Experience obtained with structures executed in Belgium

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Experience obtained with Structures Executed in Belgium.

Erfahrungen bei ausgeführten Bauwerken in Belgien.

Observations sur les ouvrages exécutés en Belgique.

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The present report refers to welded bridges only. At the end of 1935 there were in Belgium some thirty bridges entirely welded, either in use or under construction. The majority are fixed bridges and with the exception of a few plated girder bridges the others are road bridges of the Vierendeel type.

Since 1932, when the first semi-welded, semi-riveted bridge was constructed, great strides have been made. The present tendency may be summed up as follows: As little welding as possible, the use of rolled sections wherever possible, systematic use of butt welded joints, placing of joints where there are the least stresses, filling the roots of joints with electrodes of small diameter and subsequent reducing the number of passes by using electrodes of larger diameter, avoidance of front fillet welds over the whole width of plates and of the meeting in one point of side fillet welds, no sharp angles, but gradual curves of large diameters and less prejudice against welding in difficult positions.

On the other hand a number of structural engineering firms have equipped themselves for carrying out welding in the workshop in normal positions, and for preventing deformations. Such methods are mostly the private properties of these firms.

The spans of the bridges vary between 35 and 90 m. The width of the roadway is 3, 6, 9 or 12 m, whilst that of the sidewalks is from 1 to 3.50 m. Some of these road bridges are built to carry also local trains with two engines of 30 tons and a number of 15 ton wagons.

Mild steel used in these bridges has the following properties: minimum rupture strength 42 kg/mm^2; limit of elasticity 24 kg/mm^2, elongation 20 to 24 %, quality coefficient 1000, fatigue strength 12 kg/mm^2 without wind and 13 kg/mm^2 with wind.

A typical cross section for road bridges is shown in Fig. 1. The longitudinal beams are encased, and the bondage between steel and concrete ist established by small horizontal and vertical welded lugs and stirrups, the whole is of great rigidity. Tests carried out with such bridges show that the distribution of live loads is well assured. When the span of the bridge exceeds 50 m the decking and the longitudinal beams should have expansion joints. In fact, the shrinkage of the reinforced concrete decking slab introduces compressive forces in the longitudinal beams. As the lower flanges of these beams do not follow such deformations dangerous torsion stresses might be set up in the cross girders. The longitudinal beams are butt welded over the cross girder in normal manner. They are fixed to the cross girders by lateral fillet welds of from 5 to 8 mm thickness.

When the available height for the decking construction is insufficient the connection-type shown in Fig. 2 is employed; which as proved by fatigue tests

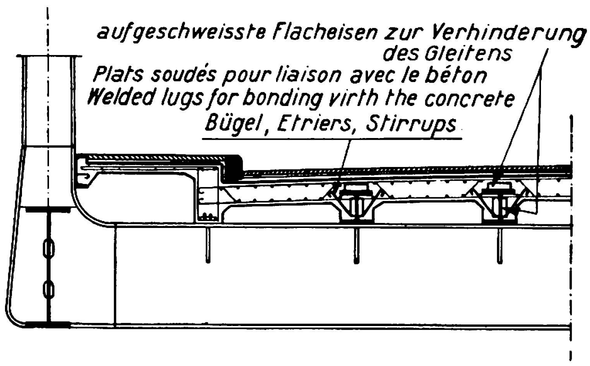

Fig. 1.

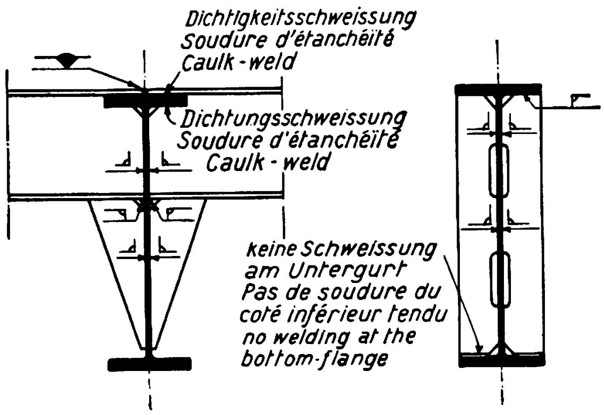

Fig. 2—3.

offers great strength. The cross girders are Grey-sections, but in case of very wide bridges without upper wind bracings composite sections are sometimes necessary. They are then plated girders with thin and deep web plates. When stiffeners are welded to the web plate care must be exercised to avoid internal stresses which may lead to damages to the web. One kind of web stiffener is shown in Fig. 3.

The cross girders are welded to the uprights by gradual transitions without sharp corners or abrupt changes in cross sections (Fig. 1). The butt-welding of flange plates is done in normal position. The welding of the web is carried out in vertical position, the joints being V-shaped with re-welded root, or X-shaped. It is necessary to make sure by preliminary tests that the electrodes are suitable for this kind of work. It should only be done by specialists who have passed an examination.

