Present-day tendencies in large-sized reinforced concrete constructions

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IVb 1

Present-day Tendencies in Large-sized Reinforced Concrete Constructions.

Neuere Gesichtspunkte für den Bau großer Eisenbeton-Bauwerke.

Tendances actuelles dans les grands ouvrages en béton armé.

S. Boussiron,
Paris.

The ambitions in the conception of large structures of reinforced concrete have been faithfully supported, if not provoked by the progress in the manufacture of cement and by the studies on its better utilization with given aggregates.

The resistances to crushing of 400 to 450 kg/cm² which it is possible to obtain on the building yard without having recurrence to any exeptional cares (the constancy of which could not be assured) permit the adoption of a working coefficient of 100 kg/cm² in round figures for reinforced concrete and of 150 kg/cm² for concrete with lateral reinforcements up to 1.10 %. This coefficient can even have as a limit 0.6 of the resistance to crushing, or 240 kg/cm² if the lateral reinforcing is done up to 3.6 %.

But more than ever is it necessary to say here that the solution of a great problem is not the amplification of an average one. The adoption of such stresses disengages the action of diverse phenomena, the study of which must be more deeply concentrated on.

The influence of the permanent load in large spans demands a reduction of all the sections to their proper limits; but this must be accompanied by a careful verification of the degree of stability of these sections with regard to an increase of the loads, or, a displacement of the pressure line. Calculation methods are therefore necessary which are not only reliable to disclose all stresses, but are also quick, enabling the author of the project to perceive early enough the difficulties of the arrangements made.

Finally, the construction of large structures can only be considered with practical and safe solutions for the scaffoldings, which are the most important position of expenditure.

In the following, we are indicating the tendencies which can be drawn from what has been done in France in this line in the course of the last few years.

PART ONE.

Arrangements and Calculations.

We are limiting our treatise to arched bridges. In fact, the arches constitute the only solution to which reinforced concrete is economically suitable as soon as spans exceeding 100 m are concerned. Any other solution would only be an adaption to steel constructions or to suspension bridges, and would have the disadvantage of causing the designer to solve such tensile force and tensile joint problems, which, without being irrealisable, are far from representing a judicious application of reinforced concrete.

The study of these arched structures actually marks a definite tendency towards a more scientific determination of their characteristics: shape, rise-span ratio and stresses.

Until now the general shape adopted for arched bridges was that of masonry bridges, perpetuated since the origin of these structures. No systematic research has been undertaken in order to determine the influence of the shape and other characteristics of the arch on the stresses produced in the sections nor of their repercussion on the size even of the sections.

The first research in this direction was made on the occasion of the construction of the Fin-d'Oise Bridge[1]; subsequently this research was completed by various studies[2] which accurately fix the scientific conditions for the design of these structures.

As the full application of these studies has only just been made on the occasion of the construction of the last realized large arched bridge (Roche-Guyon Bridge over the Seine) we consider it best to expose the method of efficient determination of an arch, by description of the research work that has been carried through in order to establish the characteristics of this structure, viz:

1) Study on the influence of the shape of arch (Variation of the moments of inertia of cross sections),
2) Selection of the rise-span ratio,
3) Selection of shape of sections,
4) Selection of working stresses of concrete.

We are then going to show that the limiting span of arched bridges can be deducted therefrom and are comparing the type adopted with other types of arches.

We are finally giving with some detail the mode of precise calculation which has served to determine the stresses in the arch studied.

In a second chapter, we are exposing on the other hand a few considerations on ordinary and special three-hinged arches.

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[1] See "Génie Civil" of February 1st 1930.

[2] *Vallette:* "Génie Civil" of May 9th 1931 and 2nd volume of Reports of the International Association for Bridge and Structural Engineering. Chalos, same work.

1st CHAPTER.

Statically indeterminate systems.

I. Variation of the moments of inertia.

Let us examine the curve of limiting values of the maximum moments which are produced in an arch of constant equivalent inertia and section (curve 1, Fig. 1) the centre of gravity of its pressure line being at $^1/_3$ of the rise (fig. 2, type I).

If keeping the same moment of inertia at the apex, by means of appropriate variations of inertia, the maximum moment at the quarter span points is increased, that at the springings automatically diminishes. The curves of limiting

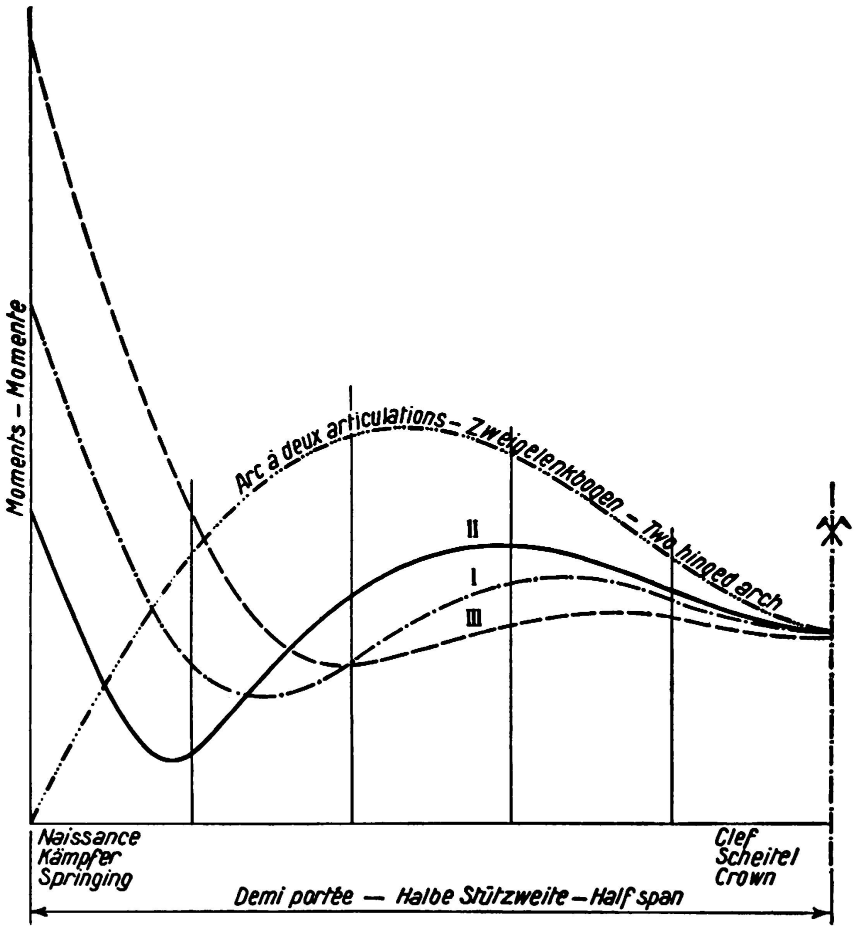

Fig. 1.

Enveloping curves of moments for three types of arches, I, II, III, with equal spans, equal span-rise ratio; and equal moment of inertia at the crown, as well as for two-hinged arches with constant reduced section of equal moment of inertia at the crown as for the above arches.

values of the moments show the course of that marked II in Fig. 1. They correspond to arches with equivalent moment of inertia decreasing from apex to springings, having the centre of gravity of the pressure line in the lower two thirds of the rise (Fig. 2, Type II) and which tend at the limit towards the two-hinged arch (Fig. 1) for which the moment at the quarter span points attains the highest maximum.

On the other hand, if the variations of inertia diminish the moment at the quarter span points, it is noticed that the moment at the springings increases. The curves of limiting values such as III (Fig. 1) are pertaining to arches with equivalent moment of inertia increasing from apex to springings, having the centre of gravity of the pressure line in the upper third of the rise (Fig. 2, Type III) and tending at the limit towards two cantilevers, connected by a hinge at the top.

