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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.

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VII a

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VIIa
General Report.
Generalreferat.
Rapport Général.

Dr. Ing. K. Klöppel,

Leiter der technisch-wissenschaftlichen Abteilung des Deutschen Stahlbau-Verbandes, Berlin.

During recent years the steady pursuit of greater economy in construction has been combined with a desire to allow due weight to aesthetic feeling in the design of our bridges and in the development and application of steelwork generally. The artistic side of bridge design is now so much stressed that not infrequently a solution conspicuously good in this respect may be preferred even when it is not the cheapest that would satisfy the problem. To say this is not to imply that the aesthetic aspects of bridge building have been neglected in the past; for it is to those bridge builders who were at once engineers and architects, and who enjoyed the most unchallenged freedom of choice in their work, that we owe the existence of bridges of a aesthetic excellence beyond doubt and beyond the reach of change of fashion. We can only wonder at the boldness of the engineers who built long spans like the Britannia bridge and the Weichsel Bridge in Eastern Germany in the middle of the last century. Nor, in our present recognition of the primacy of the artistic side of bridge design, must we underrate the achievements of those engineers who, swept along by the rising flood of statical development which marked the years before and around 1900, envisaged their main objectives as mastery over statically difficult systems and in the choice of those structural forms which called for the smallest quantities of materials. The difficulties of that period — now almost at its end — were inherent in the designers' preoccupation with statical science, and they led, inevitably while they lasted, to a divergence between the engineer and the architect, the origin of which is now too readily forgotten.

This consideration is relevant when considering many of the old lattice bridges (such as the classical cantilever construction by Gerber over the Main at Hassfurt) which, though their "mathematical" form has become out of date, still stand as valuable and noteworthy monuments in the history of great bridge construction. From this intermediate phase of vigorous statical development we have now won through to a fortunate position in which we are enabled by the high standards attained in metallurgy, rolling practice, steel erection and statical procedure to subordinate the purely technical problems of bridge engineering to such refinements as (for instance) the choice of uniform and effective spacings for the stiffeners on the outside faces of plated girders, providing any additional

necessary vertical stiffeners on the inner face together with those horizontal stiffeners which may be particularly effective under conditions of heavy bending stresses.

Once again the eye is ready to recognise the beauty of clear-cut forms, and hence the reviving trend of co-operation between the engineer and the architect. But such co-operation is no longer understood to imply merely the addition of architectural trimmings; the architect now lends his aid in the actual detailing of steel bridges (as for instance in so arranging cantilever footways that they appear as a lateral closure of the bridge). In this it is right to acknowledge an important step — perhaps even a decisive step — forward in much debated ground; a step which marks the final escape of steelwork from that past period in which the architect looked upon steel as a medium foreign to his art, wherein he would build only with reluctance, and wherein his early efforts were often unpropitious. It was, however, a condition of this recent advance that the

architect should consent to some initiation into the fundamental statical and constructional problems of the engineer.

The aesthetic value of a bridge is conditioned — apart from subjective aspects — by the primary laws of design in nature — namely, eurhythmy and symmetry. The fulfilment of these laws, which will give recognition to builders for all times, lies in the employment of the simplest forms of construction. In this matter beauty and suitability are one; always provided that suitability is understood as meaning not merely the solution of the problem with the least possible use of material but its solution against a background of general understanding of the way in which a structure operates — an understanding which is, in fact, much more wide-

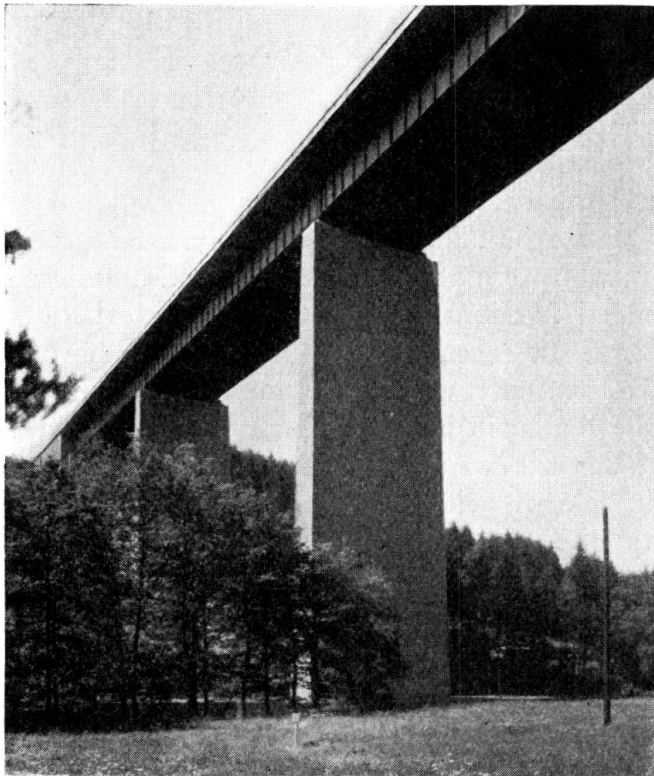


Fig. 1.

Reichsautobahn bridge at Siebenlehn.

spread than the engineer is disposed to admit. It is logical enough, therefore, that while the predominant type of bridge on the Reichsautobahnen is one of extreme simplicity advantage is being taken of the present development of technique to utilise economical forms of design of the beam type with the roadway above the girder, even in spans where arch bridges or compound forms of girder would previously have been preferred. In Germany there exist some masterly examples of this type of bridge, as for instance the Mangfall bridge at Darching (see Fig. 10, page 1348 in the Preliminary Publication). Later, in constructing

the viaduct near Siebenlehn (Fig. 1) the slender concrete piers were encased in natural stone, their vigorous pattern and varied tint being in pleasing contrast to the restful flatness of the rigid line of the steel superstructure. Another new development in the arrangement of steel bridge supports in the form of plate webbed portal frames (see Fig. 17, page 1355 in the Preliminary Publication). These supports give a paramount impression of boldness and slenderness which particularly suits the woodland setting of the bridge, and at the same time they serve to express some sense of the ease with which the interplay of forces is regulated and the great reserve of strength inherent in the steelwork.

Where it is possible through organic design of a deck girder bridge so to increase the constructional depth of the main girder that a trussed lattice construction becomes possible, and where this solution is not ruled out by the ver-

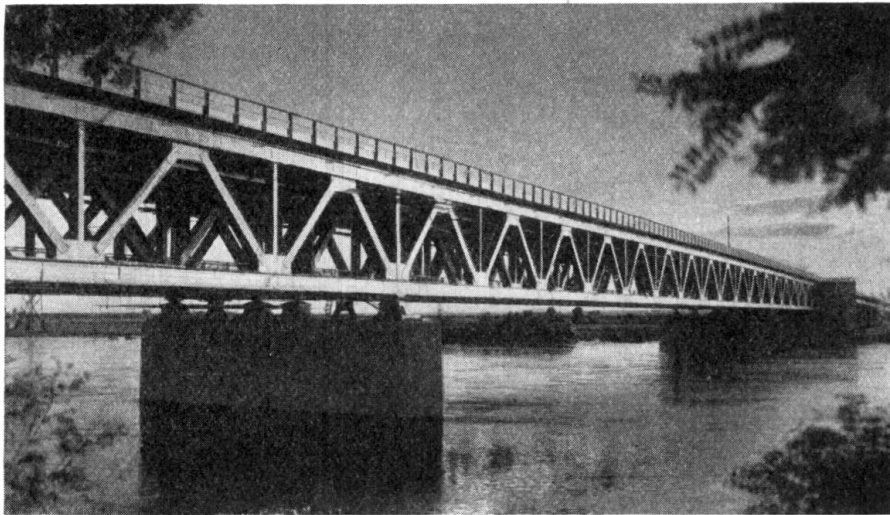


Fig. 2.

Reichsautobahn bridge at Hohenwarte.

tical and horizontal dimensions of the whole, the use of a truss should be contemplated not merely with a view to economy in material but also on account of its aesthetic effect which is in no way inferior to that of a plate webbed deck girder (see Fig. 2). It is generally and wrongly assumed that architects are flatly opposed to the use of trusses, whereas the fact is that they appreciate the varied expressiveness of this type of design and exult in the ornamental way that the interplay of forces is mastered therein. It is in this very choice of harmonious arrangement in the design of a truss — a choice always to be made with due regard to the scale dictated by the surroundings a matter too often neglected in the past — that co-operation between the architect and the engineer may hope to achieve its happiest results. In a suitable situation there should be no hesitation even in reverting to the use of fine meshed lattice work, for the well known objections to this form of construction no longer hold good, and it is one which can be made to harmonise excellently with a beautiful wooded landscape. The truss is the original form of steel bridge construction and the one which best lends itself to calculation, and there could be no justification for engineers ceasing to

develop this form of design. For long spans, in cases where the main girder must necessarily be placed above the roadway and where a trussed form of girder is, therefore, essential, its design has long been brought to a masterly degree of perfection (see Fig. 22, page 1358 of the Preliminary Publication).

Among the other basic types of steel superstructure the open-webbed arch with a suspended roadway is one which, during the last decade, has been used with advantage over very large spans of the order of 500 m, as for instance in the case of the Sydney Harbour Bridge. Here, just as was once the case in contemplating bridges of 100 m span, the impression of boldness tends to make one overlook the successful aesthetic effect. It is true that with spans of about 300 m slender plate-webbed arches also produce a very satisfactory aesthetic effect. In cases where the constructional depth is very small and the possibility of placing

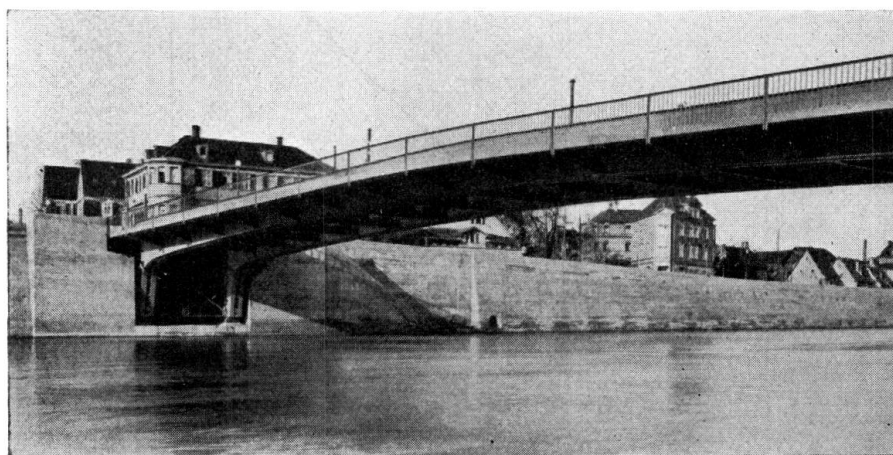


Fig. 3.

Wilhelm Bridge over the Neckar at Cannstatt.

the main girders above the roadway is limited by considerations of visibility, so that beam bridges with more than two main girders are ruled out, the true arch with a suspended roadway should again find scope for adoption over spans of less than 100 m. A recent development in this field has been the *Langer girder* (a particular type of bow-string girder, or beam reinforced with arch), presumably the result of difficulties in foundations.

The Swedish Mälarsee bridge, partly of welded construction, will be well remembered mainly on account of the instructive international competition of which it formed the object. In this design steel bridge engineers were offered the attractive (but unfortunately rare) problem of bridging across a wide sheet of water by means of an arch with an overhead roadway (see Fig. 1, page 1327 in the Preliminary Publication). Many of the details, also, are pleasingly successful (see Fig. 4, page 1329 of the Preliminary Publication) including the tubular supports, for which the use of welding proved particularly well suited. Welding may, indeed, be the means of reintroducing the use of tubes in bridge work to an important extent.

For small spans where the maximum possible rectangular opening is desired, as for instance over navigable waterways, a recent development has been the *rigid*

frame (see Fig. 3) in place of the arch, and there are many difficult problems from the points of view of construction, traffic or hydraulics to which no other satisfactory solution could be found than this space-saving type of steel bridge.

Finally, a brief reference must be made to *suspension bridges*. In America bridging problems reach an extreme which nothing but the suspension bridge is capable of solving, and it is there that the engineering world is now being staggered by the construction of the largest bridge in the world, namely the cable suspension bridge over San Francisco Bay with the prodigious span of 1,280 m (see Fig. 4).

The attraction of suspension bridges, both to the general public and more particularly to structural designers, is justification enough for the endeavours



Fig. 4.

Golden Gate Bridge.

that have been made in recent years to reduce the cost of such construction so that it shall be suitable even for the smaller spans encountered on the European continent and be competitive with other types. By improving, amplifying and simplifying the application of the "deflection theory"¹ — a method of design which takes account of the fact that elastic deformations relieve the loading of the stiffening girder and lessen its deflection — notable savings have been realised even in the case of suspension bridges of 200 m span with ground anchorage (see the original design for the Reich bridge at Vienna in Fig. 2, page 1278 of the Preliminary Publication). This form of calculation is true to the facts; in the case of pure arch bridges of long span it implies indeed an increased consumption of material (though the increase is small compared with suspension bridges), but when the span exceeds about 350 m it implies a reduction in the moments of the stiffening girder by more than 30 %, and a reduction in the deflection by as much as 50 % by comparison with the approximate method of calculation. If account is taken also of the economies that can be realised through using lighter roadway construction (reduction in the ratio of

¹ H. Bleich: Berechnung verankerter Hängebrücken (Springer, 1935). — F. Stüssi: Publications. I.A.B.S.E., Vol. 4, 1936. — W. Blick: V.D.I.-Zeitschrift, 1933, Vol. 77, No 34, p. 921.

live to dead load) there is ground for expecting a more frequent adoption of small suspension bridges, and the likelihood of this would be increased if it were possible to reduce the proportion of cost incurred in the cable.

This point may serve as a transition to considering questions of economics and development in steel construction; with special reference to the use of *high tensile steel* (a material to which many beautiful girder bridges owe their slender appearance) as well as the development of *welding* and the resulting use of *light construction*.

Recent fatigue tests² indicate that results obtained in usual static tests provide no reliable criterion for the fatigue resistance of various kinds of steel and

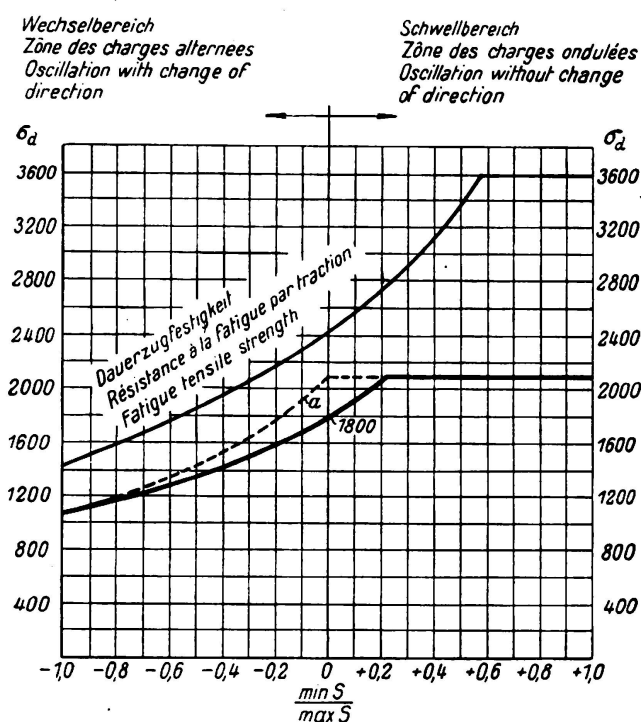


Fig. 5.

Permissible stresses according to the German Regulations for the Design of Steel Railway Bridges (Heavy Traffic), and tensile fatigue strength of riveted members in St. 52/44. Curve a applies when max S is compressive.

subject to the same considerations as those which entail a reduction in that quantity, in railways bridges, within the range of alternating stress and of the lower part of the range of pulsating stress. By the same reasoning there would appear to be scope for its use in road bridges and in building frames.

The applicability of results obtained on small specimens in a pulsator testing machine to the structural arrangement and conditions of stress in an actual bridge is problematical. Even though we know that the frequency of loading, and perhaps also the length of the intervals between traffic, are of no importance

structural members. Under repeated, non-alternating stress, the amplitude of stresses in St. 52 is not much greater than in St. 37, while on the other hand the suitability of St. 52 for fatigue stresses is indicated even in cases where the initial stress exceeds the yield point of St. 37. Hence in the German regulations for railway bridges the use of St. 52 is contemplated mainly for members in which the initial stress is considerable (as for instance on account of dead load), which is the usage suggested by the shape of the curve correlating permissible stresses with the ratio of maximum to minimum stress (see Fig. 5). This is the reason why originally St. 52 was developed for very large bridges, for in these the conditions of stress are such as to render an increase in the permissible stress of St. 52

² Klöppel: Gemeinschaftsversuche zur Bestimmung der Schwellzugfestigkeit voller, gelochter und genieteter Stäbe aus St. 37 und St. 52. Tests to determine the tensile surge load strength (tensile fatigue strength) of solid, perforated and riveted bars of steel 37 and steel 52. Stahlbau 1936, N° 13/14, p. 97. And other references.

within the range which concerns us, many questions nevertheless remain unanswered. Lack of space prevents going closer into these here, though they are questions which must exert a decisive influence over the further development of the regulations. It will merely be stated that experience shows the dimensions prescribed by the regulations to be well on the safe side as judged by the results of fatigue tests carried out to date.

The concepts hitherto recognised such as yield point, ultimate strength, elongation at fracture, and reduction in cross section, are inadequate to explain the differing behaviour of the available steels under fatigue stresses, or many other observations which affect the suitability of designs — for instance, the circumstance that the effects of concentrations of stress increase with the size of the specimen, even though geometrical similarity and the same flow of forces is preserved. These are questions relating to the fundamentals of steel construction which can be cleared up only by reference to the *mechanics of materials*, and their use as criteria for the safety of a structure makes it necessary that they should be understood both in the sense of the mathematical theory of elasticity, and in that of general statical, constructional and practical application to the purpose in view. It follows that the mechanics of materials are an indispensable study for the steelwork engineer. He alone is in a position to determine which of the results will enable the known lacunae in the principles of steelwork construction to be filled. He must become familiar with how the mechanics of materials depend on physical data, while recognising that resistance is limited by susceptibility to corrosion and flow and is effected by concentrations of stress due to the presence of notches. In order to keep step with improvements in our methods of calculation it is necessary that our knowledge of the actual resistance of materials should be increased, and from this point of view research on the phenomena of plasticity acquires an additional though indirect importance beyond that appertaining to it in Question I of this Congress.

The welcome saving in weight represents no more than the first of the possibilities of increased economy in steel construction that can be realised through the adoption of *welding*. In this matter we are at the start of a development which is destined to be revolutionary in so far as riveted connections are replaced by weld seams, for welding, where advantageous, will direct the trend of steel construction towards monolithic forms of design which are of the very essence of light construction. The road leading to this goal is doubtless a difficult one, but the extraordinarily rapid development which has already taken place offers encouragement to follow it: a development illustrated in the application of completely welded sitejoints in plate-webbed railway bridges (as in the Rügendamm bridge) and of butt joints in main girders of large bridges on the Reichsautobahnen (as, for instance, at Rüdersdorf). The increasing reliability of butt welds gives grounds for supposing that butt-welded tensile and bending members may be subjected to higher specific fatigue stresses than riveted members, because the permissible stress thus escapes the considerable notch effect due to the rivet holes, well made butt welds with smooth surfaces over the seam being practically free from this notch effect. This requirement is satisfied in rolled beams only when they are made free from holes at heavily stressed parts.

In nearly all fields of application of metals constructional development is tending in the direction of *light construction*, an idea which has become a stimulus for new and revolutionary endeavours. On no account should this be understood as a mere makeshift in construction. It should rather be regarded as a higher stage in the utilisation of material, attained through careful structural arrangement taking due account of the inherent properties of the material, which renders the structure lighter and, at the same time, more resistant and stiffer than existing forms of design intended for the same purpose. Thus a reduction in the amount of material is made the means of improvement in quality. In reviewing the application of steel this fact must not be overlooked, particularly since development of certain elements of steel work is proceeding in the direction of light construction. Such elements will be primarily steel plates, flats, tubes and weld seams. In so far as it is not possible to publish tables of carrying capacity based on experiments, the calculation of increased capacity conferred by the monolithic character and three-dimensional action of these structures must be effected by simplifying the more difficult methods of calculation and permitting approximate methods of design to be used under the regulations. The successes attained in aircraft construction as, for instance, with torsion-proof rigid box shaped girders and posts or load carrying partitions from corrugated sheets, encourages the idea that in steelwork, also, a more economical type of construction may be realisable which shall be no less resistant than the present form. To set limits to the course which these developments are likely to take in the long run, on the basis of experience now available, must be abortive, for the economical application of light construction entails far reaching alterations in method of fabrication. Even to-day large edge bevelling machines are making their appearance in steel fabricating shops, and the adoption of cold pressed sheets (a trend which may greatly reduce the importance of rolled sections) is drawing attention to steels with increased ageing resistance. Thin rust-proof steel sheets, also, are attracting more and more attention. The stimulus towards overcoming the difficulties of the transition stage may be found in the certainty that light construction is destined to open out new fields of work.

In road bridge construction a conspicuous feature is the development of *light weight decking*, the most notable line of endeavour in this direction being the use of a fine mesh grillage which behaves statically as if it were a slab, and which has been rendered possible through the introduction of welding. It is true that in the suspended plates (trough plates) we already possess a form of supporting member which makes the use of plates possible in steel construction. But these are exposed to certain disadvantages which form an obstacle to reducing the weight of the roadway, namely the considerable weight of the concrete filling and also, the risk of corrosion which arises if trough plates are used over larger areas than has hitherto been the practice. Attempts are being made considerably to increase the area of trough plates with a view to economising in the number of girders of the deck construction, and, although it is not found that this increase gives rise to difficulties from the point of view of strength, there is no general regulation governing calculation of these plates. There is a risk, moreover, that under unfavourable conditions of loading (especially where the

trough plate with its relative lack of stiffness is used) the concrete may separate from the plate and allow water to enter. This matter will be mentioned again later.

With a view to reducing the dead load of the concrete filling in the suspended plate the expedient suggests itself of adopting a floor construction of flat sheets which are made to function as membranes, as has already often been done in movable bridges. These call for a very close spacing of the longitudinal joists. Another solution to the problem is offered by the use of welded grid plates often known as "cellular steel floors" (Fig. 6); the strength and stiffness, as deter-

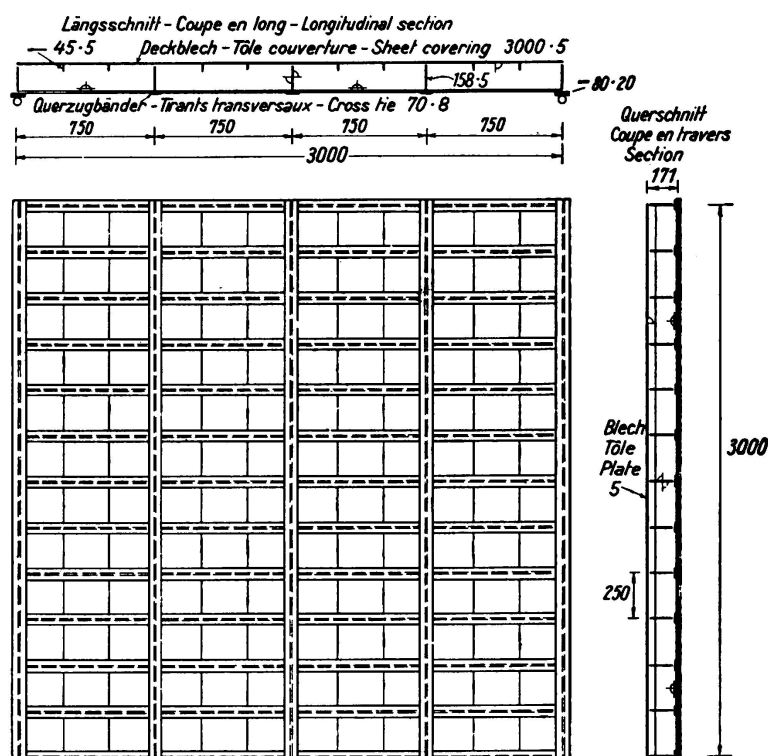


Fig. 6.

Tests on bridge floor members.

mined on specimens similar to those illustrated, are excellent, but the cost unfortunately is very high, as may be appreciated from the larger number of welded connections. Decking of this type has already been used with economy for field track crossings over the Reichsautobahnen. The limited amount of constructional depth required for the light-weight decking also enables economies to be realised in access ramps and other ancillary works, and in the case of small crossing bridges this may turn the balance of economy in favour of steel as against other methods of construction. The form of specimen shown in Fig. 6 (designed for a load of 110 kg/m^2) lends itself to a simple method of calculation applicable to these members which depends, in the first place, on an understanding of the load distributing action of the flat member in relation to its fixation around the edges and to the constructional details. In this way it is possible to arrive at a simple, realistic and economical method of calculation, similar to that employed for reinforced concrete slabs reinforced in two directions, and to incorporate this

in the regulations. The first of the relevant experiments at Stuttgart arouse the hope that justification may be found for adopting the approximate formulae for slabs stiffened against torsion which is laid down in the German Reinforced Concrete Regulations, seeing that in these steel members the necessary stiffness is easily

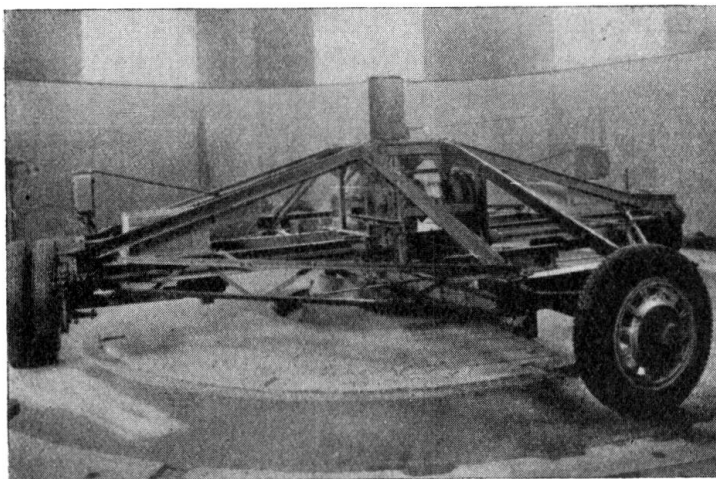


Fig. 7.

Rotary apparatus for testing plated steel floor.

attainable by means of suitable connections and flat bracing bars welded to the corners of the plate. It is obvious that these members represent an important new type of load bearing element in steel construction, which is designed to be widely employed especially in building work.

Since the pavement, the sand and the reinforced concrete protective and insulating layers weigh something like 450 kg/m^2 , which is about one half of the

total weight of a reinforced concrete slab floor, investigations were embarked upon to ascertain whether requirements might be met by covering the flat sheet

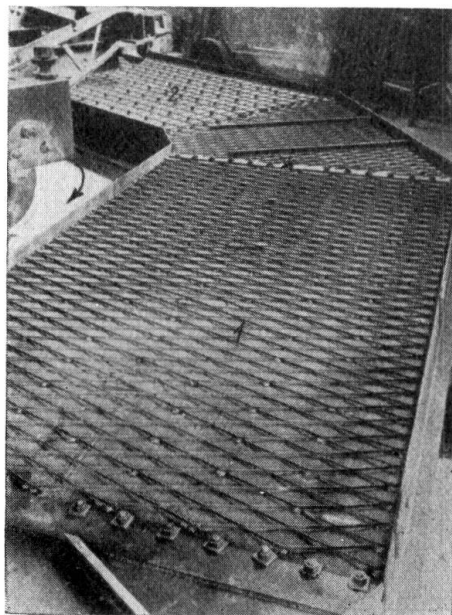


Fig. 8.

Expanded metal welded onto steel plates.

merely with an intermediate layer 4 cm thick of asphalt overlaid by a wearing course 3 cm thick of hard asphalt containing basalt chippings, stone dust and quartz sand as fillers, the tests being carried out on a circular road testing machine (Fig. 7) at Stuttgart. The flat plates were given a varying degree of stiffness from one section to another so that under the same load the deflections varied from 0.11 to 0.6 mm. After a test period which would correspond to four years of intense road traffic it was found that no separation had occurred between the covering material and the plate where these two were connected by expanded metal (Fig. 8), neither was there any sign of corrugation even for the softer plate. Powerful weathering effects were imitated in the tests and these disclosed the remarkably high resistance of the covering. In this way it is possible to construct light-weight decking with a depth of 80 mm weighing

280 kg/m^2 not including the longitudinal girders (Fig. 9). It is obvious, of course, that adequate stiffness and watertightness must also be ensured.

A number of other interesting ideas for light weight deck construction are now

being tried out. In America steel grillages of the open type have been used as roadway deckings, and light non-ferrous metals are also being tried, but the question arises whether these will possess the necessary fatigue resistance.

When it is remembered that the weight of the decking, including the longitudinal girders, may vary between 1,050 and 300 kg/m², the necessity for

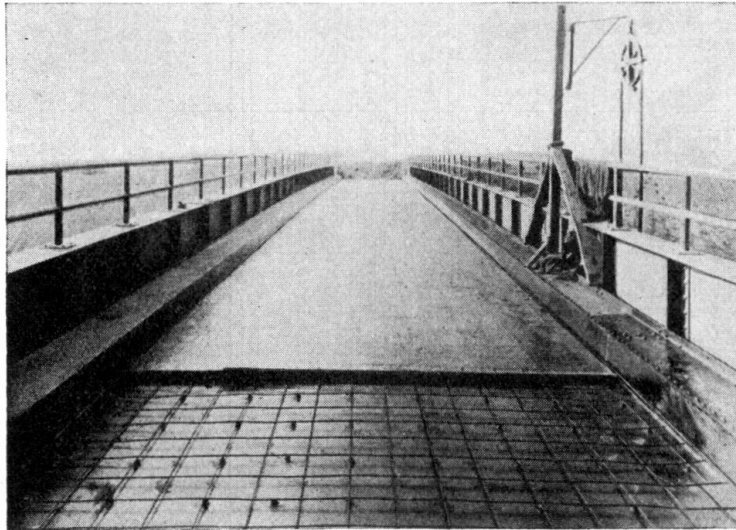


Fig. 9.

Light floor — Schaper system.

these endeavours becomes indubitable, even though the cost of the lighter construction may often be considerably higher than that of the type of decking hitherto in use. The economic problem is by no means settled merely by answering the question whether the reduction in weight and cost of the main girders

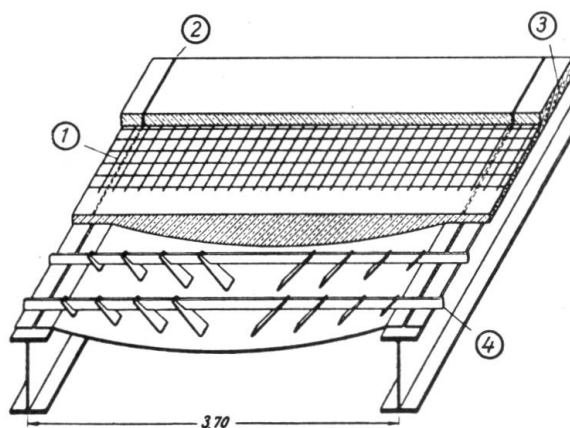


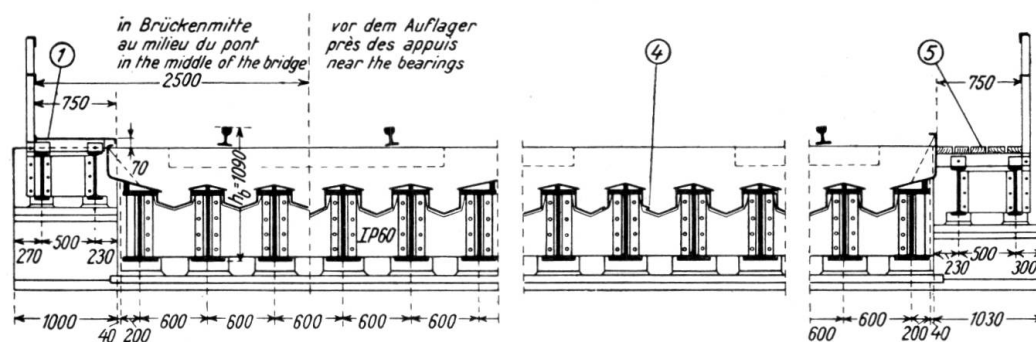
Fig. 10.

Stiffened arch-plate.

- ① Intersecting reinforcements on steel plate.
- ② Longitudinal joints over the longitudinal girders.
- ③ Concrete containing 300 kg of cement per m³ deposited in a single layer.
- ④ 80 × 8 mm flat steel bars at 45 cm centres.

through adopting a lighter decking balances the greater cost of the latter, for, as already indicated, many cases arise where the reduction in constructional depth may be the deciding factor. Moreover it is relevant to point out that the economy of this new method of construction depends on the use of special workshop

methods which are far from having been fully exploited. The result is that present-day practice has made greater use of the long span arched plates and trough plates already mentioned, but in order to reduce the weight of the concrete filling these plates are being made with a very flat arch by comparison with earlier practice.



- ① Footbridge of chequer plate covering.
- ④ Arrangements of joints.
- ⑤ Footway with timber planking.

Fig. 11.

Section through the bridge.

Where arched plates are used it is necessary to provide an obstacle to any movement between the concrete filling and the plate itself, which may be done by stiffening the latter with blades welded on to it, as for instance was done on the arched plates of 240 cm span and 8 mm thickness used in the Sulzbachtal



Fig. 12.

