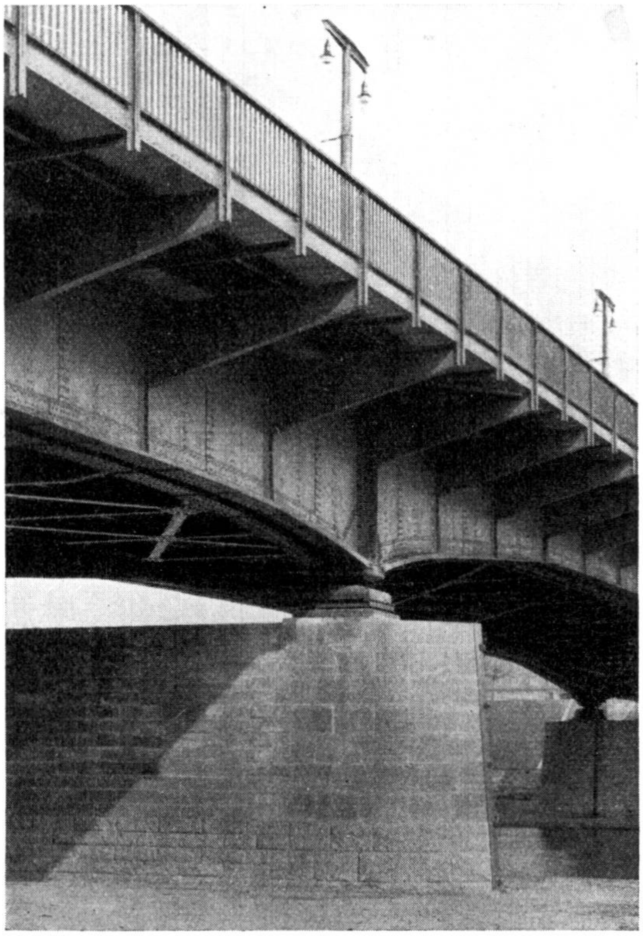

Fig. 14.
Adolf Hitler Bridge over the Neckar at Mannheim (86 m span, 1926).

of the joints. The other transverse girders have suitable stiffeners, so that the joints are no longer visible from outside. This bridge was constructed on the free cantilever erection principle over the 105 m wide middle span, and the work of erection was not rendered difficult by this arrangement of joints. The same remark applies to Fig. 15.

Fig. 10, on the other hand, exhibits vertical cover plates in every third panel. The accumulation of vertical rows of rivets spoils the appearance when the structure is looked at from a short distance. In Fig. 14, the web-plate joints were made in all the panels, thus giving the bridge a uniform appearance. This particular arrangement will of course not be considered in ordinary conditions. In many