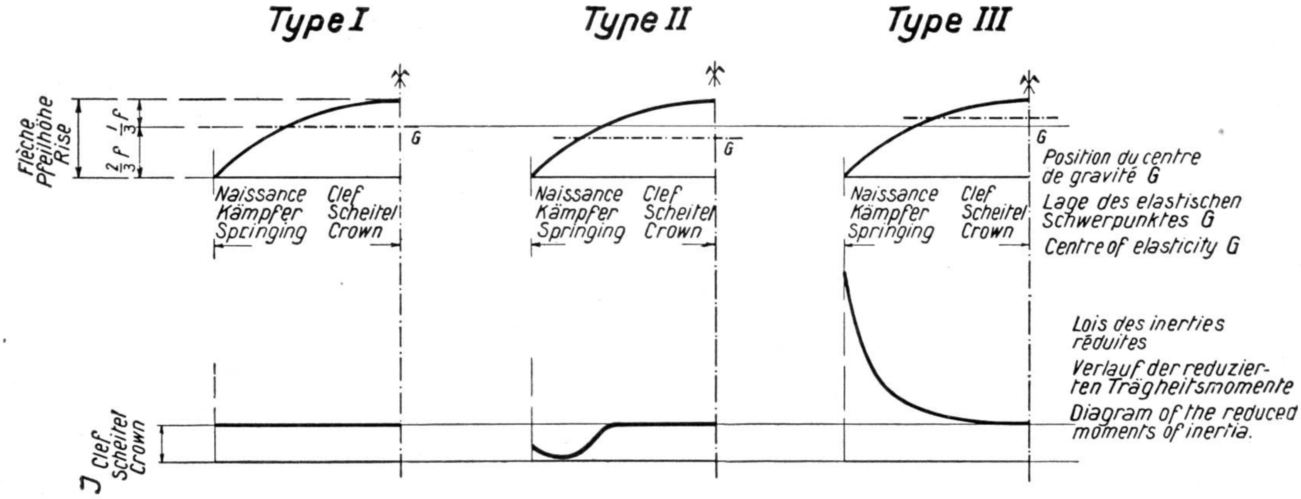

Fig. 2.

Position of the main axis of inertia, passing through point G of the middle fibre; diagram of reduced moments of inertia for arch types I, II and III.

It is obvious that between the two extreme cases, the smallest of these maximum moments will be obtained if the variation of the moments of inertia is such that they are equal at the springings and at the quarter span points. This research has led to the particular type of arch which has already been utilized

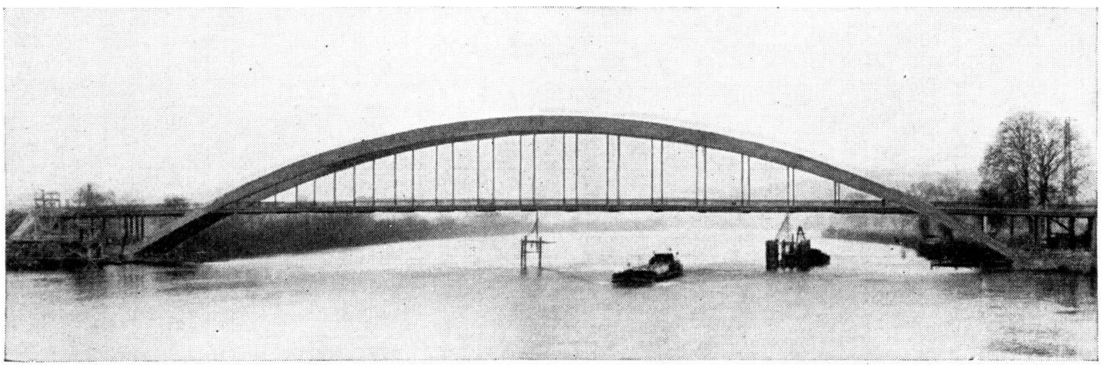

Fig. 3.

Arch-bridge of 161,0 m span over the River Seine at la Roche-Guyon.

in France in 1929 for the construction of the Conflans-Fin-D'Oise Bridge with a span of 126 m and which has just been applied in a still more interesting manner in the bridge constructed by us of 161 m span, across the Seine at La Roche-Guyon (Fig. 3 and 4).

In view of the extent to which we have carried the study of this structure, we shall take it as a basis of comparison with the different conceptions.

In order to do this in a clearer way, it is first of all necessary to determine the selection on the other characteristics of this arch.

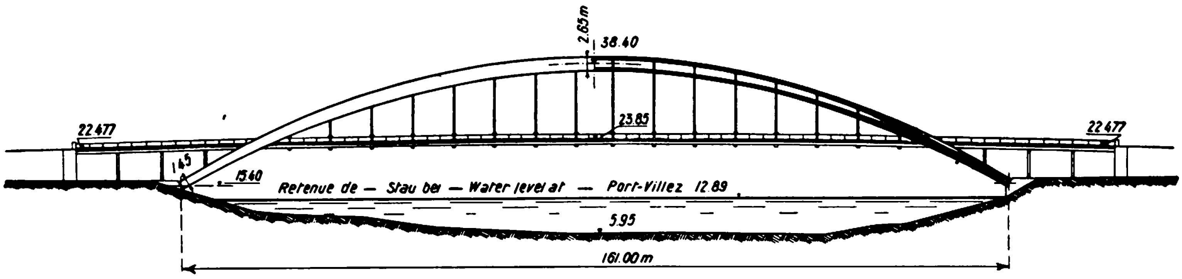

Fig. 4.

Bridge at la Roche-Guyon. Elevation and longitudinal section.

II. Selection of the rise-span ratio.

In order to determine the conditions of this selection curves B (Fig. 5) have been established which show the variation of the average section as a function of the rise-span ratio. On examination of curve 80^k — 161 — II which corres-

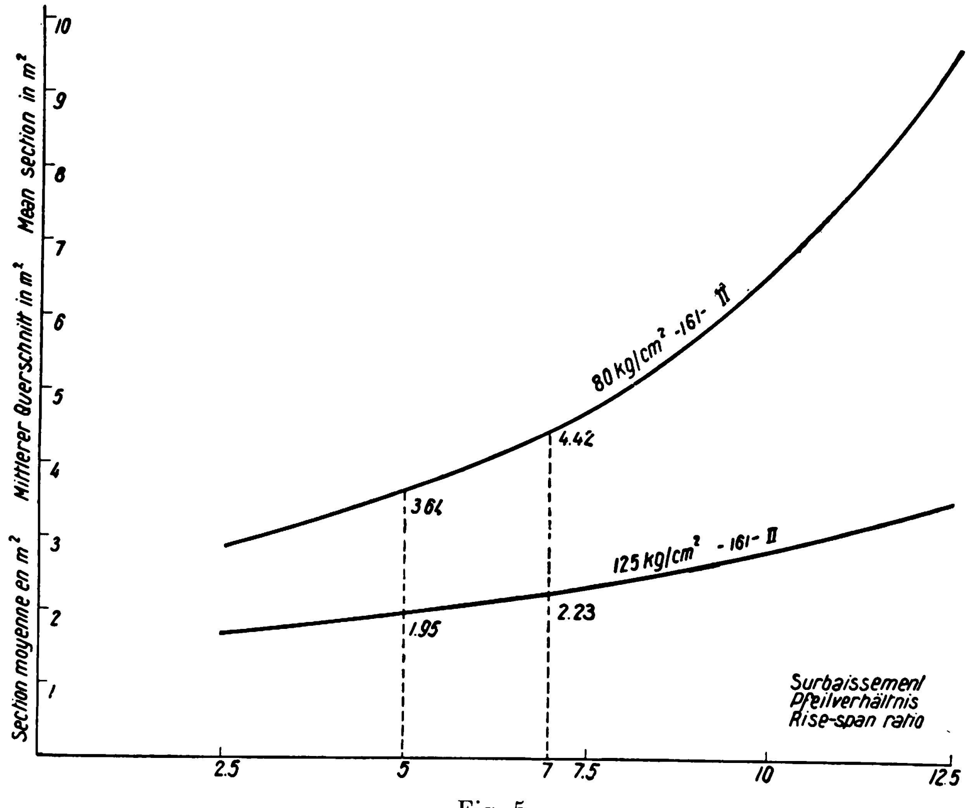

Fig. 5.

Curves B.

Changes of mean sections in relation to the span-rise ratio for arch type II of 161 m span and stresses of 80 and 125 kg/cm² respectively.

ponds to the arch at La Roche-Guyon with a span of 161 m and a maximum working stress of 80 kg/cm², shows that if the ratio rises from $\frac{1}{7}$ th to $\frac{1}{5}$ th the average section decreases from 4.42 m^2 to 3.64 m^2, which gives a ratio of 1.21 between the two areas.

