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VII

Application of steel in Bridge and structural engineering and in hydraulic construction.

Anwendung des Stahles im Brückenbau, Hochbau und Wasserbau.

Application de l'acier dans la construction des ponts et charpentes et dans la construction hydraulique.

VIIa

Application of steel in bridge and structural engineering.

Anwendung des Stahles im Brückenbau und Hochbau.

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VIIa 1

Progress made in Belgium from 1932 to 1936 in the Use of Steel.

Fortschritte in der Anwendung des Stahles in Belgien 1932 bis 1936.

Progrès réalisés de 1932 à 1936 dans l'application de l'acier en Belgique.

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It is not our intention to give in this report a general review of the uses of steel in bridge building and structural engineering in Belgium during the period 1932—1936. We shall confine ourselves to outlining certain original and significant features of the progress made in our country in the field of steel construction during these years, as for example, innovations as regards method and technique more typical than those which have been witnessed for several decades. Although we may consider that the initiative for this progress emanated from ourselves — a fact that is also generally acknowledged — we feel that we must refrain from framing our explanations in a subjective light. We do not intend either to propound theses or to refute criticisms, but primarily to deal with structures that have been actually carried out. If the new ideas whose application we elucitade claim too much attention, it is because, of their nature. Our own credit lies only in that we have worked reasonably, in conformity with the laws of Nature; that we have been able to overcome the difficulties we have encountered without any help but that of engineering.

The progress achieved may be divided into the following stages; in the first we are obliged to go back a year or two in advance of the period under consideration.

1930—31 Construction of the riveted multiple-frame steel skeleton of the Institut de Chimie-Métallurgie, Val-Benoit, Liége.

1931—32 Construction of the Lanaye bridge, 68 m span, welded in workshop, riveted at site.

1932 Construction of the entirely welded multiple-frame steel skeleton of the Institut du Génie Civil, Val-Benoit, Liége.

1933 Construction of Bridge C at Hérenthals, span 57.50 m, entirely welded.

Each of these constructions was the first of its kind in Belgium, and they introduced there the systems of multiple-frame steel skeletons with rigid panel-points, either riveted or welded, and the Vierendeel road bridges, at first partly and soon afterwards entirely welded. As far as we know, at the time of their conception and construction, these structures had no equivalents in other countries either. These bridges and other structures have certain elements in common, namely, the rigid panel-points forming the principal riveted, welded or combined connections. They determine the evolution and the chronological order of the structures mentioned. Without departing from basic principle, their forms have altered in a characteristic manner in the course of the stages cited, gradually producing types that have been since reproduced again and again without any essential modifications.

One of the present authors has already drawn up a detailed report on the subject of rigid panel-points; we would refer the reader to this and confine our attention in this paper to the character of constructions actually carried out with the system of rigid panel-points.

Vierendeel Road Bridges.

Since 1896 Mr. A. Vierendeel, engineer and professor, has continuously been working on the task of superseding triangulated lattice girders by girders with arcaded webs, without diagonals. The first forms given to this type were various, and most of them have since been discarded. A certain number of bridges of this construction were executed in steel before 1931, but, on the strength of the experience gained with the Lanaye bridge and C bridge at Hérenthals, more Vierendeel road bridges have been constructed between 1933 and 1936 in Belgium and throughout the world than in the period from 1896 to 1933. It is even possible that this result has encouraged the construction in Belgium of a large number of long-span Vierendeel railway bridges, all riveted with the exception of the Val-Benoit bridge over the Meuse at Liége. In this, one of the most recent constructions, electric arc welding was locally and partially employed.

The Lanaye bridge, as constructed, represents the result of a counter-project brought forward in 1931 at the time when Prof. A. Vierendeel submitted his project of a riveted road bridge. The Société Métallurgique d'Enghien Saint-Eloi proposed the substitution of welding for workshop riveting. The Société did not deem it possible at that time to employ welding at site as well as in the workshop, and riveting was retained for erection purposes. Nevertheless this workshop welding, and particularly that of the panel-points, had the following advantages. It permitted

1) a considerable saving in labour by eliminating all hand riveting and by substituting broad-flanged rolled sections for composite girders forming the booms, uprights and traverses, except in the case of the upper boom, which was constructed of a double T-section and welded.

2) a reduction in weight of about 10%. This saving did not reach the maximum; in subsequent constructions the same designer effected savings in weight of from 20 to 25%. The reason for this lay in the fact that pioneer constructions always involve difficulties outside the actual technical sphere,

while in addition there were at that time no broad-flanged sections to be obtained in Belgium; these only began to be manufactured after their efficiency had been demonstrated in the Lanaye bridge.

3) a reduction in the cost of construction owing to the two foregoing factors. The saving in cost, too, was not extremely great in the case of the Lanaye bridge, a fact which is easily explained by the cost of preliminary studies and tests and by the labour entailed in adapting the workshop to the new requirements. As against subsequent projects, however, the difference in price between a welded and a riveted construction proved to be so great (approximately 15%) that riveting has been practically abandoned in the construction of road bridges in Belgium.

The innovation offered such advantages that it soon came to be looked upon as a definite advance and was eventually approved in principle after a preliminary test had been made with a scale-model welded panel-point, as already described at the 1st. International Congress for Bridge and Structural Engineering, held in Paris in 1932. However, the novelty of the innovation was such that all the designer's proposals were not accepted, certain alterations in the dimensions being required, and the stipulation made that a number of details of the initial project be retained. The harmony of the structure suffered under these restrictions, it has a hybrid character which detracts both from its composition and from its appearance. Subsequent experience has shown that it would have been better to adopt the more homogeneous details of construction proposed by the designer. It may be said, however, that the real innovation was at once accepted, namely, the welded panel-points with their characteristic influences on the elements (booms and uprights) of the girders and on the constructive members of the decking (Final Report of the First Congress 1932, p. 258, Figs. 8 and 9).

The tests carried out in May 1933 on the bridge itself proved just as satisfactory as those made with the model panel-point (see *Santilman:* Annales des Travaux Publics de Belgique, December 1933).

The construction of the bridge at Lanaye was succeeded by that of five bridges of the same type as regards the main idea: panel-points welded in the workshop and riveted at site. On the strength of the favourable experience made with the Lanaye bridge, however, their construction became more homogeneous as regards the wind-bracing, decking beams, latticework of the uprights, etc.

One of the first of these bridges following the construction of the Lanaye bridge, was the swing bridge over the Muide at Gand (*M. Storrer* and *A. Spoliansky*. L'Ossature métallique 1933. See also *Hawranek,* Bewegliche Brücken 1936). At the present time it is still the largest movable welded bridge in the world.

We may also mention the Schooten bridges (No. 39 and 40), constructed in 1933—34 and the Lanklaer and Lanaeken bridges, also constructed at the same period.

Having gained experience of this type of construction, the workshop that executed the bridge did not hesitate in 1933, when projects were invited for bridge C at Hérenthals, to submit plans for an all-welded bridge as against Prof. A. Vierendeel's project for an all-riveted construction. This counter-project was adopted by the Department for Roads and Bridges because of its technical and economic advantages. The all-welding process completed within lees than three years the evolution begun by the Lanaye bridge. The project for the Lanaye bridge as executed was conceived at the end of 1931; the all-welded

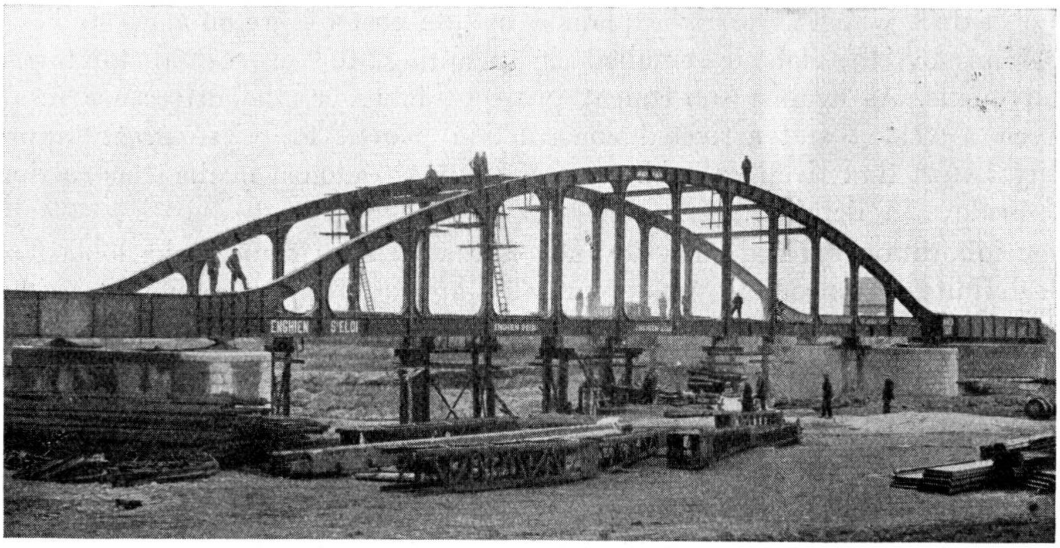


Fig. 1.
Bridge C in Hérenthals (1933 - 34).

steelwork of bridge C at Hérenthals was completed in February 1934 (*A. Spoliansky.* L'Ossature métallique 1934). Figs. 1, 2 and 3 illustrate three characteristic aspects of the steel structure.

Since welding has come to be more generally employed it has become possible to confirm the advantages of the technical features already used in the con-

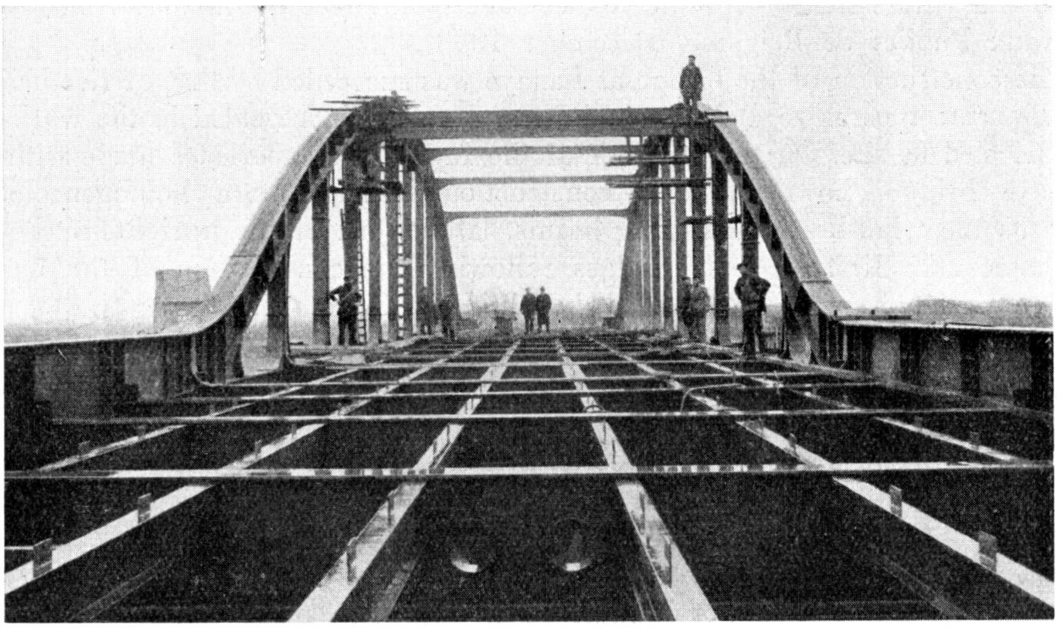


Fig. 2.
Bridge C in Hérenthals (1933—34).

struction of the Lanaye bridge. Their full description and analysis will be found in the paper by *A. Spoliansky* in Vol. 3 of the Publications of the I.A.B.St.E., 1935. (Les ponts soudés en Belgique. Revue Universelle des Mines, Vol. XI, N° 8, 1935. See also L'Ossature métallique 1935 and 1936.)

Bridge C at Hérenthals has since become the prototype for numerous structures in Belgium, particularly for bridge A at Hérenthals (constructed at the

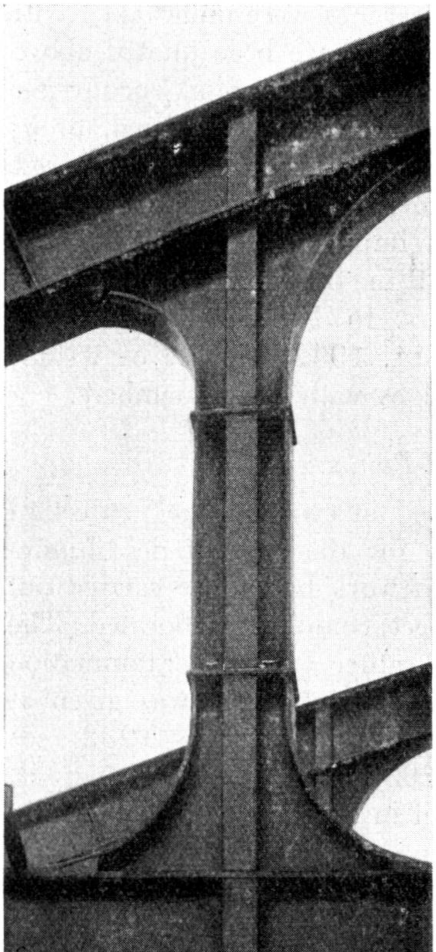


Fig. 3.
Bridge C in Hérenthals (1933—34).

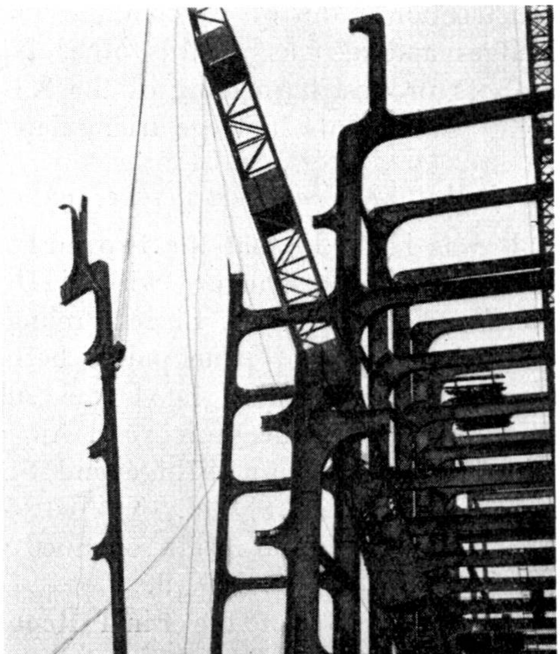


Fig. 4.
Building of the Chemical-metallurgical Institute
Main entrance details (1930—31).

same period), for the Bocholt (1934) and Dilsen bridges, etc. (8 bridges over the Albert Canal 1934—35), at Gheel and Oolen (4 bridges 1934—35), 2 bridges at Dudzeele (1934—35), at d'Eygenbilsen (1935), at Sutendael (1935), 2 bridges at Monsin (1935), at Vivegnis, d'Hermalle and Haccourt (3 bridges of 90 m span, 1936); in short, more than thirty-one bridges in Belgium. Those at Nuth in Holland and Michalovce in Czechoslovakia were also constructed exactly after the original design.

These various structures only differ from one another in details which do not alter their fundamental analogy to the prototypes. All the bridges in Belgium were carried out in Etat belge 42/50 type steel. The rapid multiplication of this system to such a prodigious extent constitutes a real innovation in steel construction. It has raised a large number of important problems, both from a designing and constructional point of view and from that of execution and organisation in the workshop and at the site. It has created in the Belgian steel construction industry a spirit of activity and emulation which is evident in the amount of research work carried out. The first researches were undertaken with modest resources and along general lines; their results have been quoted above. At the present time more detailed research work is in progress, encouraged perhaps by the indisputable and rapid developments already mentioned, or by the observations made during construction and the difficulties encountered. Among these difficulties should be mentioned deformations due to heat in the welded members. It is to be expected that this new impulse will enable the steel construction industry to continue its development in the domain of railway bridges and also as regards other types of girders. In 1932 one of the two authors proposed (meeting of the A.B.E.M., May 11, 1932) the use of welded rigid panel-points in large triangulated lattice bridges with rigid members.

Steel Multiple-Frame Structures with Rigid Panel-Points.

It was in 1929 that the two authors collaborated in evolving the project of the riveted steel structure with rigid panel-points for the Institut de Chimie-Métallurgie, Val-Benoit, Liége. Preliminary research work had to be carried out on scale models of panel-points before this project could be elaborated. The structure, completed in 1931, was subjected to a rather extensive examination in the course of the same year. An account of this investigation was given at the 1st. Congress for Bridge and Structural Engineering in Paris 1932 (see also R.U.M. Series 8, Vol. IX, Nos. 5, 6 and 7, 1933).

A summary of the results obtained will be useful in enabling the reader fully to appreciate what followed.

Fig. 6, p. 534 of the Final Report of the lst. Congress in Paris, 1932, reproduces the diagrams of measured and calculated tensions in respect to loadings of a girder, clearly shown in the illustration. The remarkable agreement between the respective curves of these diagrams will be noted, and the characteristic feature that the measured tensions are lower than the calculated ones. The comparison becomes more striking when a study is made of the transverse deformations (flexions) and angular deformations (rotation of the panel-points) and of the extent to which the girder is encastret at its ends.

Angular deformations of panel-points 1 and 1′

Test	Calculated values	Measured values	Difference in %
No. 1	$225 \cdot 10^{-6}$	$166.90 \cdot 10^{-6}$	26
No. 2	$246 \cdot 10^{-6}$	$174.39 \cdot 10^{-6}$	29

Deflections ad middle of beam 1 and 1′

Test	Calculated values	Measured values	Difference in %
No. 1	8.49 mm	11.70 mm	27
No. 2	11.48 mm	14.40 mm	20

Degrees of restraint at panel-points 1 and 1'

Test	Calculated values	Values received by measurement
No. 1	0.965	0.975
No. 2	0.970	0.980

These degrees of restraint are real degrees, corresponding to the relation between the moment of restraint and the angular deformation. If these coefficients are determined as functions of deflection or moments of maximum binding of the girders, it will be found that approximately 1.10 is given for Test No. 1, and about 1.08 for Test No. 2, values devoid of physical significance (see reference literature).

The differences arise chiefly from the imperfect method of calculation, which regards the members as being prismatic between their point of axial intersection and neglect the stiffening influence of convergent members and of gusset plates. This is revealed by a more elaborate method of calculation taking into consideration the reinforcement of the beams by the panel-points, assuming the presence of a mean fictive moment of inertia, exaggerated in value. The differences in respect to the measurements are substantially reduced, as is shown by comparison between the degrees of restraint at the ends of the beams. The values found for the real degrees of restraint at the panel-points 1 and 1' are as follows:

Test	Values obtained by measurement	Calculated values
No. 1	0.971	0.968
No. 2	0.974	0.969

Based on the deformations at the middle of the beam 1—1', the calculated values become 1.04 for Test Nr. 1 and 1.015 for Test No. 2.

These results also show the quasi invariability of the degree of restraint of these structures; this factor, however, cannot be applied theoretically in a rigorous manner. In fact, the degree of restraint is determined practically as being equal to one.

Figure 4 shows a characteristic and novel feature of this type of structure.

The steelwork of the Institut du Génie Civil, Val-Benoit, Liége, constructed by one of the two authors, was executed in 1932—33. Begun after the Lanaye bridge, it was completed before the latter. Although the principle of rigid panel-points employed in the preceding structure was adhered to, improvements in form were effected and the construction was entirely welded; besides this, on the energetic initiative of the firm S.A. d'Ougrée-Marihaye, a special 58—65 kg/cm² steel was used. This structure, remarkable from three points of view, has not yet been described in detail, and space is not available for the purpose here. Fig. 5 illustrates the type of truss most frequently used in the structure. Not a single hole had to be bored in any part of the whole steelwork. Fig. 6 shows the principal results obtained from an examination of a truss of another type.

This examination was carried out under much more ideal conditions than those obtaining when the riveted steelwork of the Institut de Chimie-Métallurgie was examined. The loading was effected by means of a hydraulic press acting on two successive beams forming part of a single frame. This arrangement permitted the tests to be repeated in a very short time by a simple manipulation and progressive application, and without abrupt loading. The fact that this stressing was of an exceptional nature not allowed for in the design of the structure, was of no importance. The stressing itself was easy to calculate and the results enabled a comparison to be made quite simply with those of the examination. The arrangement adopted increased the general precision of the results of the examination, the stresses exerted by the hydraulic press being exactly determined by means of very accurate manometers. The method was so convenient that practically the whole truss system could be examined. Fig. 6 shows the diagrams of the two extreme fibre stresses, calculated and measured,

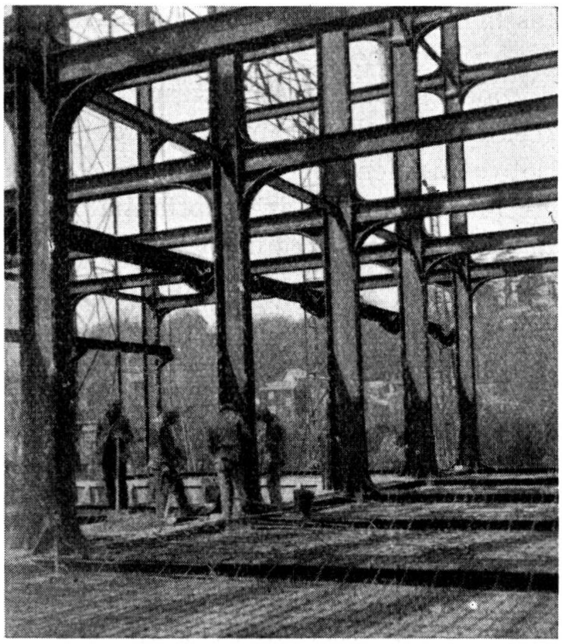


Fig. 5.
Steel structure for the Civil Engineering Institute.

for all the bars. The general agreement of the curves is very striking. The differences are much smaller in comparison with the examination of the riveted steelwork of the Institut de Chimie-Métallurgie (7% instead of 13%) because the welded panel-points are much less complicated. Nevertheless, the maximum measured stresses always keep lower than those calculated, for reasons already explained. We do not know of any other examination of an actual steel structure giving such good agreement between calculation and reality. A slight disturbance of the upper beam will be noticed, emanating from a longitudinal supporting a monorail track and welded to the members of the neighbouring trusses. Its action was computed as accurately as possible and allowed for in the calculation. Better agreement was obtained by doing so, although the curve is still imperfect.

On other occasions we have ascentained that disturbances often looked upon as secondary influences frequently emanated from similar causes. Their action does not greatly affect the precision of the structure.

The following results were obtained for the calculated deformations as against those measured:

Angular deformations at panel-points

Point	Calculated values	Measured values	Differences in %
1	$3525 \cdot 10^{-6}$	$2770 \cdot 10^{-6}$	21.5
1′	$-4377 \cdot 10^{-6}$	$-3547 \cdot 10^{-6}$	19
2	$-3701 \cdot 10^{-6}$	$-3215 \cdot 10^{-6}$	13
2′	$4326 \cdot 10^{-6}$	$2935 \cdot 10^{-6}$	32
0	0	$-496 \cdot 10^{-6}$	
0′	0	$473 \cdot 10^{-6}$	

Deflections at middle of beams

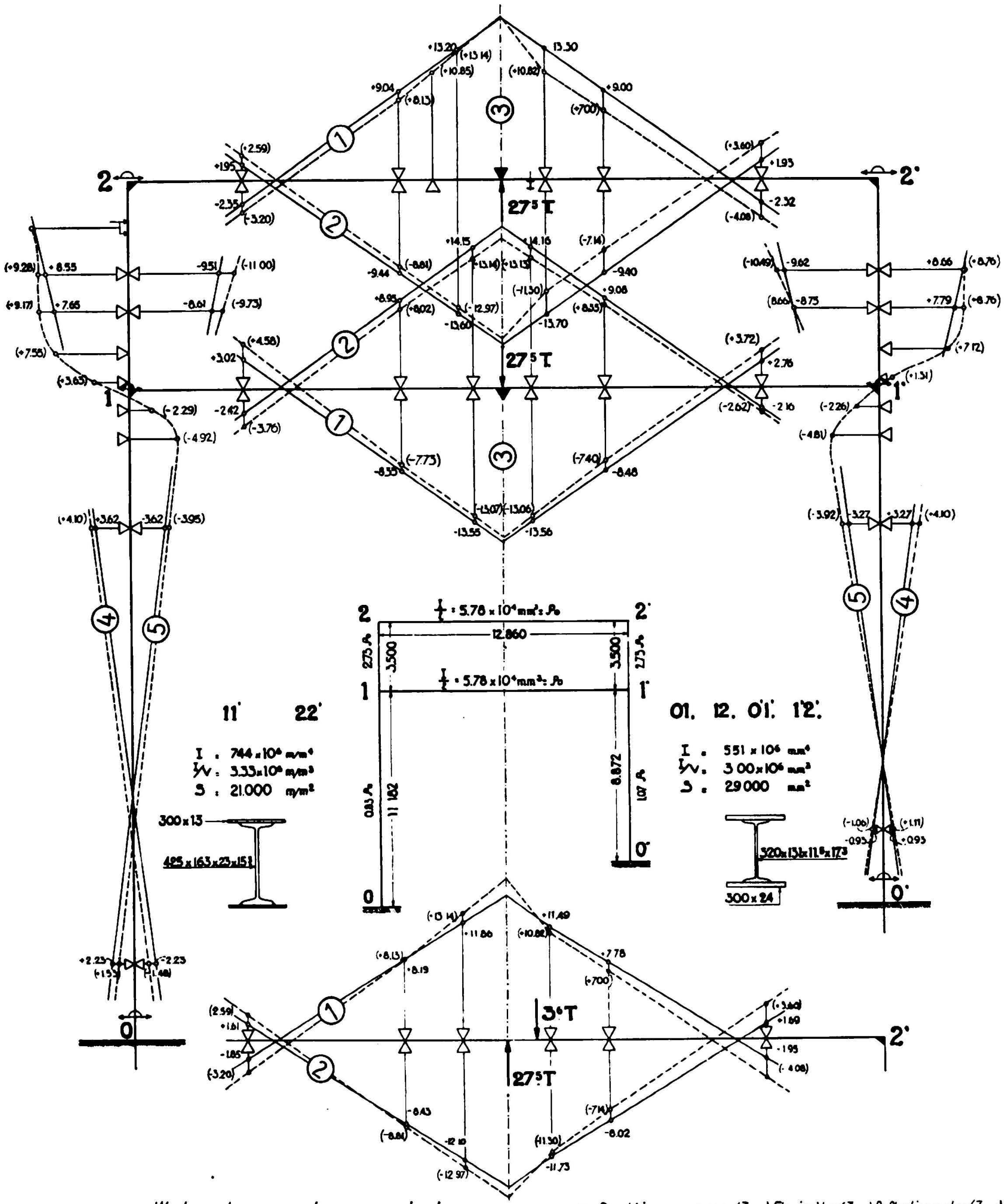
Point	Calculated values	Measured values	Differences in %
1 1′	31 mm	24.91 mm	19.7
2 2′	33.45 mm	23.88 mm	28.6

It will be noticed that the differences for deformations are on an average just as great as those in the riveted structure of the Institut de Chimie-Métallurgie, although the stress differences are less. There can be no doubt that the stress measurements taken in the second examination are more accurate. Moreover, it is probable that welded joints are less subject to deformation. The difference between the results for loadings N° 1 and N° 2 in the riveted steelwork of the Institut de Chimie-Métallurgie would seem to indicate a slight slippage of the riveting.

The above figures show that the hypothesis of perfect restraint of the column bases O and O′ on which the calculation is founded, is inexact. These bases O and O′ have undergone slight but measureable rotation.

We would remark that, more likely than not, this is due to the manner of loading, which does not stress the columns supporting only the bases of the truss, hydraulic press and testing platform. For real loads it is most probable that the restraint of the bases would have revealed better action. When carrying out an examination, it is possible to make allowance in calculation for the effects of measured rotation. This increases the differences between the calculated and the measured elements and tends to give similarity with those found in the examination of the Institut de Chimie-Métallurgie steelwork, where the restraint of the column bases was observed to be practically perfect. Finally, it should be noted that for the stressing to which truss IV of the Institut du Génie Civil steelwork was to be subjected, the assumed degree of restraint is again theoretically less accurate than in the case of the other examination described, though practically it is present with its full value. It would be impracticable, as well as beyond the projected scope of this paper, to discuss these points in detail here.

In our opinion, the above results are in themselves eloquent enough to be adequately appreciated without further comment on our part. However, it would certainly be interesting with a view to checking the principles of the calculations, to elaborate our analysis still further; this will be done in a subsequent paper. Here we shall confine ourselves to drawing the conclusion that these two successive experiments prove that entire confidence can be placed in steelwork of this type provided it is well designed and well executed; that they demonstrate the practical possibility of calculating these structures to any useful degree of accuracy without being obliged to carry the calculation to extremes



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—— *gerechnete Werte · valeurs calculées · calculated values*
\+ *Druckspannungen · compressions · compressive stresses*
− *Zugspannungen · extensions · tensile stresses*
▽ *Akustischer Dehnungsmesser Schaeffer (Maihak) · Extensomètre acoustique Schaeffer (Maihak) · Acoustic tensometer Schaeffer (Maihak)*
Klinometer (Stoppani) · Clinomètre (Stoppani) · Clinometer (Stoppani)

▼ *Durchbiegungsmesser (Zivy) · Fleximètre (Zivy) · Deflectionmeter (Zivy)*
① *unterer Rand · bord inférieur · lower edge*
② *oberer Rand · bord supérieur · upper edge*
③ *2 Durchbiegungsmesser · 2 Fleximètres · 2 Deflectionmeters*
④ *innerer Rand · bord intérieur · inner edge*
⑤ *äusserer Rand · bord extérieur · outer edge*

Fig. 6.

Tests executed on the structure of the Hydraulic Laboratory.

of precision. Simplified methods are perfectly legitimate provided they keep to the principle of the construction — in this connection common sense will prove more valuable than theory.

The riveted steelwork of the Institut de Chimie-Métallurgie weighed 24.8 kg/m³ of constructional volume (1817 tons), that of the Institut du Génie Civil 20.15 kg/m³ of constructional volume (595 tons). In 1933 one of the present authors was instrumental in the design and construction of a new all-welded structure of the same type, executed in special steel. The weight was reduced to 16.69 kg/m³ in the project. During building operations riveting

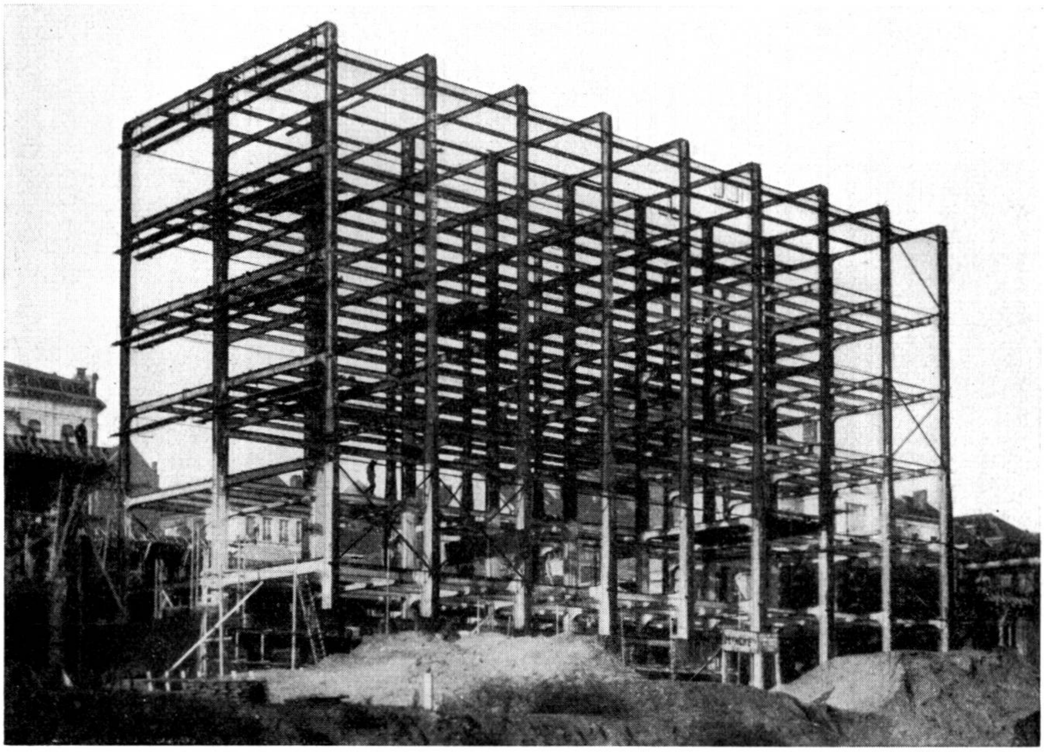

Fig. 7.
Steel structure of the Technical Laboratories of the Gent University (1935).

was substituted for welding. The weight finally worked out at 17.34 kg/m³ (146 tons).[1] In 1935 one of the authors elaborated a competitive project for the construction of technical laboratories for the University of Gand. It was accepted on the strength of a wide margin of superiority over the other projects submitted. The structure, which was of the above mentioned type, was executed in ordinary steel, all welded, and weighed 16.60 kg/m³ (414 tons).

It should be noted that the floor loads for this building were considerable (1000 and 3000 kg/m²). Moreover, joists were expressly stipulated with intervals of 2 m, each capable of carrying a partition wall. The result was that these elements had to be designed rather heavy (4.5 kg/m³). Figs. 7 and 8 show characteristic features of this steel skeleton.

[1] It should be noted that these three structures were the first in Belgium to be entirely encased in concrete.

In 1936 one of the authors submitted an alternative project for an important 7000 ton steel skeleton with continuous members and rigid panel-points, all welded and in special steel. The reduction in weight, price and amount of welding in respect to the primary project submitted was very substantial. The latter envisaged a dead weight of approximately 75 kg/m³, and a volume of welding of 80 electrodes per 100 kg of steelwork.

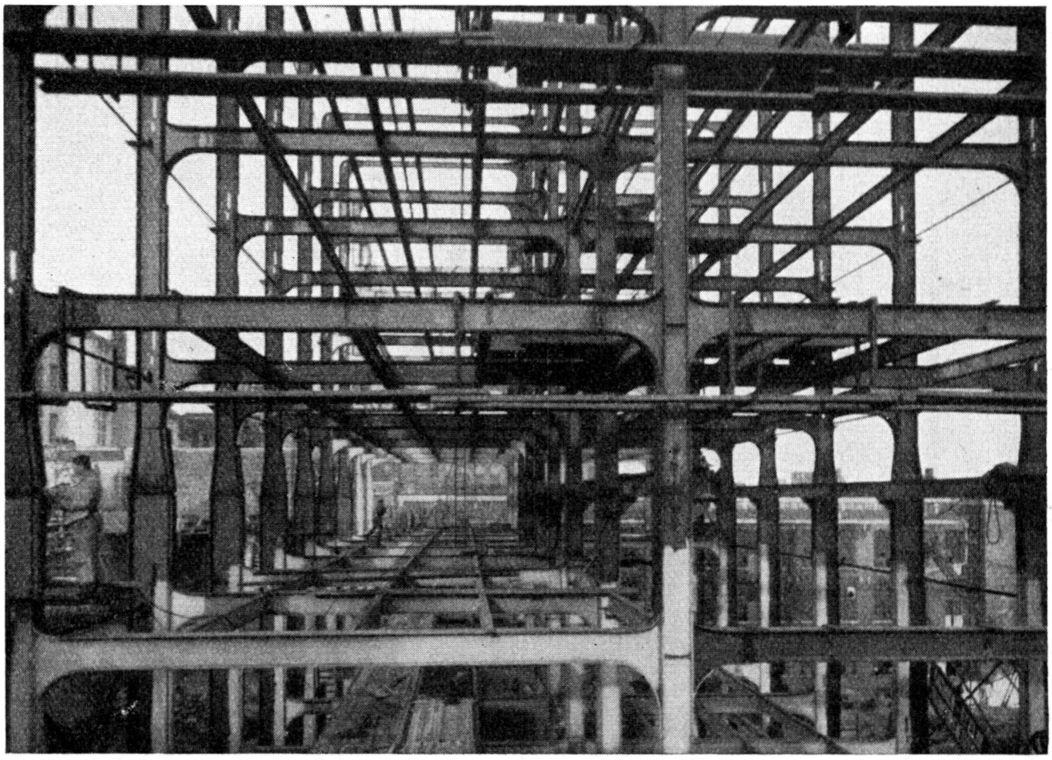

Fig. 8.

Steel structure of the Technical Laboratories of the Gent University (1935).

In the alternative project the respective figures were only 40 kg/m³ and 40 electrodes per 100 kg. It is evident that this reduction is due not merely to the employment of highly tensile steel, but also to more advantageous design. Besides, the unitary price per ton of the construction was practically the same in both projects, in spite of the great difference between the cost of normal and high-strength steel (for plates the increase is 70%). Another example is that of the Anti-Cancer Centre in Brussels, projects for which were submitted quite recently. For this 35 m high building a project similar to that of the technical laboratories of Gand University was placed first with a weight of approximately 16 kg/m³. The building, which will be executed on the principles described, is to be erected shortly.

We would remark that steel skeletons with continuous members and rigid panel-points as described in the foregoing have not been reproduced to the same extent as have the Vierendeel welded road bridges. The reason for this lies in the fact that, after prototypes of these bridges had been established, the Ministry of Bridges and Roads itself elaborated the projects for subsequent structures, thus conferring on the whole steel construction industry the benefits arising from the initiative of one workshop. Nothing of this kind occurred in the case of the steel skeletons; a single workshop took up this type of structure as its principal line, yet a number of the skeletons mentioned above had been constructed by other enterprises. Wherever it has entered into competition with other designs, however, its technical and economic advantages asserted themselves. Its ability to compete with reinforced concrete, combined with the progress which steel constructional work is likely to make as more and more experience is gained, would seem to predict a good future for this new type of structure. We might observe that the question of the form (shape) of the panel-points — to which we are giving our special attention in this case — is not essential. The principle of continuity, introduced by us into the construction of steel framework, does not require the rational and perfected type of panel-point that we have always employed. Other forms of points may be used, according to the designer's originality. But we have proved, by tests, work executed and the results obtained from an industrial and economic standpoint, that the rational form of panel-points we advocate is the best in every respect. Other types are technically less convenient and safe, and have not the same economic advantages. This latter fact was clearly proved on numerous occasions when it was a question of direct competition. A more detailed description of the principles will be found in the paper delivered by one of the present authors at the 2nd. National Scientific Congress at Brussels, 1935.

Summary.

The authors state that between 1932 and 1936 (or more properly 1930 to 1936), steel construction in Belgium was given a new impulse by the erection of a number of original structures designed on similar principles. These new buildings were multiframe skeletons with continuous members and rigid panel-points, either welded, riveted or both, and executed in ordinary or special steel; and furthermore Vierendeel road bridges partially or entirely welded.

The common features of these constructions, which were transferred and reproduced as the respective prototypes of the structures were elaborated, consist in rational types of rigid panel-points having as their object the realization of the most perfect properties of continuity in the structures.

The type of structure developed on this principle has been reproduced in numerous examples in Belgium and abroad. The authors describe the majority of these structures and refer to investigations which were carried out on some of them and which firmly established their technical advantages. Economically there is absolutely no comparison between these constructions and other types; the former have proved absolutely superior everywhere there has been direct competition.

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VII a 2

The Construction of the Kincardine-on-Forth Bridge.

Der Bau der Kincardine-on-Forth Brücke.

La construction du pont Kincardine-on-Forth.

R. G. Edkins,
B.A., AM. Inst. C.E.

Special Note on the Mechanical Part of the Swing Bridge Span.
Bemerkungen über den mechanischen Teil der Drehbrücke.
Remarques sur la partie mécanique du pont tournant.

J. G. Brown,
M. Inst. C. E.

Introduction.

The Kincardine-on-Forth Bridge has been constructed with the dual purpose of improving road transport facilities in Scotland and of relieving unemployment in the districts near the site of the bridge.

The initial promotion of the scheme was carried through in 1931 by a Joint Bridge Committee formed by the County Councils of Fife, Clackmannan and Stirling, and the Borough Counsils of the towns of Dunfermline and Falkirk.

The Joint Bridge Committee appointed Sir Alexander Gibb & Partners as their Technical Advisers and Consulting Engineers.

The Bridge was designed and tenders were called for and a firm of Contractors were selected. This firm, The Cleveland Bridge & Engineering Company, Limited, of Darlington, commenced work at the beginning of January 1934.

A grant of financial assistance was obtained from the Ministry of Transport, from the Road Fund, to the extent of 75 % of the total cost of the scheme, the remainder being contributed by the above Local Authorities.

Selection of Site.

The choice of Kincardine as a site for a crossing of the Firth of Forth is a natural one. The banks of the river draw together at this point to form the second narrows in the river above the famous Forth Railway Bridge, which lies 13 miles to the East at Queensferry, the site of what may be called the first narrows.

Kincardine lies 10 miles downstream and East of Stirling, whose castle has guarded the route to the towns of the North East, Perth, Dundee, Aberdeen and

Inverness, from historical times and whose bridge is the most Easterly road crossing of the Forth at present.

The new bridge will, therefore, provide a shorter alternative route to these towns, will correct the present tendency to congestion of traffic through the town of Stirling and, in addition, will provide a much shorter direct road access between the industrial area of Glasgow and the comparatively isolated peninsular of East Fife, which lies between the encircling pincer-like arms of the sea formed by the Firth of Forth on the South and the Firth of Tay on the North.

It will also compete to a considerable degree with the half-hourly vehicle ferry at Queensferry, by affording an alternative crossing 20 miles nearer to Edinburgh than the existing alternative route through Stirling.

This ferry is the vital link connecting Edinburgh with the main highways leading to the North of Scotland.

To complete the case for a bridge at Kincardine, it is only necessary to add that it will cost almost exactly one tenth of the estimated cost of a recently proposed road bridge at the first narrows site near the great railway bridge at Queensferry.

Conditions affecting the Design.

The passage of sea-going ships to and from the port of Alloa, which lies 4 miles above the bridge, has necessitated the provision of a very large opening span, and the interests of less important river traffic have demanded a head room in the centre of the bridge of 30 feet above high water.

Consequently, the roadway to the bridge rises from the comparatively low lying ground on either bank in a vertical curve which has its highest point at the centre of the Swing Span.

On the South shore the ground is so low the river bank is not very clearly defined.

The river, which is tidal with a range of 18 feet, is 3,000 fest wide at periods of extreme high water, when the flats on the South shore are covered to a depth of one or two feet, but at ordinary high water periods the flats or "saltings" which are covered with coarse grass, are dry and the width of the river is then about 1,800 feet.

This additional 1,200 feet width of "saltings" is in fact a bank of silt some 50 feet deep which has been deposited on the gravels and clays of the river bed and its existence has meant that a considerable length of approach structure has had to be provided.

An approach embankment covers a little more than 500 feet of the "saltings" but owing to the rising curve of the bridge profile, and the impossibility of placing a high embankment on such soft ground, foundations for a crossing of the remainder of the "saltings" have had to be sought at a greater depth and the bridge structure proper commences here, about 700 feet from normal high water mark.

On the North shore the bridge is extended beyond the high water line in order to cross the London & North Eastern Railway which runs close beside the shore.

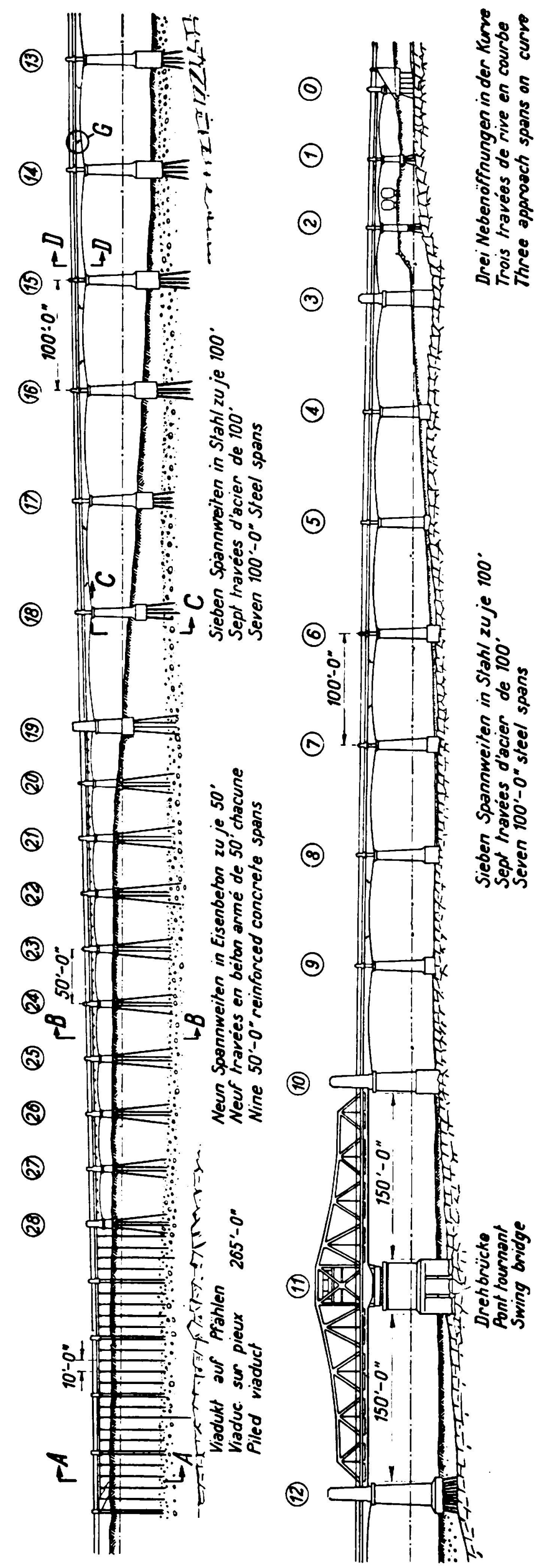


Fig. 1.

These additional lengths of bridge at either bank make the complete distance between abutments 2,692 feet.

The geology of the site is interesting. It provides excellent foundations on the sandstone rock of the ●arboniferous period at no great depth. But this is only the case over half the width of the river from the North bank, until the pier of the Swing Span is reached. From there Southwards, the level of the sandstone drops away to increasing depths and one of the original trial borings, made at about the middle of the "saltings", failed to find rock at a depth of 45 feet below the average level at which it is found on the North side of the river.

There are evidences of an important geological fault on the centre line of the river and also of the existence of the old preglacial river valley, supposed by some to be of great depth. Either or both of these suppositions may explain the change in the rock level.

Whatever the cause of this sudden dip of rock, its result is that the Southern half of the bridge has had to find foundations above the rock level.

This problem was greatly simplified by the presence of a bed of hard bound gravel, which was disclosed by the trial borings during the initial survey and which by its extent and its level suggests that it formed the river bed at some comparatively recent period — (geologically speaking) — and has been buried in silt since then. Below this gravel and above the rock are clays containing boulders and stones.

General description of the bridge and approaches.

The bridge, as can be seen from the accompanying illustrations, is a multiple span structure employing a large variety of types of construction.

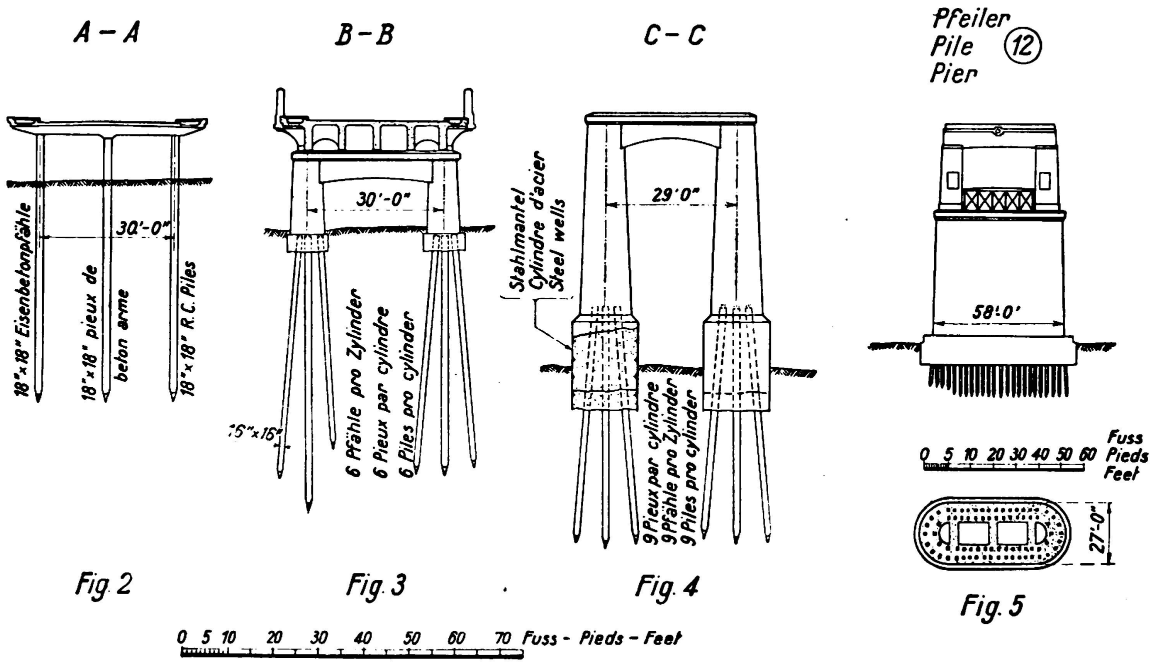


Fig. 2 Fig. 3 Fig. 4 Fig. 5

It carries a 30 feet wide carriageway and two 5 feet wide footpaths.

At its centre is the large Swing Span which constitutes its most interesting feature and this is flanked on either side by a symmetrical arrangement of 14 steel spans of 100 feet, seven on each side. These spans may be described as the main steel bridge which covers the normal width of the river.

On the South side a considerable length of reinforced concrete approach structure is made necessary by the presence of the "saltings" described above.

This approach structure consists of 9 spans of 50 feet each, and 260 feet of viaduct carried on bents of piles at 10 feet centres.

On the North side 3 approach spans are necessary to cross the railway and these are steel spans of 62'6" each and are placed on a curve in plan in order to allow the approach road and embankment to take the most economical line through the town of Kincardine.

Foundations and piers.

In describing the structure of the bridge in detail the Swing Span and its three piers will be treated separately as they form the most important and interesting features of the bridge. The remainder of the structure divides itself into five parts, namely: —

1) The Approach Spans and Piers on the North Side.
2) The 100 feet steel Spans and their Supporting Piers on the North Side.
3) The corresponding section of 100 feet Steel Spans on the South Side.
4) The Reinforced Concrete 50 feet Spans on the South Side.
5) The Reinforced Concrete Approach Viaduct on the South Side.

The section will be descrribed in numerical order and the methods of construction will be described at the same time in order to avoid repetition.

1) The three steel approach spans at the North end of the bridge rest upon the Abutment, Piers Nos. 1 and 2, and Pier No. 3. The Abutment and its wing walls is a mass concrete structure supported upon a piled foundation, consisting of 36 reinforced concrete piles 16"$\times$16" in section and 20 feet long, which were driven down on to the rock. Piers Nos. 1 and 2 are similar mass concrete piers, rectangular in plan and supported on 20 piles each, similar to the Abutment. The methods of construction are without special interest. The piles were driven to refusal and, therefore, no special final set per blow was required.

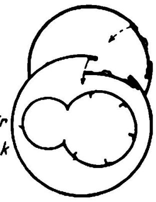
2) The seven 100 feet steel spans are supported by Piers Nos. 3 to 10. Pier No. 3 is a special pier with dimensions made larger than the normal pier in order to accommodate an architectural feature in the form of a pylon on either side oft the bridge, marking the commencement of the bridge proper. It consists of two very wide tapered columns, circular in plan and joined at their heads by a heavy reinforced concrete cross beam. It was founded in the open in the same way as the other piers on solid rock.

Piers Nos. 4 to 9 are all similar piers which stand on twin cylindrical bases 14'6" in diameter (Fig. 4). The bases end just below low water and the piers rise from that level in the form of two tapered columns, having a diameter of 8 feet at their heads where they are tied together by a reinforced concrete beam 4'6" wide and heavily reinforced to form a monolithic portal with the columns. The columns themselves have an external skin 9" thick of rich concrete of 1:1:2 proportions in which is embedded a moderate amount of vertical and circumferential steel reinforcement, the vertical steel being carried down into the base. Within the 9" skin is a core of 1:3:6 concrete (Fig. 4).

Piers Nos. 3, 4 and 5 were founded on rock within open wells. These wells were built up of cylinders 14'6" in diameter and about 4' high, made of 5/16" steel plate stiffened with angle rings and were lowered on to the river bed and reached above high water level. Enough of these cylinders were supplied to allow work to be done at two piers simultaneously. They were sunk until the cutting edge, formed by a heavy doubling plate, reached rock by excavating inside the wells with a grab. The water inside the wells was pumped out and the rock surface excavated by labourers working inside the wells until a firm level bed of sound rock was obtained. The wells were then filled solid with concrete up to low water level to form the two base cylinders. Steel forms were placed on the base concrete and the construction of the tapered columns continued until the work was brought up above high water level. The upper portion of the wells above the section filled with solid concrete were then removed by breaking a bolted joint and used again on another pier. The removal of the bolts had to be done by a diver working at slack water periods.

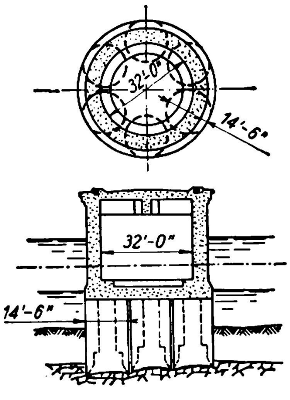
Fig 10

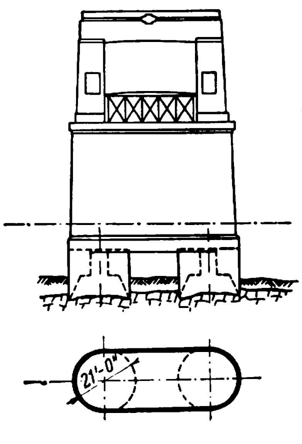
Schema der doppelten Luftschleuse
Schéma du double sas à air
Diagram of double air lock

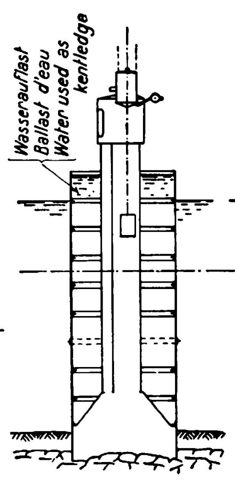


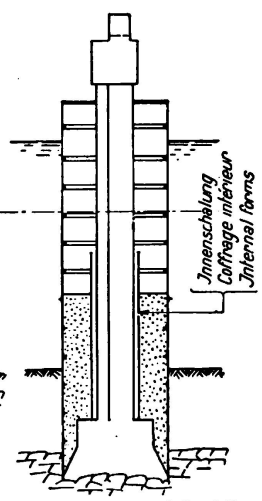
Mittelpfeiler
Pile médiane (11)
Centre pier

Pfeiler
Pile (10)
Pier









Schema der Verwandlung eines Senkbrunnens in pneumat. Fundation
Schéma de la transformation d'un puits foncé en fondation pneumatique
Diagram of steel well converted for use with compressed air

Fig. 6 Fig. 7

0 10 20 30 40 50 60 70 Fuss Pieds Feet

Fig. 8 Fig. 9

0 5 10 20 30 Fuss Pieds Feet

This method worked well on Piers Nos. 3, 4 and 5, but as the work got further out into the river the depth of the soft material overlying rock decreased until it reached a minimum of only 2 feet. At Pier No. 6 where it is about 5 feet deep, it was found that the material was not sufficiently stable and impervious to prevent the hydrostatic pressure outside the well, when it was pumped dry, from forcing the water through the ground and under the cutting edge and so flooding the well. Accordingly the necessary plant was brought to the site to enable the remainder of the piers to be completed by the method of compressed air caisson.

This method resulted in rapid and consistent progress being made and enabled careful inspection and perfection of the foundations to be carried out before concrete was placed.

The wells already described were adapted to take an access shaft carrying a double air lock at its top, by the provision of a substantial cone-shaped plate which formed the roof of the working chamber and reduced in diameter from that of the well at its bottom to that of the access shaft at its top. The double lock consisted of a vertical lock for materials and a horizontal lock for men (Fig. 8 and 10).

The portion of the well above the cone plate was filled with water to a level above that of the river outside in order to provide sufficient kentledge to prevent the well floating.

For purposes of economy the cone shaped adaptor plate was removed from each well and transferred to another, only two such plates being provided. In order to do this it was necessary to remove the water kentledge and supply about 200 tons of solid kentledge in its place, until the concrete filling inside was heavy enough to overcome any possible uplift on the well.

The first layer of concrete was, of course, placed on the rock while the well was still under compressed air, 3 or 4 bent pipes were passed under the cutting edge and brought up inside to a height above the top surface of the first lift of concrete. This prevented any possibility of the air blowing through the concrete during deposition and causing porous channels and leaks. After the concrete hat set, these pipes were closed with screwed caps, the pressure was taken off the well, the cone plate, access shaft and air lock were removed and the construction of the columns proceeded with as in an "open" well.

3) Piers Nos. 10, 11 and 12 are the large piers specially provided for the Swing Span and will be described later. Piers Nos. 13 to 18 are the same as Piers Nos. 4 to 9, described above, but they are not founded on rock as are the latter. They are founded on the gravel bed which stretches under the bank of silt which forms the South shore of the river. The foundation consists of 18 piles, 9 in each cylinder, 4 of which are driven vertical and 5 of which are driven to a slight batter of 1 in 12, 3 at right angles to the centre line of the bridge and 2 in line with it. Pier No. 19 is the same as Pier No. 3, except for its foundations which are similar to those of Piers Nos. 13 to 18.

These piles are 18" $\times$ 18" square in section and vary from 45' to 65' in length, but the great majority of them were made 55' long. They have cast iron points and are reinforced with four $1^1/_2$" diameter rods in one continuous length bound by 3/16" stirrups at 9" spacing. This spacing is reduced at the head and the point to strengthen the pile against bursting.

The sequence of operations with these piers was to sink the wells, which were the same as those described above, to a depth of 10 feet below the surface of the river bed by excavating with a grab and then to drive the piles inside the well.

Excavation in some of these cylinders was delayed by the difficulty experienced in penetrating the gravel owing to the pressure of large boulders and stones. The driving of the piles was done from a travelling frame which could both turn and travel on a steel gantry over the wells. A single acting steam hammer, the moving part of which weighed 5 tons, was used.

The piles were driven until the penetration under six blows of the hammer was less than 1". The fall of the hammer was altered to suit the length of the pile. For a 65" pile it was 4'3" and for a 55' pile it was 3'9".

As a general rule the piles began to drive easily and came against the hard stratum fairly suddenly and then could be driven very little further. At the few places where the piles penetrated the hard stratum a period of steady driving followed, gradually becoming more difficult until the required resistance was reached.

The heads of the piles were protected during driving by a helmet of cast steel which was packed with paper or sacking. The hammer struck on a hard timber dolly about 24" high, fitted into the top of the helmet.

During the driving of piles within the wells near the middle of the river it was necessary to lower the level of the water inside the cylinder below that outside, by pumping and to rely on the seal, provided by the 10 feet of material into which the well was sunk, to resist the resulting external hydrostatic pressure.

By the provision of a sluice valve in the cylinder below low water level it was possible to limit the time that the cylinder was under pressure to the minimum necessary to allow the piles to be driven and disconnected from the guides of the pile frame. This latter operation could only be performed when the water was 5 feet below the head of the pile.

The greatest pressure to which these wells were subjected during the course of the work was that due to a head of 15 feet.

When pile driving was completed a plug of concrete about 3 to 4 feet in height was deposited in the bottom of the well, through water, using special buckets with hinged bottoms which opened to discharge the concrete as soon as they were hoisted after being lowered on to the bottom, according to the Panchard system.

When this plug had been given time to set the well was pumped dry and men went down and cleaned off the slurry of cement and mud which covered the top surface of the plug to the depth of about 12", until firm material was uncovered. Concrete was then deposited as for the other piers, the piles being roughened to give the concrete a strong grip and finally cut off at a level which allowed them to protrude 3 feet into the tapered columns (Fig. 4).

A description of these piers would not be complete without mention being made of the riveted steel forms used in their construction. These forms consisted solely of a steel skin fitting the surface of the concrete of the tapered columns and the arched cross beams, which was sufficiently stiffened by angle sections to support the weight of the wet concrete without outside support. They were comparatively light and rigid and the various pieces were bolted together by means of flanged joints and they could be erected and dismantled easily and quickly without the necessity of employing tradesmen.

On the other parts of the work welding was used instead of rivets which had the advantage of still lighter construction and left no marks of counter-sunk rivet heads in the concrete (Fig. 21).

4) The nine 50 feet spans of reinforced concrete are carried on piers similar in every way to those above described, but of smaller dimensions (Fig. 3). The diameter of the tapered columns being 6 feet instead of 8 feet. They are supported on 6 piles per column instead of 9, and the piles were 16" × 16" in section instead of 18" × 18". The penetration obtained was greater, but this was due merely to the increasing depth of the overlying silt as the bank of the river rises.

All the foundations were above high water so that no problem was presented in their construction, except that of access of which more will be said later.

Steel forms similar to those used for the 100 feet span piers were used for these piers also.

