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Some Examples of Welded Steelwork in Czechoslovakia.

Einige Beispiele von geschweißten Stahlkonstruktionen in der Tschechoslowakei.

Quelques exemples de constructions soudées en Tchécoslovaquie.

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The applications of electric welding in the special fields of bridge and large building construction show a great deal of progress during the last few years, due to the introduction of this process. Thus in 1935 a number of large hangars covering a total area of 1500 m² were constructed, the most notable part of the construction being the girder of 50 m span over an entrance (Fig. 1) which serves to carry lattice trusses spaced at 10 m centres (Fig. 2).

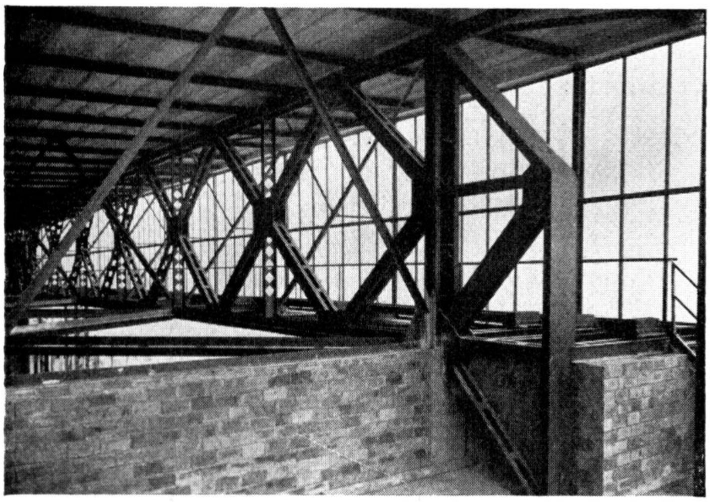

Fig. 1.

With a view to appraising the advantages of arc welding the whole design was worked out both for riveted and welded construction, and a comparison between the two solutions disclosed some interesting facts. At first the welded design for the 50 m span was based on the use of ordinary steel C 38, while the riveted design was made on the assumption that high tensile steel C 52 would be used; yet although in the second case the permissible stresses were taken 50 % higher than in the first, the weight of the girder worked out approximately the same in either.

As regards the actual supporting structure of the hangar for which ordinary steel C 38 was used in both cases, the saving in weight due to the adoption of welding worked out at 20 % (5,210 kg as against 6,500 kg). In view of these results, as well as economic considerations, the construction was, in fact, carried out in the ordinary steel C 38, electrically welded both in the fabricating shop and on the site.

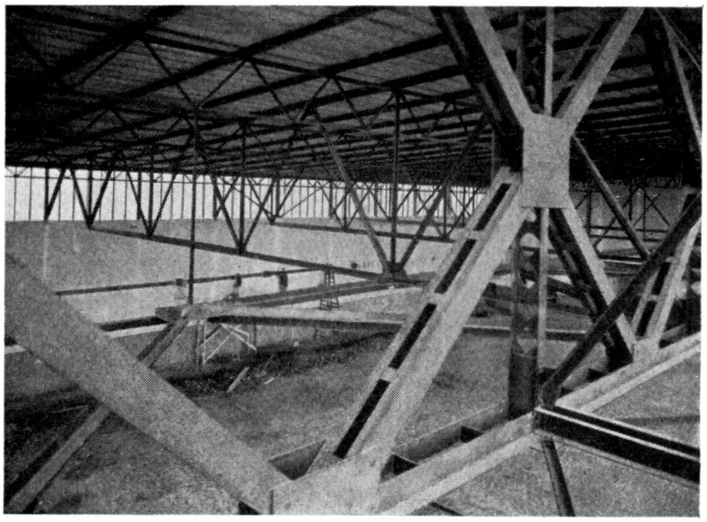

Fig. 2.

Two different types of electrodes were used in welding, giving the essentially different mechanical characteristics indicated by the minimum values specified as in Table I.

Table I.

Types of electrode	I	II
Tensile strength, in kg/mm^2	38	42
Elastic limit, in kg/mm^2	23	26
Elongation, %	12	20
Resilience (Mesnager), in kgm/cm^2	3	6

The use of Type I electrodes was authorised for various parts of the structure having a span of less than 15 m, as well as in side fillet seams of constructional members with a greater span, subject to a lower value for the permissible shear stress in these.

The maximum stresses allowed both in the parent steel and the weld metal are given in Table II in relation to the different stresses imposed.

Table II.

Permissible stresses	Parent metal	Weld metal	
		Type I	Type II
Tension	$\nu = 1200$ (1400) kg/cm^2	$0.75\ \nu$	$0.85\ \nu$
Compression	$\nu = 1200$ (1400) kg/cm^2	$0.95\ \nu$	$1.00\ \nu$
Shear	$\tau = 850$ (1000) kg/cm^2	$0.60\ \nu$	$0.65\ \nu$

Note. The values shown bracketed were allowed in cases where all external effects had been taken into account in the calculation, namely the effects of temperature and of wind pressure.

Before welding work was begun the types of weld seam and the welders were subjected to various tests, the specified minimum results of which are given in Tables III and IV.

Table III.