With an increased working stress, the variations of section are far from being as rapid. Curve 125^k — 161 — II shows that for the same structure with a working stress of 125 kg/cm^2 the average section would vary from 1.95 m^2 to 2.23 m^2 in passing from a rise-span ratio of $\frac{1}{5}$ th to that of $\frac{1}{7}$ th (see Fig. 5). The ratio of these average sections is reduced to $\frac{2\,23}{1.95} = 1.14$. In fact, in taking into account the respective developments of the two arches, the ratio of the volumes of materials in only $\frac{1.10 \times 1.95}{1.054 \times 2.23} = 1.09$.

Wind effects are more intense on an arch with a large development and may necessitate extra material. This increase will diminish still further the ratio of 1.09 found above.

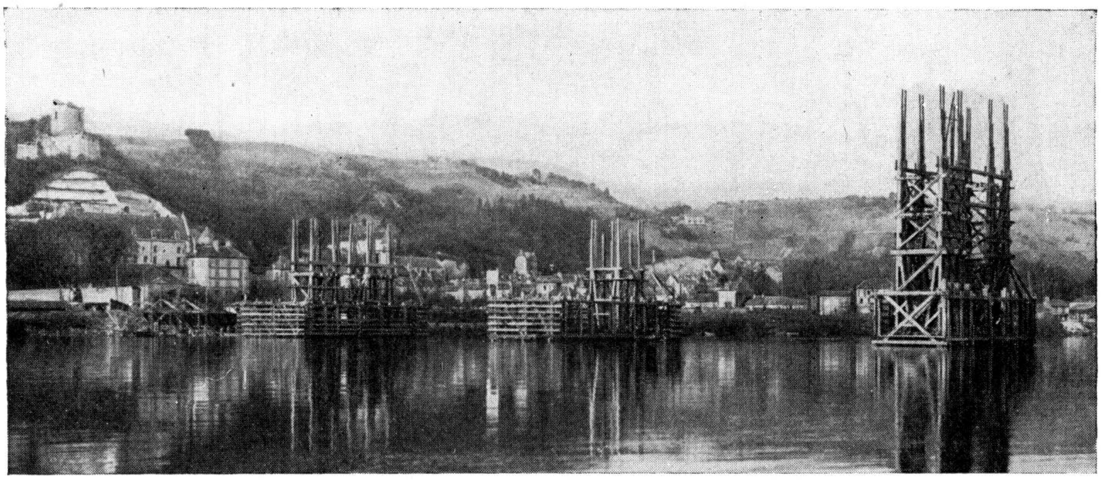

Fig. 6.

Bridge at la Roche-Guyon. View of steep right-hand bank.

Finally, in this particular case, the determining factor in the choice of the rise-span ratio will be the increase of the thrust on the abutments which is nearly proportional to it. Without considerable repercussion on the average section, the degree of reduction of the thrust will be adapted to the facility of abutment foundation, in accordance with construction conditions and aesthetic reasons. At Roche-Guyon, we have adopted a rise-span ratio of $\frac{1}{7}$ th which correctly proportiones the height of arch above the decking with the landscape, dominated by the cliffs on the right bank (Fig. 6).

The limitation of the rise is besides in concordance with the desire of restricting the height of the scaffolding above the flooring.

These conclusions only apply to the types of arch for which the variation of the moments of inertia has been judiciously studied; for other types, the rise-span ratio, according to the working stress, may be of considerable influence to the sections.

III. Selection of the shape of section.

The analysis of the influence of the shape of the section on the working stress would show the necessity of choosing a large section made up of relatively narrow members. (See "Génie Civil", 9th May, 1931. *M. Vallette.*)

In the case of an arch with suspended roadway the span of the cross beams should not be too greatly increased.

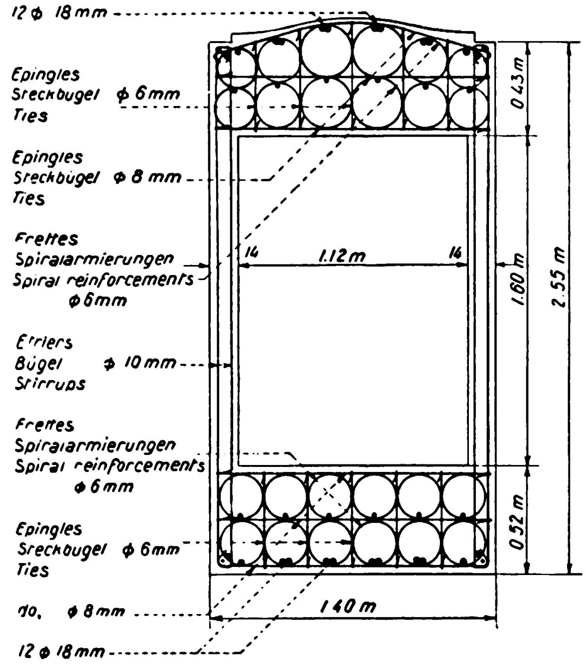

Fig. 7.
Cross section trough crown.

For this reason a breadth of 1.40 m was adopted for the whole of the arch above the level of the roadway. Then, in order to have the smallest possible ratio $\frac{h}{l}$ since a large, low arch is considered (See "Génie Civil", 9th May 1931.

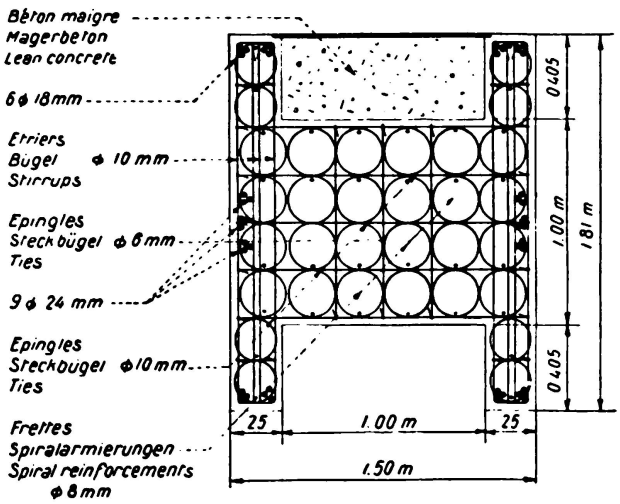

Fig. 8.
Cross section through quarter-span,

M. Vallette), the breadth of the arch was progressively increased up to 3.00 m and its height decreased to 1.45 m or less than $\frac{1}{110}$ th of the span. (See Figs. 7, 8 and 9.)

IV. Selection of the working stress.

The choice of suitable working stress is of great importance, as is shown by the full curve II (Fig. 10) which gives the variations of the average section as a function of the stress for an arch of 161.00 m span with a rise-span ratio of $\frac{1}{7}$ th carrying the required live load and differences of temperature.

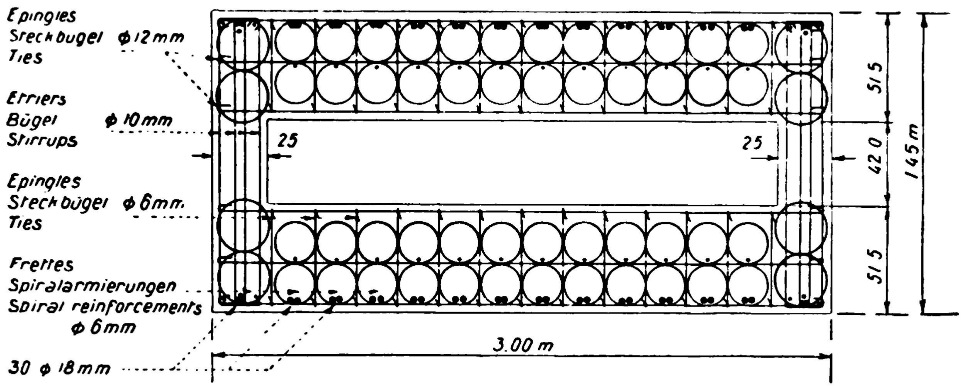

Fig. 9.
Cross section at springing.

If we had not analyzed these variations, we might have been tempted in order to avoid lateral reinforcement of the concrete to choose a current working stress. From the curve it will be seen that for a limit of working stress of 80 kg/cm^2 the mass of the arch would have been twice that necessary for the arch which was finally chosen having a working stress of 125 kg/cm^2. Curve 161 — 7 — II; curves I and III are relative to other laws of variation of the moments of inertia which will be mentioned later in the comparative study.