Schröder-type floor.

bridge (Fig. 10 herein, and Fig. 16 on page 1354 of the Preliminary Publication). Horizontal flat bars are provided to ensure that only vertical reactions can occur. It was found by experiment that these compound elements, when calculated in accordance with reinforced concrete theory, gave a factor of safety against statical breakage of at least 8 when subjected to the prescribed loading. The weight, not counting the longitudinal joists, is 470 kg/m², this low value

being explained by the omission of a waterproofing and protective course. The wearing course is a single layer of concrete placed in situ immediately over the concrete filling, 40 mm thick. Special attention must be paid to the impermeability of the concrete, and drainage of the surface is ensured by a cross slope of 1.5 ‰. Such a decking will also function as wind bracing, and its

use replaces the shuttering which is otherwise necessary in building a reinforced concrete floor.

Further experiments must show how far it is necessary to stiffen the plates where these take the form of *trough plates* (3.5×5 m) under a covering of

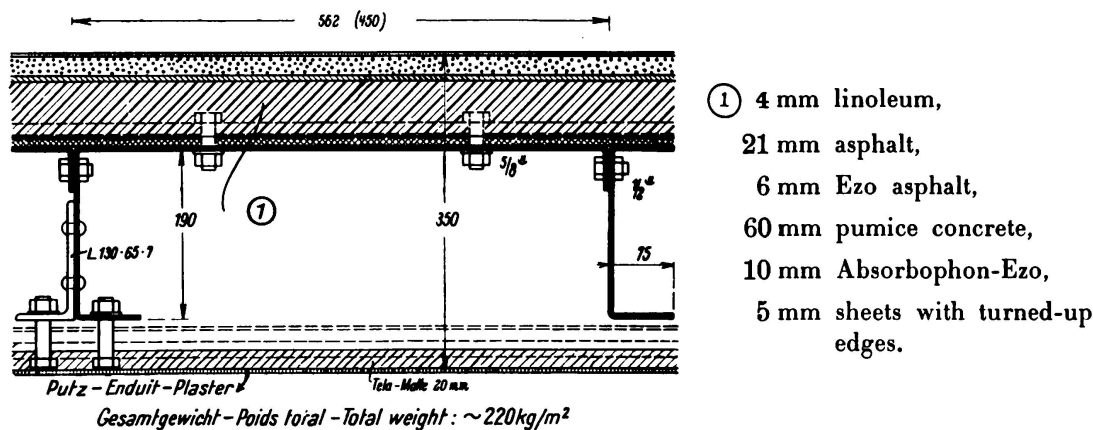


Fig. 13.

Light steel floor.

special materials with the possible addition of a wire netting, and with the use of a special working procedure. The question of troublefree maintenance of the continuous layer of concrete over the floor girder may be a matter of some considerable difficulty unless a gap has been left for the purpose.

The form of bridge construction illustrated in Figs. 11 and 12, in which there are no cross girders but the joists are *embedded in concrete*, may also be looked upon as a form of lightweight construction. Measurements of deflection made on these joists and on the plates welded to them have shown that the joists and the plates co-operate somewhat after the manner of T-beams in reinforced concrete. It is to be hoped that further measurement of deflection will show whether in the design of a construction of this type it is permissible to assume some width, of plate as co-operating with the joists, or whether an increased deflection, say $1/700$ instead of $1/900$, may be allowed in the joists themselves.

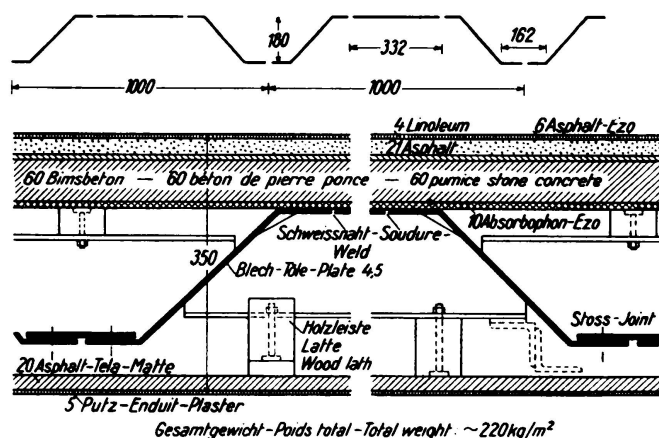


Fig. 14.

Light steel floor with doubly bent steel sheets.

In building work *light floors* as shown in Figs. 13 and 14 have been used, after careful tests under load and investigations of their acoustic and thermal properties have proved their suitability. The dead weight amounts to only about

220 kg/m² for a span of 6.50 m, a depth of 350 mm, and a live load capacity of 500 kg/m². The construction is readily adaptable to different conditions of loading and span by altering the thickness of the plate and the other dimensions,

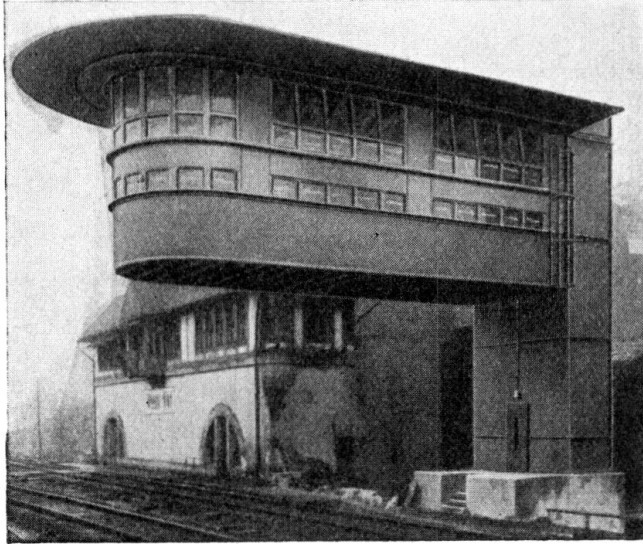


Fig. 15.

Signal box at Mainz station.

and this can be done very economically. The lightness of such a floor is an aid to new methods of constructing steel building frames with partial use of welding, and an example of this is provided by the two-storeyed signal cabin at Mainz (Fig. 15) in which the plating serves both as a curtain wall and as a cantilever girder.

As regards new design of aircraft hangars, reference may be made to Fig. 25 on page 1323 and Fig. 24 on page 1321 of the Preliminary Publication. In America suspended plates have been used for roofing over a bunker with the aid of wel-

ding.³ Fig. 16 shows an example of development in aircraft hangar construction in Germany, in which the self-supporting roof constructed of troughing with tie bars serves also as the upper flange of a truss which is well adapted to carry



Fig. 16.

Self-supporting roof formed of doubly bent steel sheets.

point loads, and the lower flange is provided in the form of a tie bar. Considering the advantages of a steel roof-covering the weight of this type of construction is surprisingly low. Surface structures (as, for instance, roofing over halls) which are stressed mainly in compression throughout their effective cross section, and which are kinematically sensitive, require special investigations of stability (theory of the second order) as relatively small differences between the ori-

ginal and the actual shape of such a system may have a decisive effect on its safety. Fortunately steelwork construction is not affected by the phenomena of deformation due to the material itself.

³ Stahlbau, 1933, p. 152, No. 19.

In the construction of large halls, as for instance the new airship hangars at Frankfort on Main and at Rio de Janeiro, the trussed form of construction still predominates.

Further attention has been paid to the co-operation of steel construction and concrete. In Switzerland small road bridges (Fig. 17) have been made with the upper flange of the main girders and the longitudinal girders embedded in the reinforced concrete slab: the combined action enables the longitudinal girders to be made lighter and also has a favourable effect on the natural condition of vibration of the bridge. There are also special cases (as where St. 52 is used) where the compound action serves to reduce the amount of steel otherwise involved in the larger girders necessitated by considerations of deflections. In Germany the Reinforced Concrete Regulations do not allow this co-operation between rolled steel joists and concrete to be taken into account unless the joists lie entirely in the tension zone, and apart from this the Reichsbahn insists on permanent accessibility of the rivets in the flanges, a requirement which operates in favour of welding.

In the design of steel building frames, the use of which has become particularly widespread in France and Great Britain in recent years and of which an example exists in the new construction of the Reichsbank in Berlin at the present time, it is permissible to allow in the design for the co-operation of steel columns with their concrete cores,⁴ and this results in increased economy. Here again the use of light flooring has proved very advantageous in certain cases. Finally a wide range of fire tests on loaded steel columns of full size, variously encased, yielded valuable results and have indicated that it is possible with limited means to attain a high degree of fire resistance⁵ in such columns (see DIN 4102, part I).

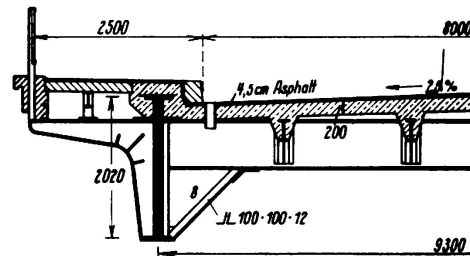


Fig. 17.

Bridge over the Limmat at Engstringen
(Switzerland).

⁴ Stahlbau, 1934, p. 59, Nos. 7 and 8; Zentralblatt der Bauverwaltung, 1935, No. 23.

⁵ Report on the XXIXth scientific meeting of the Reichsverein Deutscher Feuerwehringenieur (published by the same, Berlin, No 15).

VII a 1

The Aesthetics of Steel Bridges.

Formgebung stählerner Brücken.

L'esthétique des ponts métalliques.

F. Eberhard,

Direktor der M.A.N. Mainz-Gustavsburg.

Increasing attention is now being paid to the aesthetic side of steel bridge design and the occasion is opportune, therefore, to draw a few examples and counter examples from the wealth of present-day practice and to examine them critically.

The fundamental principle of all architecture is to reveal closely the purpose of the structure, and to the bridge builder this means that the passage of the



Fig. 1.

Road bridge over the Rhine at Neuwied.

roadway over an obstacle must be clearly indicated. The purpose is best fulfilled when the roadway is carried above the structure, but to repudiate every solution in which this condition is reversed would be to go too far. For instance, in the case of the Rhine bridge at Neuwied (Fig. 1) if the roadway were placed above the bridge, the access ramps would have to be several kilometres in length and would predominate instead of subserving the road itself, forming a whole which would appear as a disturbing excrescence from the flatness of the landscape. The design of the bridge should not stop at the abutments but should extend to the continuation of the roadway, for only thus will the bridge be made to merge itself into the landscape, whether mountainous or flat.

As examples of typical bridges in flat country we may take the taut and slender looking plate girder structure which carries the Reichsautobahn over the Main at Frankfurt (Fig. 2) and the road bridge over the Elbe at Meissen

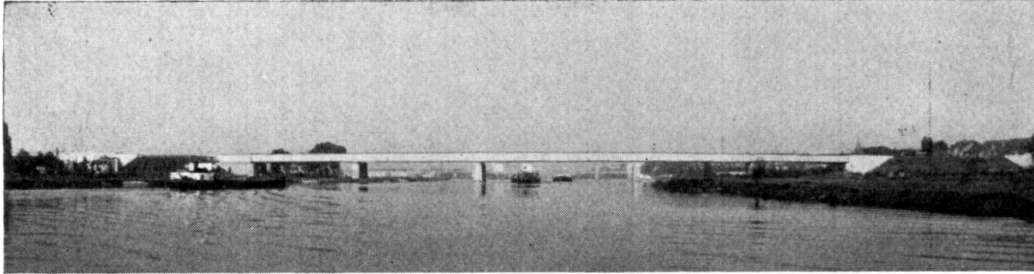


Fig. 2.

Motor road bridge over the Main at Griesheim.



Fig. 3.

Road bridge over the Elbe at Meissen.

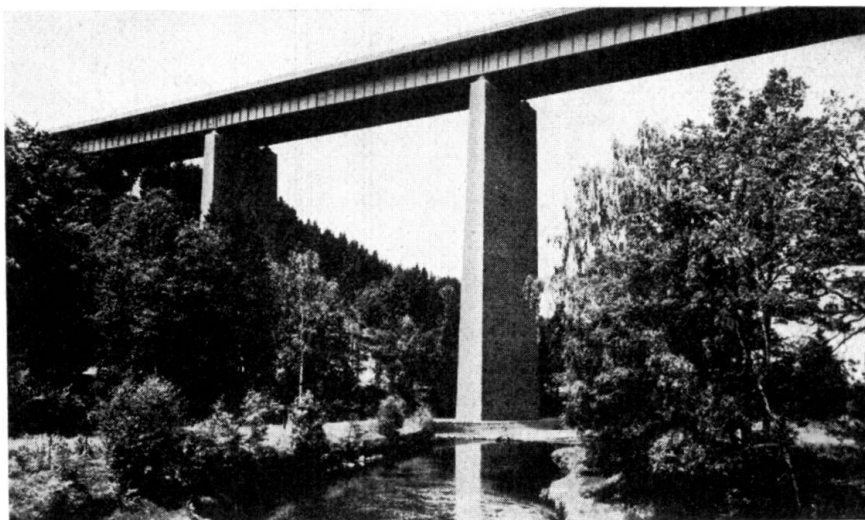


Fig. 4.

Reichsautobahn bridge at Siebenlehn.

(Fig. 3). The latter, with the picturesque Albrechtsburg in the background, illustrates that there is no need to imitate antiquity in order to harmonise the old with the new.

That the plate girder bridge is equally capable of merging into a mountainous landscape may be seen from the Autobahn bridge over the Freiburger Mulde (Fig. 4).

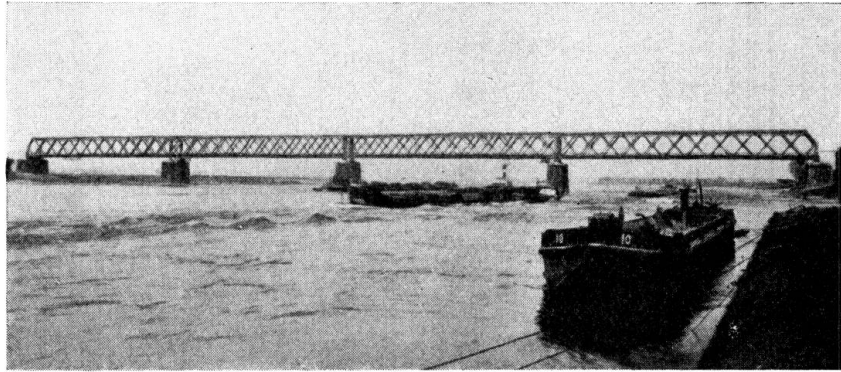


Fig. 5.

Railway bridge over the Rhine near Wesel.

A lattice girder, also, is capable of effective incorporation into either a flat or a mountainous landscape. This is illustrated by the railway bridge over the Rhine near Wesel (Fig. 5) and by the railway bridge near Freudenstadt (Fig. 6). The determining factor is not whether the bridge is solid or open webbed and not whether the country is flat or mountainous; — art consists in laborious and

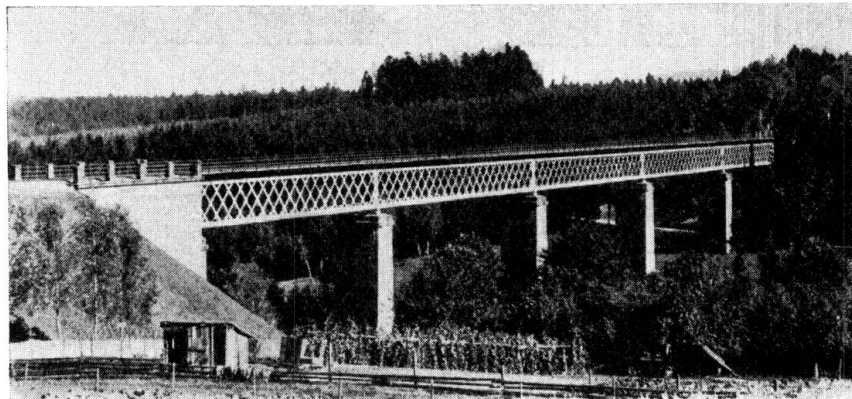


Fig. 6.

Lauterbad Bridge at Freudenstadt.

careful study to settle the height of the girders, of the spans, of the proportions of the piers and of the width of the bridge and its corbels.

In high bridges a parallel girder may very well be combined with an arch, for in such a case the arch predominates as long as an equilibrium between the abutment and the arch is obvious to the eye. The bridge over the Mälarsee near Stockholm, otherwise pleasing (Fig. 7), suffers from the defect that too little of the abutment can be seen.

The bridge over the North-East Sea Canal near Grünthal (Fig. 8) owes its appearance of boldness and elegance to the obviously right choice of the positions of the springings; at the same time the prominence given to the



Fig. 7.

Road bridge over the Mälär Lake near Stockholm.

roadway line serves to mask the disadvantage of the road passing from above to below the arch.

The result is much less happy in those arch bridges where the arch rises above the roadway but fails to free itself therefrom. Thus Fig. 9 shows the

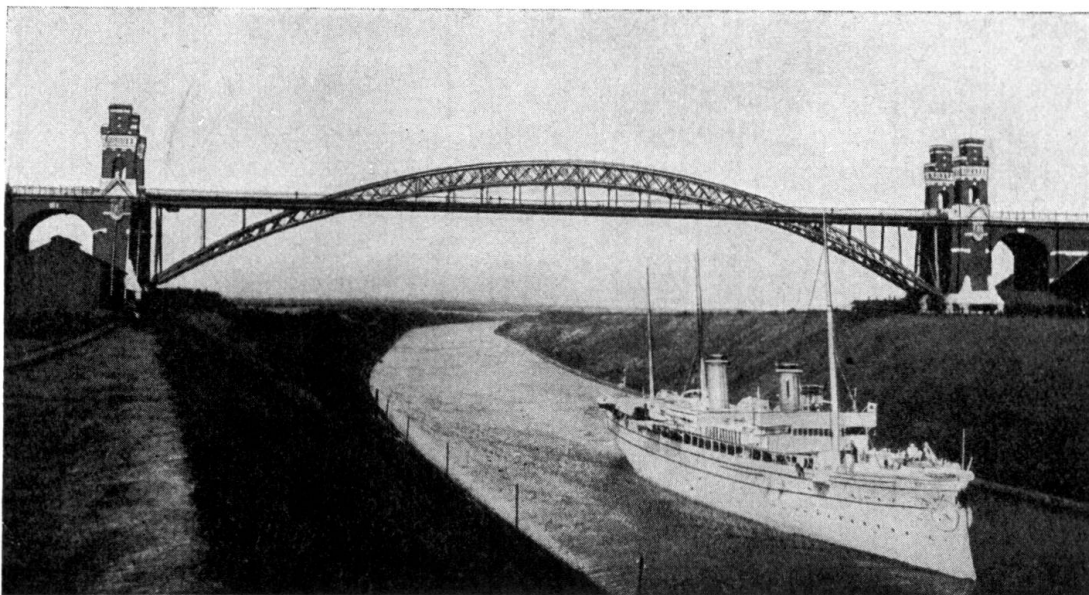


Fig. 8.

Road bridge over the North-East Sea Canal at Grünthal.

road bridge at Coblenz before reconstruction and Fig. 10 indicates how its appearance has since been improved by lifting and widening the floor. Whereas, in this bridge, all three openings are equal at 96 m, in the case of the Rhine bridge at Mainz the span increases from 87 m at the banks to 102 m in mid-

stream; but few people looking at the bridge will be aware that its faultless harmony is the result of this increase in the spans. Fig. 11 shows the bridge after its reconstruction. Originally the abutments carried toll houses and the

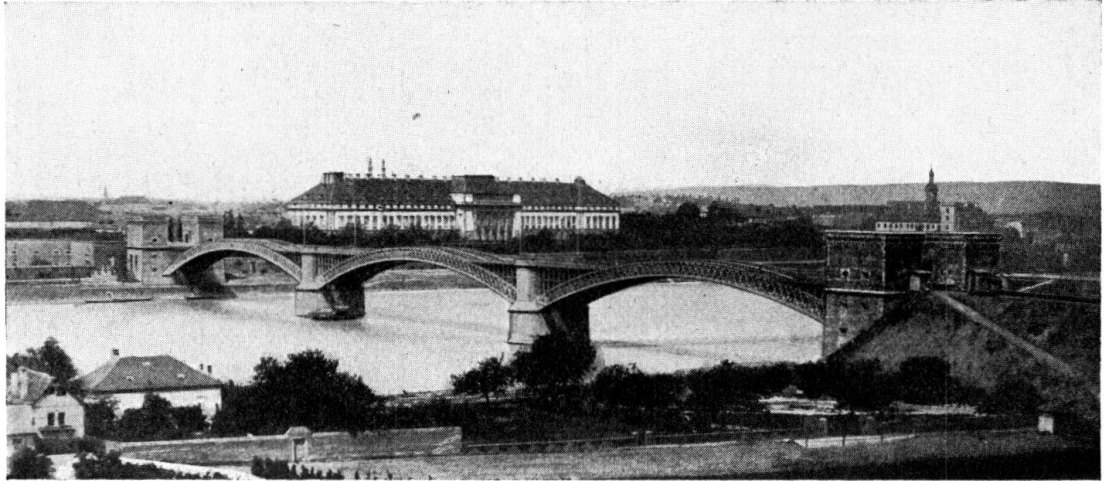


Fig. 9.

Road bridge over the Rhine at Coblenz before reconstruction.

piers ornamental pilasters which broke up the pleasant swing of the roadway and also its connection with the access ramps. The horizontal lines intersected the vertical, with mutual interference. To-day, however, the roadway, which it is the purpose of the structure to carry, clearly dominates the whole.

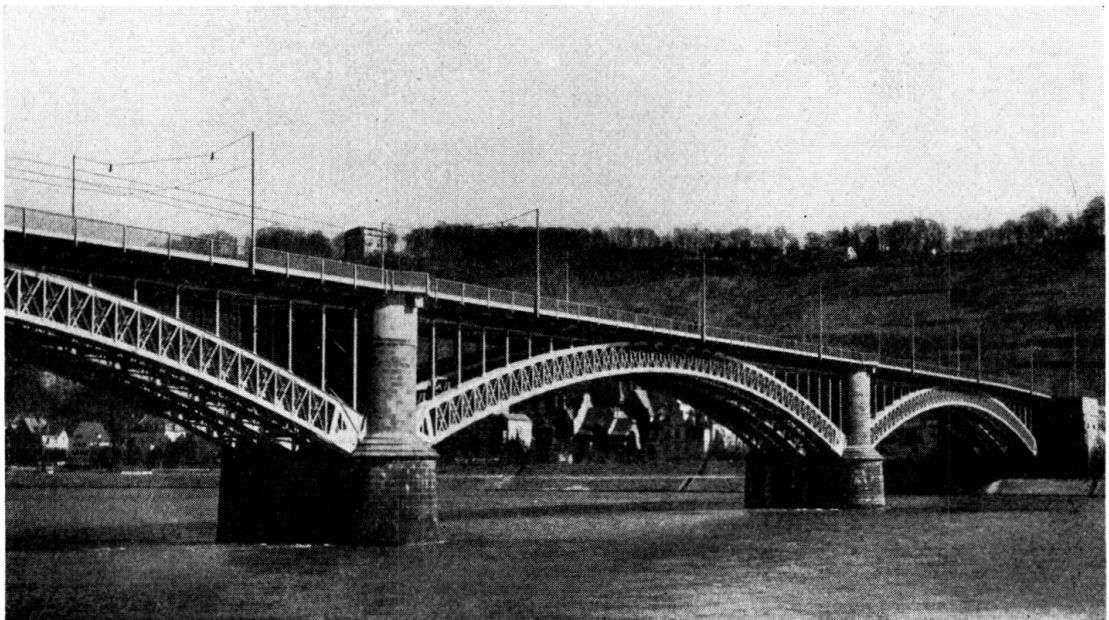


Fig. 10.

Road bridge at Coblenz after reconstruction.

It is not always that nature offers the bridge builder valleys anywhere nearly symmetrical in cross section, but where symmetry is lacking there need be no hesitation in exploiting the lack — which may, indeed, be a source of special

charm. The design for the Autobahn bridge over the Saale near Lehesten (Fig. 13) is characterised by the fact that the span increases towards one side and that the roadway is on a gradient, making it possible to increase the height of the girder to correspond with the increase in span.

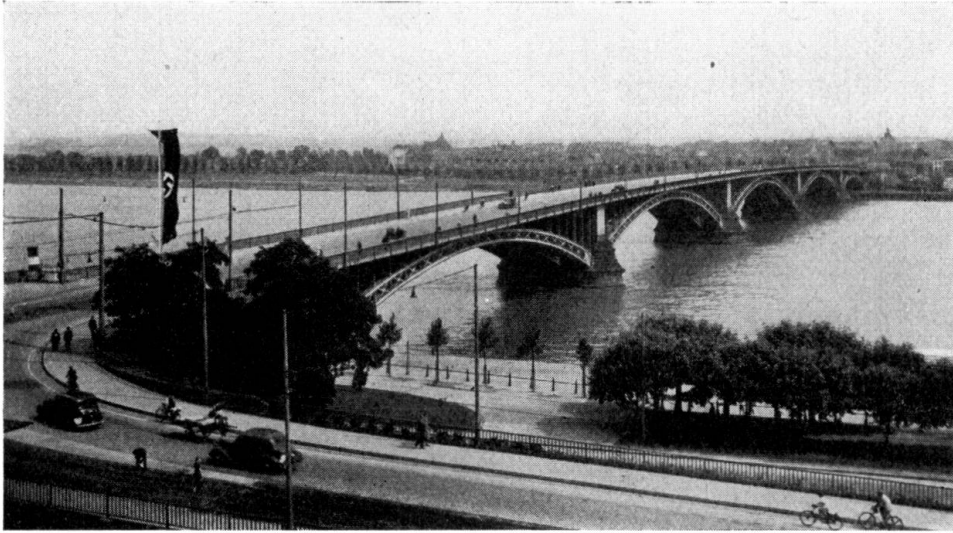


Fig. 11.

Road bridge over the Rhine near Mainz after reconstruction.



Fig. 12.

Road bridge over the Rhine near Mainz before reconstruction.

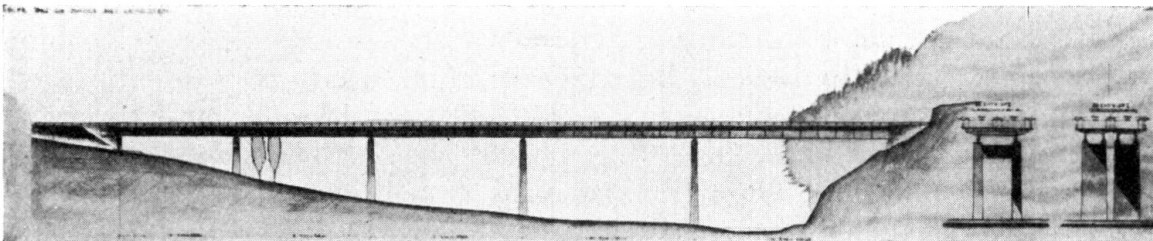


Fig. 13.

Design for a motor road bridge near Lehesten.

When the designer is prevented by conditions laid down by other agencies from following his own rules the problem of creating a harmonic structure becomes almost insoluble. The Rhine bridges near Maxau and Speyer (Fig. 14) are subject to unfortunate conditions of this kind, for the curvature of the river causes the navigation channel to lie on one side and the river must

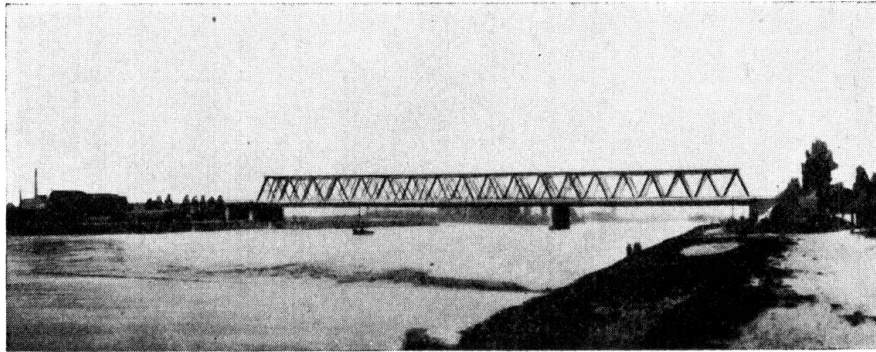


Fig. 14.

Bridge over the Rhine at Maxau.

be placed accordingly; to an onlooker, however, this lack of symmetry is incomprehensible as the reason for it is not apparent. It was found that the dominating horizontal lines of a beam bridge would emphasise the lack of symmetry least, whereas an arch (Fig. 15) would have emphasised it more.

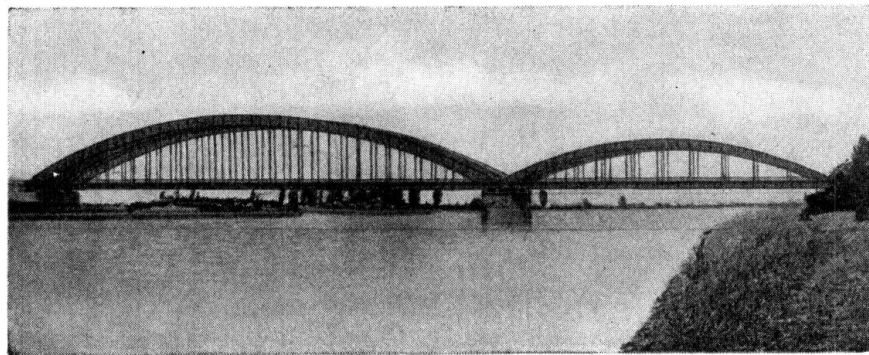


Fig. 15.

Design for bridge over the Rhine at Maxau.

Bridges over several openings must be unified into a closed line. Note the difference in this respect between the old lattice bridge at Cologne (Fig. 16) — despite the disturbing superstructures over its piers — and the jagged outline of the bridge across the Danube at Floridsdorf (Fig. 17).

Another way that the unity of line of a bridge may be completely broken is through lack of repose in the outline of the booms, as in the case of the Hassfurt bridge over the Main (Fig. 18). The road bridge at Wesel (Fig. 19), again, loses something through the circumstance that the booms rise above the river piers, and the disturbance caused by the dropping of the line of the roadway at these same places is a further defect.

How neat the old girder bridges with thin members could be has already been illustrated in the lattice bridge at Cologne. Hence the revived use of solid webbed girders, and the quest for means of filling the spaces in trussed girders so as

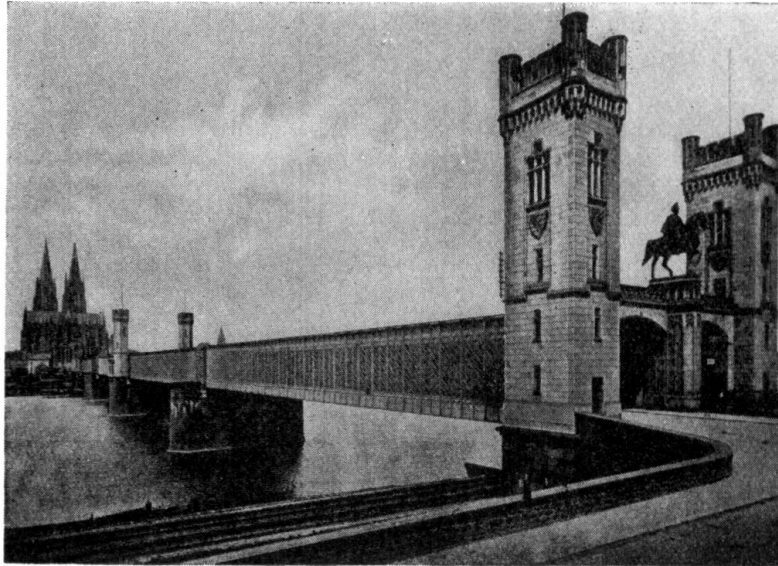


Fig. 16.

Old railway bridge over the Rhine at Cologne.

to recapture the attractiveness of the old fine-membered lines. The reintroduction of the truss with simple diagonals only is a step in this direction.

The truss with diamond (rhombic) lattice is more effective in that the girder takes on more the appearance of a wall. In the Rhine bridge at Mannheim

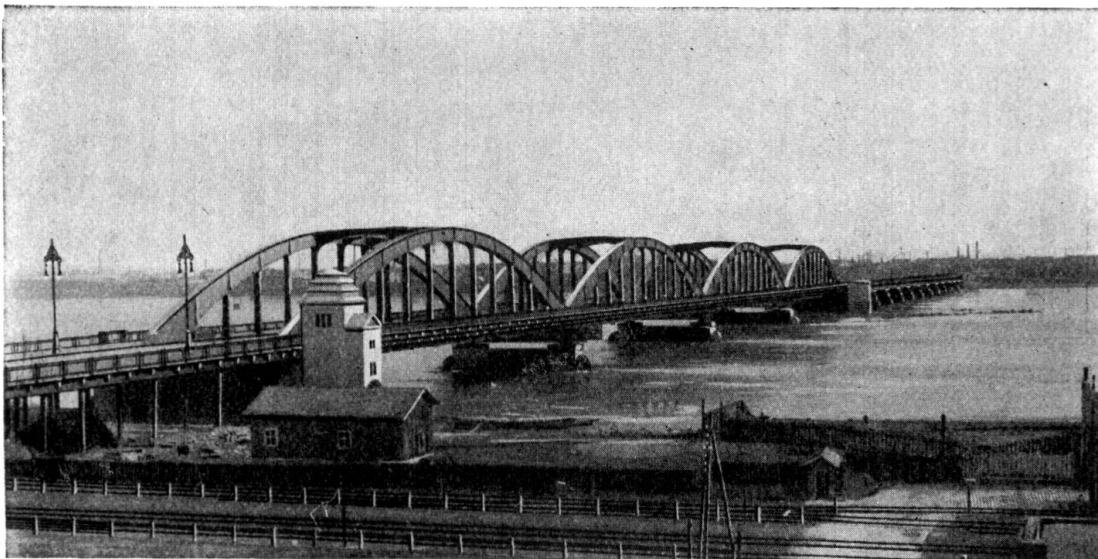


Fig. 17.

Bridge over the Danube at Florisdorf.

(Fig. 20) the end posts are carried to the full height of the girder. This gives a somewhat harsh appearance and it is better therefore, to finish off the end

posts at half the height as in the railway bridge at Wesel (Fig. 21). An end portal with a kink in the uprights pleases neither the engineer nor the layman.

