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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

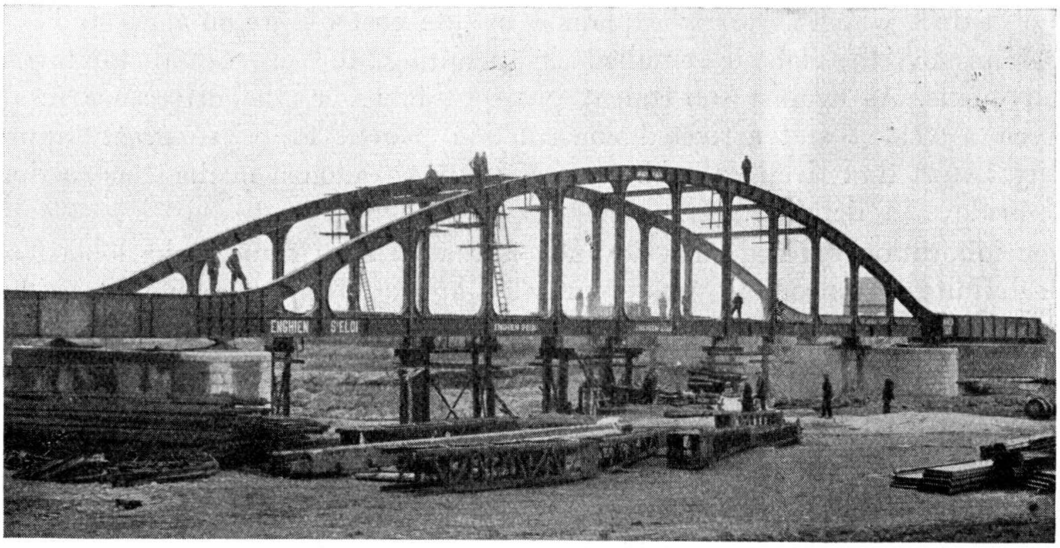


Fig. 1.
Bridge C in Hérenthals (1933 - 34).

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-

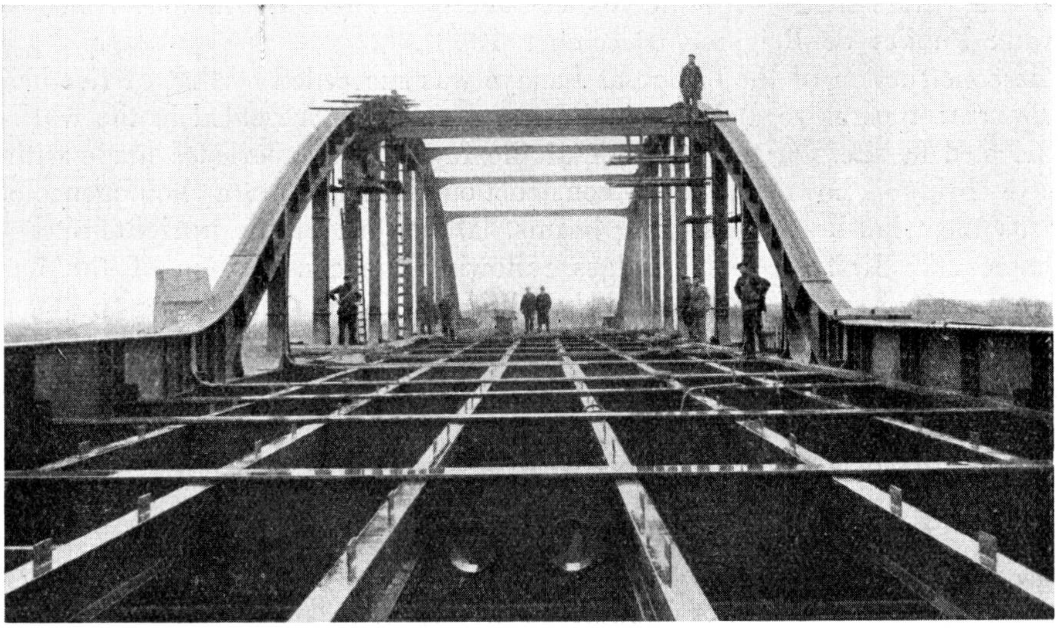


Fig. 2.
Bridge C in Hérenthals (1933—34).