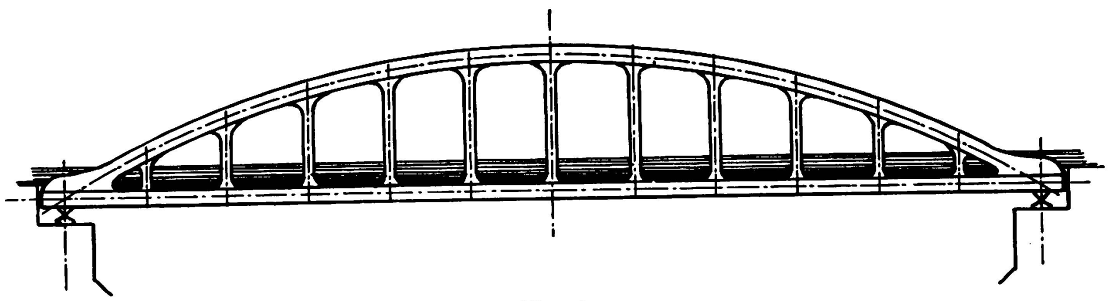

Fig. 4.

The lower wind bracings are temporarily bolted and welded only after the welding of the main girders.

The main girders are of the Vierendeel type with 8, 10 or 12 panels (see Fig. 4). The upper chord is a parabola of the second degree. The rise is usually between $^1/_7$ and $^1/_8$ of the span. This type of girder is economical because, under dead weight and uniform live load the various parts of the structure receive only normal and shear forces. The moments are nil if the deformations due to normal forces are not taken into account.

As regards appearance this is a very satisfactory type of bridge, and for the eye it gives the impression of an easy flow of forces, recalling bow-string girder bridges.

Prof. *Vierendeel* has published a simplified method of calculation for such bridges. Professors *Campus* and *Magnel* likewise teach simple methods of calculation which permit easily the tracting of influence lines for M, N and T of the different members.

Tests carried out on bridges in use which were calculated according to these methods have shown that the stresses at places of highest stressing are less than those calculated.

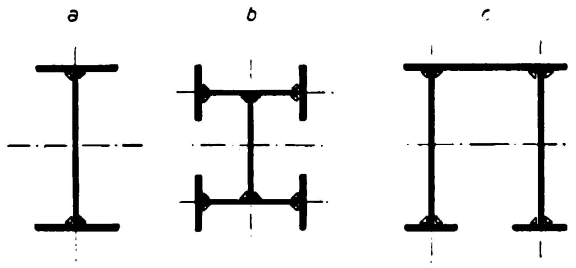

Fig. 5.

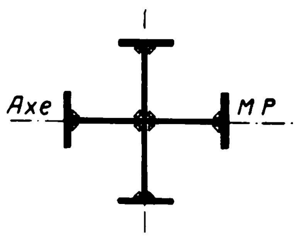

Fig. 6.

The upper chord is normally a composed section of I-shape (see Fig. 5). When there is no top windbracing a section is used as shown in Fig. 5b. In case of large bridges box girders as shown in Fig. 5c have to be used. The uprights are usually composed of T-sections, laterally stiffened (Fig. 6). The half-length of a rounded gusset varies between $^1/_3$ and $^1/_4$ of the distance between two panel points (Fig. 7), a length which assures the rigidity of the angles. The gussets are cut in quadrants.

A large number of tests were carried out on models and on bridges in use. The stresses in the curve of the gussets do not vary evenly, there is, however, increased straining at the beginning of the curve where the radius changes suddenly. Hence the circle is not the ideal shape for the gusset. Possibly a parabola or an ellipse would allow to avoid these stress accumulations, but the circle has been maintained for the gussets on account of easy execution.

Besides, fatigue tests have shown that the quality of the welding plays a preponderating part. The gussets in shape of quadrants, even if imperfect, are in any case better than straight-line gussets, which sometimes have been employed (Fig. 8). Corners only little rounded are the seat of local overstressing and therefore offer but little resistance to changing straining efforts.

Tests have likewise shown that over-stressing takes place at the junction between curved flange plates and web plates (Fig. 9). In order to keep this kind of stressing small the ratio of flange width to the thickness of the flange should be below a certain value which varies with the radius of the gusset.

The distribution of stresses in the gusset plate is of a very complicated nature. Tests have demonstrated that thin webs with a number of radial stiffeners are the seat of considerable residuary stresses due to thermal shrinkage. It is better to employ thicker web plates without stiffeners.

Weldings carried out in workshops do not present any difficulties, where they can be executed in normal position.