5) The remainder of the bridge on the South side is carried on a reinforced concrete viaduct consisting of a 10" reinforced concrete slab, which forms the carriageway, reduced to 6" on either side to carry the footpaths.

This slab is supported on transverse beams set at 10 feet centres which are supported on 3 piles, 1 in the middle and 1 at the line of the kerb on each side. The portions of the beams supporting the footpath slab are cantilevers (Fig. 2).

The piles used are 18" × 18" reinforced concrete piles 65 feet long. These piles have the greatest penetration of any on the bridge, they are founded at a depth of 50 feet below ground level.

The part of the viaduct abutting on the approach bank is braced by diagonal and horizontal members of reinforced concrete, which are built monolithic with the piles in five bents.

Stagings for Construction.

Mention was made above of the problem of gaining access to the line of the bridge. This was a question of the greatest importance to the Contractors and particularly affected the work on the South side.

During the initial consideration of the general scheme of construction it was, of course, necessary to make a choice between the use of floating plant or the provision of a temporary staging upon which cranes and other plant could stand and travel. The Contractors very wisely, in view of the strong fluctuating tidal currents in the river, decided to provide a staging over the whole length of the bridge, except for a gap at the Swing Span to allow the passage of ships during the construction of the bridge. But whatever decision had been made elsewhere, on the South side it was necessary to provide a staging both across the "saltings" as a means of access and along the line of the bridge as a working platform. After several trials it was decided to abandon the use of piles which it was found would have to be very long to support the load of a crane track and a crane and, consequently, most uneconomical, but instead to rely on a mattress of close set sleepers with a superstructure of timber cribwork to distribute the load over the grass grown crust of the "saltings" sufficiently to keep it within its bearing capacity. This proved quite successful and was used also for the double track on either side of the line of the bridge, which supported the travelling gantry used for driving piles.

In the river, timber piles were used to support the staging, which was heavily braced with steel angle sections. The crane track was carried on 18" joists (Fig. 21).

Steel and reinforced concrete spans.

Recommencing at the North side, the design and construction of the steel and concrete spans of the bridge will be described.

The approach spans on the North side of the river, crossing the railway, are

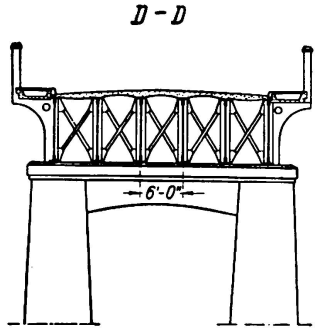
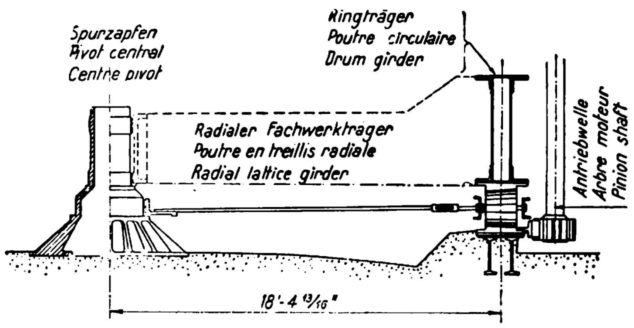


Fig. 11

0 10 20 30



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12 0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15

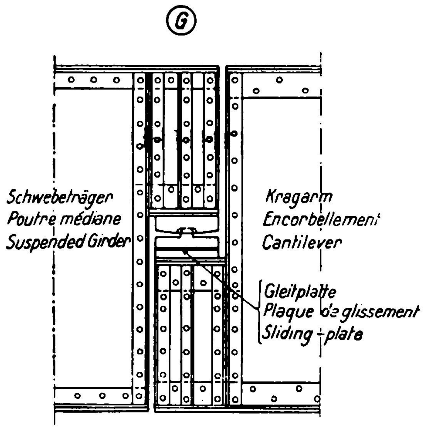
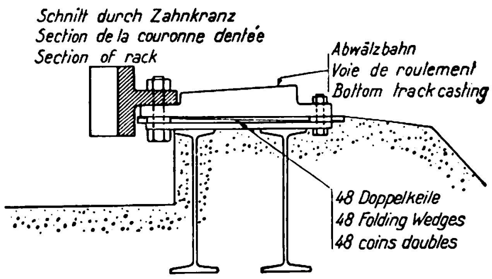


Fig. 12

12 0 1 2 3



Einzelheit der Abwälzbahn
Détail de la voie de roulement
Detail of lower roller track

Fig. 14

12 6 0 1 2

continuous steel spans on a curve. The continuity of the spans was retained in order to allow an arched form of girder to be used which would conform with the design of the main bridge.

Their detail design is similar to that of the 100 feet spans which will be described later, but they are reduced in size in proportion to their span.

The chief point in which they differ is that on account of the curve of the bridge the carriageway has been given a moderate degree of super-elevation and has been widened over the curved portion. In order to reduce the dead load which would be imposed by merely thickening the concrete, which forms the carriageway, on one side so that the required slope could be obtained, the girders themselves have been raised on their bearings from the inside of the curve, outwards, so as to reduce the required thickening of the concrete to a minimum. This has resulted in a rather complicated piece of steelwork, particularly in the case of the cross bracing and footpath brackets.

2) and 3) The 100 feet steel spans are deck spans designed on the principle of alternate spans continuous with two cantilevers at each support alternating with short suspended spans resting on the ends of the two cantilevers (Fig. 1 and 12).

This arrangement has the advantage of bringing the maximum bending moment over the supports where the greatest depth of girder occurs and simplifies the provision of expansion joints. The arched form of girder resulting from this arrangement gives the bridge a pleasing appearance.

Each span consists of six girders spaced at 6 feet centres, built up of web plate with angle stiffeners and flanges of double angles and plates. They are 9'1" deep at the supports and 5'2" deep at the centre. The girders are tied together with angle cross bracing and by the deck of buckled plates. The plates are concave upwards at the sides of the carriageway and concave downwards in the centre in order to suit the camber of the concrete filling which forms the road foundation.

The footpaths consisting of steel plates covered with concrete are carried on brackets springing from the webs of the outside girders.

Fig. 15.

General view of bridge. Looking up-stream and west. From north tower of electricity power crossing. Conductors cross the picture in the left hand corner.

The bearings of the girders on the piers are alternately fixed and sliding bearings. The sliding bearings consist of steel plates fixed to the girders sliding on phosphor-bronze plates, fixed in cast iron seatings bolted to the piers. Slotted holes are provided for the holding down bolts.

The bearings of the short suspended spans rest on stiffened brackets at the ends of the cantilevers (Fig. 12). One end is sliding and the other end is fixed in position by a knuckle working in a shallow trough, which allows a rocking motion at the point of contact.

The design of these spans greatly simplified the problem of erection. The steelwork was delivered by rail in three standard fabricated units: —

I) Pieces formed of a cantilever and a similar portion of the girder continuous with it, which took the form of a double cantilever on either side of the support.

II) The centre sections of the continuous girders.

III) The short suspended girders complete.

Two 5 ton Locomotive Cranes picked up the double cantilever pieces i), at each end and travelled with them out on to the construction staging on the

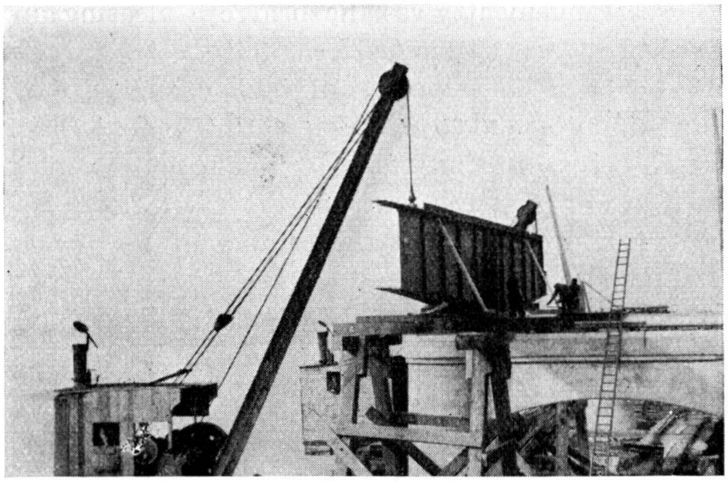


Fig. 16.

Steel erection 100 feet steel spans. 5 ton locomotive cranes lifting double cantilever section onto top of pier.

downstream side of the bridge and placed them on the piers (Fig. 16). A second double cantilever piece was erected on the adjacent piers and then the centre section II) of the continuous girder was joined to the two double cantilever sections, the projecting ends of which were steadied by temporary timber staging.

This completed one continuous girder and two projecting cantilevers, a second girder was similarly erected, the two were braced together and then pushed over on a roller track to the upstream side of the piers. The weight of these two girders together was about 55 tons.

The other pairs of girders were erected in the same way and pushed into position by the use of jacks. This completed the erection of six girders on one pair of piers. Work then passed on to the next pair and when the six girders were erected on these there remained only the gaps for the suspended girders iii) to be filled. These girders were also erected on the downstream side and were pulled over, one by one, on a temporary skid rail placed on the tops of the ends of the cantilevers.

The site riveting consisted only of riveting the joints between the centre sections and the cantilever portions of the continuous girders, the cross bracing and the buckled plate deck.

The whole of the steel for the South side was delivered by rail to the North side and floated across the river on pontoons.

Three full spans, two girders wide, were erected at the Contractors' works at Darlington, to test the workmanship of the steelwork before despatch to the site and this was clearly shown later when the whole of the 14 spans were erected and fitted together without any difficulty.

4) The reinforced concrete approach spans on the South side (Fig. 3) consist of 5 reinforced concrete beams spaced at 7'6" centres continuous over two supports with an expansion joint every 150 feet for which steel bearing plates sliding on phosphor-bronze seatings are provided.

The two intermediate bearings are merely arranged to be non-monolithic, by interposing a layer of bitumen between the top of the piers and the underside of the beams. The beams have a similar arched form to the steel spans and are 5'8" deep at the supports. The beams are monolithic with the deck slab which is 10" deep and the footpaths are cantilevered out and are supported on brackets in order to imitate the design of the steel spans. The footpath brackets which, naturally, project from the side of the outside beam are reinforced by jack arches in line with the brackets between the outside beam and its neighbour.

These brackets and jack arches complicated the design of the form work, which had necessarily to be self-supporting between the piers. Any intermediate supports of a temporary nature placed on the "saltings" would have been so unreliable, due to the yielding nature of the ground, as to have been dangerous.

Steel forms were used, the sides of the beam forms being sufficiently strong to support the whole of the concrete of the span including the deck. It can be easily imagined that the large gaps in the side forms of the outside girders, necessary for the formation of the brackets and jack arches, weakened the beam strength of the forms seriously.

In practice, the beams were poured separately up to the underside of the deck. Closing spaces were left over the supports to guard against shrinkage cracks and to allow the beams free action initially as simply supported spans, because the tensile steel necessary over the supports for continuity, was at this stage above the level of the concrete.

The forms were removed after 7 days and the deck shuttering was erected on the beams themselves, being supported by the bolts left in the concrete for the purpose.

This method also had the effect of facilitating the stripping of the beam forms.

The closing spaces were filled when the deck slab concrete was poured.

5) The piled viaduct structure has been described earlier. It is only necessary to add that the form work for this section was entirely of timber and that the expansion joints are provided at every 50 feet. These are arranged for by widening every fifth cross beam and supporting it on 5 piles instead of 3. The free end of the deck slab rested on half the width of this widened beam, discontinuity being ensured by painting the bearing surface of the concrete with bitumen. These joints work easily.

Swing Span piers.

The three piers of the large Swing Span are the most important units in the foundations of the bridge and, consequently, occupied the most time.

They will be described in the order in which they were constructed.

Centre Pier.

The centre pivot pier was designed as a hollow cylindrical pier, 42 feet in diameter, founded on rock inside an open sheet pile cofferdam. The cofferdam was driven, using Larssen No. 2 steel sheet piles, and attempts were made to dewater it which failed several times. This was due partly to the presence of boulders in the 8 feet of gravel and clay overlying the rock, and partly to the buckling and twisting of the lower ends of the piles causing the grips to open. Finally, when the last attempt to dewater was made and the bottom was actually uncovered in places, a very violent blow in, under the feet of the piles, occurred at high water, which scoured out the internal material holding the feet of the piles and forced in a length of some 15 feet of the sheeting up to the point where it was held by the lowest waling. After this the attempt to work in the open was abandoned and the foundation of the pier was re-designed.

This arrangement consisted of six wells of 14'—6" diameter to be sunk under compressed air to rock, arranged with their centres on a circle of 32'—0" diameter (Fig. 6).

Just below water level these wells, which were filled solid with concrete, are tied together by a heavy circular slab of concrete 4'—0" in depth and heavily reinforced with steel. On this slab the walls of the cylindrical pier, which are 5'—0" thick, are built according to the original design. In order to overcome the difficulty of kentledge, which was described in connection with the compressed air wells used for the 100-feet span piers, the six wells for the centre pier were built of concrete inside the steel shells. The cutting edges were suspended from three pairs of jacks by perforated straps and as the concrete was filled into the cylinders and the load on the temporary staging supporting the jacks was increased the wells were lowered down on the straps in order to obtain partial support by floatation (Fig. 23). The wells were finally lowered on to the bottom and excavation began under compressed air, until a sufficiently stable bearing had been obtained to allow the support of the jacks to be dispensed with.

Centre Pier.

All these wells were sunk without any serious delay and excellent foundations about 4'—0" into hard sandstone were obtained. The working chambers and the access shaft space were then filled with concrete and work commenced on the concrete slab.

This slab, as mentioned above, was below low water level, but the cofferdam was sufficiently watertight to allow a moderate difference of level between the inside and outside water levels to be maintained.

Work, thereafter, proceeded without special difficulty and the pier was raised above high water.

At the top this hollow pier carries 4 steel lattice girders 6'—6" deep, arranged radially in the form of a cross and embedded in concrete. These girders support the central pivot of the swing bridge, the weight of the bridge, however, is carried by the walls of the pier on a roller path.

The reinforced concrete roof of this pier is supported on the radial girders. All the concrete in this pier was of $1^1/_2$: 2: 4 proportions.

As a protection to the Swing Span at this pier a timber jetty has been constructed, which is slightly wider than the steelwork of the Swing Span and is 420 feet long from end to end. Stretching up and downstream this jetty is provided to assist any vessel passing through the bridge which is not entirely under control, and protect the steelwork of the bridge when it is open, from any error of judgment on the part of the pilot. The timber used for this jetty, which contains 212, 14"×14" piles 50 to 60 feet long, is of British Columbian Pine and for protection against deterioration the timber was creosoted under a pressure of 180 Lbs per sq. in. In order to obtain as reliable a protection as possible, the surface of the large timbers was incised before being creosoted.

The pitch of the incisions, which are $^1/_4$" deep, is 8" and the lines of incision are 1" apart, the incisions being staggered so that the incision on one line comes opposite a space between two incisions on the adjacent line.

The average results obtained were 4 Lbs. of creosote per cubic foot and the gain in penetration due to incising with this timber was 0.72 Lbs per sq. ft. of exposed surface.

The upstream and downstream ends of the jetty are strongly braced radially to resist collision and the remainder of the jetty is braced in the vertical plane.

Galvanised iron bolts, washers, plates and dogs are used for the fastenings. The jetty also formed a very convenient staging for the erection of the Swing Span steelwork (Fig. 22).

North End Rest Pier.

After the experience gained with the central pier it was decided not to attempt the foundation of the North end rest pier of the Swing Span inside an open cofferdam. The pier, which was originally designed with a mass concrete foundation similar to the centre pier, was re-designed on similar principles (Fig. 7).

Two large cylinders of 21 feet diameter were used in this case and were sunk under compressed air in a manner similar to that used elsewhere. The two wells being built of concrete inside steel shells, but owing to their greater size were supported on 4 pairs of jacks instead of 3.

As in the case of the centre pier a heavy slab joins the heads of the two cylinders just below low water and use was made of the cofferdam to maintain a moderate difference between external and internal water levels. The pier above the foundation cylinders and slab consists of a hollow rectangular pier with semi-circular ends. The walls are 5'—0" thick at the base, tapering to 3'—6" at the top, and the internal space is divided into two rectangular and two semi-circular compartments, by cross walls 3'—6" thick.

The top of the pier has a reinforced concrete roof and carries, in addition to the ends of the 100-feet span girders, the bases for the rollers upon which ride the two wedges which lift the nose of the Swing Span into position at each end; a socket for the locking bolt of the Swing Span and electrical devices for bringing the bridge to rest and centering it over on the carriageway.

Above the level of the carriageway this pier carries a large portal structure which is partly of agricultural and ornamental significance and partly necessary to house the safety gates which are lowered horizontally across the road and footpath when the bridge is opened to allow the passage of a ship. When the road is open the gates are hoisted and hidden inside the portal beam.

South end Rest Pier.

The third pier for the Swing Span was also designed to be constructed inside an open cofferdam, but in view of the uncertainty of the level of sound material, which was not shown very clearly by the survey borings, and the excessive depth before rock could be obtained, it was decided to drive piles in this foundation.

Seventy six piles were driven around the circumference of the pier, under the walls, in two rows. The outer row of forty two piles were driven to a batter of 1 in 10, the remainder being vertical. These piles were 18" by 18" in section and 40 feet long and were driven to refusal with an average penetration of 20 feet below the river bed.

This foundation was treated in much the same way as the small foundations for the 100-feet span piers.

The cofferdam was driven to a penetration of about 15 feet and the ground was excavated inside by grabbing to a depth of 10 feet below the river bed level.

The piles were then driven and 3 feet of concrete was placed on the bottom, through water. When this had set the cofferdam was pumped dry and construction continued inside (Fig. 5).

The base of the pier consists of 6 feet of concrete and above this level the structure is the same as the North end rest pier.

The piles were roughened or broken down and the steel reinforcing rods incorporated in the concrete of the walls of the pier.

For all three of these piers the outer face shuttering was of welded steel construction and was of particular advantage at the curved corbels at the tops of the piers (Fig. 21).

The Swing Bridge.

The steelwork of the Swing Bridge is 364 feet long from end to end and, when closed, spans two openings of 150 feet clear width.

The 30 feet carriageway passes between the two warren type girders of the span, at a level slightly above the bottom boom, and the footpaths are cantilevered out on brackets springing from the vertical members of the truss. The weight of the bridge which is 1600 tons, rests upon 60 cast steel rollers which run on a cast steel track supported over the centre of the wall of the central pier.

To commence then at the bottom, the roller track will be described first. The foundation for this track consists of a circular box girder built of two joints 20" deep with $^7/_8$" cover plate. On this bed the cast steel sections of the roller track were laid. For purposes of level adjustment 48 pairs of machined steel folding wedges were interposed between the foundation plate and the track castings, and the beds for these wedges were ground plane. The track castings were placed and adjusted for radial inclination and circumferential level to limits of $^{1}/_{64}$" on temporary wedges. The pairs of wedges were then checked for

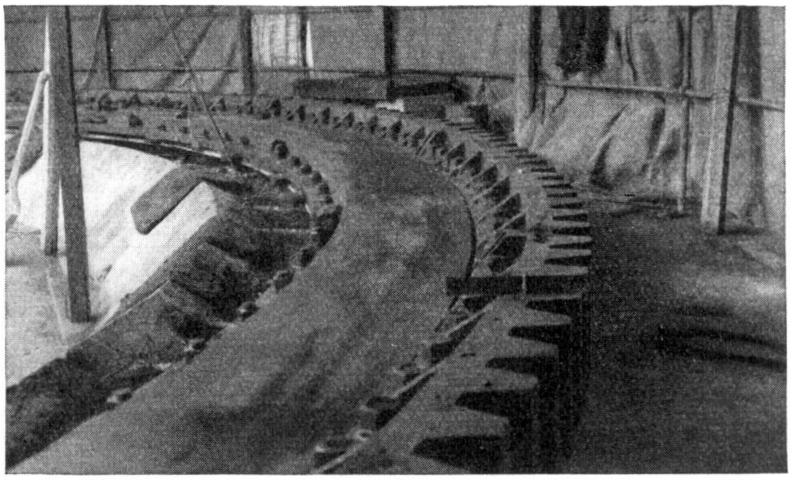


Fig. 17.

Swing bridge. Lower roller track castings bolted down with rack sections bolted on.

correct taper to give the right inclination to the track, by using a $^{4}/_{1000}$" feeler between them and their top and bottom bearing surfaces. Any errors of taper were rectified and the wedges replaced.

The circumferential level was finally adjusted and when correct within the

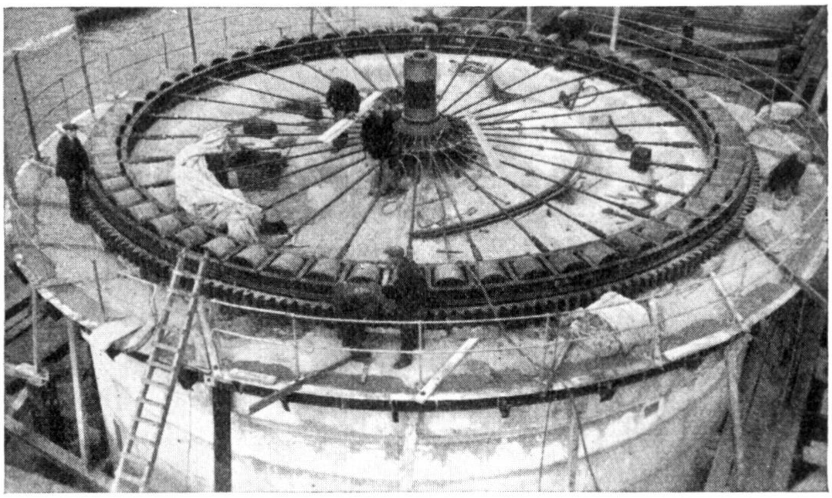


Fig. 18.

Swing bridge. Centre pivot. Radius rods and cast steel rollers on centre pier (Nr. 11). Shows rack bolted on.

limits of accuracy obtainable, the track castings, wedges and foundation plate were drilled simultaneously and bolted together with turned bolts (Fig. 14).

The spaces between the wedges were then filled with a dry hard rammed mortar of cement and sand of 1:1 proportions.

A toothed cast steel rack section was bolted to the track castings and the cast steel rollers were assembled. These were fabricated in cages of channel sections holding 5 rollers and each alternate roller has a radius rod which is connected to the centre bearing, which turns on a machined journal on the centre pivot.

The upper roller track castings, which are identical with the lower track, were placed on the rollers and a very stiff drum girder, 5 feet deep, of box section was lowered thereon (Fig. 19, see also Fig. 17 and 18).

This drum girder performs the function of distributing the weight of the bridge as evenly as possible upon the rollers, and the upper track casting was adjusted from its lower flange plate by inserting thin packings (Fig. 19) until

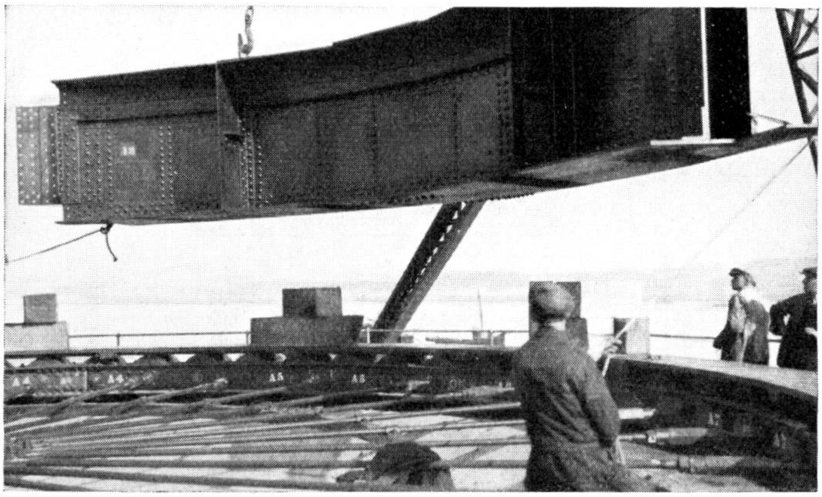


Fig. 19.

Swing bridge. Drum girder being lowered into position on top of upper roller track castings which are resting on top of the rollers.

a $^4/_{1000}$" feeler could not be inserted at any point between a roller and the upper track.

The drum girder and superstructure were rotated into different positions for the purpose of this test.

The drum girder is radially braced to the centre pivot of the bridge by lattice girders bolted to the castings turning on two machined journals on the pivot at the top and bottom (Fig. 13). These lattice girders support the floor of the machinery room and its machines.

The drum girder receives the weight of the bridge at 8 points. These 8 points are the bearings of 4 very deep girders, 2 to each girder. These are the main distribution girders, which are built integral with the centre section of the bottom boom, and are box girders 8'—4" deep, and two main cross girders, which are stiffened web plate girders, across the line of the bridge, 11'—0" deep.

The space between the four walls formed by these four girders is used as the machinery room.

The main girders of the bridge are of normal construction, the top and bottom booms are of box section, one side at least being formed of open lattice bracing. The diagonals are deep H sections formed of a web plate and four unequal bulb angle sections. The verticals are I girder section. All sections are 22″ deep. The deck bearers are 20" joists placed longitudinally at 5 feet centres on cross girders at 20'—3" centres, which are 5 feet deep.

The decking is of buckled plates similar to the 100-feet spans and, like them, is covered with concrete.

In the case of the swing bridge the concrete has articulated joints at intervals, filled with bitumen and sealed with a corrugated copper strip. A system of triangular sway bracing of light lattice members, joins the two top booms and between them, in the centre of the bridge, is placed the operator's cabin from which the swinging of the bridge is controlled.

Machinery.

The operation and control of the bridge is electrical.

The bridge is swung by a dual pinion and rack system, the rack being fixed to the pier is, consequently, stationary. The two pinions turn in bearings fixed to the bridge and, consequently turn with it. The pinions are driven by $9^1/_2$"

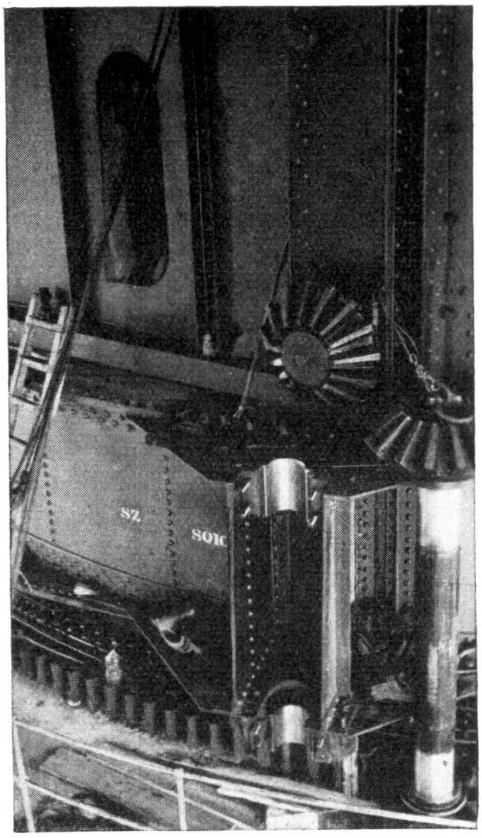


Fig. 20.

Swing bridge. Pinion shaft and bevel gears, Pinion shaft bearings. Toothed circular rack. Door of machinery room.

diameter vertical shafts connected by bevel gears to horizontal main shafts passing through the walls of the machinery room, that is to say, the webs of the main cross girders. These main horizontal shafts are driven through a train of reduction gearing by two 50 H. P. Direct Current Motors. The two drives are entirely independent.

The supply of power to the bridge is Alternating Current obtained from the Local Electrical Company and is conveyed by a cable running along the bridge, descending below the river in order to cross to the central pier which it enters by means of a duct through the wall placed above high water level, and thence, after subdivision into small cables, reaches the machinery room through the hollow shaft of the centre pivot to a vertical series of collector rings and brush

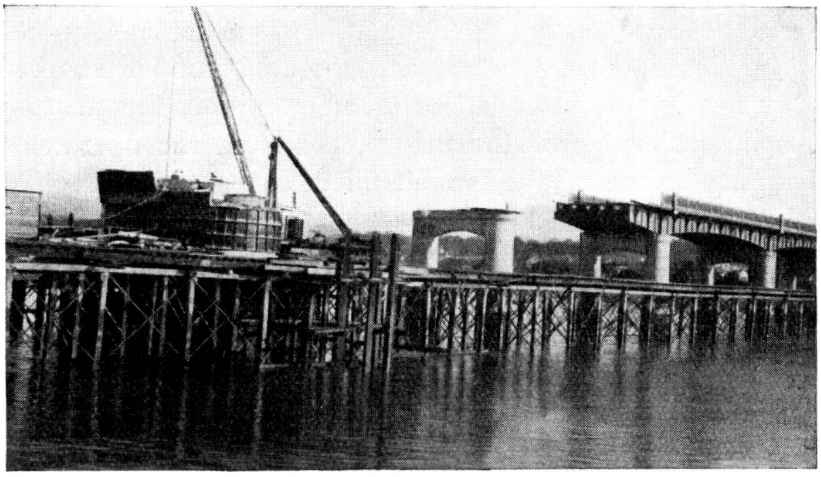


Fig. 21.

Pier Nr. 10 under construction. Shows welded steel form work. Projecting cantilevers of steel spans with brackets for suspended girders. Temporary timber staging on piles.

contacts. The A. C. supply is converted for use in the D. C. Motors by a Ward Leonard Motor Generator Set, and a standby set, in case of a failure in the normal supply, is also provided. This consists of a 150 H. P. 4 cylinder, horizontally opposed, solid injection Diesel Engine, which drives a 40 K. W.

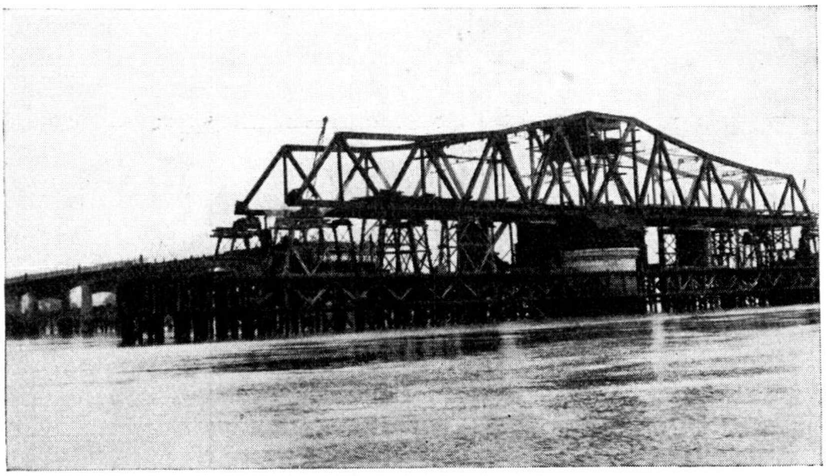


Fig. 22.

Swing bridge nearing completion. Shows permanent timber jetty and temporary trestles for erection.

Direct Current Generator and a 36 K. W. Alternator for lighting the bridge. For the hydraulic power by which the wedges and locking bolts are operated, a 25 H. P. Direct Current Motor is provided, coupled to a six ram oil pump which has four rams working at 3000 lbs. per sq. inch, and two rams working at

1000 lbs. per sq. inch, the former are for the wedges and the latter for the locking bolts. All this machinery, including the usual compressed air starting system for the Diesel Engine, is remotely controlled from the control cabin.

In view of the exceptional size of the swing span every care has been taken to ensure that the various operations in opening and closing the span are automatically carried out in proper and regular sequence by electrical control to eliminate the risk of accident due to an error on the part of the operator. In addition, duplication of essential plant and controls has been kept in view wherever possible to prevent the complete breakdown of the operations in the event of a fault occurring in any of these.

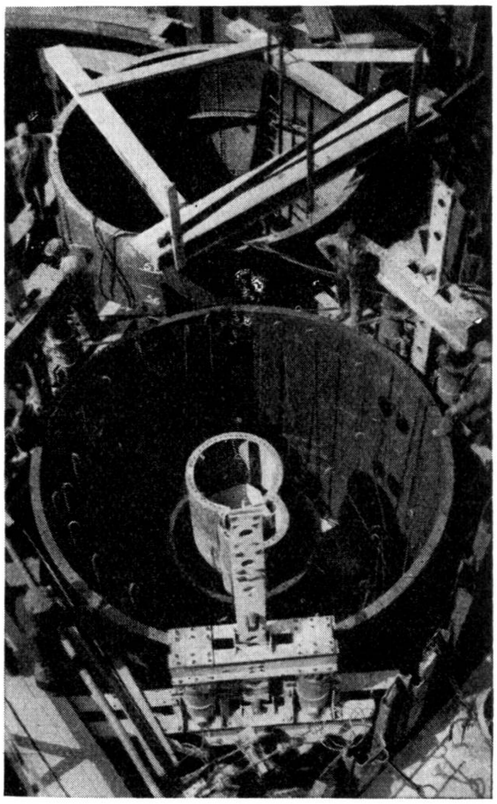


Fig. 23.

Centre pier No. 11. Compressed air wells at pier No. 11. Showing access shaft. Internal shuttering. Reinforcing rods and steel well suspended from perforated straps on hydraulic jacks. Compare with fig.8, 9 and 10.

The operator stands at a control desk in the cabin, where a hand wheel as it is turned round carries out the proper sequence of operations, the completion of each operation being indicated by coloured signal lamps on the desk.

The operation of closing the roadway to traffic and opening the Bridge to river traffic is as follows: —

1) Power supply switched on for Ward Leonhard set and hydraulic pumps.
2) Road traffic signals at both ends of the Bridge turn from green to red and warning bell rings. Interval until assistant in roadway indicates to operator in cabin by gong that roadway is clear of vehicular and pedestrian traffic.
3) Gates lowered across roadway and footpath at both ends of Bridge. Red light at end of jetty changes to amber, semaphore arm drops and syren sounds to indicate to vessel that Bridge is about to be opened.

4) Wedges holding up ends of Bridge withdrawn by hydraulic power.
5) Locking bolt withdrawn by hydraulic power.
6) Bridge swung through 90^0 to open position. The acceleration to maximum uniform speed and deceleration at the end of the travel is controlled automatically by limit switches.
7) Second red light at end of jetty turns to amber and another semaphore arm drops to indicate that passage is clear for shipping. A syren is also sounded.

For the closing of the Bridge the operations are reversed. It is possible by manipulating the handwheel in the cabin to make the Bridge open or close clockwise or counter-clockwise, or end for end, as desired.

For ensuring the exact centering of the Bridge a photo-electric device is incorporated. This consists of three coloured lights on the control desk, the lighting of the lamp at either end indicates that the Bridge is nearly centred, whilst the centre lamp when alight shows that the Bridge is centred within $1^1/_2$" or sufficiently near the dead centre for the locking bolts to be driven home. The two locking bolts and the four wedges under the ends of the Bridge each travel 27 inches, and their relative movement is indicated by dials on the control desk. In the event of a breakdown of one of them this is instantly shown on the desk by a warning light, and the operation stops. A hand pump is installed in the machinery room to allow for the defective wedge or bolt being operated by this means.

In the event of a failure of any of the essential traffic control lights, this is indicated by a warning lamp in the cabin.

The whole operation of opening or closing the Bridge occupies four minutes as follows: —

Withdrawing wedges	$1^1/_2$	minutes
Withdrawing locking bolts	$^1/_2$	minutes
Swinging Bridge through 90^0	2	minutes
Total =	4	minutes

The shipping using the river will pass the site of the Bridge within one or two hours on either side of High Water. It will be necessary, however, to have the Bridge Staff constantly in attendance for maintenance etc., and the intention is to have three groups of Bridge operators and assistants, each group working an eight hour shift.

Summary.

The bridge, which crosses the river Forth, was promoted by a Joint Committee of the interested Local Authorities and the Ministry of Transport contributed largely to the cost.

The site of the bridge is the most Easterly site consistent with moderate cost, and the bridge will improve road communication between Glasgow and Edinburgh and the Nort East of Scotland.

River traffic has necessitated the provision of a large opening span and the site conditions have resulted in a bridge $^1/_2$ a mile in length employing many different types of construction.

The foundations of the Northerly half are all on rock at easy depths and compressed air caissons adapted in a simple manner from open steel wells were largely used for these foundations. On the Southern half of the bridge use was made of reinforced concrete piles up to 65 feet in length and 18"×18" in cross section.

The 14 deck spans of 100 feet are constructed of steel girders 6 to a span, employing the cantilever and suspended girder priciple.

The road, which is 30 feet wide, is formed of steel buckled plates covered with concrete and a bituminous surface.

The two footpaths throughout the bridge, each 5 feet wide, are carried on cantilever brackets on either side of the main structure.

The approach spans on the North side are constructed on a curve and are of steel with super-elevation built in.

The approach on the South side consists of 9 reinforced concrete spans of 50 feet, having 5 girders each and 260 feet length of viaduct supported on piles. The 50 ft. spans involved some difficult form work. The forms for concrete throughout the bridge were of steel.

There are 3 large piers supporting the Swing Bridge, 2 of these were founded on rock by use of compressed air caissons and the Southern pier was founded on piles.

The Swing Span, 364 feet long, is a rim bearing Swing Bridge of the largest type with a moving weight of 1600 tons.

The roller track supporting the bridge involved some accurate workmanship.

The bridge itself is a through span of the Warren girder type, with a steel plate deck covered with concrete for the carriageway.

The turning machinery, which is electrically operated, is located beneath the roadway and is remotely controlled from a cabin suspended between the girders above the road. Wedges and centering bolts are hydraulically operated and photo electric equipment is used for centering the bridge accurately.

A standby generating set driven by a Diesel Engine is provided.

The bridge can be swung open in four minutes.

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VIIa 3

Bemerkungen über Stahlbrücken in Dänemark.

Remarques sur les ponts métalliques au Danemark.

Steel Bridges in Denmark.

A. Engelund,

Professor an der Polytechnischen Lehranstalt, Copenhagen.

In many countries the developement of design for main girders of bridges is known to be in the direction of preference for the solid webbed girder over the lattice girder even for considerable spans.

While the current preference for maximum possible simplicity of form may be a partial explanation of this, it is certainly an exaggeration to state, as is done in various quarters, that solid webbed construction is, preferred merely because it is, so to speak, the fashion at the present time.

According to experience in bridge construction in Denmark during the last ten years, the following may be taken as an outline of the situation now obtaining.

Up to 35 metres span the amount of material used does not vary appreciably whether for solid webbed or framed girders, and where this is the case the solid webbed girder will clearly be the more economical. For larger spans — that is, up to 60 to 70 metres in the case of beam bridges and up to about 120 to 140 metres in arched bridges — a greater expenditure on material is as a rule necessitated for solid webbed construction, but the cost is usually not greater than for framed construction because the cost of manufacture and frequently also the cost of erection per ton is lower. Finally, the solid webbed girder with its smaller surface area is cheaper to maintain.

To attain the full economic advantage of solid-webbed construction it would be desirable to introduce more rational estimate of construction and maintenance, and with this object in view a few figures are given below which must be taken into consideration when making a more rational comparison between solid-webbed and framed girders, namely:

a) Number of rivets per ton of steel work.

b) Weight of rivets per ton of steel work.

c) Exposed surface area per ton of steel work.

The following are the values of these quantities on various bridge works in Denmark:

	Span m	Steel weight tons m	No. of rivets per ton	Weight of rivets kg per ton	Exposed surface sq. m per ton
(1) Warren girders with diagonals and verticals	36.0	0.78	204	45	10
(2) Plate-girders	32.9	0.86	87	38	8
(3) Parabolic truss-girders with diagonals and without verticals and without continuous rivet lines (rolled sections)	67.0	0.97	65	21	12
(4) Bow-string bridges with stiffening plate-girder	67.6	1.03	190	48	12
(5) Parallel lattice girders with posts and 4 systems of crossing members . .	64.5	1.10	154	38	12
(6) Parallel girders with diagonals and posts . . , ...	70	1 75	150	48	13
(7) plate-girders	60	1.86	75	40	6.5

The longitudinal beams referred to in 4), in a road bridge 8 metres wide (Fig. 1), represent a form of bridge girder which is frequently applied. In order still further to simplify and cheapen bridge constructions of this

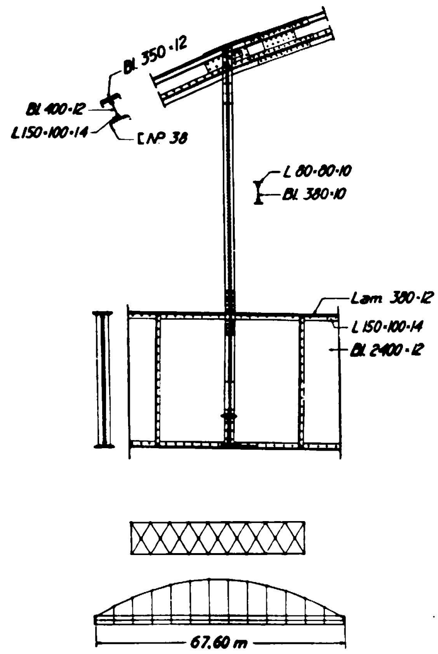

Fig. 1.

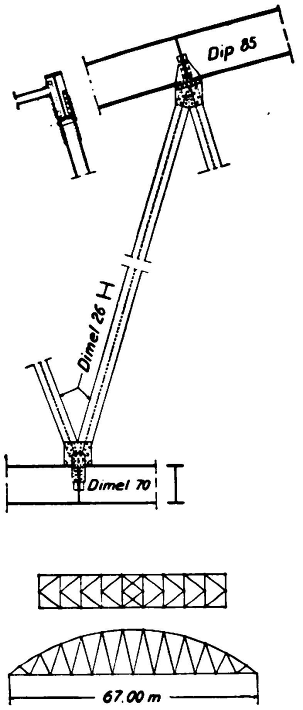

Fig. 2.

order of magnitude the form of girder referred to in 3) and represented in Fig. 2 was designed, consisting of parabolic girders with diagonal members but no verticals. Hitherto this form of girder has been very infrequently applied in steel construction, but systems similar to this have several times been carried out in reinforced concrete under the designation of arches with inclined suspenders (see *O. F. Nielsen:* Arched girders with inclined suspension members. Publications of the I. A. B. S. E., Vol. I., 1932, p. 355).

The system shown in Fig. 2 offers the following advantages as a form of girder for steel construction:

a) The maximum thrust is approximately constant throughout the length of the boom, so that no changes in cross section are necessary and the boom can, therefore, be formed from a rolled I-section without continuous rivet-lines. The upper boom may be curved where this is an advantage as regards appearance.

b) All the web members carry considerable tensile forces resulting from the dead load, so that the additional loading due to traffic causes no compressive forces or only small ones. Eeach bar may be formed from a relatively light I-section, so that continuous rivet-lines are eliminated.

c) As a result of the small forces in the diagonal members the connections can be very easily carried out, either by riveting or by welding. Fig. 2 shows an example of riveted construction.

A comparison between line 3) and 4) in the table shows that the parabolic girder means a considerable simplification in comparison to the bow-string girders, an advantage which in the case of light road bridges up to about 70 metres span may always be realised.

Summary.

Wide spans in solid web construction requirer in general more material than lattice constructions. Jar the purpose of calculating the economical advantage, which as a rule is with plated constructions, a mumber of fundamental comparison figures are given, which were derived from various bridge constructions in Denmark.

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VIIa4

Noteworthy Steel Structures in Austria.

Bemerkenswerte Stahlbauten in Oesterreich.

Constructions métalliques intéressantes en Autriche.

Dr. Ing. F. Glaser, Wien.

Since the Paris Congress of 1932, which unfortunately met when the world economic crisis was in full swing, Austria has witnessed renewed activity in all branches of the building trade thanks to the magnificent programme introduced by the Federal Government with a view to reducing unemployment. The opportunities thus offered have also been made available for the somewhat less extensive branch of engineering with which we are concerned, namely, the structural steel industry.

One of the most outstanding constructions in Austria is the new Reichsbrücke over the Danube, which is being built at present. This monumental bridge construction will be one of the largest steel structures in Europe. A classification of the biggest European suspension bridges according to the length of the main span may be of interest. The list is as follows:

(1) Cologne-Mülheim (cable) span $1 = 315$ m
(2) Budapest (chain) ,, $1 = 290$ m
(3) Belgrade (cable) ,, $1 = 261$ m
(4) Vienna (chain) ,, $1 = 241$ m.

Thus the Reichsbrücke at Vienna takes the fourth place among suspension bridges. It ranks second among chain bridges and second also, if placed according to the amount of material used, in fact, from this point of view it follows very closely on the Cologne-Mülheim Bridge.

The plan was obtained by means of a competition organised in 1933 by the Federal Ministry for Trade and Transport. Altogether 22 plans were submitted and produced a large number of very valuable ideas which were brought to bear on the solution of the exceedingly complicated engineering problem which confronted the architects.

An official publication dealing fully with this very interesting technical event is being compiled and the authorities intend it to appear at latest directly after the completion of the bridge. The present communication is not to encroach in any way on the official report and the information submitted now will therefore only be general in character.

The Judging Committee, composed of leading representatives of the official departments concerned and of the Technical College at Vienna, after carefully examining the plans, reached the decision that, from the point of view of economy and navigational technique, the plan "Freie Donaufahrt" (Freedom of Navigation on the Danube) (Fig. 1), with its main span of 170 m bridged by a trussed arch with solid web brace girder, was the one to be recommended for execution. This plan and three others were awarded prizes as offering a combined solution.

During the negotiations dealing with the letting of contracts for the work at the end of 1933, however, the Judging Committee's recommendation was dropped, the decision revised — mainly for reasons of appearance — and a decision taken in

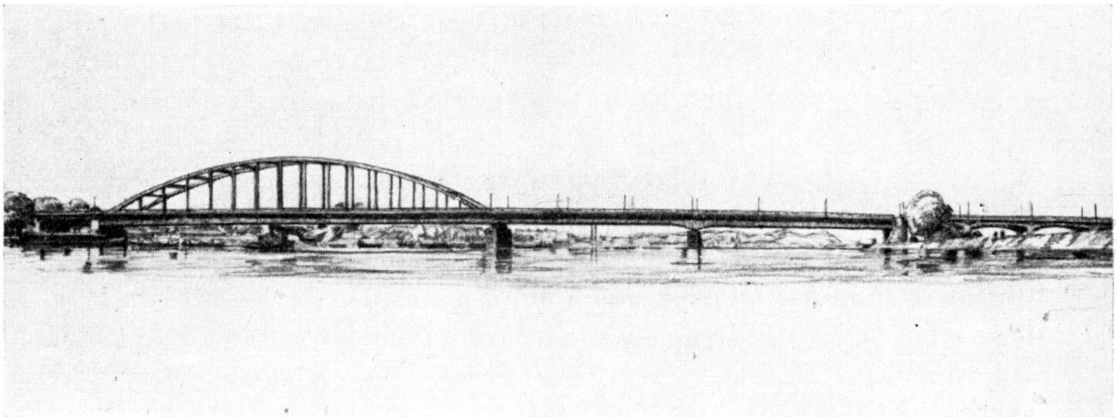

Fig. 1.
Reconstruction of the Government Danube bridge in Vienna.
(Design "Freie Donaufahrt" recommended for execution.)

favour of another solution which had also been awarded a prize, namely, the "chain bridge". This plan included an alternative design of a "cable" suspension bridge. The question as to whether the suspension member should be a chain or a cable

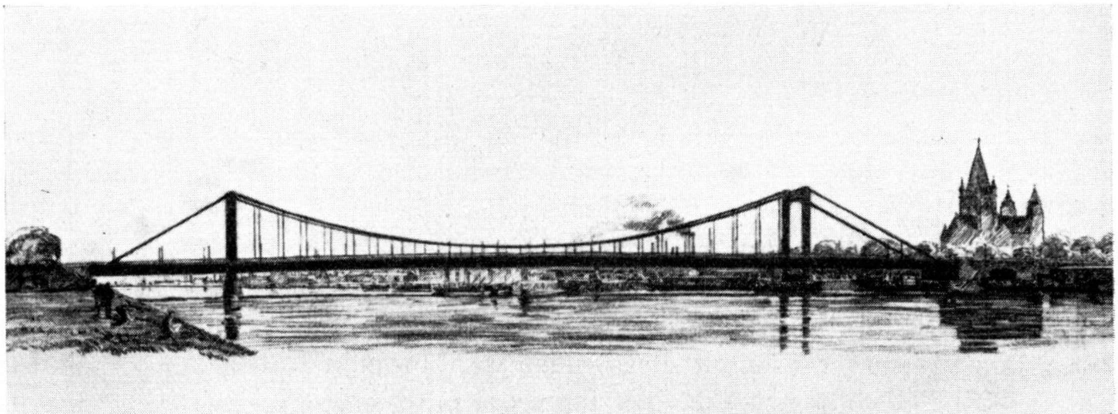

Fig. 2.
Reconstruction of the Government Danube bridge in Vienna.
(Design "Kettenbrücke" accepted for execution.)

was then raised, as indeed it is raised nearly every time a suspension bridge is planned. On the basis of an Advisory Opinion given by Professor *F. Hartmann* (engineer), it was finally decided to select the chain. The determining factors in this connection were the economic aspect, the increase of rigidity and last, but not least, considerations of appearance.

The plan accepted for execution had been designed by the Bridge Building Company Waagner-Biro, Ltd., of Vienna and Graz, and provided for an anchored chain bridge with a stiffening plate girder 4.30 m high, for bridging the central span of 241.2 m. The two side spans, each 65 m long, were made of solid web plate girders, the one on the left bank in one span, while the other — on the right bank — was subdivided by two columns, hinged top and bottom. The approaches to these girders are solid constructions. The chain over the side spans runs in a straight line from the pylons to the anchorage blocks (Fig. 2). The cross section of the design shows the roadway, 16.5 m wide with two footpaths each 3.5 m wide. The main girder distance is 19.1 m. The bridge has a capacity of two tramway tracks and four lanes of vehicular traffic. It will be wood paved, the pavement resting on a concrete bed which covers the suspension plates (Fig. 3).

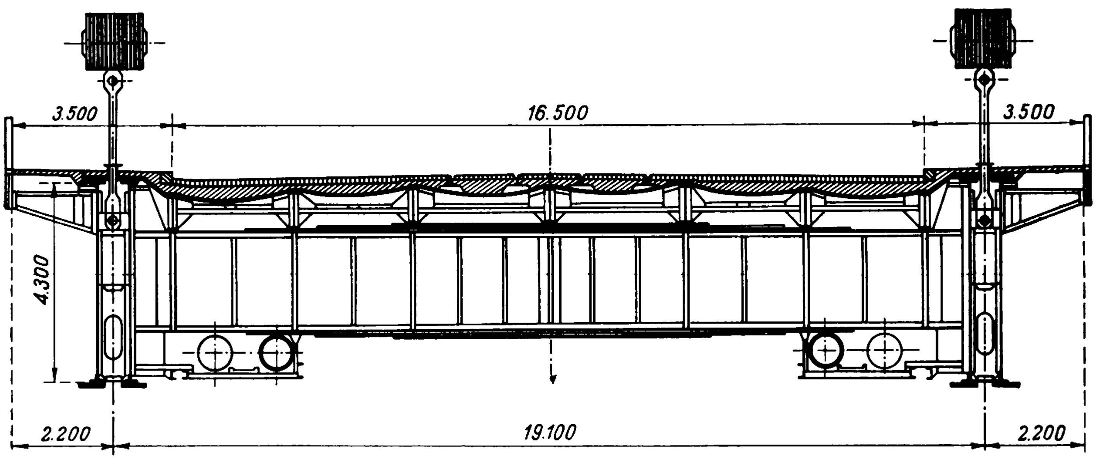

Fig. 3.

Reconstruction of the Government Danube bridge in Vienna.

Original cross section of river span.

The plan aroused great interest on account of its dimensions which are unusually large for bridge building technique. This interest was considerably increased when it came to putting the plan into execution.

Before the work on the new bridge could be started, the supporting structure of the old Reichsbrücke had to be removed (Fig. 4). In order to clear the building site for the new bridge, the axis of which coincides fairly closely with that of the old one, it was found necessary to move the existing bridge some 26 m down stream. The displacement of this continuous structure was carried out in one operation on September 12th, 1934, and took five hours to complete. The old bridge on its new site is being used as a subsidiary bridge while the new one is under construction. The cutting of the connections of the old bridge prior to its removal and its subsequent junction to the previously prepared wooden connecting bridges was accelerated to such an extent that pedestrian traffic was held up for approximately 30 hours only, while the interruption of road traffic lasted only about 48 hours.

As soon as the old bridge had been removed, work was started on the scaffolding and this was quickly followed by the erection of the new bridge. This preliminary work included putting together the stiffening girders on a staging about 85 m long and 25 m wide. After moving this structure toward midstream new sections were added at the shore end (Figs. 5 and 6). The usual procedure for advancing over rollers and fixed fulcrums was not followed, instead of this the structure was moved forward on trolleys running on fixed horizontal rails. On account of the limited length of the rails, the lengths of which were determined by the width of the intermediate supports, the advance had to be accomplished in steps. Supports had to be supplied in keeping with the carrying capacity of the stiffening girders. The piers of the old bridge, together with three wooden temporary supports, served this purpose. After one journey of approximately 2.4 m the bridge was raised by upright hydraulic presses placed beyond the trolley track, the trolley was then placed in its original position, and after the structure was lowered on the trolleys, the next

Fig. 4.
Reconstruction of the Government Danube bridge in Vienna.
Old construction immediately before shifting.

section of the journey was effected (Fig. 7). The erection of the stiffening girders, cross girders and wind bracing was completed by March 1935.

Meanwhile construction proceeded apace. The new abutment for the left hand pylon was erected by means of pneumatic caissons. In this connection the contractors had a surprise for ground conditions proved to be different from what had been expected after the examination of the soil made previously. It was found that the soil was not capable of bearing the loading that had been contemplated. As a result the bearing area of the foundation of the piers had to be extended by consoles which projected beyond the edges of the caissons. But serious doubts were entertained as to whether it would be wise to build an anchored chain bridge.

The plan which had been awarded the prize had to be revised and extended and its re-examination led to the putting forward of two proposals concerning the continuation of the bridge: (1) to increase the dimensions of the anchorage blocks, or (2) to revise the whole steel structure planned, and instead of making an anchored chain bridge to construct one with compensated horizontal thrust, this latter having vertical foundation loadings only. It was decided in the end to alter the steel structure. The decision was influenced by considerations of increased economy, and a higher safety factor, as, even with much larger anchorage blocks the risk of considerable lateral displacement would not have been eliminated.

The main thing was to reinforce the very carefully calculated stiffening girder which had been designed on the basis of a very simple procedure elaborated when

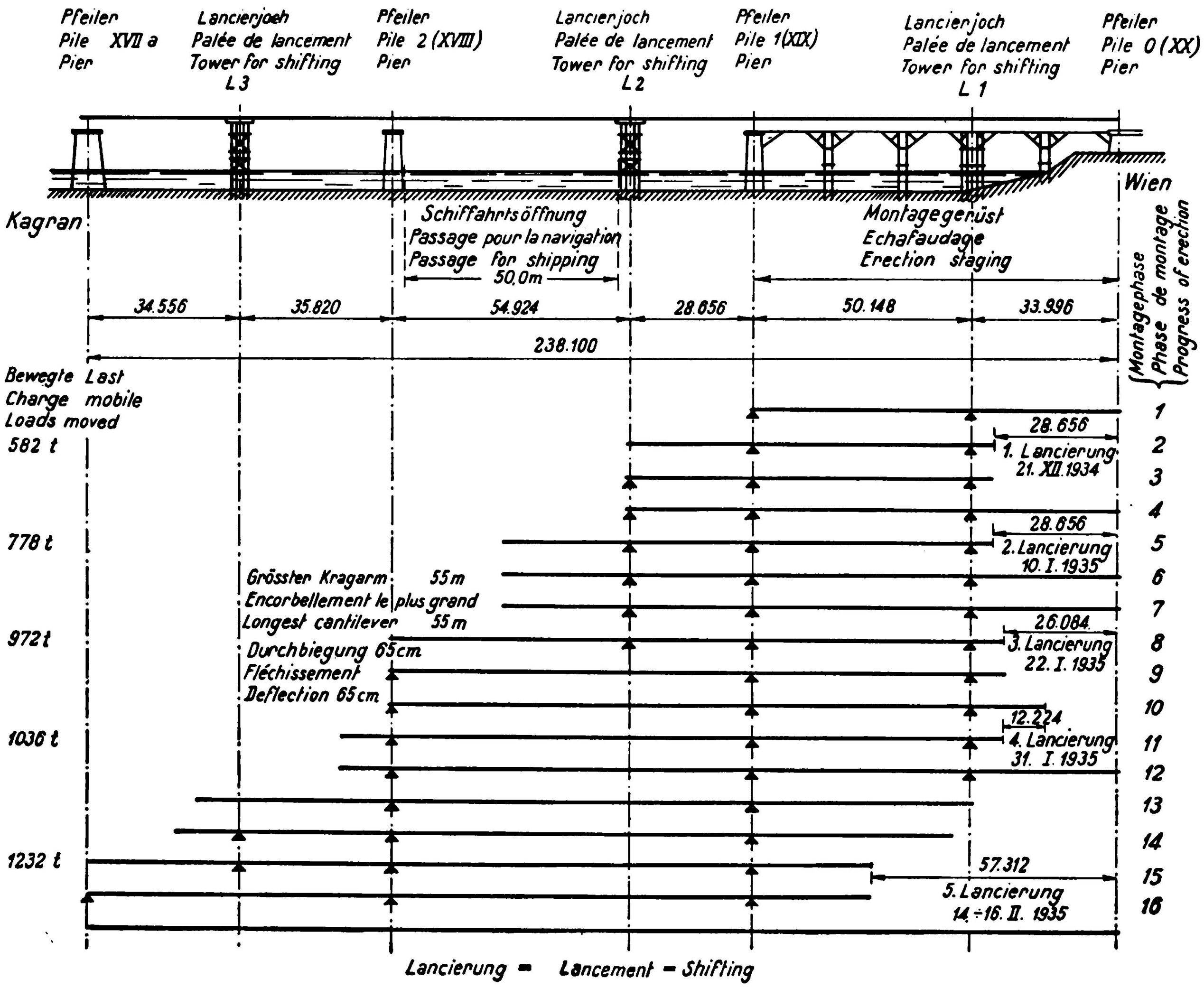

Fig. 5.

Reconstruction of the Government Danube bridge in Vienna.

Phases of shifting.

the bridge was built (called deformation theory) and enabling very precise calculations to be made. This reinforcement would have to meet the additional horizontal shear of some 7,000 tons transmitted by the chain. This was done by inserting in a vertical position four sets of plates having an average dimension of 640.150 mm (Fig. 8). Two such sets of plates, connected to the webs of the stiffening girders, were well braced so as to form buckling-proof members, and in the calculations they were considered as such. It must be remembered that the stiffening girders had already been erected. It would take us too far if we were to refer separately to all the very interesting arrangements made for inserting the reinforcement. We will merely

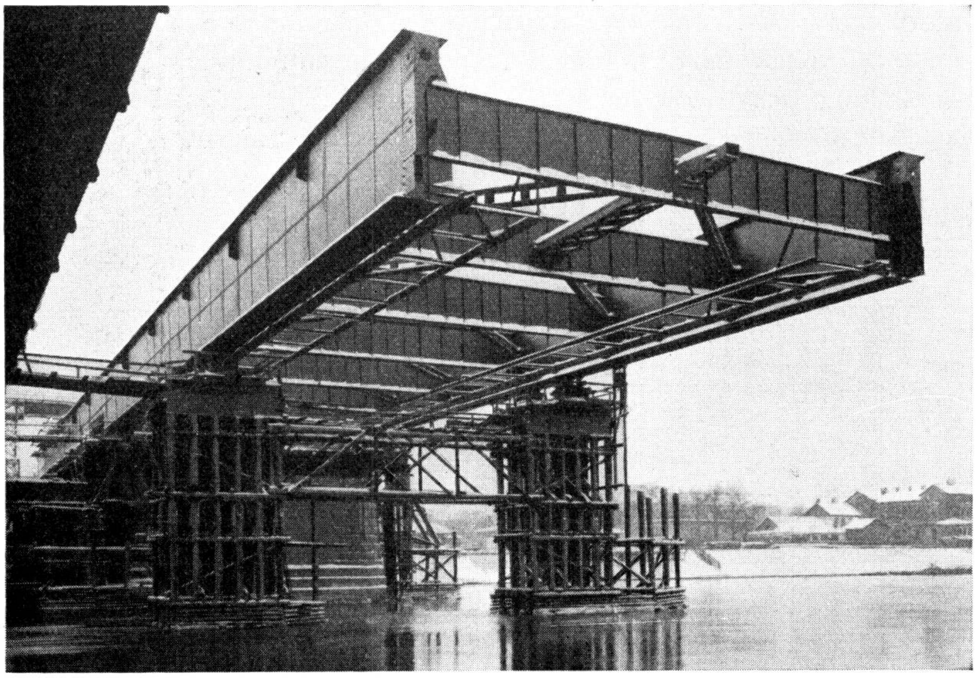

Fig. 6.
Reconstruction of the Government Danube bridge in Vienna.
State after 2nd phase of shifting.

mention that by altering the height of the supports for the stiffening girder which rested on seven supports, the part of the structure being worked on was relieved of all strain.

