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Autor: Pascal, R.

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V 2

Semi-Experimental Method of Designing a Typical Structure.

Halb-experimentelle Berechnungsmethode eines grundlegenden Bauwerktypus.

Méthode de calcul semi-expérimentale d'un ouvrage classique.

R. Pascal,
Ingénieur conseil, Paris.

It is proposed to describe here a semi-experimental method of design which has been applied to two steel arch bridges, of 67 and 82 m span respectively, crossing the Seine between Neuilly and Courbevoie. These bridges are 35 m wide and are separated by a concrete arch of 32 m span between two massive abutments. The width of the roadway is 35 m but that of the central arch with its abutments is approximately 70 m. The work, occupying the site of the

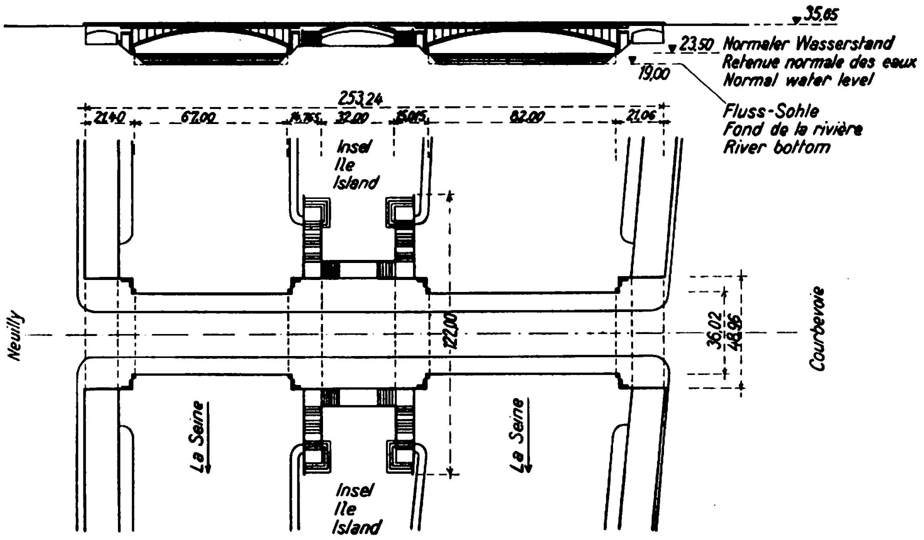


Fig. 1.

famous bridge built by Perronet in the reign of Louis XV, is at present under construction.

Fig. 1 is a key plan showing the general layout. The two steel bridges are of the same type. Each of the twelve arch ribs is hinged at its two ends, and between the springing and the quarter points nearest to the crown is rigidly connected to a cross frame composed of longitudinal members and verticals, the moment of inertia of these being much smaller than that of the arch. The frames so formed are connected by bracings below and cross girders above, so constituting an arcade in which the vertical members are also those of the main frames. In the central portion, adjoining to the crown, only the bracing members are provided. The sections of the arch ribs, verticals and bracings are hollow rectangular suitably stiffened. The cross members and longitudinals are rolled steel joists T. The work is to be carried out in steel 54, and all the connections are being made by welding. The bearings are of cast steel and the roadway is formed of reinforced concrete slabs.

The contractors for the masonry and foundations are the firm of *Leon Ballot*, and for the steelwork *Baudet, Donon & Roussel*. The work is being done for the Département de la Seine under the principal supervision of M. *Levaillant*, Ingénieur en chef des Ponts et Chaussées, and under the immediate charge of M. *Louis Alexandre Levy*, Ingénieur des Ponts et Chaussées, assisted by M. *Kienert*, Ingénieur T.P.E.[1]

Methods of calculation.

Each structure is a complex whole due to the main girders and cross bracings being inter-connected. The procedure followed in calculating the main girders will be explained first, and then the design of the cross bracings.

1) *Design of Main Girders.*

The type of structure under consideration, dictated as it is by the limited constructional depth available at the crown, is one that gives a pleasing effect to which the eye is well accustomed. The point to be noticed here is that in the central portion of the arch the moment of inertia increases outward from the crown as far as the first of the cross frames, which may be regarded as rigidly fixed, by the agency of its vertical member, to the solid central portion of the arch.