Tests for weld metal	Type of electrode I	II
Tensile strength, in kg/mm^2	38	42
Shear strength, in kg/mm^2	28	30
Bending angle, in degrees	120	180
Elongation, % . . . , ...	12	18

Table IV.

Tests for welders	Type of electrode I	II
Tensile strength, in kg/mm^2	34	40
Shear strength, in kg/mm^2	26	29
Bending angle, in degrees	90	120
Elongation, %	10	15

In the final design all the connections were worked out with a view to taking advantage of the latest improvements in welding technique. Considerable use was

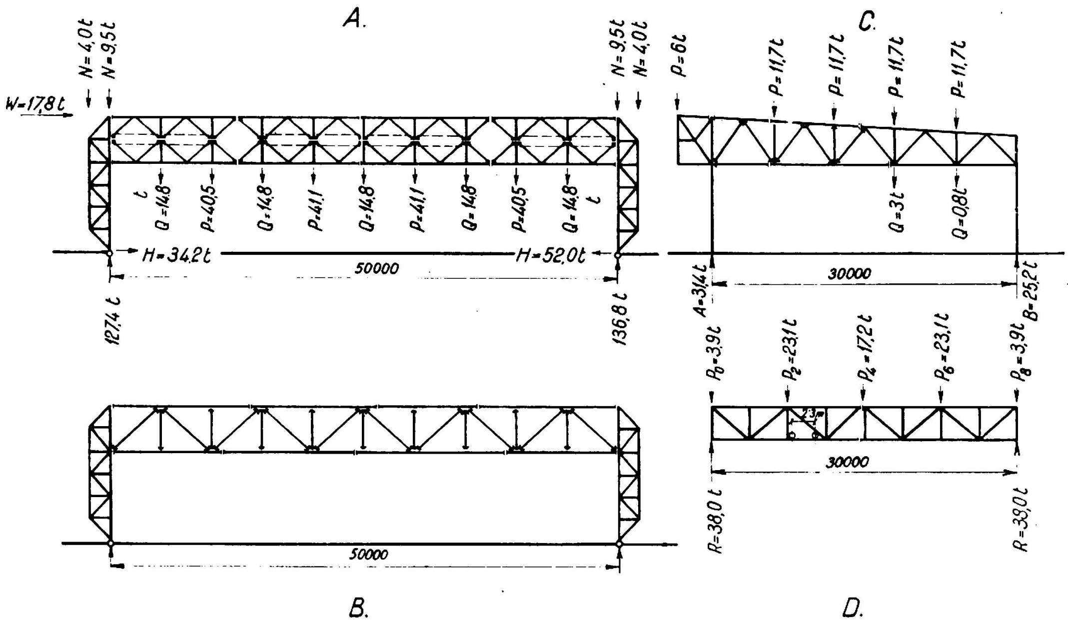

Fig. 3.

made of simple T sections obtained by cutting ordinary rolled joists in halves with the cutting blowpipe.

The boom member over the doorway was formed with a very simple cross section (500 × 500 mm plates and 100 × 180 × 18 mm angles) which was particularly well adapted to the conditions of stress arising therein, the total axial load in the boom being 318 tonnes.

Certain members such as the verticals of the girder over the doorway were formed of sections obtained by cutting ordinary rolled joists along a zig-zag line and then welding together the points of the two parts so separated after moving them opposite one another. In this inexpensive way it is possible to