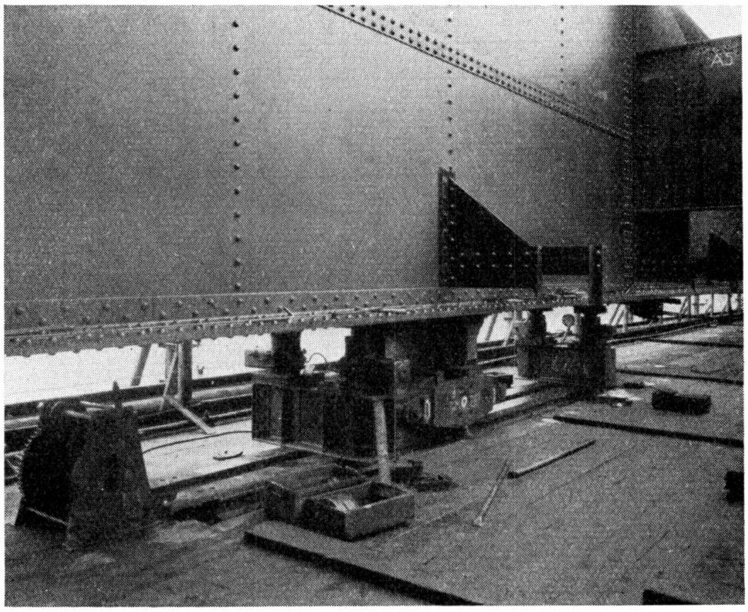

Fig. 7.
Reconstruction of the Government Danube bridge in Vienna.
Shifting track with jacking arrangement.

The increased weight of the structure produced higher strains in the chain. To be able to maintain the sections of the chain, its sag was increased by 2 m. To accomplish this, the length of a section of the pylons had to be increased.

The outside main girders of the lateral spans, which had originally been of the single-web type, were now converted into powerful box girders so as to meet the horizontal shear induced by the chain.

A few brief remarks on construction details may be mentioned here. The most important supporting member, the chain, has an average height of 1.20 m and is composed alternating of 13 elements of 22 mm thickness and of 12 elements of 24 mm thickness. The links of the chain are rectangular and at the ends where the chain bolts of about 450 mm diameter pass through them, they are reinforced by plates. The dimensions and connections of these were determined by the Material Testing Laboratory of the Technical College at Vienna (Prof. Dr. *F. Rinagl*, engineer), after very thorough experiments had been carried out. We have already referred to

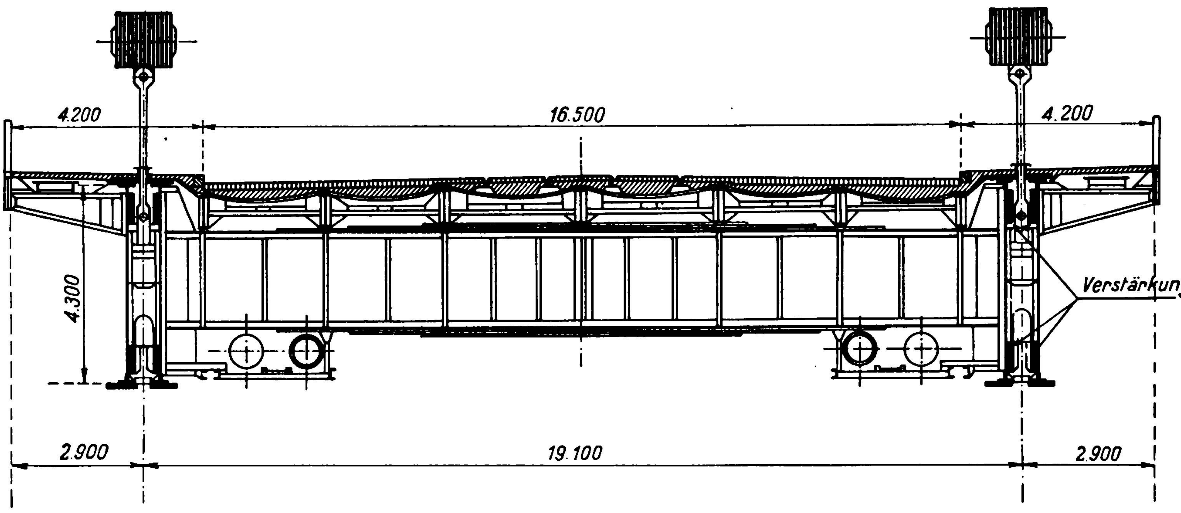

Fig. 8.
Reconstruction of the Government Danube bridge in Vienna.
Strengthened cross section of river span.

the stiffening girder (4.3 m high) and its reinforcement. The connection between the back stays and the outside main girder of the side openings is effected by large bolts having a diameter of 900 mm and which are placed slightly below the axis so as to partially relieve the bending moments in the outside main girders. The point of intersection of pylon and stiffening girder raised structural difficulties as both these members had been terminated. This complication was solved in the following manner: in order to transmit the horizontal shear, stout steel castings were fitted to the central and side openings, these pass through comparatively small windows in the pylon and are shored up against a spherical bolt which is perpendicularly supported and situated in the axis of the pylon. This prevented a weakening of the upright post of the pylon and at the same time a clear transmission of the forces was secured. Further particulars will be contained in the report the publication of which has been announced.

The following building material is being used: carbon steel St. 55, 12 for the chains, suspension members, pylons and outside main girders of side spans; carbon steel St. 44.12 for the stiffening girders, roadway and interior main girders of the side openings; cast steel Stg. 60,81 B for the bearings, forged steel St. 55.11 for the bolts and connections for suspension members. The total weight of the steel used is approximately 12 000 tons.

Next year, with the completion of this bridge, Vienna will possess an additional attraction.

Another bridge engineering feat was accomplished towards the end of 1933 when the old double track railway bridge belonging to the Eastern Railway Company and crossing the Danube at Stadlau, near Vienna, was replaced by a new one (Fig. 9).

Fig. 9.

New structure of railway bridge over Danube in Vienna-Stadlau.

The new bridge spans the Danube with a large meshed truss over four openings of about 80 m. The trusses are continuous over two spans. On the side of the town the bridge is joined to the Kaibrücke which is a solid web cantilever girder of the "Gerber" type, having seven bays of 12 m span each. In the even bays are the cantilever girders, which form with the two columns two legged bents. On the Stadlau side there are two bays of 40 m and then ten bays of 36 m each which span that part of the Danube which is often flooded. These bays are connected by continuous two span solid web girders, just as in the case of the main span. The two last bays form an exception; on account of a bend in the railway track the continuous girder is replaced by a cantilever "Gerber" girder.

With the exception of the main span, both the old and the new bridge had been designed so as to form two singletrack structures placed alongside of each other. This was done to maintain traffic on one structure while the other was replaced by the new one. In the case of the main span, it was necessary to shift the bridge to provide space for the new one. The old bridge was different in that it had five bays over the river; this was done because when it was built (in 1870) no definite information was available concerning the size of the river bed which was at that

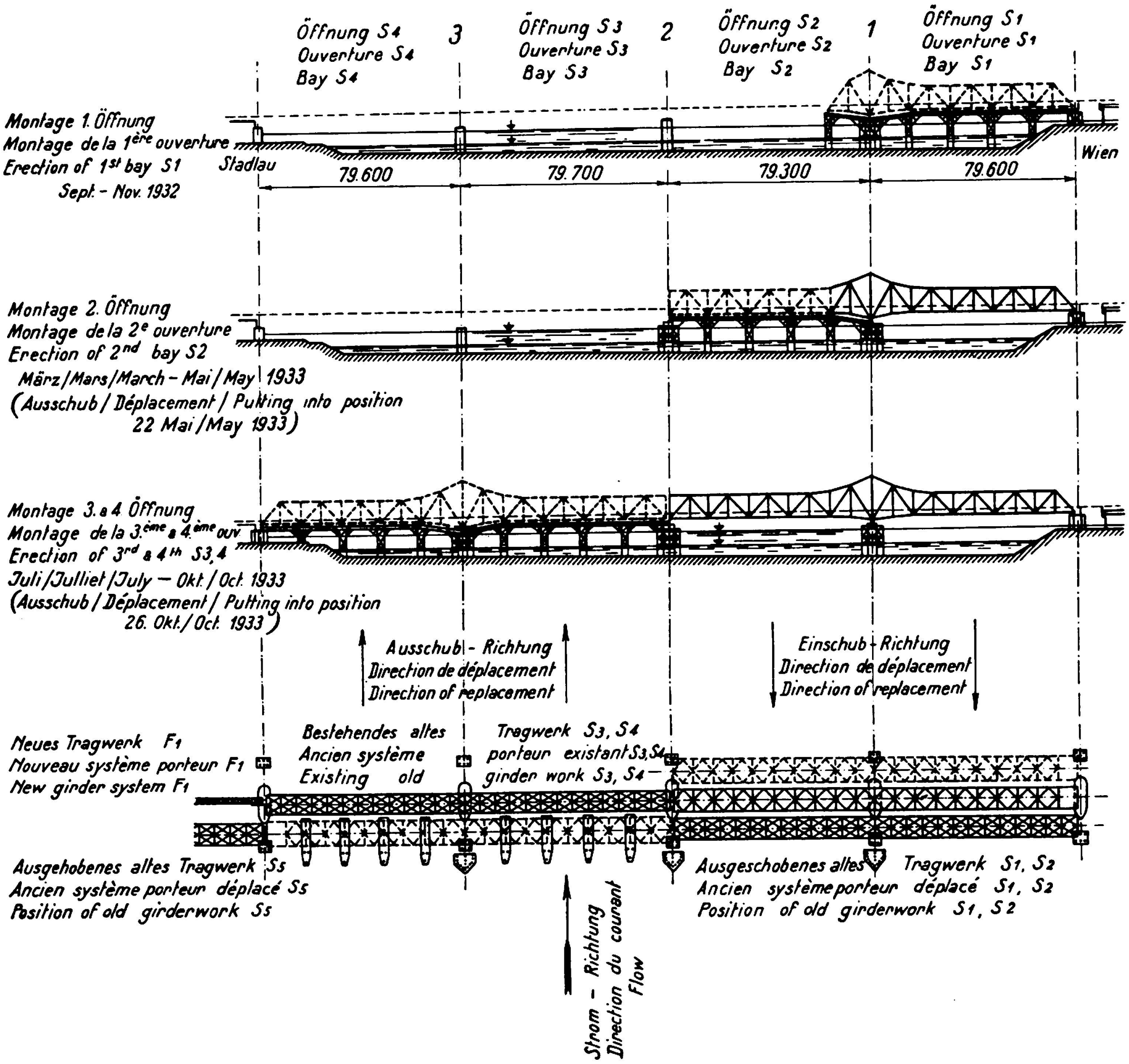

Fig. 10.
Erection of railway bridge over Danube in Vienna-Stadlau.

time excavated. To-day the bed of the river is not quite as broad as the four openings, so that there was no need to extend the new bridge over five openings. After cutting the old bridge over the second and fourth piers and partly strengthening them by wooden bents, the new structure of the two first spans could be built on the down stream side of the existing bridge. After shifting the bridge upstream the new bridge could be shifted in its final position on the old piers. The work proceeded in the same way at the last (fifth) opening. Here a second intermediate pier had to be erected, as the old bay spanning the river was replaced by a continuous plate girder of two spans. The new structure of the third and fourth span over the river was erected next to the old bridge on the up stream side of the existing bridge. That was done to simplify the transport of the material which passed over the first two bays of the old bridge. Fig. 10 shows clearly the processes just described. In the case of the two main displacements (first and second, or third and fourth openings over the river) the load to be removed was about 2 000 tons. The distance over which it had to be transported was roughly 11.5 m.

About 7 300 tons of structural steel were used in the construction, the quality being St. 44.12. The wrought-iron bridge weighed only about 3 200 tons.

About road bridges the following is worth mentioning: the reconstruction of two bridges in Vienna. In the year 1931 the Augarten Bridge over the Danube Canal was completed. This is a bridge of 80 m length, consisting of seven continuous plate girders entirely below the bridge decking. From aesthetic point of view this bridge can be regarded as giving full satisfaction and has been well received by foreign experts (Fig. 11).

The other bridge in Vienna, the Rotunden Bridge also over the Danube Canal is at present under construction.

Before concluding we would mention a noteworthy highway bridge: the Rotundenbrücke (Fig. 12), at present under construction over the Danube Canal at Vienna. The structure consists of solid web two-hinged tied arches of 67 m span. The supporting structure could not be built below roadway, as available space was very limited. The method of erection of this bridge is interesting. A fairway of 35 m width was required for the river traffic. The scaffolding could not be erected over the whole site so that a kind of "catapulting" process, not unlike that described in the case of the Reichsbrücke, had to be carried out. In order to construct the span over the fairway, the steel structure was first made to project some 17 m beyond the scaffolding. Then an open 670-ton tug, fitted with suitable staging, was floated below the projecting part of the bridge. Leverage was effected from the ship and so the two fulcrums of the bridge on the river side were removed. Then the bridge, resting on two trolleys on the fixed scaffolding and on the tug was moved along its axis some 18 m. After the structure had been supported on the other shore the tug was released. This work necessitated stopping the river traffic for two days. Building was then proceeded in the same way as before by advancing in stages.

The latest application of welding in bridge construction is found in the highway-bridge across the Mur in Styria. This continuous solid web girder bridge with two spans of 39 m each was erected by exclusive use of electric welding (Fig. 7, III d Zelisko).

In both of these constructions referred to, high grade structural steel St. 44.12 was used. At present this quality is being increasingly applied in Austria as a standard building material.

Leaving steel bridge engineering we will now give a brief survey of the application of steel to structural engineering.

There is not a great deal to report about large structures erected recently.

The widespread development of broadcasting and the improvements constantly being made to broadcasting stations have given an impetus to this industry in Austria and new installations are being erected in many places. The most important

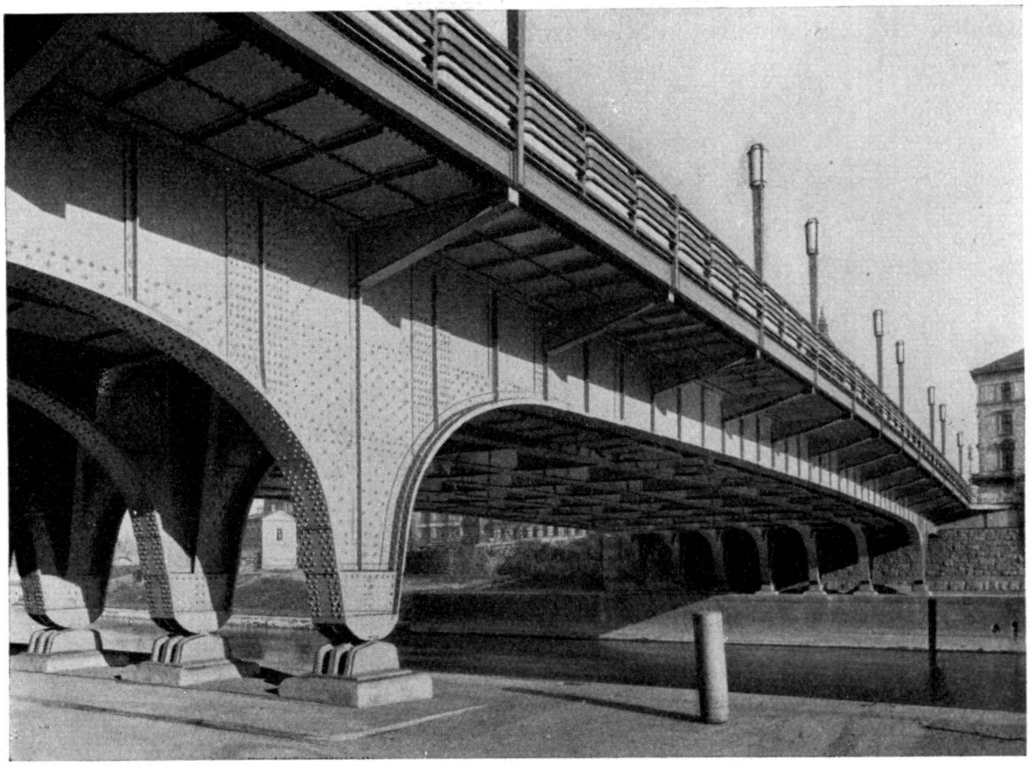

Fig. 11.
Augarten Bridge over Danube canal in Vienna.

of these is the Viennese broadcasting station on the Bisamberg near Vienna. Fig. 13 shows its two masts, each 130 m high.

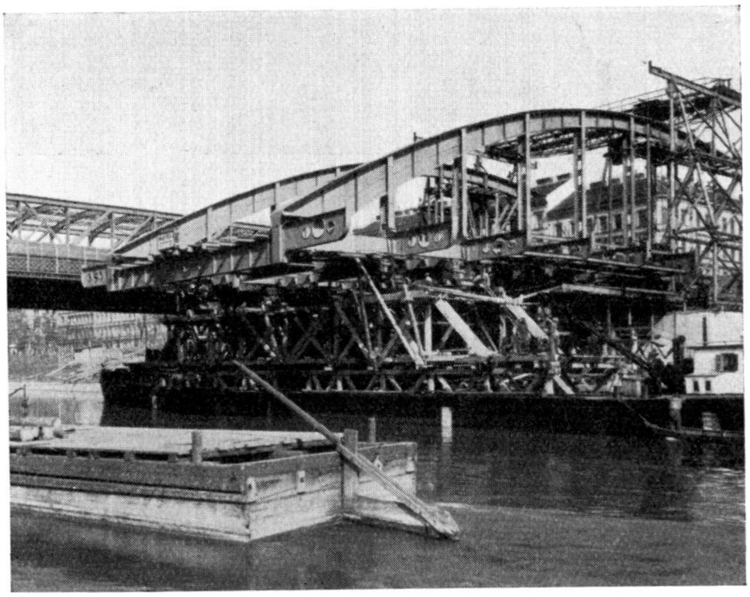

Fig. 12.
Rotunden Bridge over Danube canal in Vienna.
During shifting with pontoons.

This Paper aims at presenting a small selection of structures exemplifying what is being done by the Austrian structural steel industry. Even if the post-war political machinery of Central Europe has turned Austria into a small country and deprived her of nearly all her foreign markets, the Austrian structural steel industry has been able to maintain its technical standing. It collaborates constantly with scientific research and as a result of this, many new policies have been laid

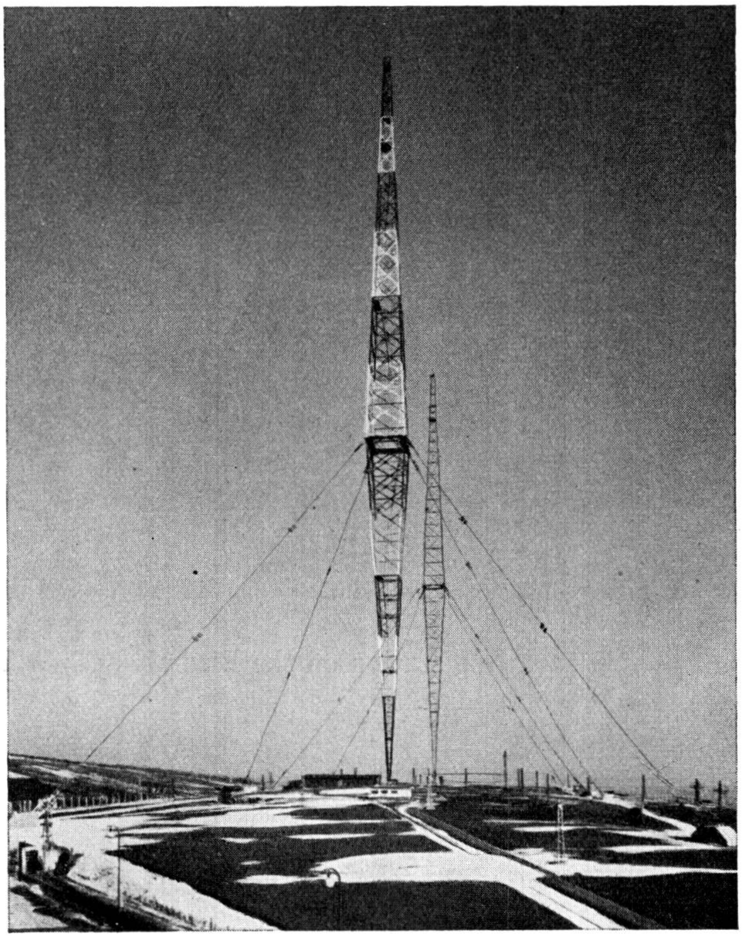

Fig. 13.
Broadcasting masts on the Bisamberg near Vienna.

down thanks to the initiative of Austria. Already in 1919, Austria was instrumental in inducing a better utilisation of this building material, and her example was finally followed all over the world. Her latest move, which is based on the most recent results of scientific research and practical experience aims at further improving the methods of utilisation of structural steel along systematic lines.

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Summary.

In this Paper a review of Austrian activity in structural steel building in the past few years is given by means of a brief description of certain selected steel structures. Special emphasis is laid on the Reichsbrücke across the Danube at Vienna which is at present being built and which is the second largest chain bridge in Europe.

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Some French Steel Structures executed during 1932—1936.

Einige in den Jahren 1932 bis 1936 in Frankreich ausgeführte Stahlbauten.

Quelques constructions métalliques exécutées en France de 1932 à 1936.

Colonel L. Icre,
Direkteur de l'Office Technique pour l'Utilisation de l'Acier, Paris.

I. General considerations.

The years from 1932 to 1936 were a period of stabilisation after the crisis which had affected every branch of industry and the metal industry in particular. Following on the exceptional years 1929 and 1930 the production figures still declined, but they did so less violently. From 1932 onward they became stable or showed some slight trend towards recovery: an index, if a slight one, of the reviving activity.

One of the most striking of the characteristics discernible in the work of the French builders of metal structures is the quest for improved or new methods designed to secure lower costs through better use of the possibilities inherent in steel.

Among these new methods a special place must be given to electric arc welding which has steadily progressed to a level of perfection that is a guarantee of safety, and in doing so has opened up new horizons to the art of construction in metal. At the same time real advances have been made in resistance welding, so that heavy structures are now included in its scope.

The demand for lightness of structure without detriment to solidity has led to the wider and wider use of high-tensile steels. In certain cases even rustless steels, despite their higher cost, have been found advantageous.

The same search after lightness has given rise to some new and original solutions, particularly in reference to floors of buildings, bridges, and aeroplane sheds.

It will not fall within the scope of this report to describe all the striking ideas that have been brought out during the past few years — most of them still in an uninterrupted course of development. We shall confine ourselves to a brief review of the most essential of the achievements.

II. Steel frames of buildings.

The number of steel framed buildings built between the years 1932 and 1936 is so great that only a selection can be dealt with, and five of the most important are mentioned here as representing different fields wherein the use of steel has been found particularly well adapted to the end in view.

1) The Shell Company's building in Paris (Figs. 1 and 2).

This structure was erected by the firm of Borderel and Robert together with Baudet, Donon and Roussel. It covers an area of 8000 m² and is notable not only for its architecture but also for the details of its planning.

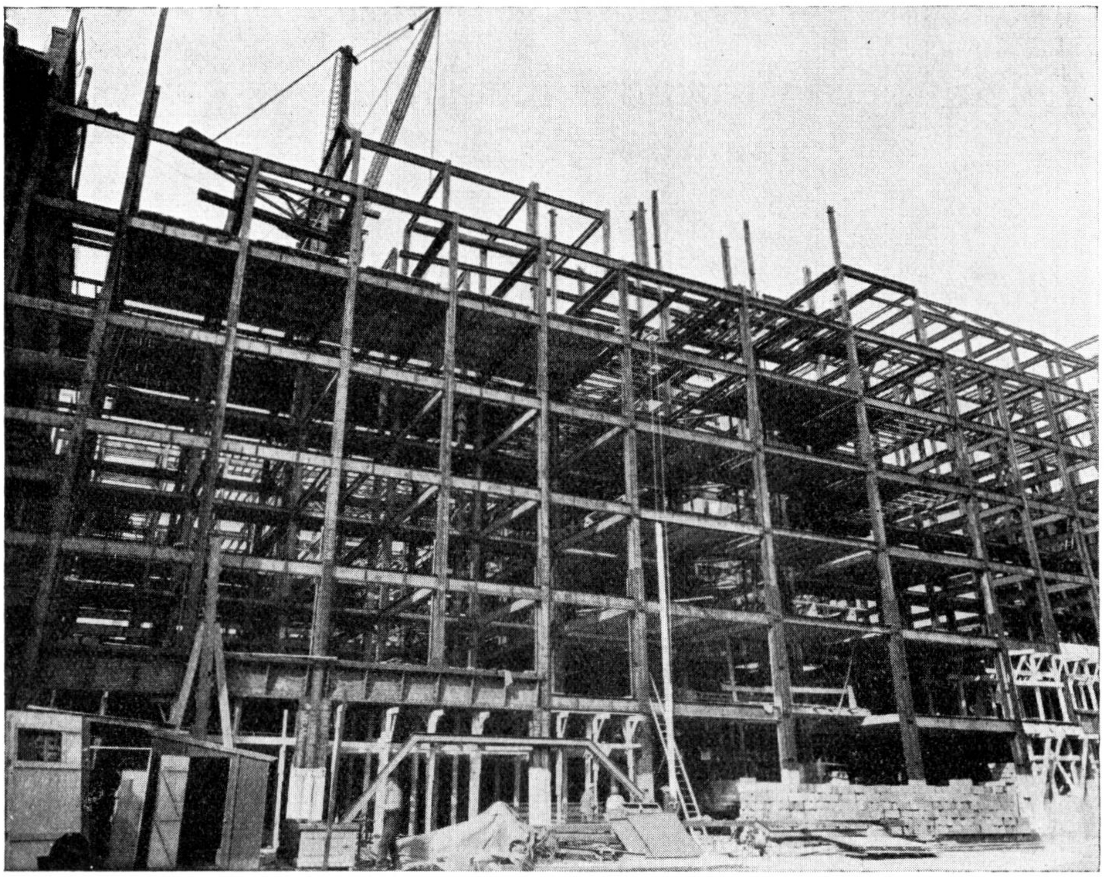


Fig. 1.
"Shell" Building. Steel frame work.

The building includes foundations and three basement storeys in reinforced concrete and above these some 5000 tonnes of steel framework enclosed in concrete to reduce the risk from fire. Everything has been designed and calculated with a view to a maximum attainment of those features which offer the greatest advantage. For instance, a point has been made of using members with very broad flanges up to sections such as 500 mm $\times$ 300 mm and no compound members have been used, the whole of the framework being encased in reinforced concrete to protect it from fire.

The facings of the building are attached to the framework by the use of new methods of construction, and their architectural composition is both sober and distinctive. It is characterised by high pilasters, surmounted by pinnacles, which emphasize the vertical lines. The great length of the facades is broken by projections of greater height, bearing the name "SHELL' in large letters.

2) *The medical school at Lille* (Fig. 3).

The new medical school was included in the first stage of construction for the *"Cité Hospitalière"* at Lille. The premises consist of a symmetrical group of buildings extending over a frontage of about 140 m and having a developed length of 230 m. The central portion and the wings have six storeys, each with an area of 3100 m². The west and east lecture theatres, with their adjoining rooms, each cover an area of 360 sq. m. and correspond in height to two storeys of the adjoining portions.

The framework, including floorbeams, main and secondary girders and stancheons, is of steel, and in general the stancheons are supported on the first floor which is of reinforced concrete. 2000 tonnes of steel were used for the

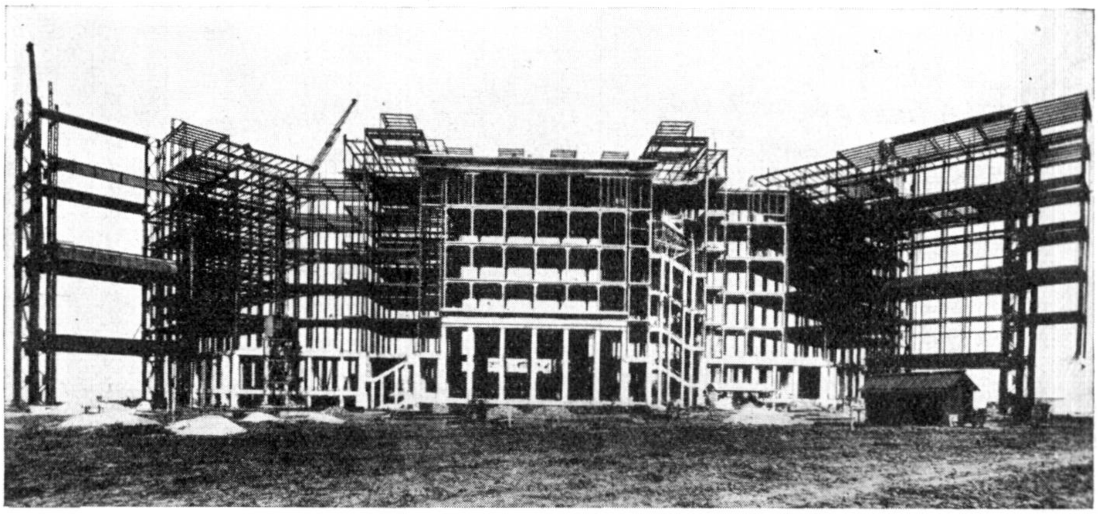


Fig. 2.

Medical Faculty Building, Lille. Steel frame work.

framing as a whole. At the seventh storey, over the whole length of 90 m between the east and west theatres, there is a steel roof construction over the accomodation for animals.

Further applications of steel framing occur under the rising seats of the Great Theatre seating 600 and in the roofs of the Dissecting Room and Operating Demonstration Theatre.

This imposing steel frame was raised from the ground in the course of a few weeks with the aid of electrically operated machinery and derricks. Notably in the construction of the facade, visitors never failed to be attracted by the combination of gracefulness and strength present in the powerful electric travelling and turning crane, with three motors, the jib of which could reach a height of 42 m and could reach horizontally a distance of 20 m.

Reference will now be made, from a technical standpoint, to an interesting detail in this superstructure: at the request of the commission of experts under the chairmanship of Monsieur Dautrey, Director of State Railways, the firm of Paudon et Cie. to whom the steelwork was entrusted made use of the principle of continuous and superimposed frames. This implies an arrangement of specially designed connections to ensure the vertical and horizontal bond between the columns and the girder work carrying the floors.

Furthermore, the columns were formed of high-strength steel so as to lighten the metal framework while at the same time adding to its strength.

3) The Rex Cinema in Paris (Fig. 3).

The Rex Cinema covers a trapezoidal-shaped site of about 1900 m^2 and was built by the firm of Baudon jointly with that of Venot-Peslin.

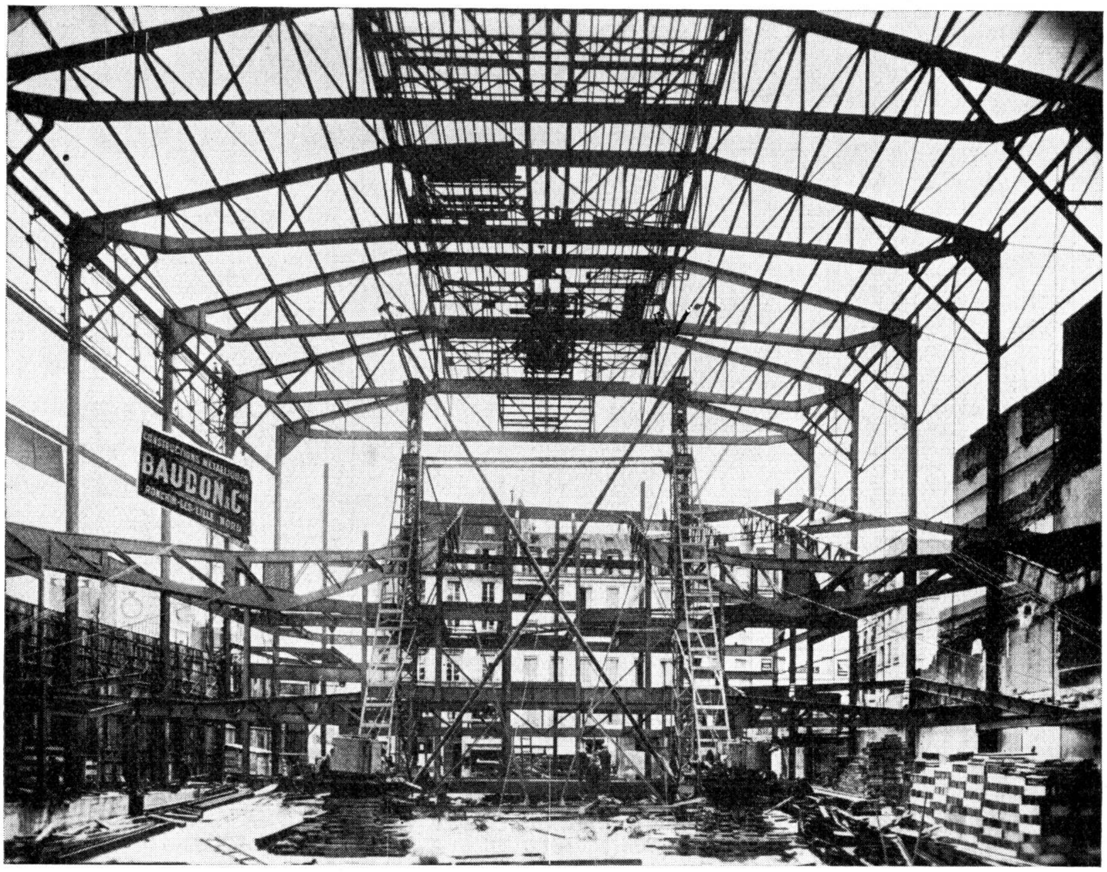


Fig. 3.
"Rex Cinema". Steel frame work.

The auditorium has an average width of 34 m and a depth of 31.5 m along its axis. The substructure of the building, up to ground floor level, is in reinforced concrete, and the remainder is supported on a steel framework.

The principal members of this framework were erected into place by means of two large masts 36 m high, each separately capable of lifting 15 tonnes. As soon as the general framework was in position work was begun on filling in the walls and the screens intended to isolate adjacent buildings in case of fire; meanwhile the work of erecting the steel framing of the balconies procceded inside.

The wall-fillings were erected with great speed in the form of hollow concrete blocks. These were pre-cast on the site and laid in cement mortar several courses at a time with reinforcement in each joint; subsequently they were bonded and reinforced by round steel bars placed vertically through the hollows into which concrete was poured. In this way a monolith was obtained when the concrete had set.

Roof trusses.

The main roof trusses, seven in number, have been built as lattice structures, and are so designed as to produce no laterial thrust on the supporting columns which are relatively light in section. A further consideration was the necessity to ensure that the upper boom should remain absolutely horizontal under load as it forms the support for office floors; this requirement was met by fabricating the work with a suitable camber.

The middle part of the upper boom is horizontal over a length of some 12 m to carry the floors of offices which are situated over the whole length of the auditorium. The lower boom is horizontal over the greater part of the span but rises on each side towards the supports, where it is reinforced with large gusset plates which also serve to make the connection with the supporting columns. At the point of intersection with each of the oblique members there is placed a strut to afford lateral bracing and to pick up the suspension bars carrying the edge of the cupola over the auditorium. This strut is relieved in its middle portion by two diagonal members, and at the last truss, which is the smallest, it is replaced by a horizontal bar which is in line with the horizontal portion of the lower boom and connects with the stancheons so as to afford lateral bracing also.

It may be noted that in no case does the axis of the truss coincide with the centre of the span, the difference in length of the two half-units being 1.40 m in the largest truss.

The connection between the bracing, the diagonals and the stancheon by means of bolts in oval-slotted holes was not finally made until the truss was under load, so that the deflection should already be present and there might be no risk of causing any thrust at the points of intersection.

As the building is trapezoidal in plan all seven trusses have different spans, varying from 33.90 m to 38.60 m, and their heights likewise range from 3.50 m to 6 m. The largest truss, weighing over 24 tonnes and carrying a total load of about 200 tonnes, is the closest to the screen. Consequently the depth available between the ceiling of the auditorium and the floor of the offices over the theatre varies from one end of the building to the other, being at its smallest over the upper seats of the amphitheatre. This depth is turned to account for housing part of the equipment, especially the ventilation ducts.

Galleries.

The general framework carries the steel frames of the two galleries which are not supported on any intermediate column. The balconies are borne by cantilevers each erected behind one of the columns of the main framing, and their projection is reduced by an intermediate girder running parallel to the front of the screen which picks them up near the middle of their length and in turn rests on two skew girders supported on columns forming part of the framework. So far as possible the span between supports has been made to balance the cantilevered portion, when under average live load.

The two skew girders carrying the trimmer of each gallery (the span of which is thereby reduced to two-thirds the width of the auditorium) are supported externally on a steel column in the outside wall and internally on a column in the curved wall dividing the auditorium from the foyers. Each of the skew girders consists of two twin lattice girders forming a box of pentagonal (almost triangular) shape, the greatest height of which is in the plane of the intermediate girder. The latter carries the cantilever beams of the galleries and is likewise a lattice box girder but of constant height.

The eight cantilever girders which carry the floors of the galleries are of lattice construction and nearly triangular in shape. At the back they frame into one of the columns of the curved wall of the foyers, and under each gallery six of them are picked up on an intermediate girder while the other two bear directly on the skew girders at their intersections with the latter. Being thus supported near the middle of their length these brackets form true cantilevers, the upper portion between supports serving to balance the projecting portion.

The gallery brackets have been designed and tested under a moving live load of 500 kg/m^2.

4) The new Citroën works at Paris (Fig. 4).

Finally, mention may be made of the group of steel frameworks built by the Compagnie Saint-Quentinoise de Construction for the Citroën Company. This workshop structure, erected on the Quai de Javel in Paris, comprises a central

Fig. 4.
Citroën Motor Works.

hall of 24 m span by 228 m long, with a height of 12 m to the underside of the roof trusses. The stancheons carry a gantry for a 10-tonnes travelling crane.

Five bays of 16 m span connect on either side of the central bay, and these include steel floors for live load of 500 kg/m².

The steelwork as a whole amounts to 8000 tonnes, and it is of interest to observe that delivery and erection was effected at the rate of 1000 tonnes a month.

III. Bridges.

From among the many bridges erected by French constructors between 1932 and 1936 we shall select three of different types which are noteworthy for their size and the neatness of their design.

5) Rolling-lift bridge at Dunkirk (Fig. 5).

This is a combined bridge for both rail and road traffic and was built by the firm of Dayde over the new dock in the Port of Dunkirk.

It includes a roadway 5 m wide with a railway of standard gauge laid on the centre line, and two footpaths 1.25 m wide. The bridge is of the Scherzer rolling lift type, and its maximum opening corresponds to a rotary movement

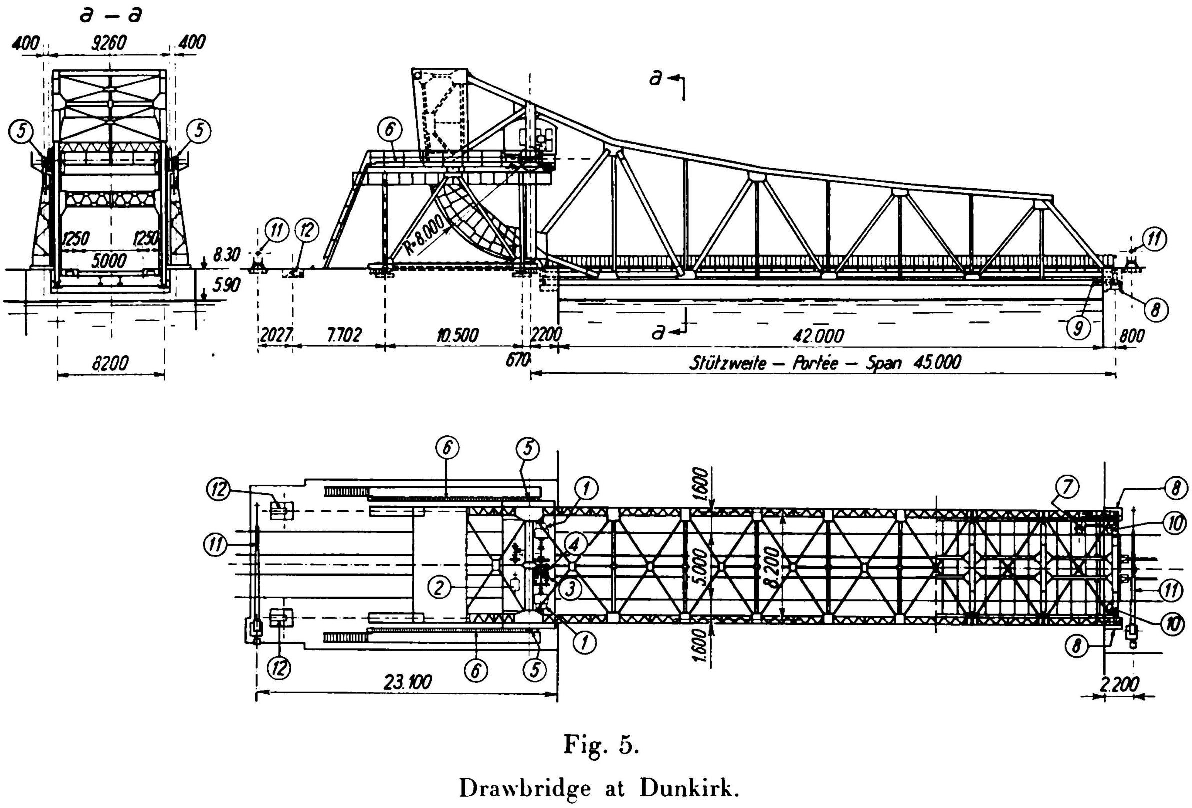


Fig. 5.

Drawbridge at Dunkirk.

1) Electric motor 162 HP.
2) Petrol motor. Renault 46 HP.
3) Mégy brakes.
4) Electro-magnetic brakes.
5) Cog-wheel drive.
6) Cog-rail.
7) Motor for locking gear.
8) Bearings.
9) Locking gear.
10) Shock absorber.
11) Turn pike.
12) Anchorage for bridge when open.

of 85^0 combined with a movement of translation of 11.84 m, leaving a clear distance of 42 m between the walls of the dock.

The lifting span includes the following parts:

— A pair of stringers carrying the roadway, railway and footpaths, these being lattice girders of varying depth and of 45 m span spaced at 8.20 m centre to centre, connected by cross girders at 5.5 m centres carrying the deck.

— A rolling portion formed of two girders in the shape of circular arcs, inter-connected above the roadway by a steel framework.

— A counterweight supported on this framework.

The side of the abutment under the rolling portion of the bridge is furnished with two rolling tracks 12.8 m in length, and when the bridge is opened or closed the circular girders roll in these. The abutment also carries two mountings for racks arranged to engage with toothed pinions forming part of the operating gear.

Operating gear.

All the operating mechanism is mounted on a trolley with a horizontal platform guided to move horizontally when the bridge is lifted or lowered, through a patented device known as the Daydé system. The motion is given by a direct current electric motor of 162 H.P. fed from a Léonard transformer set in the control cabin, and the latter is supplied by 3-phase alternating current at 50 periods with 380 volts between phases. A standby motor placed opposite the first one can be brought into use in case of need.

The bridge is held fast in any position by the agency of two electromagnetic brakes when the current is cut off. An automatic brake is provided which opposes any tendency of the bridge to move in the wind. A petrol engine for emergency use is installed below the machinery platform and there is also hand operating gear worked by hanging chains.

The bridge can be opened or closed in 100 seconds against a contrary wind pressure of 50 kg/m^2.

Locking gear.

A lock bolt, operated electrically from the control cabin and interlocked with the lifting gear, is provided at the end of the cantilever.

Anchorage in open position.

The bridge, when fully opened, can be anchored by means of connecting rods securing the counterweight box to a metal pedestal embedded in a foundation block. This anchorage is designed to withstand a wind pressure of 150 kg/m^2.

Shock absorbers.

Two shock absorbers are arranged at the end of the lifting portion of the bridge to take up any impact that may occur on completion of closure.

Barriers.

At the entrances to the bridge over each abutment there are two lifting barriers worked electrically from the control cabin. These are fitted with both audible and visual signals.

The work is the fourteenth lifting bridge of the same type built by the firm of Daydé.

6) The Moissac bridge (Fig. 6).

The old Moissac bridge carrying the Bordeaux-Sète railway across the Tarn had failed through settlement of the piles resulting from a severe flood in that river. The new bridge, built by the firm of Daydé, has three continuous spans of varying height, namely —

120 m in the central span,

95.7 m in each side span, these being supported on two piers in the river and on two abutments.

The main girders are of lattice construction varying in height, and are at 9.5 m between centres. The lower booms are horizontal and the upper booms

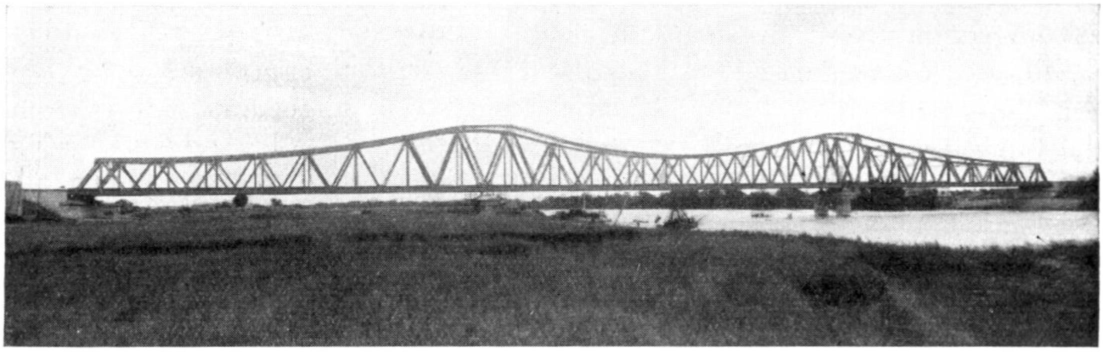


Fig. 6.
Moissac Bridge.

form polygons with a horizontal portion over each of the piers. This elevation is the result of omitting vertical members over the piers and it confers on the bridge an effect of great lightness.

The height of the girders is 18.5 m over the piers, diminishing gradually to 10.5 m at the middle of the central span and 8.5 m at the ends over the abutments.

The upper and lower booms of the main girders are connected by a simple lattice in the form of a V with verticals joining into the upper intersections of the lattice.

The cross girders, spaced at 5.4 m to 7 m centres, are connected by four lines of rail bearers of I-section, placed under the rails.

The bracing system consists of cross members in the planes of the compression diagonals, together with a horizontal triangulation, in the shape of a St. Andrew's cross, lying in the plane of the seatings of the lower booms. These cross members, the diagonals and the corresponding cross girders constitute rigid frames which serve to carry wind stresses from the upper part of the bridge to the horizontal lower bracing.

The bearings are of the rocker type fixed on one pier and mounted on rollers on the other pier and on each abutment.

Erection and assembly of the steel superstructure.

Erection on falsework was ruled out by the fact that the nature of the ground on the side of the job made it impossible to drive piles to a sufficient set, and moreover the designers favoured the method of erection by launching, even having regard to the great size of the bridge.

Each half of the superstructure was erected on a suitable platform over the bank and was drawn into its final position by an electric winch, moving on rollers over the erection platform, the abutments and the piers. To relieve the stresses in members of the main girders the latter had to be reinforced temporarily by an additional lattice system. In addition to this the stresses due to the maximum cantilever effect were reduced by the employment of a steel forerunner 40 m long.

As soon as the two halves of the bridge were fully launched they were aligned with one another, an operation which required to be done in such a way that the internal forces arising would correspond to those calculated on the hypothesis of continuity of the spans. To ensure this, the ends over the abutments were lowered by an amount calculated to bring the middle portions of the lower booms into a horizontal position, and the parts thus aligned were then riveted. Finally the decking was lowered onto its supports and the lower booms brought to their proper relative level.

The adoption of this procedure gives an absolute assurance that the stresses in all the bars are the same as if the bridge had been erected on falsework.

In the Moissac bridge the total weight of steelwork is 2714 tonnes. The simplicity of line and pleasing aspect which characterise this bridge are convincing evidence of the aesthetic feeling of its designers.

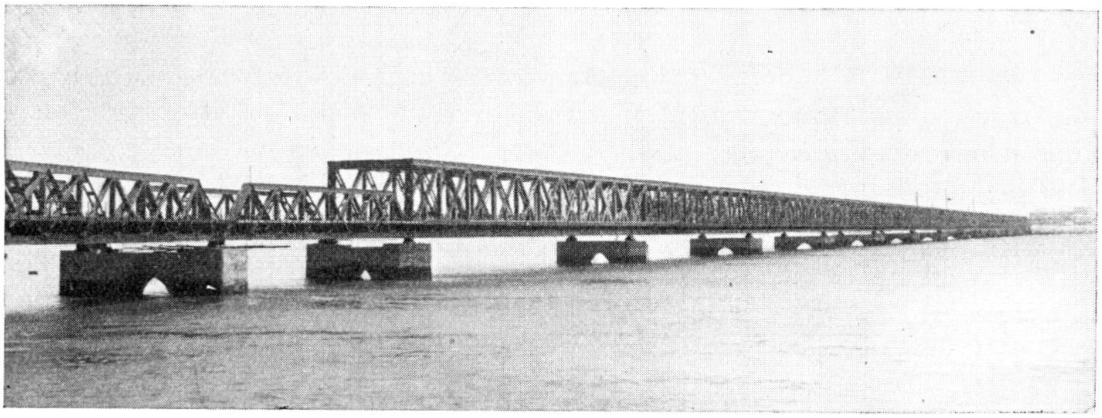


Fig. 7.
Approach Bridge to Mole at La Rochelle-Pallice.

7) La Rochelle-Pallice viaduct (Fig. 7).

The landing jetty now under construction within the port of La Rochelle-Pallice will be connected to the shore by a steel viaduct supported on concrete piers. This viaduct was the first part of the work to be undertaken and is now finished.

Starting from the land end, it includes: 1) two series of six continuous spans each, on a straight line of total length 840 m, and 2) six separate spans, arranged in plan with their longitudinal axes forming a polygon enclosing a circular arc of 192.5 m radius and 280 m development. What follows refers to the continuous spans in a straight line, which have been built by the firm of Daydé.

Each span is of 70 m between centres of supports, and they carry a double carriageway with a standard-gauge railway alongside. The clear width between the main girders is 10 m, and there are two footpaths 1.5 m in width corbelled out from the sides.

The main girders have parallel booms and are 9.5 m high from back to back of angles. The lattice work is in the form of St. Andrew's crosses with the addition of verticals connecting the lower booms to the intersections of the diagonals. The booms are of double construction.

The decking below the roadway is formed of cross girders spaced at 7 m centres and connecteted by seven rows of longitudinals, two of which come immediately below the rails of the railway.

The girders are connected at the top by lattice struts and by horizontal wind bracing. A second system of horizontal wind bracing is arranged in the plane of the bearing plates of the lower booms.

The decking is covered with reinforced concrete. The sidewalks, likewise, each consist of a reinforced concrete slab carried on two lines of longitudinals which in turn rest on steel cantilevers connecting to the vertical members.

The two sets of continuous spans were erected on dry ground at the approaches of the viaduct and were brought to their final position by successive launching operations. The total weight of the 12 straight spans is 5600 tonnes, and at the end of the launching process of each group of six spans the total weight being moved reached nearly 3000 tonnes.

IV. Welded Bridges.

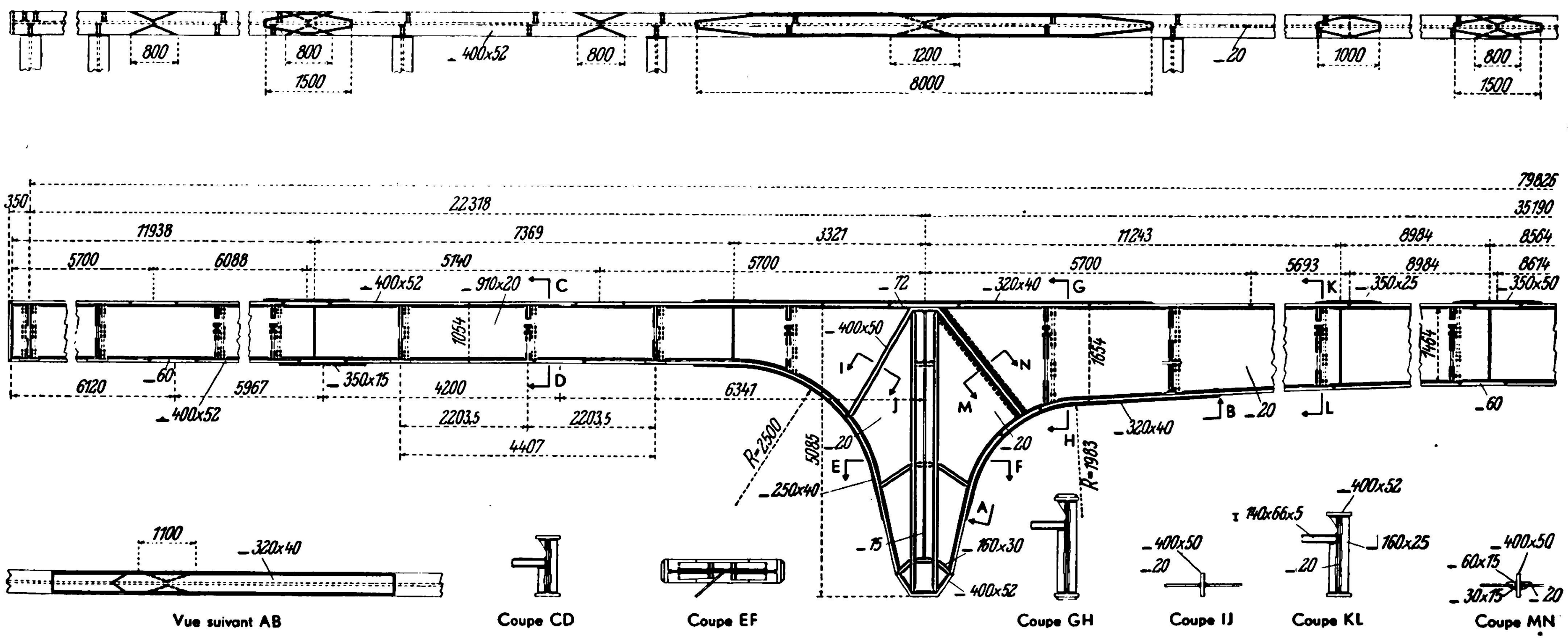
Welded construction has undergone considerable development during the past four years. Structural engineers at first held back, but little by little they have come to appreciate the possibilities and advantages of welding as applied to the field of steel frames and bridges.

From among a great number of steelwork jobs carried out by welding, we have selected the three bridges described below as best exemplifying the degree of strugth and flexibility which can be afforded by this new method of connection.

8) *The "Porte de la Chapelle" Bridge in Paris* (Figs. 8—10).

The works undertaken by the City of Paris with a view to improving traffic conditions at the exit from La Chapelle city gate involved the replacement of existing railway structures by a skew railway bridge at an angle of about 41°.

The new structure has been carried out by the Schwartz-Hautmont bridgeworks in accordance with a general plan drawn up by the Nord Railway company. It comprises two steel floors, practically identical, placed side by side, each borne by two main girders and supported in the middle by columns, the ends of which rest freely on the abutments through the medium of expansion bearings (giving a vertical reaction). The intermediate columns rest on the ground by means of rollers.



Coupe = Section Vue suivant = seen from below.

Fig. 8.

The "Porte de la Chapelle"-Bridge in Paris.

Details.

The bridge has a total length of 79.8 m in three spans, the central one being 35,2 m and the two side spans 22.3 m each.

In each floor the two main girders spaced at 4.05 m centres are connected by cross girders at right angles, and these in turn carry two lines of longitudinals to which the sleepers are fixed by hook bolts. At the ends of the floors the longitudinals are cut short and bear on the abutment through cast steel shoes (Fig. 9).

As the railway is on a curve, the rail bearers under each of the decks are arranged along the sides of a polygon without any interval at the connections between the railbearers and the cross-girders. Footpaths are carried on the

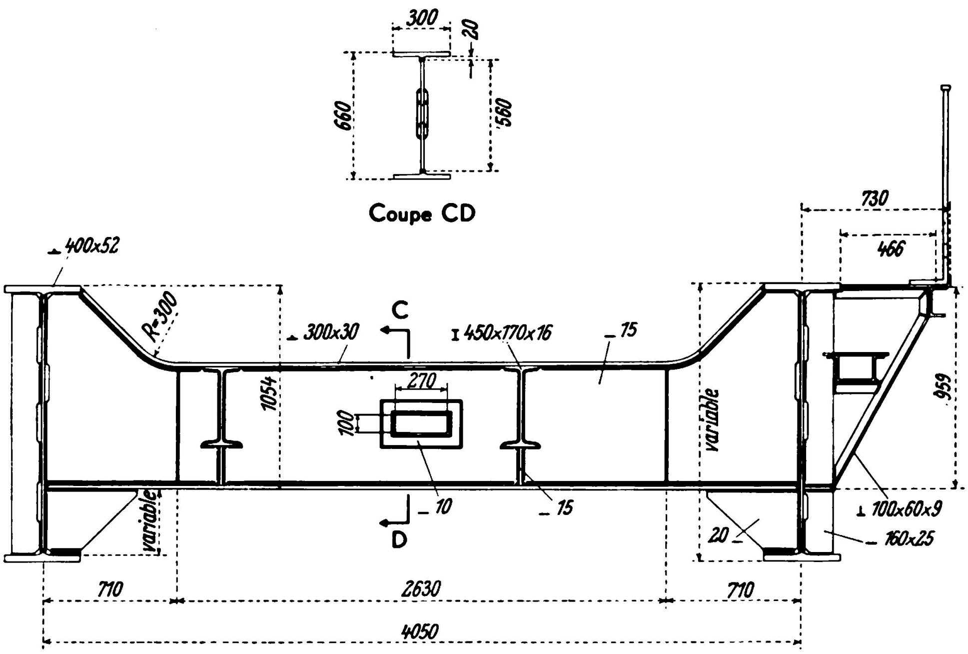


Fig. 9.

Porte de la Chapelle Bridge, Paris.
Section trough decking,

outside main girders by means of cantilevers, the ends of which are connected by joists along the edge. Since it is necessary that the decking should be watertight, this is secured by the use of continuous plates 8 mm thick, which cover the girders, cross-girders, rail-bearers and side joists. A system of piping and guttering is provided to carry the rain water from each span of the bridge into the gutters at the sides of the road underneath.

The intermediate supports have a plate web 20 mm thick, to which the flanges are welded. Each of the latter is a specially rolled section with a central protrusion allowing the use of butt-welded joints. The flanges are 400 mm wide and 52 mm thick, and the lower flange is in each case continuously connected to the wings of the struts. The joints in the web are butt-welded, but some of the joints in the flanges are welded with an X-seam in accordance with the regulations issued by the relevant Ministry in June 1935.

The main girders of the central span are 1.32 m high over the web at the centre, increasing to 1.51 m at the beginning of the strutted portion. The side girders have a uniform height of web of 0.91 m and their webs are provided

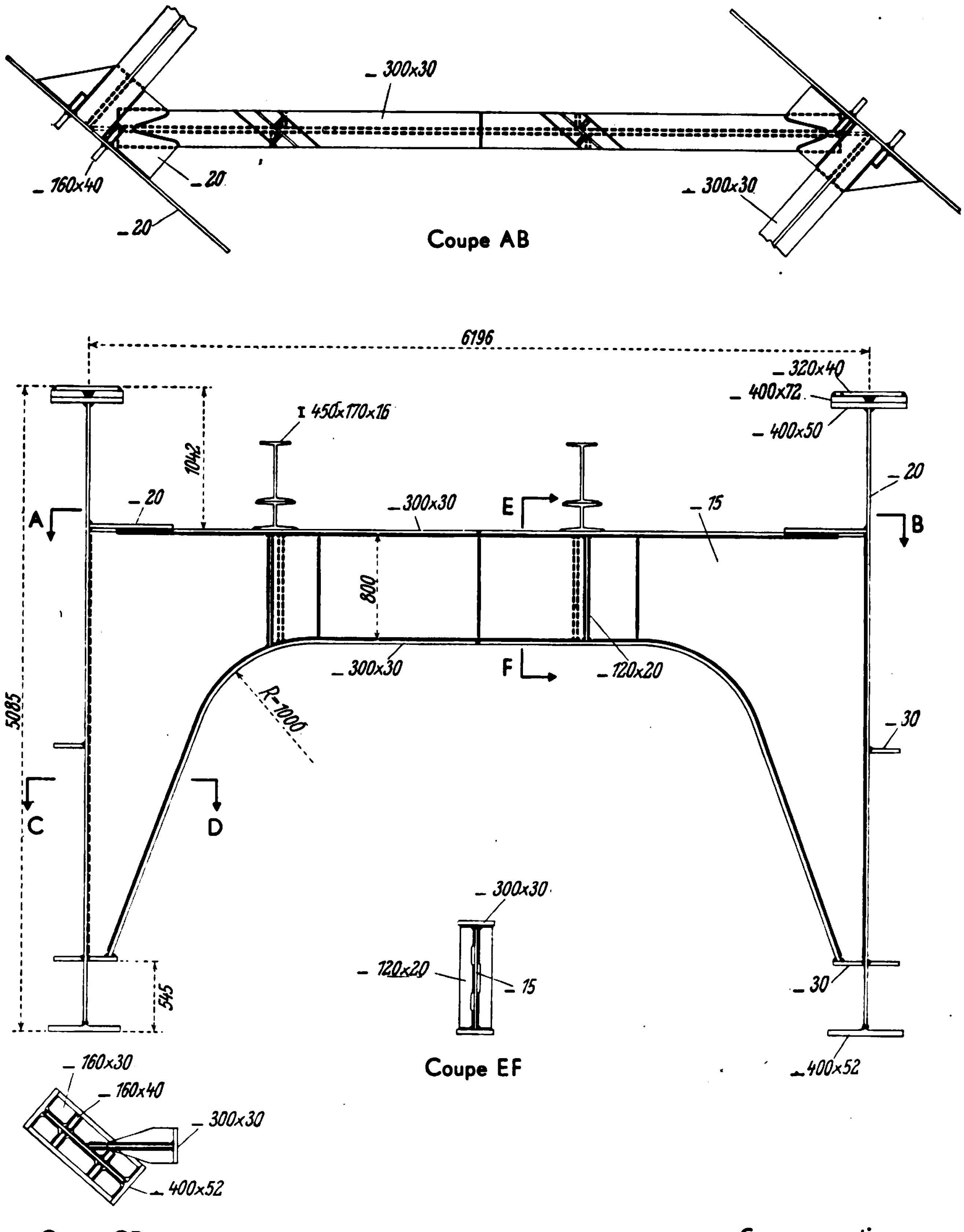


Fig. 10.