Structures similar to this have frequently been designed on the assumption that the frames were pin-jointed and, therefore, held at one end only. In the present case, however, the Administration des Ponts et Chaussées had specially asked the designers to consider each main frame as a rigid whole, and it was laid down that the roadway slab should not be taken into account for calculating the strength.

An attempt to work out the full continuity of the structure led, despite the simplicity of the method used, to impossibly complicated calculations and had to be abandoned. Advantage was accordingly taken of the work of Rieckhof and of the apparatus known under the name of Nu-Pu-Best. The steelworks were asked to build a metal model, to a scale of $1/25^{th}$, with its component elements so designed that their moments of inertia would be proportional to those of the

[1] Occurrences in France during 1936 led to many contracts being cancelled. In 1937 those for the steel bridges were awarded anew to contractors who are now engaged in carrying out the work on nearly the same lines, but in accordance with designs calculated in the offices of one of the firms concerned. The present paper is nevertheless being published in order to place on record a method of design believed not to have been previously described which is not affected by the circumstance that, for economic reasons, it was not carried into effect.

scantlings adopted in the preliminary design. By means of the Rieckhof bending apparatus it was then possible to determine the points of inflection, both real and virtual, for those members which had a small moment of inertia, but for reasons which will be evident this method could not be applied to the arches.[2]

By placing any given load in turn at every intersection point of the frame there was thus obtained the influence line for the shear force in the first vertical member to the left, and it was merely necessary to calculate the end frames in order to eliminate two hyperstatic quantities. Fig. 2 shows the shape of the influence line so obtained, and Fig. 2a the influence curve for the secondary thrust.

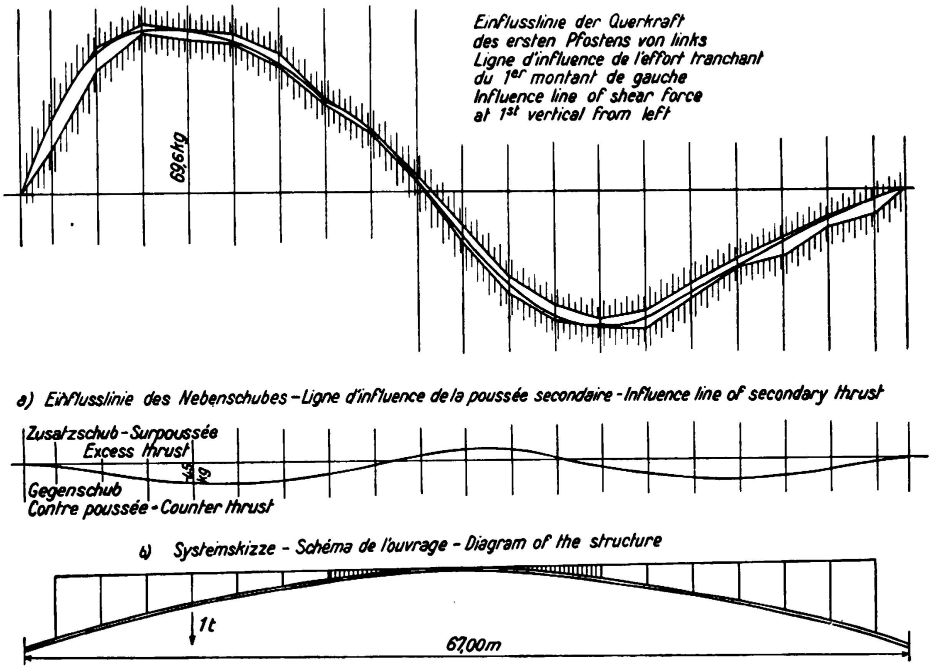


Fig. 2.

The arch had been investigated beforehand under the assumption that it was isolated, in order to determine the influence line for thrust —

1) under the action of unit vertical load;
2) under the action of unit horizontal load;
3) under the action of unit moment.

The calculations were made taking due account of the variations of the moment of inertia in the central portion. From the influence line described above, which is reproduced in Fig. 2, it was possible to work out influence tables for all the forces in all those sections of the structure which were to be welded, and at the same time a special investigation was made of thermal variations.