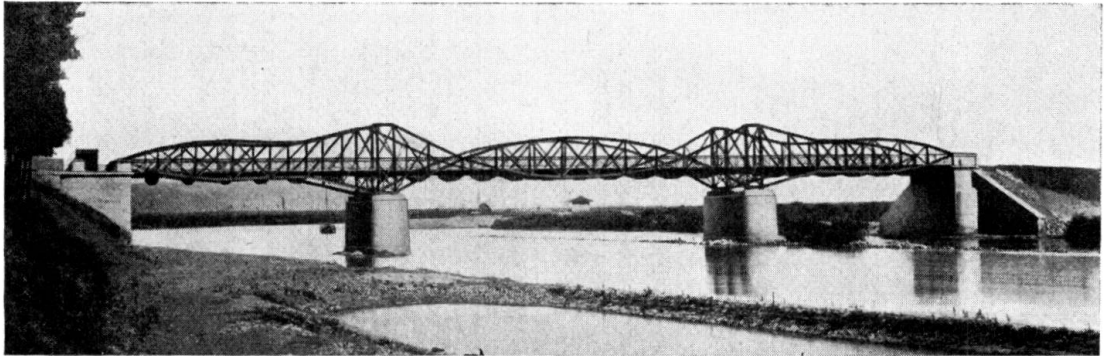


Fig. 18.

Road bridge over the Main at Hassfurt.

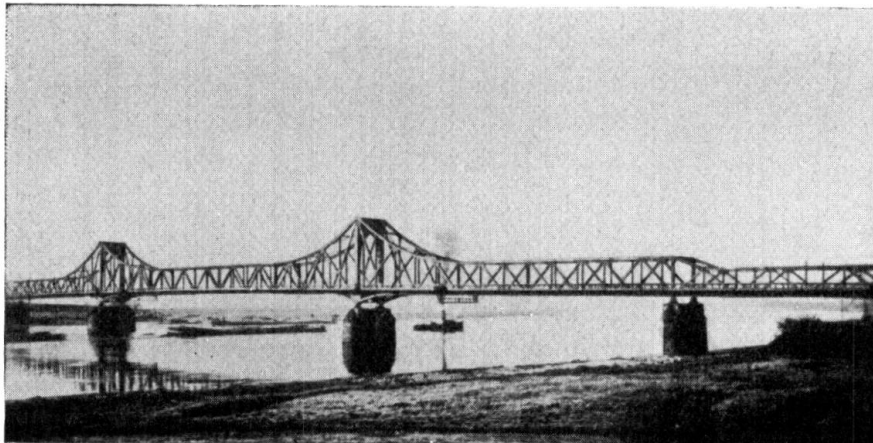


Fig. 19.

Road bridge at Wesel.

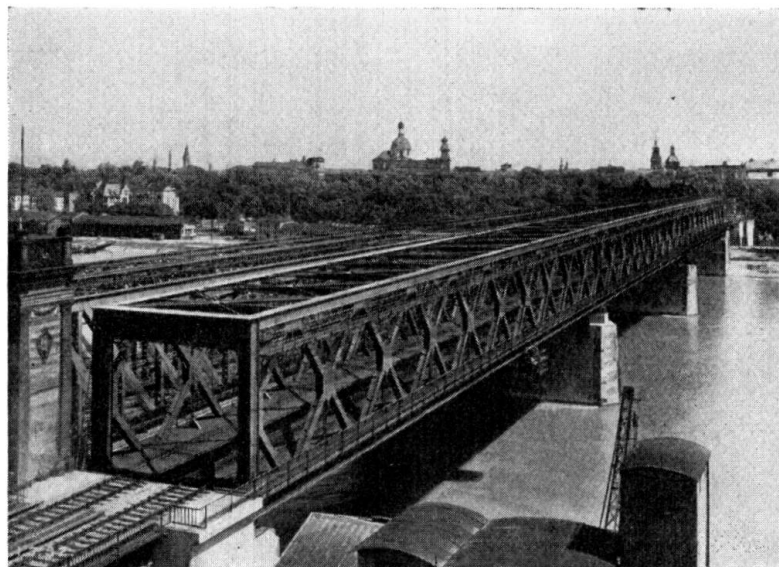


Fig. 20.

Road bridge over the Rhine at Ludwigshafen.

The search for a new way of shaping the panels of a truss is a problem of modern engineering which calls for solution through the combined efforts of the engineer and the architect. But the principle of combining a beam with an



Fig. 21.

Railway bridge at Wesel.

arch seldom leads to happy results — for how is the spectator to know that an arch with a tie has vertical reactions similar to a beam? And who is to answer his question whether it is the arch or the stiffening girder that is the



Fig. 22.

Footbridge at Oberschöneweide.

principal member? It is only in a girder bridge strengthened with an arch, when one principal opening is specially emphasised by the arch, that this type of bridge can be satisfactory. Fortunately we have learned to avoid arbitrary

mixtures of arches and beams, and opposing curves in the lines of the booms, while for such a structure as the footbridge at Oberschöneweide (Fig. 22) our technical terminology has not even a name. In the North Elbe bridge at Hamburg (Fig. 23) the roadway appears to the layman as if it were an afterthought incidentally suspended from the structural parts. Structures of this kind are largely responsible for the false idea, not yet everywhere dead, that a steel bridge

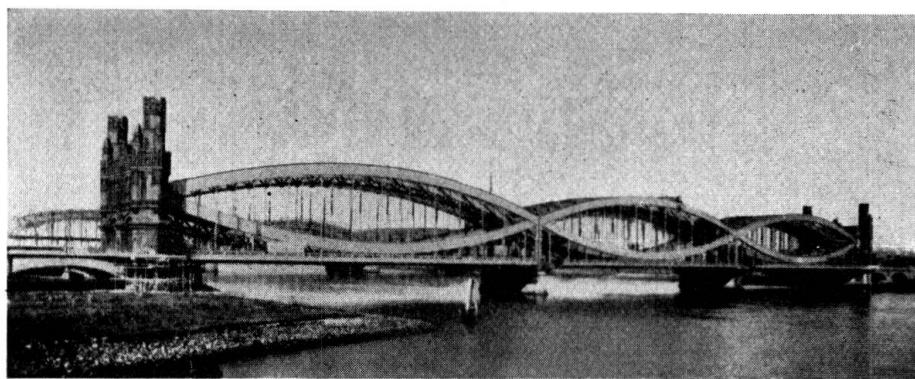


Fig. 23.

Bridge over the Elbe at Hamburg.

is no more than a makeshift for adoption where a massive bridge cannot be built.

Nor can a mixture of beam and suspension bridge give satisfaction. In the Main bridge at Bamberg (Fig. 24) the suspension boom, which is the most purely tensile member, has its central portion functioning as the upper flange

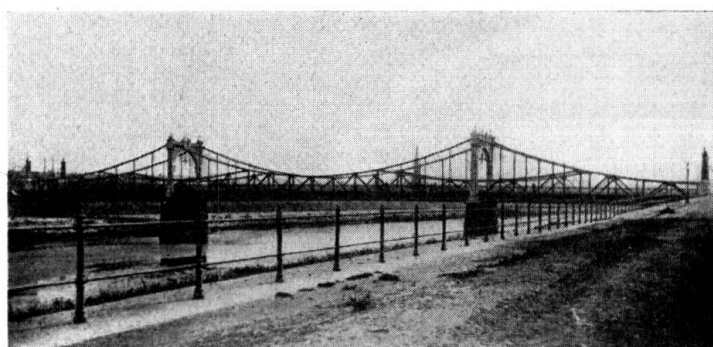


Fig. 24.

Road bridge over the Main at Bamberg.

of a beam, and therefore as a compression member: hence the confusion in the result, for no member ought to be called upon to serve two contrary functions at the same time.

The pure suspension bridge which the engineer is naturally inclined to use for long spans is all the more attractive in appearance because it is obvious that the towers carry the suspension cable, the roadway hangs from the suspension ties, and the stiffening girder carries the loads. How great are the constructional

possibilities of such a design is illustrated by the towers for the Philadelphia-Camden bridge (Fig. 25) and by the Rhine bridge at Cologne-Mülheim (Fig. 26).



Fig. 25.

Philadelphia. — Camden bridge.

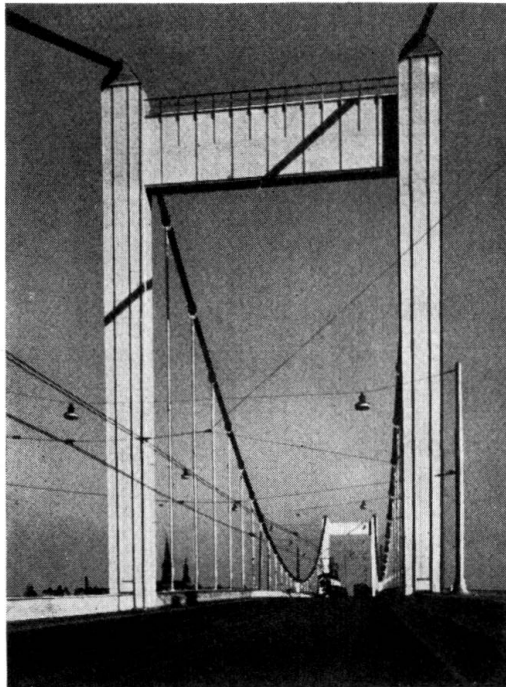


Fig. 26.

Bridge over the Rhine at Cologne-Mülheim.

The abutments and piers are a constituent part of a bridge, and it is often a matter of lively debate whether steel, masonry or concrete piers are to be preferred for these. In this respect the bridges for the Autobahn have given rise

to an entirely new problem on account of their great width. The problem is not yet mastered, though some attractive solutions to it are now on record such as the Sulzbach bridge (Fig. 27) and the bridge over the Kleine Striegis (Fig. 28).

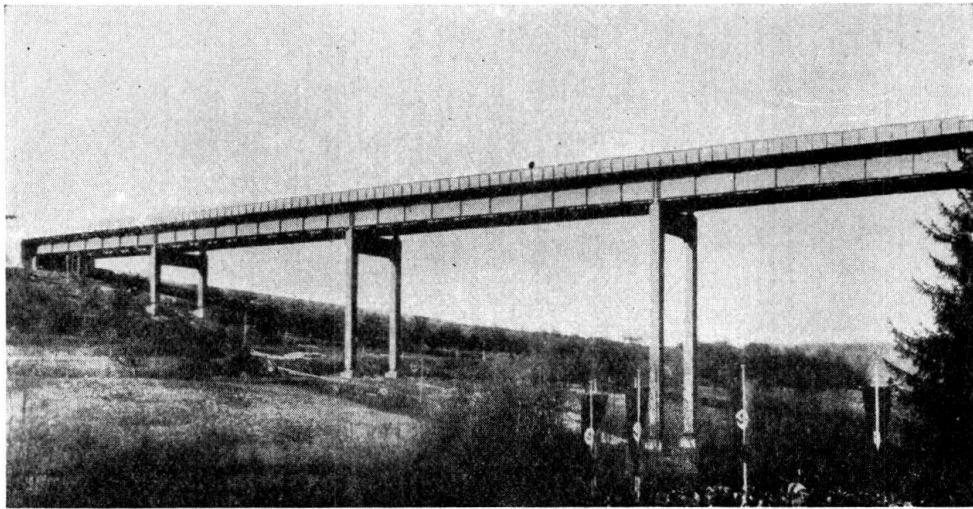


Fig. 27.

Motor road bridge over the Sulzbachtal.

The trend of design in steel bridge work is marked by a striving after clarity and truth in expression, and the problem of the bridge engineer is to choose from among those possibilities of statics and materials the ones which most naturally

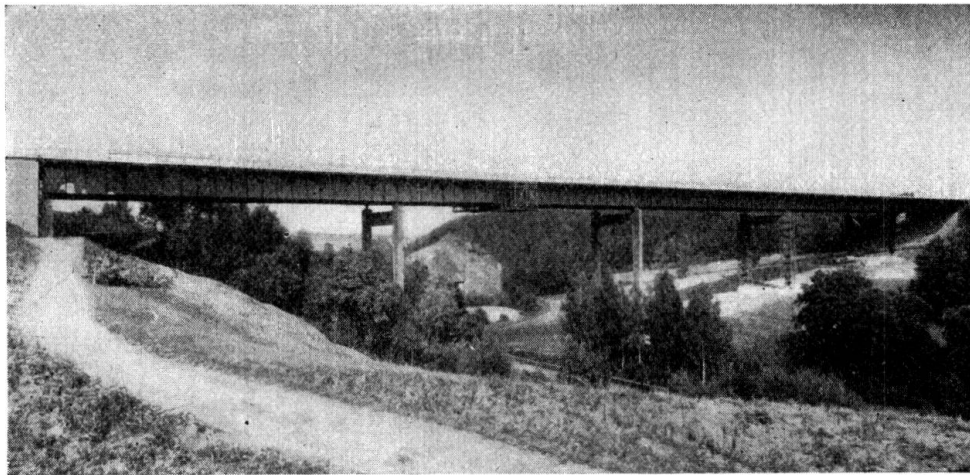


Fig. 28.

Bridge over the Kleine Striegis.

fulfil their purpose. A bridge is not a structure that stands alone but is part of a line of communication. It becomes a work of art when as a whole, and in all its members, it clearly expresses its purpose of carrying and guiding traffic.

VIIa 2

Unbuilt Bridges.

Brücken, die nicht gebaut wurden.

Ponts qui n'ont pas été construits.

Dr. M. Klö n n e,
Dortmund.

At meetings and in the literature, discussion is usually confined to structures which have actually been built, reference being made to their general arrangement, their constructional details and the advantages and disadvantages possessed by this or that type of work.

It is proposed here to direct attention to bridges which have never been built, or, more correctly speaking, to designs which have not been carried into effect at any rate in the particular circumstances for which they were drawn up.

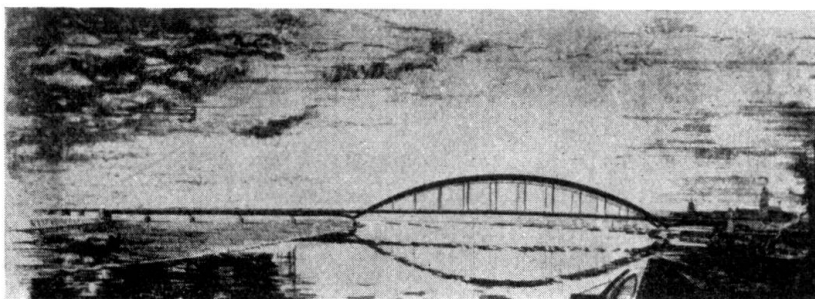


Fig. 1.

It will be suggested, moreover, that in the case of those designs which have gone no further than the design stage, it will often happen that either the external form or the underlying idea can successfully be taken up again later. The intention here is not so much to lament the tragedy of an idea being born before its time as to suggest to engineers that it is well worth while devoting attention to designs that have fallen by the wayside in competitions. Much thought of the highest quality is stored away in the prodigious labour applied to such designs by first class bridge builders, and it happens frequently that this may advantageously be drawn upon in other circumstances. This is true as regards both the design as a whole and the execution of its details.

The author wishes to declare at the outset his intention of quoting the actual names of the works in question, and he trusts that this will be excused

by his colleagues, as he considers such a course more useful than mere mention of the designs without closer designation. He also asks for indulgence in mentioning mainly works, to the design of which he himself contributed.

First a few examples will be given of the general arrangement of bridge design.

In the competition for the Rhine bridge at Cologne-Mülheim an arched design was proposed (Fig. 1) which, having a span of more than 300 m, would have resulted in a bold and remarkable structure. Perfectly feasible technically, it would have had the advantage of allowing motorists on the bridge an unobstructed view of the river (Fig. 2). For reasons which would take too long to explain here, a suspension bridge was chosen instead.

To carry the Autobahn over the Elbe at Hohenwarthe, the author proposed a suspension bridge (Fig. 3), the lightness of which would have harmonised excellently with the long lines of the approach spans (Fig. 4). Actually a lattice girder bridge was built instead.

For the Elbe bridge at Dömitz he suggested a truss bridge in which all the web members would have been struts of equal inclination (Fig. 5). As there were to have been only two main girders, the

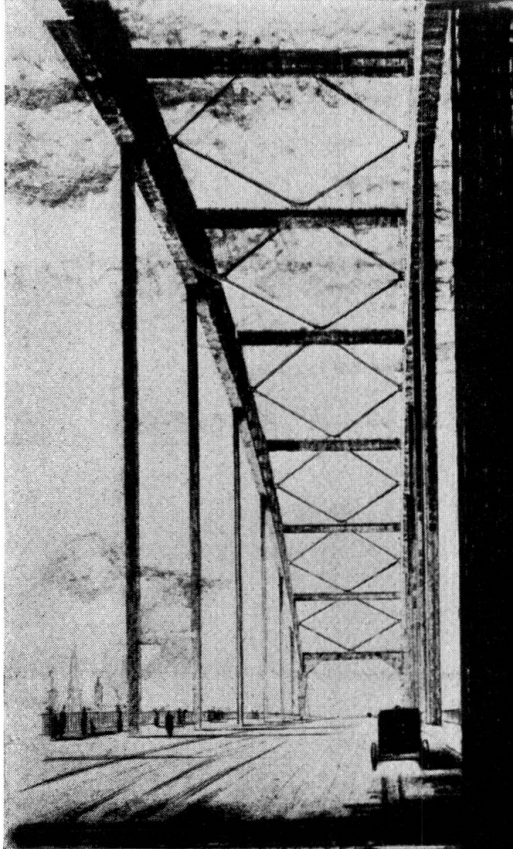


Fig. 2.

effect even of an oblique view through them would have been one of repose. The design actually chosen was a bow-string girder bridge.

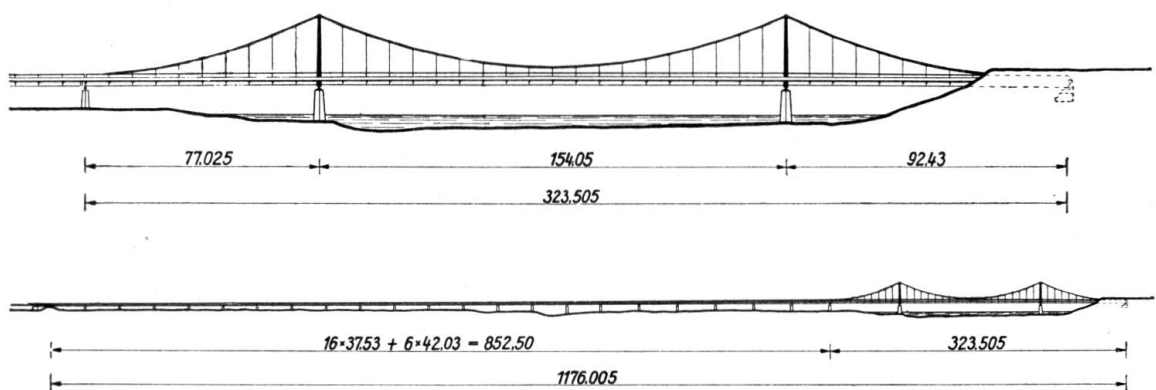


Fig. 3. (Dimensions in metres.)

This completes the circle from arch bridge to suspension bridge, from suspension bridge to trussed girder, from trussed girder to bow-string girder.

Although in these cases the wide-span arch bridge and the suspension bridge were not in fact carried out, the author believes that some day an opportunity will arise to make use of ideas embodied in these old designs and to apply these structural systems in similar cases elsewhere.

What is true of designs as a whole is also true, perhaps even more markedly, as regards constructional details. Here two examples will suffice.



Fig. 4.

In the construction of large viaducts with continuous main girders the recent practice has been to adopt tall steel hinged columns or hinged frames for the intermediate supports. In Figs. 6 and 7 a comparison is drawn between this form of construction and the use of slender concrete piers, designed for the Mulden bridge at Siebenlehn. The use of hinged portal frames of great height came originally from a suggestion made by the author in reference to the

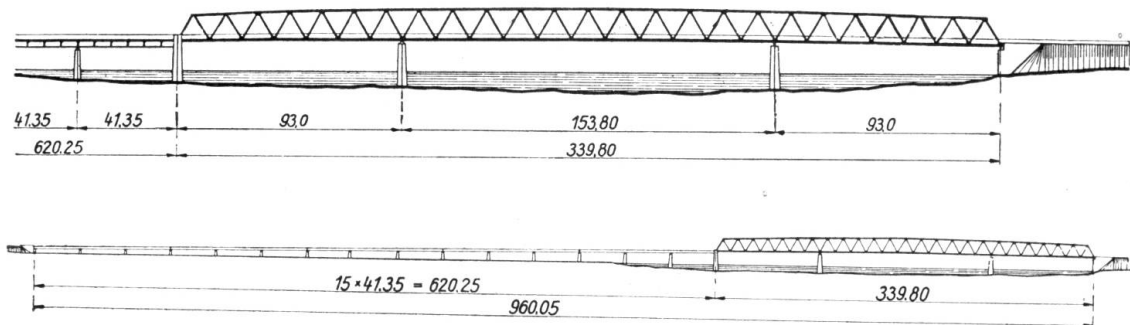


Fig. 5. (Dimensions in metres.)

Mangfall bridge, which on that occasion was not adopted. Figs. 8 and 9 are an elevation and a view from underneath of the bridge, showing the two alternative designs of solid-webbed and open webbed main girders.

The second example that may be cited in this connection is that of the overhead wind bracing. There can be no doubt that from an aesthetic point of view the Vierendeel form of bracing is very attractive, its plain cross members without diagonals giving an effect of repose. In 1934 the author proposed an

overhead wind bracing of this design for the Autobahn bridge at the Kaiserberg near Duisburg, but for statical reasons the deciding authority preferred a K-bracing. More recent bridges of this kind are in fact being built with Vierendeel

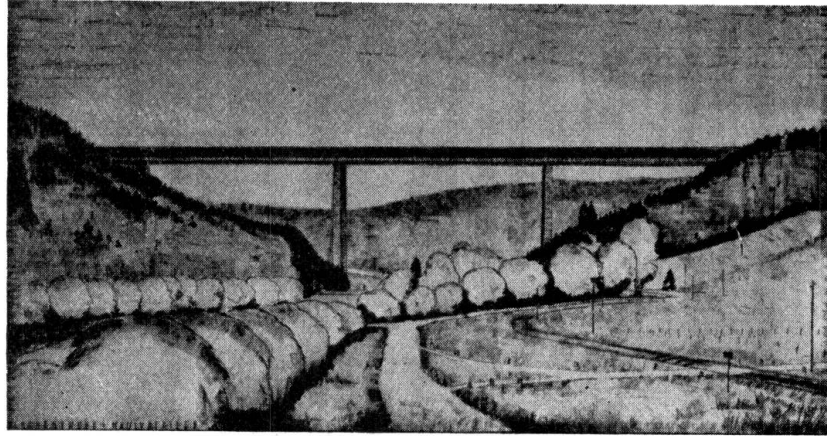


Fig. 6.

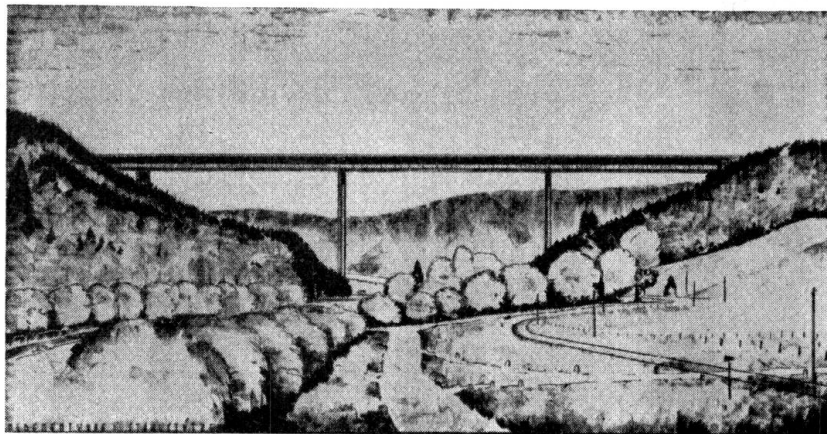


Fig. 7.

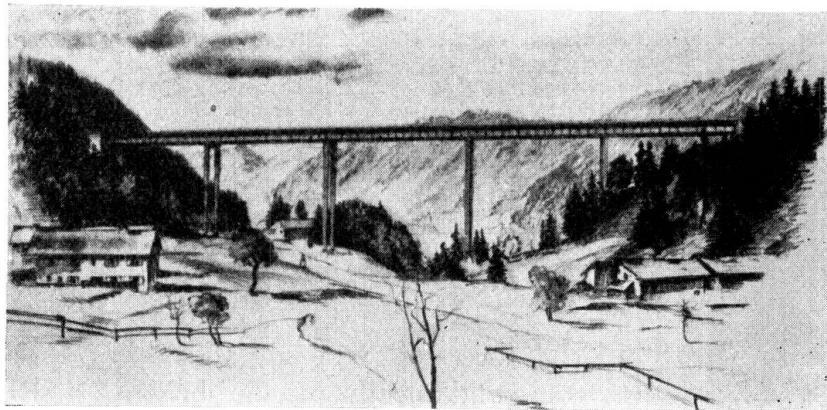


Fig. 8.

type of wind bracing, as, for instance, the Autobahn bridges over the Lech near Augsburg and over the Rhine-Herne canal near Duisburg, the latter having a span of 140 m.

Finally, a few words may be said on the subject of arch bridges. There are a considerable number of cases where an arch bridge is not only technically advantageous and competitive, but is also entirely satisfying from an aesthetic

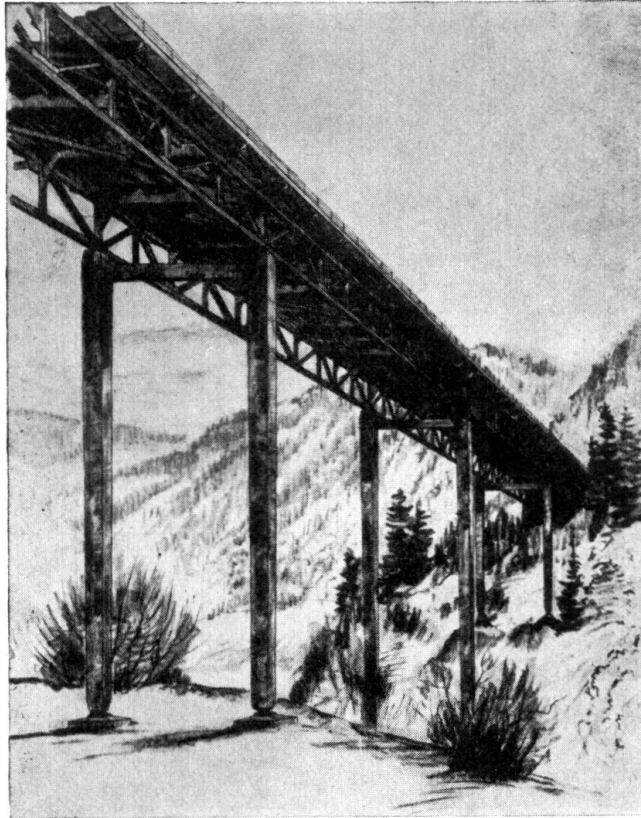


Fig. 9.

point of view, and if this were realised the present urge towards the adoption of nothing but parallel plate girder and lattice girder bridges might happily be varied. Here the author has in mind arch bridges properly so called (whether

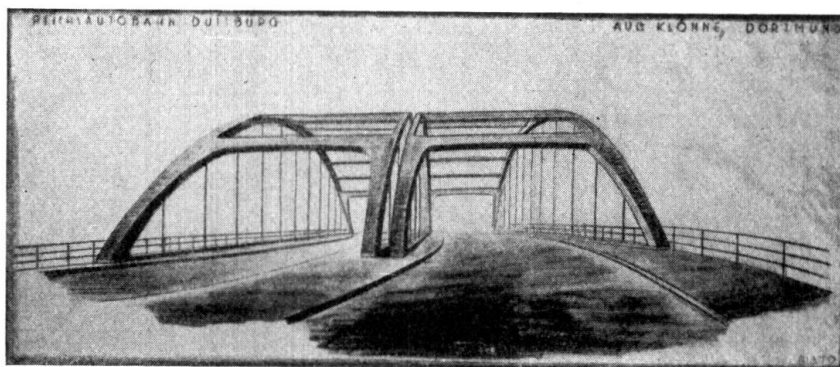


Fig. 10.

with or without a tie bar) as distinguished from girders stiffened by arches in which the girder dominates the design and emphasises the horizontal line. Even a "through" arch bridge (with the arch ribs rising above the roadway) may present a very attractive appearance. Apart from the example already given

of the arch design for the Cologne-Mülheim bridge, the author puts forward for comparison a design for an arched girder and one for a sickle girder,

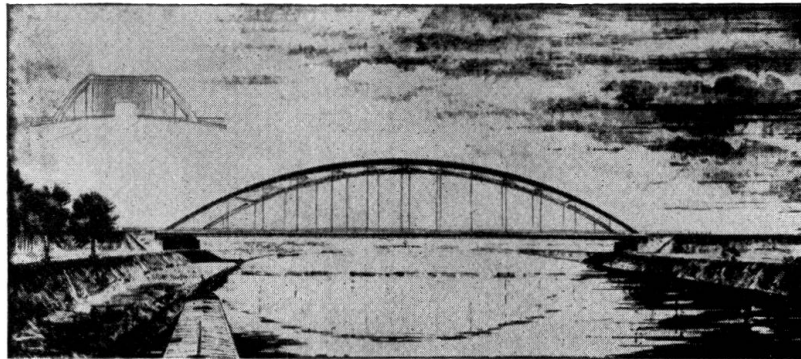


Fig. 11.

appearing in Figs. 11 and 12 respectively, which were worked out for the Rhine-Herne canal bridge.

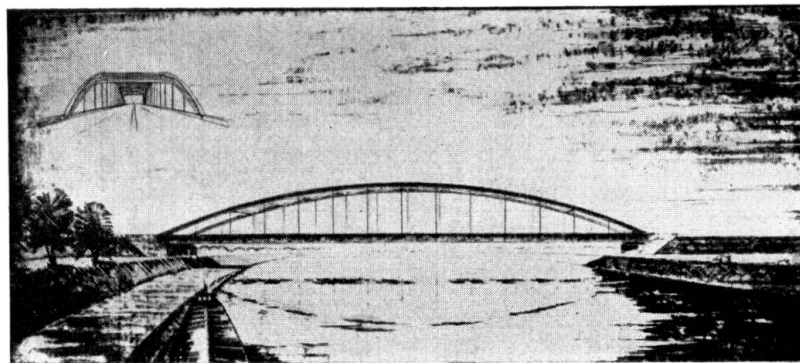


Fig. 12.

It is certain that the principal field of application for the arch bridge, whether of the sickle design or as a fixed arch, will continue to be that of bridging

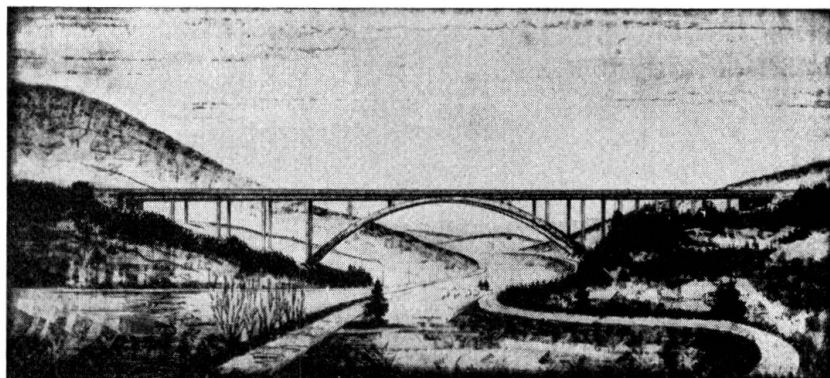


Fig. 13.

wide and deep valleys so that the whole of the arch may be placed below the level of the roadway. Fig. 13 shows a fixed arch for the Helderbachtal bridge,

which serves to illustrate how well this form of construction merges itself into the landscape. The principal dimensions of the design are indicated in Figs. 14 and 15, the former showing a transition from the arch to the girder and the

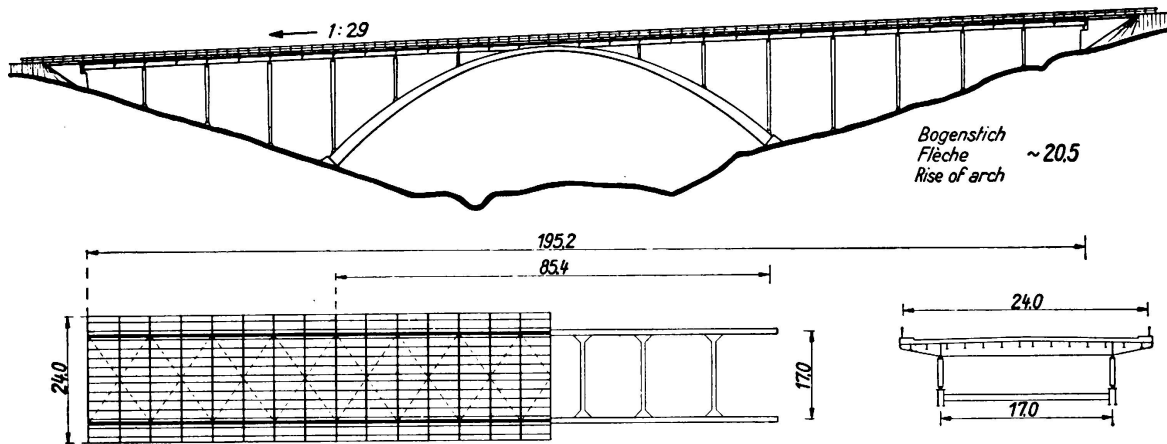


Fig. 14. (Dimensions in metres.)

second showing these two elements separated. The second form is the one which the present author prefers.