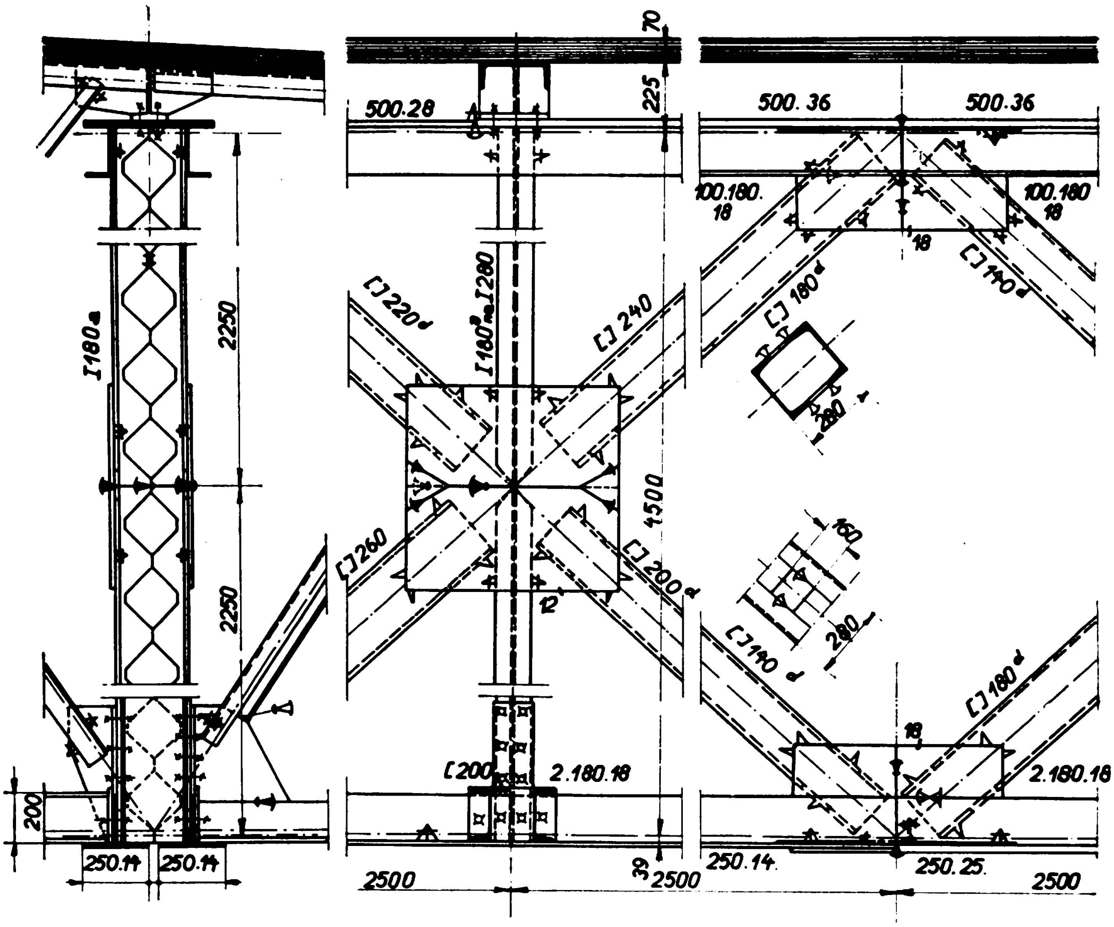

Fig. 4.

obtain an open webbed beam of the same weight as a normal beam but much more rigid.

For the most part, in connecting the various sections together, use was made of butt welds. The arrangement of the erection joints is shown in Fig. 3.

All the girders were assembled and welded on the ground, so far as possible in a horizontal position, before being offered up into their final positions and the last of the joints closed with them vertical. With a view to this procedure the erection welds were so arranged as to be easily accessible while being carried out.

The guiding idea in the design of the doorway was to reduce the number of site joints to a minimum. It was found possible to deliver the end verticals of the frame in single pieces, but the upper boom member (Fig. 4) was too long and too high for this to be done and had to be divided into a number of sections. In order to facilitate the assembly of these the lozenge type of

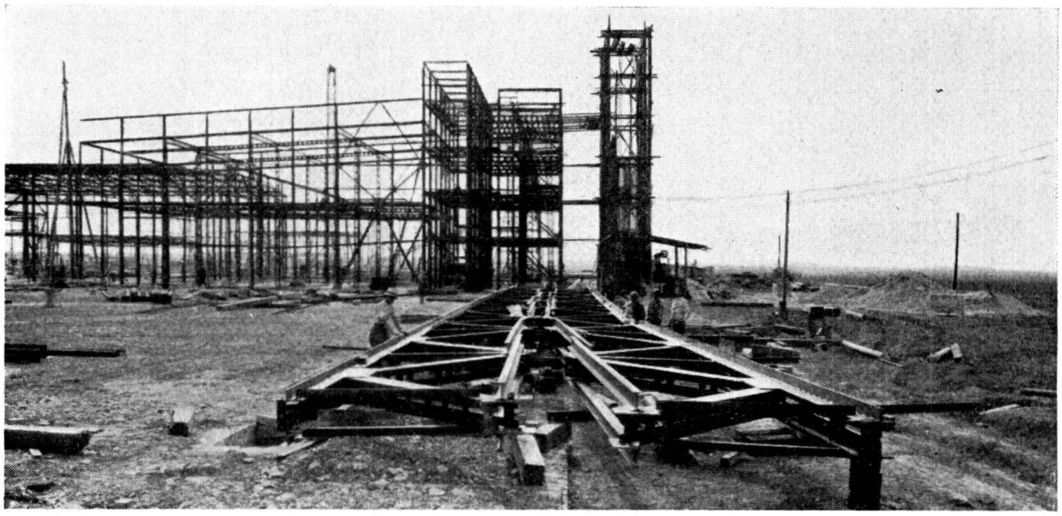

Fig. 5.

girder (Fig. 3) was adopted, thus reducing the number of sections to eight as against the 27 which would have been required with the usual triangular form (Fig. 3B).

With a view to rigidity combined with ease of transport, the various constituent parts were fitted to temporary boom members attached to central gussets, and, in an ingenious way, the roof trusses of the hangar were made to serve as booms (Fig. 5). The erection joists were placed in the boom members and in the central gussets as may be clearly seen in Fig. 3. The intersection of the diagonals is of some interest (Fig. 6). To facilitate the welding of the lower gusset the corresponding upper plate was notched, and its triangular complement was welded to the upper plate after the welding of the lower plate had been completed.