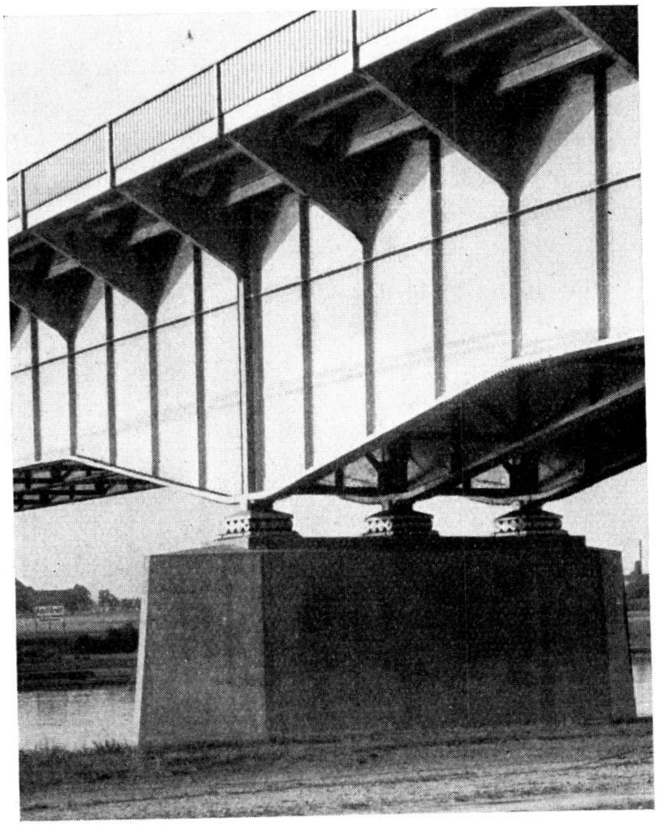

Fig. 15.
Kaditz Bridge over the Elbe near Dresden
(115 m span, 1930).

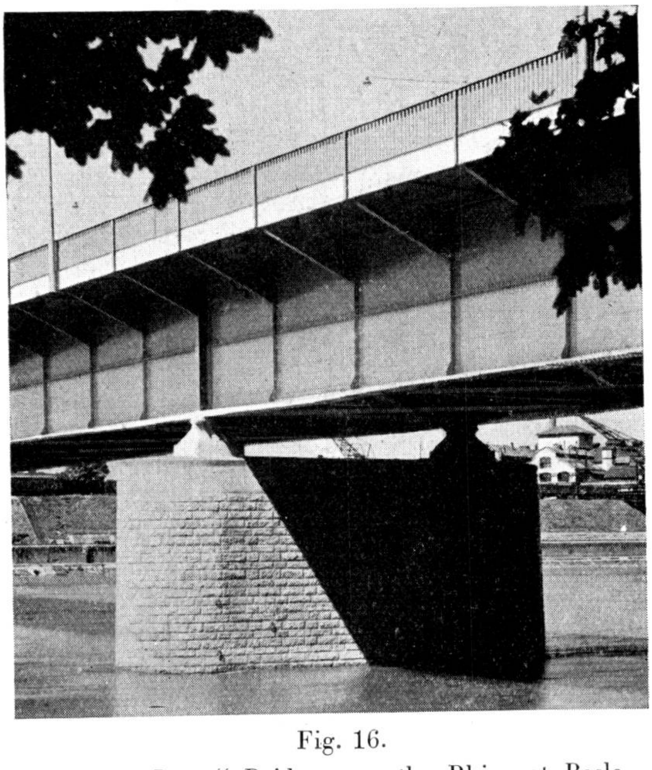

Fig. 16.
"Three Roses" Bridge over the Rhine at Basle
(middle span 105 m, 1934).

cases, however, the appearance of the bridge will be improved by avoiding irregularities due to cover plates, and this can be done without appreciably increasing the cost.

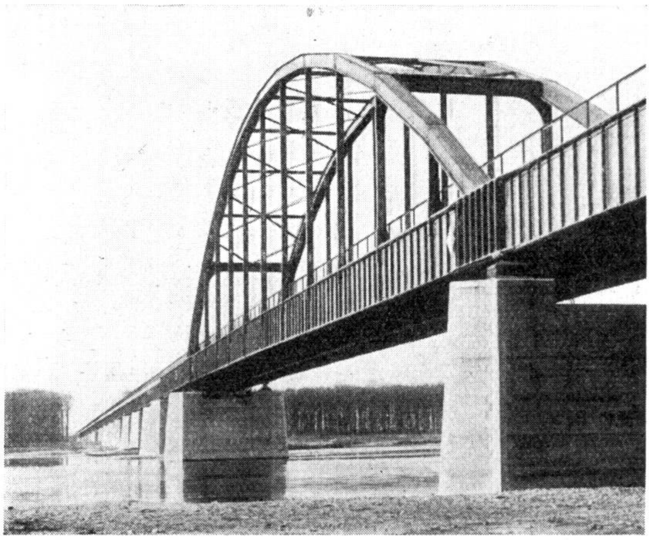

Fig. 17.
Bridge across the Elbe near Fort Dömitz (span 153,8 m, 1936).