He hopes that these brief comments will have served the purpose of demonstrating that bridge designs which have not in fact been carried out are often worthy of notice, and may, indeed, predict future lines of development.

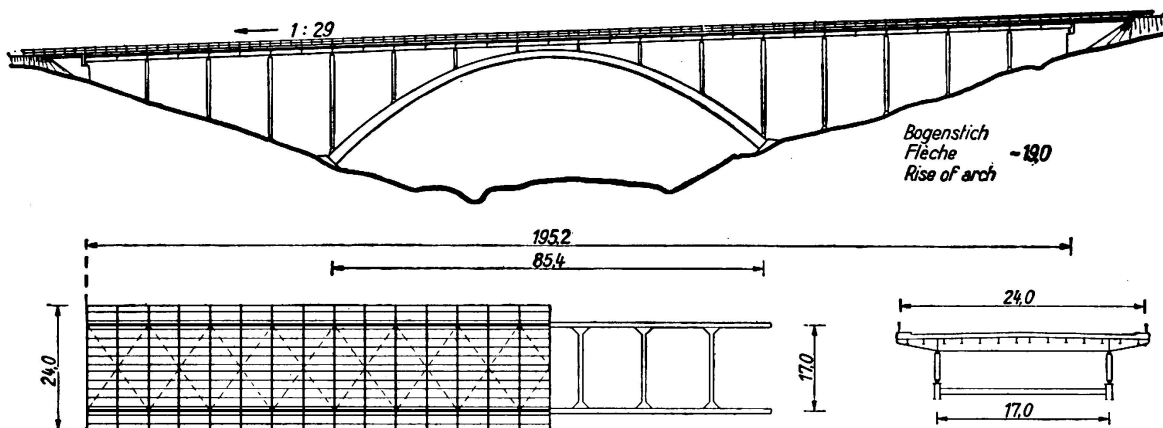


Fig. 15. (Dimensions in metres.)

We ought always to remember that bridges should be built not merely with a view to their structural performance; but, just as in appraising a man we look first for character, so should we expect to find that quality in a bridge. Such bridges cannot be "constructed", they must be "created".

The engineer thereby establishes his mastery, in that he brings the requirements of economy into harmony with those of beauty — a factor which he must never leave out of account.

VII a 3

Applications of Structural Steelwork in Italy.

Die Verwendung des Stahls beim Bau von Stahlkonstruktionen
in Italien.

L'application de l'acier aux constructions métalliques
en Italie.

Dr. Ing. A. Fava,

Chef de Service aux Chemins de fer de l'Etat, Rome.

1) It will be appreciated that in Italy the shortage of ore and coal render steel a very expensive material, and that its applications are consequently limited to those cases where it is an absolute necessity or offers special advantages by comparison with other materials, leaving many fields in which it is unable to compete against its rival, reinforced concrete.



Fig. 1.

2) One of the fields in which it is generally necessary to make use of steel is that of railway bridges which, frequently, are too low in height for the development of masonry arches or reinforced concrete structures. The Italian

State Railways system includes some 7000 steel bridges having a total length of nearly 100 km and during the last ten years about one third of these bridges have been renewed, a process which has usually involved the replacement of the existing spans by new ones except in the case of those of less than 10 m.

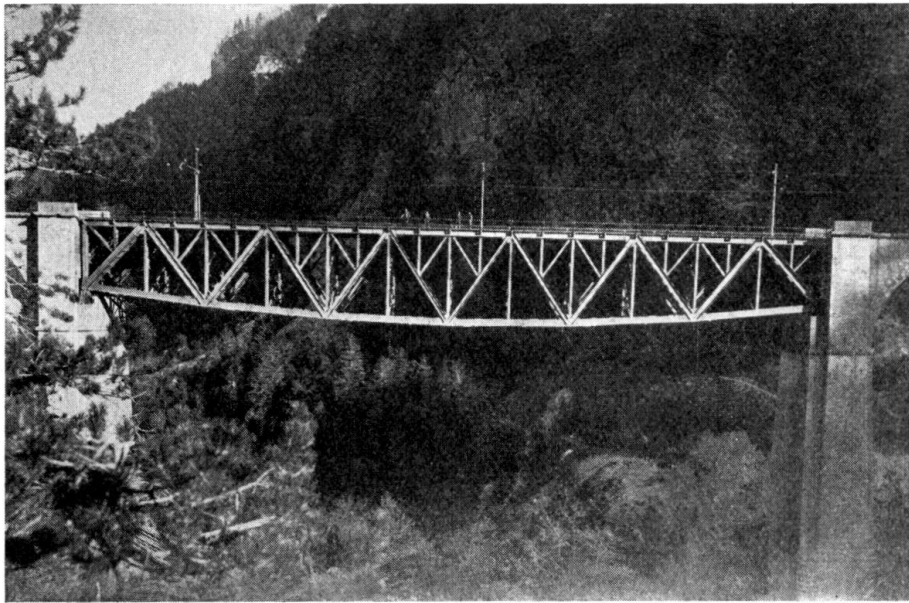


Fig. 2.

Fig. 1 shows a girder over three spans which may be considered as typical of recent work as regards constructional details. It will be seen that all the members consist of large section without small lattice work. The absence of the latter gives an effect of repose and great simplicity.

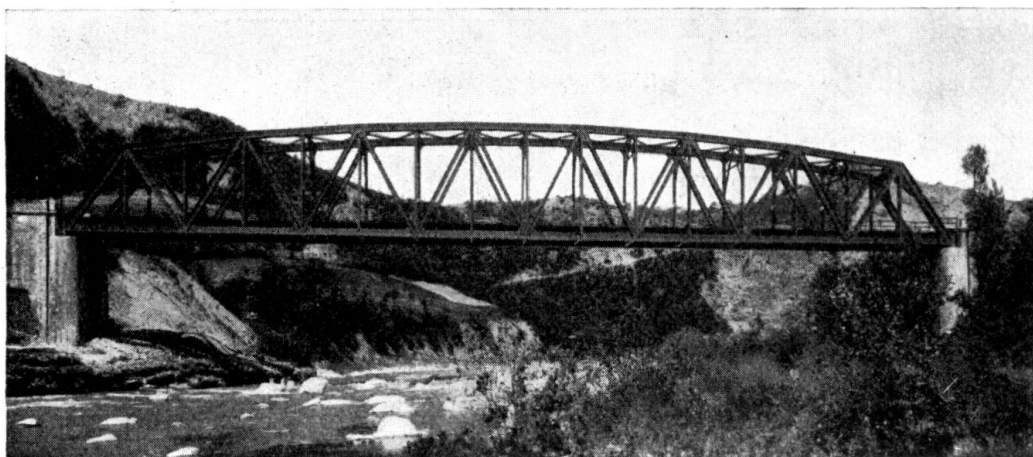


Fig. 3.

Fig. 2 shows one of the larger types of lattice bridges having a parabolic lower boom, and Fig. 3 a through span of about 90 m, again with a parabolic boom, which is representative of a great many bridges. Fig. 4 illustrates a continuous girder over two openings of 77 m each, with the exceptional feature

of intersecting diagonals in the main girders, this being adopted here as the most suitable arrangement in view of the bridge being very much on the skew.

In reference to this bridge it may be observed that for some time past the State Railways have abandoned the use of the continuous type of girder as it was found that in practice many such bridges underwent notable settlement of the supports, which seriously disturbed the distribution of forces. The continuous type is now again being adopted, but is made subject to careful

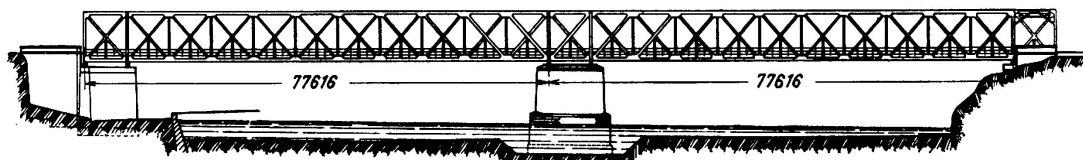


Fig. 4.

check of the levels of the supports. This is done with the aid of calibrated hydraulic jacks and precision manometers, whereby the span is lifted simultaneously from all of its supports and the respective reactions are adjusted to correspond with their theoretical values. In this way not only is it possible so to control the reactions that their values correspond to the several supports being on a level, but as an alternative it is possible to ensure experimentally,

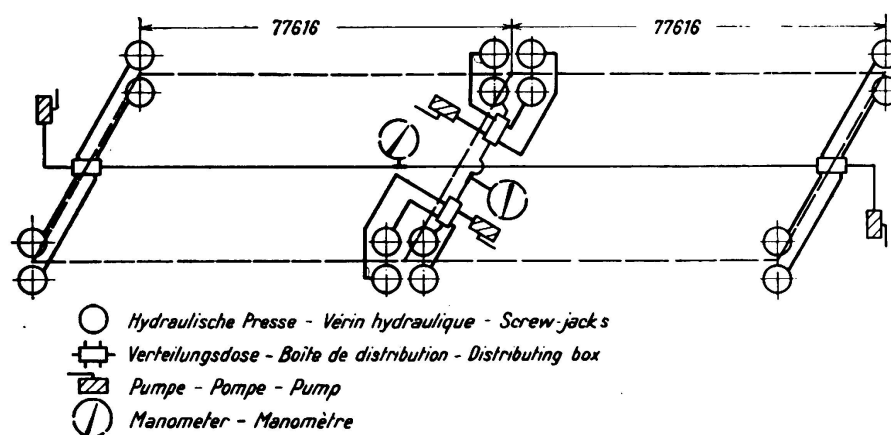


Fig. 5.

with every confidence, that the reactions assume any other values that may be the most expedient in order to obtain the greatest possible economy in the structure.

Fig. 5 shows diagrammatically the arrangement for setting the adjustment of these supports of the continuous girder illustrated in Fig. 4. The operation was combined with measurements of the stresses existing in different members of the structure, and it was found that had the adjustment not been made the reactions would have been very unfavourably influenced.

3) A very large number of public halls, varying in type, have been built entirely in steel, as well as steel framed buildings and roofs for civil, industrial and military purposes, during recent years. In this field of work welding,

especially electric arc welding, has fully established its position in Italy to such an extent that it may be expected in the course of a few years entirely to supersede the use of riveting.



Fig. 6.

From among the many examples which might be mentioned here we will take the steel roofs of the new railway station at Florence, which, at the same time, is a notable instance of modern welded construction.

These roofs are divided into two groups. The first of the latter, which covers the approach for motor cars and the booking hall, forms a structurally separate entity (Figs. 6 and 7). The supporting structure consists of eight large portal

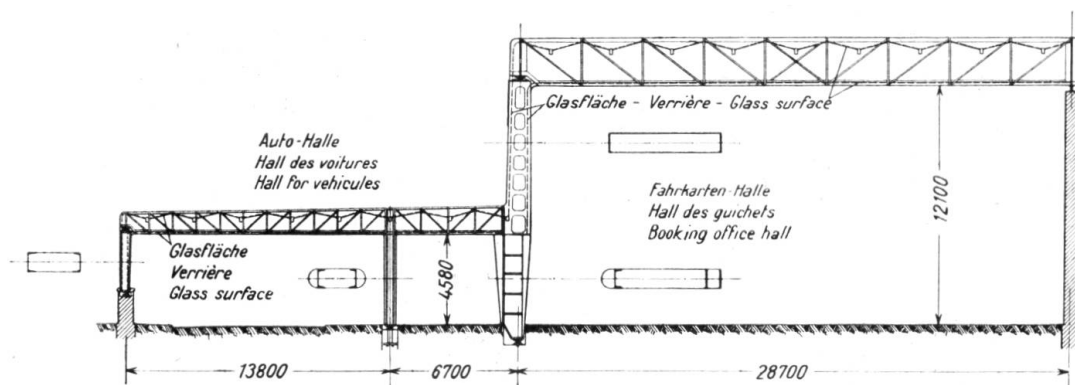


Fig. 7.

frames of three spans each, connected to one another by hinges in order to allow of thermal expansion. These carry two glass coverings, the outer one containing wire mesh glass and the inner one of Thermolux glass, both of them continuous horizontally and vertically, which gives a notable decorative effect. Between them are arranged the sources of artificial lighting for use at night. The outer glazing where it forms the roof has generally to be flat for architectural reasons, but to facilitate drainage it is built up of small elements slightly inclined to one another.

The second set of roofs at Florence is over the platforms and track (Figs. 8

and 9). Here the carrying frame consists of a series of large plate webbed girder spans of 30 m of varying double T section, built up of flat plates by welding.



Fig. 8.

For architectural reasons the shape of these girders is very special. They comprise three portions which are practically horizontal but at different levels, and are connected by steeply inclined portions. In view of the unusual shape

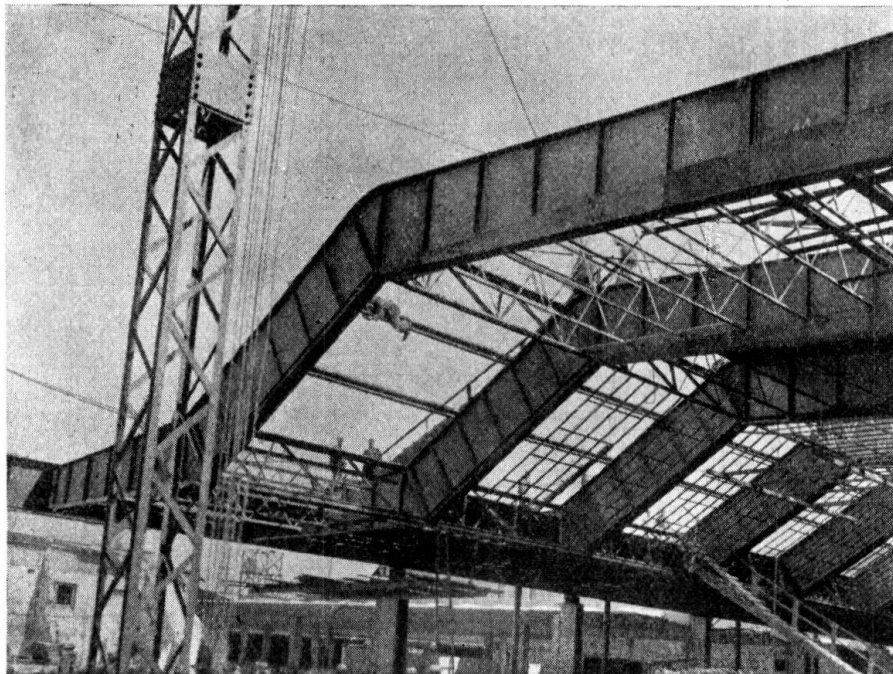


Fig. 9.

of these members their dimensions were decided after preliminary experiments on models, and after the work was completed they were made the subject of an accurate experimental investigation. All the visible portions of the steelwork at Florence have been copper coated and then burnished.

Another example of roofing is that over the public swimming baths at Milan (Fig. 10) which is notable also for the fact that a reinforced concrete construction was at first contemplated but that steel was preferred in view of

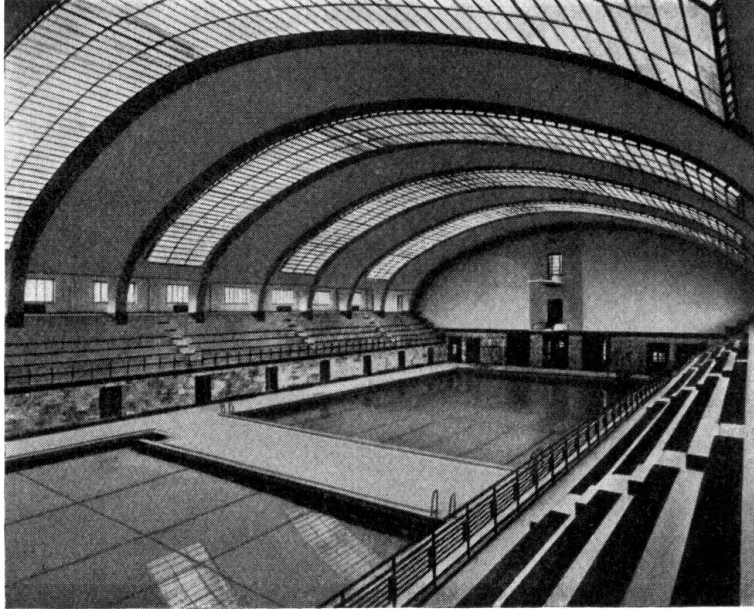


Fig. 10.

the very limited time available for carrying out the work. The arrangements for opening the glazed roof by remote control are ingenious, but these cannot be described here.

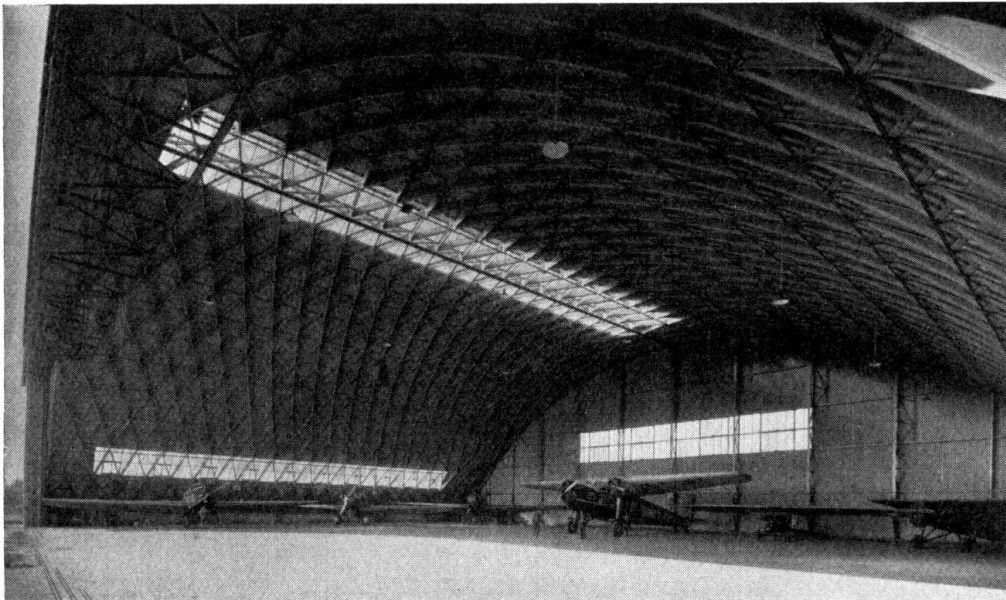


Fig. 11.

A special place among roofs is occupied by the type wherein the construction is of the coffered type. This offers not only a satisfactory solution to various technical problems but marked advantages from an architectural and aesthetic

point of view. There are some notable examples of these in Italy, especially over aircraft hangars and garages (Fig. 11).

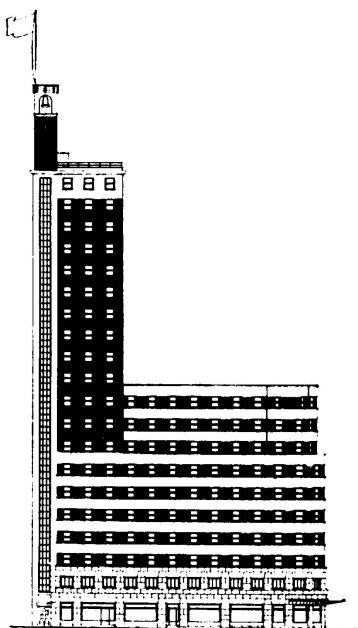


Fig. 12a.

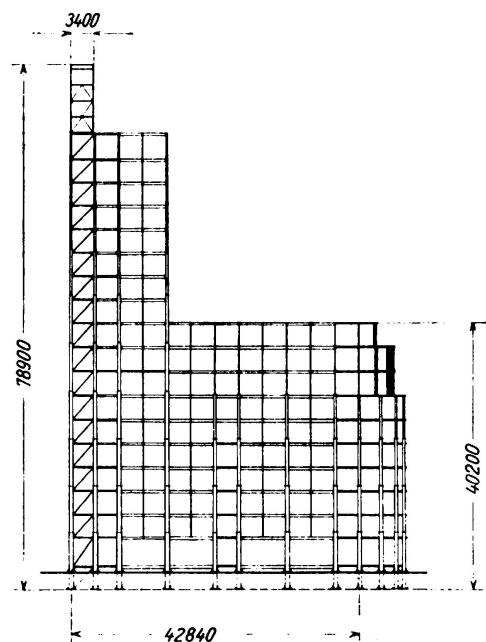


Fig. 12b.

4) In modern building practice the advantages of steel framing have become well recognised, but in Italy, on account of the high cost, steel framing is somewhat rare while concrete framing is common practice.

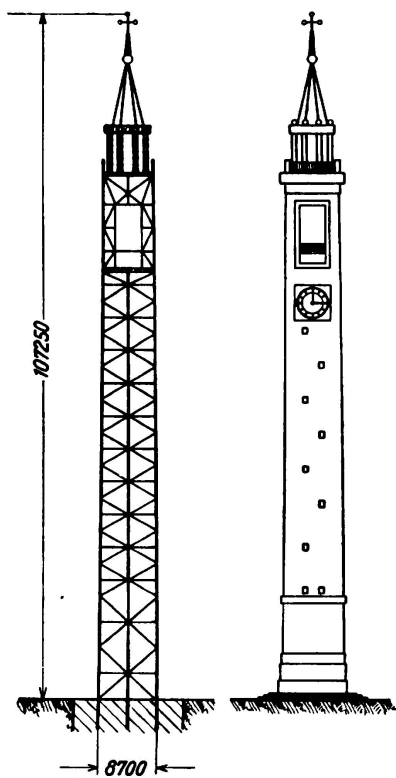


Fig. 13

An example of the latter, worth mentioning in view of its importance, is the new skyscraper at Turin (Figs. 12a and 12b) which is a large steel structure, entirely welded, reaching to a height of 80 m above ground level. Another is the bell tower at Sesto Calende (Fig. 13), some 100 m high and again of welded construction.

But rather than enlarge further upon these illustrations of rather special cases, it may be worth drawing attention to a field of work in Italy in which steel framed structures are assured of a promising future, namely that of earthquakeproof construction. Steelwork, in fact, possesses the best possible qualities for resisting seismic action — maximum lightness and maximum specific strength combined with a high degree of elastic deformability. It offers no less important advantages from a practical standpoint in that it can be manufactured in a few standardised types on an industrial scale by specialised works, and can be rapidly erected on the site by a few welders, thus reducing the call made upon local facilities which are often limited.

5) Arising from the extraordinary development of electrical installations in Italy one of the largest fields of application for steelwork is found in the construction of pylons for the transmission of energy. Allied to these, there may be mentioned, on account of the importance of the structural problems arising, the supporting towers for cable ways the number of which is continually increasing, and also aerials for wireless telegraphy and telephony.

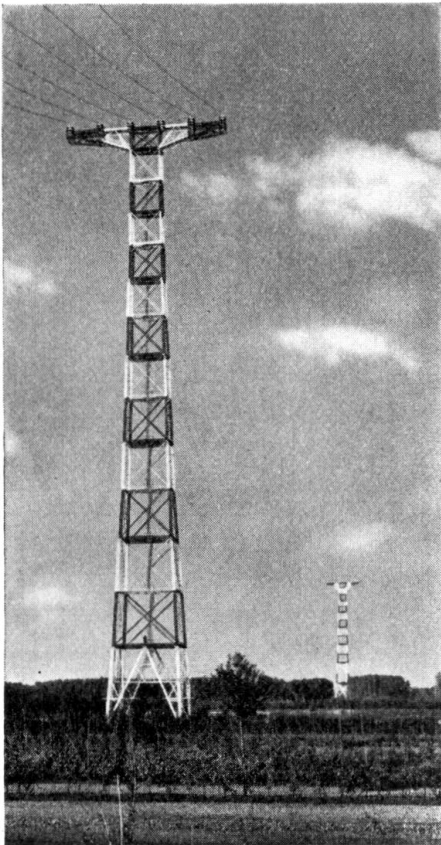


Fig. 14.

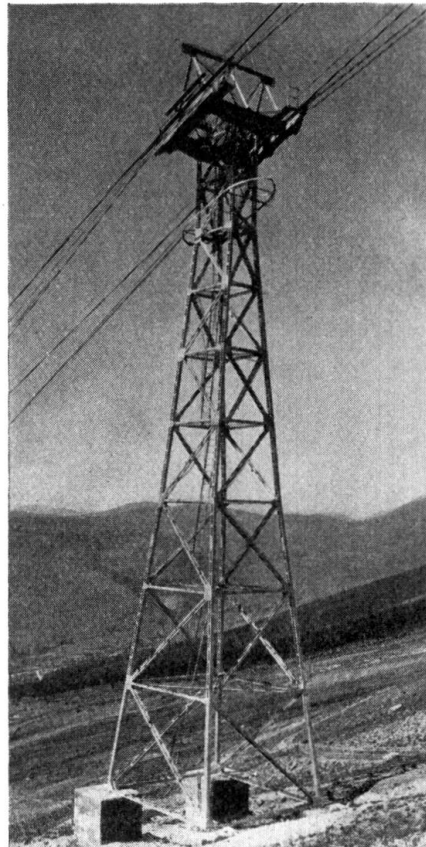


Fig. 15.

It would be of great interest to review different types of these structures as an outline of the process of evolution they have undergone in Italy, but having regard to the limited time available a few only of the most notable examples will be mentioned. Fig. 14 represents two high towers approximately 120 m above ground level for a crossing over the river Po with 1050 m span. Fig. 15 shows one of the modern pylons of welded construction to carry cable-ways namely those of the Gran Sasso d'Italia and Cervino (Matterhorn), wherein the spans are almost 1 km in length. Figs. 16a and 16b show the broadcasting station of Rome, San Palomba, which is one of the largest constructions of the kind. It reaches to the remarkable height of 267 m; the total weight of the aerial is 180 tonnes and the load on the insulator at the base is 290 tonnes.

6) A branch of engineering in which Italian industry has taken an important part is that of cranes for shipyards, factories and harbours. These are made of every kind and size, with frequent recourse to electric welding.

Here again it would be worth while to trace out the evolution of the different

types, but lack of time necessitates that reference should be confined to the crane represented in Figs. 17a and 17b, which is distinguished by its elegance

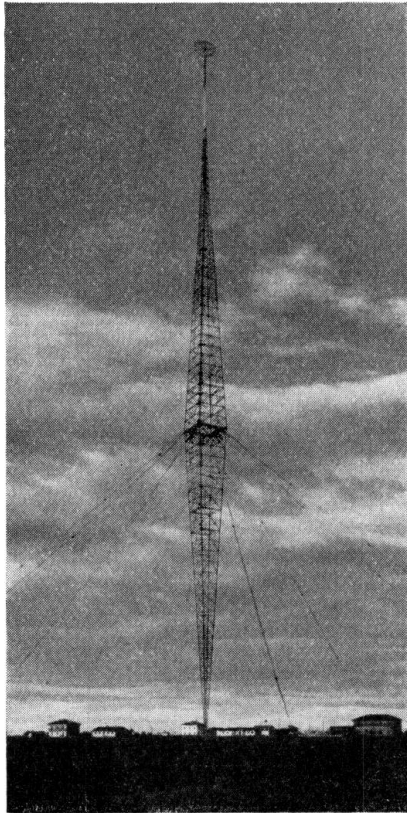


Fig. 16a.

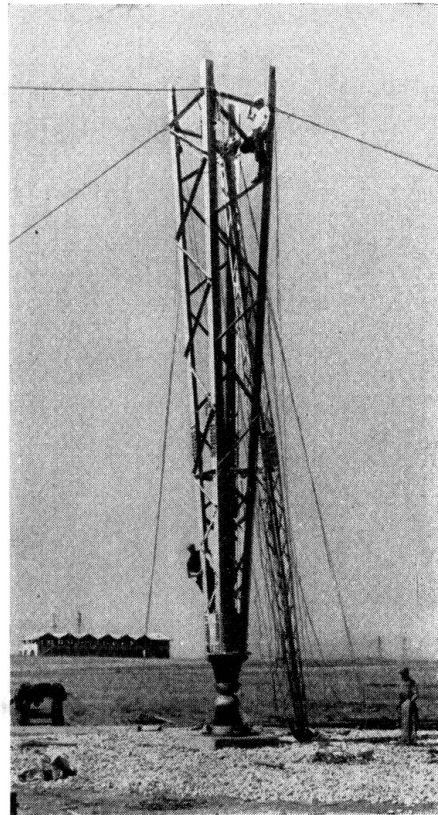


Fig. 16b.

of line and by the method of construction, entirely by welding. The vertical arm consists of two co-axial tubes; the inner tube is entirely welded and the

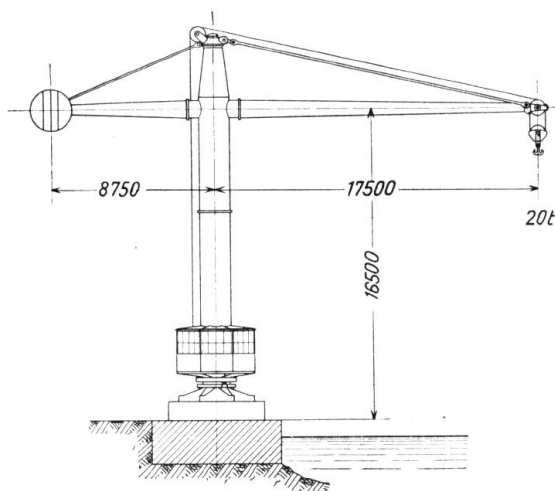


Fig. 17a.

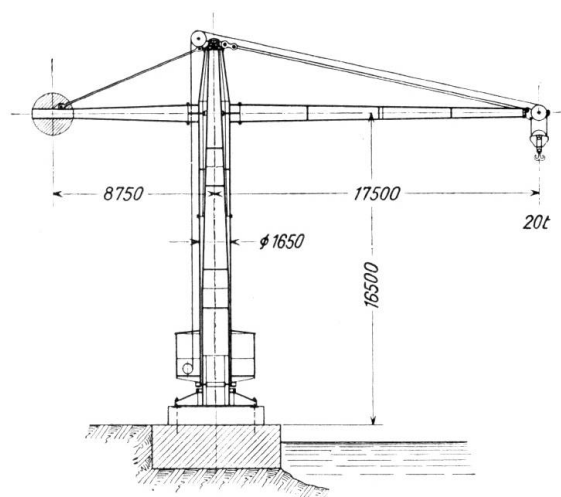


Fig. 17b.

outer one, for reasons of erection, is made in two parts connected by bolted flanges. The same form of connection is used between the two horizontal arms and the upper portion of the vertical member.

7) Finally reference may be made to some special structures made in weldless tubes of semi-hard steel, a form of work which has assumed a wide development

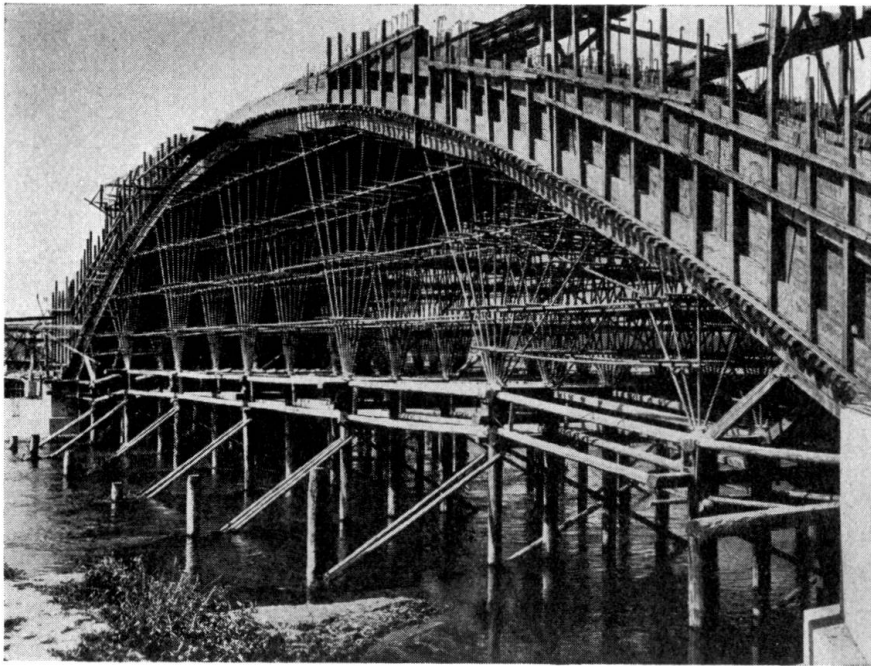


Fig. 18.

in Italy. Fig. 18 shows the centring for a concrete bridge over the river Ticino near Pavia. Fig. 19 shows the extensive falsework of the new bridge now in course of construction over the Tiber giving access to the Mussolini Stadium;

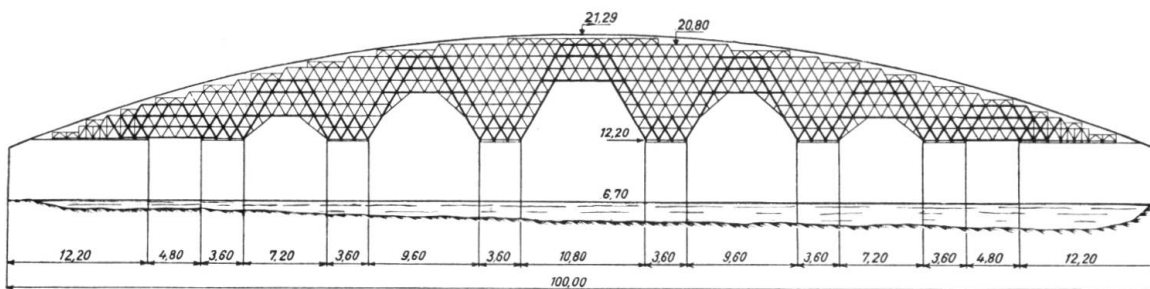


Fig. 19.

this will be a reinforced concrete bridge with a central arch of 100 m span. This centring involves the use of 64 000 m of tubing with 54 000 junction pieces.

VII a 4

Experiments on Girders with Welded Web Stiffeners.