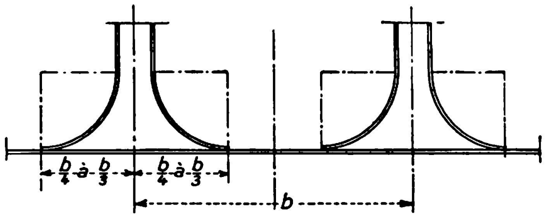

Fig. 7.

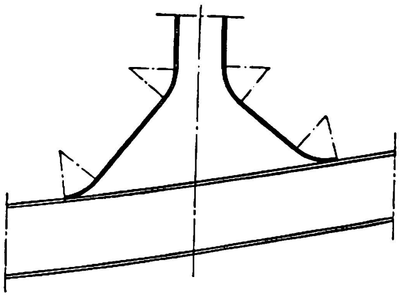

Fig. 8.

The connection between curved flange plates and chords is a delicate matter. The stresses in such connections are small being only 2 to 3 kg/mm². In Fig. 10 is shown a connection of this kind, it consists of the primary welding of a tapered machined plate to which the curved flange plate is butt-welded.

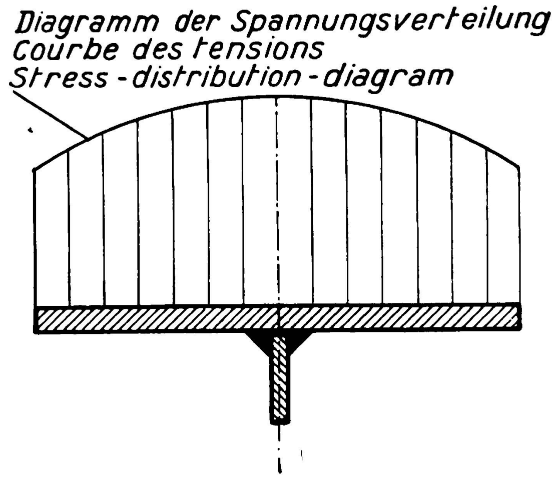

Fig. 9.

With regard to weldings done in situ, butt-welding is adopted for the chords, either with V-joints and rewelded roots, or X-shaped joints. The welds are finished-off with a distinctly convex surface. To avoid the formation of end "craters" small little plates are welded on which are removed after welding is completed. All incidental damage and burning off of parent metal must be filled up again. For welding in situ electrodes are used which supply a weld metal with a rupture strength of from 45 to 55 kg/mm² and a minimum elongation of 20 % with an elastic limit of 30 kg/mm². The erection joints should be placed at the points of counterflexure; at places of the least stresses.

With respect to erection joints of the uprights, the present tendency is to butt-weld the flange plates and to use fillet welds for the web (Fig. 11). It is but logical to place such joints in half the height of the uprights, in the point of counterflexure. But conveyance by rail must also be taken in account, hence for central uprights two erection joints are employed, placed in the straight partion of the post, near the curved gusset plates. The dimensions of the uprights and their connections are calculated generously, on account of alternating stresses caused by different positions of live loads. The uprights are the members of highest stressing.

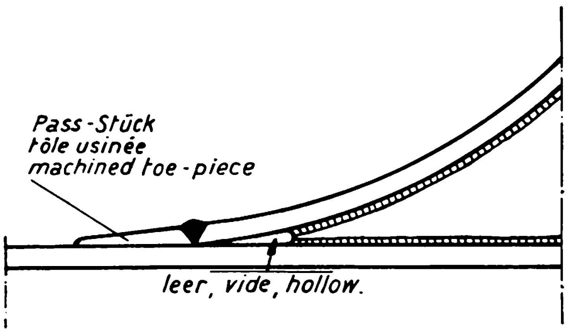

Fig. 10.

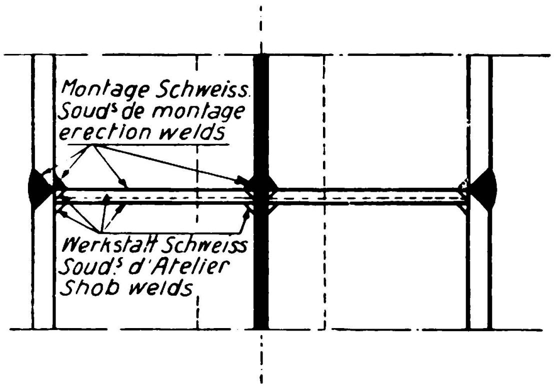

Fig. 11.

Welding at erection site is sometimes difficult as members to be assembled are not always of equal and exact dimensions, as rolling margins must be taken into account. Besides which, the welded sections show deformations which are not the same in the two parts to be welded.

Hence the welding at site must be especially studied beforehand. Devices should be provided for permitting adjustment of the various joints before welding.

The experiences made with welded bridges have shown that welding, like reinforced concrete, is valuable only according to its workmanship. Supervision and checking of the welders and weldings are of the utmost importance.

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