The engineers of Messrs. *Baudet, Donon & Roussel* suggested that it would be a great advantage if the joints in the model, which had at first been made cylindrical, were replaced by pieces corresponding in height to the sections of

[2] This method cannot properly be applied unless buckling effects have first been eliminated, but that is easily done.

the arches and the longitudinal beams respectively and a comparison of the model with the plan of the work made the utility of this suggestion apparent. Its importance was shown on examining a section of the model in polarised light, and at the same time this test furnished a useful justification for the method employed.

A calculation of the strains showed that the vertical and horizontal deflections of the monolithic frame were less than those which would occur in a pin-jointed frame.

Fig. 3 is a photograph of one of the models used. Special precautions were taken to eliminate the effect of friction due to the weight of the apparatus when arranged horizontally, and it was confirmed that the increase in the moment of

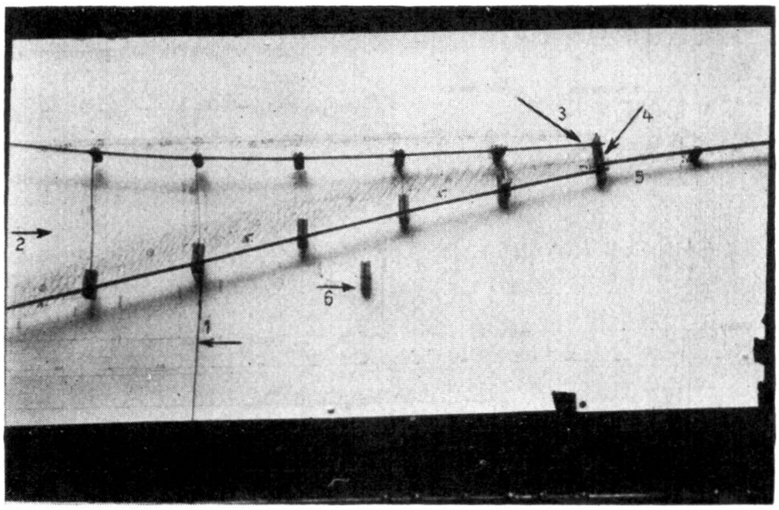


Fig. 3.

1 = A vertical load.
2 = Shear force x at the extreme left hand post.
3 = Total restraint of the last beam nearest to the crown.
4 = Unchangeable angle.
5 = Portion of variable sections (variability of moment of inertia).
6 = A single joint piece.

inertia due to the thickness of the joints had practically no effect on the results obtained.

Curve N° 2, together with curve 2A representing the influence line for secondary thrust, was plotted by comparing the results obtained in the calculation for the first frame to the left of the crown with those obtained by calculating the first frame to the right of the crown, these two series of calculations serving to check one another. In this way it was possible to delimit with approximate accuracy the region through which the curve would pass. Curve 2A was plotted at the same time as Curve 2.

2) *Calculation of the Cross Members and of the System as a Whole.*

By giving expression to the equality of the deflections occurring at each of the intersections between frames and cross members, a system of 228 equations with 228 unknowns is obtained; this can be considerably simplified, but remains quite insoluble. Could it be solved it would give the distribution of forces in the frames and the cross members for each case of loading under consideration.

What calculations could not give, it was decided to obtain by experiment. The firm of Baudet, Donon & Roussel were asked to build a model to scale of

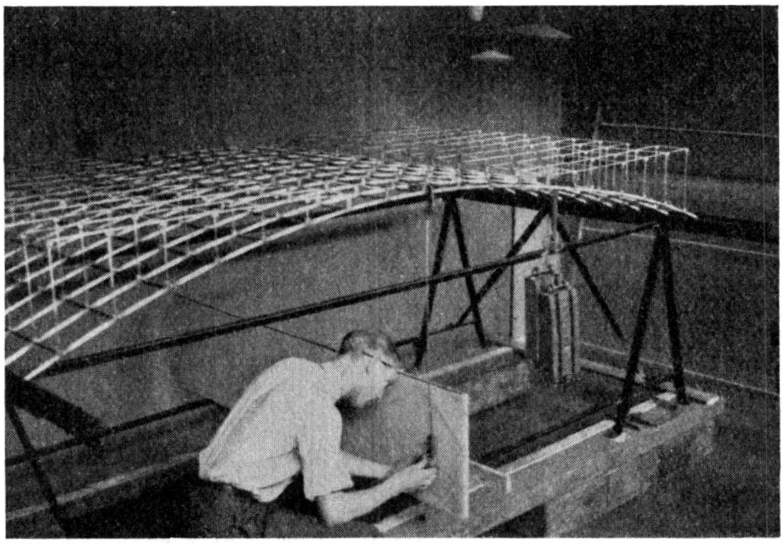


Fig. 4.
Three-dimensional model for measuring the vertical deformations.