Versuche mit Trägern, deren Stege durch angeschweißte Versteifungen verstärkt sind.

Essais sur poutrelles renforcées par des raidisseurs soudées à leur âme.

Dr. Ing. St. Bryła,

Professeur à l'Ecole Polytechnique de Varsovie (Pologne).

A series of tests was carried out to study the effect of stiffeners welded to the webs of joists, in the first place on sixteen joists PN 16, 20, 24 and 30 and secondly on six joists, PN 32 and 34, all the joists having a span of $L = 2$ m and being bent by the application of a concentrated load at the middle of the span.

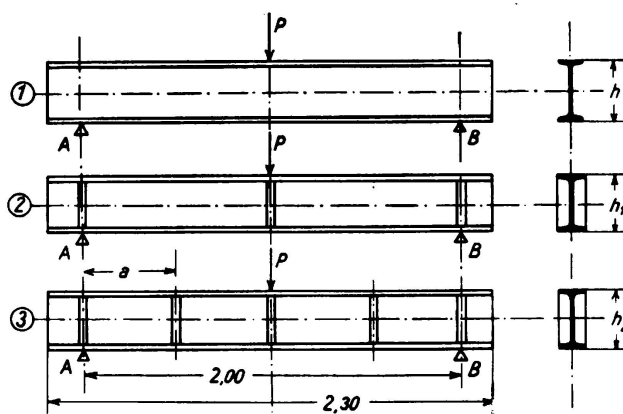


Fig. 1—3.

The joists tested were of three types.

- 1) Joists without stiffeners on the webs (Fig. 1).
- 2) Joists having three pairs of stiffeners arranged respectively over the supports and below the concentrated load (Fig. 2).
- 3) Joists fitted with five pairs of stiffeners at 0.50 m centres (Fig. 3).

Table I shows maximum values of the forces which caused failure of the joists, the suffixes denoting the numbers of pairs of stiffeners.

Table I.

NP	breaking load		
	R ₀	R ₃	R ₅
16	8.6 t	7.425 t	7.6 t
20	15.4	13.75	15.8
24	22.9	23.85	26.3
30	39.9	48.45	48.3
32	46.0	58.5	59.5
34	51.0	69.5	72.5

A study of the differences $R_3 - R_0$ as given in Table II shows that in the case of relatively deep joists the addition of three pairs of stiffeners increases the breaking load R in proportion to the depth of the joist. In the case of joists PN 16 and 20 no additional strength was obtained by the addition of stiffeners. The last column of the table gives the increase in strength brought about by the addition of five pairs of stiffeners: but in the case of joists PN 16 the strength was thereby reduced, while in the case of the other joists the increase in strength is proportionate to the height of the joist in question.

Table II.

NP	R ₃ —R ₀		R ₅ —R ₃		R ₅ —R ₀	
	tonnes	%	tonnes	%	tonnes	%
16	— 1.175	— 13.7	0.175	2.36	— 1.0	— 11.6
20	— 1.75	— 11.3	2.05	14.9	0.4	2.6
24	0.95	4.15	2.45	10.27	3.4	14.8
30	8.55	21.4	0.15	— 0.31	8.4	21.0
32	12.5	27.2	1.0	1.71	13.5	29.4
34	18.5	36.3	3.0	4.6	21.5	42.2

Here the safe load (with $\sigma = 1200 \text{ kg/cm}^2$ and $M = \frac{PL}{4}$, L being 200 cm) is

$$P_b = \frac{4 \cdot 1200}{L} W = 24 W. \quad (1)$$

The factor of safety $n = \frac{R}{P_b}$, or ratio between the breaking load and the safe load, is given in Table III for each of the cases examined.

Table III.

I NP	W cm ³	P _b tonnes	n ₀	n ₃	n ₅
16	117	2.81	3.06	2.98	3.05
20	214	5.14	3	2.68	3.08
24	354	8.50	2.7	2.80	3.10
30	653	15.67	2.55	3.09	3.08
32	782	18.75	2.45	3.12	3.16
34	923	22.32	2.28	3.12	3.25

If joists PN 16 and 20 are left out of account it will be seen that n_0 becomes less and n_3 becomes greater in proportion as the depth of the joist increases, while n_5 scarcely varies at all, but is always greater than n_0 .

Table IV gives the values of σ obtained by substituting for P the values of Q and R given in Table I and taking for W the values listed in Table III. In this way the results obtained from joists of varying depth are brought to a common measure.

Table IV.

I NP	Number of pairs of stiffeners	Stresses σ obtained by substituting the values of Q and R from Table I for those of P in equation (3)	
		Q	R
16 {	0	29.5	36.8
	3	29	31.7
	5	29	32.4
20 {	0	29.2	36
	3	27.9	32.2
	5	31	36.9
24 {	0	26.2	32.4
	3	27.4	33.8
	5	29.7	37.2
30 {	0	23	30.6
	3	29.3	37
	5	30.2	37
32 {	0	—	29.4
	3	—	37.4
	5	—	38.0
34 {	0	—	27.7
	3	—	37.7
	5	—	39.3

The diagrams in Figs. 4 and 6 are a graphical representation of the results stated in Table IV, the abscissae denoting the heights of the joists in centimetres and the ordinates denoting the stresses σ in kg/mm². Had the material of the joist been perfectly homogeneous and the tests been carried out under ideal conditions without any possibility of lateral buckling, and had the formula $\sigma = \frac{M}{W}$ been valid up to the point of failure, the curves would have been horizontal lines.

It will be noted that curve 4 drops while curves 5 and 6 are rising. The first mentioned result was to be expected, and the latter indicates that this loss of strength can be avoided by welding stiffeners on to the webs and flanges of the joists, the phenomenon being due to the collapse of the upper flange.

Figs. 7 and 8 show the manner and magnitude of the effect. Those joists which are provided with stiffeners assume after failure a double undulation

with the points of inflection at the centre of the joists (Fig. 7), whereas the joists unprovided with stiffeners (Fig. 8) assume after failure a shape containing only a single undulation; the effect of the stiffeners is, therefore, to promote the double undulation with the consequence that the critical buckling load is increased.

The phenomenon was the same in all the joists. In those with stiffeners both flanges became bent, the upper flange as much as the lower, but the joists without stiffeners remained straight with only a limited amount of deflection. In the joists without stiffeners a crushing of the upper flange underneath the concentrated load was observed and this effect increased with the depth of

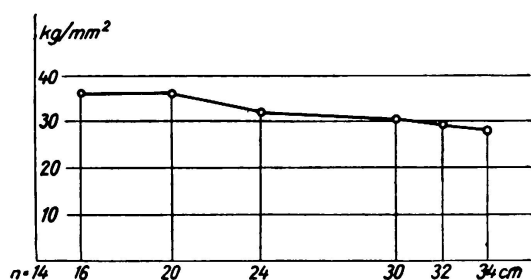


Fig. 4.

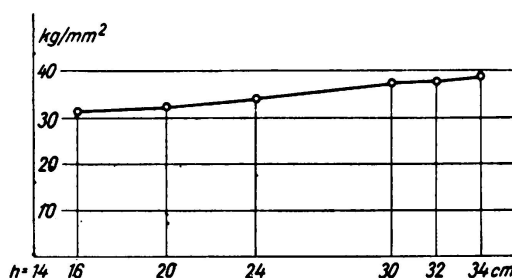


Fig. 5.

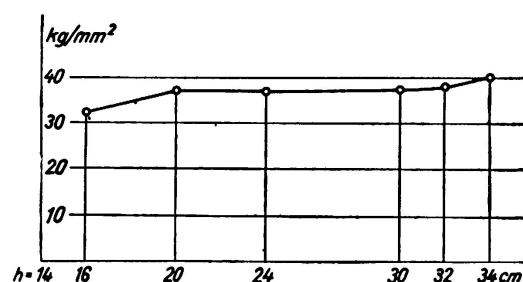


Fig. 6.

the joist. The influence of the stiffeners on the deformation of the joist becomes more marked in proportion to their depth. It follows that the resistance of the joists to bending where stiffeners were provided was on the point of being used up, and that fracture was imminent. On the other hand in the joists which had no stiffeners failure due to bending was still a long way off, and failure actually occurred on account of the buckling of the flange under the concentrated load. The collapse observed in the deep joists without stiffeners when the stresses σ were still relatively low appears to indicate that it is not these stresses which play the predominant role, but rather the normal stresses present in the horizontal section of the web immediately below the flange underneath the concentrated load, to which Professor *Huber* has given the name of transverse stresses, and has devoted several chapters of his book.¹ The present author will give a more detailed study of these transverse stresses elsewhere, and will here merely summarise the results.

¹ *M. T. Huber: Investigations of I-beams double T. Proceedings of the Technical Society of Warsaw, 1925.*

1) The reinforcing of an I-joist by means of stiffeners welded to the web below the concentrated load has the effect of increasing its resistance to bending, the increase being more marked in proportion to the depth of the joist, and

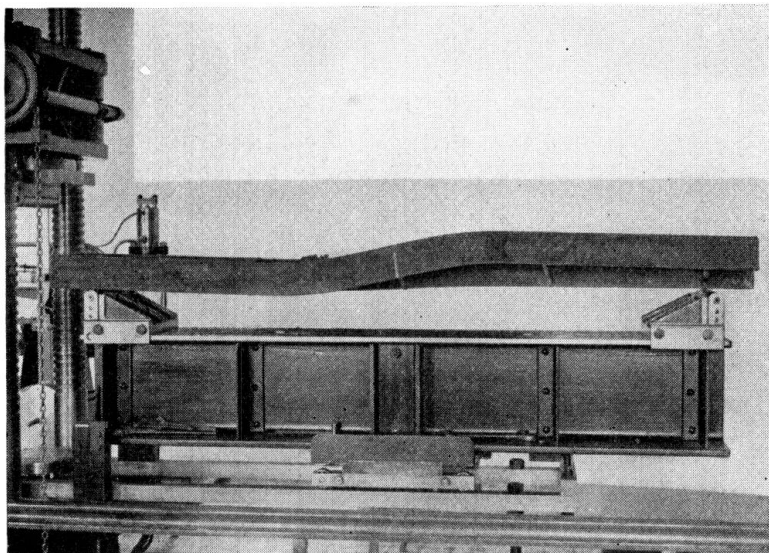


Fig. 7.

being inappreciable in a joist PN 16 but amounting to 40 % in the case of a joist PN 30. The provision of stiffeners welded to the web at other points than the concentrated load also increases the resistance of the joist, though to a smaller extent.

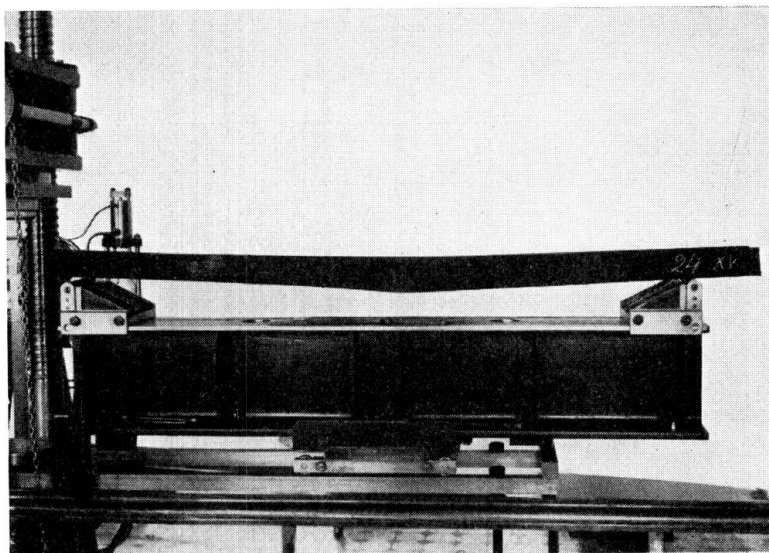


Fig. 8.

2) The maximum stresses calculated from the formula $\sigma = \frac{M}{W}$ must be reduced in the case of very deep joists. This formula cannot be used to determine the strength of joists in excess of a certain depth when subjected to concentrated loads, as such joists fail not by bending but by crushing of the flange where the load is applied. By welding on stiffeners underneath the concentrated load the failure by buckling is delayed and the formula as given may continue to be used.

VIIa 5

Illuminating and Constructional Considerations in the Arrangement of Long Span Saw-Tooth Roofs with Steel Frames.¹

Lichttechnische und konstruktive Gesichtspunkte für die Anordnung weitgespannter Sägedächer mit Traggerippe aus Baustahl.¹

Les points de vue de l'éclairage et de la construction dans la disposition des sheds de grande portée avec fermes métalliques.¹

Dr. Ing. H. Maier-Leibnitz,
Professor an der Technischen Hochschule, Stuttgart.

I. In the case of industrial single storey buildings which form the simplest special case of multiple bay sheds, the structural features are primarily influenced by the character of the daylighting, ventilation, rainwater, drainage, crane equipment and the placing of the supports as governed by operating conditions. A further condition is imposed on the design in that impressive and aesthetically satisfactory interiors as well as exteriors are to be realised.

II. As regards the arrangement of *daylight openings* in an industrial single storey building, there are a surprisingly large number of possibilities which have been tried in practice, but these vary greatly in their merits from the point of view of illumination. Ordinary skylights (see fig. 1, whether with inclined or vertical glass surfaces (Boileau roofs), result in intensities of lighting which are not uniform, and which, in their maximum values, tend to be excessive. At any time during the day, whatever may be the position of the daylight openings, sunbeams may enter. During the summer this is bound to give rise to uncomfortable greenhouse conditions of temperature in the building, and will at any time of the year tend to dazzle the workers, apart from which, in many cases, the sun's rays are undesirable for the sake of the product under manufacture.

All these disadvantages can be avoided by the adoption of *saw-tooth roofs*, which guarantee a quality of lighting in the workshop equal to that in an artist's studio, and do this under economically reasonable terms.

To assess the quality and uniformity of the daylighting use is made of the concept of the daylight ratio TQ. (*Tageslichtquotient*), which is defined as the ratio of the intensity of the illumination existing on, for instance, a horizontal

¹ Supplement to the report VIIa 9 in the Preliminary Publication: Development of structural steel-work.

element of surface within the room, to the intensity of illumination on a horizontal element of surface under the open sky.²

Fig. 1 shows the T.Q. curve for a recently completed factory building having gable skylights, which illustrates the disadvantages explained above; nor can these be overcome by the usual expedient of coating the glass surfaces exposed to the sun with whitewash or the like.

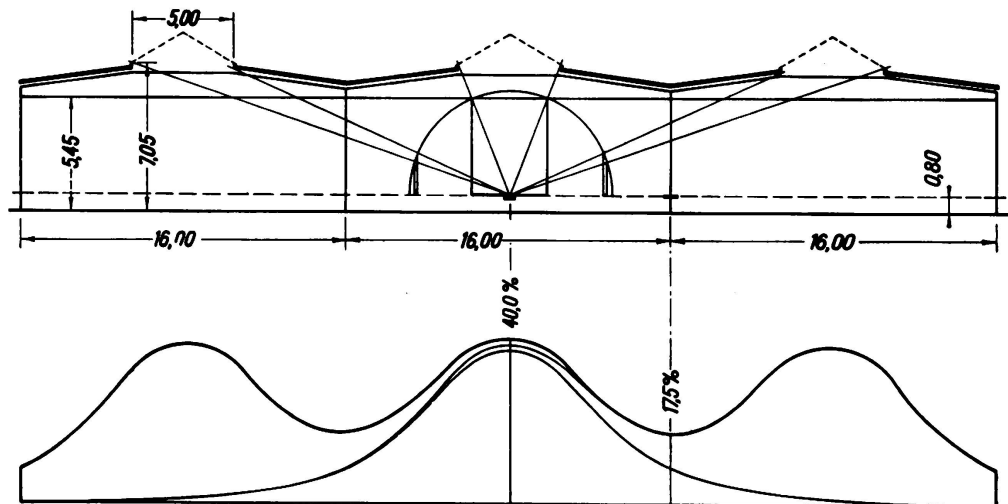


Fig. 1.

T.Q. line for saddle-shaped skylight.

It will be explained below that when a saw-tooth roof is adopted, the choice of the pitch of the transparent and non-transparent roof coverings, as well as the width of the glass surfaces, depends to a great extent on daylighting considerations. Fig. 2 shows a comparison between the following two cases, assuming a saw-tooth unit 7 m wide:

- a) Non-transparent roof covering inclined at 30° .
Transparent roof covering inclined at 60° .
- b) Non-transparent roof covering inclined at 30° .
Transparent roof covering vertical.

The T.Q. lines indicate that in case a) considerably better conditions are obtained than in case b). The T.Q. for the "characteristic" horizontal element — a concept which is also used below — in the middle of the second saw-tooth unit, amounts to 16.7 % in case a) and to 12.3 % in case b).

Fig. 3 shows how the saw-tooth units should be arranged with vertical glass surfaces in case c), so that the "characteristic" element of area already mentioned will have a T.Q. of equal value to that obtained in case a). The considerably greater amount of both transparent and non-transparent roof covering required in case c) as compared with case a) will be noticed. In the effect of illumination nothing is altered if, with a view to economy in case c), the transparent roof

² See *Maier-Leibnitz: Der Industriebau, die bauliche Gestaltung von Gesamtanlagen und Einzelgebäuden*, Berlin, 1932, p. 74 ff.; also DIN Sheet 5034. In the experiments on illumination described below very long strips of glass are assumed. The very simple construction for T. Q. may be seen from the illustrations.

covering is carried out from the upper edge of the gutter at right angles to the non-transparent covering, so that the non-transparent roof forms a kind of overhang. Such types of saw-tooth roofs would be specially useful, for instance, in the neighbourhood of the Equator where it is desired to prevent the entrance of very steeply incident rays of the sun into the room.

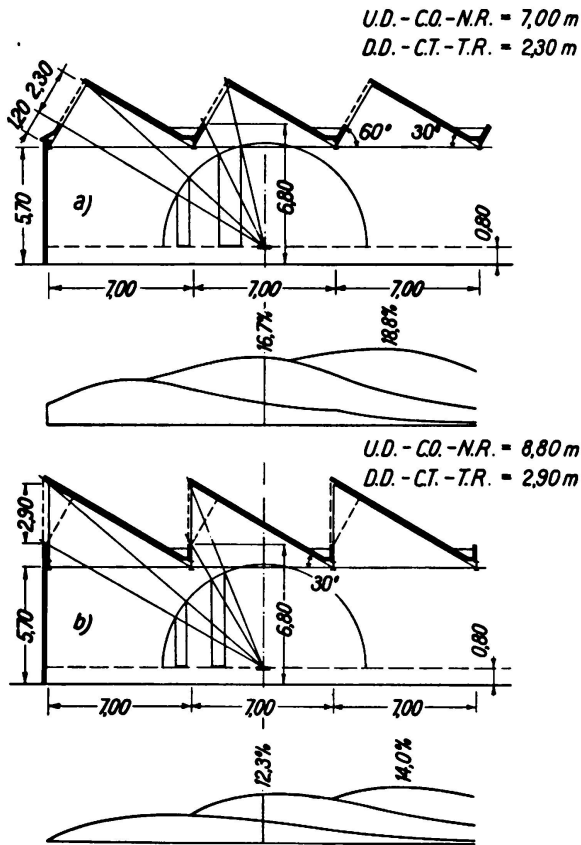


Fig. 2.

Saw-tooth roofs, 7 m — units.

T.Q.-lines for horizontal elements.

- a) T.R. = Transparent roof covering inclined at 60° .
 N.R. = Non-transparent roof covering inclined at 30° .
 b) T.R. = Transparent roof covering vertical.
 N.R. = Non-transparent roof covering inclined at 30° .

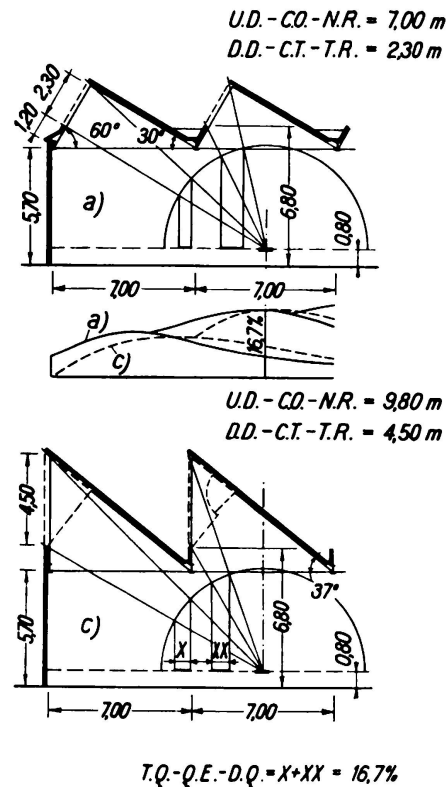


Fig. 3.

Saw-tooth roofs, 7 m — units.

T.Q.-lines for horizontal elements.

- a) T.R. = Transparent roof covering inclined at 60° .
 N.R. = Non-transparent roof covering inclined at 30° .
 c) T.R. = Transparent roof covering vertical.
 N.R. = Non-transparent roof covering inclined at 37° .

Fig. 4 gives the T.Q. curve for case a₁); which is fundamentally similar to case a) except that the non-transparent portion of the roof covering has an overhang which extends to a point lying vertically above the lowest point of the glass strip. In the "characteristic" horizontal element of the second saw-tooth unit there is obtained a T.Q. value of 12.3 %, equal to that for case b), whereas in case a) the value is 16.7 %. In case d) it is shown to what width the strip of glass has to be reduced (1.7 m instead of 2.30 m) in order that the characteristic element may receive the same T.Q. as in case a₁). If, in case d), the upper

portion of the non-transparent roof covering is made movable, a certain "dosage" of daylight may be obtained, and just as in case a_1) the entry of sunbeams into the room may effectively be prevented under even unfavourable conditions, such as the height of summer.

Without detriment to the T.Q. value further savings in the transparent and non-transparent roof coverings might be effected by reducing the pitch of the transparent covering to, for instance, 45° .

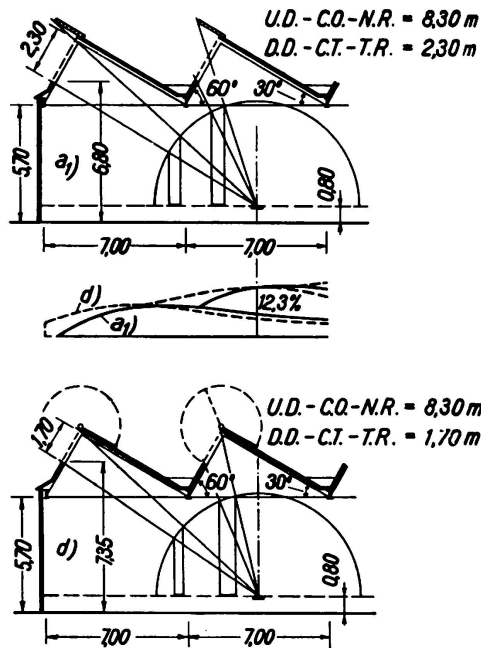


Fig. 4.

Saw-tooth roofs, 7 m — units.

T.Q.-lines for horizontal elements.

- a_1) T.R. = Transparent roof covering inclined at 60° with cantilever.
 N.R. = Non-transparent roof covering inclined at 30° .
 d) T.R. = Transparent roof covering inclined at 60° .
 N.R. = Non-transparent roof covering inclined at 30° .

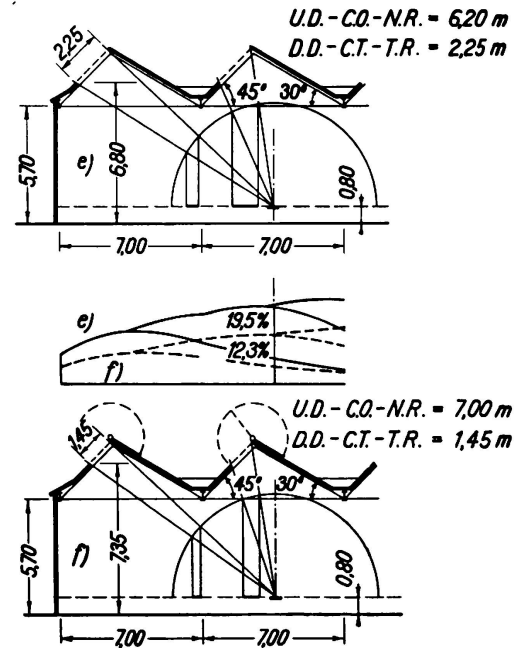


Fig. 5.

Saw tooth roofs, 7 m — units.

T.Q.-Lines for horizontal elements.

- e) T.R. = Transparent roof covering inclined at 45° .
 N.R. = Non-transparent roof covering inclined at 30° .
 Level of gutter same as for cases a — d incl.
 f) T.R. = Transparent roof covering inclined at 45° .
 N.R. = Non-transparent roof covering inclined at 30° .

Fig. 5 shows two such cases. In case e) the same conditions are chosen as in cases a) and a_1), both as regards the gutters and their top edges. At the "characteristic" point the T.Q. value is 19.5 %.

In case f) the strips of glass are only 1.45 m wide, but in spite of this there is obtained at the "characteristic" element the same T.Q. value of 12.3 % as in cases a_1), b) and d), the inclination of 30 % for the non-transparent roof covering being retained. In both cases e) and f) it is assumed that movable parts for dosing the daylight are fitted. Case f) approximates to the desired minimum of building and operating cost.

For cases a) and b) the T.Q. curves of the vertical elements of area are shown in Fig. 6.

III. As regards constructional considerations affecting the *rainage and ventilation of saw-tooth roofs*, the criticism that saw-tooth roofs are difficult to venti-

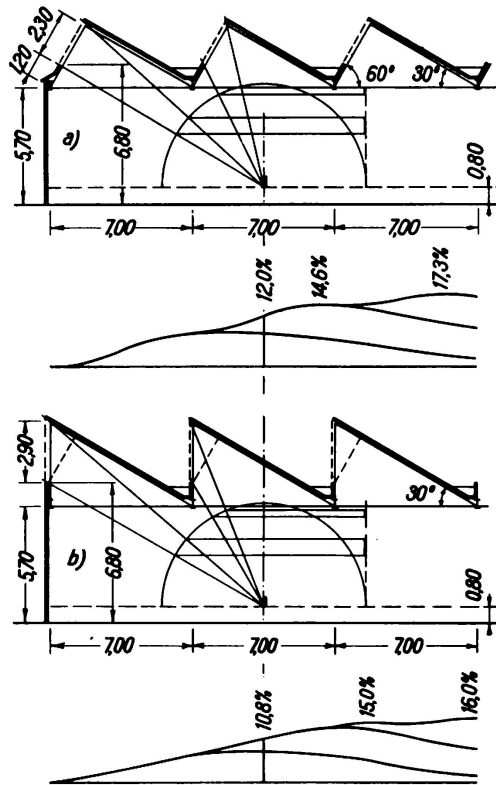


Fig. 6.

Saw-tooth roofs, 7 m — units.
T.Q.-lines for vertical elements.

a) T.R. = Transparent roof covering inclined at 60°.

N.R. = Non-transparent roof covering inclined at 30°.

b) T.R. = Transparent roof covering vertical.

N.R. = Non-transparent roof covering inclined at 30°.

late is overcome by the provision of ventilating flaps, as shown in Fig. 3 for case c) or by the provision of suitably arranged ventilators of cylindrical profile on the ridge of the saw-tooth roof. This ensures at least as good ventilation as is

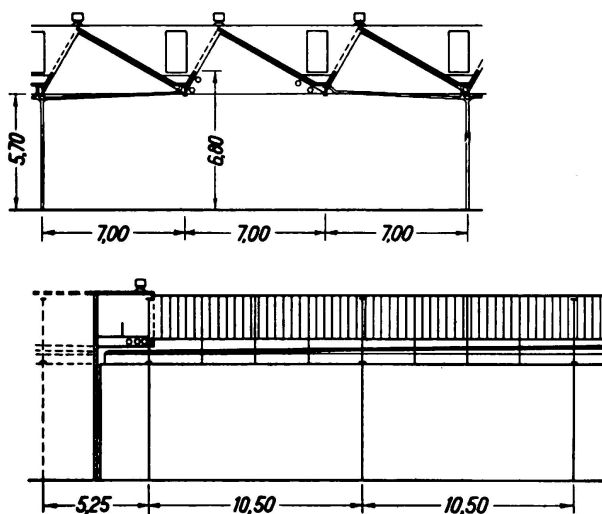


Fig. 7.

Inspection gangway on the east and west side of a building.

possible with gable skylights. Ventilation may be considerably simplified if, as shown in Fig. 7, an inspection gangway is provided either on the east or

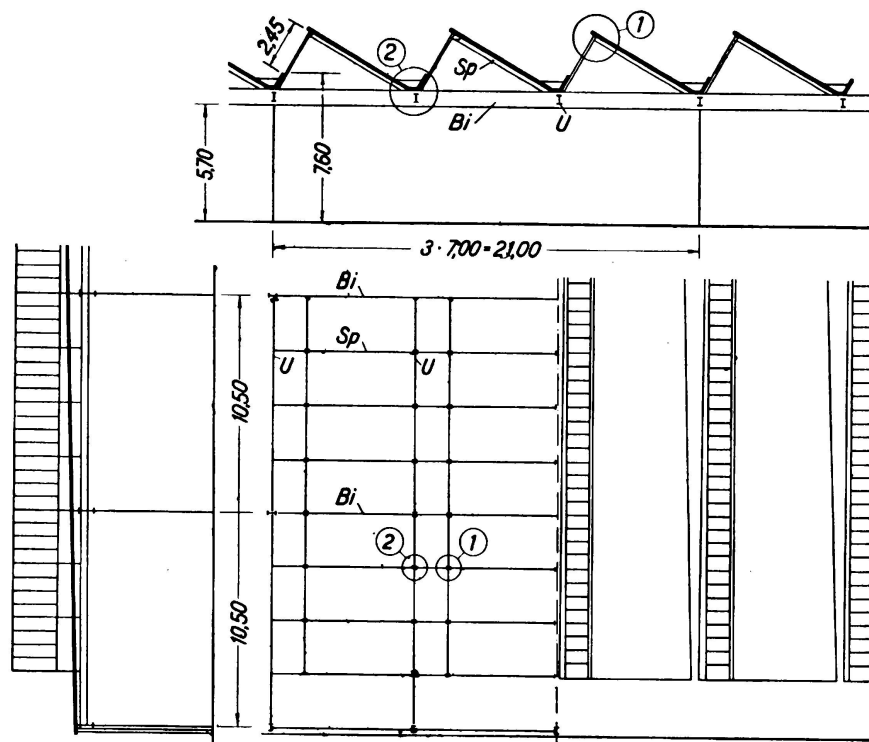


Fig. 8.

Case A: Supporting rib for bays of $21 \times 10,5$ m, plate webbed main girder running north and south.

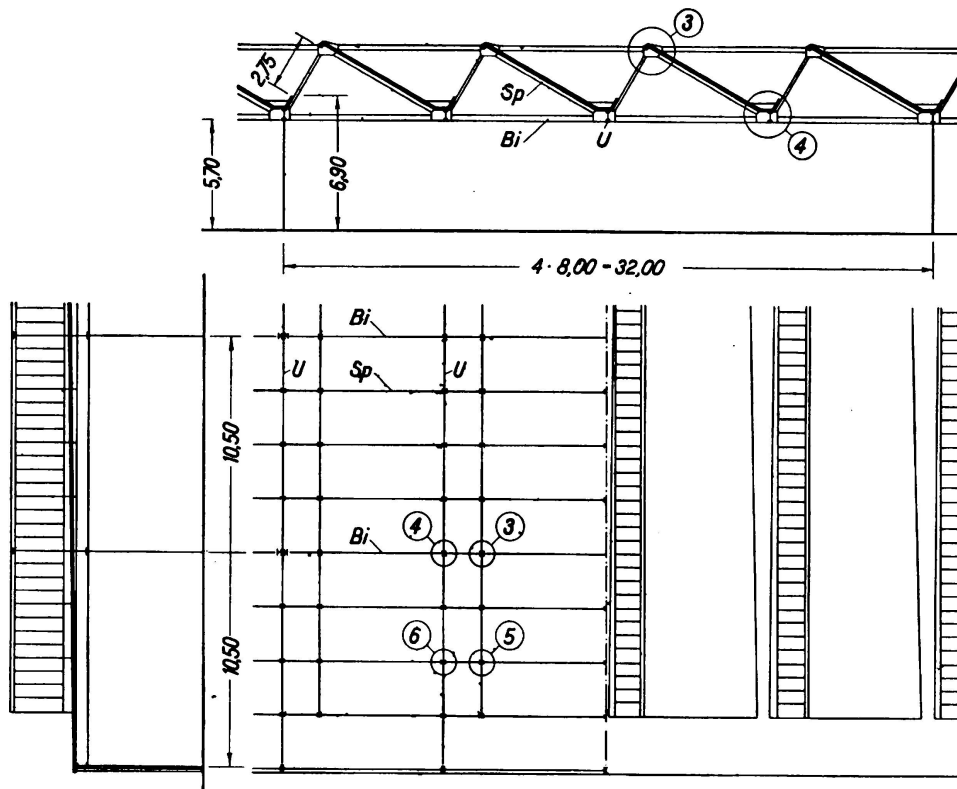


Fig. 9.