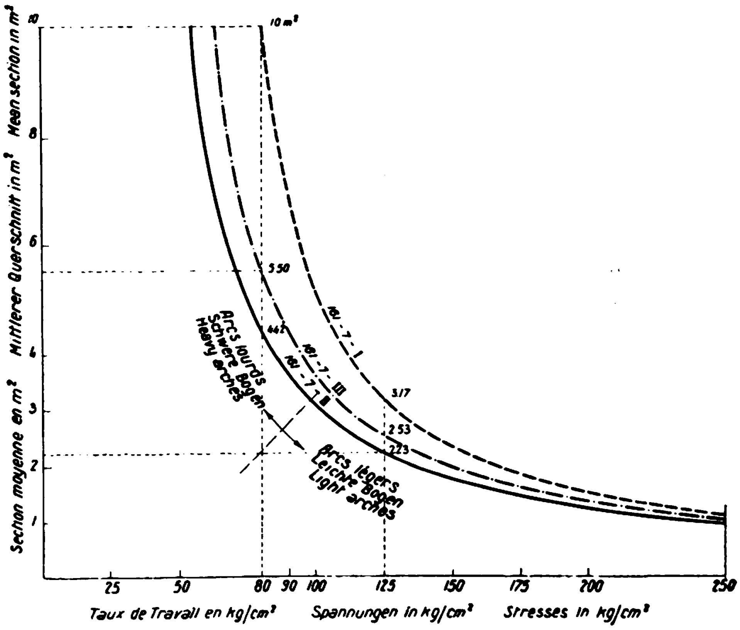

Fig. 10.
Curves C.
Changes of mean sections in relation to stresses for three types of arches of 161 m span and rise-span ratio of 1 : 7.

This choice was governed by the necessity of obtaining the smallest arch capable of retaining sufficient security against sudden increase of stress through accidental causes. For a very small decrease of section, this increase might be considerable in the horizontal branch of the hyperbola which corresponds to "light" arches. For this reason the stress was limited to 125 kg/cm². Its stability was proved by checking the working stresses of several sections to ensure that the figure of 12 kg/cm² for the tension reinforcement and 180 kg/cm² for the concrete were not exceeded when the live load was doubled.

On each of the curves a point can be found where for a decrease $d\Omega$ of the section, there is a corresponding increase dR of the working stress which is such that $\frac{d\Omega}{dR} = \text{constant}$.

The last condition defines the stability of an arch by the value of the tangent to the curve at the point corresponding to the desired working stress (or by a multiple of this tangent when the scale of the Ω and the scale of the R are different and consequently the curve is deformed). These considerations, and the statements previously made, have led to the approximate relationship.

$$l = \frac{\varepsilon R^n}{e^\alpha}$$

V. Limit span of arches.

The constant α is determined by the type of arch and for the same rise span ratio $\frac{1}{e}$, the constants ε and n, approximatively determined, give the arches practically the same stability and consequently the same character, heavy or light. Thus, the formula above gives a practical means of choosing the working stress of the concrete as a function of the span and the rise. As already seen, this choice is of fundamental importance in the design of an arch which is to be stable and economical.

By means of this formula, the limit l of the span of arch can be determined immediately.

We have traced in figure 11, for increasing spans curves, giving the value, as a function of the working stress R, of the variations of the average section of arches type II with a rise of $^1/_5{}^{th}$ of the span.

This figure also shows curves of equal stability for light arches. The curve marked 1 separates the heavy arches from the light arches. With the scales chosen, it corresponds to the value $\frac{d\Omega}{dR} = 0.005$. For "La Roche-Guyon", we have taken $\frac{d\Omega}{dR} = 0.0025$, or 0.5 on the scale of the curve.

From the above equation a curve has been plotted for a stability of 0.4 and is shown by a dotted line. This equation has the advantage of slightly increasing the stability of very large spans.

It will be seen that the curves are drawn on the assumption that, for all spans, each arch carries a load of 6600 kg/m length in addition to its own dead weight: 2000 kg/m for the live load and 4600 kg/m for the floor, the hangers, wind bracing and all other accessories. This corresponds to a free width of 8.00 m. The curves mentioned above are such that the average section of the arches is almost proportional to the live load, the latter being itself proportional to the width.

This results from the general formula (2) established by *M. Vallette* (2nd volume of Publications of the association, Zürich 1934).

$$\Omega_0 = pl \frac{C_5 e\lambda + C_6 \frac{e}{\lambda} + \frac{C_8}{\lambda'} + \frac{C_1 l}{2a^2h} + \frac{C_2 h e^2}{2l}}{R - C_4 \lambda l e + C_7 \frac{a^2}{\lambda}\left(\frac{h}{l}\right)^2 e^2 - C_3 \frac{h}{l} \times \frac{e}{2}}$$

The application of this formula to arches of type II (type La Roche-Guyon) after determination of the coefficients C, gives for the section at springings the formula (3)

$$\Omega_0 = pl \frac{0{,}124 k_1 e\lambda + 0{,}0376 \frac{e}{\lambda} + 0{,}329 \times \frac{1}{\lambda'} + 5{,}95 + 0{,}00163 e^2}{R - 0{,}191 \lambda l e + t^o e \left(0{,}0025 \frac{e}{\lambda} - 0{,}603\right)}$$

Arches of type I with constant equivalent moment of inertia and section would give (4)

$$\Omega_0 = pl \frac{0{,}121 k_1 e\lambda + 0{,}04 \frac{e}{\lambda} + 0{,}35 \times \frac{1}{\lambda'} + 3{,}57 + 0{,}005 e^2}{R - 0{,}28 \lambda l e + t^o e \left(0{,}0127 \frac{e}{\lambda} - 1{,}54\right)}$$

Arches of type III, evolved by *M. Chalos*, Ingénieur des Ponts et Chaussées, Chef du Service Central d'Etudes Techniques du Ministère des Travaux Publics[3], but improved by the selection of hollow sections, of perceptibly reduced constant area, would give (5)

$$\Omega = pl \frac{0{,}125 k_1 e\lambda + 0{,}038 \frac{e}{\lambda} + 0{,}33 \times \frac{1}{\lambda'} + 4{,}85 + 0{,}002 e^2}{R - 0{,}248 \lambda l e + t^o e \left(0{,}0037 \frac{e}{\lambda} - 0{,}61\right)}$$

In these formulae k_1 represents the ratio of the weight of the floor and the accessories to the live load. Even for a variation of this ratio from 2 to 3, which is the extreme limit, the repercussion of this variation on the formula would be insignificant. It should be borne in mind that the formulae 3, 4 and 5 only apply to road bridges. For railway bridges with the same parameters $\frac{h}{e}$ and a^2, they are modified by respective coefficients[4].

For example for a span of 800.00 m with a free width of 16.00 m figure 11 shows that the section of an arch with a stability 0.8 would be:

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[3] See 2nd volume Publications of the Association, Zurich 1934.

[4] h = height of section, r = radius of gyration = ah Ω = area of section J = Ω a^2 h^2.

$$5.90 \times 2 = 11.80 \text{ m}^2$$

with a stress of 252 kg/cm².

Even prior to the application in actual practice of the new methods from which an increase in the strength of concrete is to be expected, the current methods permit the conception of such a span. Since the working stress of spirally reinforced concrete can be raised if desired by increasing the percentage of reinforcement, the only limit is 0.6 of the crushing strength of concrete without reinforcement.

With careful workmanship a minimum strength of 420 kg/cm² at 90 days can be guaranteed which corresponds to the working stress above of 252 kg/cm².

This possibility as far as it is due to the quality of the material must not, however, eclipse the difficulties of the problem with regard to execution, the essential element of which is the scaffolding. This question will be treated later on.