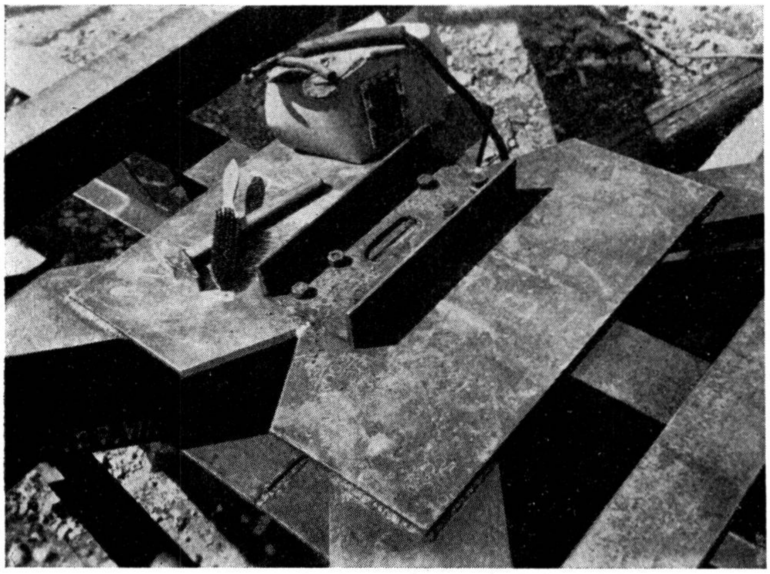

Fig. 6.

First the horizontal portions of the frame were assembled and clamped together with the aid of bolts in their correct positions; then the joints were tacked and completely welded in turn. Finally the auxiliary boom members were removed, and the footings were assembled and welded. The process of erection was begun by fitting the frames over the doorway, the whole frame, covering one span of 50 m and weighing 41.0 tonnes, being placed in position with the aid of erecting towers (Fig. 7), an operation which took four hours. The erection of the framework was then carried out in the usual way. Both in the shop and on the site the welding was done by means of direct current welding sets.

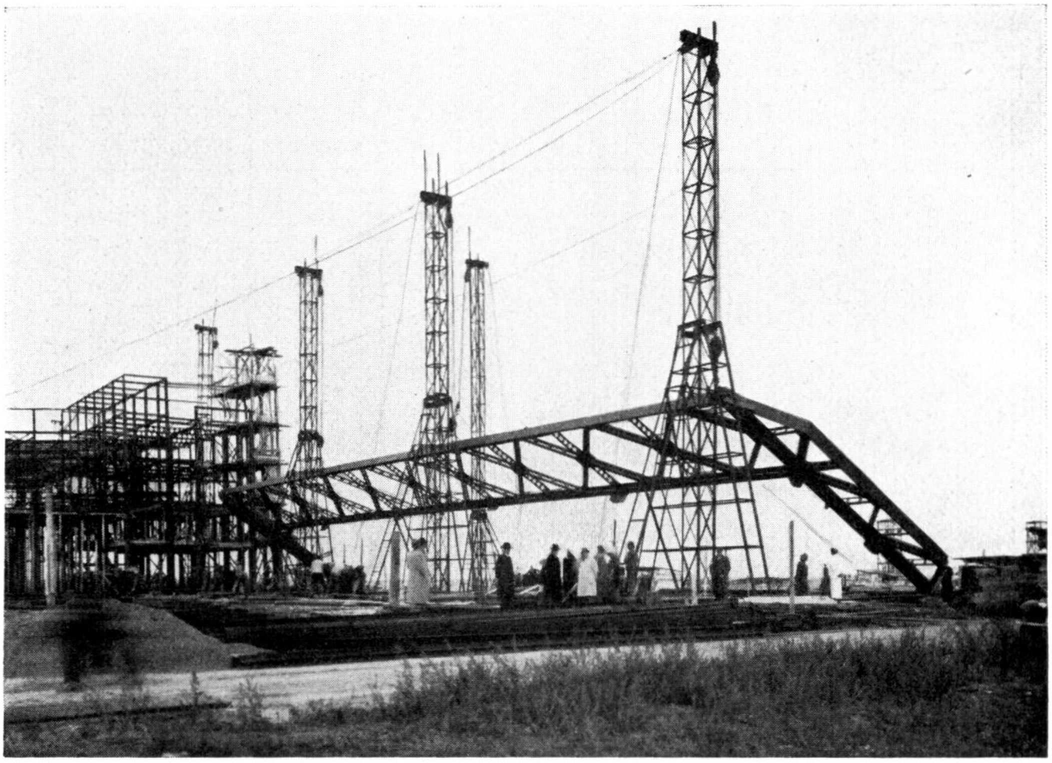

Fig. 7.

The construction of the steel frames was divided between the following two firms: —

S.A. des Anciens Etablissements Škoda, Pilsen.

Českomaravská-Kolben-Daněk, Prague.

The first mentioned firm made use of Böhler-B-Elite-KVA electrodes for welds of Type I, and of Arcos Stabilend electrodes for welds of Type II. The second firm made use of Elarc-Resistenz electrodes exclusively.

The average results obtained from the test specimens for electrodes, welds and welders are given in Table V, and against these results the prescribed minimum values have been included for comparison. Altogether 42 welders were tested in this way. It appears from the table that the minimum values, despite the high standard set, were easily obtained. The welding was closely supervised during the work, and on completion the welds were subjected to careful check and their dimensions accurately measured. A number of them were examined internally after drilling.