Case B: Carrying ribs for bays of $32 \times 10,5$ m, main girder of open frame construction running north and south.

the west side of the saw-tooth roof. Such a gangway also brings the gutters in the valleys between the saw-teeth within easy reach. The inner spaces below the saw-tooth roof are connected, for the purpose of ventilation, by a transverse

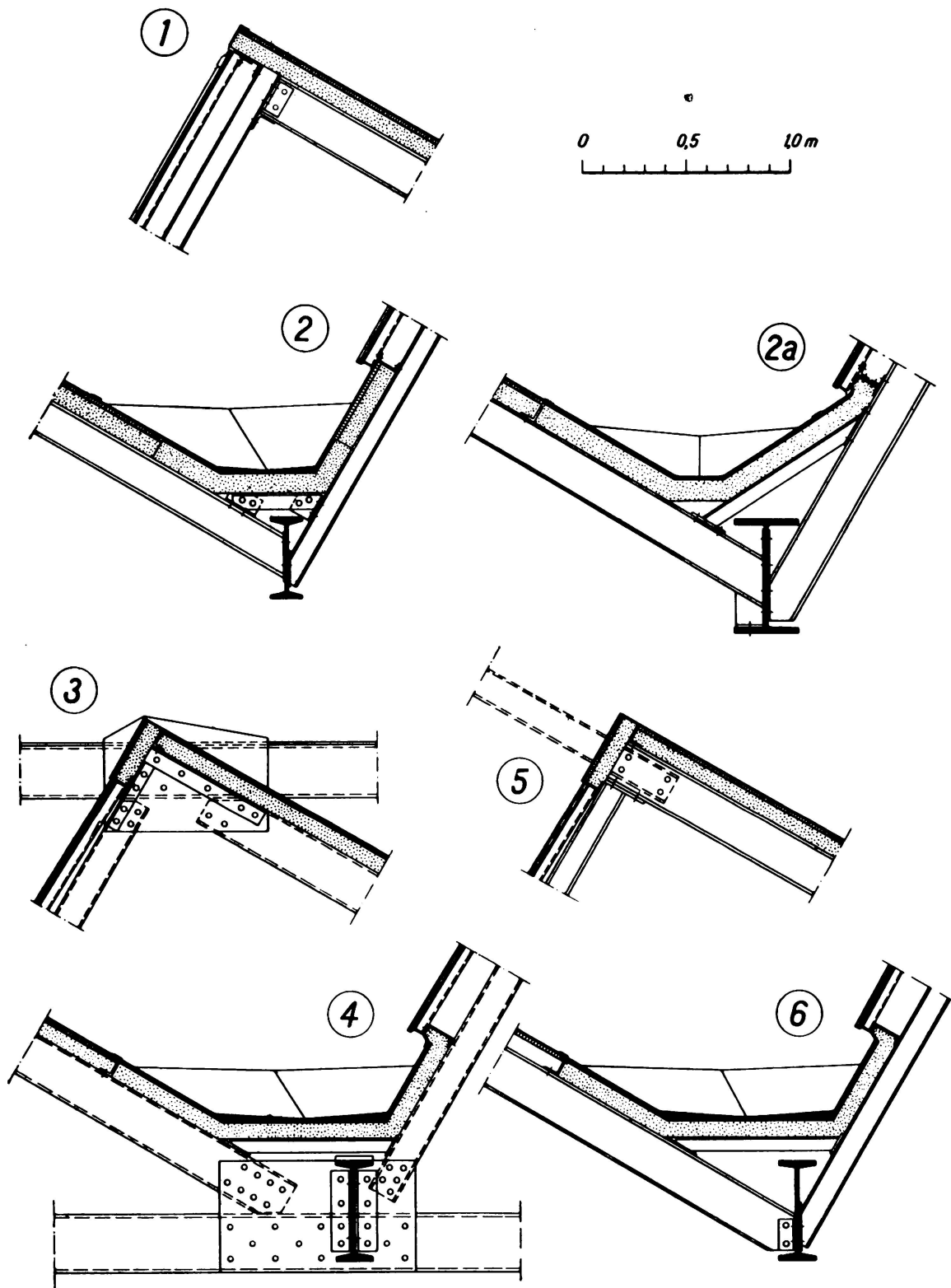


Fig. 10.

Details of the carrying ribs and arrangement of the roof covering for Figs. 8 and 9.

duct. On the transparent floor of the inspection gangway pipe lines can be laid running along the length of the building, and can easily be introduced into the spaces below the saw-tooth roof. The external design of a single storey building is greatly facilitated by the provision of an inspection gangway of this kind, and the objection that may be raised against saw-tooth roofs that they are ugly is in this way overcome.

IV. Among the different possibilities of arranging the *supporting frames of long span saw-tooth roofs*, reference will be made here only to that form in which the most important of the structural elements, namely, the main roof



Fig. 11.

Intersection point in the lower boom of a main girder in the framing system.

trusses, run north and south, i. e. the greatest spans between the stancheons are demanded in this direction. Here two cases are to be distinguished:

Case A: In Fig. 8 the spacings of the supports are at 21 by 10.5 m. The steel skeleton consists mainly of continuous plate girders (H.Bi) together with purlins (U) spanning 10.5 m, and rafters (SP) to which the non-transparent roof covering, in the form of pumice concrete slabs, is directly attached. The spaces under the saw-tooth roof are completely free from constructional members between the main girders.

Case B: Here use is made of continuous trusses of 32 m span and the purlins are so connected as to ensure their operation as continuous beams.

The details most essential to this design are shown in Fig. 10 for both cases. In the building indicated in the sketch 2a the continuous purlins are arranged over spans of $10 + 19.5 + 19.5 + 10$ m.

Fig. 11 shows a point of junction in the bottom chord of the trusses and the above mentioned connection of the purlins with a truss. There also are to be seen the lower inclined surfaces of the gutters. Enough space is left between the lower part of the gutter and the purlin to allow of running pipe lines from one saw-tooth unit to another.

VIIa 6

Observations on the Design of New Belgian Vierendeel Bridges of Wide Span.

Betrachtungen über Vierendeel-Brücken großer Spannweite,
die vor kurzem in Belgien gebaut wurden.

Considérations sur l'étude de quelques ponts Vierendeel de grande portée construits récemment en Belgique.

R. Desprets,

Professeur à l'Université de Bruxelles.

The Vierendeel girder has been adopted for many road and railway bridges. The most important of its applications under standard gauge railway lines are those recently carried out on the Belgian State Railways at Hérenthals over the Albert canal and at Malines, in connection with the electrification of the Brussels-Antwerp line. These works were completed in 1934 and are now in operation.

I. General description.

Hérenthals Bridges (Fig. 1).

The Hérenthals bridges form two series, respectively with double and single tracks, making three spans separated by concrete piers. In view of the considerable skewness of the crossing of the railway in relation to the axis of the canal, and to the desirability of using normal type of supports for the bridges, it was decided to adopt spans of approximately 90 m for the central opening and of 33 m for each of the side openings. It was also deemed expedient to adopt independent bridges carried on simple supports under each of the spans. The central openings are crossed by two straight bridges having their main girders of the Vierendeel type, the side spans by deck bridges with plate webbed girders under the track.

In order to limit the maximum width of the intermediate piers only movable bearings were provided on these, an arrangement which was only rendered possible by making the deck of the central span monolithic with those of the side spans in order to carry all of the longitudinal reactions onto the end abutments.

Malines Bridges (Fig. 2).

The two bridges at Malines were constructed one with a span of approximately 53.50 m over the Louvain canal, and the other with a span of about 90 m over

the Malines-Louvain highway. These structures are in the electrified line between Brussels and Antwerp. They are of single span for double track. The main girders are of the Vierendeel type, those for 90 m span being similar to the corresponding bridge at Hérenthals.

II. Main Vierendeel Girders.

The Vierendeel girders for the Hérenthals and Malines bridges consist essentially of a parabolic arch with a rise of $\frac{1}{7}$ divided into eleven panels. Both were designed by reference to the same numerical tables. All the members of

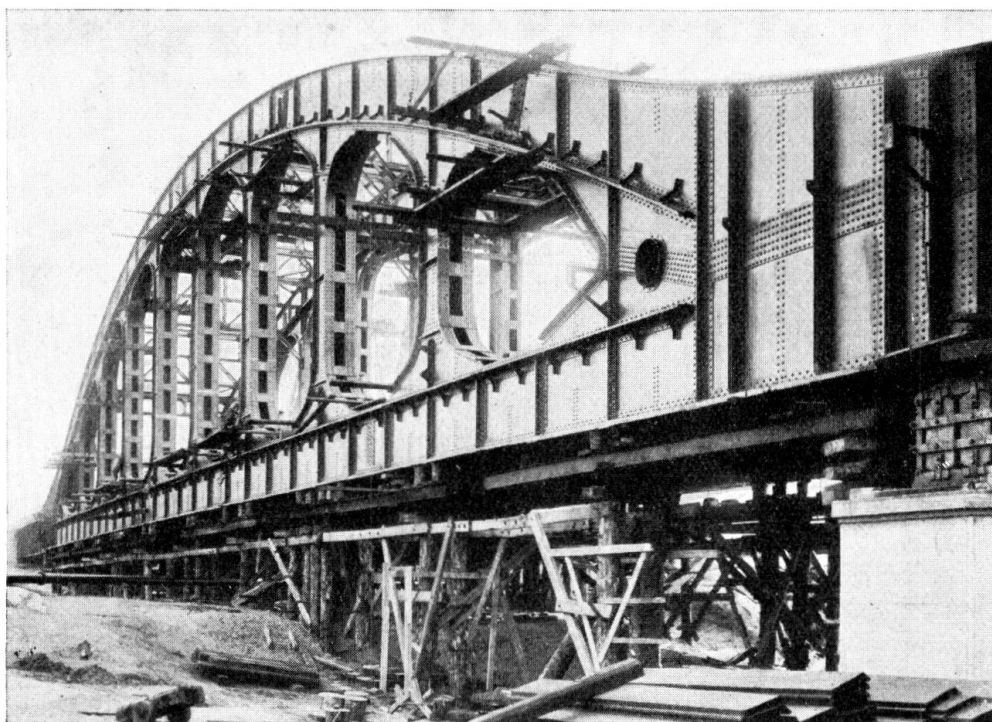


Fig. 1.

Railway bridge with Vierendeel main girders: movable bearing.

these girders are box shaped in section and are made sufficiently large to allow access to a workman for purposes of maintenance. In view of the necessity to take up bending moments in either direction the cross sections are double T; they are built up in the usual way from web plates, angles and flats. In the case of the 90 m bridge girders for the double track it was necessary to make use of special angle sections with legs 180 mm wide. As in ordinary box sections for lattice girders, cover plates have been provided only on the outside of the box, but in view of their width they are inserted between the two angles attached to the web plates and the free end is butted against the web to eliminate any tendency to bulging of the angle legs.

The verticals are, of course, run through the boxes which form the arch and the lower boom, thus making an exceptionally rigid framework. The erection joints for the verticals are made near the ends of the curved portions top and bottom and their exact position is fixed by the effective maximum width of the

gusset plate. The web of the vertical is made continuous through-out the depth of the girder. It should be remarked that the arch sections and the tie sections are continuous from end to end of the girder, an arrangement which is justified if the whole is regarded as acting like a simple bowstring girder under a continuous uniform load.

The box bracing, interior diaphragms and cross bracings were subjected to very close study in detailing, in order to ensure the greatest possible rigidity without wastage.

The girder ends connecting the arch with the tie called for particularly careful investigation in view of the large sizes of plate to be used; it was necessary

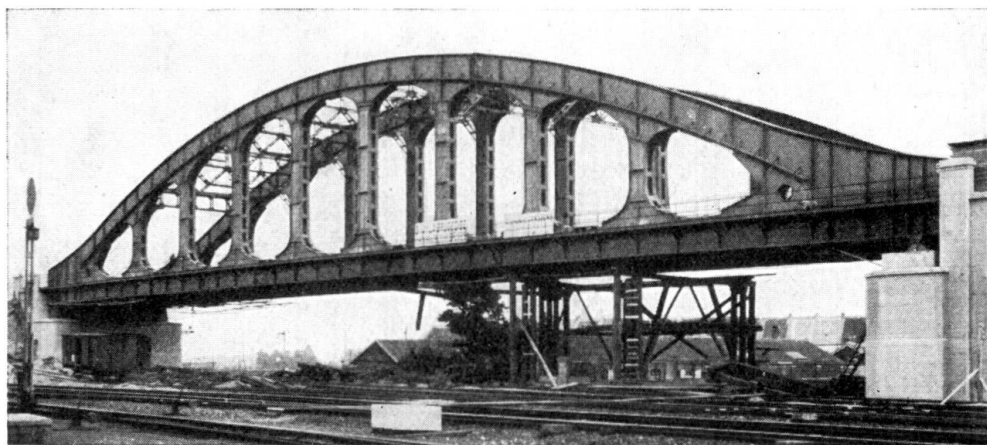


Fig. 2.

Railway bridge over the Louvain road at Malines. Elevation.

to limit the number of joints and provide a sufficient number of stiffeners, while at the same time allowing access for maintenance if required. The principle adopted is the provision of a horizontal diaphragm at an intermediate level with a series of vertical diaphragms pierced by manholes allowing access to any of the internal spaces.

Calculations.

The characteristic of the Vierendeel girder with simple verticals is the absence of any diagonal members and the consequent elimination, according to its inventor, of those secondary stresses which are so harmful in lattice girders on account of the increase in total stress which they may entail and cause to be excessive. Without again entering upon the vigorous controversy on this subject which has been maintained from time to time it may be permissible to observe that frequently what are described as secondary stresses are merely primary stresses, the term being extended to include stresses due to eccentricities of connection which admit of precise calculation and which are the result of forces acting in the ordinary way. It would be better to reserve the term "secondary stresses" to stresses caused by deformation, such as tend to arise in a lattice system on account of the angular deformations due to extension and contraction of the bars.

It is obvious that if the deformability of the intersection points is increased to an extent which makes these approximate to pin joints such stresses will disappear, and the English expression "self relieving stresses" may properly be applied to them. This result may be attained through the play of the rivets or through the plasticity inherent in the gussets and in the bars. Such stresses, thus defined, play a minor part by comparison with that sometimes attributed to them. It should be added that all structures which deform more or less are exposed to these stresses, within limits that vary according to the degree of restraint imposed upon the free interplay of forces by the rigidity of the component elements. In a lattice girder, for instance, calculations and experiments agree in indicating that these stresses are practically proportional to the inertia per unit length $\left(\frac{I}{l}\right)$ of the element.

The present girders were designed in accordance with the simplified method explained by M. Vierendeel in his course on the stability of structures. From the tables of diagrams in this work the values of the shear forces at the points of intersection of the verticals and those of the bending moments in the booms and in the verticals were read off for each loaded panel point, these elements of calculation being applicable directly to all girders of the same ratio between rise and span and the same number of panels. When the girder is under uniformly distributed load covering the whole span the force in the arch becomes a simple compression along its axis and the tie bar is uniformly stressed in tension between its supports, while the verticals are in simple tension from the weight of the floor.

It is of interest to note that in the prismatic central portions of the boom the stresses calculated on the hypothesis of a live load over the whole span are greater than those which result from partial loading. The latter condition would be less favourable only in the case of curved connections between the verticals and the booms on the assumption that these are of prismatic section. On the other hand the design of the verticals is conditioned by bending stresses when the bridge is partially loaded.

If the bending moment diagram for a panel length of the boom is examined, this diagram having been plotted on the usual hypothesis of constant cross section over the whole length of the panel, it will be seen that for certain conditions of loading the point of zero bending moment may be either within the width of the vertical member or within the covered portion of the connections. Having regard to the great increase in section as between the verticals and the booms, and to the wide extent of these transitions, it seems unlikely that these points of zero bending moment (coinciding with the points of inflection) will be displaced much outside the prismatic central portions of the booms. At any rate it may be inferred that the initial assumption of a constant moment of inertia will lead to conclusions that must be accepted with caution proportionate to the extent of the transition in comparison with the prismatic portions of the verticals and the booms. The limiting case would be reached in a girder consisting of a succession of triangles touching one another at the central points of the booms and verticals. It is, therefore, difficult to ascribe any great accuracy to a method of calculation for a Vierendeel girder which does not take account of the variations

of the moments of inertia due to the transitions, and it would appear that a simpler method of fixing the point of inflection in the prismatic portions of the booms *a priori* would give satisfactory results not necessarily more erroneous than are obtained by the method alleged to possess greater accuracy. Such a method has been devised by the German engineer Engesser and described by him in the *Zeitschrift für Bauwesen* in 1913. Engesser assumes that the verticals possess infinite rigidity, and he deduces from this that the points of inflection on the booms would be situated on the vertical line passing through the centre of gravity of each panel of the girder.

The fixing of fictitious hinges is immediately accomplished, and allows of a simple and rapid calculation of the different isostatic sections of the girder.

Relative calculations carried out to determine the stresses in a girder of about 100 m span for a railway bridge, single track, using the two methods of Vierendeel and Engesser respectively, indicate that the approximate method gives satisfactory results. It is only right to mention that before Engesser, M. Vierendeel himself had pointed out a similar simplification applicable to girders with parallel booms.

It should be noticed that the ratio of the linear moments of inertia (ratios of moments of inertia to length) in the elements of the boom, to the corresponding figure for the verticals, is of basic importance in defining the action of a Vierendeel girder. The limiting conclusions can easily be ascertained by making use of the general relationship given below (due to *Keelhoff* in his "Cours de Stabilité"):

$$\frac{(I'c)^3 + I''^3}{(I'c + I'')^3} \left[H_n^3 \frac{Z_n}{I_n} - H_{n-1}^3 \frac{Z_{n-1}}{I_{n-1}} \right] = \frac{3\lambda}{2} \frac{H_{n-1} + H_n}{I'c + I''} (M'_n + M''_n)$$

Here a single panel of the girder is considered, with the heights of the verticals given by H_{n-1} , H_n and the moments of inertia by I_{n-1} and I_n . The normal width of a panel is λ , the moments of inertia of the upper and lower booms are assumed to be constant at I' and I'' . The upper boom forms an angle φ with the horizontal for which $\cos \varphi = c$, and the lower boom is itself horizontal. If a section is taken through a panel along a vertical line passing through the centre of gravity, the bending moments in the upper and lower booms respectively are M'_n and M''_n , while Z_{n-1} and Z_n are the horizontal shear forces acting at the points of contraflexure of the verticals.

Recalling the fundamental hypotheses,

$$\frac{M'}{M''} = \frac{I'c}{I''} \quad \text{et} \quad \frac{h'}{h''} = \frac{I'c}{I''}$$

h' and h'' fix the position of the point of inflection in a vertical.

To disclose more easily the limiting conditions we will suppose that $I' = I'' = I$

$\frac{I}{\lambda} = \beta$ is the linear inertia of the lower boom,

$\frac{I_n}{H_n} = \frac{I_{n-1}}{H_{n-1}} = \alpha$ is the linear inertia of the verticals

$$\frac{1 + c^3}{(1 + c)^2} = K.$$

The general relationship takes the following form

$$H_n^2 \cdot Z_n - H_{n-1}^2 \cdot Z_{n-1} = \frac{\alpha}{\beta} \cdot \frac{1}{K} \cdot \frac{3}{2} (H_{n-1} + H_n) (M'_n + M''_n)$$

wherein the ratio $\frac{\alpha}{\beta}$ between the linear inertia of the verticals and the booms appears as a principal coefficient.

The limiting values of $\frac{\alpha}{\beta}$ are ∞ and 0, the value $\frac{\alpha}{\beta} = \infty$ or, reciprocally, $\frac{\beta}{\alpha} = 0$ corresponding to Engesser's assumption that the moment of inertia of the verticals is infinity. Applying $\frac{\beta}{\alpha}$ to the first member, the hypothesis $\frac{\beta}{\alpha} = 0$ implies M'_n and $M''_n = 0$ and since M' and M'' are of the same sign, $M'_n = 0$ and $M''_n = 0$.

We may infer that the sections of the booms on a vertical line passing through the centre of gravity of the panel are the sections of zero bending moments, whatever the loading. If nothing but the simple bending of the booms is considered, these sections will correspond to points of inflection. In the case of a girder of constant height these points will be situated at the centre of each panel.

The other limiting value $\frac{\beta}{\alpha} = 0$ corresponds to the case where the verticals possess zero inertia, as is practically the case in bowstring girders with thin suspension bars, and as would also be true of two parallel booms of equal inertia connected by vertical pin-jointed rods.

$$\begin{aligned} H_n^2 \cdot Z_n &= H_{n-1}^2 \cdot Z_{n-1} \\ Z_{n-1} &= Z_n \cdot \frac{H_n^2}{H_{n-1}^2} \end{aligned}$$

Z_{n-1} has the same sign as Z_n the proportion being that of the squares of the height of the verticals.

Under the hypothesis of vertical loads $\Sigma Z = 0$.

In the case of a girder with parallel booms $\Sigma Z = 0$ which may be written

$$Z_n \cdot H_n^2 \cdot \sum_0^m \cdot \frac{1}{H^2} = 0$$

and reduces to $Z_n = 0$. All the shear forces in the verticals are zero.

In the case of a bowstring girder with thin suspension bars the summation ΣZ_n includes a term

$$Z_0 = Z_n \cdot \frac{H_n^2}{H_0^2}.$$

If $Z_n \geq 0$, H_0 being zero, Z_0 would be infinite.

Now, in this case the value of Z_0 is determined and finite, as it corresponds to the horizontal component of the axial thrust in the arch. For Z_0 to be finite while H_0 is zero it is necessary that Z_n should be zero, a conclusion which brings us back to the ordinary definition of a bowstring girder with thin vertical members pin-jointed to the arch and tensile boom.

VII b

Application of steel in hydraulic construction.

Anwendung des Stahles im Wasserbau.

Application de l'acier dans la construction hydraulique.

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VIIb

General Report.

Generalreferat.

Rapport Général.

Dr. Ing. K. Klöppel,

Leiter der technisch-wissenschaftlichen Abteilung des Deutschen Stahlbau-Verbandes, Berlin.

During the last few decades the use of steel in hydraulic engineering has increased at a rate which makes it appropriate for this special field of application to be reviewed and assessed, and seeing that steelwork applied to hydraulics has much in common with steelwork in general it has fittingly been included in the programme of the present Congress.

As an amplification of the reports and contributions to the discussion which outline the developments of steel construction in hydraulic work and provide illustrative examples, it is proposed here briefly to summarise those questions of materials which the problems have in common.¹ First and foremost among these is the problem of *corrosion*, which is one of great economic importance even though there is no need to accept the astronomical figures which are frequently (on insufficient grounds) put forward as estimates of the total annual loss by rusting.²

The structural engineer who uses steel as a building material will be unable in future to escape the necessity of some attention to the fundamental questions of corrosion, a field in which he has an important contribution to make to the essential collaborative work.

Too much stress should not be laid on the fact that the reports submitted disclose only rare instances of severe damage. In the construction of cofferdams, especially, our experience does not yet extend over a period long enough to be decisive regarding the resistance of steel to corrosion, but the available records appear ample and favourable enough to refute the objections that used

¹ The necessary shortening of this report has been secured by omitting such portions as might be looked upon as partial repetitions of the papers covered.

² In the lay and technical press the annual loss through corrosion of iron and steel, in Germany, is frequently put at 2 000 000 000 RM, but the impossibility of the loss reaching this order of magnitude is obvious at once from the fact that in the good year 1929 the total production of steel reached a value of only 2 000 000 000 RM. *Schaper*, working from a sounder foundation, arrives at an estimated value of 120 000 000 RM. (*Stahl und Eisen*, 1936, p. 1249), and *Daeves* puts the annual loss through rusting of German rolling mill products used in steel building work, bridges, shipbuilding, etc. at a maximum of 18 000 tonnes corresponding to 700 000 RM.

often to be urged against the suitability of steel as a material for hydraulic work. To-day our experience, even in the matter of sheet piling, already covers periods corresponding to the economic life under traffic of most of our works.

The essential point about the difficulties attending the problem of corrosion is that we do not know the order of importance of the diverse and numerous destructive influences that operate together, and as long as we continue to be without a criterion for distinguishing the principal from the incidental circumstances of corrosion we run the risk of neglecting influences which alone can provide the explanation of differences in the behaviour of steel structures similarly situated. The chief lesson to be drawn from this is that whenever practical observations are made it is necessary to insist that as many detailed data as possible be recorded which may throw light on the subject. In this connection it is necessary to take account of distinctive circumstances attending each case under observation even when, so far as our present knowledge goes, they appear to have no connection with the phenomena of deterioration.

Many notable contributions to the research of corrosion (for instance, the copper alloying of steel) are attributable more to chance observations than to planned investigations. Progress depends, therefore, on the co-operation of several different groups of technicians who have at their disposal varying possibilities of observing the phenomena of corrosion. Hence in the field of corrosion research the collection and digestion of experience and observations plays an even more important part than in others, for it is well known that "accelerated tests" to assess the value of a particular kind of paint or the qualities of a particular kind of steel are of very doubtful value. The only means of compressing the process of corrosion into a shorter period of time are by strengthening the corrosive medium, increasing the temperature and increasing the rate of movement of the specimen, and the limited extent that results obtained from laboratory experiments of this kind are suited for practical generalisation is shown by the very fact that indices of resistance to corrosion comparing one metal with another fall in an entirely different sequence according to the particular acid which has been used for the test.

In our special field of work the principal weapons against corrosion are to be found in the further development of paints and of low rusting steels. Metalising processes may be left out of account since they have attained to no significance in hydraulic work.

Underwater paints, exposed as they are to mechanical, chemical, as well as botanical and zoological effects, are made the object of requirements which partly contradict one another. There can, therefore, be no one kind of paint which is equally suitable from all points of view, and it is all the more necessary to ascertain by numerous observations on completed works which points of view are of the greater importance in the choice of such a paint. There is a lack, in all countries, of any suitable principle for the collection of such information. Recently, with a view to filling this need, co-operative work on a generous scale has been begun, such as the discussions and the full sized experiments under natural conditions carried out by the paint committee of the Verein Deutscher Ingenieure; a step which encourages the hope that important progress will be made.

Hydraulic structures should preferably receive a finishing coat based on bitumen or coal tar. In sea water a hot coating (bitumen without a solvent) is used; in fresh water a cold coating of bitumen dissolved in benzol hydrocarbons.³ For the priming coat red lead continues to be preferred despite the danger of blistering due to the linseed oil it contains, but with hot painting the red lead priming coat is often held to be unnecessary. The red lead must be allowed to harden in order that the benzol of the bitumen coating which follows may not be further dissolved, and the hardening process takes from two to five weeks. This entails great difficulties in the erection and maintenance of hydraulic structures of steel, and these have led to the development of a type of red lead which dries quickly and under unfavourable conditions of weather.⁴ Success appears to have been attained even in the use of special combinations of oils and resins to produce a special type of red lead which hardens adequately in a few hours, and is insensitive to bitumen dissolved in benzol.⁵ The time required for the drying of the red lead can also be considerably reduced where benzine hydrocarbons can be used as the solvent for the bitumen. Recent experiments with paints based on *chlorinated rubber* (a material which also shows smaller sensitivity to light than the bituminous paint) promise good results with this form of protection, particularly from the point of view of resistance to abrasion, a consideration not negligible in hydraulic work.⁶

Low rusting steels have been obtained chiefly by the addition of *copper* up to 0.3 %, ⁷ but the increase in corrosion resistance is effective only against atmospheric attack and not against the continuous action of water. Copper-bearing steel rusts almost like ordinary steel, but a surface deposit of copper or copper oxide is gradually formed which combines with the rust to form a dense and strong protective layer, and this greatly retards further deterioration. If kept permanently damp, however, the layer of iron oxide becomes spongy and loses its protective value, this being the explanation of many unsuccessful experiences with copper bearing steel in hydraulic engineering.

Further development has proceeded in the direction of introducing other alloy elements with a view to increasing the corrosion resistance. It was found, for instance, that a relatively high phosphorus content (such as is characteristic of almost all weld iron) when combined with the copper content renders the protective layer very dense and also causes it to form very quickly.⁸ The superiority of the more recent low rusting steels is attributable to this discovery. Other researches to determine favourable proportions for alloying copper with phosphorus and also with aluminium, chromium and nickel encourage the hope that economical types of steel may be discovered which possess adequate corrosion resistance in water. The fact that the resistance of different

³ *Kindscher*: Stahlbau 1935, Nos. 5 and 6, p. 161.

⁴ *E. Meier*: Bautechnik 1934, p. 577.

⁵ *E. Meier*: Industrie-Lackier-Betrieb 1935, p. 1—6.

⁶ *Kappler*: Z.V.D.I. 1936, N° 7, p. 183. — *Ballé*: Der Rhein 1935, N° 2, p. 39.

⁷ *O. Carius* and *Schulz*: Mitteilungen aus dem Forschungsinstitut der Vereinigten Stahlwerke Dortmund 1928—1936, p. 177.

⁸ *K. Daeves*: Naturwissenschaften 23 (1935); 38, p. 563; idem: Mitteilungen der Kohle- und Eisenforschung G. m. b. H. 1935, p. 186.

kinds of steel is naturally influenced by the composition of the aggressive fluid is reflected in the circumstance that the copper content increases the corrosion resistance of steel in dilute sulphuric acid but not in pure water, while in the presence of nitrates the copper appears, indeed, to operate adversely.⁹

The intermittent immersion tests in artificial sea water carried out by *Eisenstecken* and *Kesting*¹⁰ have indicated, also, the great extent to which the corrosion of the steel depends both on the duration of the experiment and on the period of immersion, and have helped to explain the observation often made in practice

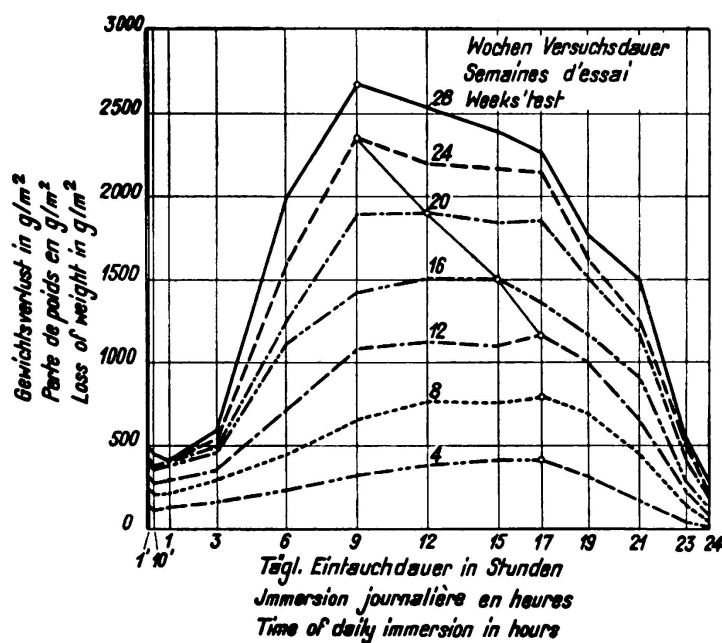


Fig. 1.

Repeated immersion of a mild carbon steel in seawater.
Seawater changed every four weeks.

that corrosion occurs in the places that are exposed to air and water alternately. The series of experiments represented in Fig. 1 covered a period of 28 weeks, the water being changed at the end of each four weeks. The specimens tested were small plates of mild carbon steel containing 0.08 % copper. A curve is plotted for each four weeks. Heavy corrosion of the specimens is observed only when the period of immersion is increased to about six or seven hours, and after the immersion the loss of weight becomes considerably slower. The effect of the duration of the test is seen in the fact that after four, eight and twelve weeks the heaviest attack was found to occur with 17 hours immersion, whereas at the end of 16 weeks it occurs with an immersion of 15 hours per day, and at the end of 28 weeks with nine hours immersion per day. The displacement of the maximum value in the direction of a shorter immersion period as the length of the experiment increases may be explained by the increasing water content of the rust layer. It may also be concluded from this that it is only with great caution that intermittent immersion tests of the kind frequently practised in the laboratory can be used as a basis for comparing the corrosion resistances of different kinds of steel and protective media. The results also show that the corrosion resistance of one and the same steel in its unprotected condition (as for instance in sheet piling) may vary greatly according to local circumstances.

Metallurgical expedients to increase the corrosion resistance are effective even in steel parts coated with paint. This welcome fact which has been established

⁹ Büttner: Bücher der Anstrichtechnik 1936, 1. Buch, V.D.I. Verlag 1936, p. 28.

¹⁰ Bericht über die Korrosionstagung 1935, V.D.I. Verlag, p. 48.

in experiments by Daeves¹¹ may be explained from the consideration that any weak spots in the covering, such as are bound to manifest themselves sooner or later however good it may be, will be countered by a retardation of rust formation at the affected parts because after a short time the thin layer of copper which separates out of the steel will cover them over.

Since the renewal and maintenance of paint coatings in hydraulic structures is particularly wasteful of time and money, there is here even more relevance than usual in the observation that it is uneconomical not to employ high resistance paints merely on account of their high price, for in any case paint represents only about $\frac{1}{5}$ th of the total cost¹²) and an increase in this item may be balanced by an increase in the life of the treatment relatively several times as valuable.

It is a matter of experience that the preparation of the steel surface to receive the coating of paint exerts a decisive influence on the life of the coating. Any defective places in the rolling skin must of course be dealt with, especially scale and rust, and here again no expense must be spared. It is worth mentioning, however, that a thick rolling skin provides in itself a very useful natural protection against rust. In the further development of low rusting steels — particularly those which are to be used for sheet piling without paint, or are only to be painted at long intervals — special attention should be paid to the systematic production of a dense and effective rolling skin. Fairly pure iron (such as Armco iron) forms a very uniform surface which offers an excellent ground for the application of paint. Apart from metallurgical effects of this kind temperature and the nature of the rolling process may possibly play an important part in the formation of a suitable rolling skin. The results obtained in treating the surfaces of small steel parts by the use of phosphates also offer a stimulus to further exploration of this avenue. The fact that the rolling skin may offer a very good rust protection is seen from a number of favourable experiences on record. For instance, *Hoffmann*¹³ reports the good condition of the paint on the North Elbe bridge broken up at Hamburg where, after several decades, the red lead was so firmly bonded to its base (described as a bluish mill scale) that it was impossible to separate it. It cannot be assumed that these steel parts had been pickled. It is difficult, however, to distinguish satisfactorily between the rust protective rolling skin on the one hand and mill scale or layers of rust on the other, and for this reason, in addition to the introduction of the sand blasting process, the tendency is to do away with the rolling skin altogether. Nevertheless the possibility of so developing the rolling skin as to form a natural protection against rust should not be neglected. Moreover, in sanded steel parts, the transition zone between the oxide layer and the steel is not usually eliminated; hence no metallically clean surface is ensured, and as the surface is known to be particularly susceptible to corrosion rapid painting or other measures to prevent rust formation are necessary though not always successful.