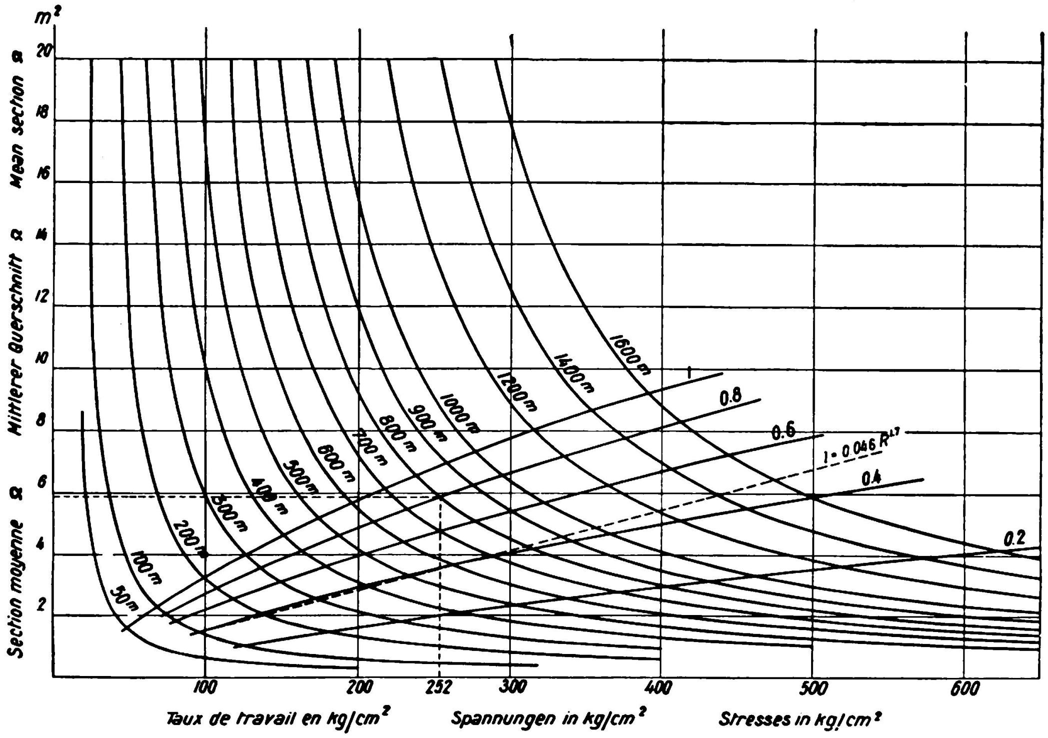

Fig. 11.

Curves C.

Changes of mean sections in relation to stresses and stability curves for arches with rise-span ratio 1 : 5 for different spans. Assumed live load 2 tn/m and change of temperature ± 25°.

VI. Comparison with other types of arches.

Curve II on Fig. 12 indicates the type of arch used, and this is compared with two other clearly defined types, namely:

A parabolic fixed arch with a constant equivalent section (I) mentioned in most text books on strength of materials.

An interesting type of arch whose equivalent moment of inertia increases from the crown to the springings according to the rule:

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$$J' = \frac{J_{crown}}{1 - \frac{K-1}{K} m^{\gamma}}$$

evolved by *M. Chalos,* who has prepared tables for its rapid calculation. (International Association of Bridge and Structural Engineering, 2[nd] volume of Publications, Zurich 1934) m designates the parameter $\frac{x}{a}$, ratio of the abscissa to the half span of the arch, and k is the ratio of the equivalent moments of inertia at the springings and at the crown.

It is to be noticed that for $K = 1$, the formula applies to arch I.

It was assumed that the arches of La Roche-Guyon were constructed to conform to these types under the same conditions of live load and temperature. In order to reduce their average section to a minimum, we made use of the

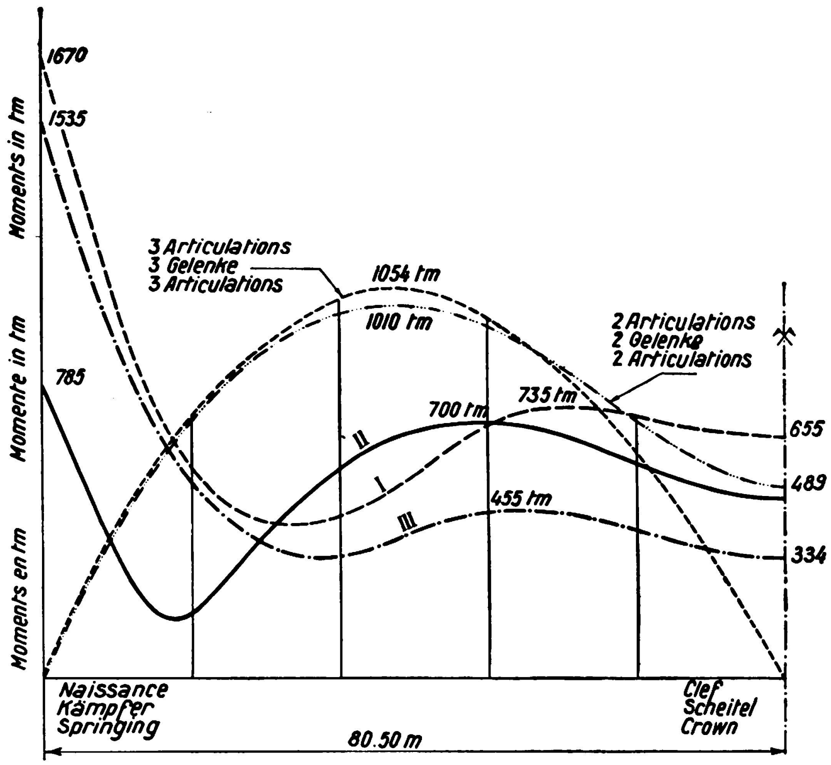

Fig. 12.

Curves of extreme values of moments for the bridge at la Roche-Guyon for five alternative types of arches.

studies of *M. Vallette* as regards shape of cross section. They, therefore, are rectangular in shape, 1.40 m wide for the part above the floor and gradually increasing down to the springings. This increase is governed by the necessity of obtaining a moment of inertia at springings, which in the case of these two types is more than five times as large as the La Roche-Guyon type and necessitates the use of a wide section.

Figure 13 shows the curves of the equivalent moments of inertia J′ and of the equivalent sections Ω′ which were used in the calculations mentioned above for the three types of arch.

Assuming, as in the case of the arch of La Roche-Guyon that the axis of the arch coincides with the pressure line for dead weight, not taking into account the effects of shrinkage, it was possible, due to the laws of similitude, to trace for these arches the curves which give the variations of the average section as a function of the stress (Fig. 10, curves I, II and III).

The type of arch for which the moments are equal is preferable for arches of average stability. In practice this is found to be the best type, as heavy arches are not economical and very light arches are not stable. Thus, for the working stress of 125 kg/cm² used at La Roche-Guyon, giving an average section of 2.23 m², a figure of 3.17 m² would be necessary for an arch of constant equivalent section, and 2.53 m² for an arch of the type recommended by *M. Chalos* i. e. $K = 5$ and $\gamma = 2$.

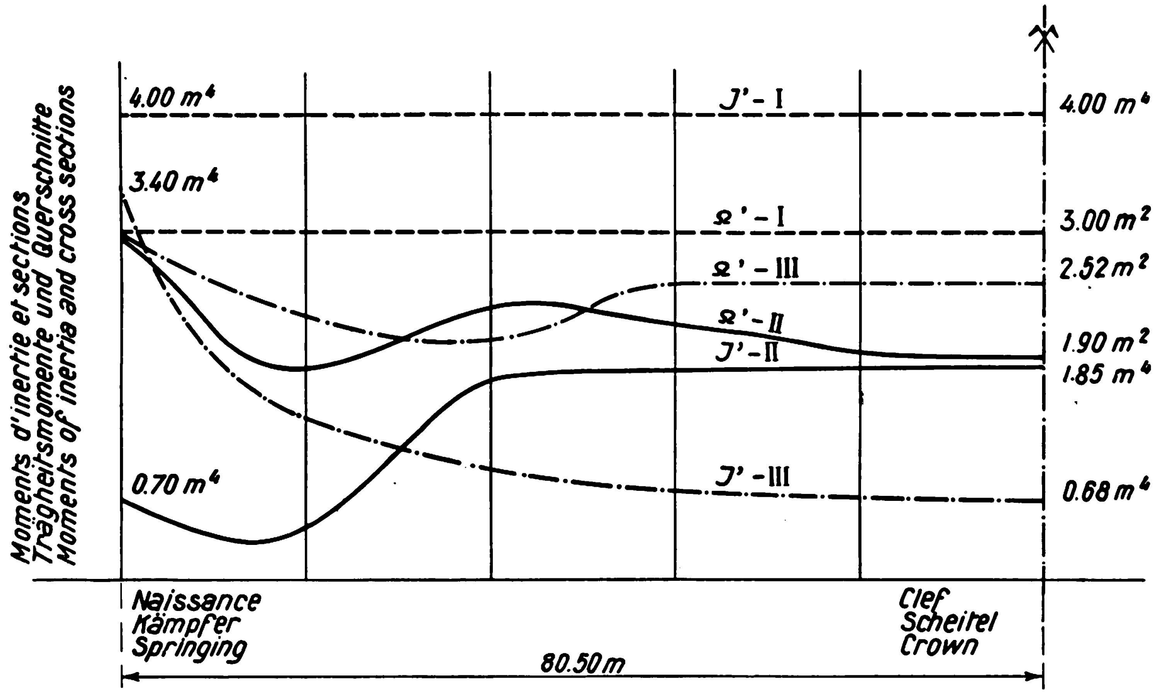

Fig. 13.