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

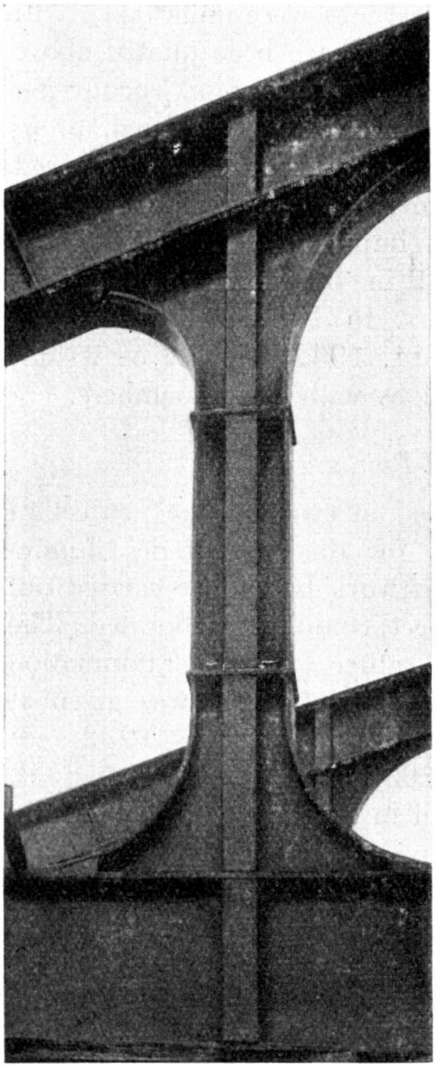


Fig. 3.
Bridge C in Hérenthals (1933—34).

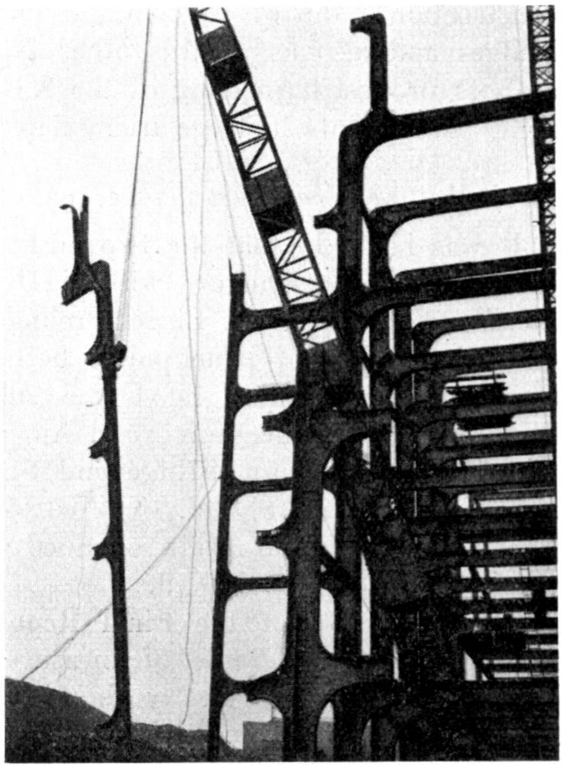


Fig. 4.
Building of the Chemical-metallurgical Institute Main entrance details (1930—31).

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 1st. 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/cm^2 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,

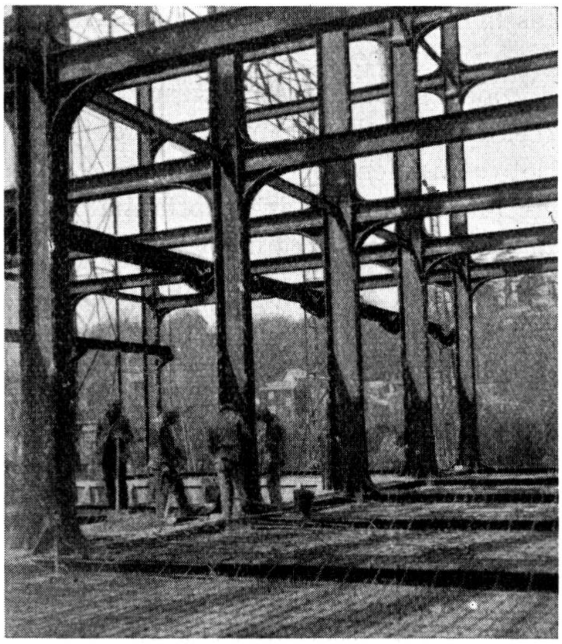


Fig. 5.
Steel structure for the Civil Engineering Institute.