Porte de la Chapelle Bridge, Paris.
Wind bracing.

with stiffeners at approximately 2.20 m centres. The latter are welded to the web and to the two flanges on either side, the inside and outside stiffeners being slightly staggered in relation to one another.

The intermediate supports, which are necessarily made continuous with the girders, are formed with a web 20 mm thick, to which the following members are welded: —

1) Connected obliquely at the top, a plate 50 mm thick, to which the web of the horizontal girder is fixed.

2) Connected by a butt joint, the bulbed section which forms the continuation of the lower flange of the horizontal girder.

Stiffeners are suitably arranged to give the necessary rigidity to the crutches, and the ground end is supported on a hinged joint. Each pair of intermediate supports is connected by crossportals to maintain the verticality of the main girders and supply the wind bracing of the bridge. These portals themselves consist of crutch frames having a web plate 15 mm thick cut to a suitable shape and having welded to it at right angles two flanges 0.30 m thick (Fig. 10).

The transverse wind bracing of the bridge is completed by struts formed of webs 15 mm thick butt-welded to bulbed flanges 30 mm thick. The webs of these struts are welded to the inside of the webs of the main girders and form stiffeners for the latter. The wind bracing system is further amplified by an intermediate strut consisting of a No. 14 rolled joist.

The rail bearers placed immediately underneath the rails are formed of standard rolled steel joists No. 45, and are welded to the cross-girders.

Welding was used both in the workshop and on the job, but a few of the secondary members were riveted.

9) Bridge at Soissons (Figs. 11—14).

Leading out of Soissons (Aisne), the Nord Railway Company have constructed a skew railway bridge with two continuous spans and a side footpath carried on cantilevers. The work was entrusted to the firm of Paindavoine

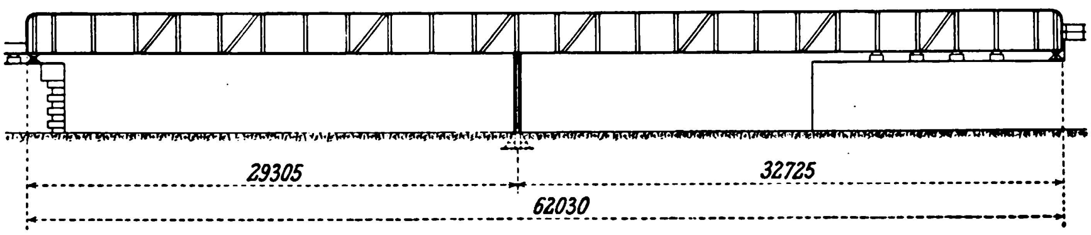


Fig. 11.

Soissons Bridge. General arrangement.

Frères in accordance with a general plan drawn up by the Nord Railway Company.

The bridge is of the straight girder type with solid web, and the main girders are spaced at 4.90 m centres, in each of the two continuous spans of 62.95 m and 62.03 m respectively. Each of the main girders is 2.25 m high and is supported close to its middle point by a stancheon 4.49 m high hinged at top and bottom, bearing at one end on a welded steel seating and at the other end under the girder itself. At either end of the bridge the girders rest on abutments through the medium of hinged expansion bearings.

The web plates of the main girders are 10 and 18 mm thick, and the two flanges are of specially rolled sections having a protrusion, which is welded to the web by butt welds. The flange plates have a uniform width of 400 mm

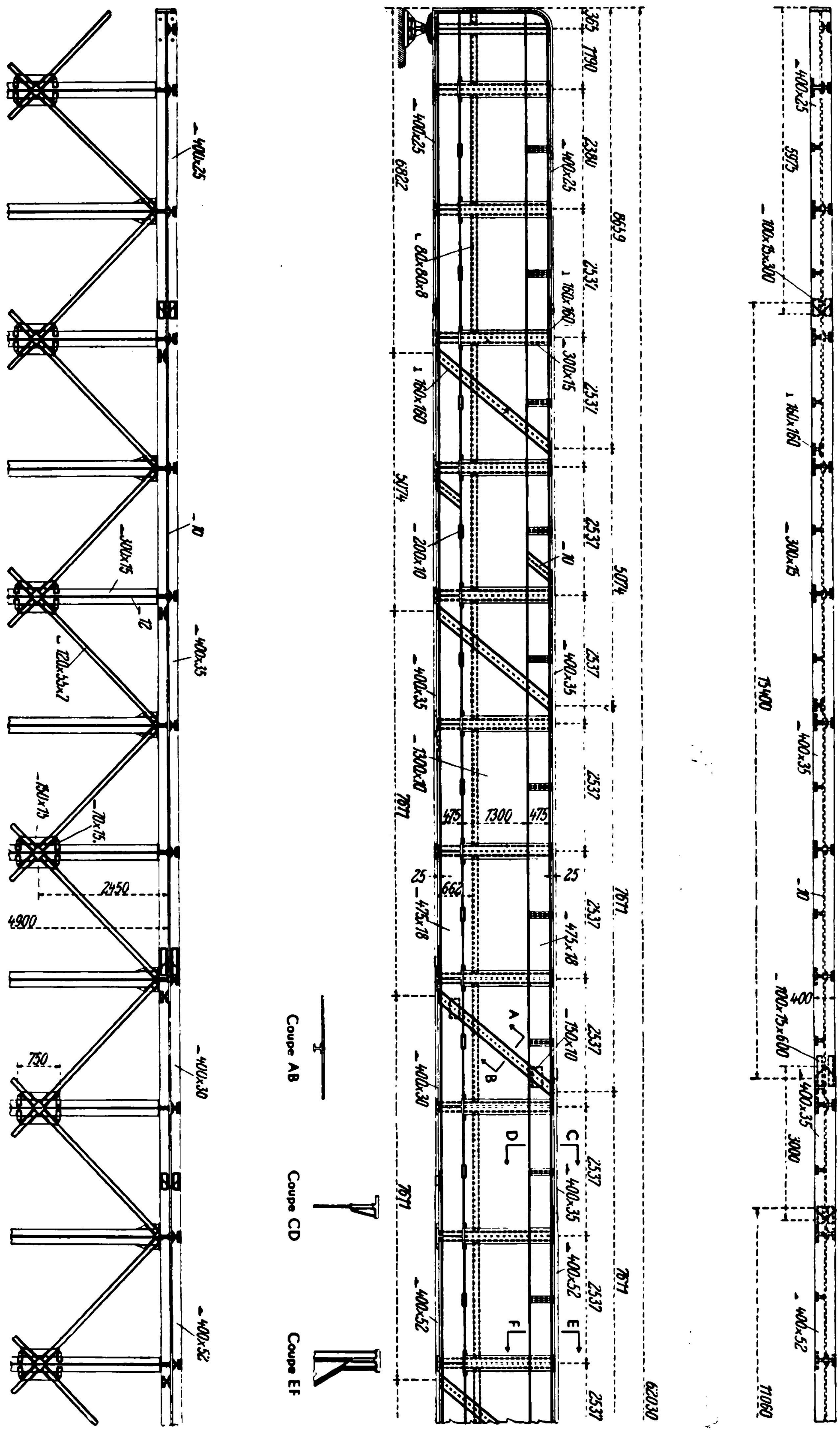


Fig. 12.
Bridge at Soissons.
Details.

and their thickness varies according to the value of the bending moment, being 25 mm, 35 mm, or 52 mm, as requisite. They have butt welded "V" joints under cover plates.

The web is made up of three successive widths of plate vertically superimposed, butt welded along the whole length, the heights of these being respectively 0.475 m, 1.300 m, and 0.475 m. The upper and lower plates are 18 mm thick, and the middle plate 10 mm. The successive lengths of plates are butted against one another on lines inclined to the vertical, the joints in the top and bottom plates being staggered with reference to the joints in the middle plate.

Stiffeners of "T" section are arranged at regular intervals of 2.537 m, welded to the web and to the two outside flanges of the bridge. The longitudinal joints of the web are further reinforced by inclined stiffeners, and on the inside there is cross bracing in the form of an inverted portal which serves to stiffen the main girders. These struts are formed of a web 12 mm thick and 0.45 m high, welded to the two flanges, these consisting of bossed sections 0.30 m wide and 15 mm thick.

The rail bearers immediately underneath the rails are formed of rolled I sections with very wide flanges 30 cm × 30 cm — these being connected to the cross girders by welding (Fig. 13).

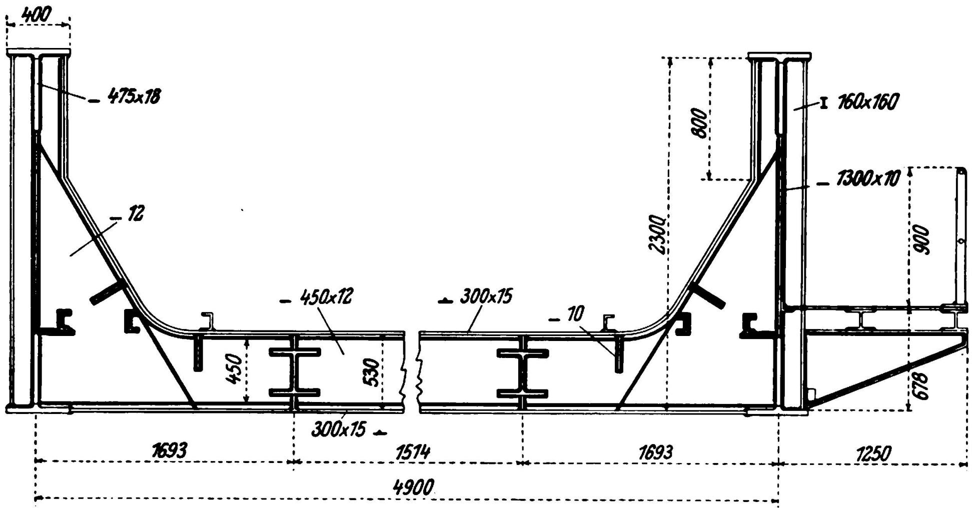
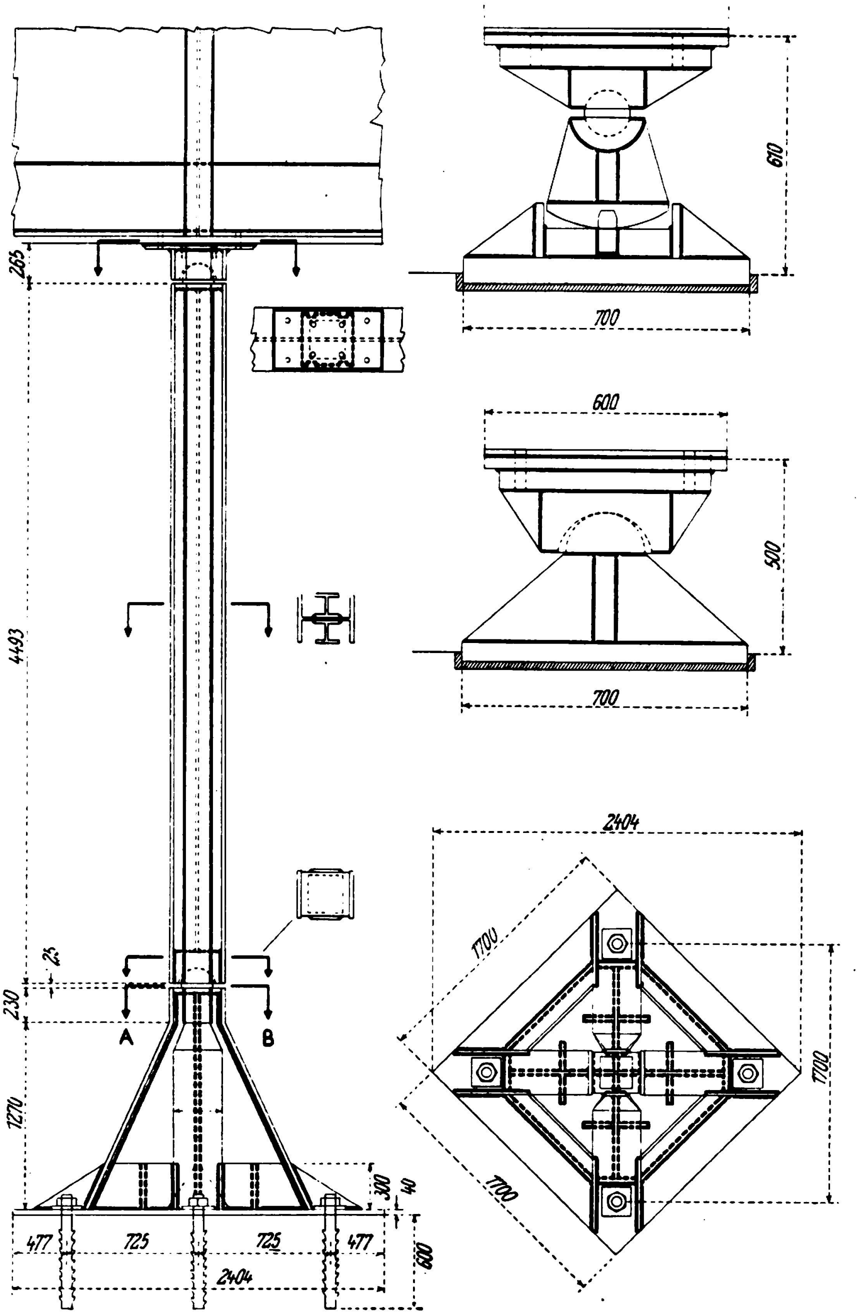


Fig. 13.
Soissons Bridge. Section through decking.

The columns under the middle of the bridge consist of a broad flange I-section 36 cm × 30 cm taken straight from the rolls with two broad flange I sections of 15 cm × 15 cm welded onto either side of its web. The top and bottom ends of the columns are provided with welded connections to carry roller joints (Fig. 14). The base of the column is a welded framework in the shape of a pyramid, formed of plates with suitable cross-pieces and stiffeners. At either end of the bridge each of the main girders rests on a hinged joint. The bearing is fixed at one end and moveable at the other, the moving end being a roller in a suitable track. In each case the work of fabrication was simplified by welding.



Section A B

Vue en plan AB = seen from above.

Fig. 14.

Soissons Bridge, Intermediate hinged columne.

10) The Neuilly Bridge.

The construction of a welded bridge over the two branches of the Seine at Neuilly near Paris was entrusted by the Administration des Ponts et Chaussées to the firms of Baudet, Donon and Roussel.

The Bridge has two arches, one over each branch of the river. One arch is of 67 m span between centres of hinges, and the other of 82 m. The two arches are practically identical in construction and dimensions, and only the one of 82 m span will be described here.

The decking of the bridge is carried on twelve 2-hinged arch ribs of 6.08 m

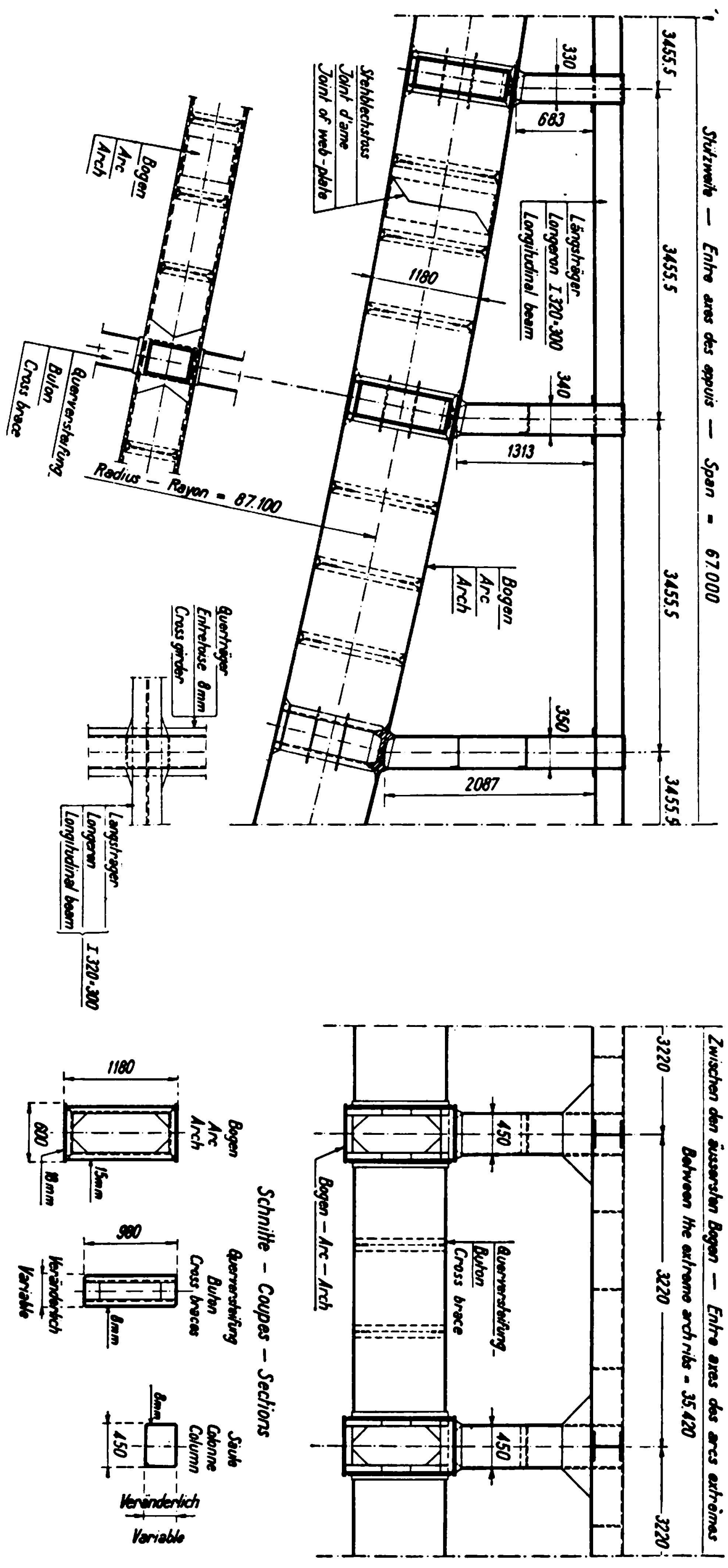


Fig. 15.
Bridge at Neuilly.

rise spaced at 3.22 m centre to centre. Each of these arches is a box section 1.18 m $\times$ 0.60 m formed by welding together four flats at right angles to one another — the box being provided with internal diaphragms formed of welded angles and flats so as to constitute cross frames. These stiffening angles are welded at their outer edges, and are spaced at one metre centres. The joint between each pair of lengths forming the arch ribs is made by butt welding, but the successive pairs of sections meet over an internal sleeve.

The arches are connected with one another by means of box shaped cross girders provided with internal diaphragms in a similar way to the arches themselves. These cross girders are 0.98 m high over the web and are formed by welding together two web plates 8 mm thick and two flange plates 10 mm thick.

The verticals are likewise of box construction, being formed by welding together at right angles four flate 8 mm thick. These are 0.45 m wide in one direction and vary in the other direction between 0.32 and 0.39 m. These verticals are spaced at 3.45 m centre to centre, bearing upon the arches through the medium of special cast steel pieces built into the upper flange of the arch.

The longitudinal girders, which are carried on the heads of the columns, consist of rolled joists 320 mm $\times$ 300 mm. The pressure is transmitted through the bracing which connects the columns together. This bracing consists of a bent plate 8 mm thick which forms a cap to the columns which it braces, and which is stiffened on the inside by flats fixed at right angles.

V. Miscellaneous Works.

11) Access bridge to rail ferry at Dunkirk.

For use in connection with the direct Paris-London passenger service, the Compagnie de Fives Lille have constructed an access bridge connecting the railway with the ferry at the Port of Dunkirk (Fig. 16). The problem of mooring the ferry, and of providing for the passage of sleeping cars from the land onto the ship, was one which involved special difficulties due to the very exacting conditions of operation. Owing to the tides there are considerable variations in level to be taken up; the transverse slope may be accentuated by

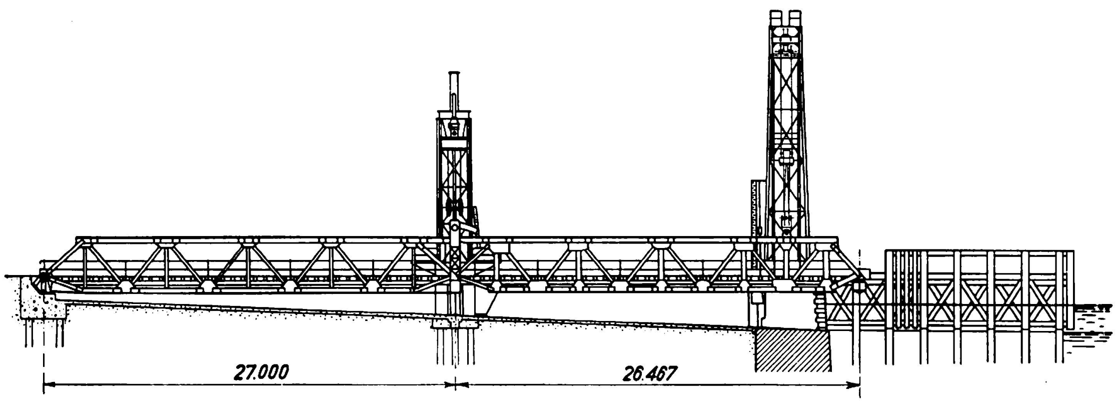


Fig. 16.
Landing Bridge for Ferry-boats, Dunkirk. Elevation.

wind and by the roughness of the water even within the harbour; finally, in view of the very crowded time table, it is essential to ensure both speed and complete certainty of operation, by night as well as by day.

These requirements have been satisfied by constructing a jointed bridge within one of the existing docks. The type is already well known in northern countries, and various improvements have here been introduced. The seaward end of the bridge is provided with a removable extension for the purpose of

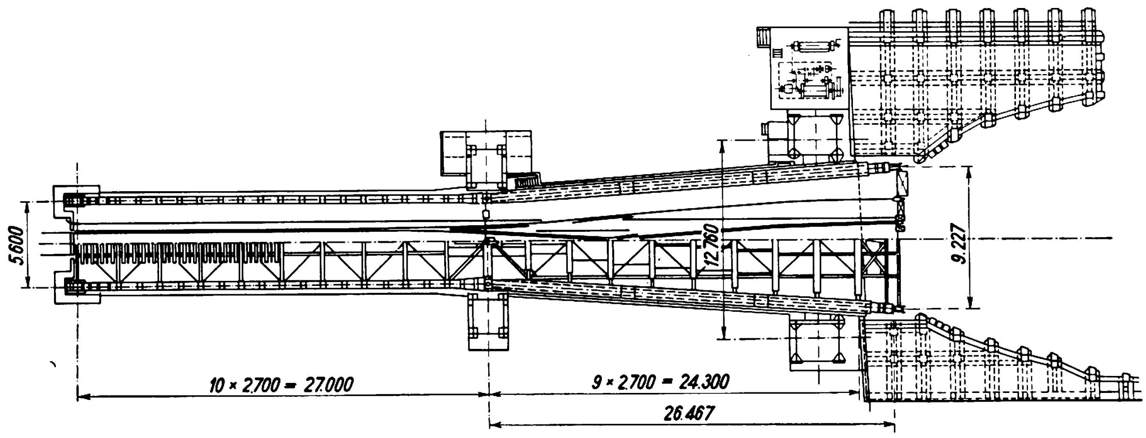


Fig. 17.
Landing Bridge for Ferry-boats, Dunkirk.
Plan.

connecting, if necessary, with the ferry now in use for the carriage of goods traffic between Calais and England, which is of a totally different design from the other.

The main dimensions of the work are, as follows: (Figs. 16, 17, 18.)

Length of the landward span 27 m.

Length of the seaward span 26.467 m.

Total length 53.467 m.

Distance between centres of girders in the landward span 5.600 m. (for one track).

Distance between centres of girders of the seaward span, varying between 5.600 m and 9.227 m (for two tracks).

Range of level of the water surface to be compensated 2.400 m.

This difference in level reaches a maximum of 3.90 m if account be taken also of the variation in the freeboard of the ship in accordance with its loading, and of the oscillations caused by a possible list which it has been assumed may amount to as much as $\pm$ 7°.

Trials carried out with the ferry on 8th September 1935 gave complete satisfaction.

12) Slipway at Lorient (Fig. 19).

For the fishing port at Lorient, in Brittany, the firm of Joseph Paris have constructed a large slipway which allows of trawlers up to 55 m in length and 650 tonnes in weight being withdrawn from the water and hauled up an inclined plane in order to place them in repair docks, of which ten are provided.

The installation consists essentially of the inclined plane, trucks for haulage corresponding in number to the repair docks, a main winch, a moving bridge and, finally, a separate winch for each of the docks.

At the top of the inclined plane, and forming a continuation of it, there is a circular pit 45 m in diameter enclosing a special type of moving bridge which

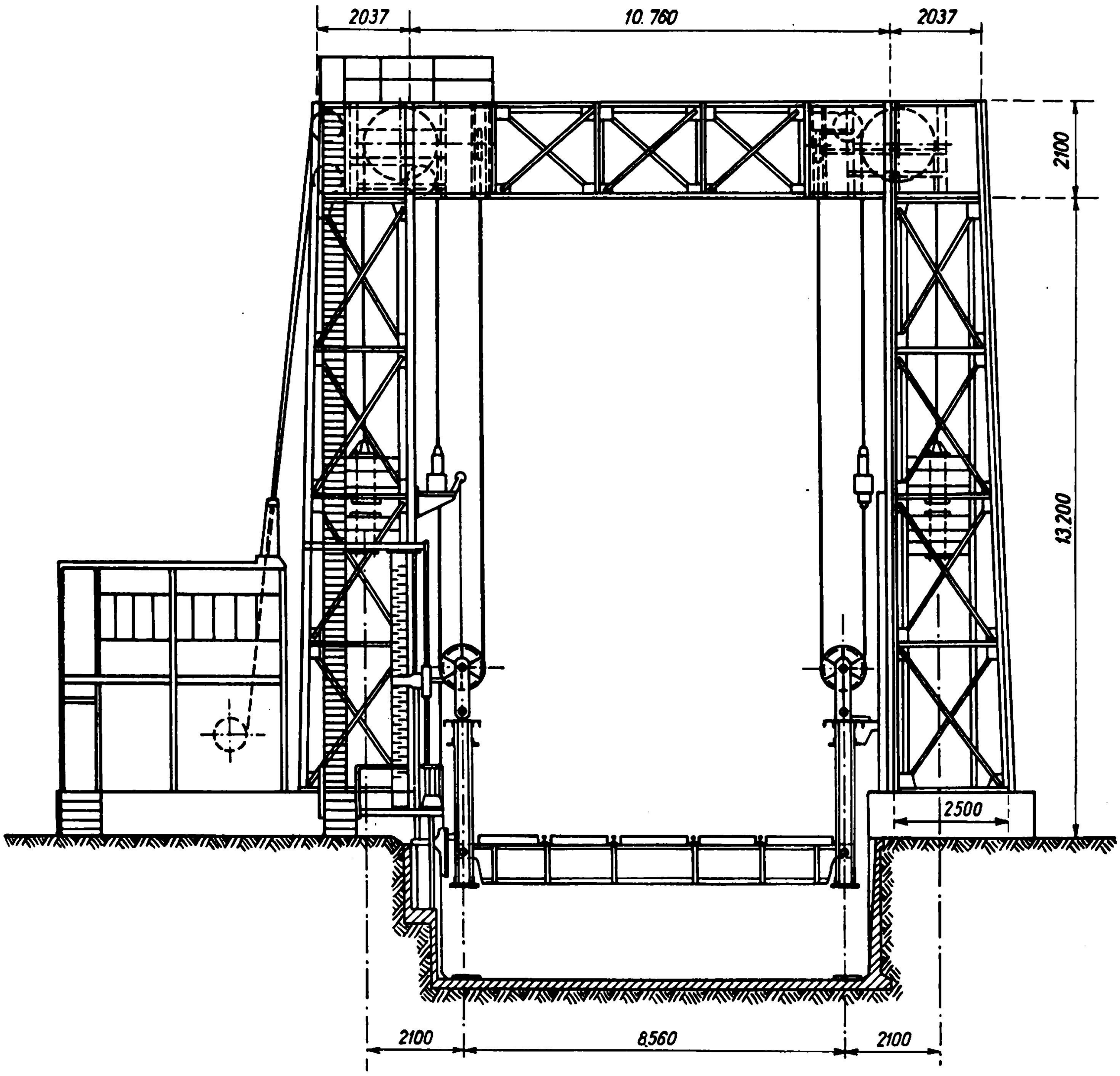


Fig. 18.

Landing Bridge for Ferry-boats, Dunkirk. End portal frames.

can both lift and turn. This bridge has rails to receive the carriage carrying the vessel, and when the latter reaches the top of the inclined plane the deck of the bridge is tilted to a slope of 6.25 % so as to form a continuation of the track. The haulage is continued until the carriage covers the whole of the bridge, which then rocks back into a horizontal position. To provide for this rocking movement the main girders of the bridge rest on the pivot through the medium of suitable roller hinges, the base of which is fixed to the pivoting sub-structure. The rocking movement is imparted by hydraulic pistons which act upon a movable horizontal beam supporting the front end of the bridge;

this beam is guided, as it rises and falls, by lateral slide bars, and it can be locked in the upper position by means of screws at the side which fix it in the proper path for its movement round the circumference of a circle. When so arranged the bridge can pivot in the same way as a locomotive turn-table. The total weight of the bridge unloaded is 450 tonnes.

Ten horizontal bays radiate from the centre of the pit, forming docks in which rails are arranged exactly as on the inclined plans and on the bridge. The sub-

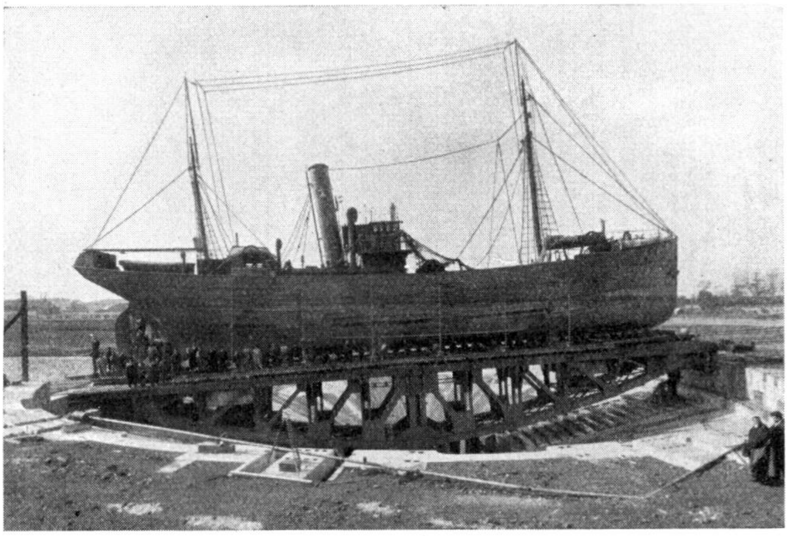


Fig. 19.
Slipway at Lorient.

structure of these bays is of reinforced concrete. None of these docks are on the axis of the inclined plane, but they are arranged symmetrically five on each side.

The bridge can be swung into alignment with any one of the docks, the turning movement being provided by special machinery carried on the bridge itself. This is worked by a 15 H.P. motor which turns a pinion at the lower end of the bridge, the pinion engaging with a rack arranged around the circumference of the rolling track. The pivot consists of a circular crown of framed construction which bears on a set of radially disposed conical rollers and is centred by a pivot rod built firmly into the masonry. When the bridge has been aligned, it is secured and finally adjusted into position by means of screws which engage with suitable seatings on the edge of the pit so as to obtain exact agreement between the levels of the ends of the rails. To remove the boat from the dock the operation of the bridge is carried out in the reverse order.

VI. Pylons.

13) Broadcasting pylons (Figs. 20 and 21).

Two years ago the French postal administration embarked upon a programme of complete renewal of its regional broadcasting stations. The power of these stations, formerly only a few kilowatts, was brought up to 100 KW and in some cases to 200 KW, necessitating the construction of aerials for effecting the radiation at high frequency with very high efficiency.

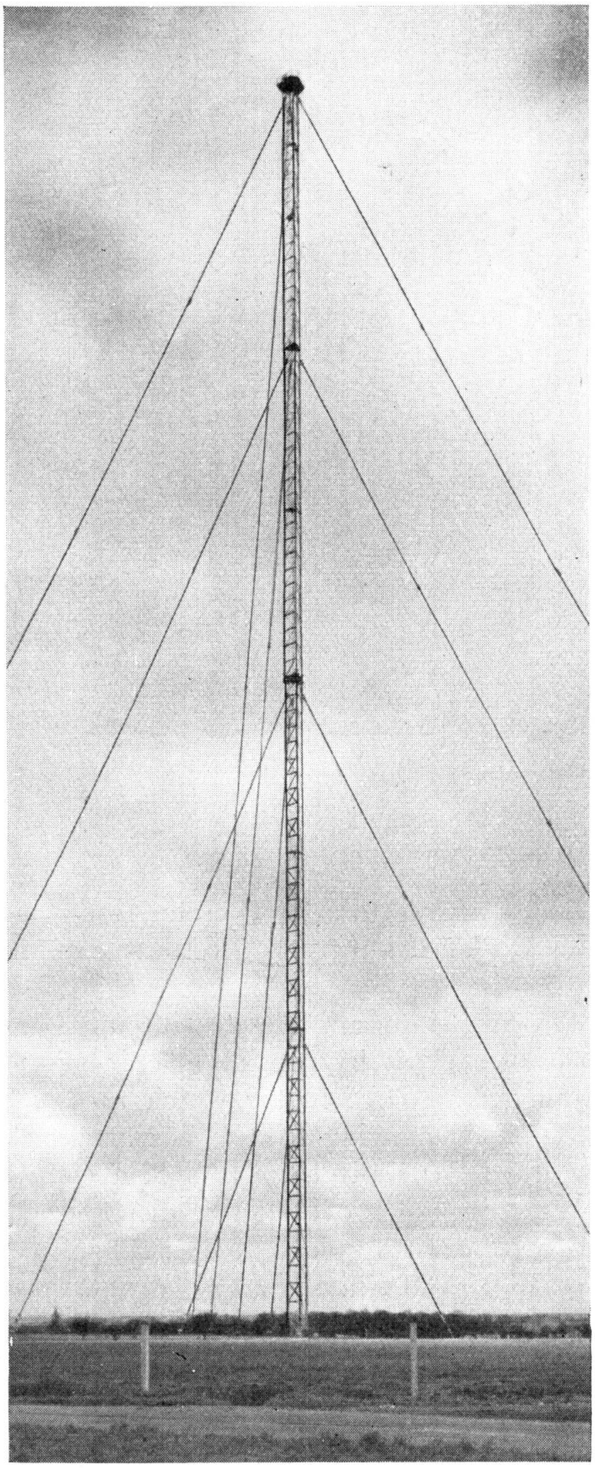


Fig. 20.
Radio Pylon.
General view.

The Administration held a competition for the supply of six pylons to comply with the following specification: —

— Height 220 m.

— Horizontal force to be withstood at the top of the pylons 2 tonnes.

— The pylons to be insulated, both at their base and at the middle of their height, to withstand an alternating difference of potential of 20,000 volts, with a frequency of 10^0 periods, corresponding to a wave-length of 300 m.

The contract was eventually entrusted to the Schwartz-Hautmont works who, before submitting their tender to the Administration, had made a rapid comparison between the tower type and the guyed type of pylon.

It having been found that the guyed solution would be the more economical, the study of this design was carried to its conclusion and a proposal was made

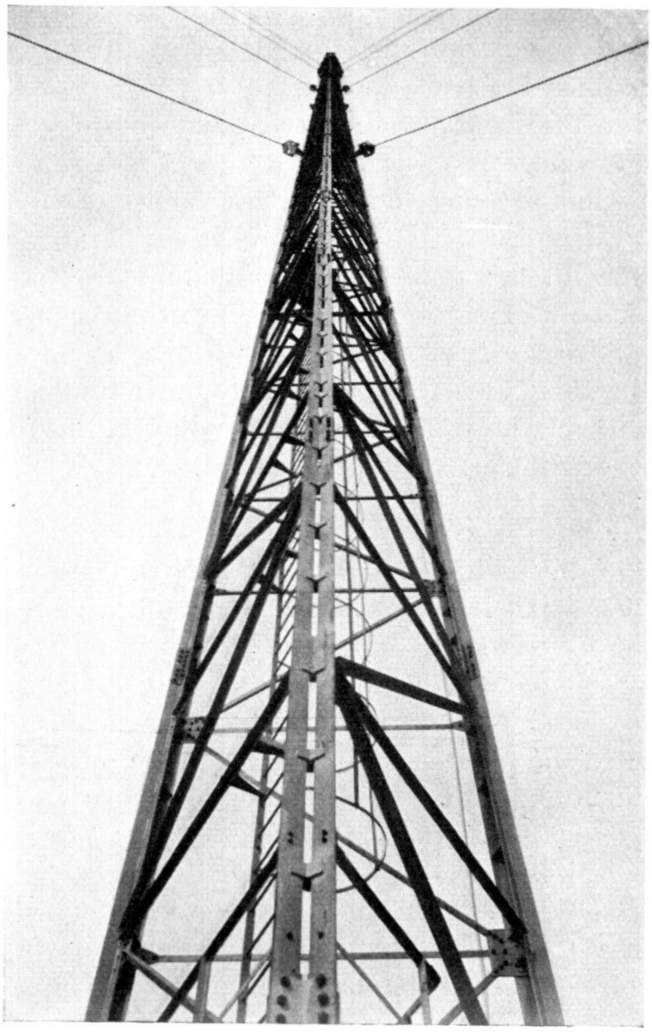


Fig. 21.
Radio Pylon.
Lattice work in detail.

to the Administration that the pylons should be built of either "54" chrome-copper steel, or in Ponts et Chaussées quality 42 kg/mm² steel with 25% elongation. The estimated cost for pylons in steel 54 turned out to be greater than that for steel 42; the difference in weight would be about 10.4%; the works cost of such a pylon would be approximately the same for either quality of steel. Considerations of economy in transport finally balanced the advantages in favour of constructing the pylons in steel 54.

This was done in the contractors' shops at Hautmont (Nord) under the direction of Monsieur Pigeaud, formerly Inspecteur Général des Ponts et Chaussées.

Calculations.

The working stresses fort steel 54, as laid down in the specification of the Ponts et Chaussées were as follows:

R 1 = 18 kg/mm².

R 2 = 19 kg/mm².

The wind pressure assumed in the calculations was 200 kg/m². The forces in the pylons resulting from wind pressure were determined on the assumption that the pylon formed a continuous beam on five supports. It was assumed firstly that the wind was blowing at an angle of 90° to one side of the pylon, secondly that the wind was blowing on two faces of the pylon in a direction bisecting the angle between the two exposed faces.

The calculations for the cables were carried out in three successive stages.

1) The pylon without wind.
2) The pylon under wind pressure, but with the points of support assumed to be locked.
3) As in stage 2, but with the connections maintaining the fixity of the centre of gravity of each pair of sections assumed to be successively released, and the pylon assumed free to turn on its lower pivot.

For the purpose of this calculation, it was found advantageous to take the co-efficient of elasticity of the cable as corresponding to its first position (see Pigeaud: "Résistance des matériaux et élasticité").

Construction (Fig. 22).

The cross section of the pylon is triangular with a constant length of side of 2.80 m. The foot is carried on a spherical pivot of cast-steel, and the pylon is maintained upright by a system of 12 guy-ropes which takes the form of three bundles of cables separated by an angle of 120°. The inclination of the guy-ropes to the horizontal is 63°. The three corner members of each pylon are formed from rolled angles, the scantling of which varies according to the stresses from 120 × 120 mm to 80 × 80 mm. To ensure rigidity and good resistance to buckling on these corner members, they are braced by diaphragms and bent plates attached by electrically deposited fillet welds. The guy-ropes consist of cables from 27 to 32 mm diameter. These cables are of the "Mines" type — the 32 mm cable being composed of 52 wires each of 3.52 mm diameter of galvanised and bituminised steel. The breaking strength of the steel used for the cables is between 180 and 200 kg/mm².

The connections between the guy-ropes and the pylon have been arranged in such a way as to approximate as closely as possible to the conditions assumed in the calculations. With this object, a platform is constructed at the level where the cables connect with the pylon, consisting of beams framing the equilateral triangle which forms the section at that level. The cables are attached at an axis 220 mm away from the vertical axis of the pylon, an arrangement which serves considerably to diminish the bending moments which may result from variations in the tensions as between the guys attached to any one crown, and also to ensure that the vertical components of the loads in the cables of a given crown are distributed almost uniformly, owing to this arrangement on three corners of the pylon.

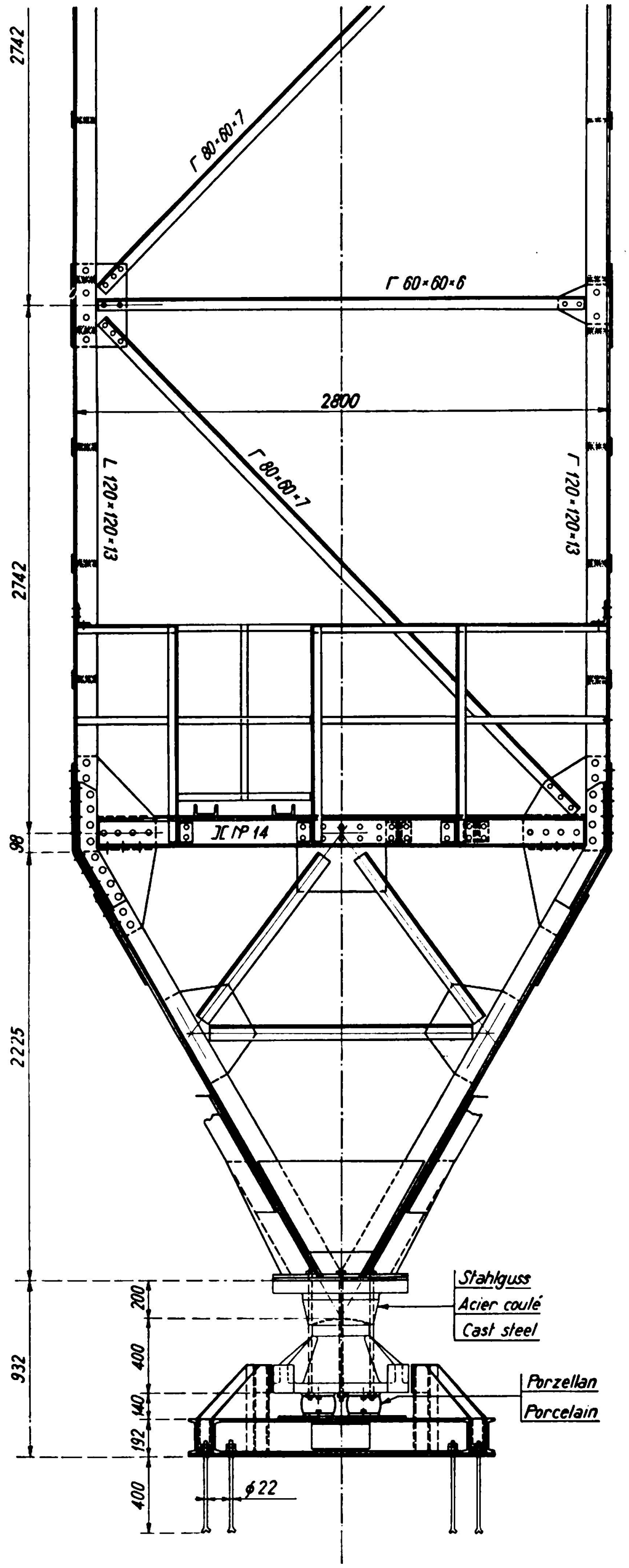


Fig. 22.
Radio Pylon.
Base of mast.

The insulation at the base has been carried out by means of a hoop of porcelain 90 mm high as illustrated in Fig. 22. Each of these porcelain members can carry a maximum load of 90 tonnes, and their dimensions have been calculated with a factor of safety of three. At the level of 110 m, in order to obtain the insulation by using porcelain as the insulating material, special measures have had to be taken in view of the fact that porcelain will withstand only compressive stresses. Some idea of the resulting complication of the framework may be gathered from Fig. 25.

At every 50 m the guys are interrupted by an insulating device, wherein the tension of the cables is transformed into compression exerted on blocks of porcelain, this being the only form of stress which the latter material will withstand.

Finally, to complete the electrical insulation, the ladder which gives access to the upper platform of the pylon itself rests on blocks of porcelain, and is divided into sections joined by blocks of porcelain.

Erection:

After trial assembly in the shops, the pylons were delivered in sections and these were erected one over the other with the aid of a tubular mast 10 m high, fixed at the centre of the pylon by means of cables suspending it from the three sides of the latter. The mast was moved from one level to the next within the pylon.

The sections were lifted into place with the aid of a mechanical winch. In the course of the erecting operations the pylon was maintained vertical by temporary cables, the tension of which was controlled by dynamometers so as to avoid the production of oblique stresses in the blocks of porcelain (as will be discussed below). At as early a stage as possible the guy ropes were added and were subjected to an initial tension, calculated with a view to reducing the oscillations of the pylon in the wind. This tension was checked by dynamometers of up to 30 tonnes capacity.

Foundations:

The foundation under the pivot is formed of a heavily reinforced concrete slab.

The tension of 30 tonnes in each of the guy ropes is absorbed by an anchorage of the "Malône" type, which consists of a pile inclined in the direction of the cable and having a length proportional to the tension in the latter and to the resistance of the ground; at the foot of the pile a spherical chamber is hollowed out, and this, like the pile itself, is filled with concrete; steel bars are embedded in the concrete and the cable is attached to these. With this arrangement the volume of earth intervening between the spherical chamber and the surface of the ground acts as an anchor weight.

VII. Dams.

14) The Chatou Dam (Seine):

A composition was held by the Administration des Ponts et Chaussées for the reconstruction of a dam across one of the branches of the Seine at Chatou, below Paris. The Moisant-Laurent-Savey works, who were awarded the contract, have completed a modern form of installation which is of special interest in view of the various novel features and improvements incorporated.

The work includes three weirs of similar design, each with a clear width of 30.50 m wide between the faces of the piers and abutments, and as the piers are each 4.50 m wide, it follows that the total width of the dam between the neat faces of the two abutments is 100.50 m. The span of the gates between supports is 32.40 m.

The crest of the three weirs is at a Reduced Level 15.50 m. Each of the weirs is closed by an arrangement consisting of a lower and upper gate which can be moved vertically by machinery. The upper gate is provided with a device which allows it to be lowered in stages in the event of a slight flood, but for a large flood the whole of both gates is removed.

The gates, when completely lifted, leave the weirs unobstructed up to R. L. 31.00 m so as to permit navigation, for which provision is normally made in time of flood, up to level 25.00 m.

A high level service bridge is provided on the upstream side along the whole length of the structure. This carries the machinery for operating the gates and also serves as one of the tracks for a double travelling gantry to which reference will be made below.

To allow passage on the upstream side of the gates when at full capacity, three moveable gangways resting on the piers and abutments are provided at a height of 0.80 m above the water level of 23.22 m. These gangways, which rise and fall automatically with the lower gates, are completely free of the weirs when the gates are lifted. The gangways were originally designed in ordinary steel but have actually been constructed in high tensile steel, as a trial of this new material.

On the upstream side of the gates vertical grooves have been formed in the piers and abutments for the attachment, if and when necessary, of a mobile metal cofferdam made in a single piece, forming a temporary substitute for the gates of any given opening in case of damage and allowing their removal and repair.

The cofferdam can be brought into position by means of a double travelling gantry which spans between the service bridge on the down stream side and a track formed on a reinforced concrete box girder on the upstream side. On the left bank these tracks are continued across an additional span reaching the island of Chatou, where the cofferdam is normally kept when not in use. The gantry can likewise be used in case of repair work to pick up any of the gates or gangways and deposit them on dry ground on the island.

Downstream of each weir, a cavity has been formed to receive a cofferdam composed of four pieces which are normally stored on the island of Chatou: this, again, can be moved about by means of the gantry. This downstream cofferdam, erected opposite any one of the openings conjointly with the upstream cofferdam, would enable the space between them to be pumped dry for effecting repairs to the gates, the grooves, or the crest of the weir. The necessary operations can be carried out either by electric winches or by hydraulic jacks.

The downstream service bridge consists of a gallery which extends over the whole length of the works, consisting of three weirs and one span over the island. On the right hand side of the piers and abutments, this gallery is built higher, forming cabins which contain the hydraulic jacks. The roof and upstream side of each of these cabins can be completely removed when it is required to erect or dismantle any apparatus by means of the travelling gantry.

The upstream bridge consists of a tubular girder which carries a track of 1.28 m gauge for the bogies of the travelling gantry. The downstream and upstream bridges are connected at their two ends by horizontal struts which, with their seatings, form cross-frames.

Between the two bridges the piers and abutments are splayed, down to level 30.50 m, to allow the passage of the gates and cofferdam when these are being transported by the gantry.

The span over the island where the gantry is normally stored has a clear opening of 35 m, allowing the upstream cofferdam or the gates to be easily stowed away at ground level between the piers, and the downstream cofferdam in four pieces on suitable supports on the downstream side.

The gates are of the Stoney type, made of steel, and the weight of the upper gate is 100 tonnes.

The end of each gate is bolted to a cast steel rolling frame so arranged as to allow the horizontal bending of the gate under the pressure of the water. This frame presses against a series of rollers which transmit the pressure onto a cast steel track built into the grooved masonry. The set of rollers is suspended from a pulley block which is carried between two passes of steel cable, one end fixed into the masonry and the other attached to the head piece of the gate. This is the characteristic arrangement of the Stoney gate, whereby friction is reduced to a minimum, being purely rolling friction.

Since the suspended rollers move at a speed half that of the gate, a point is reached in the upper part of the travel of the gate when the latter ceases to be supported by these rollers; at this point it is picked up on fixed rollers built into the masonry. On the upstream side, guidance is afforded by means of fixed rollers at either end of the frame, rolling on a vertical track formed of I section, which is held in position by horizontal attachments built into the concrete. To allow of withdrawing the gates for repair, the I-shaped section on the upstream side includes two portions which can be removed to leave a free passage for the rollers projecting from the gate when the latter is suspended from the gantry and is being moved sideways.

The lower gate is hung at each end from a double chain of steel with flat links, the two runs of this chain being connected to it through an arm linked with a block forming part of the frame of the gate. The two parts of the chain are connected above to a hydraulic jack.

The upper gate is suspended from a rigid tie-rod attached to the top of the frame but free to hinge within the latter when the gate has a movement imparted to it by the other gate. This rigid tie-bar terminates above in a Galle chain tackle, one end of the chain through the latter being attached underneath the upper service bridge and one end coiled on a reel which forms part of the mechanical winch.

The lifting mechanism includes two separate arrangements, one for lifting of the upper gate by itself and the other for lifting the lower gate or both gates together. The mechanism for the upper gate consists of an electric motor of

30 H. P. which operates two winches of 100 tonnes capacity each, through a cross shaft. The rate of lifting is 0.20 m per minute. In the case of the lower gate, the lifting arrangement consists of two hydraulic jacks of 175 tonnes capacity.

VIII. Aircraft Hangars.

15) Hangars at Bordeaux-Teynac (Figs. 23 and 24).

The steel hangars, in the aerodromes of Bordeaux-Teynac and Lanvéoc-Poulmic constructed by the firm of Daydé, form part of a servies of 27 hangars to be built on a patented system jointly by this firm and be the Forges et Ateliers de Construction Electrique de Jeumont. They have a free opening of 70 m with a depth of 66 m at Bordeaux-Teynac and 55 m at Lanvéoc-Poulmic. The clear height inside is 10 m. The framework is carried on concrete footings which extend to 1.50 m above ground level.

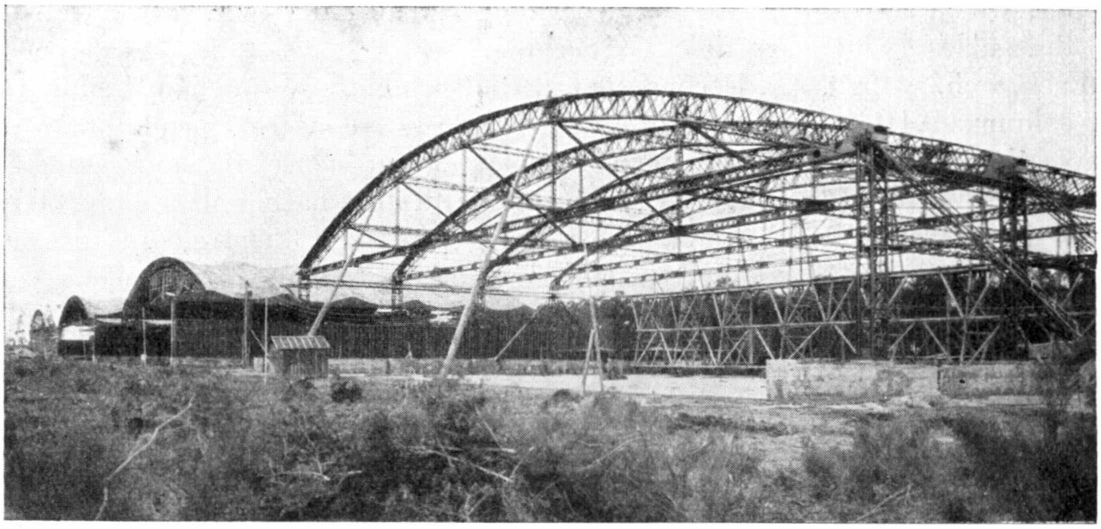


Fig. 23.
Aircraft Hangars at Bordeaux-Teynac. Frame work.

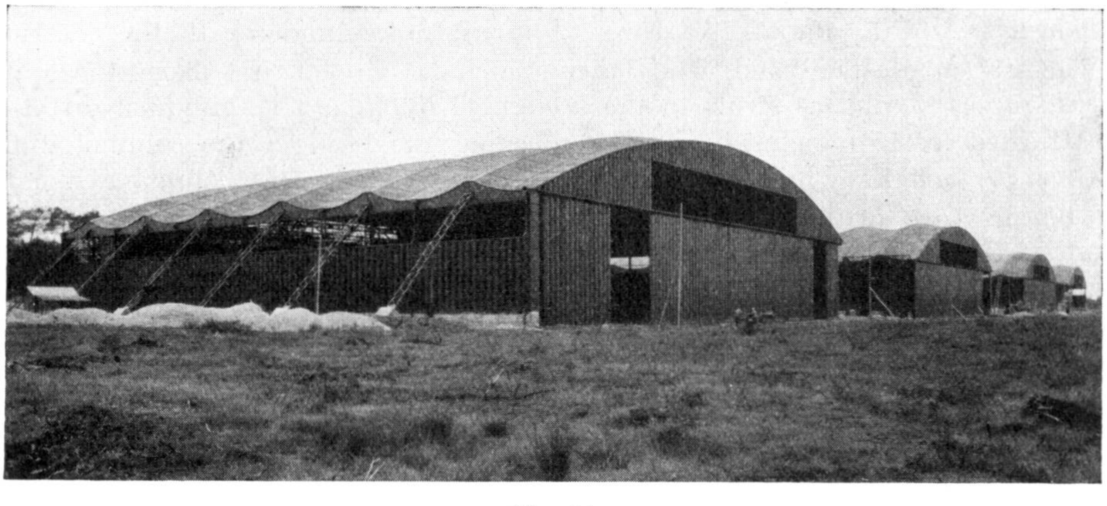


Fig. 24.
Aircraft Hangars at Bordeaux-Toynac. View of completed structures.

In the case of the 66 m hangar the framework consists of six intermediate arches relieved by ties of 71 m span spaced at 11 m centres. These are carried on vertical piers. There are also two end frames of similar span, with struts and bracing, a roof covering of sheet metal, filling and covering pieces for the two long sides and the closed end, and sliding doors for closing the open end.

The construction of the 55 m hangar is the same, with the exception that one of the intermediate arches and one span of 11 m are omitted.

The arches each consist of a lattice girder built with its axis in the shape of a circular arc, and having a rectangular cross section 1.50 m high by 0.80 m wide. The rise, measured at the axis, is 7.60 m. The ties are box girders, 0.50 m by 0.58 m, enclosed by lattice work on the vertical sides and by full plates on the horizontal faces. Each tie is connected to the corresponding arch by five latticed suspenders, and the ties are braced by five sets of box girders which serve to resist laterial buckling and to unify the two wind girders of the gables (which are described below).

Each arch, with its tie, is connected at the springings by pin joints supported on columns 9.10 m high, one of these serving as an inverted pendulum which leaves the arch and tie free to expand under the influence of temperature and loading, while the other is fixed by an inclined strut having its footing 10 m away from that of the column, either to the left or to the right of the shed.

The covering, which is the especially original feature of these hangars, consists of a self-supporting roof formed from sheeting 1.4 mm thick erected in pieces 2.50 m by 10.20 m welded together. Each such sheet is hung from its ends between two successive arches and is given the shape of a hyperbola by stiffening members 0.160 m high. Thus the roof is made up of a succession of sheet metal "tents" in the shape of hyperboloidal segments having a common horizontal axis parallel to the long sides of the shed.

In the case of the intermediate arches the tensions exerted by the strips of sheeting are balanced as between those on either side. At the two ends of the shed the sheets are terminated at the circles which fit the corresponding hyperboloids, and the tensions imposed by the sheets across the end arches are balanced by the action of five rows of thrust pieces underneath the covering.

The system of struts and wind bracing includes, in the first place, the upper thrust pieces serving as struts to the arches. Stability in a longitudinal direction is afforded by two inclined braces connecting the head of the column at the middle of each long bay with the feet of the two adjacent columns.

The pressure of the wind against the sheathing at the closed end of the shed, or against the sliding door, is transmitted to these bracings and to the middle columns through the medium of two horizontal triangulated wind trusses which are arranged against the long sides of the shed in the plane of the bracings of the ties of the arches, and through the medium of two wind girders placed against the gable ends.

The sheathing of the long sides consists of rolled sheets 1.4 mm thick, arranged in courses 2.40 m high. These are carried on small walls of reinforced concrete 1.50 m high and terminate at the top in a glazed strip 2.70 m high.

The sheathing at the closed end is formed in similar fashion to the long sides, with glazing over 35.50 m length of the shell.

The open end can be closed by a sliding door made up of 15 panels 10 m high. These travel on rollers in tracks built into the foundation at the bottom, and are guided by tracks attached to the framework of the hangar at the top.

These hangars have been constructed entirely in high-tensile steel. Their erection was carried out on moving scaffolds with the aid of a pivoting 3 tonnes electric crane of 9.50 m reach, the crane itself running along the top of the scaffold. The erection of a hangar of 540 tonnes weight was completed in 24 days.

16) Another type of aircraft hangar (Fig. 25).

The firm of Delattre et Frouard have perfected a new system for the construction of self-supporting roofs using thin sheet without any framework, and three hangars designed in accordance with these new methods have been ordered by the (French) Air Ministry.

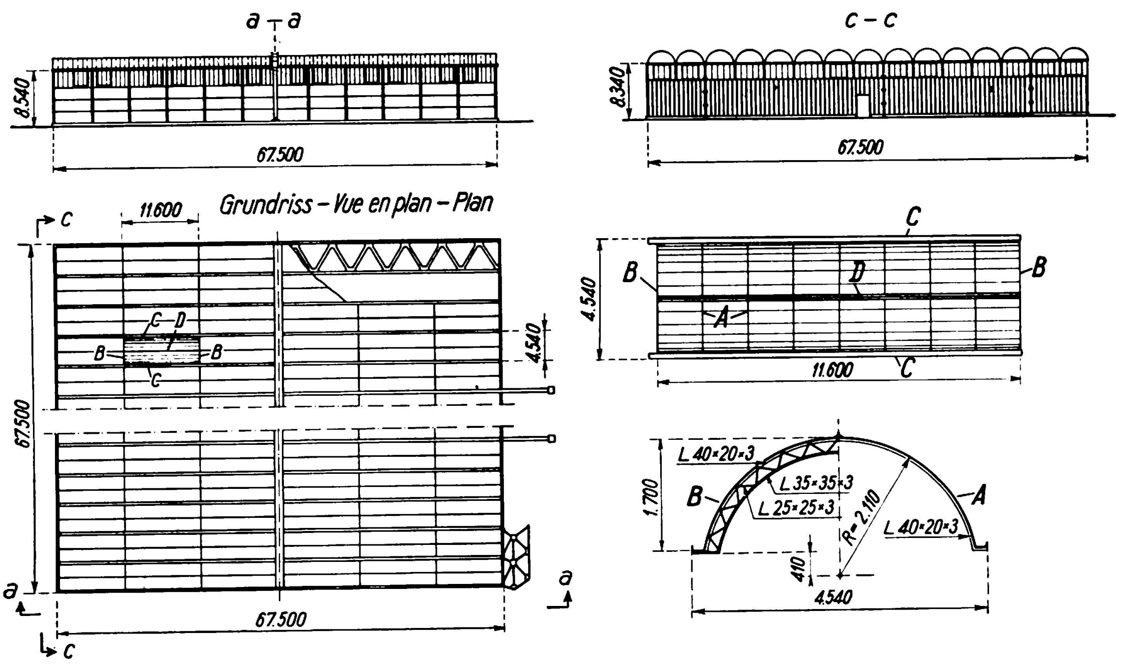


Fig. 25.