Reduced moments of inertia and reduced section for all points of the arch for the La Roche-Guyon bridge for types I, II and III. These values served to plot curve C of fig. 10, of the extreme moments of fig, 12.

We emphasise that in order to obtain this result, the last arch was modified so that the strength should be equal and the stresses amount to 125 kg/cm² throughout.

With this object any rule for the areas was chosen and only the rule for the actual moments of inertia was strictly observed

$$J = \frac{J_{crown}}{\left(1 - \frac{4}{5} m^2\right) \cos \alpha}$$

This is indeed the only preponderant rule, determining the distribution of the stresses; the areas of the sections are of influence only through their average.

The advantage of the type which we have adopted, already great in decreasing the bulk of concrete, is incontestable with regard to the abutments. It shows the most reduced moments at the springings, as is indicated by figure 12 in which the envelopes of the maximum moments have been plotted. To the advantage with regard to the moments, a further one, concerning the normal force is added. The abutment reactions are as follows:

Type II La Roche-Guyon	M = 785 tm N = 1850 t
Type III of *M. Chalos* with optional rule for areas:	M = 1535 tm N = 2060 t
Type I with constant equivalent section	M = 1670 tm N = 2200 t

It can be argued that a two-hinged arch would have avoided all moments at the abutments. But if a normal stress of 2000 tons for each arch is reached, hinges become either difficult to carry out in spirally reinforced concrete, as they

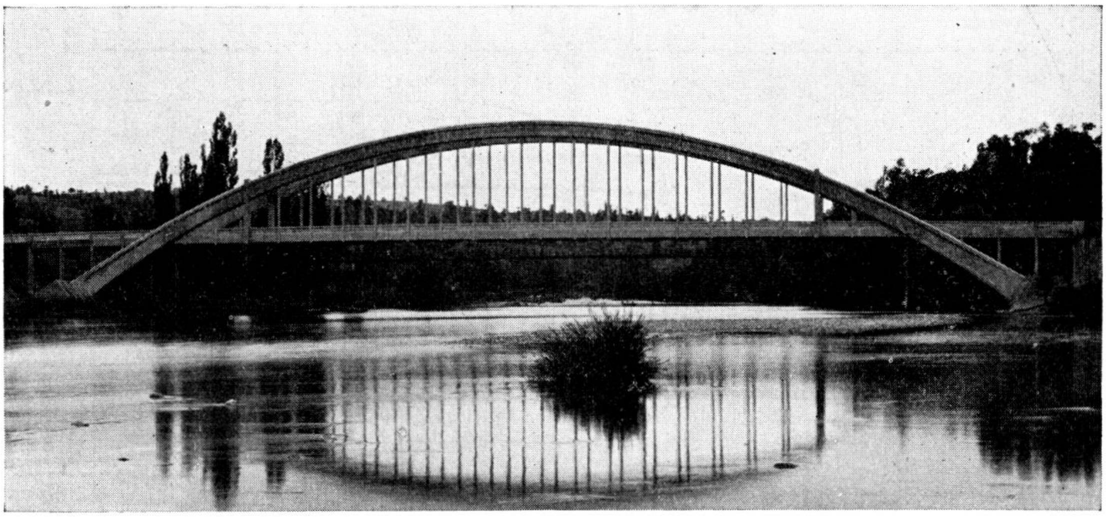

Fig. 14.
Bridge at Bas-en-Basset.

must reach widths of the order of 5.20 m per arch, or costly, if cast steel pieces are used. Further, the average section is larger than in the case of the fixed arch of the type which was used. As a prove, we have plotted the envelope of the maximum moments for a two-hinged arch applied to the conditions of La Roche-Guyon (Fig. 12).

As already seen, it is possible to obtain arches having nearly the same stress in any section, no matter how the moment of inertia varies, i. e. it is always possible to get an arch of equal strength of the reinforced concrete, by varying shape and area of the sections. But among all these types of arch of constant stress, there are two, which give the minimum bulk for the work: one is the arch whose moment of inertia increases from the crown to the springings according to the *M. Chalos* theory, the other is that whose moment of inertia decreases in the same direction of the type used at La Roche-Guyon. According to the actual problem that has to be realized, the advantage will be on one or the other side, depending on the risespan ratio and the ratio of dead weight to live load. For type III, of course K and γ have to be chosen in the best possible way.

M. Chalos' rule is particularly apt for bridges with roadway above, for which the diminution at the crown is disirable not only with regard to the free height, but also concerning aesthetics of appearance, resembling the beautiful masonry bridges enlarged at springings (Fig. 14). Obviously the foundation soil must be capable of bearing the moments, working on piers and abutments.

As examples of the structures built according to this rule, we are mentioning:

The Bridge over the Loire at Bas-en-Basset with a span of 112.00 m erected by the "Société de Constructions Industrielles et de Travaux d'art" (Fig. 14). The axis of the arch is a parabola of 4^{th} degree. The hight of section at the crown is 1.90 m and increases steadily towards springings where it amounts to 3.275 m. The favourable adaption of this type of bridge to the character of the landscape deserves mention. The aspect of scenery is obstructed only to a very small extent by construction members for the double arch and the roadway.

Fig. 15 shows the bridge over the Lignon.

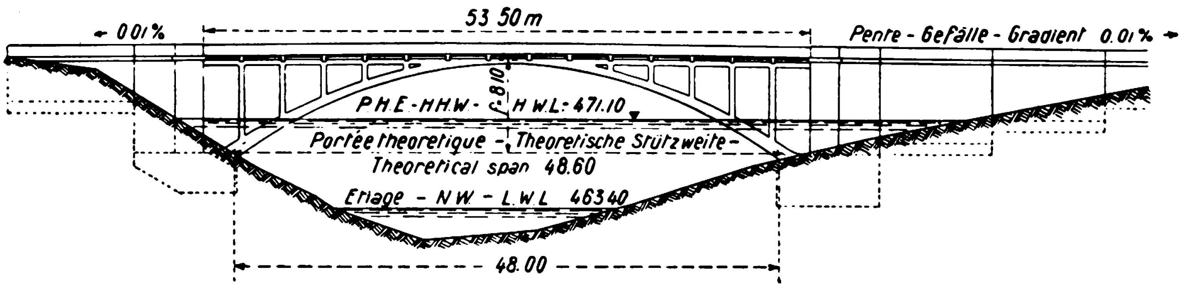

Fig. 15.
Bridge over the River Lignon.

VI. Calculation.

With restriction to that value of the coefficient of stability, below which it would not be advisable to go, — whereby Fig. 10 and 11 may be used to determine its rate —, the light forms are the only ones to be considered for bridges of large span. Contrary to this advantage, they require more accurate calculation than the massive forms, where weight itself helps to stabilise the stresses, but the results justify the extra work. Besides, the rules of general similitude[5] permit to carry out this study only once for all arches of a certain type; stresses and sections of an arch of equal shape but of different span, rise-span ratio and strength of material being deducted therefrom by simple proportion.

We have verified this on the occasion of the La Roche-Guyon Bridge.