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

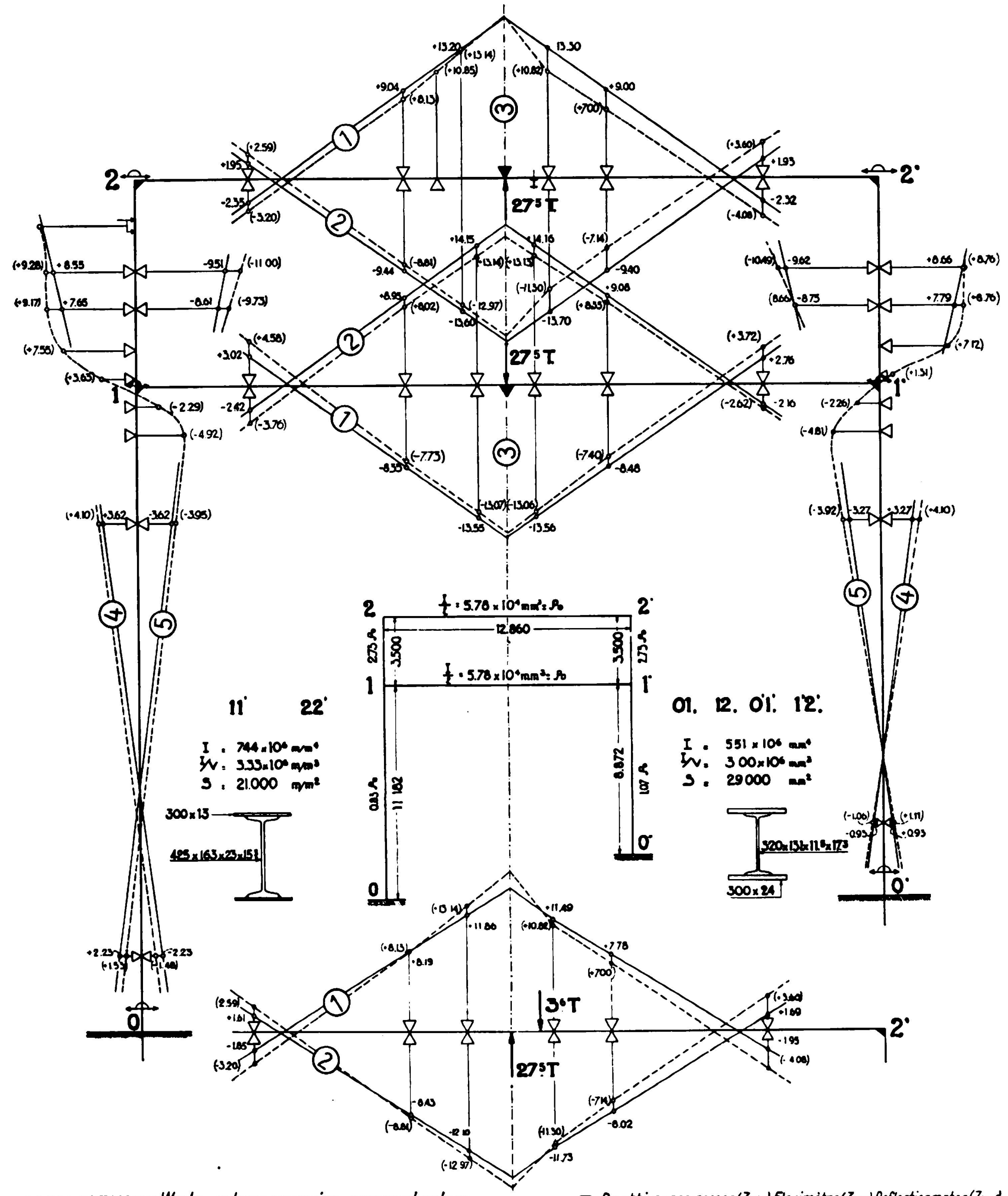
Point	Calculated values	Measured values	Differences in %
1 1′	31 mm	24.91 mm	19.7
2 2′	33.45 mm	23.88 mm	28.6