Aircraft Hangars built by Messrs. Delattre et Frouard.

General characteristics: —

The useful area (internal measurements) is 67.50 m square, equal to 4556 m². The useful height is 8.50 m. The hangars are capable of being taken completely to pieces. Apart from the joints arranged for this purpose, which are bolted, the whole of the remaining construction is assembled by electrically welded seams.

a) Roofing —

This consists of 15 identical elements which act as girders continuous over three supports. Their intermediate support is afforded by a central girder, one end of which rests on an internal column situated 22.50 m from the front face.

and the other end on a column in the back face of the hangar. The outer supports are columns situated in the long sides of the latter, these columns being standard 220 mm I-sections. The roofing element is a half-tube of 3 mm sheet bent to a radius of 2.10 m and having welded to it two side channels of 3 and 4 mm sheet. The "half"-tube is not a complete semicircle, but its central angle is 150^0 and its rise is 1.70 m. The side channels are 20 cm wide and the total width of the arch including these channels is 4.54 m.

Each separable section, approximately 11 m long, consists of eight sheets butt-welded to one another, and two such sections joined along their common upper generatrix form a complete roofing element with channels. Six of these elements constitute a complete girder over three supports. Stiffeners formed of $40 \times 20 \times 3$ mm angles bent to the required shape are welded to the sheeting at approximately 75 cm intervals; every fourth stiffener is replaced by a curved lattice frame, likewise welded to the sheet, consisting of an upper flange of $40 \times 20 \times 3$ mm angle, a lower flange of $35 \times 35 \times 3$ mm angle, and lattice work at 45^0 of $25 \times 25 \times 3$ mm angle. The front and back facing members include box girders in the plane of the channels; these are of triangular lattice construction around the half-tube, which is thus made to act as a wind girder.

b) Central girder.

The central girder consists of two plate web girders 2.10 m high and 80 cm apart. The webs of these, 5 mm thick, are stiffened by $60 \times 40 \times 5$ mm and $60 \times 60 \times 6$ mm angles spaced 30 to 50 cm apart. The booms are made from half I-sections with very wide flanges 280×280 mm reinforced by plates up to 260×24 mm at the point of maximum bending moment.

The twin girders are strutted in the first place by the arches themselves which are continuous through the webs of the girders (portions oft them being welded to either side of the web plate) and in the second place by latticed trimmers spaced at an average distance of about 4 m.

The front column is of V-section, being formed of two box girders, with a pin joint at the foot. The rear column is also of box construction and is pin-jointed in both directions at the top and at the foot.

c) Long sides.

These are built in corrugated sheeting 0.8 mm thick attached to longitudinal members (80 mm standard I-beams) resting on the columns. The uppermost 2.50 m of the sides is glazed and one-third of the glazed area is on opening sashes. The sides are borne by dwarf walls of reinforced concrete 50 cm high.

d) General stability.

This is ensured by side bracings at an inclination of 7/8, and by the central girder as regards stability in depth.

e) Doors.

These consist of panels on ground rollers which run into a suitable place at the side. The panels are formed from pressed plates.

f) Total weigtht, without the sliding doors, is 283,000 kg or 62 kg/m^2.

g) Materials used.

The whole of the hangar is constructed from sheets and rolled sections of "steel 54" having an elastic limit of 36 kg/mm^2 and a breaking strength of 54 kg/mm^2. Only the sheathing, 0.8 mm thick, is of mild steel sheet.

Summary.

One of the most pronounced characteristic in the activity of French steel designers lies in their research work and development of improved and modern construction methods with the object of reducing prime costs. The present article gives a number of descriptions of recent constructions illustrating the advance made in various fields of structural engineering in steel. The author's report is divided into 7 sections, giving a description of 15 structures in steel.

1) *Structural steel work.*
   Shell Building in Paris, Medical Faculty in Lille, Rex Cinema in Paris, New Citroën works in Paris.
2) *Riveted Bridges.*
   Bascule Bridge in Dunkirk, Moissac Bridge, Viaduct La Rochelle-Pallice.
3) *Welded Bridges.*
   Bridge near Porte de la Chapelle in Paris, Bridge in Soissons, Bridge in Neuilly.
4) *Various Structures.*
   Railway-ferry-bridge in Dunkirk, Slipway at Lorient.
5) *Pylons.*
   Broadcasting tower of P.T.T.
6) *Weirs.*
   Weir at Chauton (Seine).
7) *Airplane Hangars.*
   Hangar at Bordeaux-Teynac, Hangars with self-supporting steel plating.

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Some Recent Steel Bridges in Sweden.

Neuere Stahlbrückenbauten in Schweden.

Quelques nouveaux ponts métalliques en Suède.

Major E. J. Nilsson,
Hafenverwaltung der Stadt Stockholm.

A number of comparatively large steel bridges, both road and railway, have been erected in Stockholm during the last few years. Although these bridges cannot compare in size with the very big bridges abroad, they are nevertheless noteworthy as regards design, achitectural features, local conditions, and method of erection. The present notes briefly describe just a few of the road bridges recently completed or at present building in Stockholm.

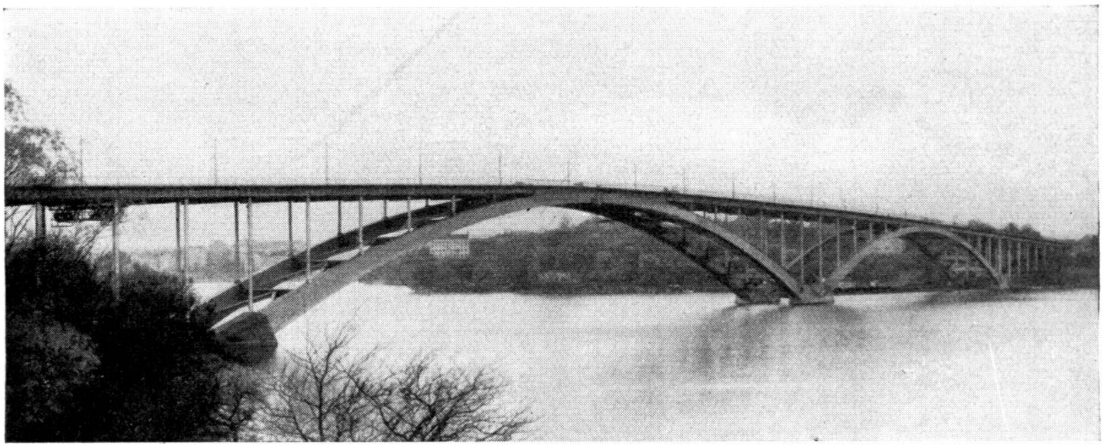


Fig. 1. Western Bridge. General view.

West Bridge:

A Road Bridge over Lake Mälar, Stockholm.

The West Bridge forms part of the main traffic route between Kungsholm and Söderhalm in Stockholm. It comprises two arches spanning the Riddarfjärd, one of which has a span of 168 m and the other 204 m, connecting with viaducts of 12.9 m span as shown in Fig. 1. The overall length of the bridge is 601.5 m, and width 2.5 + 19.0 + 2.5 = 24 m.

The roadway is 30 m above mean water-level, and is reinforced concrete on longitudinal and transverse girders. It has projecting footways. The headway for navigation in the main opening of the arch is 24 m above mean water-level over a width of 50 m at right angles to the navigation channel, and 26 m over a width of 19 m.

The substructure consists of separate concrete foundations carried down to the rock. The average distance apart of the piers, which are stone-dressed at water-level, is 18 m measured across the bridge. The max. depth of the foundations below water-level is 14 m. The foundations are put in inside cofferdams or caissons, in open pits, and partly under water.

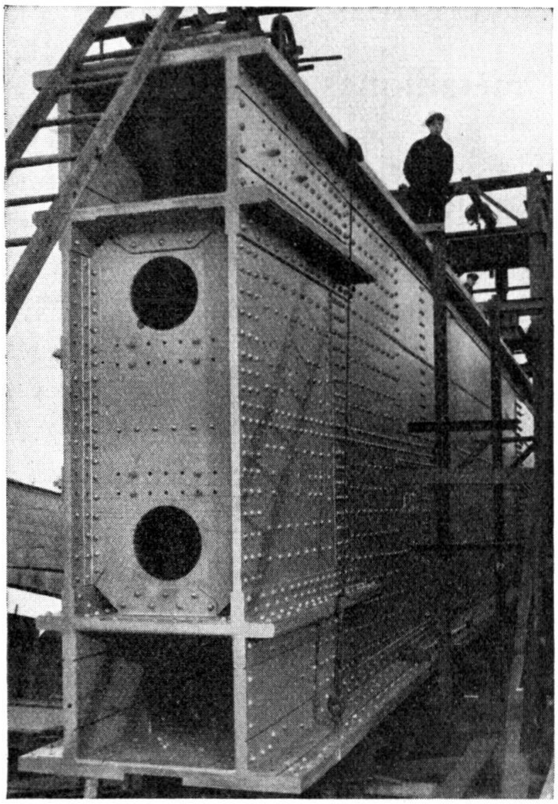


Fig. 2.
Western Bridge. Section of arch at springing.

Fig. 3.
Western Bridge. Support of arches at springing

Steel Superstructure.

Each arch consists of two arched box girders anchored to the abutments and unhinged at the apex. The main girders are 18 m between centres. The overall height at the apex and at the abutments is 2.0 and 4.0 m respectively for the smaller arches, and 2.5 and 4.6 m for the larger ones. The cross-section of the arches and the method of supporting them at the abutments are shown in Figs. 2 and 3. The arches are connected by horizontal bracing. The ratio between the rise and span of the arches is 1/8.2. The arches, including the bracing, are rivetted throughout, the former being made of St 52 Structural Steel and the latter of Steel 44.

The roadway deck consists of rivetted transverse girders spaced 12.9 m apart, and 10 welded main girders 2.13 m between centres in each panel. In the middle of each bay, the main girders are connected by means of transverse load-distributing girders. Below the roadway deck is a horizontal bracing of welded $\perp$-bars rivetted at the points of intersection. The ferro-concrete roadway platform, as also the roadway deck, is supported by tubular, partially welded columns 600—700 mm diameter. Figs. 4 and 5 show the construction of the roadway and the columns.

The transverse girders are made of St. 52 Steel, and all the other components of the roadway deck, including columns and horizontal bracing below the roadway, of St. 44 Steel.

The contract specified that the transverse girders should also be welded. However, for reasons which had nothing to do with any objections to welding, these girders were rivetted.

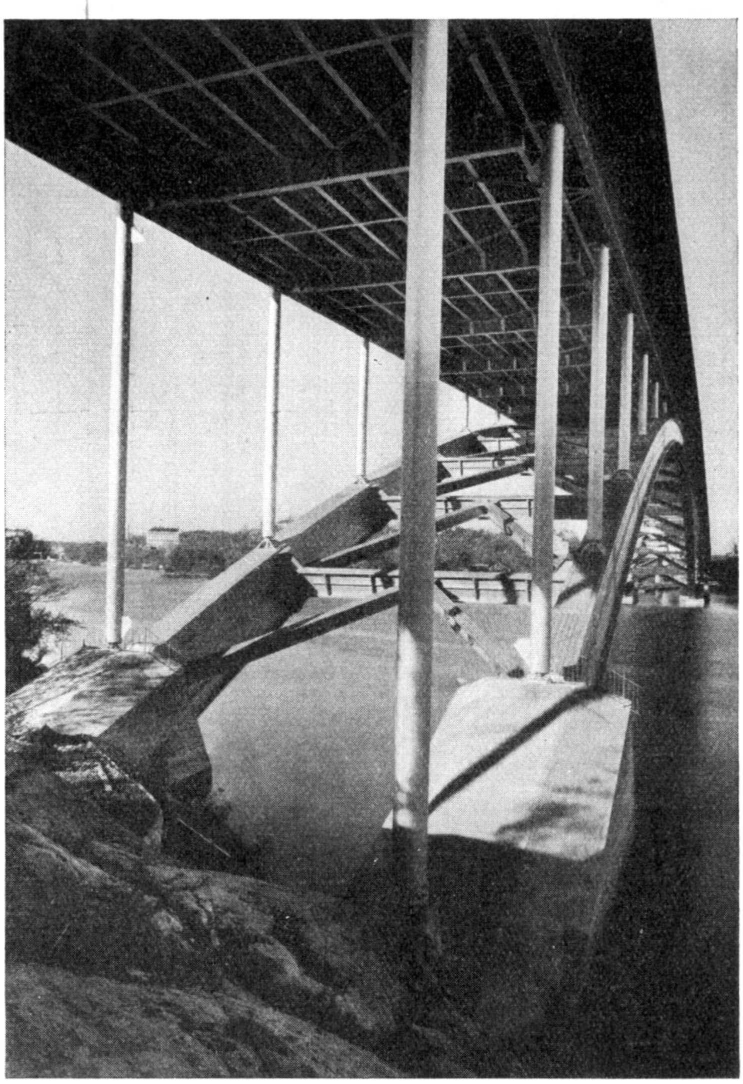


Fig. 4. Western Bridge. Road decking seen from below.

Erecting the Steelwork.

The erection of the steelwork was an interesting operation. The sections of the arches, in lengths of roughly 13 m (or the width of a bay) and weighing 65 tons maximum, were transported on craft from the shops at the Ekensberg Yard situated $1^1/_4$ miles from the site. They were then laid on a fixed staging and rivetted up (Fig. 6). The halves of the arches were then each pushed on to a separate cross-track, towed to the site with the help of a floating dock (Fig. 7), and let down on to temporary piers. When two

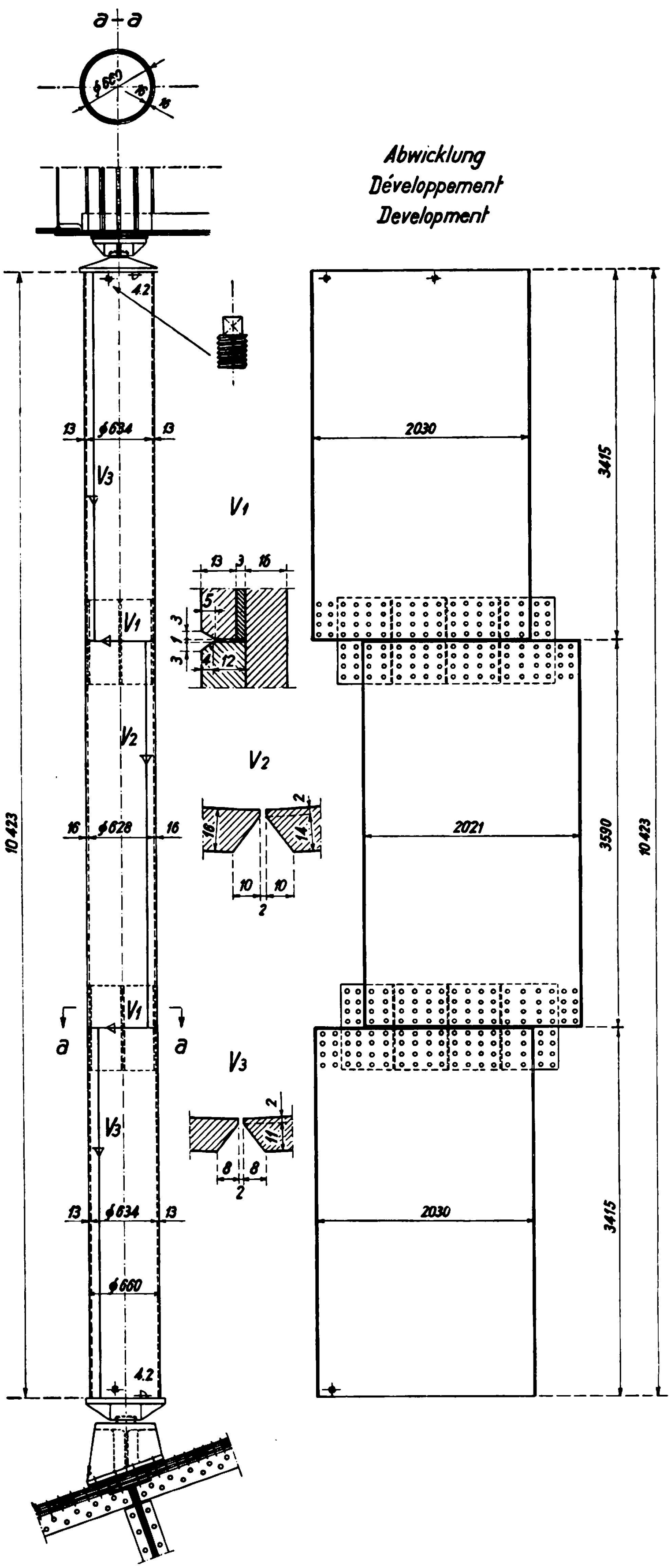


Fig. 5.
Western Bridge. Cylindrical column supporting road construction.

halves of the arch had been braced together, they were lifted in pairs by hydraulic presses mounted on an auxiliary derrick located in the centre of the arch opening, and by Gall chains (Fig. 8), the arches turning about pivots which

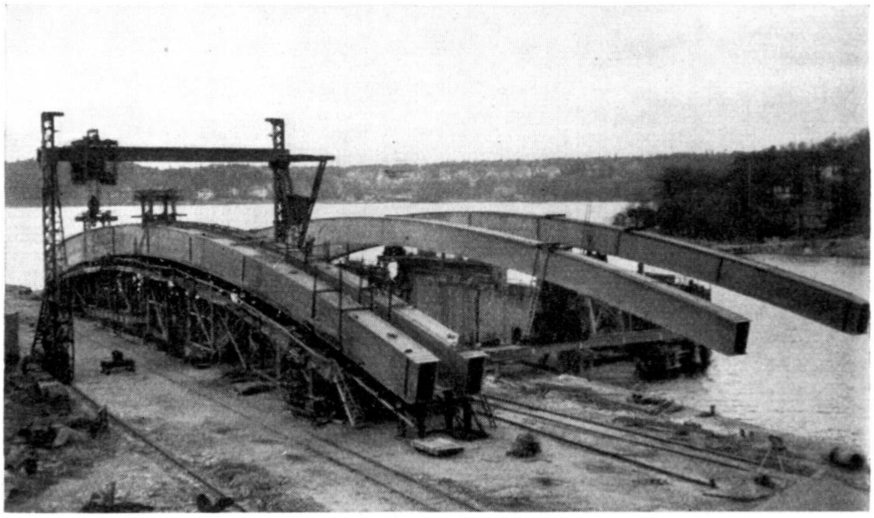


Fig. 6. Western Bridge. Half of arch at Ekensberg Wharf.

were capable of being adjusted in any direction by presses. When they had reached the proper height, the arches were controlled by means of hydraulic presses and temporary cast steel hinges at the apex, so that the optimum restraining moments at the abutments were achieved when all the load and other influences had been allowed for. The tops of the arches were then rivetted together.

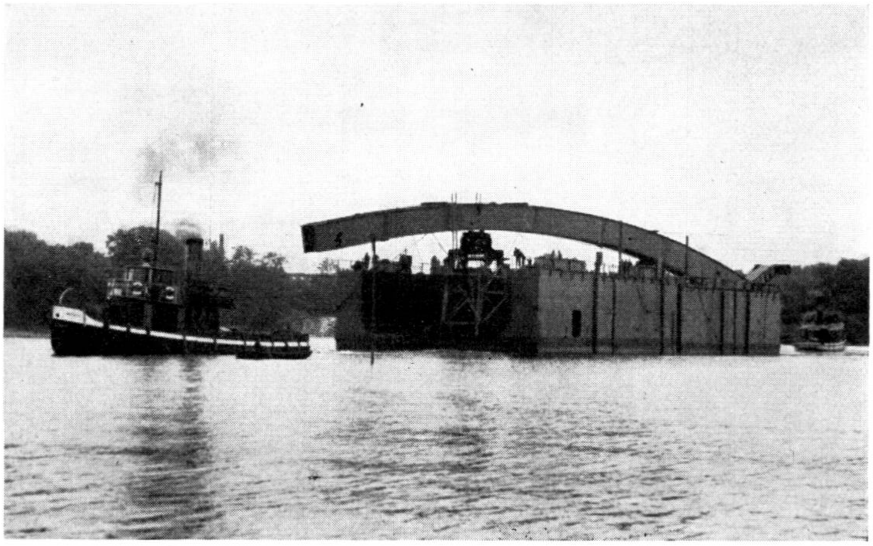


Fig. 7. Western Bridge. Half of arch being transported to site.

The roadway deck, including tubular columns, etc., were built up as shown in Fig. 9, by means of a derrick crane and other auxiliary equipment. It took 2—3 days to assemble a section in this way. The total weight of the steel

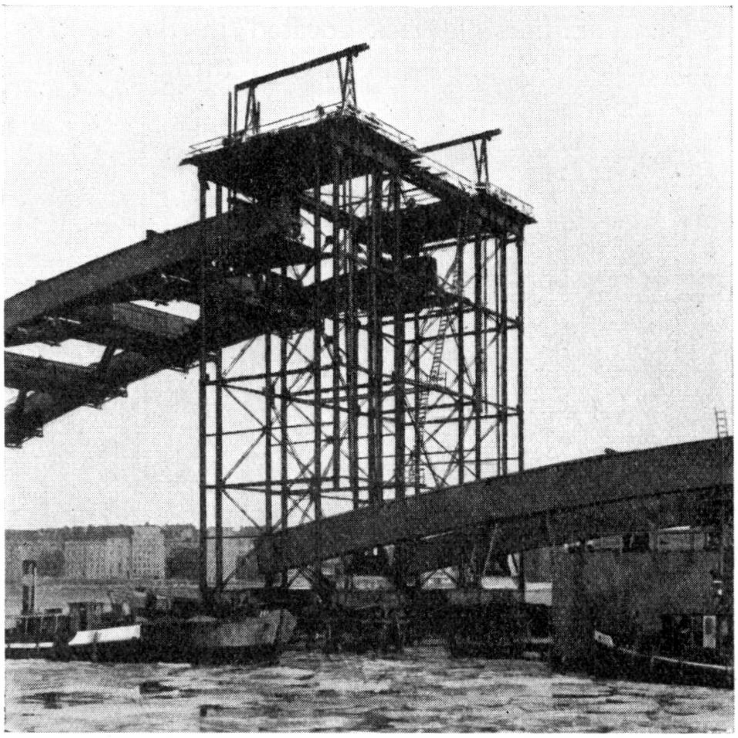


Fig. 8.
Western Bridge.
Auxiliary tower
for raising
arches.

portion of the structure is about 7000 tons, approximately 2000 tons of which are welded. The West Bridge, and the Pålsund Bridge following it, were opened for traffic in 1935. Including the substructure and roadway platform, the bridge cost roughly 6,175,000 Kroner to build.

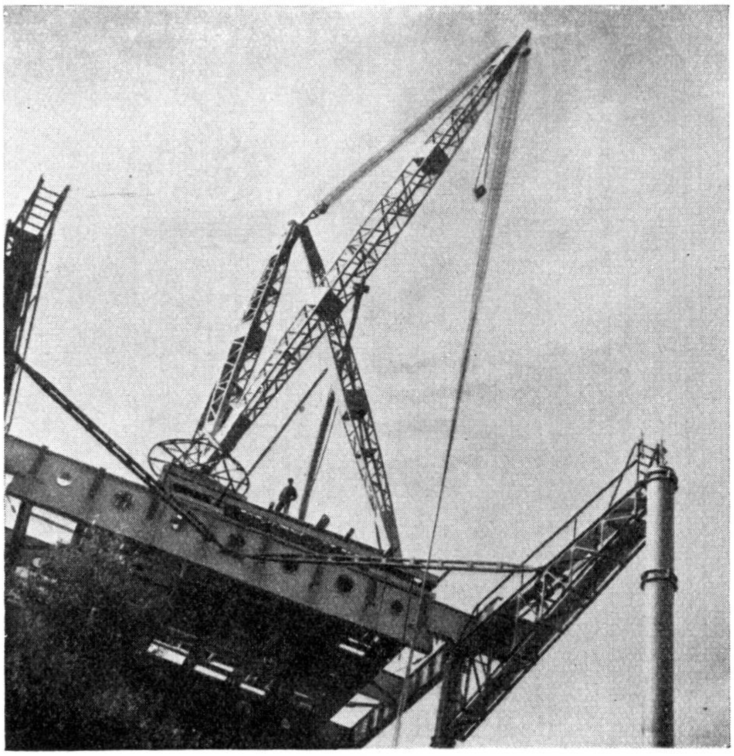


Fig. 9.
Western Bridge.
Assembling the
Viadukt.

Pålsund Bridge.

The other bridge in the line of traffic mentioned above is the Pålsund Bridge, connecting the island of Längholm with Sötermalm. This is a comparatively small bridge, but is interesting because the steelwork is of the completely welded type.

It consists of a 56 m span above the Pålsund, and connecting viaducts of 1 m span at either end (Fig. 10). The total length of the bridge is 276.6 m, and the width of the roadway 24 m. The roadway deck and roadway platform are very similar in design to those of the West Bridge, except that the number of girders in

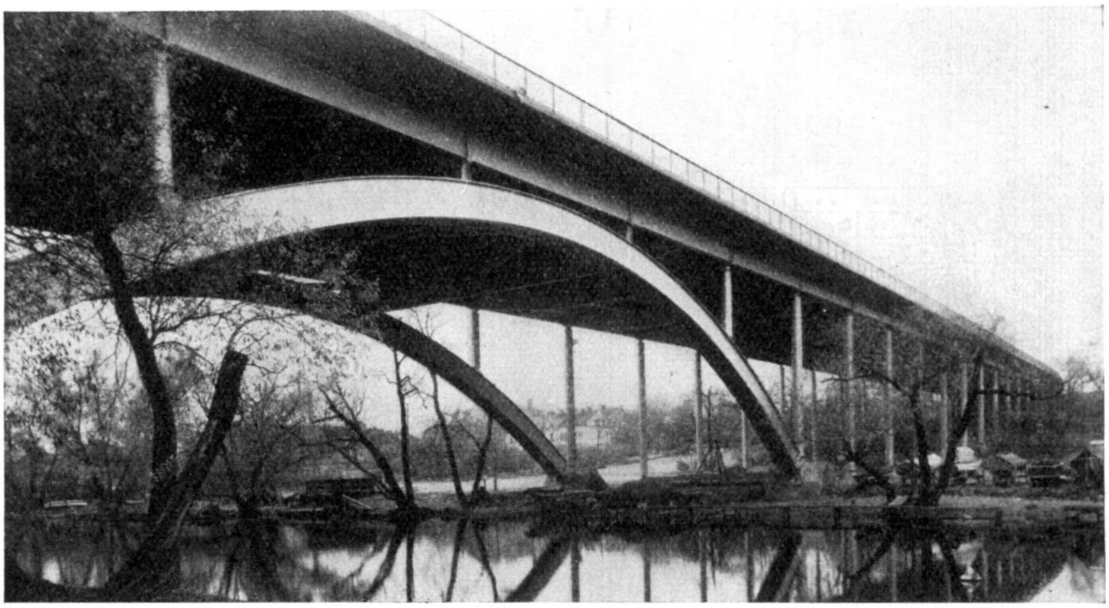


Fig. 10.
Pålsund Bridge. General view.

each bay is 7 istead of 10. A notable feature is the method of designing and making the cross-girders and arches. The cross-girders, 19.5 m long and 1.13—2.06 m high, consist of a 17 mm gauge web, and flange plates of 450 × 60 mm at the centre of the girder, and 300 × 60 mm at the supports (Fig. 11). The "lugged" flanges are welded at two points by vee butt joints, and the webs are stagger-butt welded in the middle of the girder. This latter type of welding was modified whilst work was proceeding, until the joint finally assumed the form shown in Fig. 11. The transverse girders, weighing about 12 tons each, were completed in the shops and conveyed on special trucks to the site.

The double-hinged arches of the main span were practically built up on the site, where the necessary equipment had been provided. The arches are parabolic and of box-section (Fig. 12). Each half of the arch was completed in four sections, which were then placed on a derrick and welded up (Fig. 13). Both the horizontal and vertical joints were vee-welded. The internal stiffening of the arches was carried out by means of transverse partitions or bulkheads with manholes at distances of roughly 6 m. The lower flanges also had manholes through which the men could get access to the inside of the arch for

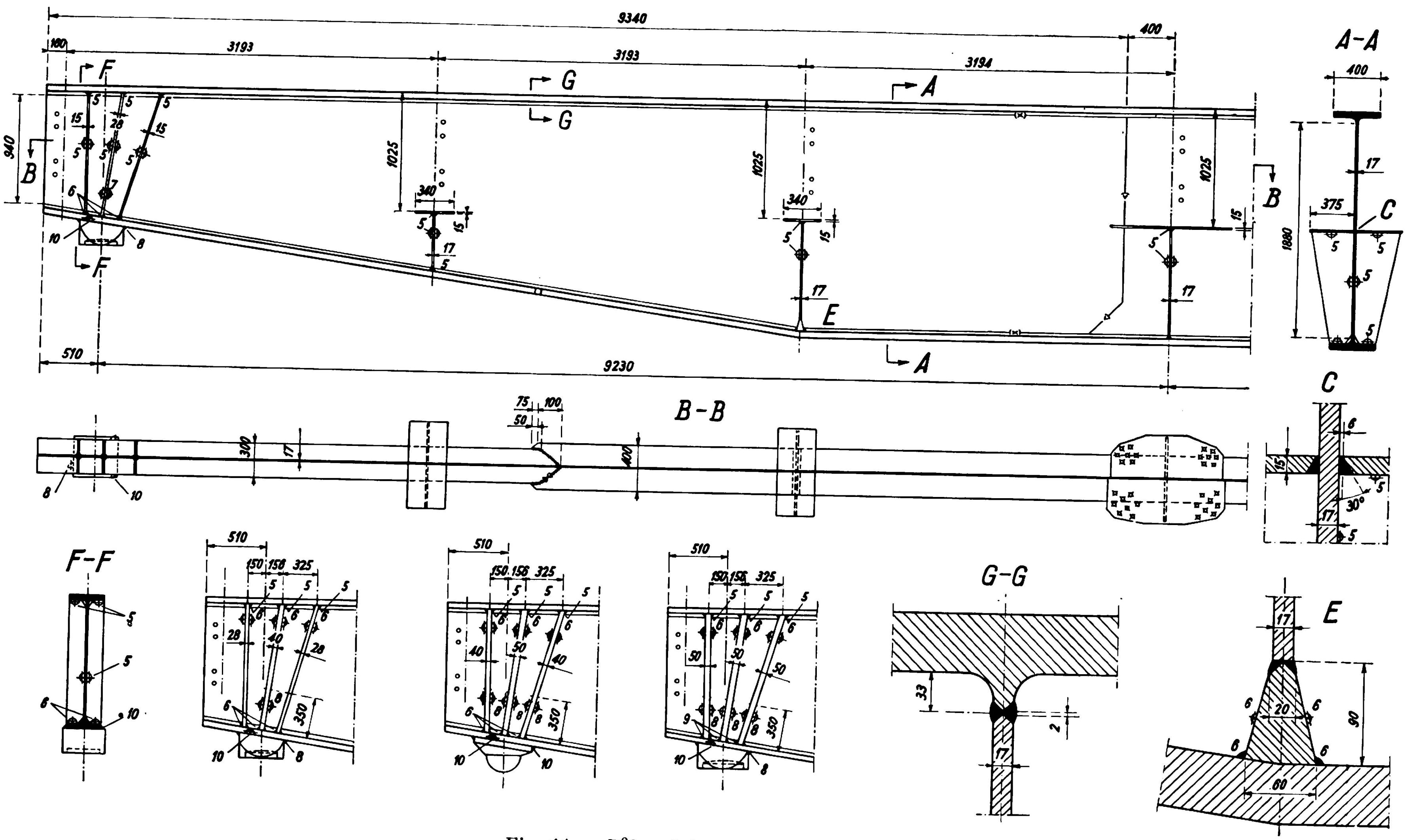


Fig. 11. Pålsund Bridge. Cross beams.

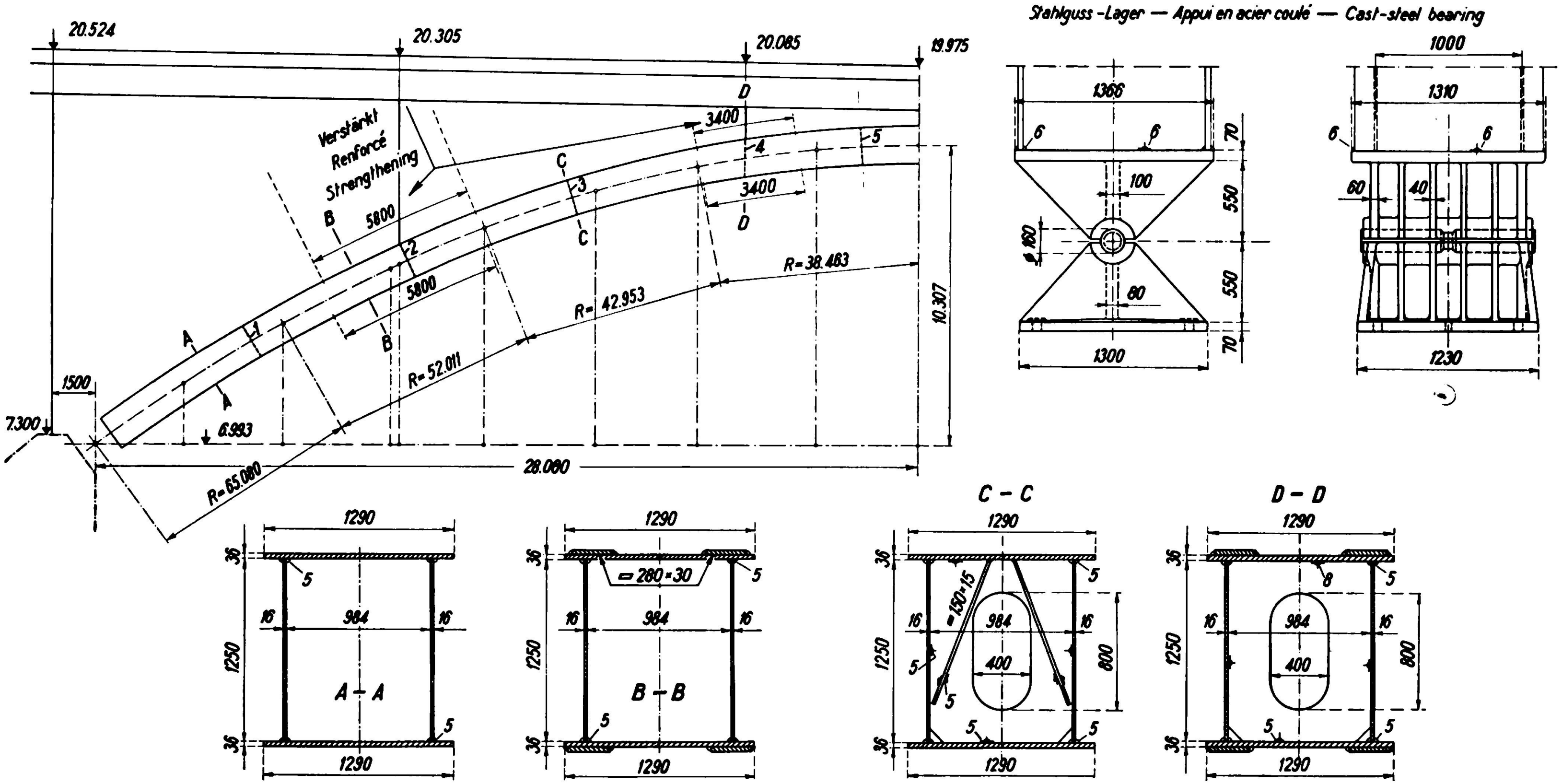


Fig. 12. Pålsund Bridge. Arch girders.

inspecting the paintwork, etc. At the places where the maximum moments occur, the flanges of the arch are reinforced by butt-straps, but were only butt-welded at the joints.

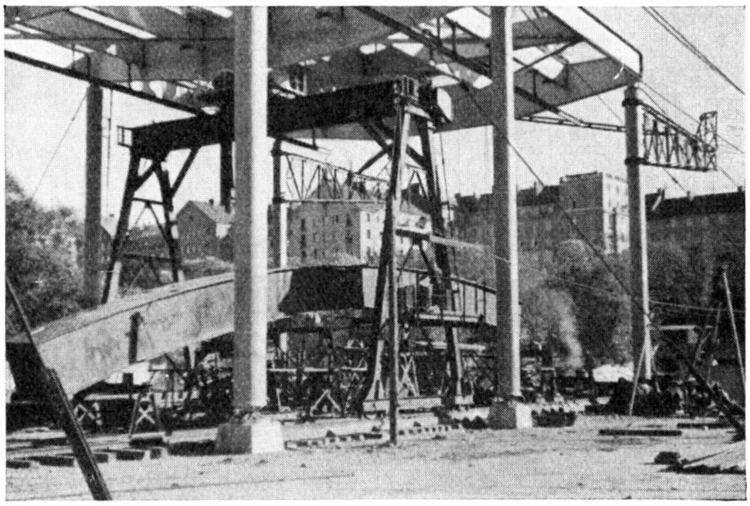


Fig. 13.
Pålsund Bridge.
Welding arch sections at site,

The following method was adopted for erecting the arches. By means of the above-mentioned derrick and trucks built inside it, the halves of the arches were run out on a track over the Sound, and there moved on other trucks and lines in the transverse direction of the bridge, and manoeuvred into the proper position. With the aid of hoisting tackle and temporary latticework trestles,

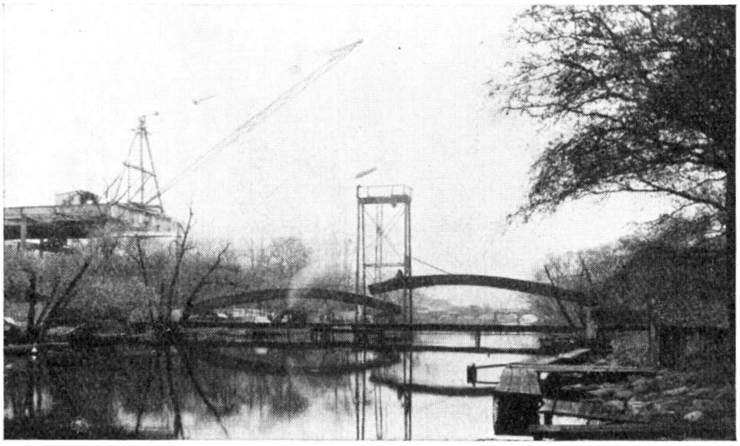


Fig. 14.
Pålsund Bridge.
Erecting the arches.

the halves of the arches were then each raised singly to the correct level (Fig. 14), pivoting on temporary pivots on the abutments. The halves of the arches were then welded together at the apex in the same way as the other butt-joints.

The total weight of the steel superstructure of the bridge is about 1100 tons, including steel castings. The transverse girders account for 265 tons. The latter are made of St 52 Structural Steel, and the other parts of St 44 Steel. The total cost of the bridge is 1,230,000 Kroner.

St. Eriks Bridge.

Last year, a start was made with the construction of new steel superstructure for the St. Eriks Bridge in Stockholm. The present bridge, only 30 years old, consists of three arch openings each of 40 m span, and two subsidiary openings with lattice superstructures each of 26.7 m span at either end of the bridge (Fig. 15). The total length of the bridge is 226 m, with a useful width of 18 m. In order to make it suitable for through traffic, for which the West and Pålsund Bridges were built, the St. Eriks Bridge is now to be widened to 24 m and adapted to carry the same load as the other two bridges. During the conversion, provision is also to be made below the roadway of the bridge for a second deck to carry a double-track suburban railway bridge. When completely converted and rebuilt, the bridge will appear as shown in Fig. 16. The present foundations (put in by

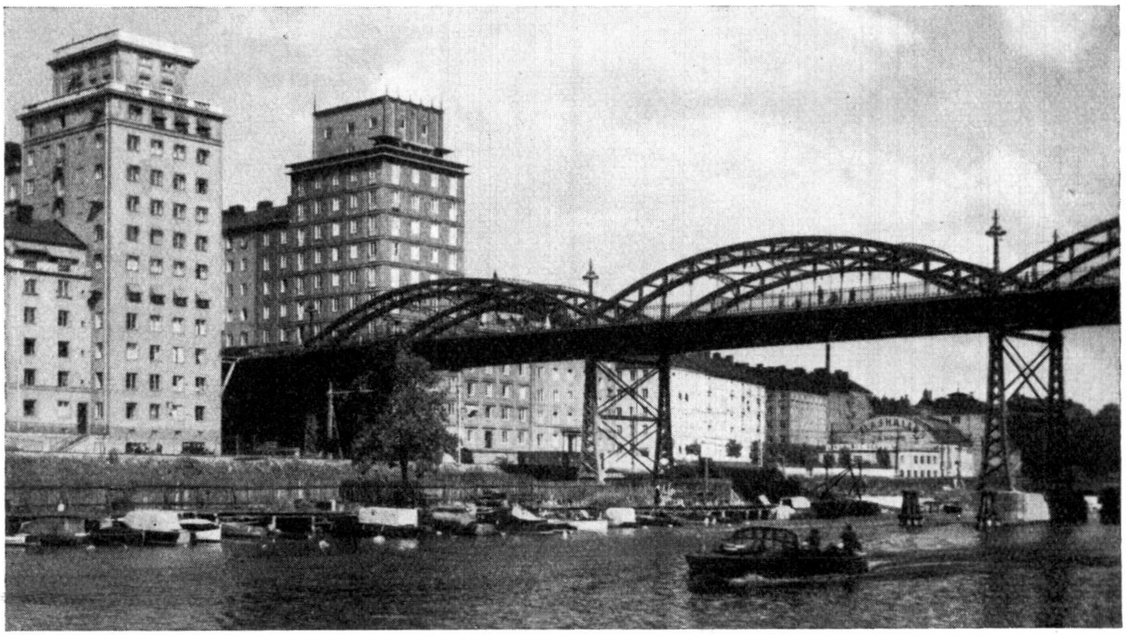


Fig. 15.
St. Erik's Bridge. The old bridge.

compressed air) will remain as they are after the concrete masonry has been strengthened by injecting cement. After the steel supports on which the existing steel superstructure rests have been grouted in and widened by brackets (Fig. 16), a new steel superstructure will be put in place, consisting of six continuous main girders, and transverse girders in the centre portion of the bridge. All the new structural parts will be welded throughout.

The main girders have webs 2 m high, and are composed of single unstiffened web-plates and fillet-welded flange-plates. The design of the steel superstructure is shown in section in Fig. 17. The erection joints of the main girders, put in place in the first stages of construction, together with the cross-bracing joints, will be rivetted, but subsequently all the joints will be welded on the site, partly to avoid the noise of rivetting.

The replacement of the steel superstructure and the widening of the bridge will be carried out in three sections. The first and second sections will be devoted to removing the old projecting footways, fitting in place the two outer main girders with their bracing, and temporarily completing the reinforced concrete roadway. During this period, pedestrian traffic in both directions will be taken care of first by one and then by the other footway. In the third stage, the entire traffic (including vehicular and street car traffic) will be carried over the completed strips, and it will then be possible to replace the middle section of the bridge.

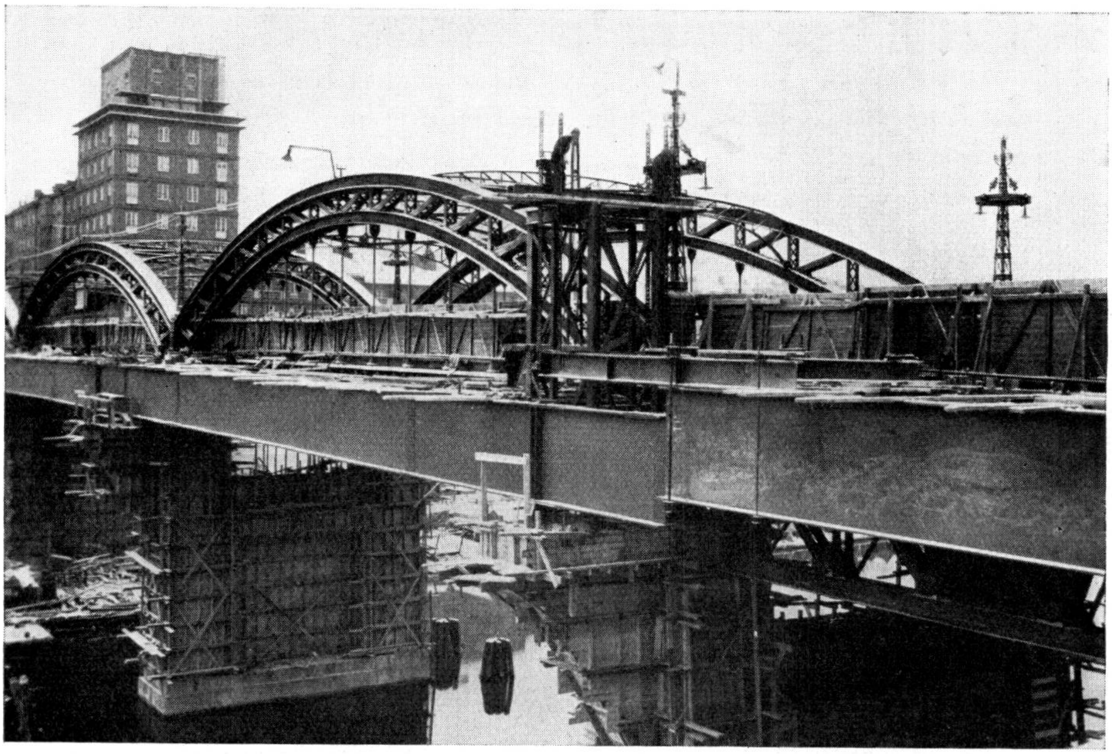


Fig. 18.
St. Erik's Bridge. Placing of new steel superstructure.

The old structure will be utilized as a support for the erection of the new steel superstructure. Subsequently, after the old steelwork has been removed, the new superstructure will be deflected into its final position on the piers with the aid of trestles on the concrete piers, hydraulic presses, and Gall chains (Fig. 18). The main and transverse girders are of St. 48 Structural Steel, and all other parts are made of St. 44 Steel. The total weight of the new steelwork is roughly 1100 tons, as compared with 1600 tons for the old bridge. The work of converting and rebuilding the bridge will be completed during the coming year.

Small Essinge Bridge (now building).

The reason for describing the comparatively small road bridge between Kungsholm and the island of Lilla Essingen is partly because the steelwork is to be welded throughout, and partly because of the special truss design. Formerly, this type of construction was used almost exclusively for timber structures, but

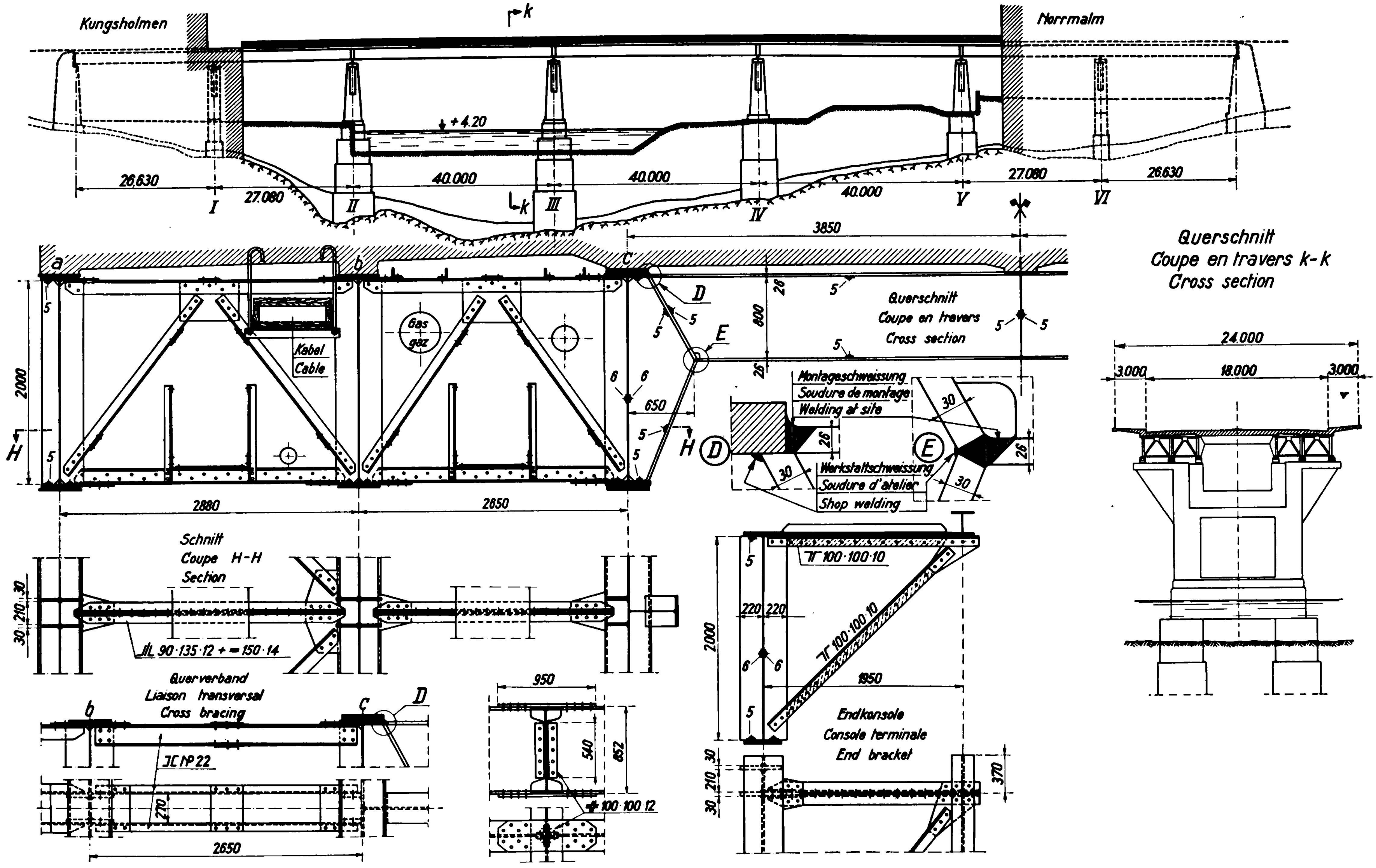


Fig. 16 and 17. St. Erik's Bridge. The bridge after reconstruction.

it also offers economic advantages when steel is used as the structural material. This construction will enable the main opening to be divided up into three bays with continuous girders, and obviate the expense of having intermediate piers in the water.

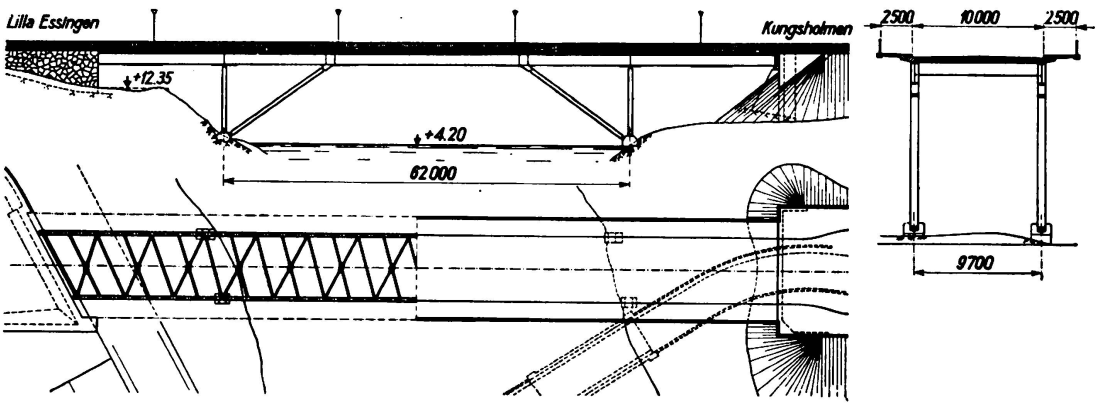


Fig. 19.
Little Essinge Bridge. General arrangement.

The bridge, of a total lenght of 109 m, consists of two continuous girders spaced 9.7 m apart, and intermediate transverse girders upon which the ferro-concrete roadway deck rests directly (Fig. 19). The river opening has a span of 62 m, and the width of the roadway is 2.5 + 10.0 + 2.5 = 15.0 m. The total weight of the welded steel superstructure is roughly 240 tons.

Summary.

During the last few years, the following comparatively large steel bridges have been erected in Stockholm, in which welding has been extensively adopted.

1. *West Bridge,* a road bridge over Lake Mälar in Stockholm. It consists of two arched main openings of 168 and 204 m span respectively, and connected viaducts of 12.9 m span. The total length of the bridge is 601.5 m, and the width of the roadway 2.5 + 19 + 2.5 = 24 m. Total weight of superstructure, partly of St 52 Structural Steel and partly of St 44 Steel, approx. 7000 tons, of which about 2000 tons are welded.

2. *Pålsund Bridge,* another bridge in the same line of traffic as the West Bridge, consists of a 56 m double-hinged arch opening and connecting viaducts of 12 m span. Total length, 276.6 m; width, 24 m. Total weight of superstructure, partly of St 52 Steel and partly of St 44 Steel, about 1100 tons. Welded throughout.

3. *St. Eriks Bridge,* a road bridge being converted and rebuilt without interruption to traffic, the steel superstructure of which consists of continuous plate girders of 26.63 + 27.08 + 40.0 + 40.0 + 40.0 + 27.08 + 26.63 = 227.42 m span. Width, 24 m. Total weight of the steel superstructure of St 48 Steel, about 1100 tons, welded throughout.

4. *Small Essinge Bridge,* a 15 m wide road bridge under construction, with a 62 m main opening, built on the truss system, and 109 m total length. Total weight of the steel superstructure of St 44 Steel, about 240 tons.

VII a 7

Use of Steel in Bridge Building. In General and in Detail.

Anwendung des Stahles im Brückenbau. Allgemeines und Einzelheiten.

Application de l'acier dans la construction des ponts; généralités et détails.

Geheimrat Dr. Ing. G. Schaper,
Reichsbahndirektor, Hauptverwaltung Berlin.

Introduction.

As is well known, in bridge-building steel is principally to be preferred to other materials such as wood, stone, concrete and reinforced concrete, when the building height available is limited or the form of substructure or other contingencies make it necessary to employ spans of the greatest practicable length. More attention is being paid today than was the case formerly, to the aesthetic factor of making new constructions harmonise with their surroundings. The designing engineer is of course expected to devote the greatest care to the artistic aspect, both in general and in detail, of his creation; yet when it is a question of an important project he will be well advised to cooperate closely with the architect. The latter must then be able to give due consideration to the flow of forces and also the question of costs. Only by really efficient collaboration between these two parties can structures be created that will not be found wanting when judged by the standards of posterity. When deciding upon the type of structure it has been found very profitable to use scale models or to sketch in the structure planned on photographic reproductions of the site and surroundings. In Germany recently new enterprises, and especially the construction of the new State Arterial Roads, have led to the building of numerous large bridges. Many of these bridges are welded, and in our paper we shall give a brief description of some of them.

I. *Frames.*

Example 1.

Fig. 1 illustrates a solid-webbed, all-welded, two-hinge frame for a bridge over a road. Its appearance gives an impression of great lightness. The distance between the hinges is 21.2 m. Over the width of the roadway below, the lower boom is almost horizontal, thus giving constant clearance. At the sides, too, the height is sufficient to accomodate the footpaths. Where the boom is bent downwards the decking is borne by supports carried by the main girders. The web-plates are butt welded somewhere in the region of the zero point of moments.

The boom plates are continuous throughot the whole length. No special measures were necassary for the formation of frame angles. The frame footings are stiffened. The arrangement of the decking may be seen from the cross section.

The whole superstructure was welded in the workshop and transported by rail to site in one piece.

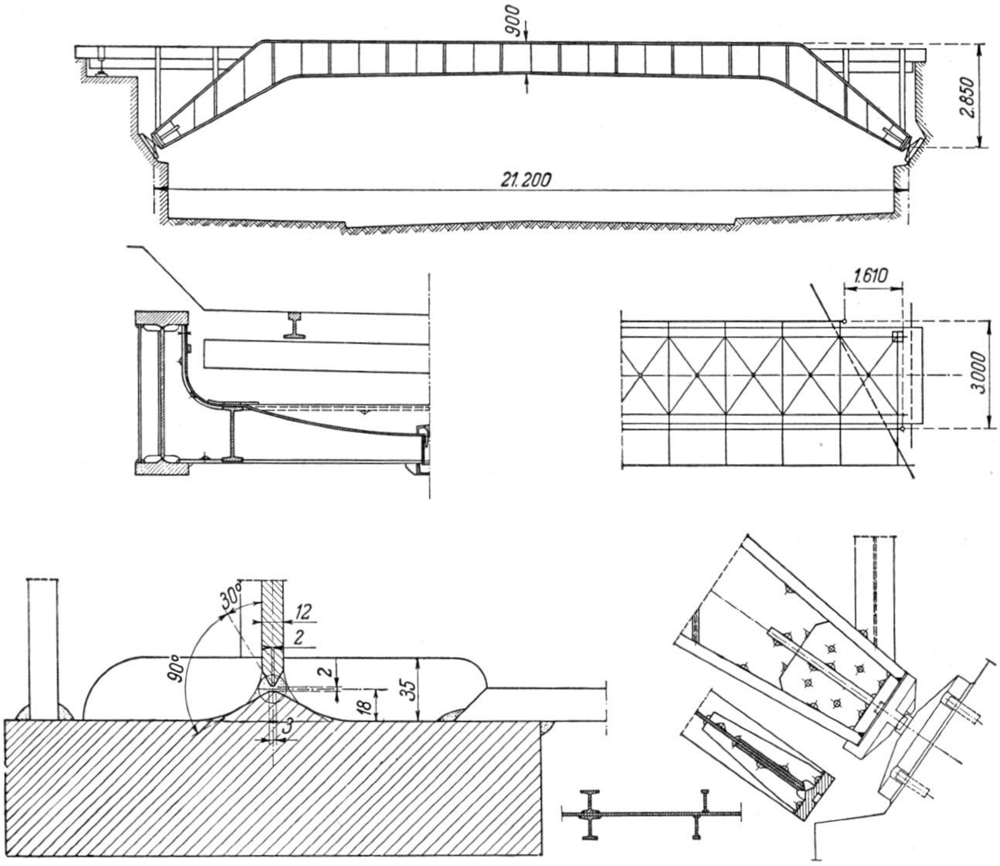


Fig. 1.

Example 2.

Fig. 2 depicts two-hinge frames used for an overbridge carrying a highroad over (right) a railway line, and (left) over an arterial motor road. In this —

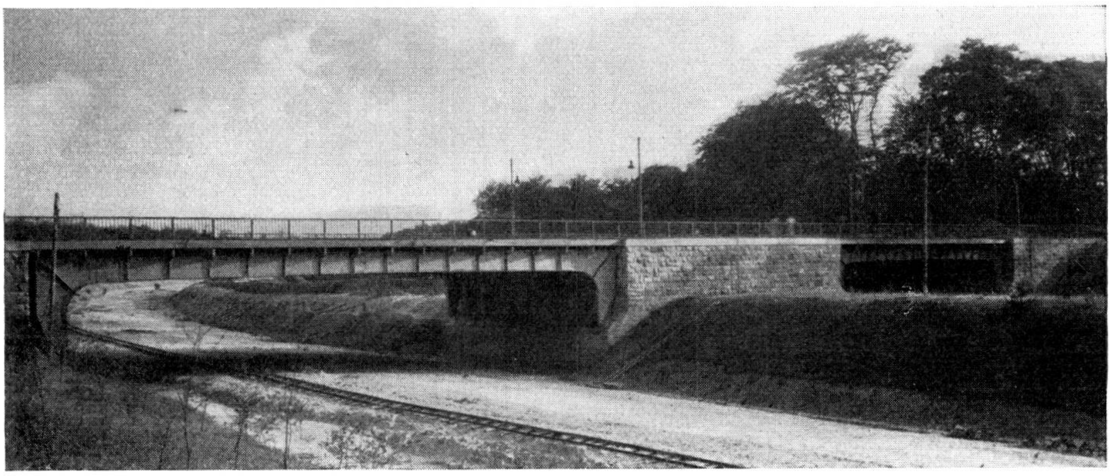


Fig. 2.

as indeed in all the constructions carried out for the State Arterial Roads — great attention was paid to outward appearance. The treatment of the masonry, the arrangement of parallel wing-walls and the continuity of the coping over the whole length of the bridge combine with the trim lines of the steelwork and the fine shadow effect thrown by the cantilevering footpaths to produce a harmonious structure that fits into its surroundings excellently. The distance between the hinges of the frame measures 33 m on the portion passing over the motor highway.

Example 3.

Fig. 3 shows a single-webbed, two-hinge frame, 3a being the riveted, 3b the welded construction of same.