A model type for the shape of arch adopted, having been established several

[5] *M. Vallette:* "Génie Civil" of 9^{th} May 1931.

years ago, has furnished, by application of the rules of similitude the following values:

At the springings	M	$= -796$ tm
	N	$= 1842$ t
On the haunches, at 32.00 m from the crown	M	$= +710$ tm
	N	$= 1646$ t
At the crown	M	$= +520$ tm
	N	$= 1620$ t

To bear out these results we have also made the direct calculation of the arch of La Roche-Guyon by using the most accurate methodes.

The formula for stability and the methods of graphic calculation described by *M. Vallette* in the "Annales des Ponts et Chaussées" (VI 1925) have been used.

The graphical method is the only one by which the distribution of stress can be determined accurately and also shows any rapid variations in the moment of inertia at given points and thus eliminates any possibility of error. The degree of accuracy is extremely high and correction to the curve of thrust can be made to within 5 %.

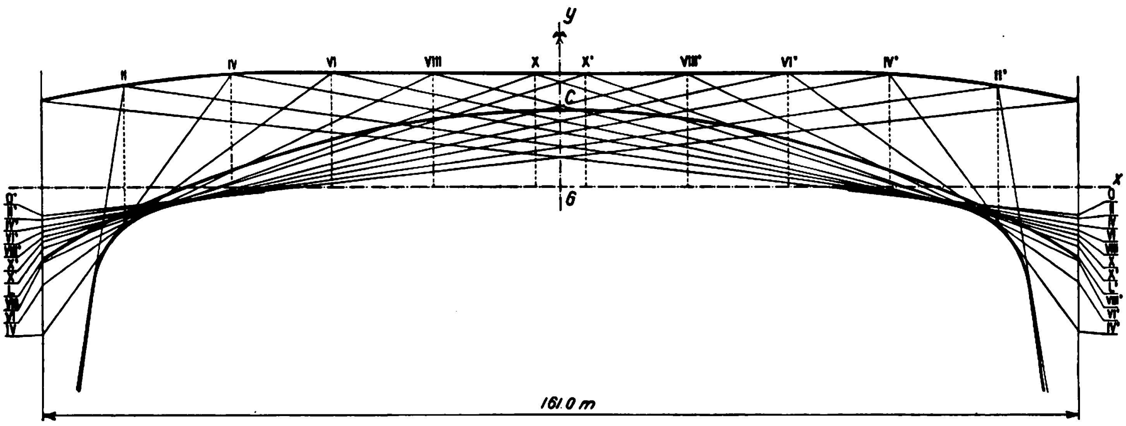

Fig. 16.

Reaction force line and enveloping curve to reaction forces at springing for te arch of the La Roche-Guyon bridge.

In spite of opinions to the contrary, it can be used throughout the whole span of an arch. It is unnecessary to make calculations for obtaining the effect of a load placed near the springings. It is only necessary to adapt this procedure at these points by taking different origins for the abscissae by which a more accurate value may be obtained for certain constants.

The moments, thrusts and shears have been determined directly by means of the graph giving the reactions, each peak being taken to the right of a hanger. The reactions for pairs of hangers are given as well as the curve of the intersections, and the curves of the reactions at the supports (Fig. 16). This proceeding is so sensitive that it is impossible to make an error on the position of the reactions. These succeed each other in order to form the complete curve without overlapping, and cut out, on the verticals of the abutments, segments which increase in size and so conform to recognised design.

It was assumed that the maximum stresses were given at every point by the maximum moment combined with the normal thrust which corresponds to it. This theory bears out the actual fact.

The forces obtained were:

At the springings	M	$= -785$ tm
	N	$= 1842$ t
On the haunches at 32.00 m from the crown	M	$= +702$ tm
	N	$= 1641$ t
At the crown	M	$= +490$ tm
	N	$= 1615$ t

If we compare them with the values previously obtained by the rule of similitude, we notice that those were entirely sufficient for the accurate determination of the arch and that the exclusive use of this procedure would have been perfectly justified, even with the minute differences in shape (funicular polygon for dead weight) which always exist between two arches.

The only want of precision of the calculation for the determination of the action of the structure could, therefore, be attributed to the value of the coefficient of elasticity of concrete. But before proceeding for La Roche-Guyon, to the compensation of the further forces from residual shrinkage dead loads of the roadway, we ascertained the value of this important factor. In this connection, we had made some very interesting experiences at the bridge over the Qued Chiffa of the normal gauge railroad from Algiers to Oran.

Each arch of vibrated reinforced concrete was shaped like a rectangular caisson, 5.00 m wide. The type adopted was that for which the moment of inertia decreases from the crown to the springings. This was rendered necessary by the large rise-span ratio, the considerable effects of the live load (locomotives), and the variations of temperature which are of considerable importance in Algeria.

In order to work with precision, we lowered the centering first, as we had noticed in similar cases, that the precision of the observations had been influenced by the fact that the yield of the compression of the timber maintains the support of the arches on the centering at quarter span points. The height by which the arches would have to be raised in order to free them from this support is considerably greater than that which corresponds to the desired compression for the lowering of the centering and the compensation and this would create considerable bending moments in the vicinity of the crown along the freed part of the arch.

The arch having been freed, it was necessary only to operate the jacks in order to obtain the theoretical deformations and to arrive at a position of the arch where the curve of pressure of the loads in action almost coincides with the neutral axis. The arch is then without secondary stress. This condition is called the "neutral state" of the arch and is obtained when, after the centering has been entirely lowered, the arch is brought back by the jacks to the position which it occupies in the first place on the centering (less the lovering) $\int N \frac{ds}{E\Omega} \times \frac{dy}{ds}$, corresponding to the compression which is negligible in this case), the opening of the joint being without rotation.

Having ascertained the trust Qe, which in the neutral state of the arch is centred on the neutral axis, and the real shortening of the arch, definite data is available for determining the coefficient of elasticity E. This coefficient has been found equal to 2.1×10^6 t/m² for the first arch and to 2.3×10^6 t/m² for the second arch. The information was carried ont 18 days after the completion of concreting work for each arch.

This applies to concrete with 400 kg of cement. We were, therefore, right in adopting at La Roche-Guyon where the ratio of mixture is the same, the value of 2.2×10^6.

It will be noticed in the second part of this report, relating to the construction of structures, that this value has been very closely corroborated by precise observations made during the lowering of the centering of successive elements and of the complete arch, as well as during the compensations.

2nd. CHAPTER.

Three-hinged Arches.

The calculation of these arches is very simple, since the reaction are determined without recourse to elastic deformations and as temperature and shrinkage have no influence.

As a means of comparison, we have traced in figure 12 the curve of the maximum moments for an arch of the same shape as that of La Roche-Guyon, but with three hinges.

The area of the Mdx, representing the size of the average moment is larger than that of types II and III, but smaller than that of the arches I and IV. The uneven distribution of the moments, passing from O to a maximum, does not correspond to a good utilization of the material. The advantage which this type may have, due to the suppression of the moments at the abutments, is opposed by the use of expensive hinges which, in our opinion are not apt for large spans because of the high local stresses in the concrete. If for the chosen coefficient of stability, working stresses of more than 125 kg/cm² are permissible and judicious, it will be difficult to use other than carefully constructed cast steel hinges in order to keep within strict limits the indeterminateness of the point of application of the reactions in the surfaces in contact.

For large rise-span ratios, it will be necessary to give full attention to the displacements of this point. If the popular arrangement of rolling a convex surface on a concave one of larger radius or on a plane one, this displacement may attain under the influence of shrinkage, in addition to that of temperature a dimension which is no larger negligible. Furthermore, the influence of the slow compressions of concrete has to be considered, on which Engineer *Freysinnet* has drawn the attention of the designers, several years ago, and which is the object of careful studies in order to determine its laws.

For the three-hinged arches the above mentioned phenomena acquire greater importance because of the unrestricted possibility of rotation. Thus, for a shrinkage of 0.022 mm/m which is to be expected after lowering the centering the fixed arch of La Roche-Guyon settles 0.0548 m whereas a three-hinged arch of the same span and the same rise would settle 0.067 m. The same values would result from a drop of temperature of 20 degrees.

If some particular circumstances were responsible for the construction of three-hinged arches of large span and of large rise-span ratio, we would judge it advisable to reserve the possibility of replacing after some time of service the jacks, used for lowering the centering in order to re-establish the arches in their original position after the effects of shrinkage or of slow compressions of the concrete.