The design for the framework was carried out by the S.A. des Anciens Etablissements Škoda, Pilsen, with special attention to simplicity of execution both in the shops and on the job. The work was carried out under the control of the Bridge Department of the Ministry of Public Works.

Table V.

Acceptance test			Types of Electrodes				
			I		II		
			Böhler B-Elite	Minimum required	Arcos stabil.	Elarc Resist.	Minimum required
for electrodes	elastic limit, in kg/mm^2		30.9	23	35.0	40.0	26
	tensile strength, in kg/mm^2		46.5	38	46.3	48.7	42
	elongation, %		21.6	12	24.9	23.6	20
	resilience, in kgm/mm^2		4.3	3	8.5	9.7	6
for welds	tensile strength, in kg/mm^2		44.7	38	48.5	46.3	42
	shear strength, in kg/mm^2		34.1	28	34.6	37.1	30
for welders	tensile strength in kg/mm^2	horizontal posn.	47.2	—	49.6	46.8	—
		vertical „	42.2	—	47.9	48.0	—
		overhead „	43.8	—	50.5	47.0	—
		average „	44.5	34	49.5	47.3	40
	shear strength in kg/mm^2	horizontal „	33.3	—	33.3	35.7	—
		vertical „	33.7	—	35.7	36.1	—
		overhead „	31.3	—	35.1	34.2	—
		average „	33.1	26	34.8	35.3	29
for welds	I-shaped: tensile strength, in kg/mm^2		—	—	46.6	47.6	42
	V-shaped: tensile strength, in kg/mm^2		—	—	58.9	42.4	42

Another very large job was the welded construction of a road bridge of 52.005 m span (Fig. 8). Here the main girders were of the Vierendeel type without diagonals, this design being chosen mainly on aesthetic grounds but also because of the advantages it offers from the point of view of welding, and for the simplicity and rigidity of the intersections. Moreover in such a girder the secondary stresses are nil, whereas in a triangulated system they may vary between 10 to 15 % of the principal stresses, owing to the large sizes of gussets necessary at the intersections and to the system of calculation which has to be used.

Hence, using the same permissible stresses in the calculations, the true factor of safety is greater in the case of the Vierendeel girder, and finally bridges with Vierendeel main girders deflect much less than those with triangulated main girders on account of the great rigidity possessed by the intersections — a fact which is very important from the point of view of maintenance.

Hitherto the sole disadvantage attending the use of Vierendeel girders has been the difficulty of the statical calculations involved, but the Beggs-Blazek method of determining the influence line has completely removed this difficulty.[1] The advantage of this method lies in the fact that it is no longer necessary to rely on simplified assumptions and that the additional rigidity due to the fixation of the vertical members is automatically taken into account. The influence line can be accurately determined at any required point, and this makes it easy to check the conditions of stability.

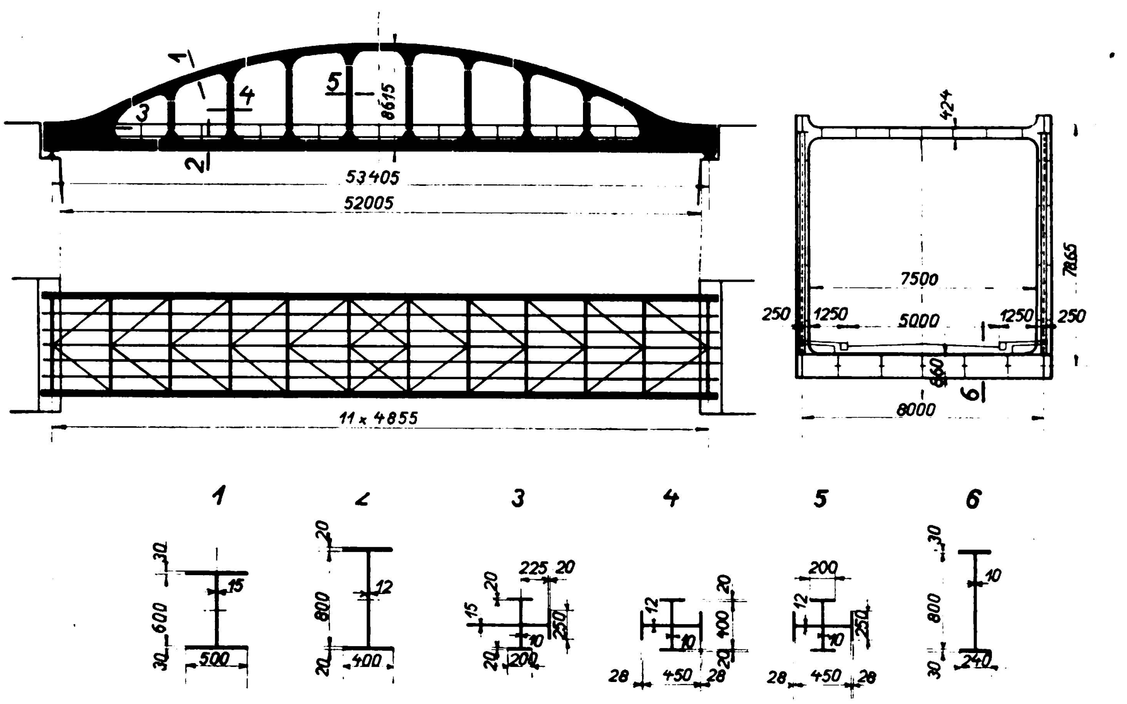

Fig. 8.