In the riveted frame (3a) the flanges of the lower brace pass in a gentle curve into the vertical. The upper flanges of the brace, however, and the outer flanges of the upright meet at right angles. The angles of the booms are mitred at this

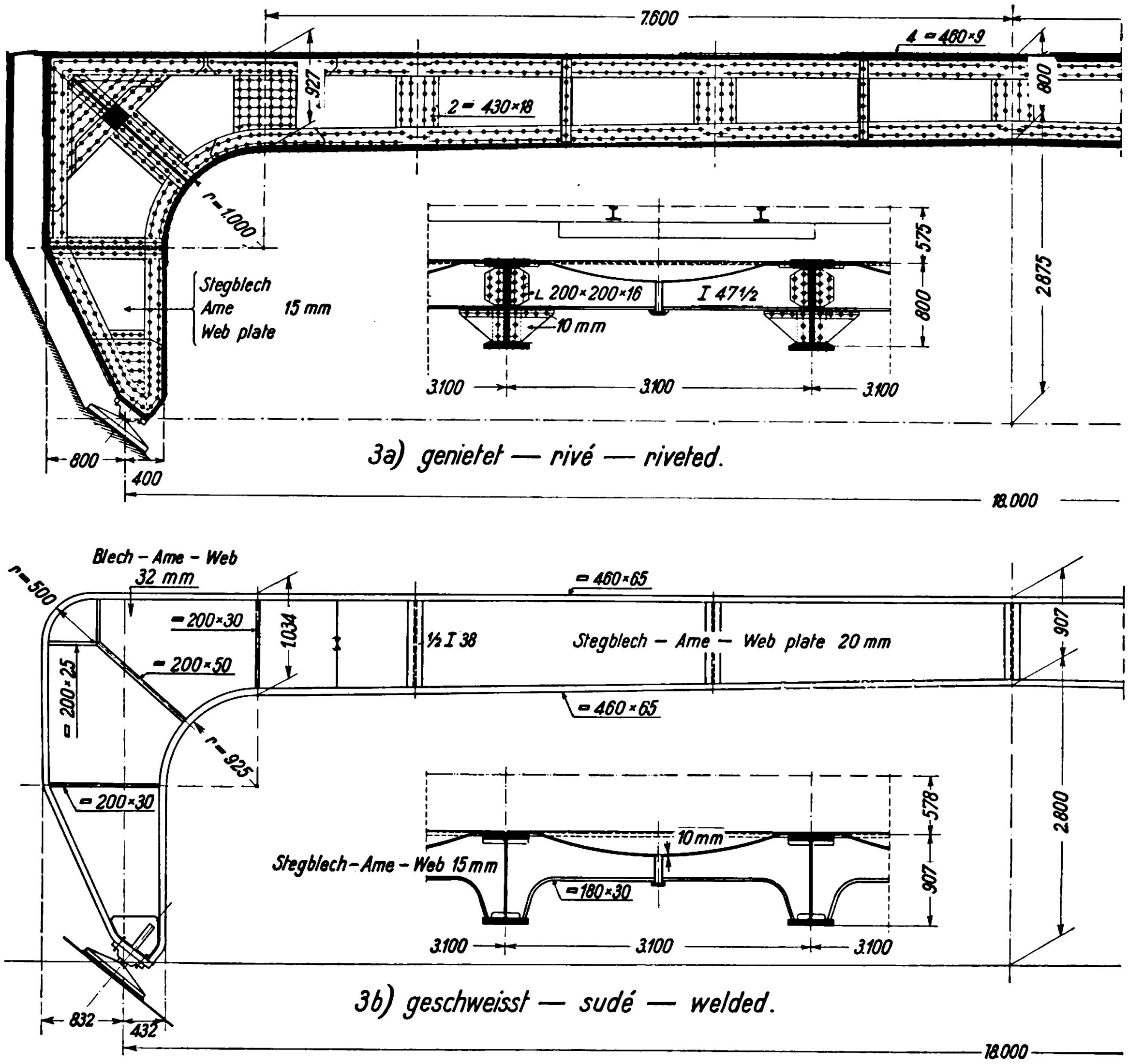


Fig. 3.

Two-hinged plate girders.

point. The flange plates of the brace, one of which is broader than the other to take the buckle plates of the decking, end in this corner, as do the outer flanges of the upright. As the forces set up in them and in the angles of the flanges cannot be transmitted by the web plate alone without its becoming overloaded, the contiguous legs of the angles of these two flanges are connected with each other by means of 30 mm thick, fitted cover plates which are also connected to the corner web plate by means of intermediary packing. The corner web plate itself is continuous right down to the bearing. The web plate of the brace is joined to the corner plate where the curve begins, the joint being covered on both sides by straps. Strong stiffening is arranged in the central line of the corner. The wall of the frame footing is reinforced by plates affixed on both sides. The angles and the flanges of the lower boom run unjointed from bearing to bearing.

Fig. 3b shows how much simpler the two-hinge frame can carried out in welding construction, and how much more satisfactory its appearance then is. Both main girders belong to the same structure, have the same dimensions and both are made of St 37. The riveted main girders weigh 19.4 t, and the welded ones only 14.3 t each. The constructional design of the welded girders is simple. The top flange plate, consisting of a Dörnen bulb section[1] measuring 460×65, also becomes the outer flanges of the uprights, that is to say, the top flange runs from bearing to bearing in one piece, across the whole 18 m width of the superstructure. The lower flange is also made in one single piece. Both flanges are interconnected at their ends by V-welds, the web being stiffened at the bearing by means of 10 mm thick plates affixed on both sides of it. These plates were fillet welded on top to the web and on the other sides tapered together with the web and welded to the flanges of the boom. (Present-day regulations for calculation would permit a further simplification, namely the use of one single web plate, of corresponding thickness, in place of the thin web stiffened by the two 10 mm stiffeners at the bearing. This one-piece web would then be butt welded to the other thin web.)

To prevent buckling under the action of the considerable compressive forces, the stiffening plates are connected with one another and with the web by means of rivets (this riveting would naturally be eliminated if the thicker web were used). Here the stiffeners are also welded to the booms. Today, the parts of the boom subject to tension would be provited with compensation plates where, in accordance with regulations, the bending stresses make this necessary.

II. *Beam stiffened with Arch (Stabbogen).*

Example 1.

In recent years numerous bridges have been constructed in Germany with Langer beams (so called after their inventor, the Austrian engineer Langer) having solid-webbed stiffening girders.

As will be seen in Fig. 4, the appearance of bridges thus designed is restful and pleasant, and the general impression one of charming harmony with the surroundings. The main span bridges the river in a single arch. The lateral spans are much smaller and comprise girders situated at the same height as

[1] See Fig. 5 of Kommerell's paper, Theme IIId.

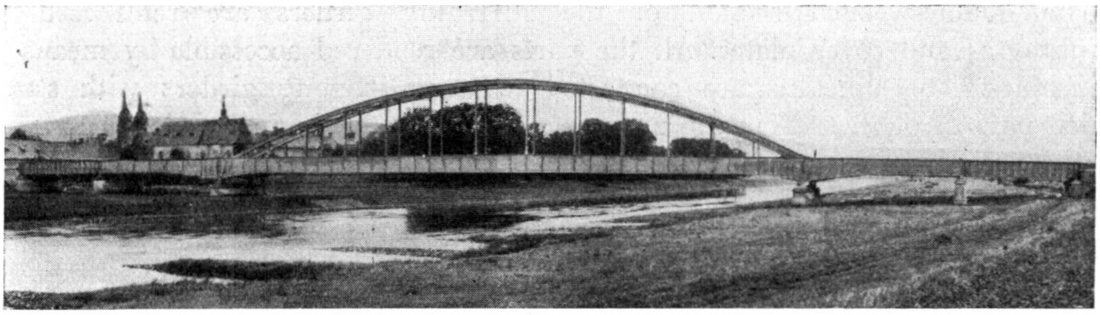
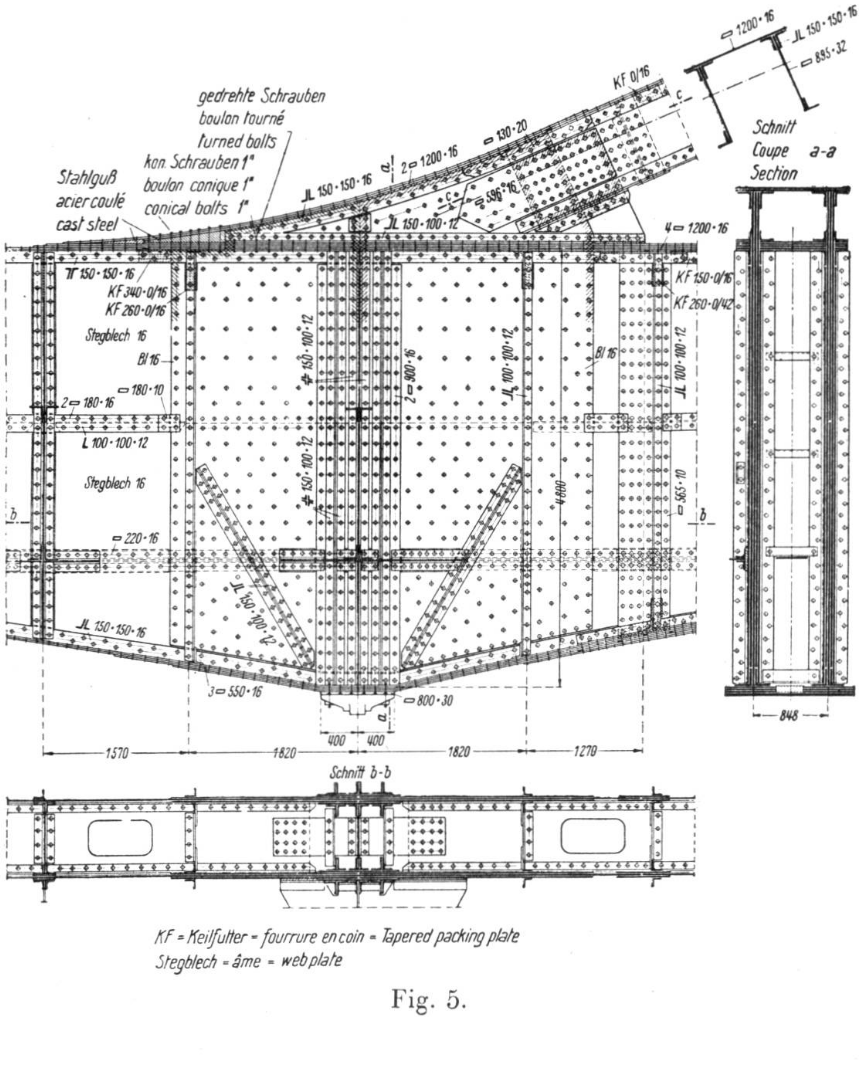


Fig. 4.

the stiffening girders of the central span and forming with them a continuous beam. In this uninterrupted system of girders the main span is set off in a natural and pleasing manner by the third boom — the arch — consisting of the Langer beam. The central span over the water has a length of about 120 m, the adjoining opening are 33.4 m and the end spans to the abutment about 26 m long. The whole bridge is riveted. The web of the single-webbed girders used in the end spans is 3.2 m high, that of the double-webbed girders in the intermediate spans increases gently from 3.2 m to 4.8 m at the piers of the main span. Fig. 5 is a detailed illustration showing the main girder at the end of the arch; Fig. 6 is a cross girder.



KF = Keilfutter = fourrure en coin = Tapered packing plate
Stegblech = âme = web plate

Fig. 5.

The double-webbed section of the stiffening girders are reinforced by diaphragms and cover plates. All the parts are rendered accessible by means of man-holes. The uprights are connected to the stiffening girders with straps which pass through slots in the upper flanges of the stiffening girder. The beam of the arch is also double-webbed.

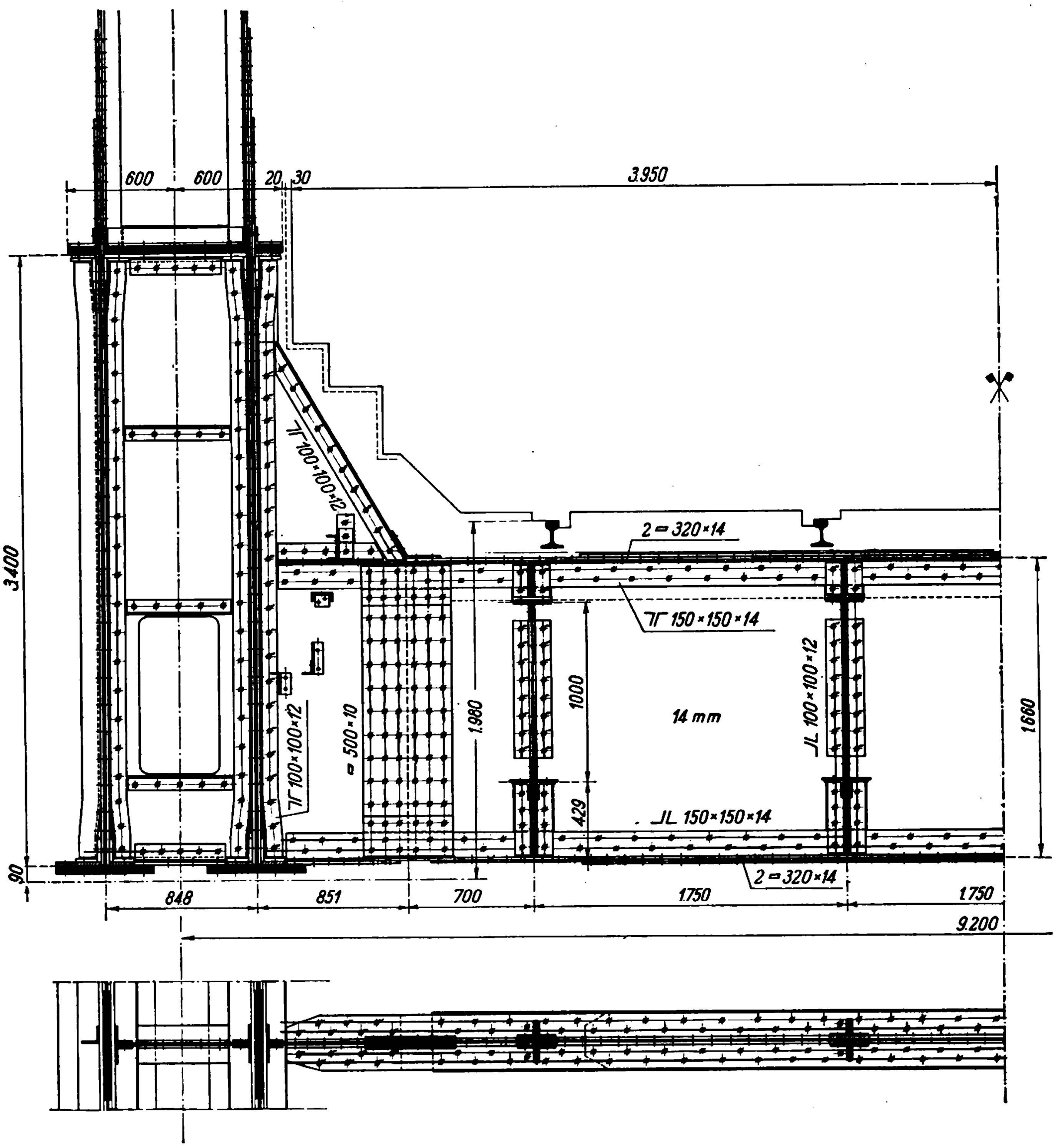


Fig. 6.

The respective arches from which the two main girders are suspended are interconnected by strong upper wind bracing (Fig. 7), whose horizontal forces are conveyed to the supports via the portals. Fig. 8 depicts the top brace of the portal.

Example 2.

Fig. 9 shows an example of a completely welded bridge superstructure of the "Stabbogen" type. Its span is 103 m, the panelpoints lying in a parabola. Each of the members comprising the arch is curved.

The stiffened girder is of one-webbed section, the arch, however, double-webbed. Suspension is effected by means of round steel bars.

Beside the finished superstructure can be seen portions of the adjacent superstructure, still incomplete, which are carried on turning appliances so that the welds can be carried out in the horizontal.

III. *Solid-webbed Girders.*

The construction of the arterial motor roads often necessitated the building of large viaducts in mountainous and hilly districts.

Example 1 is that of a continuous plate beam bridging three spans of 90, 108 and 90 m respectively (Fig. 10). Apart from the question of appearance,

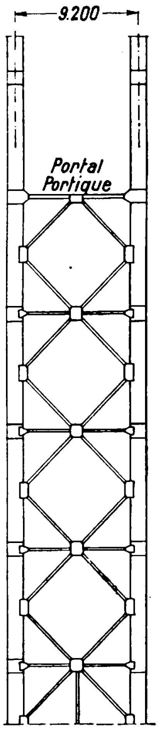


Fig. 7.

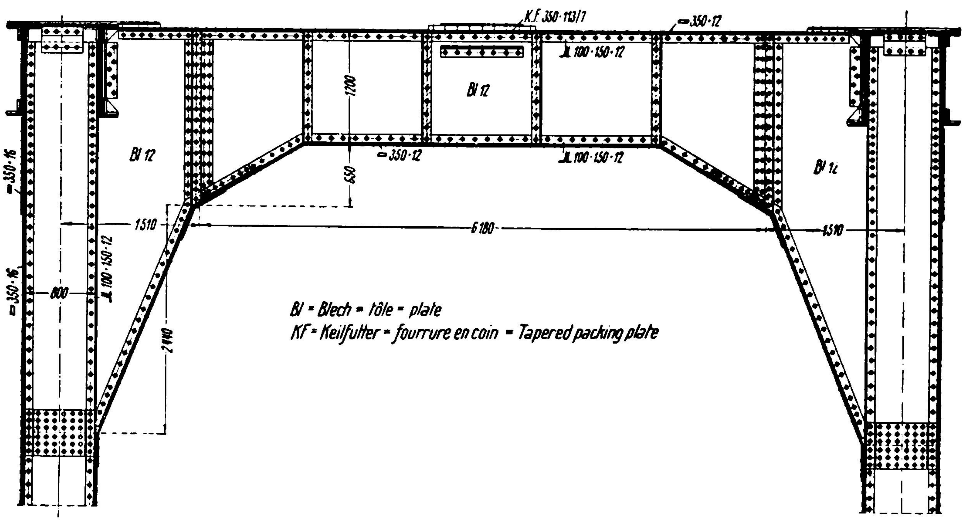


Fig. 8.

it was principally the soil conditions that led to the choice of this type of structure. The two intermediate supports consist of two pairs of hollow reinforced concrete piers, each pair on a common foundation. They are connected on top by a solid cross brace (Fig. 11). The interior of the piers is accessible from outside. The bridge carries two roadways of standard width (7.5 m each) across the valley at a height of 68 m. The decking, composed of two separate reinforced concrete slabs (Fig. 12), is borne on continuous longitudinal I 50 beams, which are rigidly connected with the cross beams, spaced at 6 m intervals, by means of small bearing plates and lateral cleats. Although the decking slab itself is extremely rigid, a k-type bracing was provided at the level of the cross-girder bottom booms; this was chiefly necessary for erection purposes. The cross girders are of the half-frame type (Fig. 13), so that they supply the necessary lateral stiffening action for the bottom booms of the main girders. The web-height of the cross girders is 1.8 m. The web of the brackets

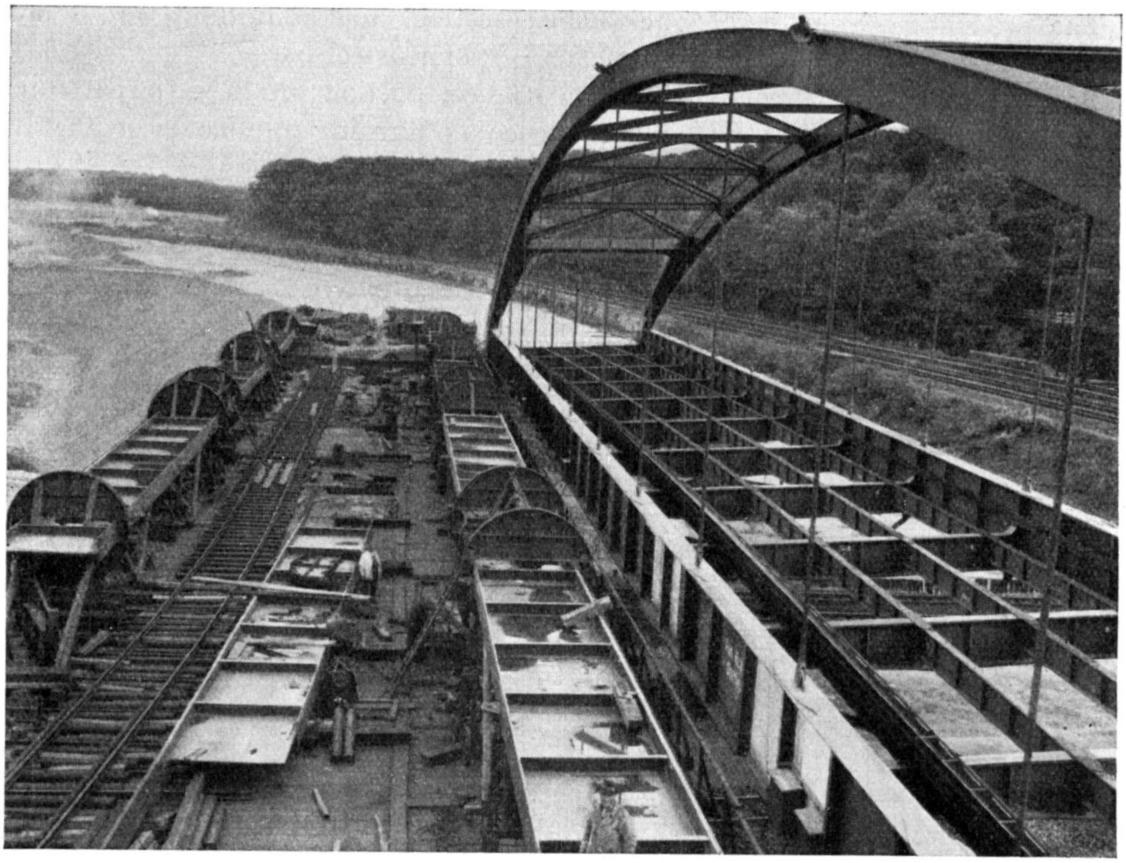


Fig. 9.

as well as that of the cross girders butts against the main girder. A direct transmission of tensile forces from the bracket to the cross girder is not possible, since the web of the main girder could not be slotted. The tensile forces are taken up by broad straps fastened to the main girder flange. The webs of the main girders are joined along their whole length at half their height. The

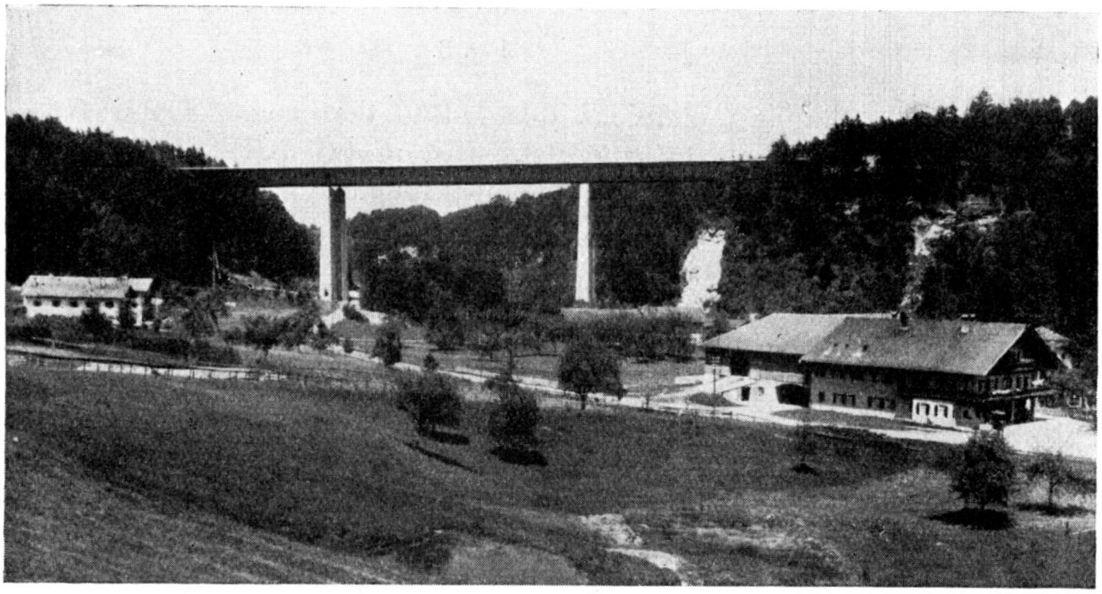


Fig. 10.

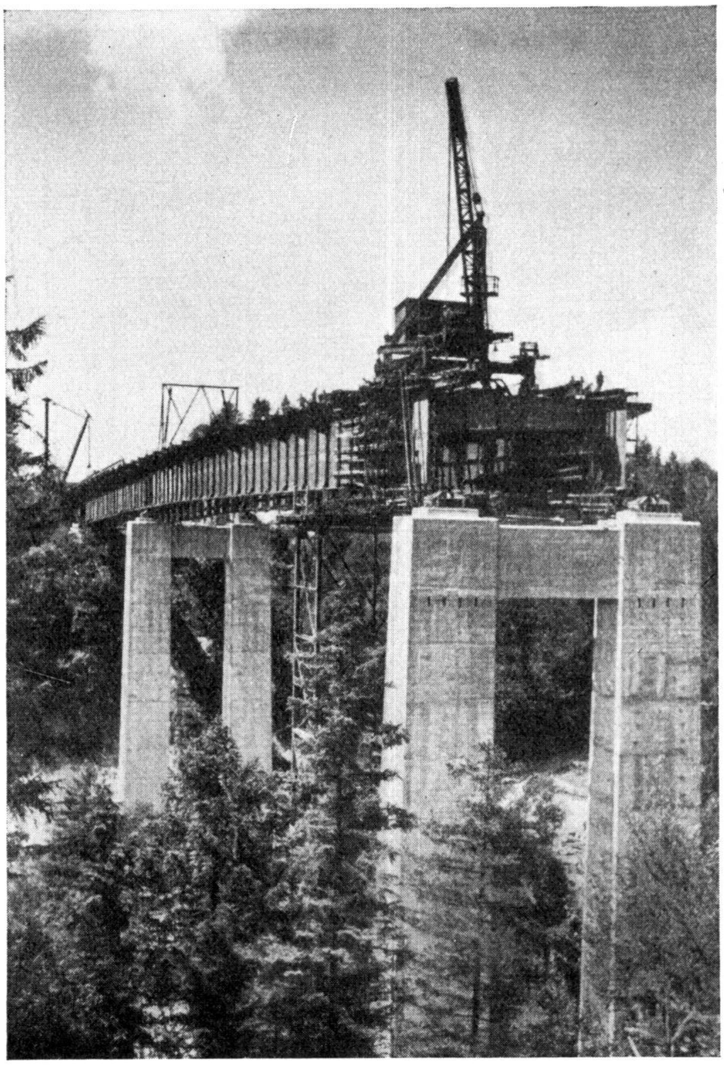


Fig. 11.

vertical connections, with a few exceptions, occur at intervals of 12 m. To improve appearance, the horizontal Z. 18 stiffeners are applied to the inner side of the main girders, so that only the vertical stiffeners are visible from outside. The flanges of the main girders (Fig. 14) are each composed of

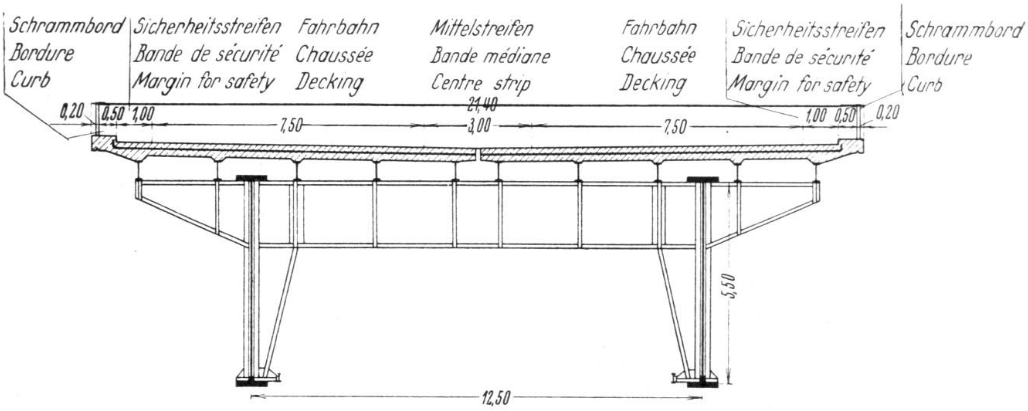


Fig. 12.

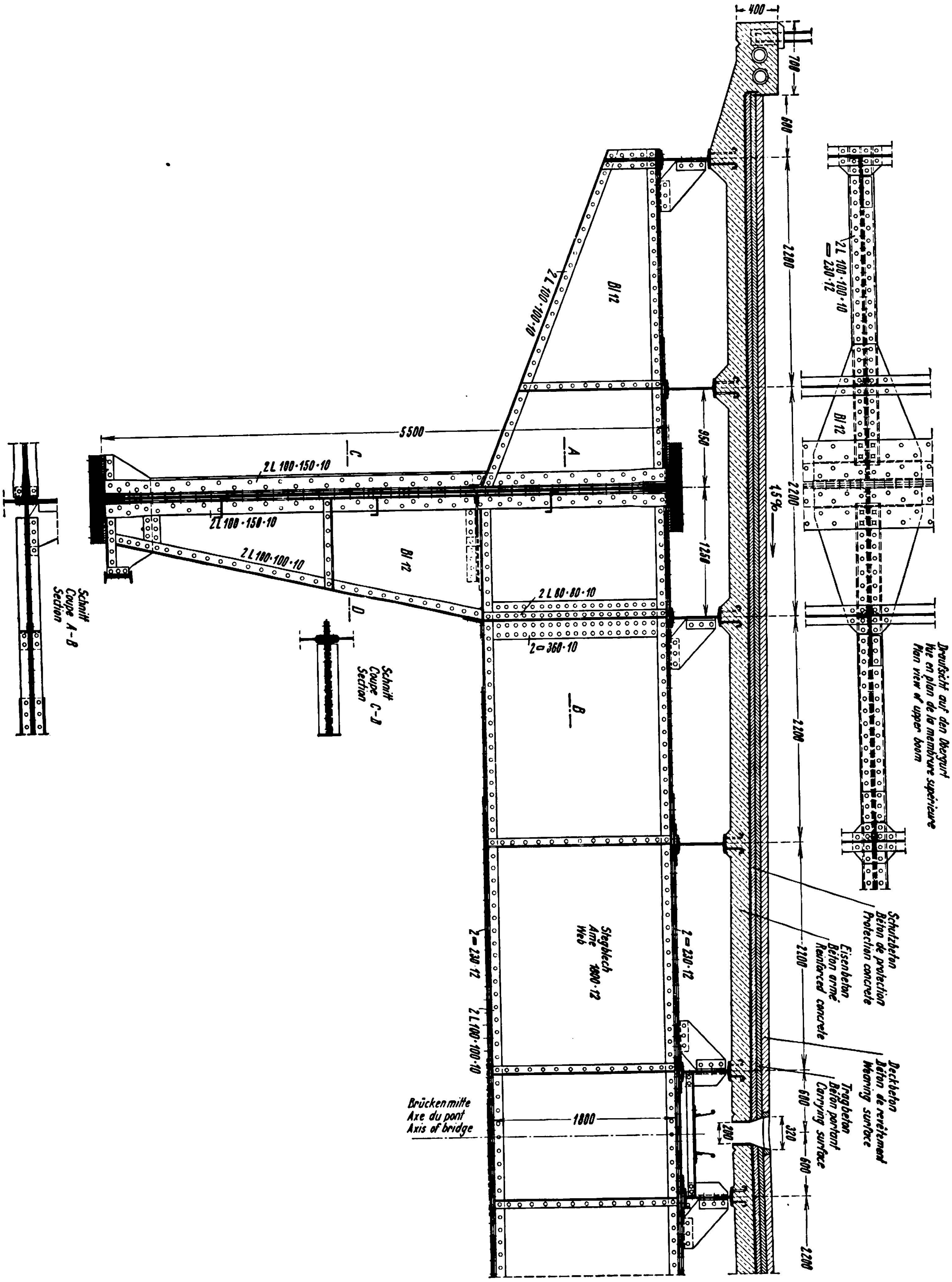


Fig. 13. Normal section of superstructure

two 200 × 200 × 20 angles and 850 × 20 flange plates, the number of which is suited to the flow of bending moment lines.

Special supports or brackets are provided at the points of support to facilitate the application of jacks.

In order to enable those parts of the superstructure lying beneath the decking and high above the bed of the valley to be maintained in good condition, an inspection carriage was installed between the main girders, running the whole length of the structure.

The main girders and cross girders are of St 52, the remaining parts of St 37.

The great height of the bridge (see Fig. 11) necessitated the application of the free cantilevering system of erection.

Example 2.

In this example the moderate height of only 43 m above the bed of the valley, and also the fact that the topographical conditions at the site were favourable, enabled the 365.4 m long viaduct under consideration to be divided into a larger number of spans. There are 7 spans in all, the largest being the middle bay with a length of 63.8 m, decreasing to 40.6 m at either side (Fig. 15).

The decking, with centre drainage, is carried by arched plates. These are provided with flat bars and diagonals which ensure good binding with the concrete filling (Direktor bei der Reichsbahn Schaechterle: Bautechnik 1934, p. 564. Neue Fahrbahnkonstruktionen (New Types of Decking). The arched plates rest on longitudinal I 45 girders spaced at intervals of 2.37 m. The erection bracing necessary in the free cantilevering system of erection is situated in the plane of the lower boom of the cross girders. Outside the main girders, in the plane of the cross girders, i. e. at intervals of 5.8 m, are arranged brackets with a cantilever of 3.3 m which carry part of the decking (Fig. 16). The tensile forces set up in the brackets are transmitted over the top flange of the main girder to the flange of the cross girders.

The flanges of the main girders are composed of ⅃L $200 \times 200 \times 16$ and flange plates 700×16, the number of which is suited to the flow of the moment line.

The extremely slender steel columns contribute very effectively to the pleasing appearance of the lengthy structure. They are composed of frames (Fig. 17) of hollow box section. The columns of the frame taper off towards the bottom. The brace, 3.2 m high, is also hollow and is provided with man-holes giving access to the interior; these can be entered from the inspection carriage. The web of the bracing was designed so high to allow the construction of a particularly rigid frame corner. The peculiarity of the soil, which is prone to slipping, is counteracted by the construction of two separate cylindrical piers for each frame column. These oppose the least resistance to any shifting of the upper layers of soil that may take place.

Example 3.

In the five-piered bridge of which a general view is shown in Fig. 18 the reinforced concrete decking slabs rest movably on the cross girders (Fig. 19). In each bay a special bracing is provided to take up and transmit the forces caused by braking. Horizontal forces are absorbed by a horizontal brace arranged in the upper third of the cross girders. The bottom flanges of the main girders are held at their sides by the portal-like cross girders.

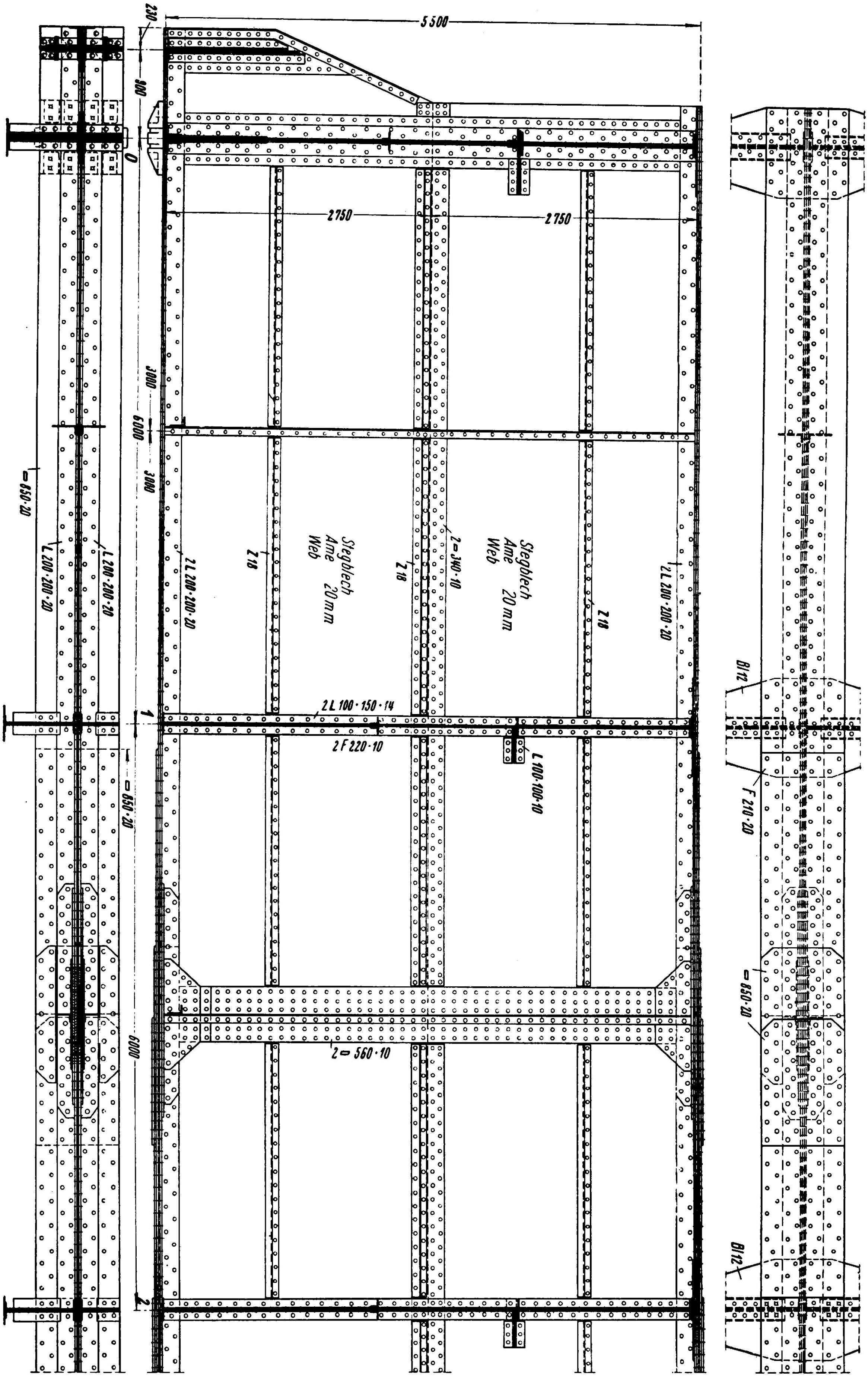


Fig. 14. Main girder between cross beams 0 and 2. Interior view.

IV. *Lattice Bridges.*

In addition to the solid-webbed structures already discussed, a large number of latticework superstructures have also been built in Germany in recent years. The most noteworthy are the following! —

Example 1.

The beam shown in Fig. 20 is continuous over two spans. Its total length is 292 m, its height 16.5 m. The latticing used is a pure strut system, i. e. one of diagonals placed zig-zag and without stays. This arrangement looks extremely effective when the four main girders of the two parallel bridges (road bridge and railway bridge) are viewed from the side. The distance between the two beams of each pair is 10 m, that between the adjactent beams in the

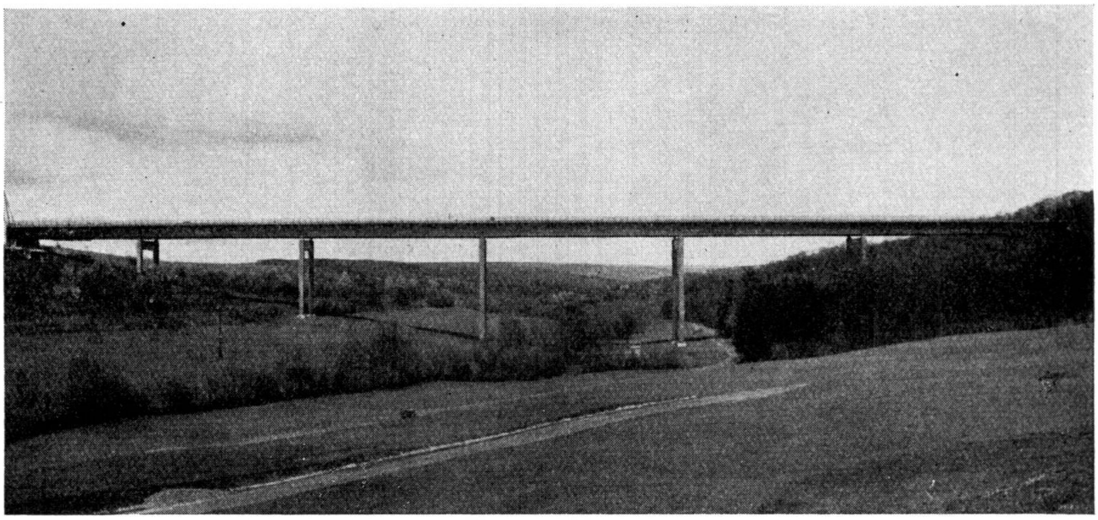


Fig. 15.

middle 4 m. In spite of the fact that when seen from an oblique angle the diagonals seem to cross and recross, the whole structure gives a restful impression, the reason for which is to be found in the lack of uprights. One peculiarity of this system as against constructions having uprights in the vertical plane of the cross girders is illustrated in Fig. 21, showing a constructional detail of the railway bridge. In a system with uprights the restraint moment emanating from the deflection of the cross girder is transmitted to the uprights. In the railway bridge, which has no uprights, other arrangements had to be devised to take up this restraint moment. It is transmitted to the vertical diaphragm situated at the connection of the cross girders by means of a corner web stiffener 2.99 m high. The diaphragm in turn transfers it to the gusset plate of the lower wind bracing and to the traverse at the height of the upper corner of the stiffener. This traverse is connected to the webs of both diagonals. Here the reaction at the support of the traverse is passed on to the diagonals, which conduct it to the upper wind bracing and, with the assistance of the horizontal diaphragm of the lower boom to which the webs of the diagonals are connected, to the lower wind bracing.

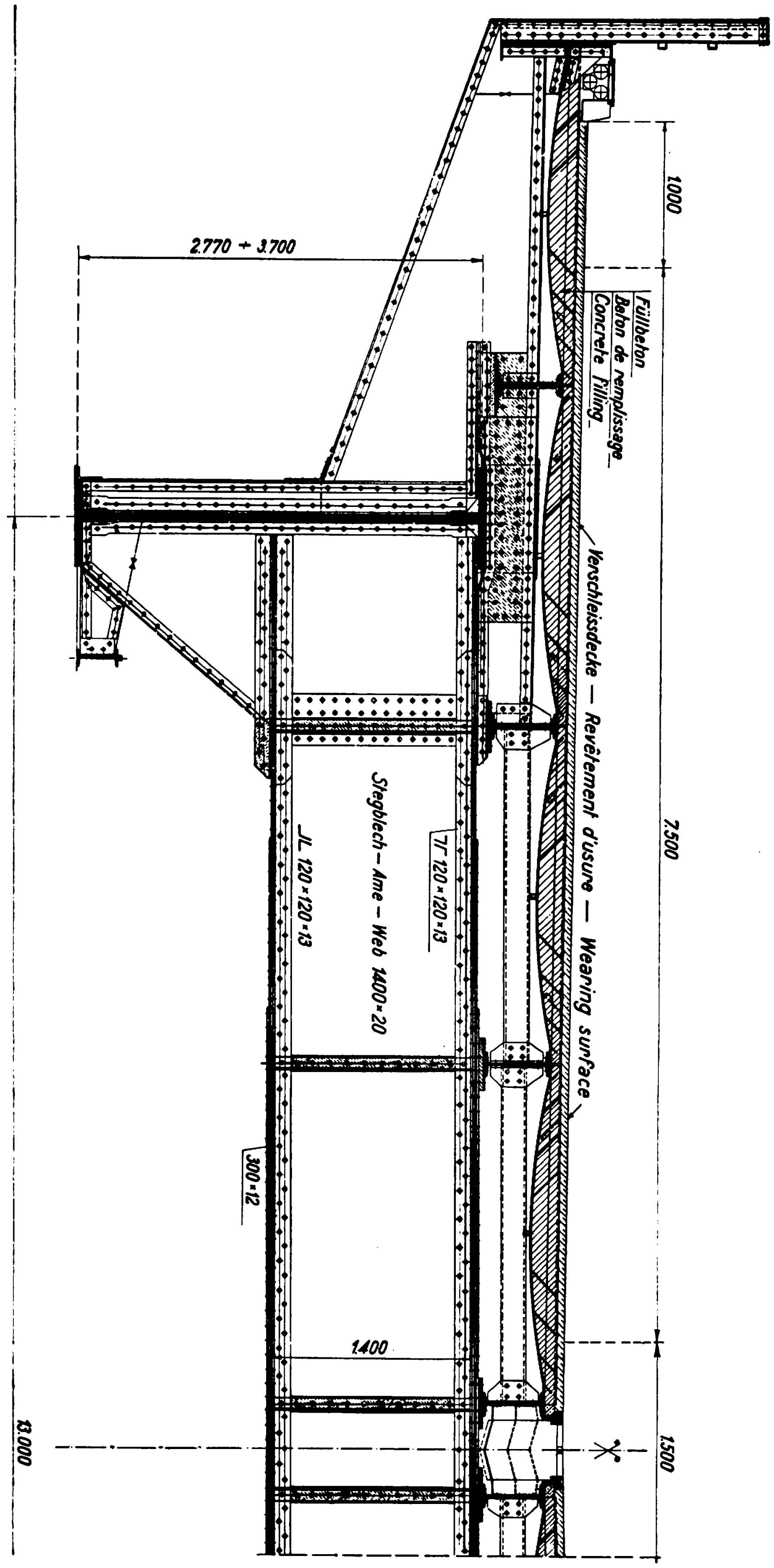


Fig. 16.
Normal cross beam.

Example 2.

Fig. 22 shows a structure of the same type and with a total length of 456.9 m. Its three spans measure 212.2 m, 66 m and 178.7 m respectively.

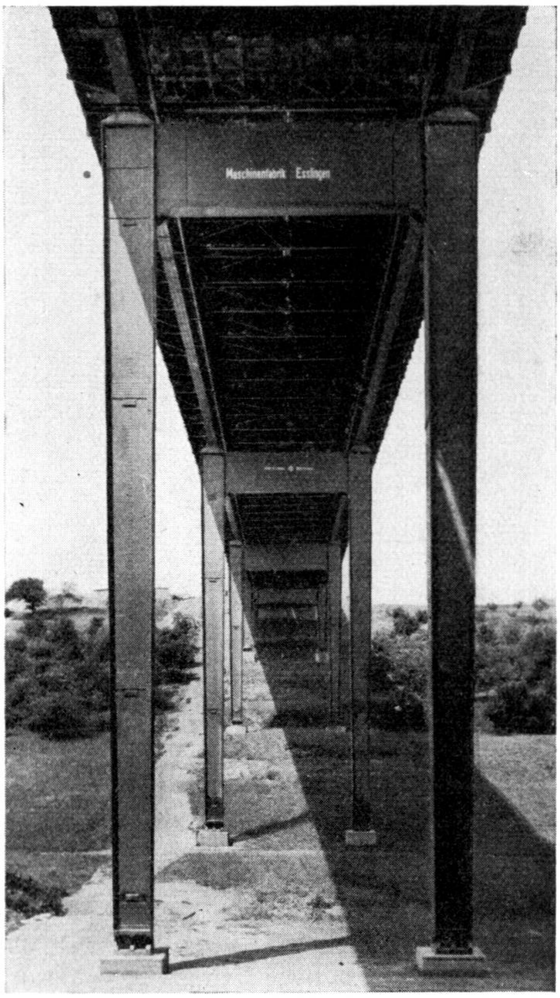


Fig. 17.

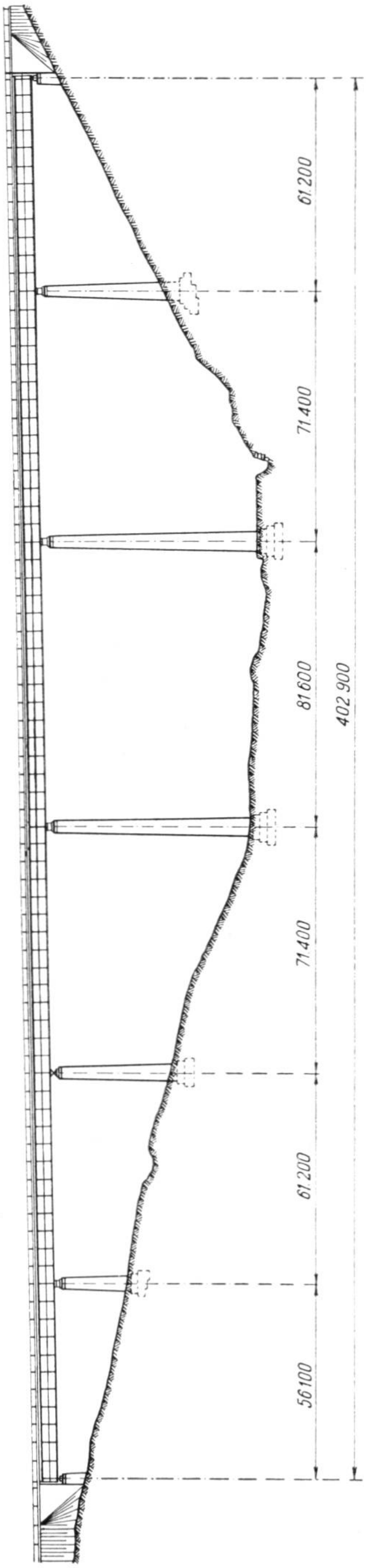


Fig. 18.

The two middle piers at either end of the 66 m span are built on an islet in the river. The main girders are 16 m high. Between them there is a decking 8.5 m wide, on each side of which runs a 2 m wide footpath. The main girders are of St 52, the other members of St 37. The surface of the roadway is formed by a rolled in layer of asphalt 6 cm thick, based on buckle plates filled with concrete. The restraint forces of the upper wind bracing, which is formed

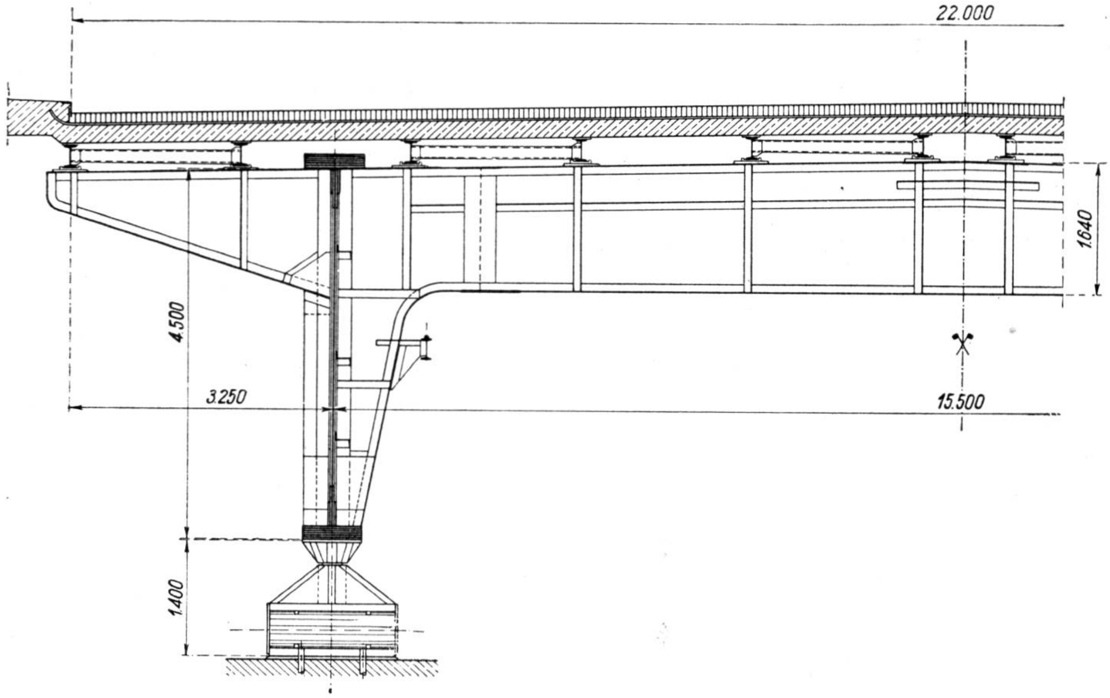


Fig. 19.
Cross beam at point of support.

of crossed bars, are transmitted to the abutments and piers by the portals situated in the planes of the diagonals.

Example 3.

Another structure worth considering is a river bridge on one of the German State Arterial Motor Roads. Its total length is 456 m, the span across the actual river being 130 m.

Before the system was finally decided upon detailed investigations were carried out with a view to ensuring proper harmony between the whole arrange-

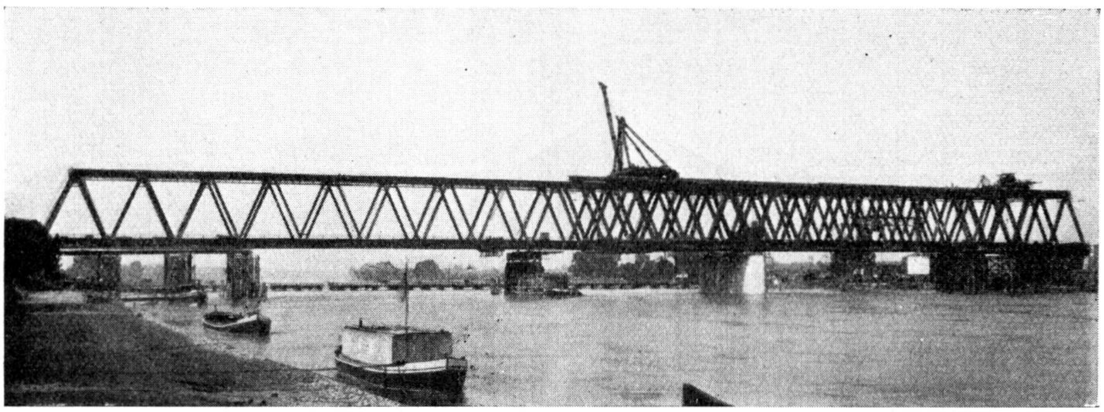


Fig. 20.

ment of the bridge and its surroundings. Owing to lack of space, the results of these investigations cannot be gone into here, for which reason we would like to refer to what has already been written on the subject (Weiß, Bautechnik 1935, p. 473).

The project as executed comprises a continuous, riveted lattice girder over five spans with decking at level of upper chords. The height of the girder

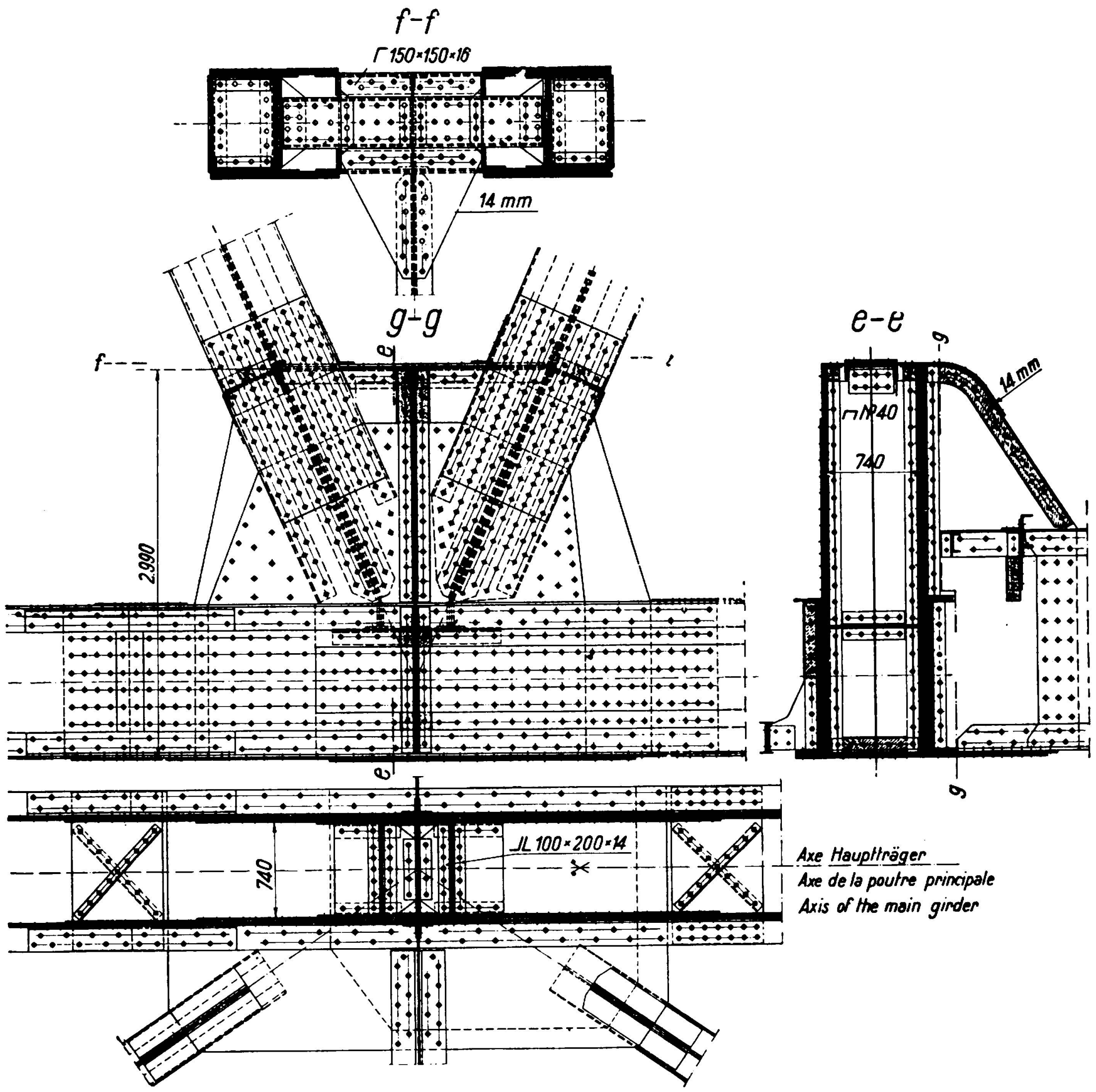


Fig. 21.

(5.2 m) is adapted to suit the length of the actual water span. The continuation on one side forms an overbridge over a road and railway track, and here the beams are welded plate girders. The decking of each of the two roadways is carried on a separate superstructure, as may be seen from the cross section shown in Fig. 24. The decking, composed of a jointed reinforced concrete slab, is carried by continuous longitudinal standard sections. These are movably supported by the cross girders and lengthy cantilevers. On one side of the roadway there is a footpath, on the other a passage for cyclists. The distance between the axes of the main girders of each superstructure is

7.5 m. The outer arms cantilever to an extent of 4.25 m, and at their connection have the same height as the cross girders. The space between the two superstructures is covered over by a reinforced concrete slab which rests on girders having their supports on small cantilevers. The upper edges of the

Fig. 22.

cross girders are connected with those of the main girders, so that there was no difficulty in transmitting the tensile forces in the upper flange of the large cantilever arm over the top of the main girders by means of tension straps.

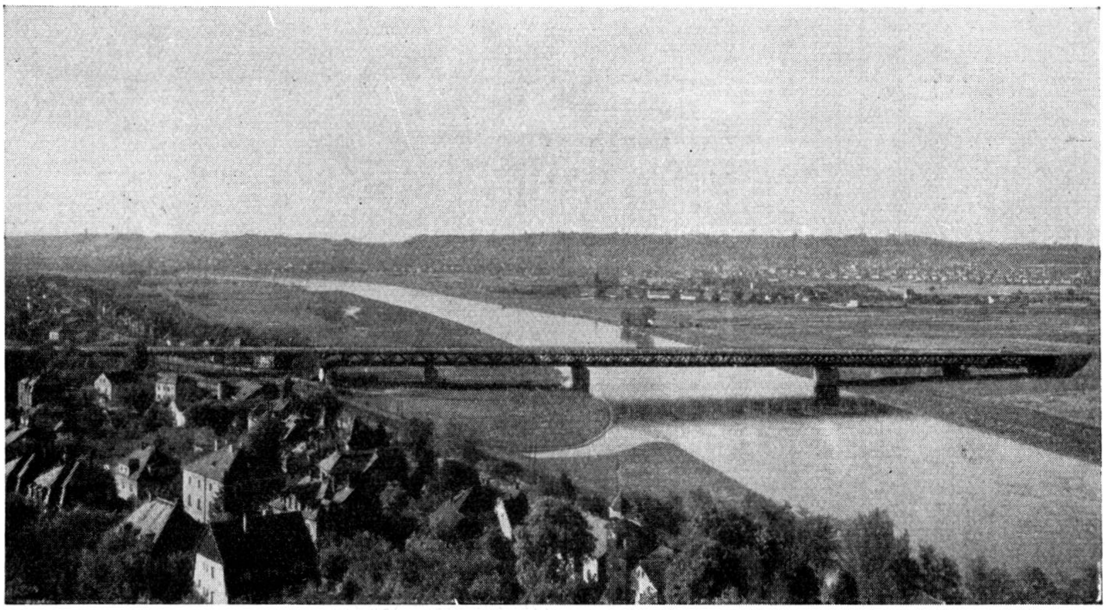


Fig. 23.