¹¹ Daeves: *Farbe und Lack* 1931, N° 21, p. 242.

¹² Klöppel: *Unterhaltungskosten von Stahlbauwerken*, Noske, Leipzig.

¹³ Dissertation, Technische Hochschule, Hannover 1921.

The further development of structural steels in the direction of increasing the permissible stresses is a matter of relatively small importance in hydraulic engineering by contrast with bridge work, since only small spans have to be covered. Indeed in many cases it is desirable to counter the dynamic effect of the water by means of a heavier mass of steel, and this explains why *high tensile steel* finds only exceptional application in hydraulic work. For sheet piling, again, according to Professor *Agatz*, ordinary structural steel is preferred, because a uniform thickness of the rusting layer weakens the cross sections of ordinary steel relatively less than that of high tensile steel, while moreover a greater resisting moment is presented and smaller deflection occurs in the sheet pile wall, and it is easier to maintain the alignment of the piles in driving them. Increased strength is, however, essential in overcoming heavy driving resistance and is desirable where heavy surface abrasion is encountered.

The *conditions of acceptance* for sheet piling are in general the same as those for steelwork in general, but it is difficult to say how far this practice is justifiable in view of the very different conditions of stress arising. Experience shows, indeed, that the criteria of quality hitherto imposed are in no way defective, but that is not to deny the possibility that other test values for steels might provide a better indication of their suitability for sheet piling. On this account, if the conditions of acceptance which have hitherto operated be found a hindrance to the further development of steel sheet piles it would be wrong to hesitate too much in departing from them.

Hydraulic engineering provides yet another field in which *welding* offers great advantages, as may be seen particularly in the movable hydraulic structures on the Albert Canal in Belgium. The monolithic character of welded steel structures tends to confer on them a greater degree of stiffness by comparison with heavy riveted work, and this is of great advantage especially in the case of flat constructions such as lock gates, where these are built of steel. The greater watertightness and the more convenient maintenance due to freedom from gaps and joints are great advantages, as is also the simplified design by this means of torsionally rigid structures, which are of great importance in hydraulic work.

To the steel engineer, the determination of the pressure and suction of water and the devising of measures to overcome vibration present additional difficulties. In order to solve these problems, as illustrated in examples by *Burkowitz*, a knowledge of the physics of stream flow is necessary, and this is obtainable from the theory of hydraulics. It would be in the best interests of hydraulic engineering — and particularly of the education of engineers for work in this field — that co-operative work between the structural steel and the hydraulic departments should be undertaken in the Universities.

VIIb 1

Steel in Hydraulic Engineering, and Model Experiments.

Stahlwasserbau und Modellversuche.

Constructions hydrauliques en acier et essais sur modèles.

Dr. Ing. e. h. Th. Becher,

Direktor der M.A.N., Werk Gustavsburg.

In the paper by *Burkowitz* and in all the contributions to the discussion on "Application of Steel in Hydraulic Construction — Movable Plant" reference is made to hydro-dynamical effects, vibration, etc. It is proposed here to develop that section of the subject which relates to the design of steel hydraulic structures on the basis of model experiments for studying the hydro-dynamical effects.

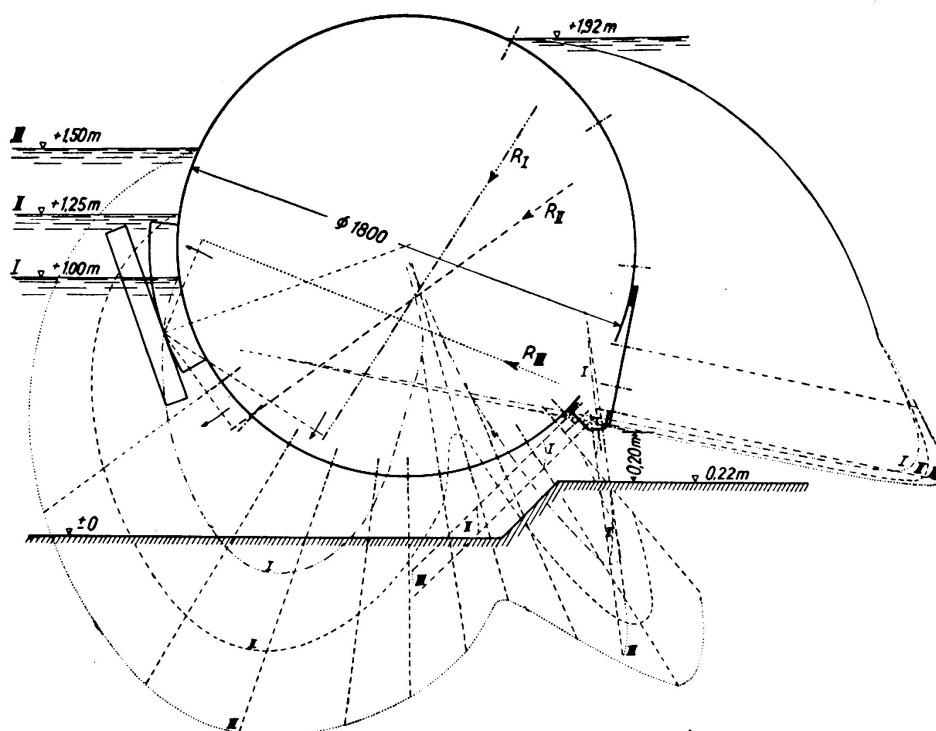


Fig. 1.

This study was first embarked upon as the result of certain diseases of infancy in the hydraulic plants which had to be cleared up by experiments on models, and the second phase was to use such experiments as a means of deducing principles capable of application for measuring the forces and dimensioning the openings and operating gear in plant designed in the ordinary way. From this

it was easy to pass to the next stage, namely that of using model experiments to develop forms which would offer the most favourable possible hydro-dynamical qualities. A few examples will now be given.

Among the earliest roller weirs there were some which consisted merely of a supporting cylinder with a small attachment for stop planks. On one occasion such a weir was seen to be in a vigorous dancing motion, which finally led to the destruction of the whole gauge house and to the roller coming out of its

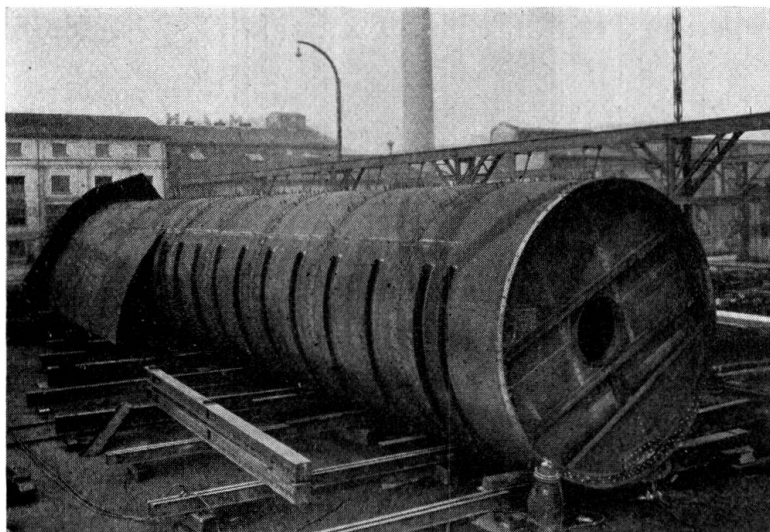


Fig. 2.

guides. As it was difficult to explain this occurrence otherwise than on the assumption of some outstanding fault in operation, experiments on models were undertaken (Fig. 1). These showed that with a roller and crest of that shape the current of water shooting over the top experienced violent variations of pressure with the result that the roller was subjected alternately to pressure and suction

effects. As a result of this knowledge all rollers were subsequently made with larger shields (Fig. 2). The most favourable relationship between the diameter of the roller and the height of the inlet was also experimentally determined. In the double gates of a large weir the upper gates were provided with a cover which the overflow water was discharged when the upper shields (upstream) were lowered. This cover consisted as shown on the left of Fig. 3 of a flat wooden sill inclined downstream. After every period of high water it was found that these overflow sills were to some extent damaged and required renewal. The wooden sill was replaced by a steel sill, but before doing

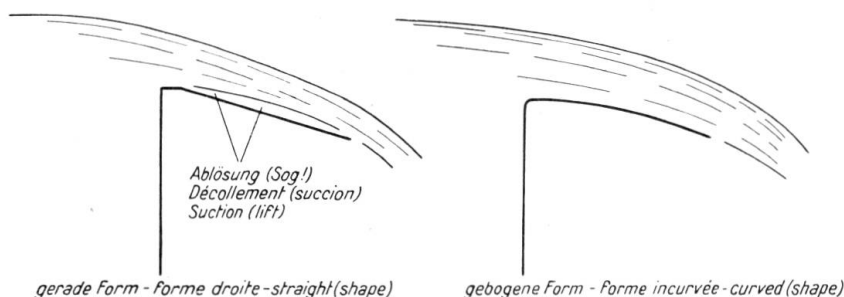


Fig. 3.

so experiments were carried out to a scale of 1 in 5 to determine the proper shape for the plate, whereby a curved shape was decided upon, as shown on the right. The original inclined shape was found to cause an entirely irregular distribution of pressure, with numerous suction hollows, and the form finally decided upon for the overflow plates gives a harmonic distribution of pressure without any such suction hollows. At the same time it was found possible to incorporate two further impor-

tant improvements. As it was difficult to explain this occurrence otherwise than on the assumption of some outstanding fault in operation, experiments on models were undertaken (Fig. 1). These showed that with a roller and crest of that shape the current of water shooting over the top experienced violent variations of pressure with the result that the roller was subjected alternately to pressure and suction effects. As a result of this knowledge all rollers were subsequently made with larger shields (Fig. 2). The most favourable relationship between the diameter of the roller and the height of the inlet was also experimentally determined. In the double gates of a large weir the upper gates were provided with a cover which the overflow water was discharged when the upper shields (upstream) were lowered. This cover consisted as shown on the left of Fig. 3 of a flat wooden sill inclined downstream. After every period of high water it was found that these overflow sills were to some extent damaged and required renewal. The wooden sill was replaced by a steel sill, but before doing so experiments were carried out to a scale of 1 in 5 to determine the proper shape for the plate, whereby a curved shape was decided upon, as shown on the right. The original inclined

tant improvements in the new design: the water pressure load on the wall was reduced from 82 tonnes in the old to 38 tonnes in the new form, with a corresponding reduction in the power required for operation, and at the same time the discharge capacity was increased by some 20 %, showing that with the aid

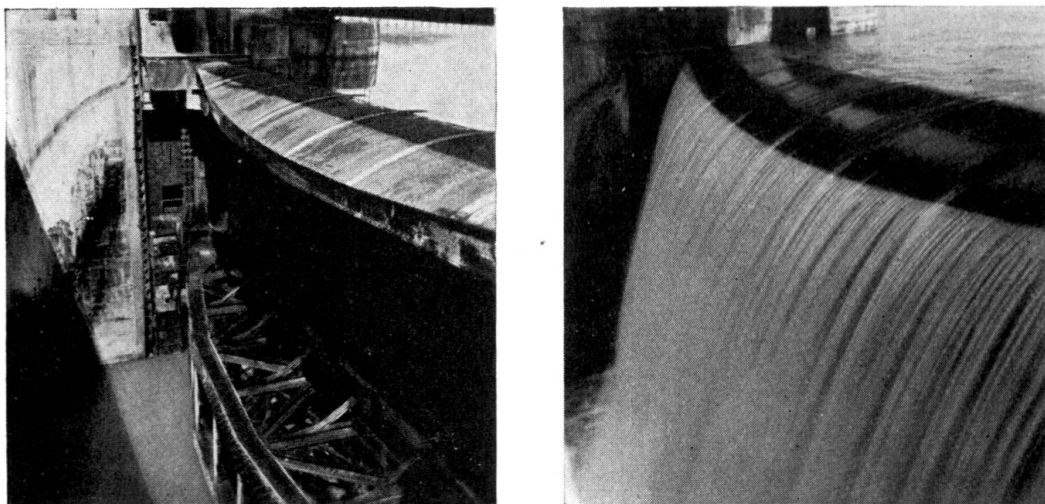


Fig. 4.

of a suitable overflow sill it is possible to economise in the length of the weir. Fig. 4 shows the constructional arrangement of a weir with one of the M.A.N. "hook gates" at Ryburg-Schwörstadt with the overflow shutter of the upper stoney-sluice. With a total height of 12.5 m these gates allow the upper gate to be dropped by 4.5 m. Their dynamical behaviour thus calls for very careful study.

A further problem was that of the vibration experienced by weirs when water flows underneath where only a small opening exists, an effect which was apt to lead to damage when the retained head became large. It was soon found by experiment that the cause of these vibrations lay in the design of the wooden sill and that by taking suitable measures the trouble could be mitigated (Fig. 5). Here again it is a question of obtaining a positive and stable line of pressure.

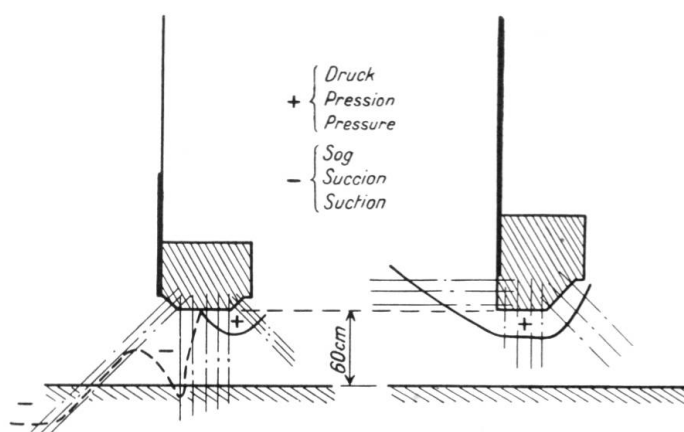


Fig. 5.

In yet other weirs of the stoney-sluice or bear-trap type vibrations were found to occur where there was an excess flow, especially where the height was limited. First of all, by experiment, ample ventilation of the space between the gate and the overflow was ensured, but this did not always cure the trouble, for it was found that the thin, coherent stream of water flowing over the flat sill was

extremely sensitive to every impulse, and thus in itself tended to cause vibration. The experiments also showed however that this sensitiveness of the water stream is destroyed if cohesion is disturbed throughout its length, as by cutting it up or making it wavy (Fig. 6). This was accomplished by the introduction of

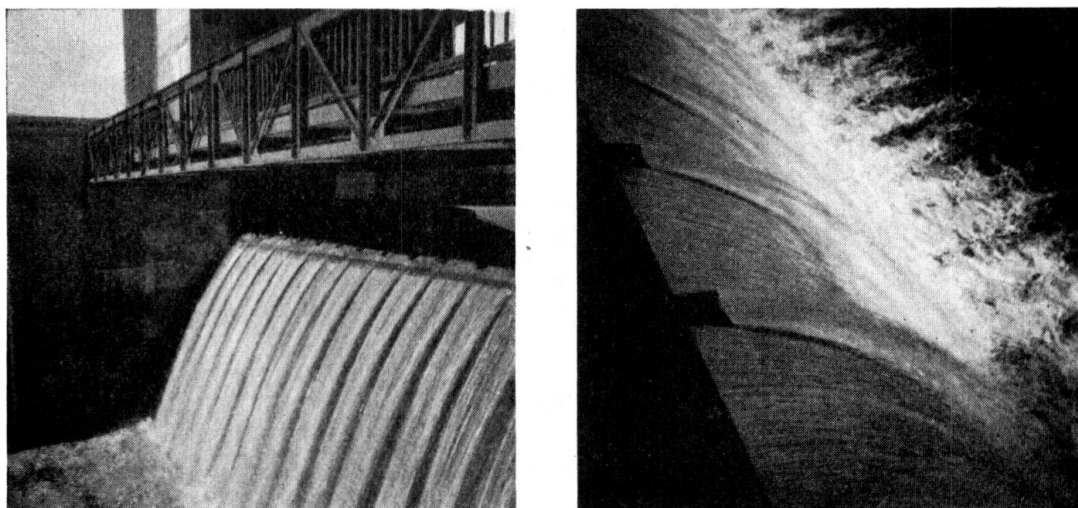


Fig. 6.

dovetailed strips of plate on the sill of the weir, and the illustration shows the stream flowing over the gate after this had been done.

Again, in the M.A.N. "fish bellied shutter", the chief feature of which is the operation from one side (Fig. 7), a current interrupter of this kind has been

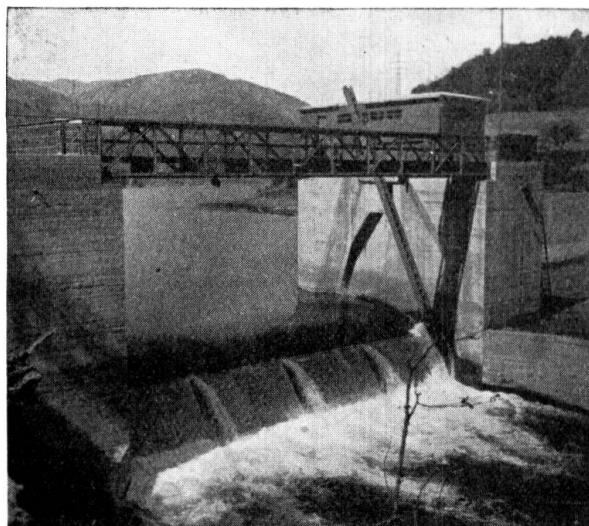


Fig. 7.

introduced. The example shows a sill of this kind in the weir at Heinbach measuring 18.0×4.0 m.

In the case of trap weirs it is also found that when the tail-race water rises, with the barrage down, alternating forces are set up therein, and in the lowest position these may easily tend to develop vibrations. The idea arose of so forming the barrage that in all cases its weight, together with the pressure of the water, would produce a positive moment in the direction of overturning on the body of the weir thus holding it steady (Fig. 8). The curves in the diagram

give the turning moment due to the water pressure in the different positions of the gate, the small negative portion below the abscissae being compensated by the moment due to the own weight of the structure which always acts in a positive direction. The problem was solved only after numerous measurements of pressure had been carried out in the testing laboratory. Indeed it is essential, if accurate work is required, to carry out a fresh

set of experiments for each new weir, since the shape of the fixed portion of the weir and of the tailrace, the depth of water in the latter, and the possible height of water over the sill, are always different and all affect the result. In the course of these experiments accurate determinations were also made of the turning moment due to the pressure of the water to be carried by the body of the barrage, and also of the operating pressure on the turning gear; at the same time the installation was calibrated, that is to say the discharge over the weir was measured for each stage of water level.

A particularly difficult problem arises in the design of under water closures (Fig. 9). Here the M.A.N., with the aid of their laboratory resources, have achieved several successful new solution during the past few years. This entailed

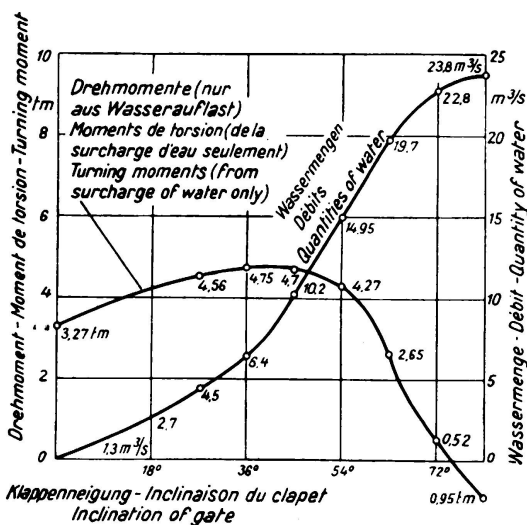


Fig. 8.

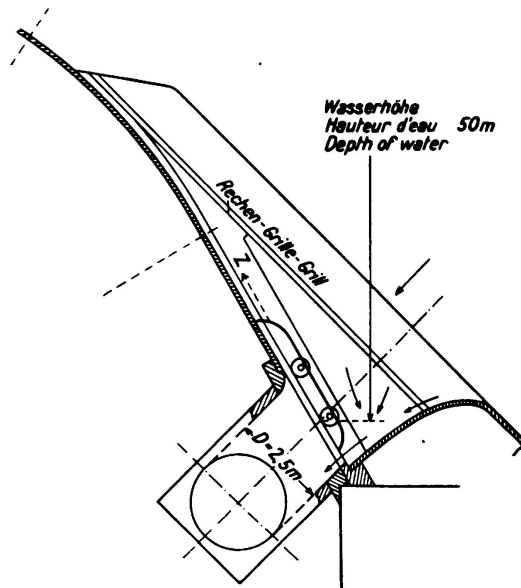


Fig. 9.

working in the laboratory with very considerable pressures in order to enable the transition to be made to full scale. The illustration (Fig. 10) shows the under water gate used in the Odertal dam in the Harz mountains, which is designed to withstand a head of water of some 50 m. As the weir is required closed under its own weight when subject to the full pressure of water, very extensive experiments were necessary to determine the design of the weir and of the structure as a whole.

The contest with vibration, and the study of the effects of moving water which it entailed, has had yet other repercussions on design, for these are factors which influence not only outward arrangement of the design but also its detailed dimensions. Flowing water must always be opposed by a definite mass, but the possibility suggests itself of reducing the moving masses in movable weirs by more suitable construction or by the use of high grade materials. Considerations of cost either of the construction or of the electric current required for operating the weirs should not be allowed to interfere with such developments, for in weirs where such operation is seldom required this factor is of no importance, and in the cases of sluices or lifting gear the same consideration applies. The choice of St. 52 as a material would not appear to be very appropriate in the

construction of weirs unless special local considerations make it necessary, for the elasticity and therefore the tendency to vibration become greater in structures where this is used. The permissible stresses, moreover, should not be as high as

in ordinary steel work, having regard to corrosion. For instance in steel applied to hydraulic engineering St. 37 should not be stressed in excess of 1200 kg/cm^2 .

Finally, a few illustrations from workshops and finished weirs may be given as indications of the magnitude now attained by steel structures in application to hydraulic work. Two of these examples will be drawn from the numerous large weirs on the Main and Neckar canalisation. Firstly Fig. 11 shows a barrage with three-boom framing and fish-bellied shutter for the installation of Faulbach in the Main-River during the process of erection. This barrage 35.0 m weir width and 6.7 m high, of which 1.60 m is taken up by the movable member. Among the Neckar weirs the installation at Heidelberg (Fig. 12) is especially notable, both on account of the pleasing way in which it has been incorporated in the landscape of the

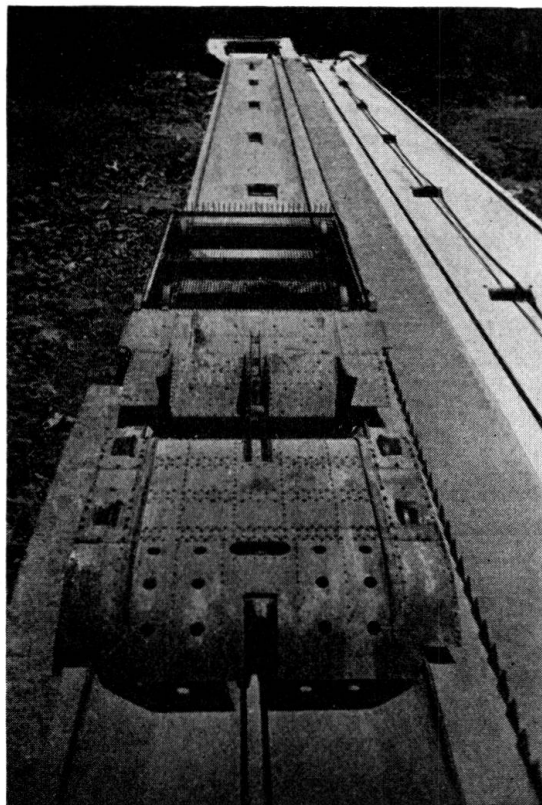


Fig. 10.

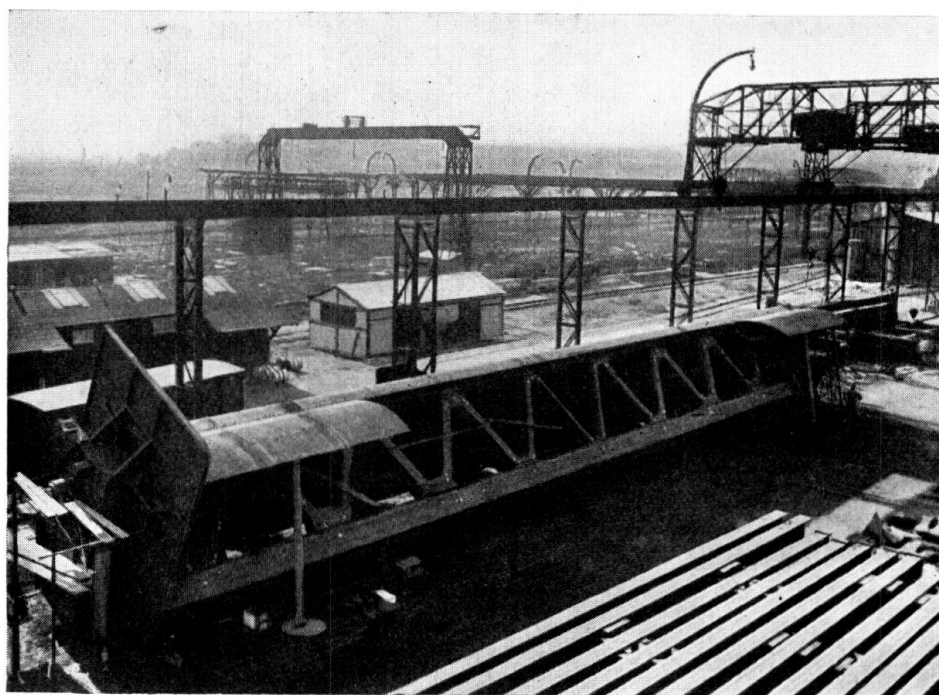


Fig. 11.

town — here so important — and on account of its dimensions. The three rollers at Heidelberg, for a retained head of only 4.10 m, have a clear width of 40.0 m and can be dropped through a depth of 0.60 m.

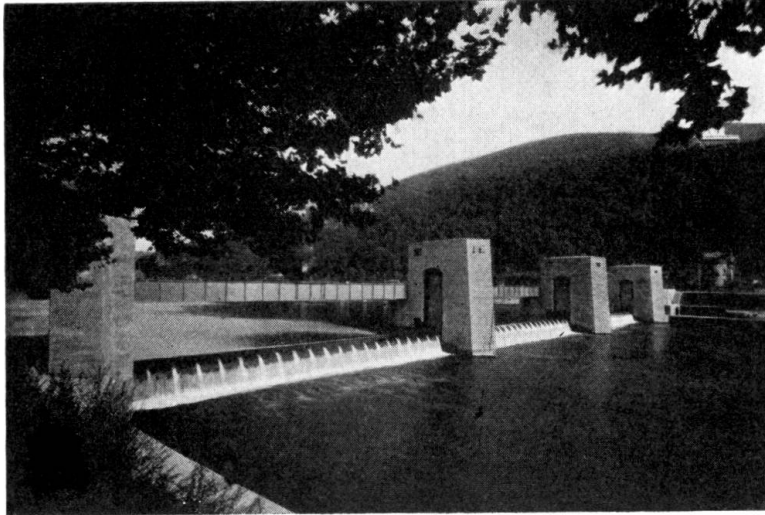


Fig. 12.

Fig. 13 shows an example of a roller weir in course of construction, intended for Solbergfoos in Norway; one of the three rollers is shown in course of being

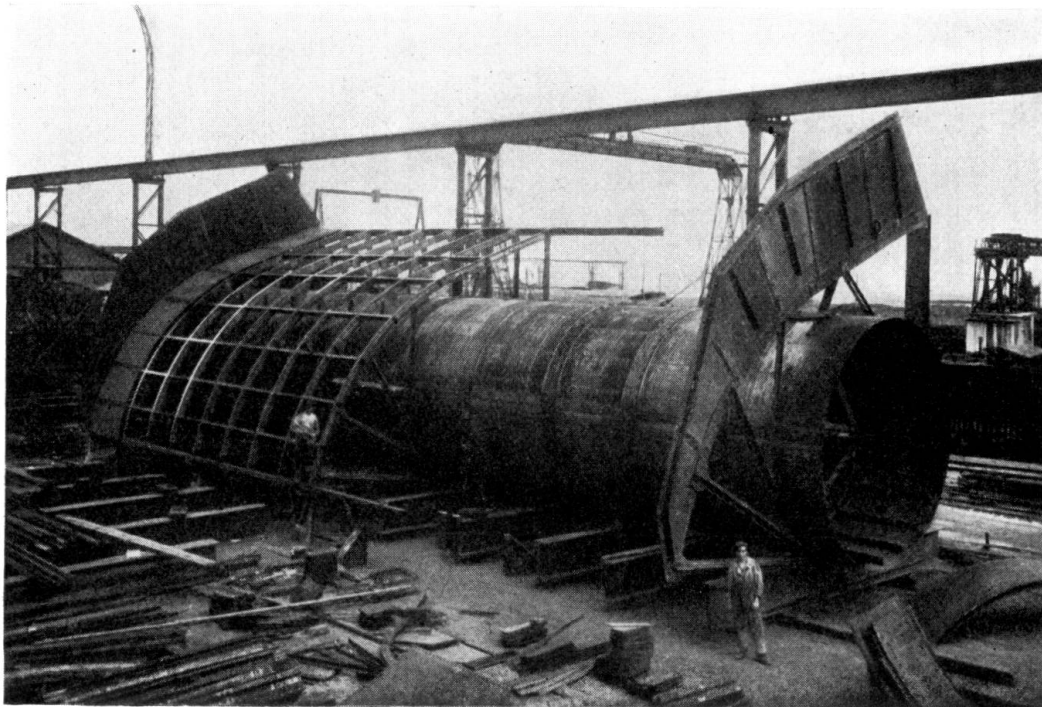


Fig. 13.

erected in the workshop. This method is economical as in the case of all roller weirs where large heads of water are to be retained and where the widths are relatively

small. For instance the installation at Solbergfoos will have three rollers each of 20.0 m clear width with 8.75 m retained depth (Fig. 14).

The Ryburg-Schwörstadt installation has already been cited as an example of large weirs with double shutters or "hook gates". The M.A.N. double gate has

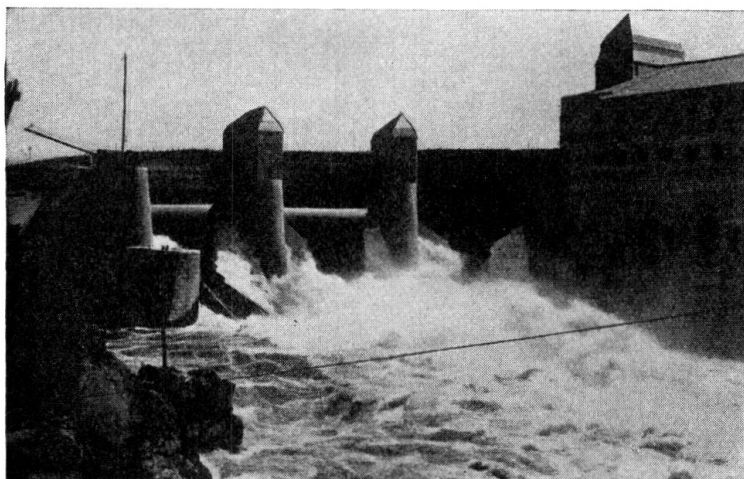


Fig. 14.

also been adopted in the large weir installation at Donau-Kachlet, which has six such gates of 25 m clear width and 11.5 m height, this being one of the largest installations anywhere in the world. Finally Fig. 15 shows the installation at

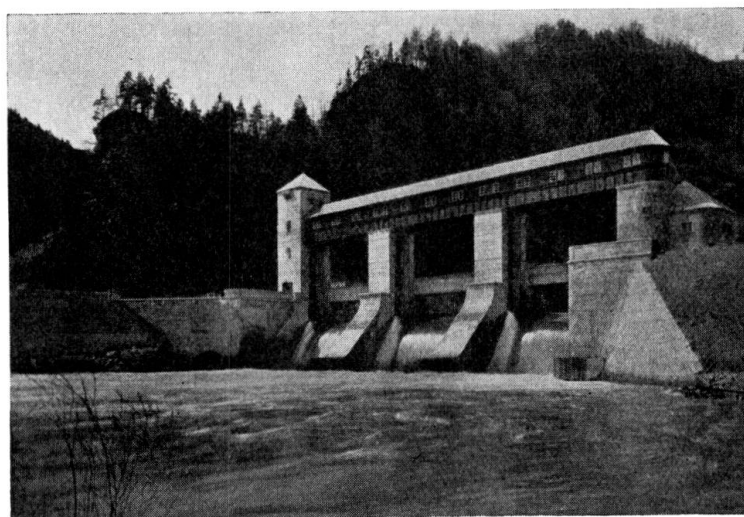


Fig. 15.

Pernegg in the Mur district, which again is a good example of the way that modern barrages are made to blend with the landscape. This includes three double openings measuring 15.0 m by 11.60 m. The extent that the double gates can be dropped amounts, as a rule, to one quarter of the total height, but the "hook gates" can be sunk as much as one third of the total height.

VIIb 2

The Use of Steel in Hydraulic Engineering, General Remarks and Details.

Anwendung des Stahles im Wasserbau, Allgemeines und Einzelheiten.

Application de l'acier dans la construction hydraulique, généralités et détails.

Prof. Dr. K. Dantscher,
Oberbaudirektor der Rhein-Main-Donau A.G., München.

In the two German reports by Messrs. *Agatz* and *Burkowitz* respectively reference is made to the use of steel in hydraulic engineering under the two headings of fixed and movable structures, the properties of this material being treated with special regard to the requirements of hydraulic work and its performance in such construction being dealt with at some length. Contributions to the discussion will serve to amplify these reports by describing the developments of steel construction in hydraulic engineering and by giving relevant examples from German practice.