Since they do not as a rule belong to the load carrying capacity, longitudinal stiffeners need not be joined to the transverse girders, etc. They can be cut off straight on the inside of the web-plates, while it will be advisable to shape the

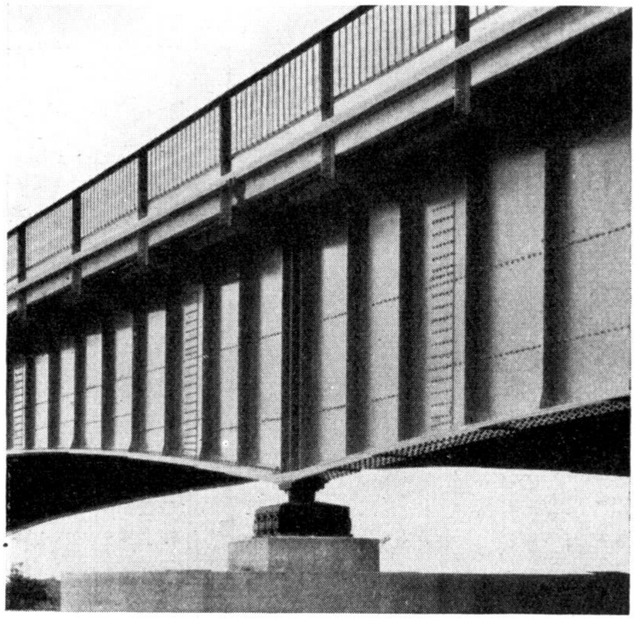

Fig. 18.
Leda Bridge near Leer (E. Friesland) (Middle span 63 m, 1933).

projecting portions on the outside by means of oblique cuts, so as to give continuous lines.

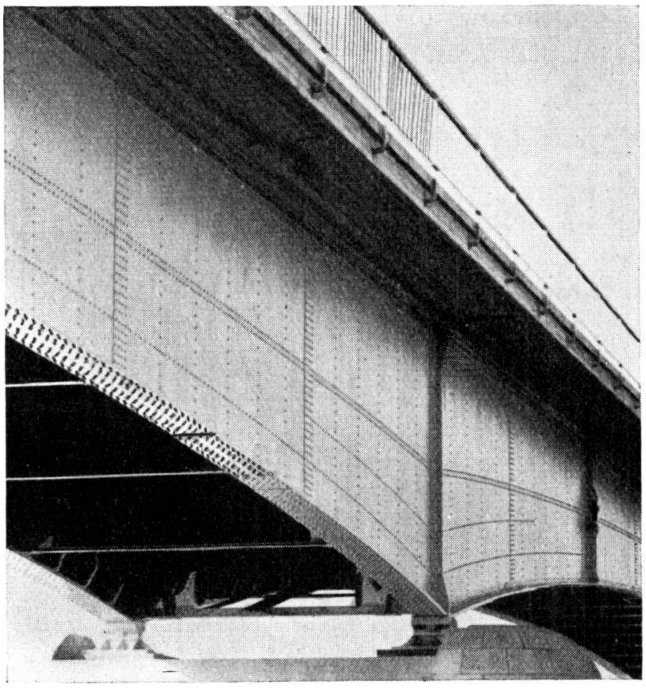

Fig. 19.
Oder Bridge near Poppelau (middle span 79,60 m, 1934).

In the following table the bulging stresses are compared for different methods of stiffening. Two typical cases are considered:

(1) Web panels of 200 × 200 cm area and 14 mm gauge, of Steel 37.

(2) Web panels of 400 × 400 cm and 20 mm gauge, of Steel 52.