In the planes of the upper and lower flanges braces are provided to take up the lateral forces. This bracing, like the main girders, is composed of girders on six supports. The upper bracing transmits its restraining forces to the supports via stout portals (Fig. 25). In the solid-webbed, welded superstructures of the approach bridge (Fig. 26) the distance between the main girders is greater than in the case of the river section. The object of this arrangement was to reduce the additional forces set up in the brackets owing to the smaller effective spans, by shortening the cantilever arm of the brackets.

Example 4.

Fig. 27 illustrates a river bridge whose length far surpasses that of any of the structures discussed in the foregoing. The middle span of this bridge is 250 m long, and on each side of it there is a secondary span of 125 m. Six flood bridges of lattice girders each with a span of 45 m, and an 87 m long transitional structure link up via the approaches with the roads of the district. The total length of the whole construction between the abutments is 857 m. As it was stipulated that from any point of the bridge an open view of the river

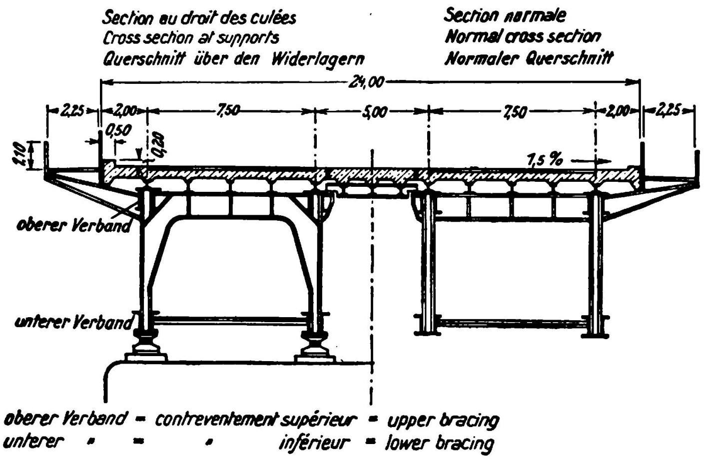


Fig. 24.

and both its banks should be obtained, the type chosen was that of a cantilevering bridge. Over the approach bridges the roadway has an up-gradient of 1: 60, and over the spans adjoining the central section a rise of 1: 125. This enables the specified clearance of 9.1 m above the highest navigable level of water to be attained for shipping. The two portals have a height of about 40 m above mean water level. The total width of the bridge between the railings is 19.5 m, of which 11 m is allotted to the roadway, 2 × 1 m to the two paths for cyclists, and 2.15 m for each of the two footpaths, which run outside the main girders.

Example 5.

In conclusion, the longest-spanned lattice bridge in Germany should be mentioned (Fig. 28). The lattice girder with rhombic arrangement of members is continuous over two spans, the main span being 256 m and the secondary span 154 m. The height of the girder is 24 m. Flood and approach bridges link up on both sides with the roads of the district. The total length of the structure is about 756 m. Between the main girders runs a roadway 12 m wide, on each side of which there is a 1.5 m wide path for cyclists. A can-

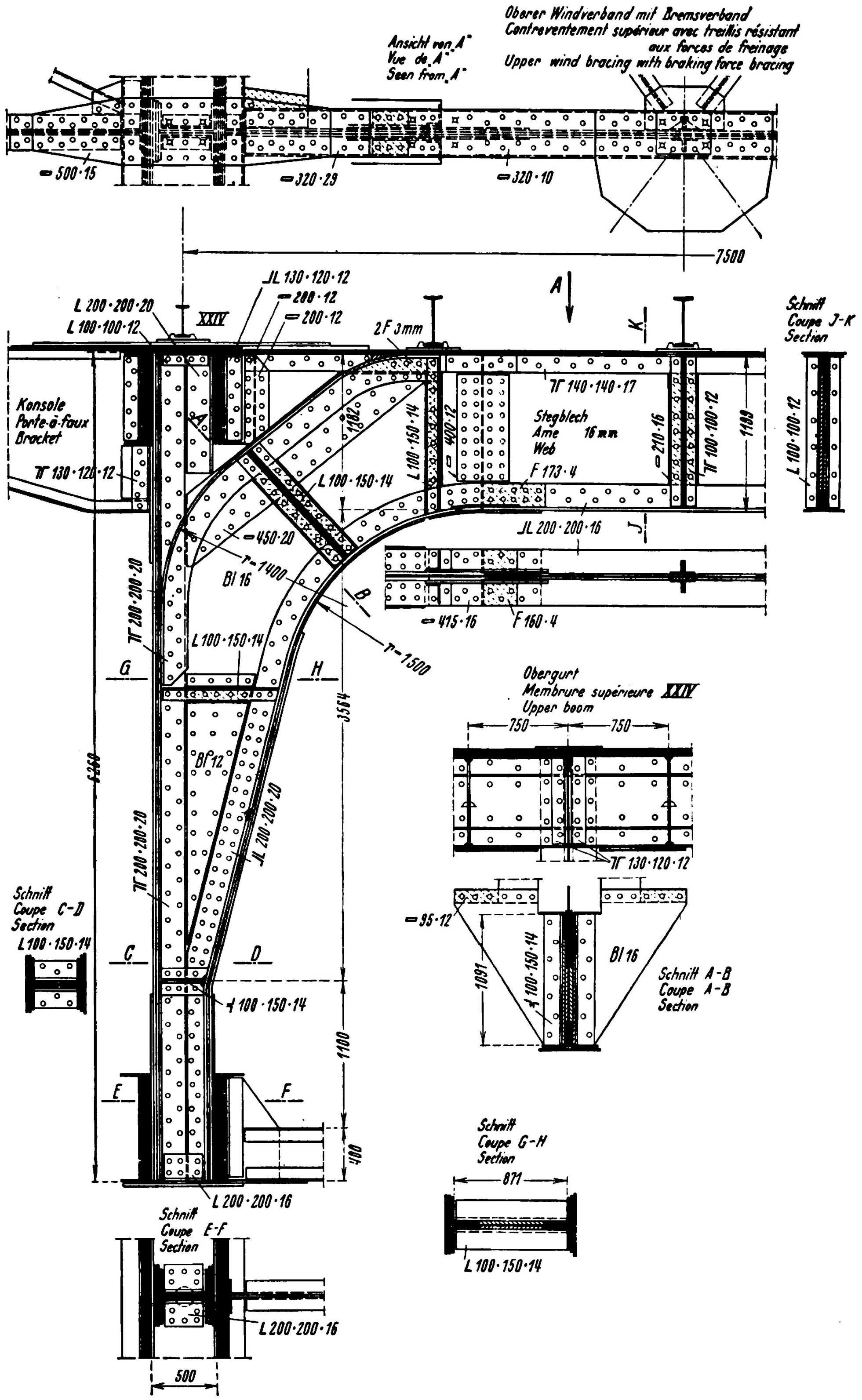


Fig. 25.

Section of portal above piers II and III.

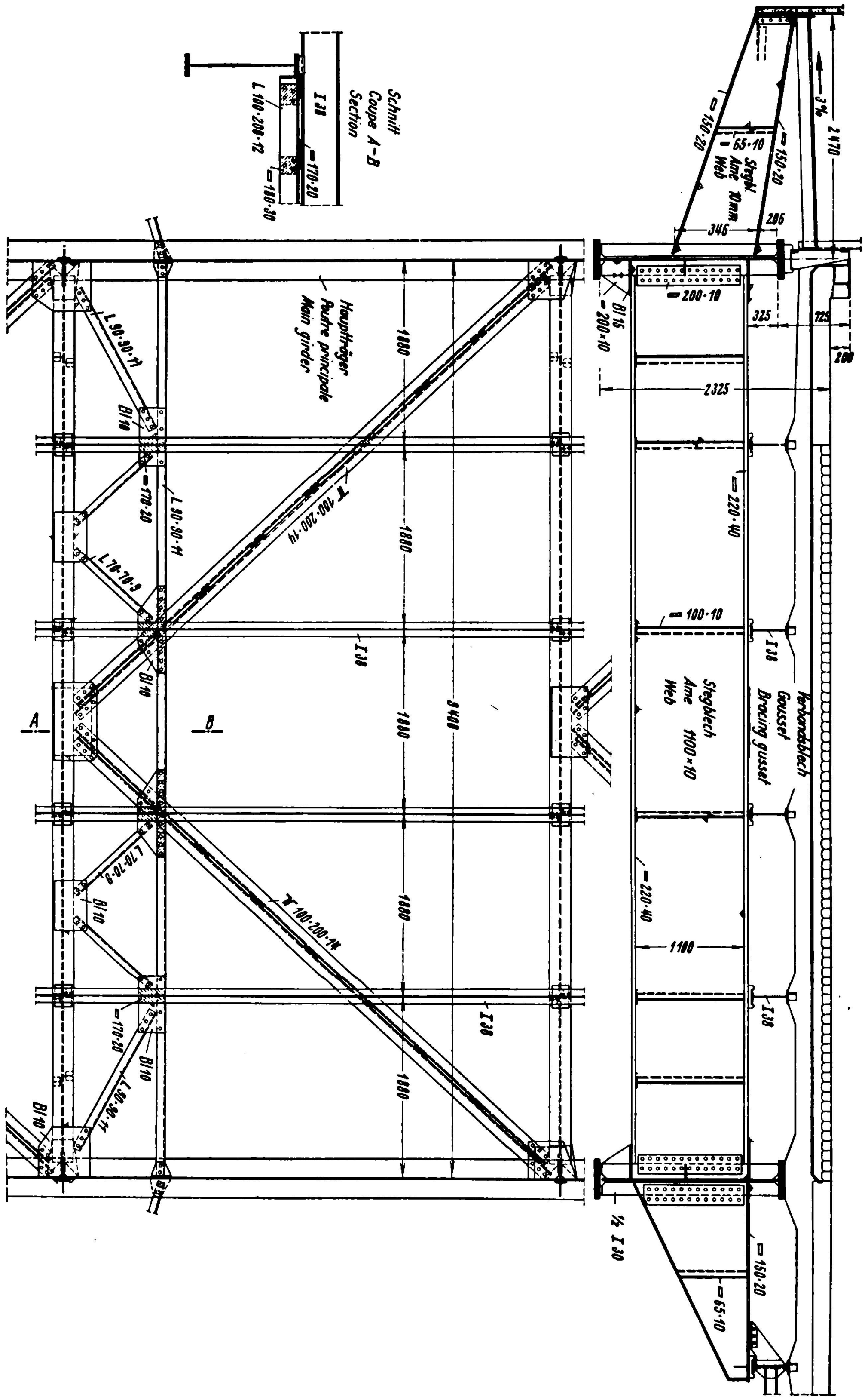

Schnitt
Coupe A–B
Section
Hauptträger
Poutre principale
Main girder
Stegblech
Ame 1100×10
Web
Verbandsblech
Gousset
Bracing gusset
2325
1100
3%

Fig. 26.

Fig. 27.

tilevering footpath, 2.75 m wide, runs along the outside of each main girder. In order to reduce the weight of the decking, the bays between the principal panel-points of the rhombic lattice are divided up by posts which transmit their

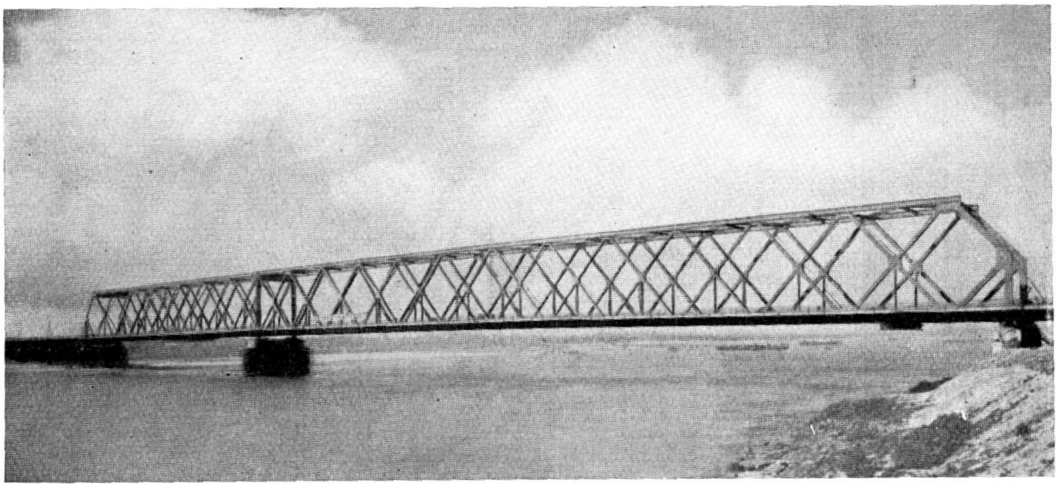


Fig. 28.

forces at the intersection point of the diagonals. The horizontal forces are absorbed by two braces, the upper of which transmits its restraint forces via the two end portals and that over the central pier to the supports.

Summary.

Many new steel bridges have been built in Germany in recent years. In contrast to former times, when such constructions were planned in the first line with a view to purely economic and technical efficiency, today the arch-

itectural standpoint is given due consideration, and new bridges are required to harmonise with their natural surroundings.

When discussing the frames of the bridges dealt with in this paper, reference is made to their harmonious lines and practical detailing.

Three examples are given of large viaducts constructed under the German Arterial Motor Road scheme. With the exception of their actual construction, these solid-webbed bridges, all of which are supported on concrete piers or steel columns, are alike.

Further examples of the use of steel are given in the description of a number of lattice bridges of various types.

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VIIa 8

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.

Dr. Ing. F. Schleicher,
Professor an der Technischen Hochschule, Hannover.

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.

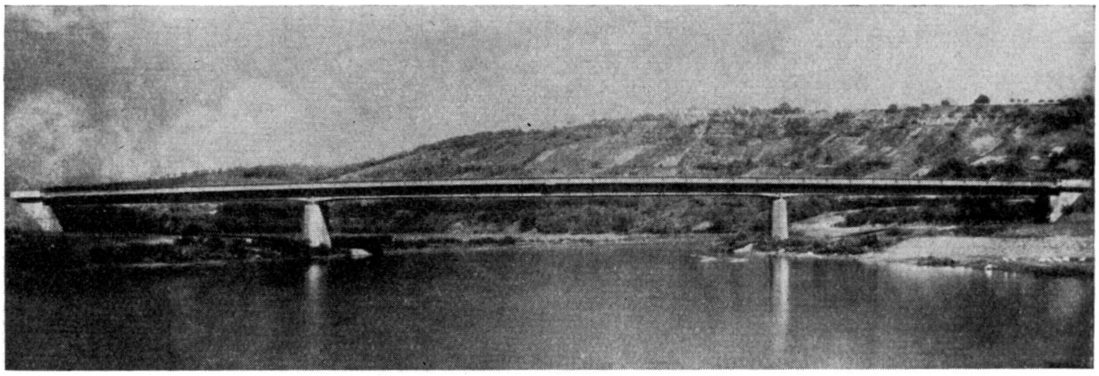

Fig. 1.
Neckar Bridge at Obrigheim-Diedesheim (central span of 90 m, 1936).

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

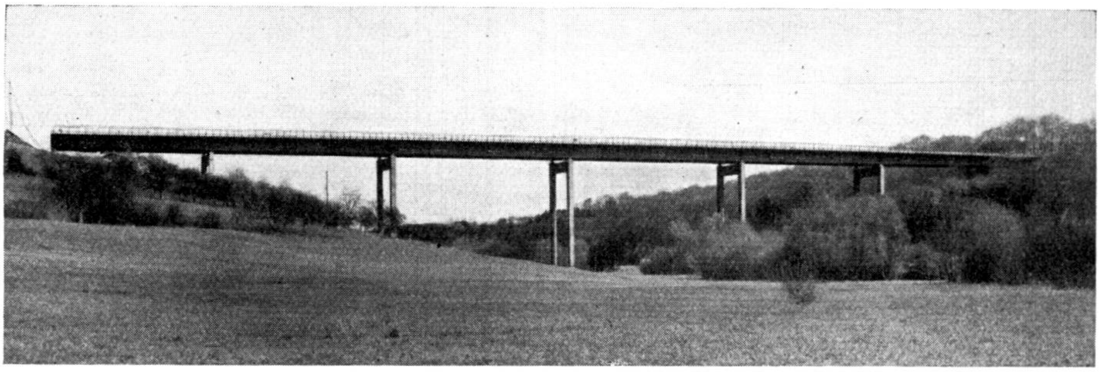

Fig. 2.
Sulzbach Viaduct near Denkendorf (largest span 63,8 m, 1936).

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.

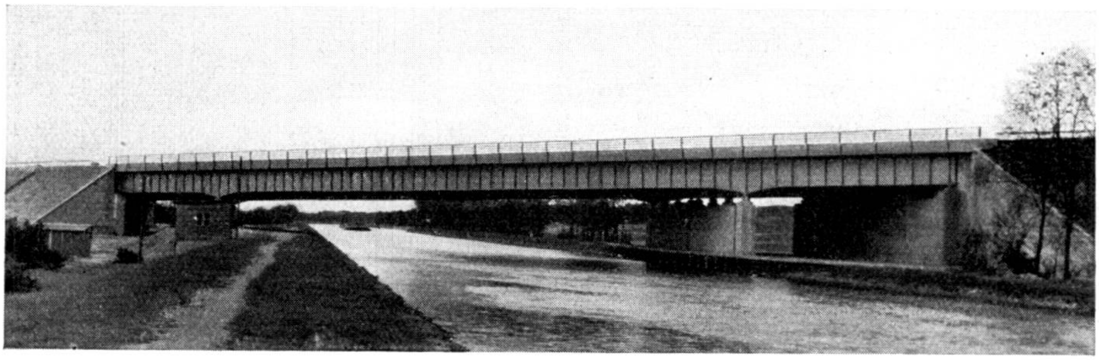

Fig. 3.
German State Arterial Road Bridge over the Hohenzollern Canal near Finowfurt.

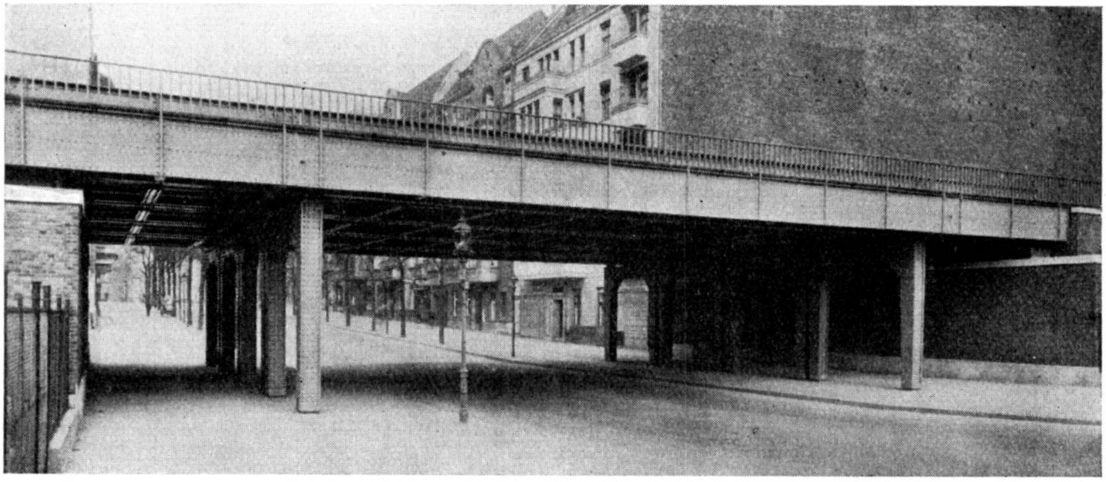

Fig. 4.
Schlüterstraße Overbridge Berlin.

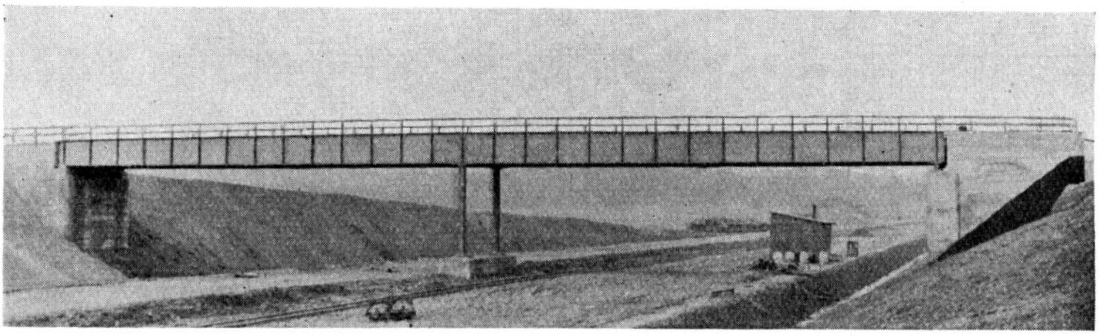

Fig. 5.
Overbridge, German State Arterial Roads.

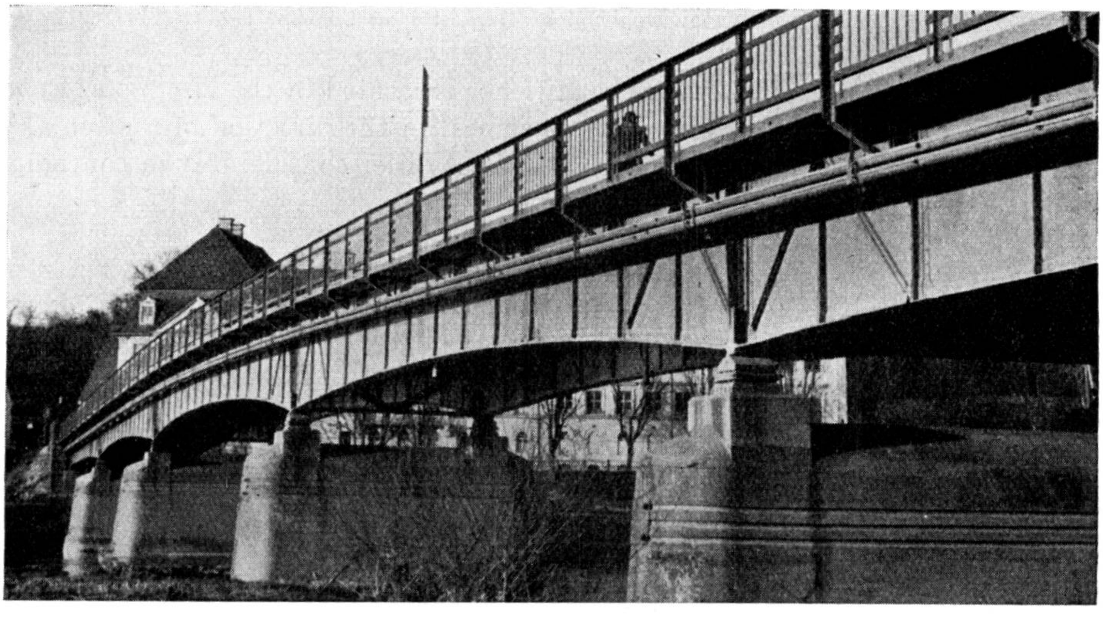

Fig. 6.
Road Bridge over the Danube near Donauwörth (largest span 32 m, 1876).

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.

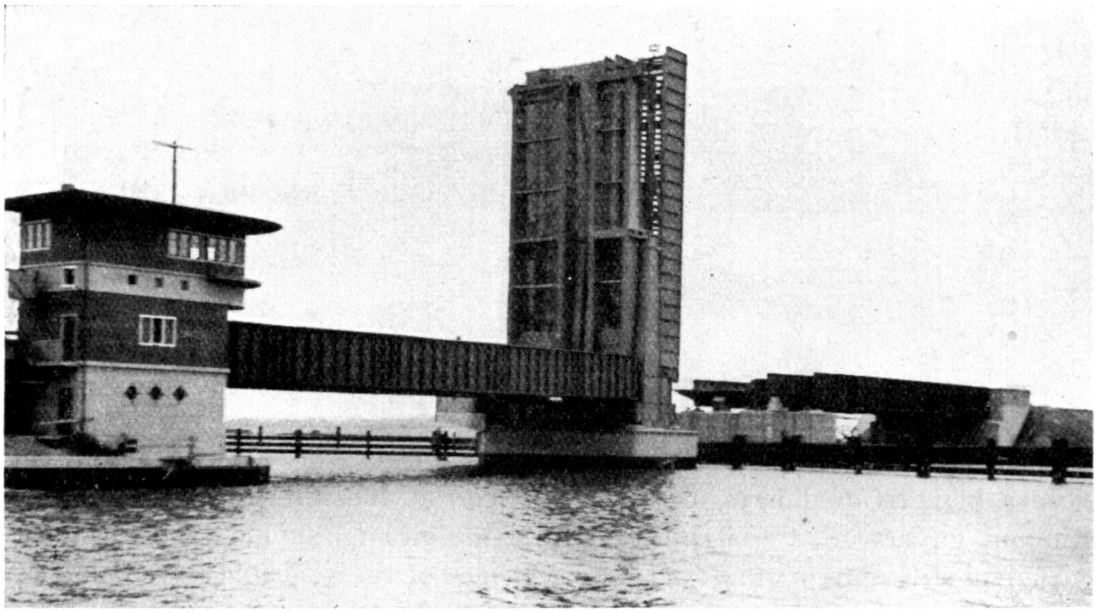

Fig. 7.
Ziegelgraben Bridge on the Rügen Dam.

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

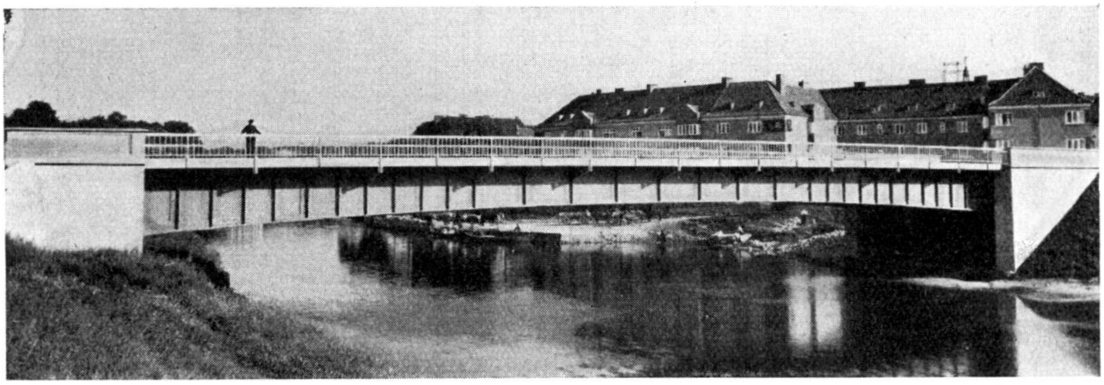

Fig. 8.
Angerapp Bridge, Insterburg (two-hinged frame, 1934).

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

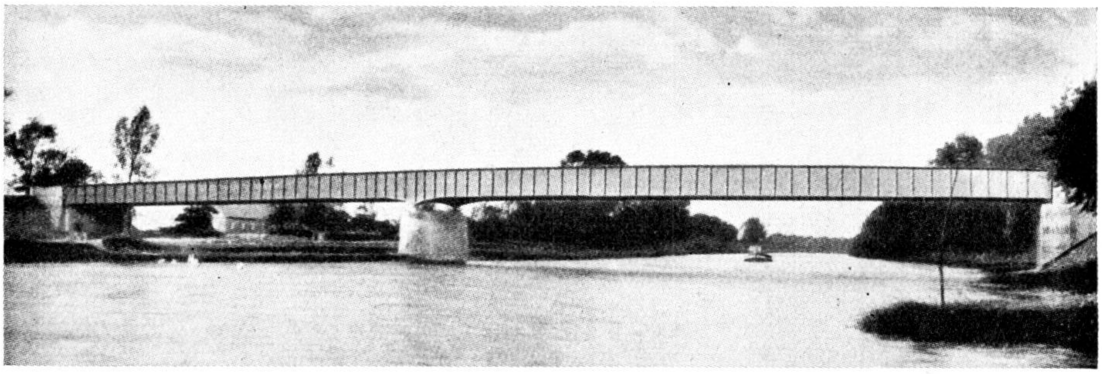

Fig. 9.
Bridge over the Ems near Steinbild (span 57,4 m, 1935).

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.

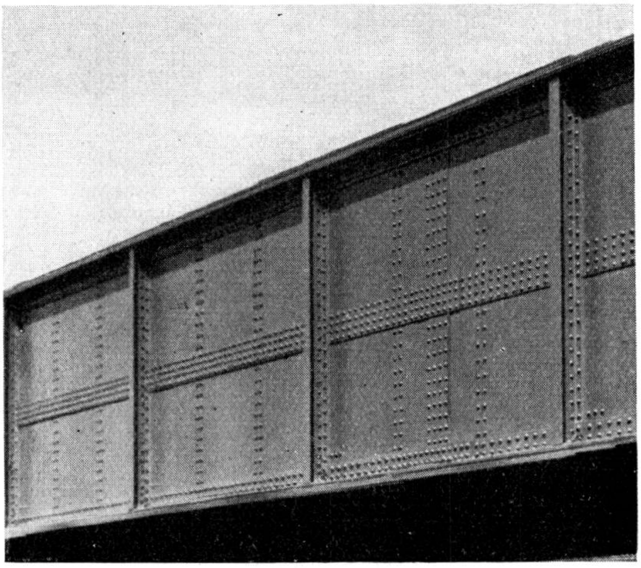

Fig. 10.
German State Arterial Road Bridge at Friedrichsfeld/Baden Railway Stn. (Middle span 61,6 m, 1935).

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.

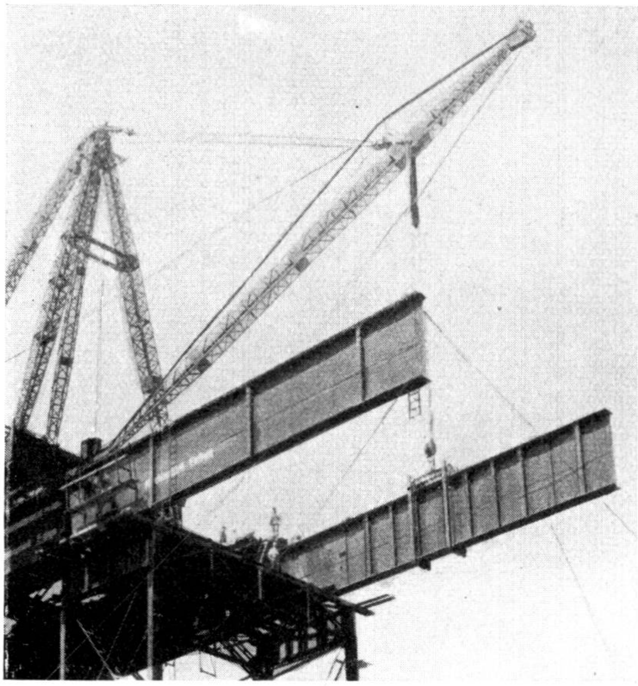

Fig. 11.

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

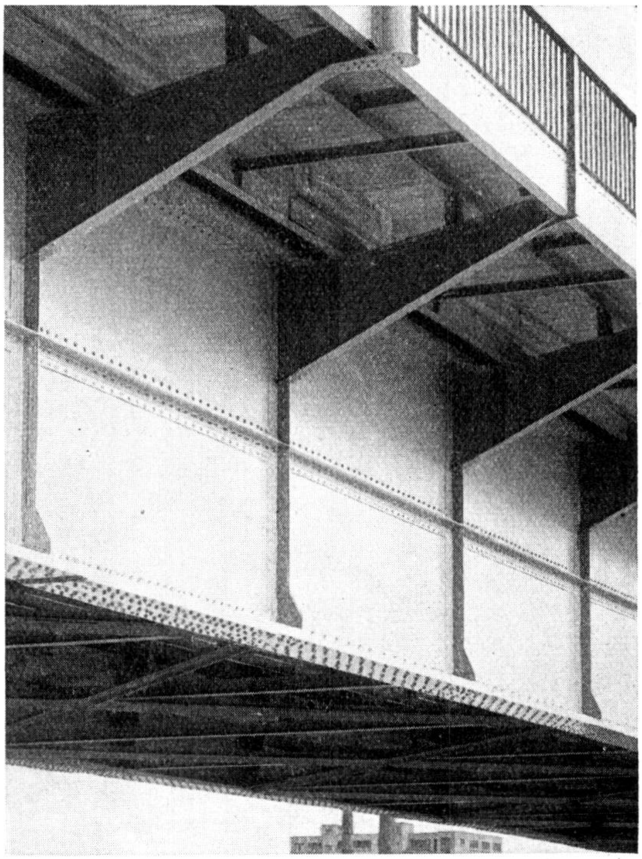

Fig. 12.
"Three Roses" Bridge at Basle (middle span 105 m, 1934).

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

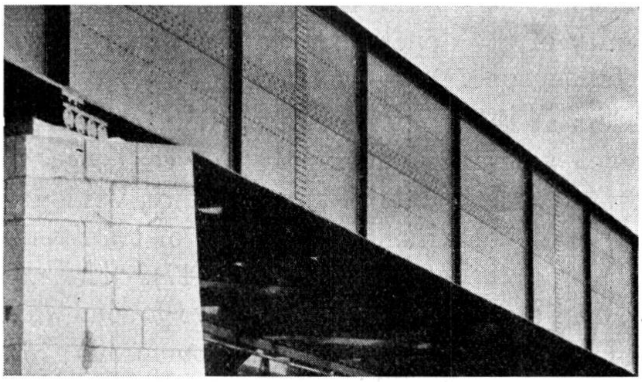

Fig. 13.
German State Arterial Road Bridge over the Main near Frankfort-Griesheim (largest span 70 m, 1934).

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

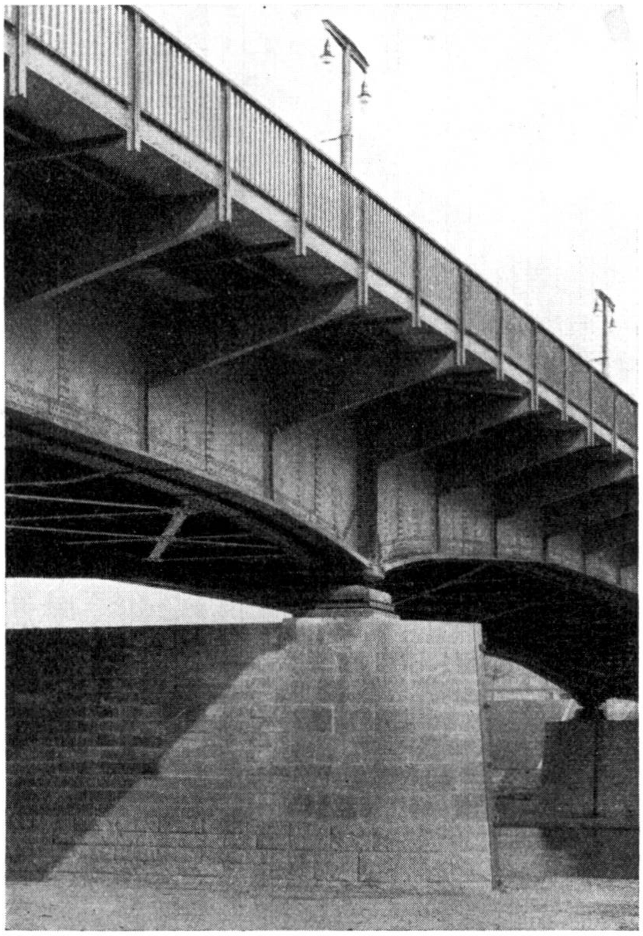

Fig. 14.
Adolf Hitler Bridge over the Neckar at Mannheim (86 m span, 1926).

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

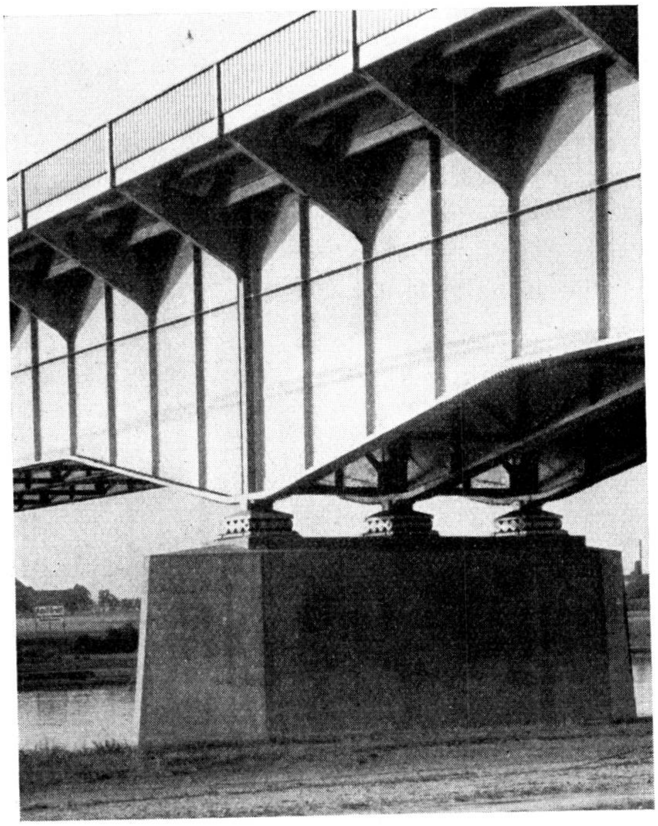

Fig. 15.
Kaditz Bridge over the Elbe near Dresden
(115 m span, 1930).

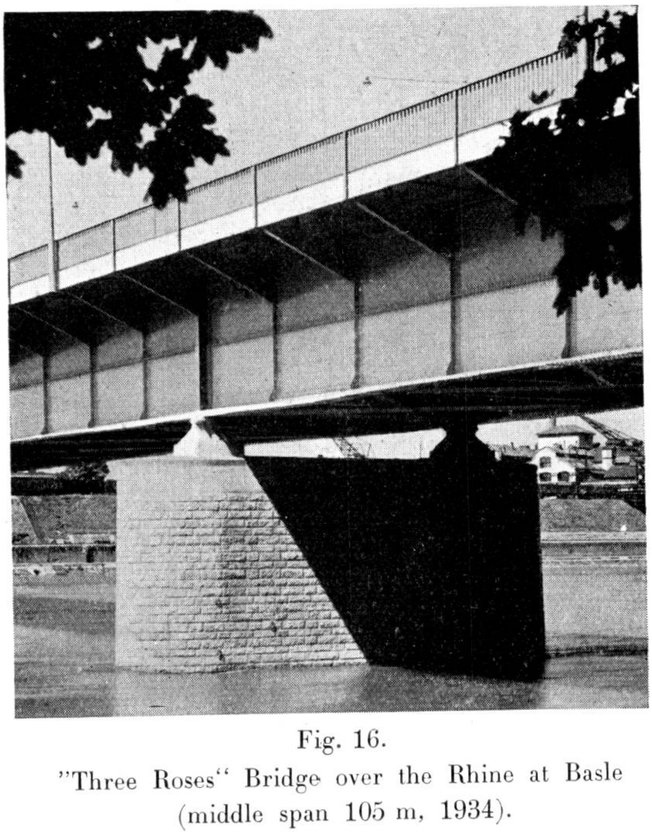

Fig. 16.
"Three Roses" Bridge over the Rhine at Basle
(middle span 105 m, 1934).

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.

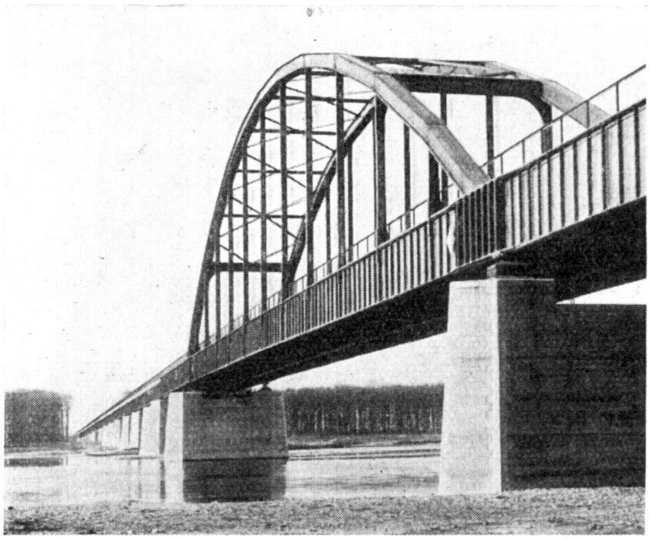

Fig. 17.
Bridge across the Elbe near Fort Dömitz (span 153,8 m, 1936).

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

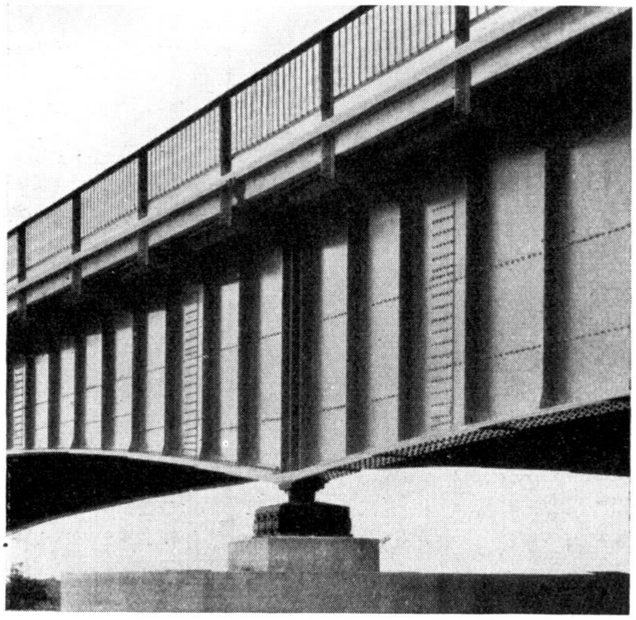

Fig. 18.
Leda Bridge near Leer (E. Friesland) (Middle span 63 m, 1933).

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

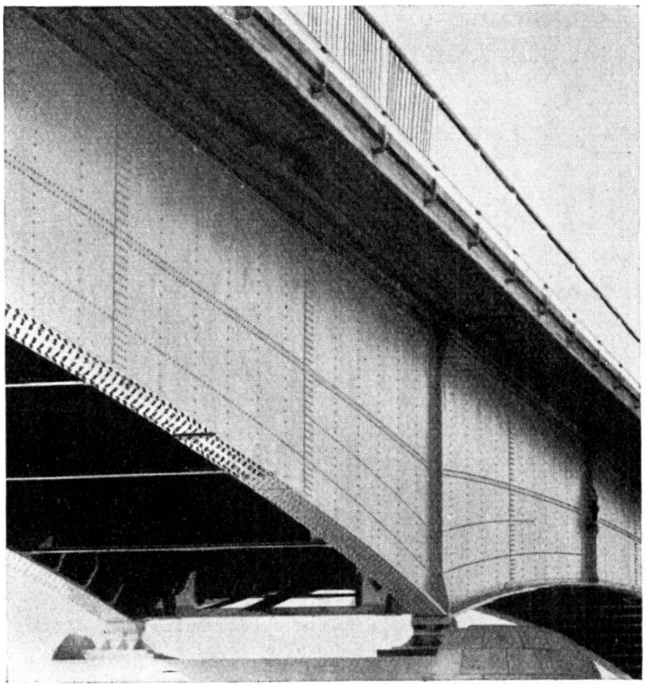

Fig. 19.
Oder Bridge near Poppelau (middle span 79,60 m, 1934).

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.

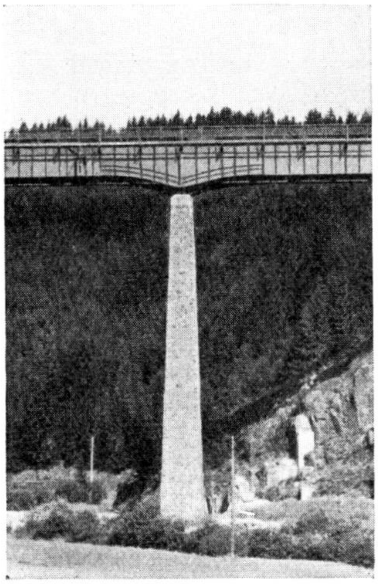

Fig. 20
Wettera Bridge
(58,8 m span, 1929).

	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.

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[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 ( ), 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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VIIa 9

Development of Structural Steel-Work.

Entwicklungslinien im Stahlhochbau.

Le développement des constructions de charpentes métalliques.

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In considering structural steel work, various methods may be adopted. The mode of representation generally found in technical literature — particularly in text books — is to classify the structures in accordance with the static action of their members; consequently girder structures are treated by themselves, likewise arched structures, frames, etc.

In contrast to this, the structures considered in this report will be considered with reference to the purpose for which they were intended. The historical development can then be easily followed.

As a rule the buildings treated here are German ones; foreign buildings are mentioned only exceptionally to serve as comparison.

The descriptions are based mainly on details obtained from technical literature. The literary sources are always given; if particulars of a building have been given in several publications, only the most important one is mentioned.

Railway Station Halls.

Last year the German railway could celebrate its 100th anniversary. Within that period the size, weight and speed of the trains have increased enormously. This development, in so far as a continuous increase in size is concerned, was made also by the steel station halls only at the beginning.

Dimensions.

The older station halls consisted as a rule of simple trusses on fixed supports. The trusses are designed either as sickle-shaped lattice girders — Munich[1], erected 1876—82 or as arched plate girders with tie members for taking the horizontal thrust — Münster i. W.[2], Hanover[3], erected 1878—80. The supports of these halls consist either of masonry or cast-iron. The spans are moderate (20—40 m).

But the first wide-spanned station halls (Fig. 1), whose lattice-work arches stretched over several platforms, very soon came into existence. Some of these

[1] *Jordan-Michel:* Die künstlerische Gestaltung von Eisenkonstruktionen 1913, 2nd Vol., p. 44 (The aesthetic side of steel constructions).

[2] *Jordan-Michel:* p. 48.

halls, arranged chronologically according to the period when erected, are mentioned in the following table.

Table 1: Wide-spanned halls.

Railway station hall	Erected	Length	Span	Height	Literature
Berlin, Schles. Statin	1881—82	207	54.4	19.0	Foerster[3] p. 662
Frankfurt on Main	1886—87	186	56.0	28.6	„ p. 759
Bremen	1888—89	131	59.3	28.4	Jordan-Michel[1] p. 5[illegible]
Cologne	1890—92	254	63.5	24.0	Foerster p. 770
Dresden	1895—98	174	59.0	30.0	„ p. 764
Hamburg	1902—04	173	73.0	35.0	„ p. 794

Beside these large halls there are very often smaller adjoining halls, for example in the stations at Cologne and Dresden. In some cases, several large halls are erected beside each other; for example, the main railway station at Frankfurt-on-Main has three equally large halls.

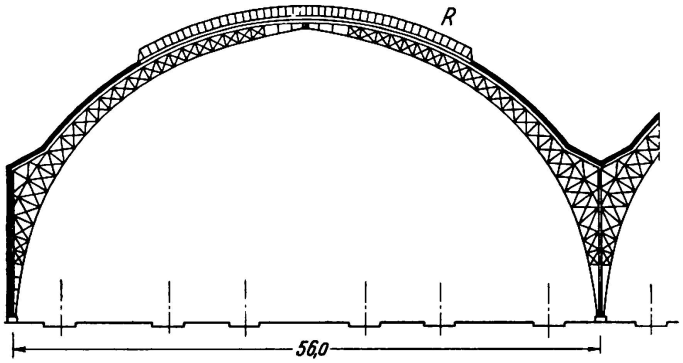

Fig. 1.
Main Railway Station
Frankfort on Main
R Skylights.

The lattice work trusses of these halls were as a rule arranged in pairs; every two trusses were united by members to form a space system. The distance between two trusses thus connected together amounts to 0.80 to 1.20 m.

These large wide-spanned railway station halls are undoubtedly very imposing in appearance; they also serve to give the arriving traveller his first impression of the town in which they stand. But, regarded from the practical point of view, they posses considerable drawbacks which became more and more evident in the course of time. Amongst these, in addition to the comparatively very high cost of building them, is the continual maintenance, which, because of the great dimensions of the halls and therefore also of the scaffolding, is extremely difficult and costly; for this reason it may very often be carried out less thoroughly than it should be.

Consequently, for a considerable number of years, the railway authorities have preferred to erect halls of medium and smaller spans, even at important and large railway stations.

In the course of time two types of halls — speaking very generally — have been developed:

a) the small hall, stretching over one platform and two lines of rails, with a span of about 20 m;

[3] *Foerster:* Die Eisenkonstruktionen des Ingenieur-Hochbaues (Steel in structural Engineering), 5th edition, 1924.

b) the medium-sized hall, with two platforms and 4 lines of rails within the hall, the span being about 40 m.

In the two following tables some small and medium-sized halls are given, arranged according to the date of their erection.

Table 2: Small halls.

Railway station hall	Erected	Leugth	Span	Height	Literature
Basle, Bad. Station	1911/12	305	24.0 resp. 20.0	11.5 resp. 11.0	Foerster p. 749
Karlsruhe	1912/13	—	21.5	13.0	„ p. 753
Oldenburg	1916	153	21.0	8.0	„ p. 754
Frankfurt o/O.	1926	178	19.35 resp. 20.75	9.47 resp. 9.76	Bautechn. 1926, p. 668
Halle o/S.	1934	103	22.75	8.8	„ 1935, p. 67
Düsseldorf	1934	—	20.5	8.3	„ 1931, p. 279 „ 1935, p. 68
Duisburg	1934	—	19.7		„ 1935, p. 68

Table 3: Medium-sized halls.

Railway station hall	Erected	Length	Span	Height	Literature
Metz	1907—09	165	32.6 resp. 42.6	18.0 resp. 22.0	Foerster p. 785
Leipzig	1911—14	204	42.5 resp. 45.0	19.7	„ p. 777
Königsberg i. Pr.	1928—29	178	37.0 resp. 43.55	13.67 resp. 15.62	Bautechn. 1928, p. 659
Liegnitz	1929	120	35.5	19.3	Bauing. 1930, p. 445
Beuthen O.-S.	1929—30	141	39.2	13.1	„ 1930, p. 846

The two medium-sized halls at Metz and Leipzig have still lattice-work trusses; those of the hall at Metz being single trusses, whilst the ones at Leipzig are double. The other medium-sized halls mentioned in the table, as well as all the small halls mentioned, have plate trusses.

Naturally both small and medium-sized halls may occur in one and the same station. For example, the Friedrichstraße Station[4], Berlin, has two halls beside each other, of which the small hall — for metropolitan trains — has a span of 19 m, whilst the other — for distant traffic — is a medium-sized hall with a span of 38.7 m.

In exceptional cases, large halls of great span are still built in modern times, especially when it is a question of replacing existing stations by new ones. As an example of this the new hall of the Schlesische Station[5] Berlin, may be mentioned, which, with a span of 54.35 m and a height of 18.41 m, has nearly the same dimensions as the old hall. However, instead of the former lattice-work double trusses, single plate trusses are now used.

Forms of the halls.

Most of the earlier, wide-spanned halls — Frankfurt-on-Main, Bremen, Cologne, Dresden — are arch shaped, somewhat as shown in Fig. 1; the halls of the main station at Leipzig are also of this shape.

[4] Bauingenieur 1925, p. 321.
[5] Stahlbau 1931, p. 292.

For the more modern small halls, the arch form proved to be unsuitable, since it is impossible to make full use of the light-space of the hall. The truss form therefore changed over from arches to frames. At first, the arch shape was retained for the upper part of the frames. For example, in the halls of the Badische Station at Basle and the station at Karlsruhe the upper parts of the frames form a practically circular arch, whilst in the station halls at Oldenburg they are somewhat more elliptical in form.

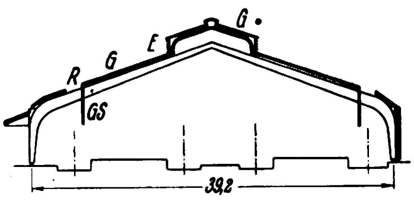

Fig. 2.
Railway Station Beuthen O.-S.
G glass, GS glass screen, E ventilation, R smoke escape.

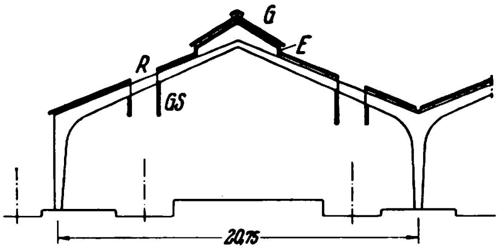

Fig. 3.
Railway Station Frankfort on the Oder
G glass, GS glass screen, E ventilation, R smoke escape.

The arched form of the upper part of the frames makes it necessary for the roof covering to be uniformly curved or intermittently bent. Consequently the line of the upper part of the frame soon began to be of straight roof-shaped form. The same development from the round shape to a sharp bend is to be seen also in the design of the corners of the frames (cf. Figs. 2—4). Of great influence on this has also been particularly the adoption of modern welding methods in construction (for example, the hall at the railway station at Halle-on-the-Saale).

It is also not without interest to note that this frame form with the upper part bent to the shape of the roof has also been adopted in some earlier lattice-work type halls; an example of this is the main railway station at Hamburg.

The new welded halls of the stations at Düsseldorf (Fig. 5) and Duisburg also show a frame form, but with intermittently bent roofing.

Lighting and ventilating.

The older, wide-spanned railway station halls had as a rule transverse saddle-shaped skylights (Fig. 1). By lifting the caps of these skylights, and also by raising the roofing over the double trusses, ventilation was provided for.

In place of the saddle-shaped skylights longitudinal rows of windows were adopted. The railway station halls at Leipzig may be cited as an example; there the skylights are arranged in steps in order to facilitate ventilation.

It can be easily understood, and experience has also shown, that the skylights get dirty rather quickly immediately below the ventilating openings. A strip of opaque roofing was consequently arranged at these spots, as, for example, in the hall at Beuthen (Fig. 2).

The railway stations hitherto mentioned have large or medium-sized halls. In the small halls, the question of lighting and ventilating was at first solved in the same way; the Badische Station at Basle may be mentioned as an example. A little later, corresponding to the example of the station halls at Ghent and Ostend[6] a continuous slit was provided over the tracks, and the smoke escaped at once through it without reaching the interior of the hall. At the platform side, the slits are bordered by a hanging glass screen, which protects travellers from any obliquely falling rain. One of the first German stations of this type is at Oldenburg.

Generally such a screen is arranged only along one side of the smoke slit, but in the station hall at Frankfort-on-Oder (Fig. 3) there are two to each slit.

A combination of the two types of ventilation is found in the medium-sized station at Beuthen (Fig. 2). The two outer tracks are ventilated by smoke slits, whilst a lantern type of superstructure is provided for the inner tracks.

Inclined glass surfaces have the drawback that they are easily rendered obscure by a heavy snowfall, and the panes are liable to get broken; experience has shown also that they become dirty much sooner than vertical skylights. The trend of development in all structures is to use exclusively vertical glass surfaces. The recent station halls at Halle on the Saale (Fig. 4), Düsseldorf (Fig. 5) and Duisburg have consequently only vertical glass surfaces.

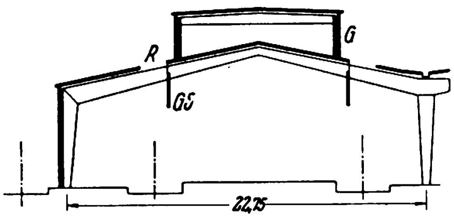

Fig. 4.

Railway Station Halle on the Saale
G glass, GS glass screen, R smoke escape.

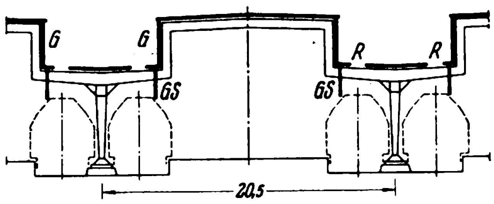

Fig. 5.

Main Railway Station Düsseldorf
G glass, GS glass screen, R smoke escape.

Of particular interest in this connection is a design which was at the time proposed for the Friedrichstrasse station in Berlin[4] (Fig. 6). The truss forms are certainly similar to those in the halls at Düsseldorf and Duisburg, but the position of the trusses with respect to the platforms, and also the solution of the question of lighting and ventilating, show essential differences. From this single comparison, the trend of development of railway station halls can already be seen clearly.

Halls for electric railways.

With the introduction of electrification, the point of view for the design of station halls has changed, the question of ventilation now recedes into the background.

The new halls of the Berlin metropolitan and circular railway, which is worked exclusively by electricity, cover together only two tracks with one platform situated between them; they belong therefore to the group of small railway stations.

[6] *Förster:* p. 734.

Table 4: Halls of the Berlin metropolitan and circular railway.

Station	Erected	Length	Span	Height	Literature
Westkreuz[7]	1928	158	21.6	—	Stahlbau 1930, p. 150
Janowitzbrücke	1932	142	14.0 to 18.0	5.35 and 8.25	P-Träger 1932, p. 77
Schöneberg	1933	160	19.1 to 23.6	9.1	Stahlbau 1933, p. 105

The cross-sections of these three halls are shown in Figs. 7—9.

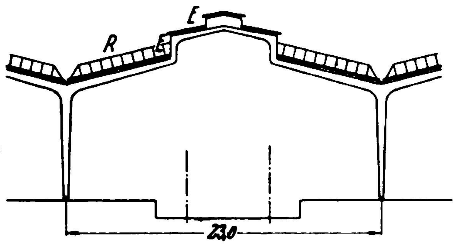

Fig. 6.
Railway Station Friedrichstraße, Berlin
(this proposal not executed)
E Ventilation, R Skylights.

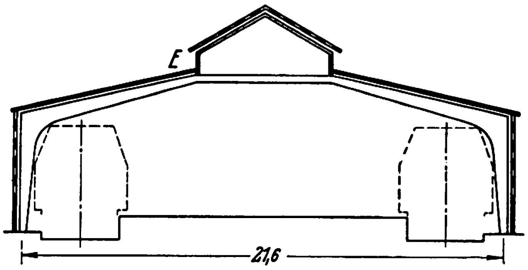

Fig. 7.
Railway Station Westkreuz, Berlin
E Ventilation.

The usual construction of such halls is plate frames with the upper part having a slight rise, as in the Westkreuz and Schöneberg stations. The adoption of the cross-section of the hall at Düsseldorf station when building the Jannowitzbrücke station will certainly remain an exception, since the reentrant corners, as long as there are not several halls adjacent to each other, are justified mainly by the necessity of leading the smoke away quickly.

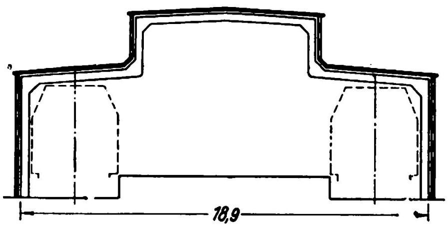

Fig. 8.
Railway Station Jannowitzbrücke, Berlin.

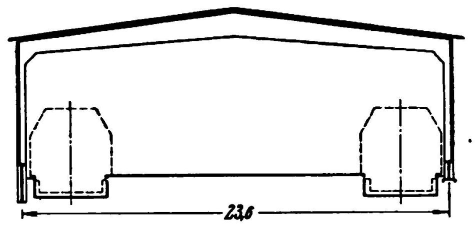

Fig. 9.
Railway Station Schöneberg, Berlin (section).

The hall of the Westkreuz station is lighted through the glazed side walls and, since the eaves lie only a little above the light-space profile, also through a glazed superstructure running in the longitudinal direction of the hall. The halls of the

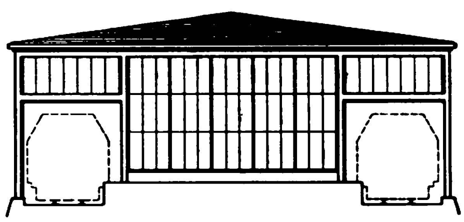

Fig. 10.
Railway Station, Schöneberg, Berlin
(Gable end).

Jannowitzbrücke and Schöneberg stations have only vertical glass surfaces. In addition, in the Schöneberg station the side walls are carried up as high that the light in the hall is ample, even when the trains come in.

[7] In publications this station is also designated „Ausstellung“.

For ventilating, slits in the walls or below the lantern on the roof, adjustable swinging sections in the glass windows prove sufficient.

At the ends, the halls can be closed if necessary down to the top of the train profile. As an example of this, Fig. 10 shows the gable wall of the Schöneberg station, which is glazed all over.

Exhibition and Sample Fair Halls.

The development of exhibition halls has been very variable. This is easily understood when it is considered that the requirements demanded from exhibition and sample fair halls may differ to an extraordinary degree. In addition, a number of other factors come into consideration, as, for example, the area of the site available, the amount of money which can be spent on the buildings, etc.; these factors have really nothing to do with the construction of the hall as such, but they nevertheless exert a considerable influence on the whole planning of the structures.

Dimensions.

In the same period as the wide-spanned railway station halls at Frankfurt-on-Main, Bremen, etc., came the large machinery hall of the Paris International Exhibition in 1889, which, with its three-hinged arches in iron of 111 m span, exceeded by a considerable degree any hall that had hitherto been erected. Also at the Chicago International Exhibition of 1892, the central hall was also erected with three-hinged arches, the span being about the same as that in the Paris exhibition, but the height somewhat greater.