The girder is designed as hyperstatic to the 33[rd] degree.

In addition the results obtained in this way were checked by reference to an approximate calculation in which it was assumed that the moments of inertia of the booms were constant in all panels and dependent on the length of the bars; at the same time it was assumed that no loads were applied except in line with the vertical members. These assumptions reduced the degree of hyperstaticity to 11 and rendered the calculations easier. The whole of this work was carried out in ordinary steel C 38 and entirely by the use of welding both in the shop and on the site. The welding was done exclusively with Arcos Stabilend electrodes.

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[1] Final Report, 1[st] Congress, I.A.B.S.E., p. 709.

The permissible stresses both for the parent steel and for the weld metal are indicated in Table VI.

Table VI.

Permissible stresses	Decking members		Main girder	
	Parent metal	Weld metal	Parent metal	Weld metal
Tension	$v = 850$ kg/cm²	0,75 v	870 + 3 l — maximum 1150 kg/cm² (1350 kg/cm²)	0.85 v
Compression		0,90 v		1.00 v
Shear	$\tau = 700$ (800) kg/cm²	0,50 v	700 (800) kg/cm²	0.60 v

Note. The values shown in brackets relate to the case where the calculations take account of all external forces (wind pressure).

For all the connecting welds the butt type has been preferred, and intersecting joints exposed to tensile stresses have been avoided on principle. Bracing members are connected to the verticals by means of butt welds. In order to avoid crowding the welds together the stiffeners of the bracings, booms and verticals have been holed at the angles, and this assists drainage.

The weight of the steel portion is 154 tonnes. The erection joints were arranged in such a way that the pieces could be delivered as large as possible (Fig. 8). The ends of the main girders, which are 9.293 m long and weigh 6.7 tonnes, were delivered to the site of the work in a single piece (Fig. 9).

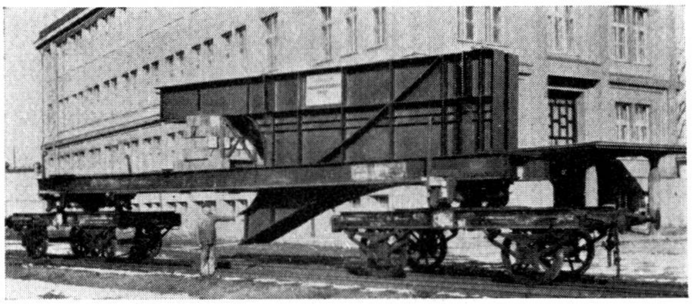

Fig. 9.

As the bridge was to be erected in spring time at the period of high water, the original intention was to carry out the welding immediately after the temporary erection by means of bolts had been completed, so that the welded points would be able, if necessary, to bear the dead weight of the structure in the event of the supporting falsework being damaged by the flood. The floor of the bridge was then to be welded in three sections, so as to lessen the stresses due to welding. The favourable weather encountered made it possible to modify these arrangements by welding the floor of the bridge to the lower boom members straight away, a procedure which helped to prevent the stresses due to the welding of the floor being transmitted to the main girders. Finally the verticals and other boom members were erected as soon as they had been welded in the shop (Figs. 10 to 12).

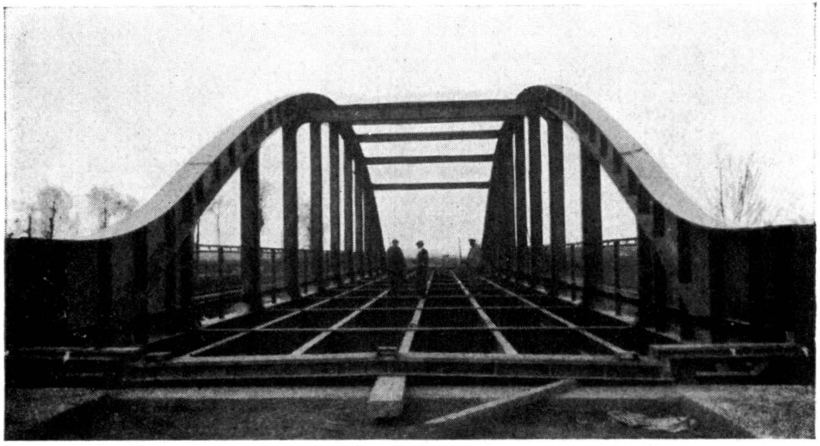

Fig. 10.

The main girders were given a maximum camber of 15 mm corresponding to the deflection under dead load together with half the life load.

In addition to the usual tests of steel, electrodes, welds and welders, fatigue tests were carried out. The fatigue limit for the weld was determined from *Wöhler*'s curve after carrying out eight tests to two million alternations of stress at 22 kg/mm² and also ten million alternations to 20.5 kg/mm². The tests were made on conical specimens in an Amsler fatigue testing machine.