The materials in use for hydraulic engineering over 100 years ago were stone of all kinds, concrete, fascines, certain kinds of earth and timber. The last mentioned has been applied mainly to the movable parts of hydraulic works such as sluice gates, weirs and piled and other foundations. There was an almost complete absence of iron and steel work in hydraulic engineering, this material being used only in conjunction with timber construction in the form of screws, nails and binding pieces. At a later stage large cast or wrought iron pieces came into use as supports or load bearing members, and these conditions generally held good until the end of the century. Only now that steel is produced in large quantities, and is available in rolled sections of the greatest variety of shapes and sizes, is the picture changing. Steel in this form is making remarkable progress as a material for hydraulic work and is almost completely replacing timber, even to some extent replacing masonry, a change attended by many new methods of work and forms of construction. A brief description of this development will be given in reference to hydraulic engineering classified under the following heads: —

- 1) Works serving for purposes of navigation.
- 2) Weirs.
- 3) Works connected with the exploitation of hydraulic power.
- 4) Foundations.

I. Works Connected with Navigation.

In reference to navigation the first point to be noticed is that the vessel itself has been in process of change from timber to iron or steel for as long as fifty years. In marine work this transition is by now almost completed and in inland navigation, in Germany, it has run the greater part of its course. As a material for ship construction iron or steel offered greater stability and strength and led to an increase in dimensions, such as would have been quite unthinkable using the earlier material, namely timber. A corresponding development has taken place in all forms of construction appertaining to navigation, and an example may be found in one of the basic elements of navigational engineering — namely, the lock. Until about fifty years ago lock gates continued nearly always to be made of timber, except for occasional iron parts of the frames. A clear width of six to eight metres was normal, and heads of this dimension were regarded as large. The introduction of 600 ton vessels called for a clear width of 10 m in the locks, and that of 1500 ton vessels a width of 12 m, while in the case of marine locks the requirements gradually increased up to 40 or 50 m. It was not feasible to construct locks of these dimensions in timber, and nothing but steel would solve the problem; hence the transition from the timber to the iron gate in inland navigation. This particular type of lock gate has been in use for centuries, and its design has developed so effectively from long experience in handicraft, informed by so perfect an appreciation of the forces arising, that up to to-day the ironwork has been unable to do better than imitate the old timber gate with its hinge and jamb uprights, its transoms, back struts and tie.

The introduction of iron work brought with it elaborate methods of statical analysis, and there followed a period in which attempts were made to free the design of such gates from statical indeterminacy, to eliminate bending moment from them by introducing curvature, and so on; later, however, a reversion took place to the typical form of lock gate. In the meantime, the dimensions have grown considerably greater and a lock gate of 12 m clear width is now the normal type. The gates for the lock at Kachlet on the Danube are of 24 m clear width, corresponding to the dimensions of the first North East Sea Canal. Even with these large dimensions the stiffness so necessary in lock gates is easily attained through the adoption of a strong and well riveted skin, if necessary of double thickness.

The introduction of iron and steel work has, however, also led to other developments such as entirely new forms of gates. Examples of gates on horizontal hinges and of sliding gates may, indeed, have occurred at earlier times where the conditions were easy, but it would be true to say that these types of gate have become really practicable only with the introduction of steel. This applies to the lifting and the segmental types of gates of which the second contributor to the discussion, *Dr. Becher*, has given examples. The sliding gates adopted in recent large locks which afford entrance to inland harbours would be altogether impossible without steel construction, and for the development of these large harbours, as at Bremerhaven, Antwerp, Amsterdam, or in the German North East Sea Canal, large locks of this kind are essential. There is no need here to elaborate the statement that all the operating arrangements increase in size with

the gates and that all such equipment is made of steel; one point appertaining to steel construction must, however, be noticed — it has given rise to yet another method of filling locks. In the small locks of primitive design the filling was effected through a small opening in the timber gate; later the practice grew up of providing diversion channels in the side walls. The considerably greater stiffness of a steel gate makes it possible, however, to arrange a large opening in the gate itself, and the result is that in, for example, the Neckar canalisation the filling of the lock takes place through openings in the steel gate, which can be closed by means of segments. Either the lifting or the segmental type of gate, which can be constructed only in steel, enable the lock to be filled and emptied without the need for side channels, as they can be operated against the pressure of the water. Where large heads have to be overcome the shaft form of lock, as favoured in Germany, would not be feasible in large dimensions were it not for the availability of steel lock gates.

In the case of ship lifts the inclined plane is a form of construction in which steel is not necessary to any great extent in the smaller dimensions, but other types of lift involve very notable amounts of steel construction, and this is a field in which German engineering has made great progress, as exemplified in the ship lifts at Henrichenburg and Niederfinow.

So far reference has been confined to the substitution of steel for timber construction, but it should also be mentioned that in the field of navigation steelwork is beginning to compete with concrete, for building locks, quays and canal walls. The steel sheet piles exhaustively treated in Prof. *Agatz's* paper have been found particularly suitable for this purpose, and in Germany a number of locks have been constructed entirely in the form of sheet piling, the largest examples being those at Griesheim and Eddersheim on Main, which are 350 m long by 14 m wide. In these works the steel piling takes the place of ordinary walls to the locks, and where the ground is such that piles can readily be driven the advantages of this type of construction are evident. Numerous examples of this application exist along the edges of harbours, and there is evidence also that the system of using sheet piles with anchorage in the ground behind is one which gives good results. A noteworthy example of the adaptability of steel piling is provided by the widening of the Dortmund-Ems canal to accommodate a larger type of vessel; here long sections of the widening work were carried out by first driving sheet piles in the solid ground on either bank, and then excavating the material in front of them, a method which allows the cross-section of the canal to be enlarged by a very simple procedure without interrupting traffic. With good pile driving the watertightness of canal walls formed in this way is very great.

II. Weirs.

The weirs needed for various purposes in hydraulic engineering were at first mainly of the fixed type and in many cases the body of the weir itself was built of timber, openings for the removal of detritus and ice being always protected by small timber screens. The canalisation of rivers, which began to be undertaken about a hundred years ago, called for a type of weir capable of movement across the whole of the stream, and to meet this need there were developed needle weirs,

the hinged barrage and the drum barrage. In these forms the needles or shutters were made of timber and were of small dimensions, only the supporting portions of iron, usually either cast or wrought. Poirée's type of service bridge as adopted in such weirs provide a notable example of the limitations of iron construction at that time. Improvements in the art of forging led to somewhat better solutions as, for instance, in the needle weir. The reason why Chanoine's shutter weir did not prove a success at the time lay partly in these constructional limitations of iron; the revival of this type by Pasqueau in the eighties was the result of improved possibilities in iron construction, and of making supports of cast steel. The favour it is experiencing at the present time is due entirely to the improved lifting gear made possible by the use of steel.

Formerly the commonest type of sluice in hydraulic work was the gate weir which nearly always was used in conjunction with a fixed weir. For instance, the weirs used for hydraulic power stations and on rivers with heavy gradients were almost invariably fixed weirs, as the gate type could be built only in limited dimensions. This form of construction gradually increased in size until finally the movable part became so large that in the latest developments the fixed portions has entirely disappeared and movable weirs are being adopted even in power plants, a development made possible by the use of steel to construct large and reliable opening gear. In regard to regulating the rise of a flood and diverting boulders and ice this constitutes a great advance.

With the normal forms of gate construction using timber it is not possible to exceed a clear width of 8 m. Greater widths than this were, however, always being demanded, as for instance on the Main at Schweinfurt where a clear opening of 30 m was required in the weir on account of the heaviness of the ice formation. This particular problem led to the leading Bavarian engineer, *Eickemeyer*, proposing the use of a steel cylinder in view of its large resisting moment, and from this suggestion *Carstanjen*, the managing director of the Gustavsburg bridge construction works near Mainz, was enabled to develop the roller type of weir which is feasible only in steel construction, and which makes it possible to build a movable weir with a clear width quite impossible with other forms of gate, while at the same time it is simple and strong and is, therefore, exactly what is required in hydraulic engineering. In weirs for the canalisation of rivers the roller type was soon adopted, and its most notable examples are in this field. In canalisation work ice, flotsam and dirt must be capable of release from storage without lowering the water level, and from this point of view the old forms of weir were at a disadvantage. By the development of the submersion roller, and of the roller with additional shutters, the roller type of weir was enabled to answer these requirements also. As already mentioned the earlier types of movable weir could not be made to exceed 6—8 m, and the large widths required to cope with rising floods were obtainable only by the use of movable uprights; this clumsy form of construction, however, soon had to be superseded, and by adopting steel as a material, it was found possible to enlarge the gate itself to a greater clear width, resulting in the design where a screen of steel plates is used to transfer the pressure of the water onto a horizontal framing. This form of construction enabled considerable progress as regards width, but the forces required for lifting the weir became very heavy. A solution to this problem was

provided by the Stoney sluice in which the raising and lowering motion takes the form of rolling in addition to sliding; here the roller carriages necessitated a new form of staunching, which was supplied in the Stoney sluice by a staunching rod, and in the steel sluice developed by the Maschinenfabrik Augsburg-Nürnberg (M.A.N.) by an elastic packing plate.

In Germany, particularly in the South German water power plants, further development has been given to the gate type of weir, and the M.A.N. firm have introduced the double sluice and the hook sluice which allow the upper portions of the construction to be lowered; the Dortmunder Union have produced the three-boom gate and the gate with shutters. Clear spans of 40 m and more have already been obtained, and were it not for the problem of coping with the vibrations that are encountered in still larger widths, weirs of more than 50 m clear opening would now be in existence.

Those forms of sluice which are operated by the water pressure, known as automatic weirs, have derived great advantages from the introduction of steel as a material of construction. The old bear trap barrage introduced by the American, White, a hundred years ago has been revised as the "roofed weir" using many different kinds of gate constructed in steel, providing for the automatic regulation of the water level in reservoirs and at the spillways of dams, and, when made to the fish-bellied type of design, enabling great widths to be spanned. The sector weir has been applied in Germany on the Weser by the M.A.N. for a sluice of 54 m clear opening, with 4.5 m retained height, which is the largest sluice actually in existence. The drum barrage invented by Desfontaines and originally constructed only in small sizes has been built as large as 12 m by the use of steel, and has provided an exceptionally useful form of weir for use on the Main canalisation work. Finally the segmental weir should be mentioned, a type which is possible only in steel, and which have been carried out in Germany up to 30 m clear opening.

Thus iron and steel have entirely taken the place of timber in the field of weir construction also, enabling much larger widths and heights than were hitherto feasible, and leading to the developments of quite new forms of sluice.

III. Power Plants.

The part played by steel in the construction of weirs for hydraulic power plants has already been discussed but a few words will now be added on the changes that have arisen in the power plants themselves. Not long ago the water wheel, the head race and the tail race and the whole of the substructure of the power house were constructed of timber. Turbines, possible only in steel construction, were introduced in the eighties of the last century. The more recent practice has been to build the power house of concrete, and, with the high heads now in use, the water is fed to the turbines through pipes. These pipes, on suitable pedestals, are an important part of the high pressure installation and provide a field of use for steel in which it is practically without competitors. It is true that pressure pipes have been constructed in timber and in reinforced concrete, but only for limited heads and discharges, and in all the larger water power plants built to-day the pipe line is of steel. The constructional requirements of pressure piping

for water power plant are by no means simple, for the regulation of the turbines entails continual variations of pressure in the pipe and a sudden closing down is liable to cause very notable increases in pressure and, therefore, in the stresses present in the pipe walls. In addition there are temperature effects which become very important in large pipes, and the reactions imposed by the supports. Cast iron pipes are found here and there in small installations, but in modern practice their place is invariably being taken by steel. In the Mannesmann seamless rolled pipe the German metallurgical industry has produced a constructional element almost without rival for high pressures, but if the discharges to be handled are great it becomes necessary to use piping of larger diameter built up from rolled steel plates. The usual method of forming the longitudinal and transverse joints continues to be riveting, but welding is being applied to an increasing extent.

For pipelines in power plants such as the Walchensee and Schluchsee, and also in pumping installations where the pipes have to operate under pressure, nothing but steel comes into question, and in such pipe lines an important part is played by the various forms of valves such as butterfly valves, high pressure slide valves, etc. (see the report by *Burkowitz*). Yet another new development has resulted from the requirements of hydraulic power practice. In works of this kind attention must always be paid to the maintenance of the water level, since if this is lost the wastage of energy becomes excessive. In order to be able to keep the sluices in order and to carry out necessary repairs, forms of auxiliary cofferdams have been developed, especially in the case of power plants, and these represent notable types of construction. The old timber stop planks not being suitable for large widths were replaced by iron beams; the procedure for laying these in position and for transporting, inserting, removing and maintaining them exerts an important influence on the design of a modern power plant, but even so these iron stop planks are not suitable for unlimited widths as they become too heavy to handle. Hence for use at large weirs, as in canalisation of rivers, forms of temporary cofferdam have been developed which can be put together on the site under water. There is, for instance, the type introduced by *Schön* which is constructed by the firm of *Noell* of Würzburg, in which the watertight wall is carried on iron supports consisting either of slabs or of Larssen sheet piles which are arranged like the needles of a needle weir.

Mention should also be made of floating cofferdams, which are hollow iron vessels which can be floated into position and sunk. At the Kachlet weir these have been built for 24 m clear width.

IV. Foundations.

Dr. Agatz has given a very full account of the use of steel for the purpose of foundations. This takes two forms, since either steel may be used as sheet piling to form a cofferdam around the excavation, or it may be used as a permanent part of the construction of the foundation to prevent scour and to resist pressure.

Steel has long been used in foundation work in the form of diving bells and caissons, and in special cases steel pipes have been used as piles. Since *Larssen*, in co-operation with the Dortmunder Union, brought out the first sheet piling section, the whole conceptions of foundations in hydraulic construction has

altered. Foundations built within an open excavation have always been the desideratum of engineers. The excavation was enclosed in a wooden cofferdam and was pumped dry, but a depth in excess of 5 to 6 m great difficulties were encountered. In the steel cofferdams engineers have a method of reaching more deeply into the ground and of obtaining a more watertight wall, and in addition such a wall was found to be considerably stiffer than a wooden cofferdam. Whereas, about thirty years ago, it was necessary to contemplate compressed air foundations as soon as depths of 6 to 7 m were reached, it is now possible to obtain depths of 12 to 14 m by open excavation within a steel cofferdam, and depths of 20 m have already been reached in this way. For excavations of large size and great depth as, for instance, those required for founding piers in large rivers, it is no longer customary when using this method to enclose the whole area of the site but only to enclose small sections in which adequate stiffening can be provided, after the manner of shafts, and in this way considerable depths may be reached. The use of closing piles and corner piles enables a high degree of watertightness to be obtained.

In weirs and dams it is always the case that percolations of water may tend to occur underneath the structure which, in the course of time, may impair the subsoil. It is necessary, therefore, that an impermeable stratum should be reached, either by the foundation itself or by apron walls, and in many weirs this has involved the use of pneumatic methods of foundation at great expense in time and money. In this respect the introduction of the steel cofferdam has been of great assistance, since in most cases it enables the impermeable subsoil to be reached even at considerable depth, and with good driving of the piles such a cofferdam is more watertight than a concrete wall sunk by pneumatic methods. The introduction of the steel sheet pile in foundation work has therefore made it possible to hold up dangerous precolations of water underneath the foundations of weirs much more easily, and the weirs can be founded more effectively than was previously possible.

Dr. Agatz has also referred to the use of rolled sections as bearing and retaining piles. Here we are at the beginning of a development, for this is an application in which timber still remains supreme; the reinforced concrete pile has not yet been able to displace it, and it remains to be seen how far the steel pile will succeed in doing so.

For the construction of high dams it has become a favourite method during the last ten years to make use of an earth dam, which is rendered watertight by the provision of a reinforced concrete wall at the core. Lately steel sheet piling has been adopted as a means of forming this impermeable wall, the core being enclosed between two lines of such piling. The stresses liable to be imposed upon this core during the placing and settlement of the earth dam may be very large, and if the risk of cracking is to be avoided can properly be carried only by a material which has the elastic properties of steel.

V. The Durability of Steelwork as applied to Hydraulic Engineering.

The transition to the use of steel in hydraulic work raises the important question of the length of life to be expected from this form of construction.

This matter has been dealt with in the paper by *Agatz*, who concludes that experience is as yet too short for any definite pronouncement to be made. It is scarcely to be expected that steel structures under water will have as long a life as the Roman aqueducts or of timber piles which are permanently under water, but the life that may reasonably be attributed to them should be about equal to that of concrete structures, and certainly equal to that of reinforced concrete. The measures necessary to ensure durability of steel when applied to hydraulic engineering are stated at length in the German report. The question of painting is one to which considerable attention has been paid in German hydraulic practice, and one on which no definite conclusion has as yet been drawn.

VIIb 3

Welding in Hydraulic Engineering.

Schweißkonstruktionen im Stahlwasserbau.

La soudure dans les travaux hydrauliques.

G. Wittenhagen,

Oberingenieur der Dortmunder Union Brückenbau A.G., Dortmund.

Hitherto welding has found relatively little application in hydraulic engineering by contrast with building work and bridges, but it may safely be said that in this special field, also, the engineer is making increasing endeavours to utilise the obvious constructional, technical and economic advantages which welding offers.

In steelwork applied to hydraulic construction there are, for instance, emergency closures, stop beams, lock gates, etc. which are mainly exposed to stresses of a purely statical character, and in this kind of work, as in building, welding is very suitable and is also economical.

In movable weirs, however, dynamic forces play a predominant part, and the phenomena which arise are even more difficult to follow than those occurring in bridges which, likewise, are exposed mainly to dynamical stresses. The dynamic forces in weirs are difficult to calculate and have not yet been fully cleared up from a scientific point of view. The barrage must be capable of being operated against the full water pressure according to requirements, and the water may flow either under or over. The whirling and rolling which attends the movement of the water, and the variations in pneumatic pressure, promote vibration which may under some conditions be dangerous. So long as the cause of these vibrations and the measures suitable for overcoming them are not fully understood it would appear advisable to make the body of the barrage as rigid and as heavy as possible.

It is true that the leading constructors of weirs, in close co-operation with universities and laboratories, have for some years been carrying out exhaustive experiments and scientific researches with a view to determining the origin of these vibrations and the means whereby they can be avoided. The results already obtained are very satisfactory. They show that by suitable design and shape and other measures it is indeed possible to avoid at least those vibrations which are likely to endanger the structure, but no complete clarification of this important problem has hitherto been obtained. The fact remains that despite all care in design and construction vibrations arise in operation, the effect of which has however, been reduced sufficiently to remove any risk of damage to the structure. In order to render these vibrations which are not yet theoretically understood as harmless as possible, the designer is compelled for the present to aim at combining a high degree of rigidity with the maximum possible inertia. Hence that

reduction in dead weight which is attainable through the use of welding is not a possibility of which the engineer is at present enabled to take much advantage. For the same reason the present tendency is to avoid the adoption of high tensile structural steel in weirs, however desirable it may otherwise be on economic grounds.

For the present, then, there is no intention of constructing weirs entirely by welding, but there remain at least a few detailed members which may advantageously be constructed by this means. For instance in most of the three-boom barrages constructed in the last few years by the Dortmunder Union the end wall, booms and shutters have been welded, the remainder riveted.

The end wall if built to a welded design can easily be adapted to the cross section of the weir, and the resulting saving in weight has no effect on vibration as the wall is supported on piers.

The welded construction of the booms and gusset plates simplifies the design and gives convenient connection and simple arrangement of the staunching timbers.

For large widths the shutters must be made rigid against torsion. With welded construction the joints in the pipe provided to take up the torsional forces can easily be made and this can be connected to the remaining portions of the structure so as to be stiff against torsion. The curved form of plate covering which is necessary in order to avoid vibration can be made cheaper by welding than by riveting and in a welded plate the presence of projecting rivet heads is avoided; these would be exposed to a risk of abrasion by sand.

Apart from these dynamically stressed weir structures, welding is, as mentioned above, employed with special advantage in hydraulic constructions subject mainly to static stresses.

The simplest static conditions are those which arise in stop beams, and the Dortmunder Union have adopted welded construction for the stop beams in the machine house of the Albbruck-Dogern installation. These "beams" consist mainly of nosed plates with web plates. The saving in weight by comparison with riveted work was found to be about 14 % and this reduction has a very favourable effect on the design of the lifting gear.

A further example of the advantageous use of welding occurs in the lifting gate for the Niegripp lock, recently constructed by the Dortmunder Union. The main girders and the end posts are welded plate web girders built up from nosed plates and the side joints in the skin plate are riveted. The saving in weight achieved in this way by comparison with all-riveted construction amounts to about 11 %, and the counterweights are, of course, lightened by a corresponding amount, thereby imposing smaller loads on the winches and on the operating frame.

A new and particularly interesting application of welding in steel hydraulic work is provided in the floats, 10 m in diameter by 35 m high, now under construction for the ship lifting gear at Rothensee forming part of the Elbe connection on the Mittelland canal. The first of these floats is now being erected. Here again the saving in weight is considerable, amounting to abt. 10 to 12 %.

The results obtained by the application of welding to steel structures used in hydraulic engineering may not hitherto have been as conspicuous as those in bridge and building work but they are nevertheless notable, for the difficulties with which this branch of engineering has to contend are particularly great.

VIIb 4

Steel Dams.

Stahldamm.

Barrages d'acier.

Prof. G. Krivochéine,

Ing., General-Major, Prag.

A field of application which offers great scope for the use of steel as a constructional material is that of dams. It is known that in North America there are thousands of dams constructed in masonry, concrete and reinforced concrete, but only four or five steel dams; but whereas dams built of stone, concrete or reinforced concrete are not completely watertight, steel dams can be made completely so.

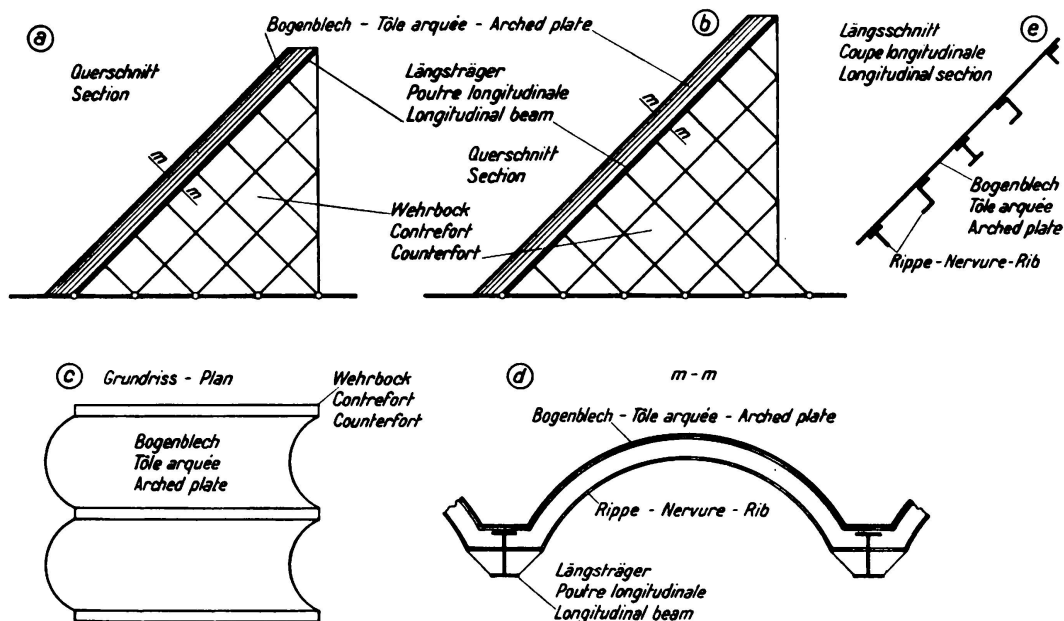


Fig. 1.

The system of dam construction patented¹ by Eng. *Fultner*, Dr. *Sekla* and Professor *Krivochéine* (Figs. 1a, b, c, d) consists of a series of curved and inclined steel plates of wide span (8 to 15 m) strengthened by ribs. The arched steel plates formed in this way are carried on inclined longitudinal girders without any cross girders, and supported directly on counterforts of open frame

¹ Amer. patent, U.S.A. N° 2, 033.027.

construction. The latter are built in an original shape, being made triangular without diagonals; a form of structure which possesses theoretical hinges at the intersections, and which is both rigid and statically determinate.

An arched form of dam in accordance with this invention is completely elastic, and allows expansion to take place under variations in temperatures although no gaps are left. Another very important advantage, from a technical point of view, is the reduction in width of the base.

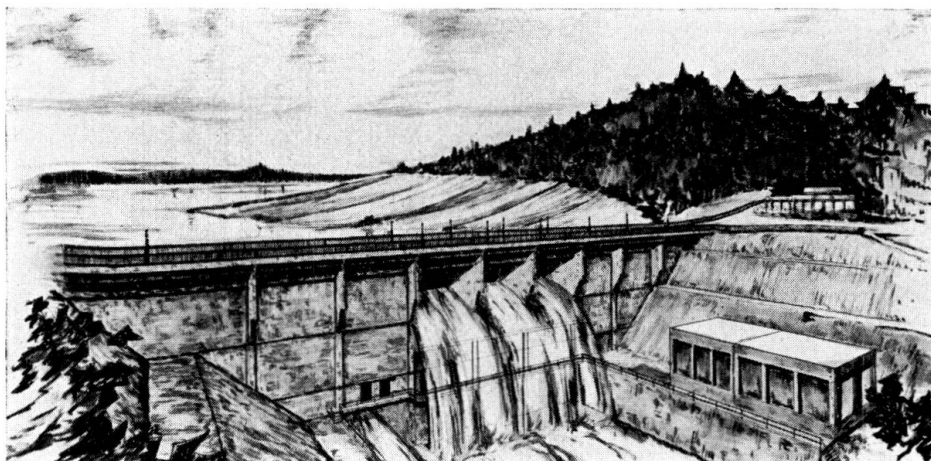


Fig. 2.

Design of steel multiple-arch dam on the Svatka River at Kniničky, Czechoslovakia.

Patented by J. Fultner, Prof. G. Krivochéine and Dr. J. Sekla.

In a competition in Czechoslovakia the author had an opportunity of proposing a steel dam of this kind for Kninicky in the neighbourhood of Brno (Fig. 2)² and the tender put forward by the Witkowitz works, according to the patent mentioned, worked out same 22 to 53 % lower than the respective fifteen tenders for masonry or concrete dams.

² The steel dam at Brno was covered with Monier reinforced concrete slabs.

VIIb 5

The Thickness and Rusting of Steel Sheet Piles.

Wandstärke und Abrostung bei stählernen
Spundwänden.

L'épaisseur et l'oxydation des palplanches métalliques.

Dipl.-Ing. W. Pellny,
Hamburg.

The wide usage and numerous applications to which steel sheet piling is put has led to the development, in the course of a few years, of a special science in their use which may now be regarded as an important branch of hydraulic engineering. The variety of technical knowledge has become so great that it is no longer possible for the engineer to follow developments in all branches and to be aware of the latest position in technology, or even to know what is being treated in the literature. For this reason alone, quite apart from the importance of such knowledge in public economy, everything technical ought to be expressed in a clear and simple form so as to relieve the engineer of the need for unnecessary and time-wasting cogitation.

It was, therefore, a matter for congratulation when the four leading German rolling mills agreed to adopt the same series of rolled sections for steel sheet piling. To-day the corresponding sheet piles are of the same cross section wherever produced; they have the same resisting moments and are of the same weight and quality. Likewise the freight on them is the same.

There is, however, one point which continues to cause the conscientious engineer a good deal of worry, and that is the question of the thickness of the piles in relation to rusting. There is no doubt, of course, that in the greater number of cases where engineering works have had to be renewed up to the present it has been for technical operating reasons — because the arrangement was out of date, the capacity had become insufficient, or the operating costs were too high — and seldom because the structure of the works had, in fact, become dangerous. It has always been sought to eliminate this last contingency, or at least to delay it as long as possible, and for this reason it is easy to understand why the thicker sections of piles should have been preferred.

There are many who profess to see a difference in this respect between the different sections, basing their arguments on the listed information of the producers who specify, for instance, 13 mm thickness in one case and 11 mm in another. It would, however, be a serious mistake to assume that on this account alone the first mentioned sheet pile would withstand heavy corrosion better than

the second. This should be pointed out clearly once and for all, because there are many engineers who, having to deal with sheet piling and reasoning very naturally in this way, have apprehended difficulties which were in fact non-existent.

When reference is made to the thickness of the pile it is apt to be forgotten that the flanges of these Z sections are made up of not only the listed thickness but also a large additional cross section at the interlock. If the latter were to be recalculated as redistributed uniformly over the whole width of the flange it would correspond to a strip approximately 5 mm thick, and if similar calculations were to be made for the corresponding sections of different types it would be found that this calculated or *virtual thickness is approximately the same in the corresponding sections produced by the different mills*. In order to simplify the matter it is sufficiently accurate to regard the resisting moment as being equal to the area of the flange multiplied by its distance from the neutral axis.

However, the criteria for examining or evaluating a steel sheet pile wall at the end of a life of 50 or 100 years is not the original thickness and the amount of rusting, but the *residual resisting moment* at the end of that time.

A rusting of 1 mm means the same reduction in resisting moment in *any* of the corresponding rolled sections, those rollings which are of small constructional depth being somewhat more favourably situated in this respect. The heaviest amount of rusting which is at all possible corresponds to the thinnest place in the web and if the rusting should become deeper still the sheet pile must fail at the interlock.

Fortunately the maximum bending stress which sheet piling is designed to withstand occurs in most cases at a point which lies considerably deeper than the part exposed to the heaviest rusting, so that it is possible for extensive weakening through rust to take place before the piling is in fact endangered.

Careful measurements have shown that under ordinary conditions in Europe the amount of rusting that is to be expected in the course of 100 years may be about 2 mm at the places most affected. (Compare Professor *Agatz* in the Preliminary Report.)

Knowing the smallest resisting moment that must be retained at the heavily rusted place in order that the prescribed degree of safety may be counted upon, the life of the sheet piling can very quickly be calculated, or the proper thickness of piling to give a desired length of life can be determined, thus fixing the section to be adopted. But even if the rate of rusting were in fact assumed too high this would be to the advantage of the steel.

In the course of years a consolidation of the ground occurs in the backfill, with the result that the *true* earth pressure falls off and becomes distinctly smaller than the *calculated* earth pressure which was taken as a basis for the design. This affords no justification for any great reduction in the additional margin of safety to be demanded in an old structure, in view of special circumstances, but it does mean that one need be a little less apprehensive — always provided that the load bearing members are accessible to inspection and that the material itself does not alter in its properties. In the case of steel both these conditions are usually fulfilled.

As regards the rusting itself, a clear distinction should be drawn between apparent and true rusting. Very often the flaking which is wrongly described as rust is merely thick layers of dross or other suspensions which the rusty water causes to adhere to the sheet pile, and if their thickness is recalculated as metallic iron it amounts to only a fraction of a millimetre. If, therefore, it is desired to observe the rusting on an actual job, it is essential to measure both the original and the remaining thickness.

VIIb 6

Steel Pipes of Large Diameter Subject to Heavy Internal Pressure.

Stahlrohre für Druckleitungen mit großem Durchmesser
und hohem Innendruck.

Tuyaux en acier pour conduites forcées de grand
diamètre, sous de hautes pressions intérieures.

Dr. Ing. h. c. M. Roš,

Professeur à l'Ecole Polytechnique Fédérale et Président de la Direction du Laboratoire fédéral
d'essai des matériaux et Institut de recherches — Industrie, Génie civil, Arts et Métiers — Zurich.

It has been found possible to obtain valuable information as to the distribution of stresses and strains, and to arrive at the factors of safety enumerated below, by means of tests carried out in the Swiss Federal Laboratory for Testing Materials during the years 1930 to 1935. The tests were carried through until the pipes burst under the effect of the internal pressure and they were correlated with extensive tests and investigations of actual pipe lines as well as of the materials employed. Diameter $D = 1.8 - 4.6$ m; pressure head H up to 1750 m; coefficient of capacity $H \cdot D^2 = 1500 - 3000$:

Type	Factors of safety as calculated		
	for static failure	for yield	fatigue rupture
1. Welded pipes with normal or helical welds, "Sulzer Winterthur"			
Normal quality steel, tensile strength $\beta_z = 38-42 \text{ kg/mm}^2$	3.5	2.4	1.6
High quality steel, tensile strength $\beta_z = 42-48 \text{ kg/mm}^2$	3.5	2.4	1.4
2. Hot bound pipes, "Ferrum" Katowice			
Tensile strengths $\beta_z \cong 38 \text{ kg/mm}^2$ in pipes $\beta_z \cong 60 \text{ kg/mm}^2$ in binding	3.4	2.3	—
3. Cold drawn, previously bound pipes, "Autofrettage G. Ferrand" by Bouchayer et Viallet, Grenoble			
Tensile strengths $\beta_z \cong 38 \text{ kg/mm}^2$ in pipes $\beta_z \cong 94 \text{ kg/mm}^2$ in binding	3.9	2.0	—
4. Pipes bound with steel wire from a reel, "Monteux", Paris			
Tensile strengths $\beta_z \cong 42 \text{ kg/mm}^2$ in pipes $\beta_z \cong 197 \text{ kg/mm}^2$ in binding	4.5	2.0	—

The four types which were subjected to close investigation are found to compete with one another technically and economically, and the helical welding is seen to make the competition keener. Each type offers technical and economic advantages of its own — whether with respect to weight, freedom from rusting, maintenance or safety — which require to be established by careful comparative experiments in each particular case.

As regards complicated shapes such as branches, junctions, inlets to turbines, large manholes, and sockets an accurate estimate of the true conditions of stress and safety can be obtained merely by measuring the stresses and strains under a multi-axial system of loading. Further valuable information can be obtained from a study of crack formation and lines of flow in specially applied coatings of lacquer. Fatigue tests and photo-elastic experiments offer means of estimating reductions in concentrations of stress.

The system of stresses and strains corresponding to the operating pressure must nowhere reach the yield value. Nothing but a combined application of all these methods of testing, in a perfected form, will allow the effective degree of safety of pressure pipe lines to be correctly assessed.

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