Just as in the case of the railway station halls, this period of giant structures was very soon followed by a certain moderation. For exhibitions which last only a limited time, the buildings are often chosen intentionally of such dimensions that the halls can be adopted for other purposes after the close of the exhibition. A good example of this is the machinery hall[9] of the International Building Exhibition at Leipzig in 1913, which, after the exhibition was over, was taken down and re-erected at Kiel as a workshop hall.

Among permanent structures is the domed building of the exhibition hall[10] erected in 1908 at Frankfurt-on-Main. The middle part, which has a floor area of 67 $\times$ 54 m and is elliptical in plan, is bounded by two rectangular side halls of 49 $\times$ 29 m area. The whole hall is free from inner supports. In this connection the German machinery hall of the Brussels International Exhibition in 1910[11] should be mentioned. Even if the span of this three-bay hall with a total width of 40 m should appear very modest in comparison with the large halls first mentioned, the really great height of the structure relatively to the span is very striking; in the middle of the hall the clear height is 22 m.

The permanent exhibition and sample fair halls erected in Germany since the war can be divided roughly into two large groups:

a) the large halls, which span the whole space without any inner supports. In this group are also to be placed the halls with several bays, where the span of the middle bay is a multiple of the spans of the side bays.

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[9] Bauingenieur, 1924, p. 745.

[10] Stahlbau, 1928, p. 221.

[11] *Jordan-Michel:* 2nd Vol., p. 66.

In the following table are given the dimensions of some structures falling in this group.

Table 5: Large halls.

Hall	Erected	Span	Length	Height	Literature
Ausstellungshalle I on the Kaiserdamm, Berlin	1914	49.8	225	20.4	ZdVDI 1915, p. 45
Ausstellungshalle II on the Kaiserdamm, Berlin	1924/25	47.0	146	20.45	Deutsche Bauzeitg. 1925 Design and construction, p. 137
Sample Fair Hall 7, Leipzig	1928	97.8	139	21.0	Stahlbau 1928, p. 2
Sample Fair Hall 19, Leipzig	1928	60.0	140	19.5	Stahlbau 1928, p. 161
Sample Fair Hall 20, Leipzig	1929	50.0	80	18.3	Bautechn. 1930, p. 347
Deutschland Hall, Berlin	1935	58.2	95	28.5	Deutsche Bauztg. 1935, p. 1003
Sample Fair Halls on the Masurenallee, Berlin	1936	23.3 41.15	45 97	35.0 18.0	Nothing yet published

For the sample fair hall in the Masurenallee in Berlin, the first row of figures refer to the middle hall, whilst the figures in the second row give the dimensions of the side exhibition halls.

b) In the second group have to be placed halls where the span has to be comparatively small because of the necessity of providing an overhead crane. As a rule these halls have three bays, a high middle bay being bounded on both sides by bays which are lower and in some cases also narrower.

Table 6: Small halls.

Hall	Erected	Middle bay		Side bays		Length	Literature
		Span	Height	Span	Height		
Sample Fair Hall 8, Leipzig	1924	21.88	15.6	11.06	8.0	195	Bautechn. 1925, p. 4
Sample Fair Hall 9, Leipzig	1924	19.5	19.59	19.5	13.1	173	Bautechn. 1924, p. 490
Nordic Sample Fair, Kiel	1925	28.0	15.6	7.0	5.2	171	Bautechn. 1926, p. 33
Sample Fair Hall 21, Leipzig	1925/26	24.0	18.1	10.0	8.0	155	Bauingenieur 1927, p. 1

Truss forms and lighting.

To a far greater extent than in all other halls, particular importance is attached in exhibition and sample fair halls to good and uniform lighting. The form given to the trusses is consequently often conditioned by the lighting requirements.

In the machinery hall of the Iba 1913, curved roughglass panes are provided, where the frame uprights connect to the ridge, for lighting the middle bay. Quite similar in form, as also in dimensions, is the same fair hall 21 at Leipzig (Fig. 11). The middle bay is also lighted through rows of windows in the walls above the side bays; here, however, the glass surfaces are absolutely vertical. Also the sample fair hall 9 at Leipzig has rows of vertical windows under the eaves of the middle bay.

The main hall for the Nordic Sample Fair at Kiel (Fig. 12) has a stepped roof with rows of vertical windows in the separate risers. In contrast to this construction, in which the woodwork of the roof is strung on the steel three-hinged frames, in the sample fair hall 8 at Leipzig (Fig. 13) particular importance was laid on the

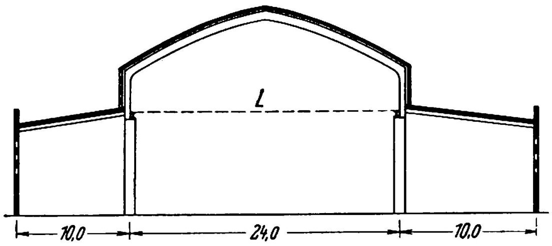

Fig. 11.
Market hall 21, Leipzig
L travelling crane 20 t capacity.

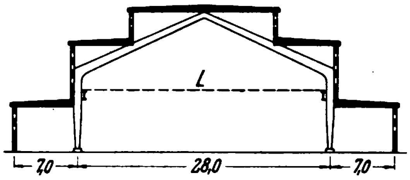

Fig. 12.
Market hall, Nordic Fair, Kiel
L travelling crane.

roof covering, as seen from the interior of the hall, running as far as possible in line with the supporting structure. There had to be no space anywhere between the supporting and the supported parts.

The smaller halls mentioned above have all been constructed with plate trusses, but in the wide-spanned halls lattice-work predominates.

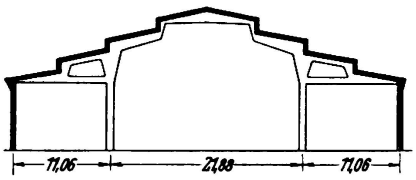

Fig. 13.
Exhibition hall 8, Leipzig Fair, Leipzig
Provisions made for future crane.

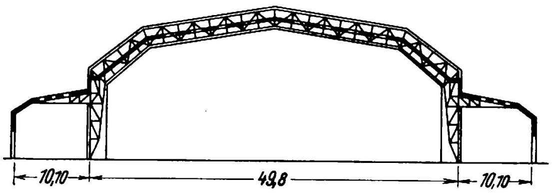

Fig. 14.
Exhibition hall I on Kaiserdamm, Berlin.

At the desire of the jury, the lattice-work construction of exhibition hall 1 on the Kaiserdamm in Berlin (Fig. 14) was completely encased. In order to avoid having a double ceiling, the roof covering is located at about half the height of the trusses. The part of the lattice-work trusses projecting outwards is covered with wood casing and asphalted sheeting; the inner part of the trusses has a Rabitz covering. The skylights are arranged in the middle part of the trusses.

Fairly similar, both in dimensions and also in the shape of the trusses, but executed fully plated, is hall II on the Kaiserdamm in Berlin (Fig. 15). The three-hinged trusses are uniformly curved and are left quite uncovered. The skylights are on the outer parts of the roof of the middle bay, where the slope is greatest. By means of the gallery running round the side bays, the available exhibition space is considerably increased.

The construction of hall 20 of the Leipzig Building Fair (Fig. 16) is also plated. The trusses are riveted sheet-metal girders, resting fixed at both ends on fixed supports, which are of lattice-work type in the lower part. The hall is lighted by vertical glass surfaces, 12 m high, in the side walls.

As the day-light intensity curve[12] shows, in such wide halls with windows only in the side walls, the brightness of the light in the middle of the hall is only about half of what it is near the windows. In hall 19 at Leipzig (Fig. 17), which is about

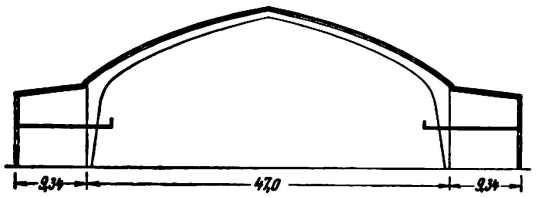

Fig. 15.
Exhibition hall II on Kaiserdamm, Berlin.

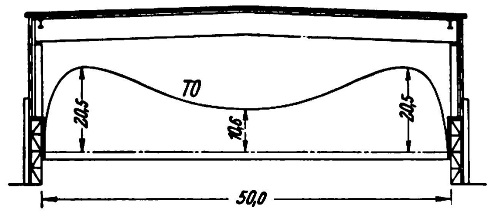

Fig. 16.
Exhibition hall 20, Leipzig Fair, Leipzig
TO Curve of daylight ratio.

10 m wider, an endeavour was made to distribute the light more uniformly, and yet to manage only with vertical skylights; by adopting an arrangement of glass surfaces as shown in Fig. 18; this will require no further explanation.

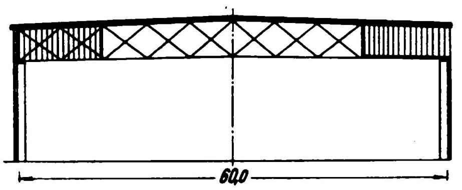

Fig. 17.
Exhibition hall 19, Leipzig Fair, Leipzig (section).

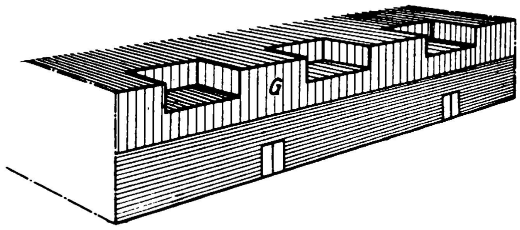

Fig. 18.
Exhibition hall 19, Leipzig Fair, Leipzig (side view)
G glass.

Sample fair hall 7 at Leipzig (Fig. 19) has two-hinged framed lattice-work trusses with saddle-shaped skylight. The upper members of the trusses form, as shown in Fig. 20, the ridge of this skylight, whilst the two inwardly inclined glass

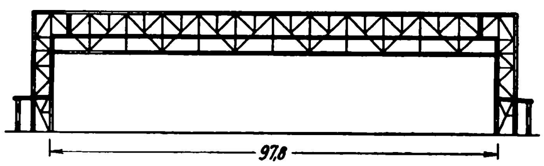

Fig. 19.
Exhibition hall 7 Leipzig Fair, Leipzig.

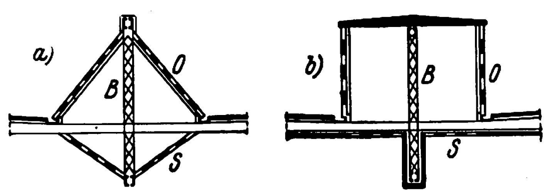

Fig. 20.
Design of skylights
B truss, O skylight, S dust screen.

surfaces of the dust roof come together at the lower members of the trusses. Naturally, vertical skylights may also be adopted here, as shown for example in Fig. 20b; such a construction has been proposed for, amongst others, a congress and exhibition hall at Hamburg.[13]

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[12] Determined according to the Burchhard light-measuring system. Cf. *Maier-Leibnitz:* Der Industriebau 1932, pp. 77ff.

[13] Stahlbau 1935, p. 40.

In the Deutschland Hall in Berlin, only the roof construction over the middle part, which measures 95 $\times$ 58.2 m, is constructed of steel; the walls supporting it are in masonry. As shown by Fig. 21, the trusses, which are designed as single lattice-work girders of 58.2 m span, are supported at each end on the roof beams, which are Gerber lattice-work girders passing along the whole length of 95 m. Transverse to the trusses run lattice-work purlins — also designed as Gerber girders; they support the rafters of the two-ply waterproof roofing and also the protecting roof, below, which is made of asbestos sheets.

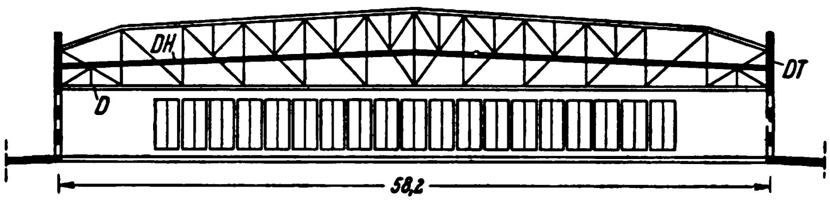

Fig. 21.
Roof construction of Deutschlandhalle, Berlin
DH roof cover, DT roof girder, D suspended ceiling.

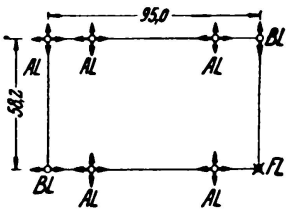

Fig. 22.
Supports of roof construction of Deutschlandhalle, Berlin
FL fixed bearing, BL bearing movable in direction of arrows, AL bearing movable in all directions

In such large connected roof structures, particularly when the supporting structure is of a different material, provision must be made to allow for mutual expansion. How this has been solved quite simply in the present case is shown diagrammatically in Fig. 22.

The Deutschland Hall serves not only as an exhibition hall; it is used particularly for large meetings, sporting events, etc. For lighting, it was consequently found sufficient to provide a row of windows running round below the steel roof.

The sample fair halls in the Masurenallee in Berlin, consist, as already mentioned, of two lateral exhibition halls and a middle hall.

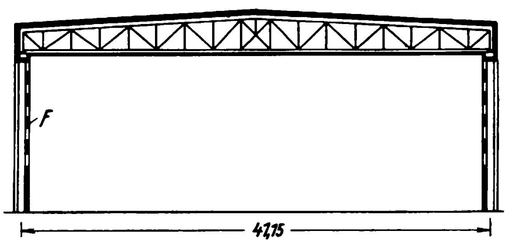

Fig. 23.
Fair hall at Masurenallee, Berlin (side view)
F glazing

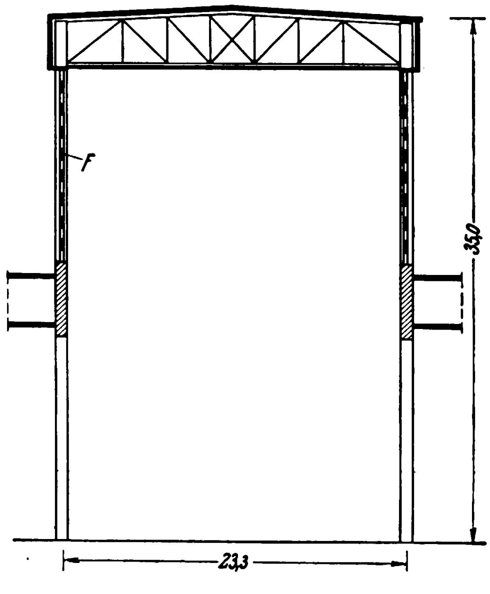

Fig. 24.
Fair hall at Masurenallee, Berlin (hall of honours)
F glazing.

The side halls (Fig. 23) have lattice-work trusses with ceiling suspended below; the plate supports are fixed at the foot. The lighting is from windows about 10 m high, in the side walls.

In contrast to the other structures, the middle hall (Fig. 24) is remarkable for its great height, 35 m. The trusses here again are normal lattice-work trusses, which are also hidden from view by a ceiling suspended below them. The walls too have again windows of large area. For this reason, of course, the problem of arranging for wind pressure was rendered very difficult; a horizontal rectangular frame, closed on all sides, is provided at the height of the lower member of the trusses; this frame is supported in a frame fixed at the bottom in each of the four fronts.

Airship Halls.

The development of airship halls is conditioned by the development in airship construction. The dimensions of some airships are given in the following table.

Table 7: Dimensions of airships.

System	Designation	Frist trip	Length	Diameter	Literature
Zeppelin	LZ 1	1900	128	11.7	Moedebeck[14] p. 732
	LZ 11	1912	148	14.0	
	LZ 62	1916	198	23.9	
	LZ 120	1919	120.8	18.7	
	LZ 126 (Los Angeles)	1924	200	27.64	Engberding[15] p. 160
	LZ 127 (Graf Zeppelin)	1928	235	30.5	Engberding[15] p. 272
	LZ 129 (Hindenburg)	1936	245	41.2	ZdVDI 1936, p. 379
Schütte-Lanz	SL 1	1911	131	18.4	Moedebeck[14] p. 755
	SL 15	1916	174	20.1	
	SL 24	1918	232	25.4	
Parseval	PL 1	1909	60	9.4	Moedebeck[14] p. 786
	PL 25	1915	110	16.4	
	PL 27	1916	160	19.6	

Shape and dimensions of halls.

All the airship halls have the same ground plan, a long rectangle. For reasons of economy, halls with circular or star-shaped plan[16] have not passed beyond the suggestion stage.

In the halls hitherto built, a distinction must be made between those intended for one airship, and those for taking two airships. In the early days when airships were comparatively small, a double hall was often chosen, principally from reasons of economy. An objection to this type was, however, meanwhile found: the airships when entering or leaving the hall, especially in a high wind, got in each other's way. In addition, if any accident such as an explosion occurred, there was the risk of two airships being damaged or even destroyed. With the present dimen-

[14] *Moedebeck:* Taschenbuch für Flugtechniker und Luftschiffer, 4th edition, 1923.
[15] *Engberding:* Luftschiff und Luftschiffahrt 1928.
[16] Eisenbau 1910, p. 228.

sions of airships, it is probable that only halls for one airship will come into question in future.

Table 8: Halls for one airship.

Airship hall	Erected	Length	Width inside	Height	Literature
Tegel		100	25	25	Eisenbau 1910, p. 229
Frankfurt	1911	160	30	24	Stahlbau 1930, p. 61
Seddin		184	35	28	Engberding, p. 212
Friedrichshafen	1929	250	50	46	Stahlbau 1930, p. 61
Rhein-Main	1935	281	52	51	P-Träger 1936, p. 2
Rio de Janeiro	1935	270	50	50	Stahlbau 1936, p. 88

Table 9: Double airship halls.

Airship hall	Erected	Length	Width inside	Height	Literature
Hamburg	1912	160	45	26	ZdVDI 1912, p. 1299
Potsdam	1912	172	50	25	ZdVDI 1913, p. 681
Leipzig	1913	184	60	25	Eisenbau 1913, p. 369
Seddin	1915/16	242	60	35	Stahlbau 1930, p. 61
Nordholz	1916	260	75	38	Stahlbau 1929, p. 251
Ahlhorn	1916/17	260	75	36	Stahlbau 1930, p. 61

The two new halls in North America are of very large dimensions, as can be seen from Table 10.

Table 10: New American hals.

Airship hall	Erected	Length	Width	Height	Literature
Akron	1929	358	99	55	Stahlbau 1930, p. 68
Sunnyvale	1932	341	94	59	Stahlbau 1933, p. 7

The trusses of the airship halls have hitherto always been of the lattice-work type. The preference for plate construction, which has been displayed in all other branches of steel structural work for quite a number of years, has hitherto not been considered for airship halls.

Fig. 25 shows the cross-section of the Tegel hall; statically, it is a two-hinged arch with raised footing. The recent halls are as a rule designed or statically determinate three-hinged arches; in order to reduce the stresses in the bars, the foot hinges are here also generally raised. Fig. 26 shows the cross-section of the Friedrichshafen hall; the halls at Rhein-Main and Rio de Janeiro are of similar design.

In the two American halls given in Table 10, the cross-section has been chosen parabolic; tests made on models in a wind tunnel showed this form to be suitable.

From the point of view of design, the three rotatable airship halls[17] are very interesting; they are intended to facilitate the arrival and departure of the airships by being turned in the direction of the prevailing wind. Because of the high cost of construction, and also because of the improved landing facilities for airships (mooring masts[18], etc.), this type of construction was soon abandoned.

[17] Deutsche Bauzeitung 1914, p. 146; Bauing. 1922, p. 584.
[18] Cf., for example, Z.d.V.D.I. 1936, p. 400.

Lighting and ventilating.

The hall at Friedrichshafen[19], erected in 1909 for building airships, has saddle-shaped skylights extending over the whole hall. The later halls have all continuous rows of windows in the roof along the length of the hall, and also large windows in the walls.

The lighting surfaces of the new American airship halls are extremely small, since according to American opinion all work must in any case be done with artificial light, because the huge hull of the airship shuts out all daylight.

Great importance is attached to efficient ventilation, because of the gas filling of the airships. Generally a part of the windows is arranged to be opened; in addition ventilating valves, and special ventilating pans, are provided in the roof.

Doors.

The doors of airship halls have to conform to special requirements. When open, they must leave the cross-section of the hall perfectly free; also, in spite of their large dimensions, they must be capable of being opened and closed easily and without requiring any great force, even if there is a certain wind pressure.

As first constructed, the doors were of the ordinary hinged type, shown in Fig. 27a (airship hall Frankfurt, 1911; Baden Oos).[20] Very often double sliding doors, as in Fig. 27b, were used. Here two arrangements have to be distinguished. In one, the upper and also the lower rails for the door may project out beyond the sides (for example, airship halls Tegel, Ahlhorn and No dholz); this necessitates, of course, a special supporting structure for the upper rails. In the other arrangement, only the lower door rails project outwards beyond the sides, whilst the upper rails extend only over the width of the hall; the doors must then be designed as so-called "three-point" doors.

A combination of hinged and sliding doors was used in the airship works hall, built at Friedrichshafen in 1909. With a hall about 50 m wide, ordinary hinged doors would have to be too large, so that each leaf was divided into two. As shown diagrammatically in Fig. 27c, the inner leaf (I) was first slid behind the outer leaf (II), and then both were swung round together.

The forms of doors shown in Figs. d and e need no explanation. They were proposed for the Friedrichshafen hall just mentioned, but were not made.

When the hinged doors (Fig. 27a) are open, they form a funnel-shaped extension to the hall, and it was at first thought that this would to a certain extent protect the airships from the wind when entering and leaving. Such an arrangement can also be obtained with sliding doors (Fig. 27f), the guide rails projecting beyond the hall being not obliquely to the axis of the hall (Barkhausen system). Naturally, here also both the lower and upper rails may project out from the side; but also the lower rails alone may project out (for example, the airship halls at Potsdam and Leipzig).

In practice, however, it was found that air eddying is easily caused, and this may considerably impede the entering and departure of the airships.[21] Consequently an endeavour is now made to have the opened doors as close as possible to the

[19] Eisenbau 1910, p. 99.
[20] Z.d.V.D.I. 1918, p. 681.
[21] Z.d.V.D.I. 1915, p. 762.

side walls (Fig. 27g), in order to offer less resistance to the wind and thus be less likely to cause any disturbing eddying of the air. In the halls at Friedrichshafen 1929, Rhein-Main and Rio de Janeiro cylinder-segment doors were adopted; instead of these the American halls (Table 10) have hemispherical-segment doors.

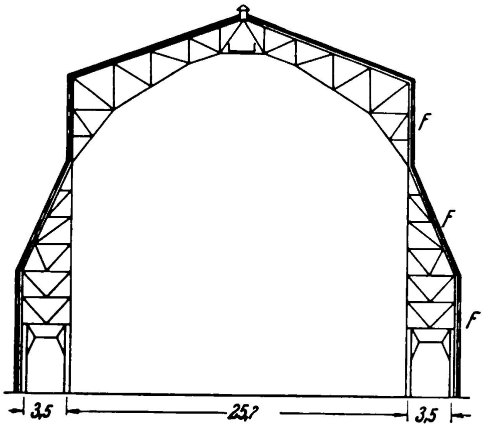

Fig. 25.
Airship Hall Tegel
F glazing

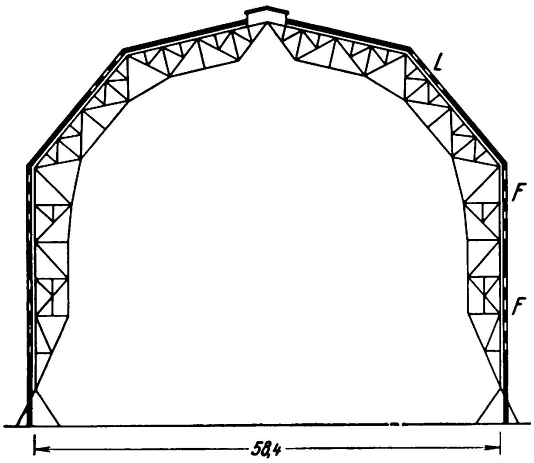

Fig. 26.
Airship Hall Friedrichshafen
L glazing, F window.

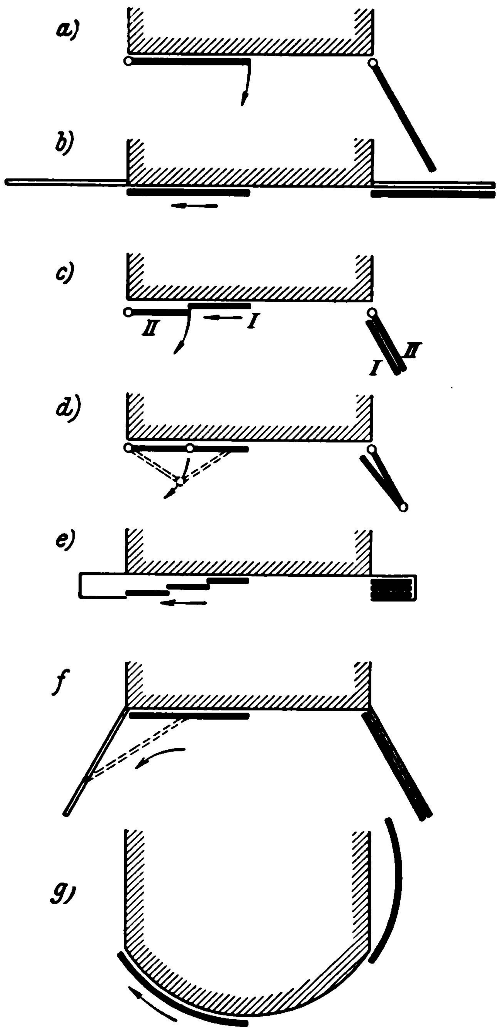

Fig. 27.
Arrangements of gates of airship hangars (outline)
left half: gate closed, right half: gate open.

The majority of airship halls have a doorway at each end. Only a few halls have doors only at one end. Since a construction of recent date is found in the Rio de Janeiro hall, where the back end has only a small sliding door, which serves for taking the mooring mast in and out and also for ventilation.

Aeroplane Halls. (Hangars.)

The development of aeroplane halls shows, in general, no fundamental changes in the design of the halls, but merely an adaptation of the design to suit the steadily increasing dimensions of the aeroplanes. Particulars of a few aeroplanes are given in Table 11.

Table 11: Dimensions of aeroplanes.

Aeroplane	Built	Span of wings	Length	Height	Literature
		Single-engine military planes			
Roland C II	1915	10.3	7.7	2.9	Moedebeck[14] p. 586
Junkers Cl I	1918	12.3	7.9	2.7	„ „
		Multi-engine military planes			
AEG G IV	1917	18.4	9.7	3.9	„ p. 595
Staaken R VI	1917	42.2	22.5	6.5	„ „
		Naval planes			
Brandenburg CC	1917	9.3	8.5	3.3	„ p. 602
Staaken L	1917/18	42.2	22.2	7.4	„ „
		Single-engine commercial planes			
Junkers	1919	14.8	9.5	3.4	„ p. 612
Dornier C III	1920/21	17.0	9.5	2.5	„ „
		Multi-engine commercial planes			
Junkers Ju 52	im commis-	29.25	18.9	4.5	Shell-Flugzeug-
Heinkel He 70	sion at present	14.8	11.5	4.15	führer[22] p. 78 a. 75

The principal dimensions of a number of aeroplane halls can be seen from the following table.

Table 12: Dimensions of some aeroplane halls.

Aeroplane hall	Erected	Depth	Door opening Width	Door opening Height	Type of door	Literature
Standardised military aeroplane hall	1914/18	22.2	22.08	4.7	Folding-sliding door	Stahlbau 1928, p. 86
Berlin-Tempelhof	1925	30.0	4×40.0	8.0	Folding door	Bauing. 1925, p. 839
Nietleben near Halle	1925	22.0	39.7	6.3	Sliding door	Stahlbau 1929, p. 28
Hamburg-Fuhlsbüttel	1925 1926	30.0 40.0	2×30.0 80.0	6.0 8.0	Sliding door	Bautechn. 1927, p. 311
Bremen		30.0	80.0	8.0	Sliding door	Bautechn. 1927, p. 443
Stettin	1927	35.2	52.6	9.0	Sliding door	Stahlbau 1928, p. 88
Schkeuditz near Halle	1927	30.0	2×60.48	10.0	Folding door	Stahlbau 1929, p. 28
Dortmund	1927	25.0	41.0	7.2	Folding door	Stahlbau 1928, p. 194
Travemünde	1927/28	61.0	60.6	12.0	Sliding door	Bautechn. 1928, p. 294
Kiel-Vossbrook		28.0	35.0	8.0	Folding door	Stahlbau 1929, p. 22
Munich	1928	70.0	2×31.0 a. 60.0	10.0	Folding door	Bautechn. 1930, p. 251
Brunswick	1928/29	30.0 30.0	2×50.0 3×30.3	9.5 6.5	Sliding door	Stahlbau 1930, p. 124
Breslau	1931/32	30.15	54.0	8.0	Folding door	Bautechn. 1933, p. 96

[22] Published privately by the Rhenania-Ossag Mineralölwerke A.-G., Hamburg, 1935 edition.

At first, when the dimensions of the aeroplanes were small, the halls consisted of any desired number of small hall units about 20 to 22 m², arranged alongside each other; the front wall of each unit was formed by the doors.

Soon, however, the increasing wing span made it necessary to have wider entrance doors. As can be seen from Table 12, the width of the door openings in recent halls is from 30 to 80 m; in the majority of cases, it is between 40 and 60 m. The clear height of the doorways amounts to 6 m to a maximum of 10 m; the only greater height of doorway is in the hall at Travemünde, where hydroplanes have to be run in on carriages.

A standard construction for aeroplane halls is shown in Fig. 28. The door beam passes over the whole width of the hall — with or without intermediate supports. Transverse to this are the trusses, which rest at the front on the door beam and at the back on the steel framed wall. These trusses often project in front over the door beam and support the upper door rails.

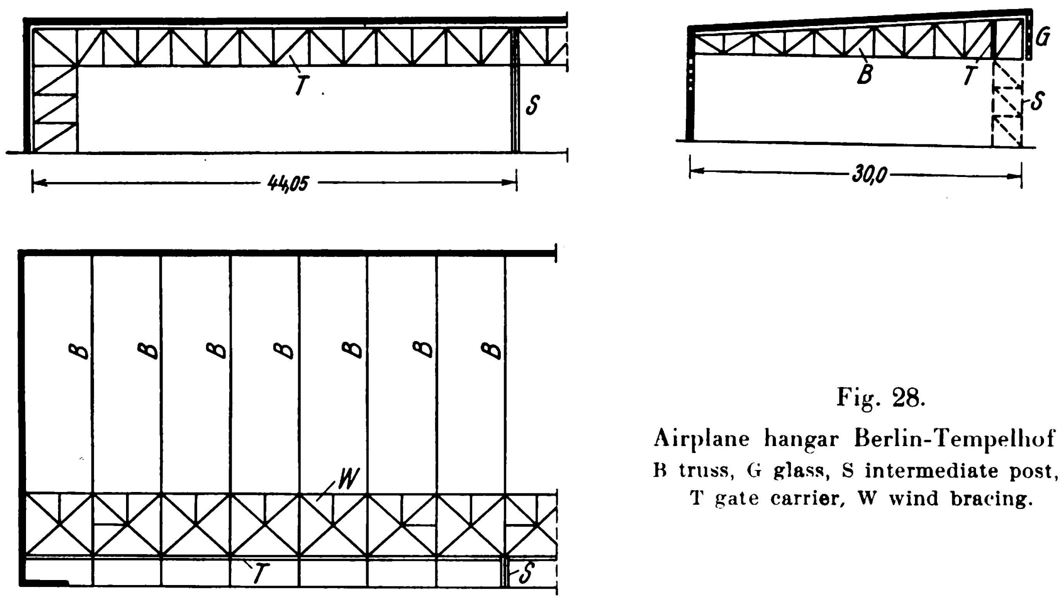

Fig. 28.
Airplane hangar Berlin-Tempelhof
B truss, G glass, S intermediate post, T gate carrier, W wind bracing.

The supporting structure of all the above mentioned halls is of the lattice-framed type. The door beams are, as a rule, of uniform depth; only in wide-span beams (for example, in the halls at Hamburg and Bremen) the depth of the beams increases towards the middle.

Because of the great height of the door beams as compared with the height required for the trusses transverse to them, it was natural to make the roof slope down to the back. At the end of the war, the endeavour to make the halls look less like sheds led to some forms of cross-section with couple-close roof[23] being evolved; with the great height of the door beams, such lines do not come into consideration.

A construction differing from the usual shapes of the halls is to be seen in the aeroplane hall at Munich-Oberwiesenfeld. As shown in Fig. 29, this hall has only one wall, whilst doors are arranged on the other three sides. This arrangement naturally makes another design of supporting structure necessary.

[23] Stahlbau 1928, p. 86.

In recent times the halls are as a rule lighted by rows of windows over the door and also in the rear wall, instead of by the saddle-shaped skylights formerly used. The glazing above the door may then be inclined; vertical windows are also often adopted (Fig. 28).

Particular consideration must be paid to the doors. Of the many systems proposed, sliding doors (Fig. 30a) and folding doors (Fig. 30b) have proved the most

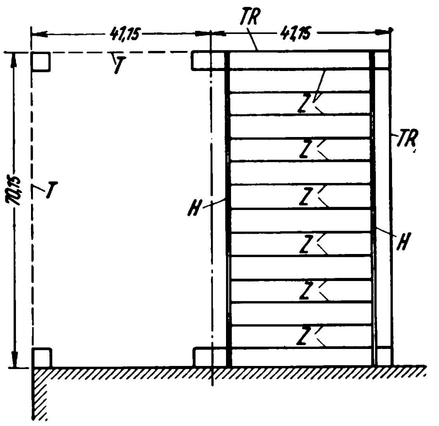

Fig. 29.
Airplane hangar Munich-Oberwiesenfeld (plan)
left half: arrangements of gates,
right half: roof construction.
T gate, TR gate carrier, H main truss, Z intermediate truss.

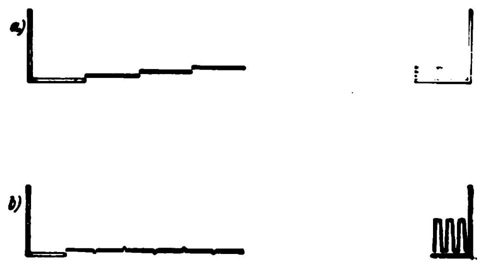

Fig. 30.
Arrangement of gates for airplane hangars
left half: gate closed, right half: gate open.

satisfactory and are exclusively adopted for recent halls. The door beams are already highly stressed by having to support the roof trusses; in order to avoid stressing them further, and also to be independent of their bending, the doors are generally supported below on running rails; the rails at the top merely serve to keep the doors always vertical.

Tramway and Motorbus Halls.

The great amount of room required for the maintenance, repairing and cleaning of tram cars and motorbusses makes it necessary, especially in large towns, to roof-over large spaces. The old existing carsheds, which were as a rule fairly small, have therefore generally made place for new large halls, particularly in the afterwar period.

Tramway Halls.

In tramway halls, the design of the structure is comparatively simple, since it is at once possible to support the roof by rows of pillars placed between the tracks. But, in order to lose as little space as possible, and also not to obstruct the view, the distance between the supports was chosen not too narrow. All new halls have a truss span of over 20 m.

The Dortmund tramway hall[24], built in 1926, has still lattice-work trusses. On the other hand, the hall built only a little later at Bochum[25] (Fig. 31a) has already

[24] Stahlbau 1928, p. 208.
[25] Bautechn. 1931, p. 691.

plate trusses; as can be seen from the longitudinal section (Fig. 31b) the distance between the supports in the direction of the tracks is fairly considerable. Consequently, in the direction of the length of the hall, joists were first arranged over the supports and on them the trusses lie.

For lighting these two halls saddle-shaped skylights are used. As an example of a construction with a row of continuous windows the tramway hall, Müllerstrasse,

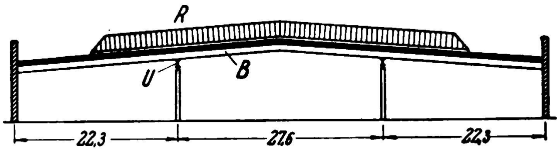

Fig. 31 a.

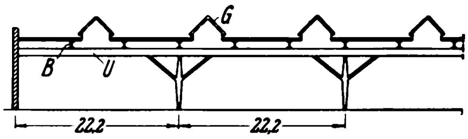

Fig. 31 b.

Fig. 31.

Tram car shed, Bochum, a cross section, b longitudinal section
B frame, R skylights, U girder, G Glass.

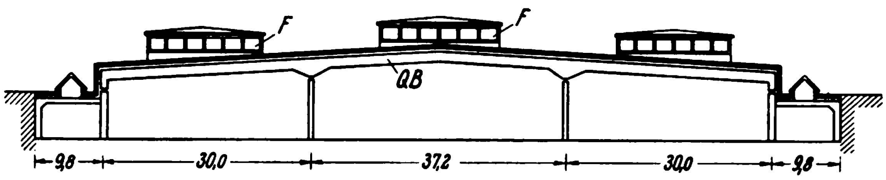

Fig. 32 a.

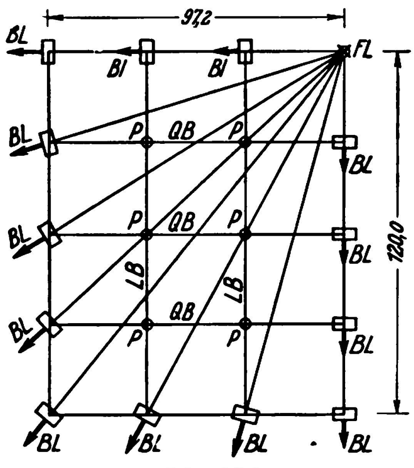

Fig. 32 b.

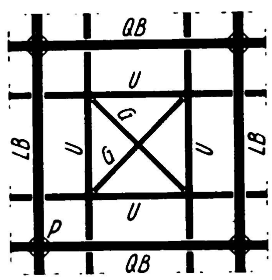

Fig. 32 c.

Fig. 32. Tram car station 16, Charlottenburg, a cross section, b plan and arrangements of bearings, c part plan
F skylights, G ridge girder, LB longitudinal truss, QB transverse frames, U beams, FL fixed bearing, BL bearing movable in direction of arrow, P hinged column.

Berlin[26], may be cited. The shape of the plated three-hinged arches is then suited to the skylights.

Very interesting from the point of view of design is the hall of tramway terminus 16 at Charlottenburg[27] (Fig. 32a). As can be seen from the plan (Fig. 32b), the hall has three transverse and two longitudinal trusses, connected at the points where they cross in order to prevent bending. Each of the spaces in the truss grating thus formed is in turn fitted with a grating of joists (cf. Fig. 32c), which supports the

[26] Bauing. 1928, p. 383.
[27] Stahlbau 1935, p. 1.

dome-shaped roof structure. The roof construction is supported on three sides on the small frames, and on the front on the gable wall. Further, rocking supports are arranged at the junction points of the truss grating. In order to prevent compression stresses, only one bearing is of the fixed type; all the other bearings are, as shown in Fig. 32b, movable radially. The whole of this large roof construction, measuring 97 × 120 m, can therefore expand and contract freely with fluctuations in temperature; measurements, taken during several years, show that the movements of the structure follow the fluctuations in temperature fairly closely.

Lighting is effected, apart from the small side frames exclusively through rows of vertical windows.

Motorbus Halls.

When deciding the span of the trusses and the distance between the supports in the tramway terminus 16 at Charlottenburg, consideration was also paid to the possibility that this building might some day be used as a motorbus hall. The conditions for motorbus halls are somewhat different: the effective area of the hall is reduced not only by the actual floor space covered by the supports; a much greater area must be deducted to allow a safe distance to be maintained between

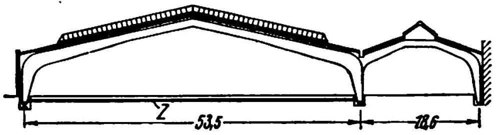

Fig. 33.
Motor-coach depot of the ABOAG Berlin,
Morsestr.-Helmholtzstr.
Z tie.

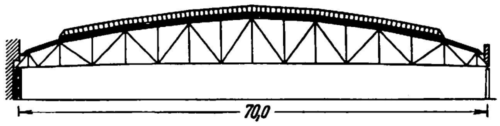

Fig. 34.
Motor-coach depot Treptow
of the ABOAG Berlin.

the moving vehicles and the supports. Another point to be considered is the great damage which may be caused to a vehicle, or even to the whole structure, in the event of a collison with one of the supports.

Consequently a hall erected solely for motorbusses has only very few inner supports, or generally none at all. The comparatively great cost of such buildings will soon be more than repaid by the extra floor space gained and above all by the spreading-up of the service[28].

Fig. 33 illustrates the hall built in 1925/26 in Berlin for the ABOAG in Berlin, Morsestrasse-Helmholtzstrasse[29], in which the plate trusses are provided with tension members below the floor. The motorbusses stand only in the large bay. The small bay is used solely as a washing room and for making light repairs. A similar structure, but with only one bay, is the motorbus hall of the Hamburg Hochbahn[30], which was erected in 1927.

The Berlin-Treptow hall[31] (Fig. 34), because of the considerable span of 70 m, has parabolic lattice-work girders. Nevertheless, in the Berlin Wattstrasse hall[32],

[28] Bautechn. 1928, p. 315.
[29] Bauing. 1926, p. 959.
[30] Stahlbau 1928, p. 25.
[31] Deutsche Bauzeitung 1931, Design and construction, p. 53.
[32] Deutsche Bauzeitung 1931, Design and construction, p. 53.

erected a year later, a return was made to plate girder construction, although the span of 63 m is not much less. A lattice-work construction would have been cheaper, but plate was chosen for aesthetic reasons and also because of fire insurance requirements.

All the above-mentioned motorbus halls are provided with saddle-shaped skylights.

Steel Framed Buildings.

The home of steel framed buildings is America. Already in 1885 the first skyscraper was erected in Chicago. How rapid the development was, is best shown by the fact that in 1929 there were in the U.S.A. about 4800 buildings with more than 9 storeys[33]. The highest steel framed building hitherto erected in the world is the Empire State Building[34], completed in 1931, which has 86 storeys and a total height of 381 m. Such high buildings are, however, no longer profitable, even under American conditions. The most recent investigations[35] have shown that skyscrapers with about 60 storeys show the best rentability in the U.S.A. The latest American buildings therefore, such as the Rockfeller Center in New York[36] endeavour no longer to create new records for height, but rather for the mass of the structure.

Also in Europe steel framed buildings were erected at an early date. As examples may be mentioned the Menier chocolate factory near Paris[37], built in 1871, and the Elbe store house at Magdeburg[38] built in 1890. However, such buildings remained for a long time quite exceptional in our part of the world. Until about the war, the usual form of steel building with several storeys was the so-called "girder construction": floor-girders, joists and inner supports were made of steel; the solid outer walls carried their own weight and also a share of the flooring. As a rule no investigations were made regarding wind pressure, since the buildings were mostly effectively stiffened by partition walls and floors. This simple method of construction proved no longer sufficient when, especially after the war, buildings became always higher and rooms as large as possible and of wide span without partition walls were demanded. Steel framed structures were then adopted also by us.

In steel framed buildings the steelwork takes the whole load; the only use of the walls is to enclose spaces. The material used for the walls must therefore be primarily a bad conductor of sound and heat; its strength on the other hand is quite a secondary matter.

This, of course, does not exclude the possibility of using a wall to support part of the load in cases where an ordinary brick wall is required, for example as a fireproof wall. The masonry can also carry some of the load when, for example, a "wind target" (for taking wind pressure), which may be designed as vertical storeyframe or as lattice, is bricked in. In consequence of its greater stiffness, the wall prevents deformation of the steel construction; only when this rigidity has been overcome, i. e. when the wall cracks, will the steelwork take over the whole load.

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[33] Stahlbau-Vorträge (Lectures on steel structures), published by the Deutscher Stahlbauverband, Berlin 1931, p. 29.
[34] Stahlbau 1932, p. 39.
[35] Bauing. 1935, p. 386.
[36] Bauing. 1933, p. 275 and Stahlbau, p. 198.
[37] Deutsche Bauzeitung 1932, p. 362.
[38] Stahlbau 1931, p. 186.

Ground plan of the buildings.

In the choice of the ground plan of a steel framed building, the designer has very seldom a free hand. First of all he is limited by the shape of the site available; then he has to consider whether the whole site should be built on, or whether — from considerations of lighting and ventilating — a part must be left free for a court. The steel framed buildings which have hitherto been constructed are consequently planned very diversely.

The simplest construction is, of course, possible in buildings where the plan is rectangular, or consists of several rectangles. Ground plans with walls making an acute angle with each other, or where the boundary lines are curved, generally entail difficulties in designing the steelwork.

In the following table, some shapes of ground plan, each with an actual example, are mentioned.

Table 13: Shapes of ground plan of steel framed buildings.

Shape of plan	Example	Fig.	Literature
Rectangle	Europahaus Leipzig	35 a	Stahlbau 1930, p. 181
Angle	Administrative buildings of the DHV, Hamburg	35 b	P-Träger 1930, p. 4
T	Sausagé factory of the Cooperative Society, Berlin	35 c	Stahlbau 1929, p. 241
I	Kathreiner Hochhaus, Berlin	35 d	Deutsche Bauztg. 1930, K. u. A., p. 85
Rectangle with inner courts	Deutsches Museum, Munich, Library Building	35 e	Stahlbau 1930, p. 109
Irregular srraightline boundaris	Rhenania-Ossag-Haus, Berlin	35 f	Stahlbau 1931, p. 43
Curved boundaries	Columbus-Haus, Berlin	35 g	Stahlbau 1931, p. 253
	Administrative buildings of the I.G.Farben, Frankfurt-on-Main	35 h	Stahlbau 1931, p. 1

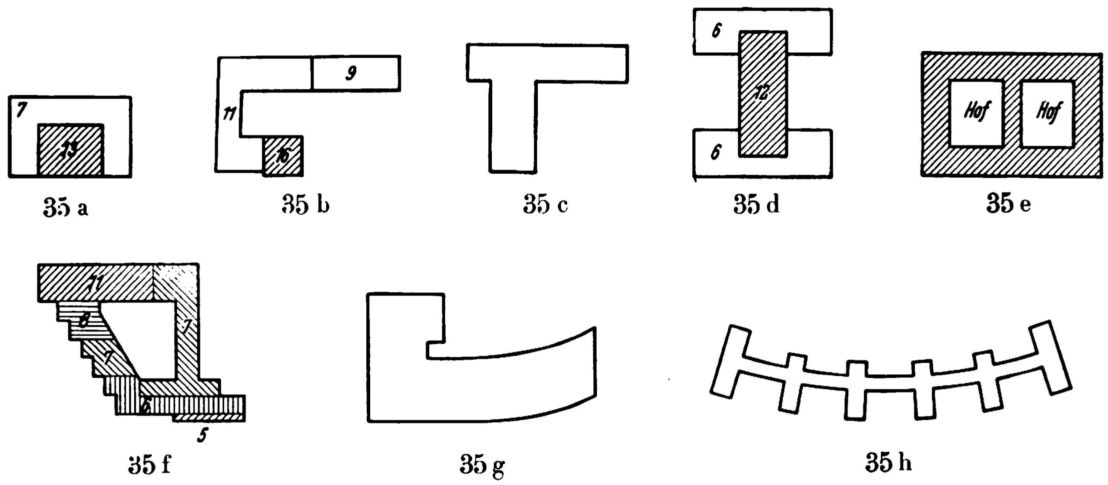

Fig. 35.
Arrangements in plan of steel skeleton constructions.

Elevations.

The maximum permissible height of a building is generally fixed by the local authorities; however, exceptions may be permitted in special cases.

The simplest section for a building is a rectangular one. In buildings in narrow streets or courts, the upper storeys must often be stepped back from considerations of lighting. An example of this is seen in Fig. 36, the section of business premises, "Samt und Seide", in Mannheim[39]. The angle inclination α of this stepping should as a rule not exceed 67^0.

The design of the structure is not quite so simple where the supports on the ground floor have to be set back, either to widen the pavement, or because of the arrangement of the shop windows. An example of this is shown in Fig. 37, a section through the Columbus-Haus in Berlin.

If only a part of the building is crowned with a tower, one speaks of a Turmhaus (tower house). For example, in the Europhaus in Leipzig, the tower, shown shaded on the plan in Fig. 35a, has 13 storeys above ground level, whilst the other

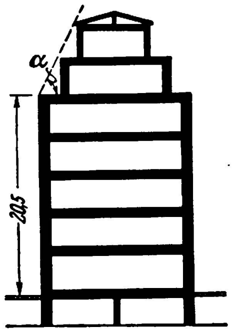

Fig. 36.
Office building "Samt und Seide", Mannheim (cross section).

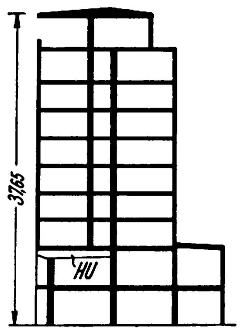

Fig. 37.
Columbus-building, Berlin (cross section)
HU main lintel.

part has only 7. The same applies to the buildings of which plans are given in Figs. 35b and d; here again the shaded part shows the location of the tower, whilst the numbers are the number of storeys above ground level.

Fig. 35b leads to the consideration of buildings of several different heights. As a further example may be mentioned the Rhenania-Ossag-Haus, Berlin, in which, as can be seen from the numbers in Fig. 35f, the number of storeys above ground level varies greatly.

An original type of building is also the spherical house[40] at Dresden, the only one as yet built.

Besides the section of a building and the number of storeys, the height of the rooms is also of interest. The following table gives these heights for a number of steel framed office buildings erected in recent years. It will be noticed that the tendency is to decrease these heights.

[39] Stahlbau 1928, p. 45.
[40] Stahlbau 1928, p. 130.

Table 14: Heights of rooms in office buildings.

Building	Erected	Ground floor	First floor	Other upper floors	Attic floor	Literature
I. G. Farben, Frankfurt-on-Main	1929	4.48	4.64	4.48—3.84	3.83	Stahlbau 1931, p. 1
Volksfürsorge, Hamburg	1929	4.2	4.2	4.2 —3,3	—	Stahlbau 1931, p. 129
Europahaus, Leipzig	1929	4.45	3.55	3.55—3.4	2.78	Stahlbau 1930, p. 181
DHV, Hamburg	1929	3.3	3.5	3.45	2.55	P-Träger 1930, p. 4
Kathreiner Hochhaus Berlin	1929	4.55	3.6	3.4	2.8	Deutsche Bauzeitg. 1930 K. u. A., p. 85
Columbus-Haus, Berlin	1931	4.8	4.96	3.42	3.83	Stahlbau 1931, p. 253
Rhenania-Ossag-Haus, Berlin	1930—32	4.0	3.6	3,3	—	Stahlbau 1931, p. 43

Steel framed structures are lighted by separate windows or by rows of windows; they are arranged according to the usual rules[41] for high buildings.

Wind bracing.

As already mentioned, as a rule no investigations were made regarding wind pressure, since the buildings were amply stiffened by the floors and partition walls. In more recent buildings, which have no or only a few partition walls, such simple procedure is no longer admissible. An obvious solution was to use frames instead of the partition walls; this led to buildings with vertical frames arranged side by side. In Fig. 38, which illustrates the plan of the business premises "Samt und Seide", in Mannheim[39], the frames are indicated by the thick lines.

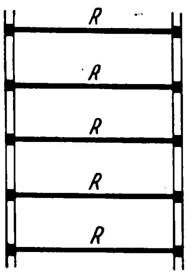

Fig. 38.

Office building "Samt und Seide", Mannheim (plan)

R Frame.

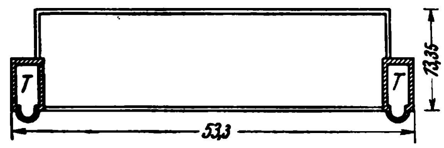

Fig. 39.

Postoffice building at Hochmeisterplatz, Berlin (plan)

T staircase.

Instead of distributing the horizontal forces among a great number of frames, they may be concentrated on a few points, where they are taken up by special "wind bracing panels". Preferably these wind bracing panels are located in the gable walls or in the staircases; they may take the form of a solid wall, or lattice-work, or frames. As an example, Fig. 39 shows the plan of the postoffice building on the Hochmeisterplatz[42], Berlin, in which the solid floors transmit all the wind forces

[41] Cf., for example: *W. Büning* and *W. Arndt:* Tageslicht im Hochbau (Daylighting in structural engineering), Berlin 1935.

[42] Stahlbau 1933, p. 68.

to the masonry staircases. In buildings of complicated floor plan, such an arrangement will often not prove sufficient. As shown in Fig. 40, which represents the plan of the Rhenania-Ossag-Haus in Berlin, a number of columns are in such cases connected together by joists to form vertical frames.

Naturally the floors must also be capable of transmitting the wind forces to the "wind bracings." If a concrete pressure-layer is provided, special iron bars can easily be laid in it (for example, in Rhenania-Ossag-Haus, Berlin). In exceptional cases an arrangement of separate horizontal bracing may be necessary (example: Administrative buildings of the D.H.V., Hamburg).

A combination of steel framed and girder constructions may be adopted, in which the wind frames or bracing are provided to take only the wind forces acting on the highlaying parts of the building; the wind pressure on the lower parts are taken by the solid outside walls. As an example, Fig. 41 shows the plan of the Werner Works in Berlin-Siemensstadt[43]; in this case, frames and lattice-work targets take only the wind forces acting above the 6th storey.

To this group belong also structures — as, for example, the library building of the Deutsches Museum at Munich, — where the wind bracings take the wind pressure only while the building is being erected. When finally completed, the solid walls are made use of.

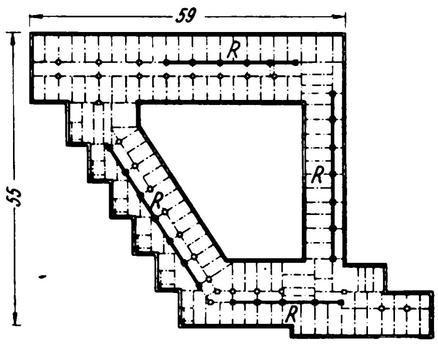

Fig. 40.
Rhenania-Ossag building, Berlin (plan)
R frame.

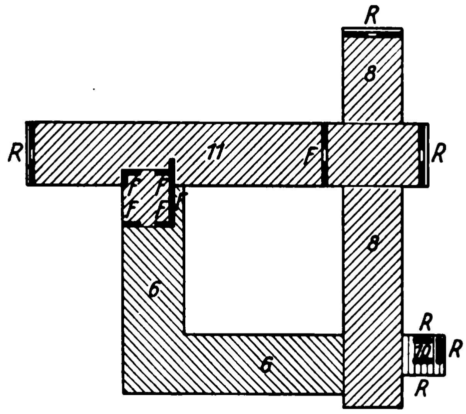

Fig. 41.
Wernerwerk, Berlin-Siemensstadt (plan)
F truss work, R frame.

Structural details.

The usual form of construction in steel framed buildings is in plate; I, IP and ⊏ steel sections are adopted. Angle sections are also used exceptionally, for example in the State Records Office at Königsberg (Prussia)[44], where the uprights of the shelving are at the same time the uprights of the steel framework.

Because of the great stiffness required, wind bracings are frequently constructed of lattice-work. A steel framed structure, completely of lattice-work, is the Pressa-Turm[45] in Cologne. Also the ribs of the Europahaus in Leipzig — except in the basement and ground floor — are all in lattice-work; special rolled sections of swallow-tail shape were adopted.

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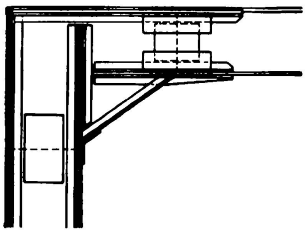
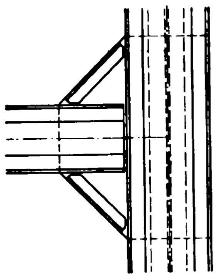
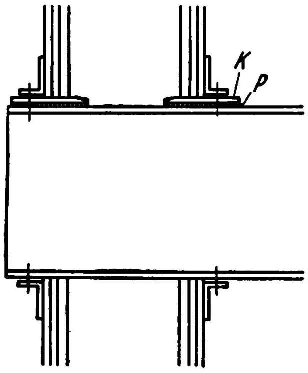
[43] Stahlbau 1931, p. 39.
[44] Stahlbau 1933, p. 207.
[45] Stahlbau 1928, p. 73.

Finally, a combination of plate and lattice-work may be used, as for example in the administrative buildings of the D.H.V. in Hamburg. Here the uprights are in plate, whilst the horizontal members, laid in the parapets of the outside walls, are in lattice-work.

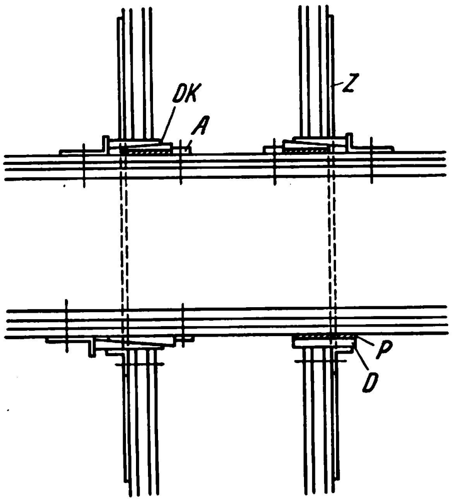
The development of special constructions is illustrated in the following Figs. 42 and 43; of these, Figs. 42d—e refer to details of the frame corners. Designs with corner plates, as for example used in the two-hinged frames of the new building "Samt und Seide" in Mannheim (Fig. 42a) or in the vertical storey frames of the Lochnerhaus[46] in Aachen (Fig. 42b), have recently been only rarely adopted. In their place come executions with feather bays and girder as frequently used, (for example in the library building of the Deutsches Museum in Munich, in the Werner Works in Berlin, in the administrative buildings of the Volksfürsorge in Hamburg, etc.). In the Rhenania-Ossag-Haus in Berlin, double bays, as in Fig. 42d, were adopted; through the tension straps, the joint could be kept comparatively small. A completely welded frame corner, as was used for example in the Haus der deutschen Erziehung[47] in Bayreuth, is illustrated in Fig. 42e.


42 a


42 b



42 c



42 d

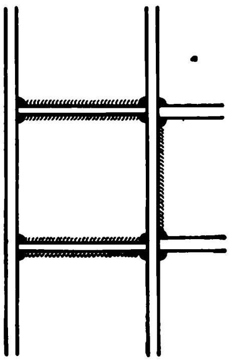

42 c

Fig. 42.

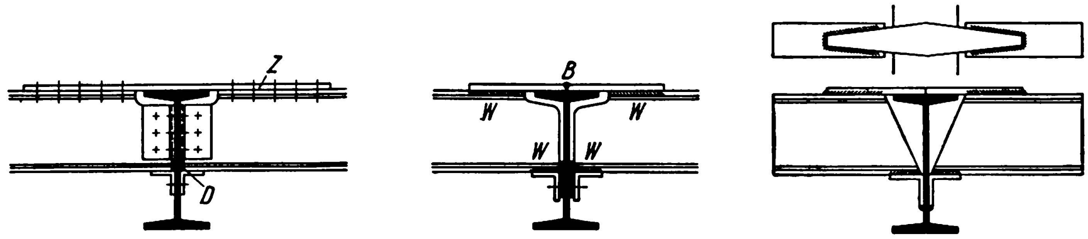
Details of frame corners

A rabbet, D pressure piece, DK folding wedges, K wedge, P adapter piece, Z tie.

[46] Deutsche Bauzeitung 1926, Design and construction, p. 41.

[47] Stahlbau 1936, p. 58.

A further development is also to be noted in the details of girder connections. The method by means of web angles or bearing plates has long been usual. Comparatively recently however, the junction has been as in Fig. 43a; tension strap and pressure pieces take the moments about the junction. A similar construction, but with welded tension strap, is shown in Fig. 43b[48]. In order to have to make as few joints as possible on site, the tension strap is divided; only the V joint marked B has to be welded on site, the others being done in the workshops. Finally, a completely welded execution — patented by the Gutehoffnungshütte, Oberhausen — is illustrated in Fig. 43c.



43a 43b 43c

Fig. 43.
Details of connections
D pressure piece, Z tensile cover plate, B erection joint, W shop joint.

Encasing the steelwork.

Floor girders, joists and the horizontal members of vertical frames are as a rule embedded in the ceiling and are thereby protected against corrosion and also against fire. The same purpose is served by encasing the uprights. If the latter consist of several rolled sections, the space between them is also filled with concrete. It is obvious that this concrete may also be used to carry load. The problems arising in this connection were treated very fully at the Paris Congress in 1932[49]; some more recent German publications on this subject are mentioned below[50].

Summary.

The foregoing article deals in a general manner with the development of hall structures and steel framed buildings. Among the hall structures discussed are railway station halls, exhibition and sample fair halls, airship and aeroplane halls, and tramway and motorbus halls. Reference is made to the dimensions and shapes of the halls, and also to lighting and ventilating. Further, in the case of airship and aeroplane halls the development in the design of the doors is described.

In steel framed buildings, the form of the steel structure in plan and in elevation is of interest. Closely connected with this is the question of taking the wind forces. After that, structural details are considered, and how they have developed in course of time. Finally, some reference is made to the problem of encasing the steelwork.

[48] P-Träger 1935, p. 7.

[49] International Association for Bridge and Structural Engineering, first Paris Congress 1932. Preliminary Publication, pp. 587ff.; Final Report, pp. 516ff.

[50] Stahlbau 1934, pp. 49ff.; 1935, pp. 81ff.; Zentralblatt der Bauverwaltung 1935, p. 536.

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