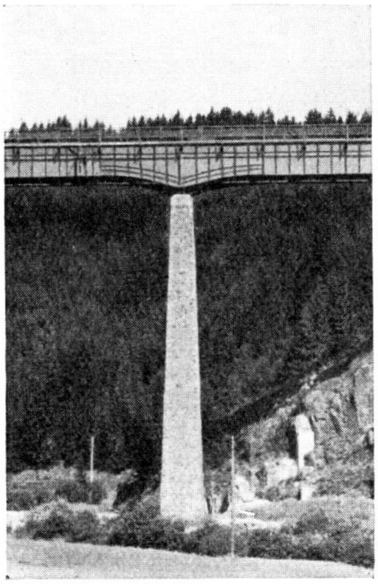

Fig. 20
Wettera Bridge
(58,8 m span, 1929).

	Anordnung der Aussteifungen / Disposition des raidisseurs / Arrangement of stiffeners	1. Beispiel, St 37 / 1. Exemple St 37 / 1st Example St 37		2. Beispiel, St 52 / 2. Exemple St 52 / 2nd Example St 52	
		σ_{ok}	τ_k	σ_{ok}	τ_k
1		2,10	0,87	1,14	0,44
2		2,13	1,33	1,21	1,20
3		2,21	1,39	1,52	1,59
4		2,32	1,39	2,14	2,08
5	b/3	2,37	1,25	2,45	0,76
6		2,40	1,33	3,16	1,20
7		2,40	1,39	3,16	2,08

Warping stresses produced by bending alone, and by shear alone respectively (in t/cm²).

The table contains the particular critical edge stress σ_{ok} for pure bending, and the critical shearing stress τ_k under a purely shearing load.[4] The edges of the plates are assumed to be pivotally supported for the dimensions given. Where the bulging stresses exceed the limit of proportionality, they were diminished in the same ratio as for a compression member stressed to the same extent[5], the buckling stress line being plotted in accordance with the regulations of the German State Railways Company. The results would not differ appreciably for any other method of reducing of the bulging stresses in the inelastic region or for any other buckling stress line of the ductile steel.

When assessing the effect of the various methods of stiffening, it should be noted that the web-plate has been assumed to be comparatively thick in both examples. The table shows that the critical shearing stress τ_k can certainly be compensated sufficiently by the vertical intermediate stiffeners but that the critical bending stress σ_{ok} remains unsufficient, especially in the second example. Large plate girders can certainly hardly be properly stiffened by intermediate stiffeners alone. Nor would rigid restraining of the web-plate in the boom plates[6] be sufficient in these cases to compensate for the deficiency.

Summary.

The new, large plate girder bridges can no longer be designed in terms of previous experience. To secure the webs against bulging which is likely to occur, for various reasons, in bridge building, various methods have been adopted in practice, some of which, however, do not give entire satisfaction from the standpoints of cost, statical strength or aesthetics.

An investigation of the stability has been found to be highly necessary for the succesful design of modern steel, solid plate girder structures.

In the case of deeper webs, it is not the bulging set up by shearing stresses, but the bulging set up by bending stresses which is the more dangerous. With fairly deep webs, even where these are of considerable gauge, is it almost impossible to attain sufficiently high bulging strength by vertical stiffeners alone. Only longitudinal stiffeners, or longitudinal and transverse stiffeners, enable the optimum limit to be reached at a reasonable cost.

Irregularities on the outside surface of the girders usually detract from the appearance of the structure. But even though additional stiffeners are necessary at certain points, and these are placed on the inside, a certain amount of caution is necessary in applying them.

[4] Über den Fall der Beulung bei gleichzeitiger Wirkung von Biegungs- und Schubspannungen (The case of bulging (warping) under simultaneous action of bending and shear stresses). Cf. *Chwalla*, Bauing. 17 (1936), p. 89, or *Schleicher:* Bauing. 15 (1934), p. 509.

[5] *Schleicher:* Intern. Assocn. for Bridge and Struct. Engrg., Final Report of First Congress, Paris, 1932, p. 131. Cf. also (⁴), p. 510.

[6] *Nölke:* Biegungs-Beulung der Rechteckplatte mit eingespannten Längsrändern (Bulging due to bending of rectangular plates with fixed edges), Bauingenieur, 17 (1936), p. 111.

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