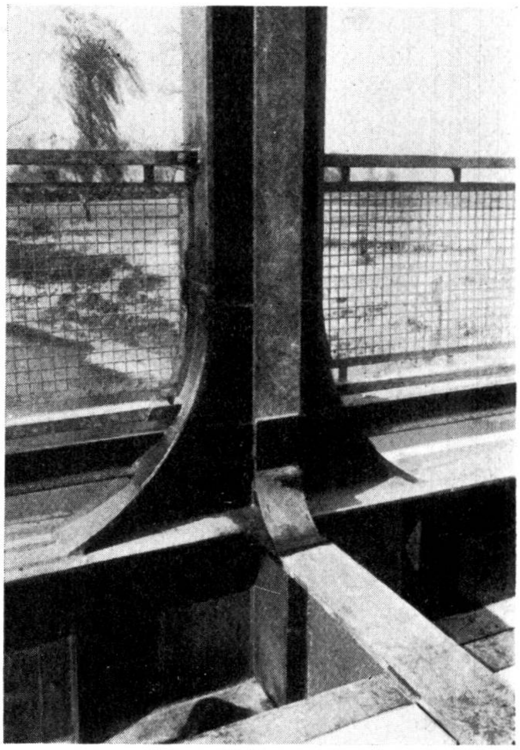

Fig. 11.

It further seemed advisable to carry out X-ray tests of the welds on a portion of the lower boom of the main girders (Fig. 13), and a model of the intersection of the lower boom was subjected to statical tests. These were carried out in the Laboratory for Testing Materials and Structures of the Czech College at Prague, enabling the stresses due to permanent loading and to assumed uniformly distributed live load to be calculated. Eventually it is intended to subject an intersection of this kind to fatigue test. When the whole structure is completed deflectometers will be applied to measure the deflections of the cross girders and main girders under stationary and moving loads.

The design of the bridge as a whole was carried out in the Bridge Department of the Ministry of Public Works, and the detailing of the final design as well as the construction of the work were entrusted to the S.A. des Anciens

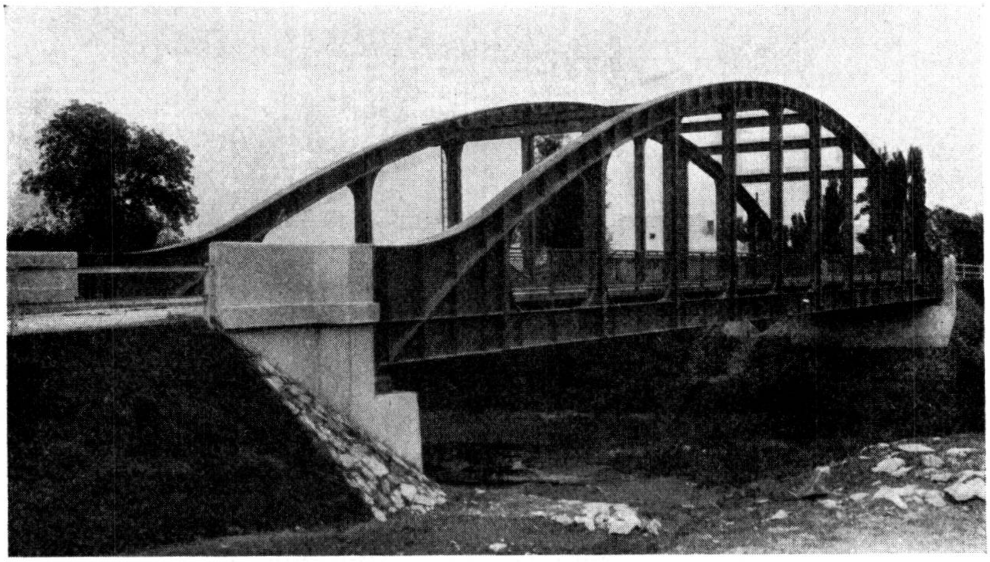

Fig. 12.

Etablissements Škoda, Pilsen, who completed the task to the entire satisfaction of the Ministry. The Bridge Department of the Ministry of Public Works were responsible for supervision.

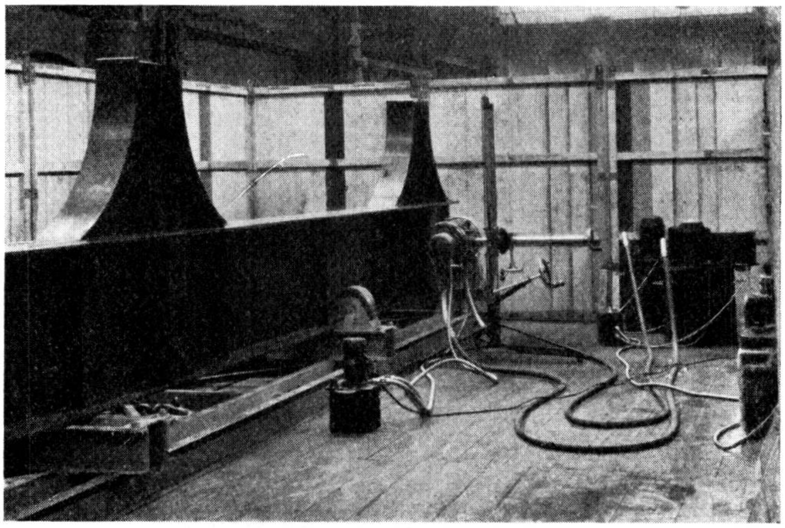

Fig. 13.