1/25th of the system to be studied, conforming in its proportional dimensions to both the transverse and the polar moments of inertia so far as possible (see Fig. 4). At the same time a model was available of two arches identical with

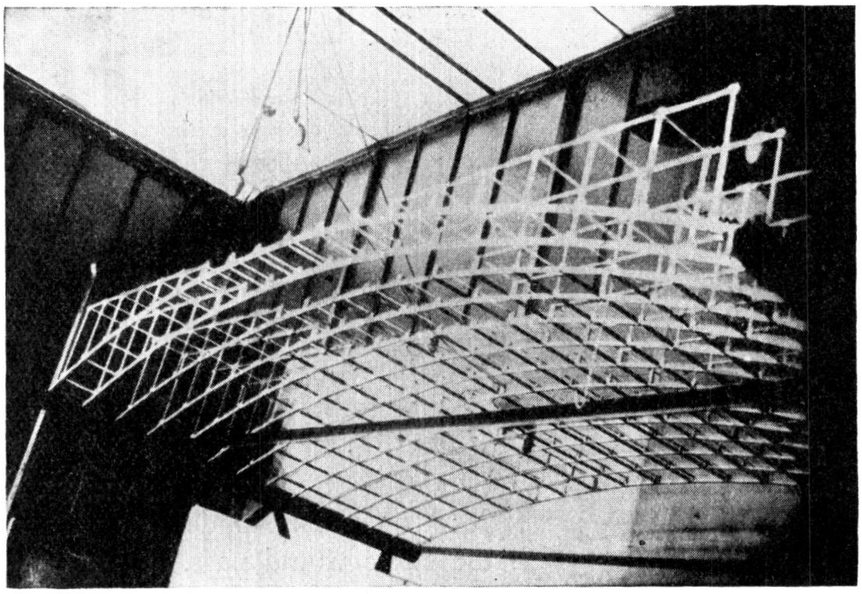
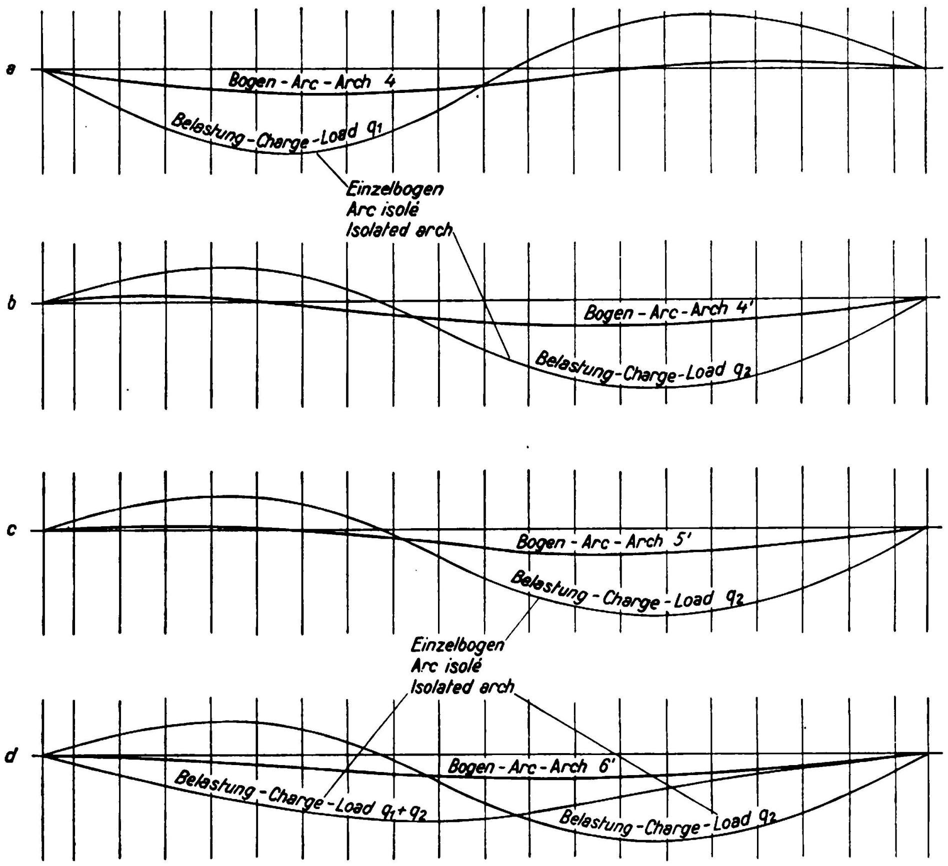