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

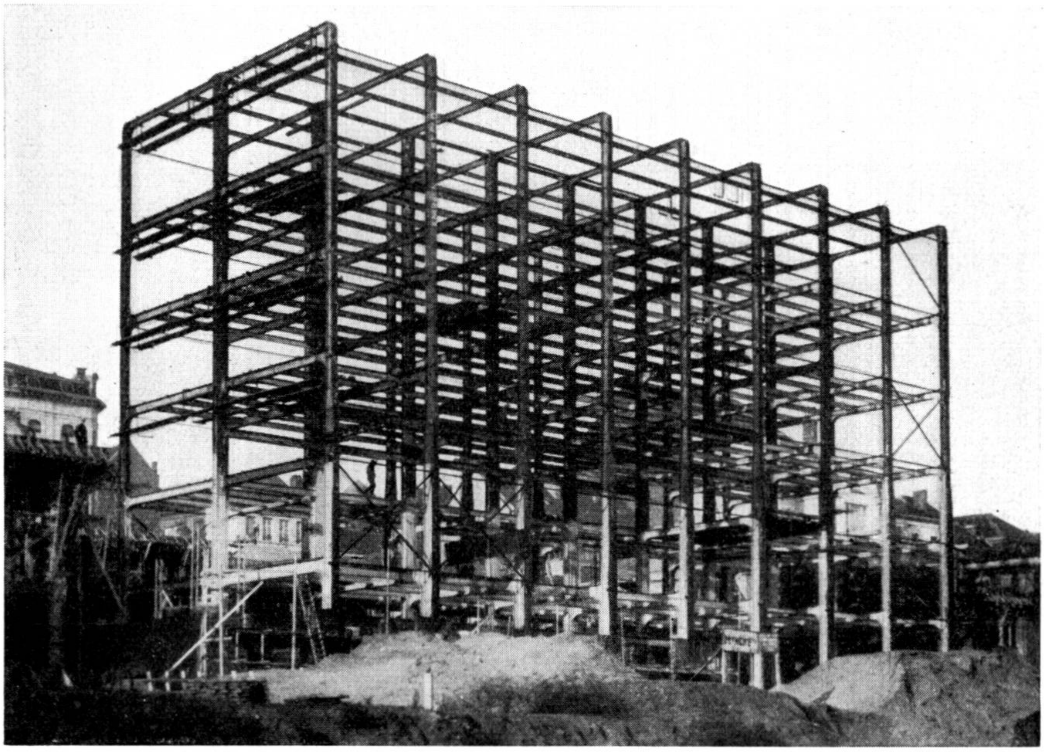

Fig. 7.

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.

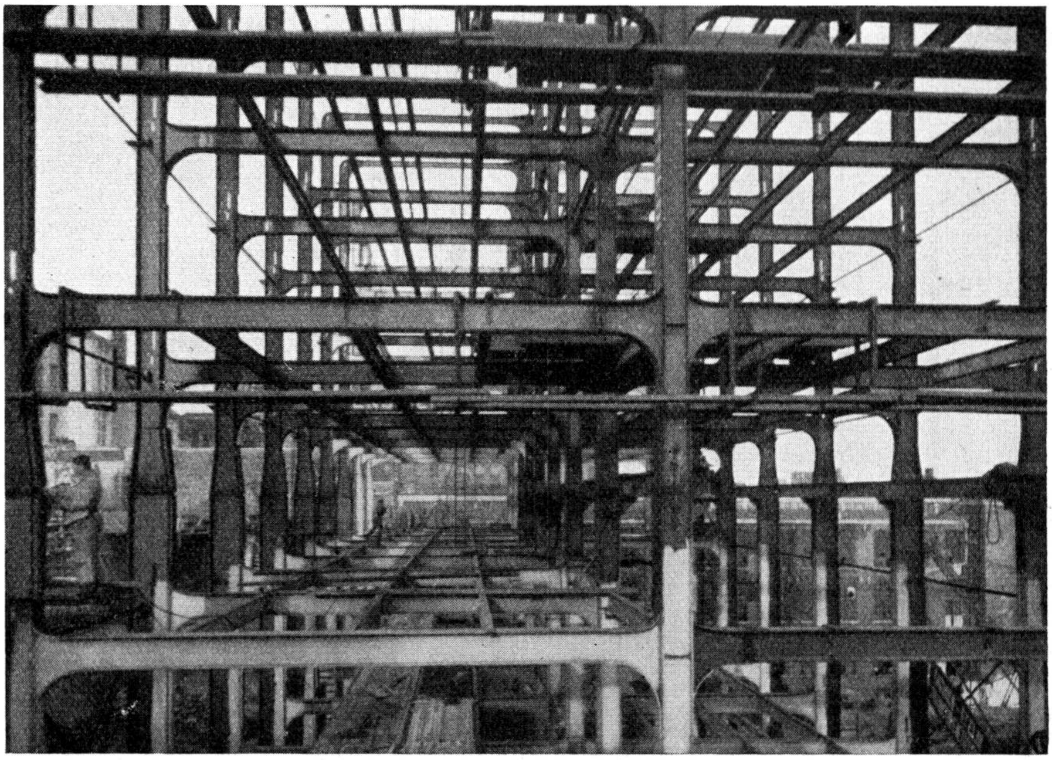

Fig. 8.

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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