Finally Fig. 14 and Table VII give particulars of the principal welded road bridges in Czecho-Slovakia completed up to the present time.

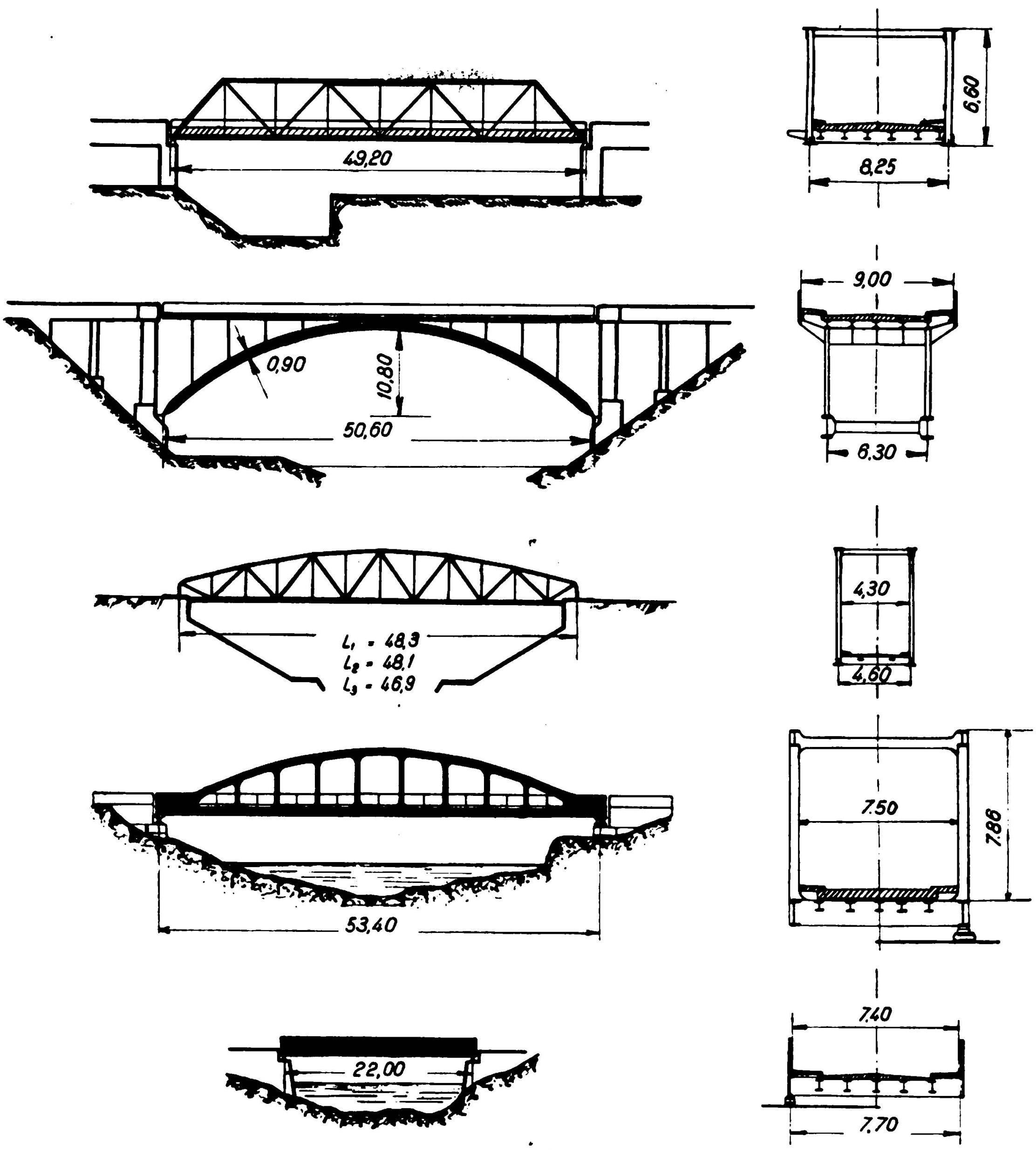

Fig. 14.

Table VII.

Bridge No.	Year of Construction	Span in m	Loading	Weight in tonnes	Construction
1.	1931	49.20	Class I	145.0	Škoda Works, Pilsen
2.	1933	50.60	„ I	111.0	„ „ „
3.	1933	22.00	„ I	37.6	„ „ „
4.	1934	48.30	„ III	52.0	Českomaravska-Kolben — Daněk
		48.10	„ III	52.0	Brno-Kralovopolská
		46.90	„ III	49.1	Škoda Works, Pilsen
5.	1936	53.40	„ I	157.0	„ „ „

Note: Class I corresponds to a uniformly distributed live load of 500 kg/m² or to a road roller weighing 22 tonnes.

Class III corresponds to a uniformly distributed live load of 340 kg/m² or to a wagon weighing 4 tonnes.