Fig. 5.
In the foreground the two-compound reference trusses;
below the tracing board.

the others, connected by bracings and cross members as in the main model. These two control arches may be seen in Fig. 5.

A load of 20 kg was applied to a large number of intersection points and the respective vertical deflections were measured. It was found that the horizontal deflections were very much smaller. When the elastic properties of the various cross members had been investigated a system of "compounding" or combined loading was established in the following way: each of the cross members whether simple or compound was replaced by a uniplanar model similar to that used for the



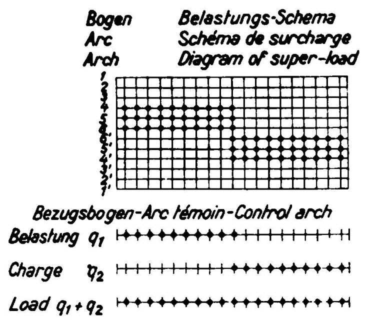


Fig. 6.
Comparison between the vertical deformations of the reference truss and the compound trusses 4, 4′, 5′ and 6′ all subject to the same external loadings.

determinations of curvature and points of inflections in the frames. The results thus obtained were not as accurate as in the case of the main frames, but it was possible to calculate the forces set up in the longitudinal, vertical, bracing and cross members to a sufficient degree of approximation. An influence table was compiled for each of the sections examined, and on comparing these tables with one another it emerged very clearly that the surcharge indicated in Fig. 6 was definitely the most unfavourable case; this applied to all of the sections considered.

It should be remarked incidentally that the cross members are not affected by dead load because the weight of the sidewalks is practically equal to that of the roadway, and the same is true in the case of live loads covering the full width of the structure.

A full investigation of the whole system would have involved "compounding" the cross frames, in exactly the same way as described above,[3] under loads assumed to be distributed in a chess-board pattern; but this would simply have furnished a check. From Fig. 6, which gives a comparison between the elastic properties of the control frame and those of each of the frames numbered 4, 5 and 6 under identical loads, it will be seen that there is a great difference in the vertical deflection which may amount to 60 to 70 %, and this shows that the corresponding actions in the various frames are considerably reduced by the presence of the cross members.

This result might indeed have been expected by analogy with the behaviour of a reinforced concrete slab supported on two edges and provided with distributing steel.

To summarise, it may be seen from this example that a structure can be designed by having recourse to experiment to provide what calculation is unable to furnish, without a disproportionate amount of labour. The essential point is to avoid empiricism and the doubtful degree of safety associated therewith. It must of course be understood that the experimental investigation should be followed up by calculation step by step. This method of using uniplanar and triplanar models can be applied to the study of a very large range of structures in two or three dimensions, composed of triangular lattice systems or frames. For instance, it would enable the study of secondary stresses in triangular lattice systems to be carried out with ease.[4]

The checks made by means of polarised light showed satisfactory agreement even in the case of very short members, and close agreement in the case of longer members.

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[3] We speak of imperfect "compounding" when the connections are merely situated at the places indicated by the three dimensional model, and of perfect "compounding" when the directions of the connections are the same also. It was established that within the limits of these experiments the difference between the two kinds of "compounding" was not negligible, but as a rule was not large.

[4] In the complete examination of a large structure over the Niger which is being undertaken it has been ascertained that the experimental method offers the only means of obtaining satisfactory results for the stresses in redundant members, and for secondary stresses.