However, it is just to recognize that up to spans of 100 m, the three-hinged type of arch has been able to furnish some interesting solutions. One of the latest applications is that over the Meuse with two bridges of a span of 97.00 m and a rise of 9.00 m according to the plans of the Société Charles *Rabut* et Cie (Fig. 17 and 18).

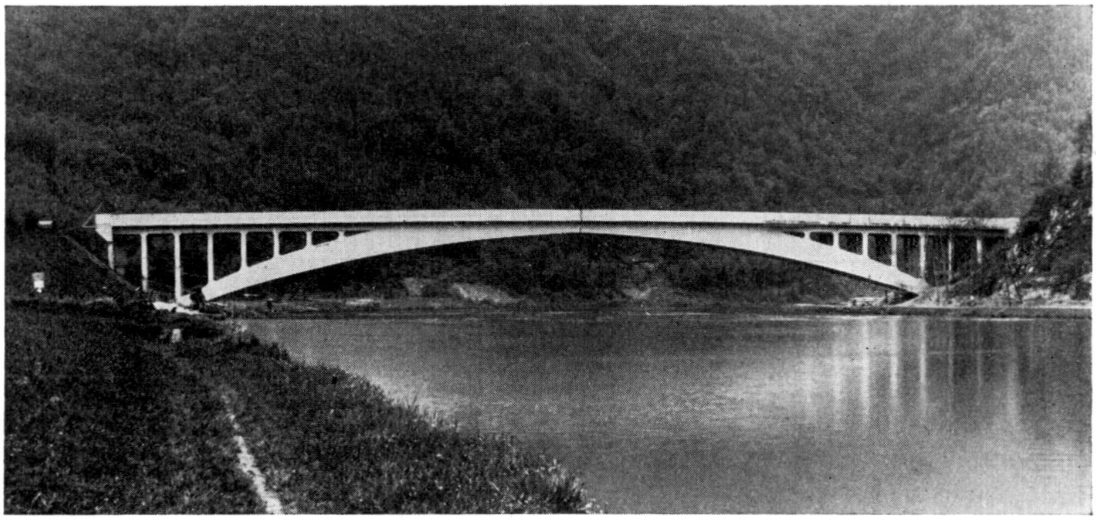

Fig. 17.

Three-hinged arch bridge of 97 m span over the River Meuse at Laifour.

The author of the project has made use of the advantage offered by the arrangement of the roadway above. The roadway platform serves as compression slab on both sides of the crown up to the point from which, with respect both

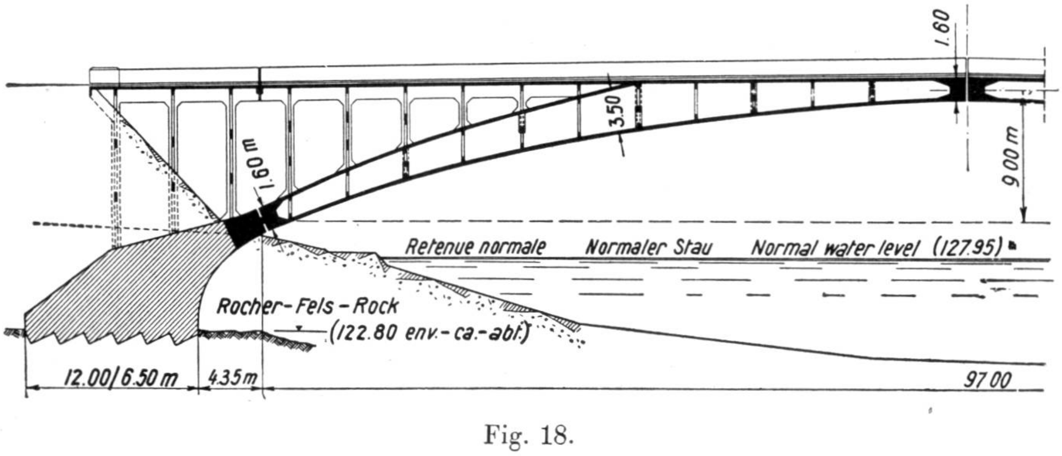

Fig. 18.

Bridge at Laifour. Longitudinal section.

to economy and appearance, the use of hollow webs and a limitation of their height is desirable. From thence, the arches have their own compression slab following the curve.

The author of the project has thus been able, in his opinion, to increase the stability of the arch of which the height at the quarter span joints amounts to 3.50 m, viz. approximately $^1/_{28}{}^{th}$ of the span. He has thereby avoided the indeterminateness of the position of the reactions within the width of arch of 0.42 m which it has been necessary to provide at the hinges.

An additional rise of 0.15 m has been provided, in anticipation of the further settlement under the influence of the phenomena discussed above.

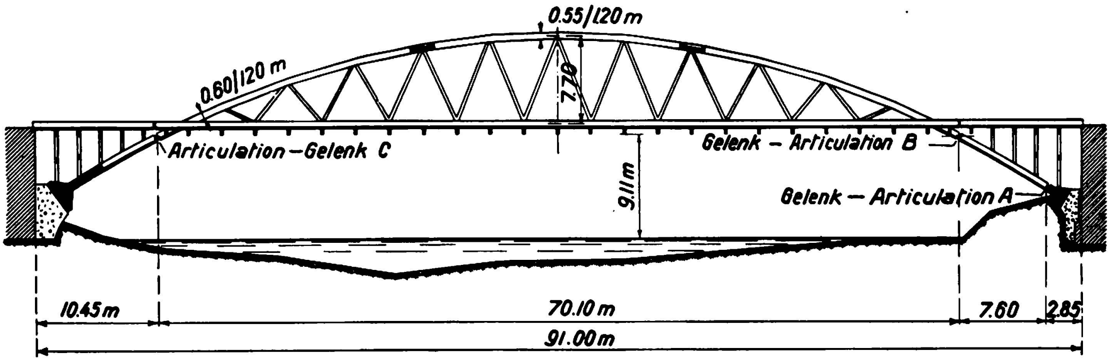

Fig. 19.

Bridge of 91 m span over River Lot at Port-d'Agrès. Three hinges under decking.

It is interesting also, to mention for the three-hinged arch system an arrangement which tends to bridge over large spans by means of parabolic lattice girders. This is made possible by the decrease and even the elimination of the stress in the tension boom, and particularly by the inclination of the reactions by means of an inclined socketed stanchion, placed underneath the roadway (Fig. 19). The 3 hinges are at A.B.C.

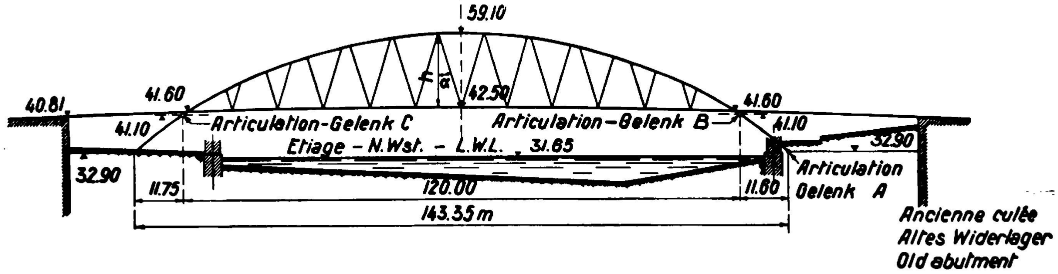

Fig. 20.

Bridge of 143.00 m span over River Lot at Castelmoron. Three hinges under decking.

The socketed stanchion AB fixes exactly the direction of the reaction on one side and consequently also that on the opposite side. The parabolical girder is treated exactly like a girder with vertical reaction, only with the difference that the stress in the tension boom is replaced by a thrust on the abutments. The lattice-work, the stresses of which are determined by the vertical component of the reactions, is exactly the same.

We have made the first application of this arrangement in 1925 for the construction of the bridge of 91.00 m span over the Lot at Port-d'Agres, the general arrangement of which is shown in Fig. 19. (Génie Civil of 18th Febr., 1928.)

The firma Christiani & Nielsen has recently carried out an even more important application with a span of 143.00 m over the same river at Gastelmoron. (Fig. 20 and 21) with the difference that the suspension struts are bare.

The suspension of the floor at the upper boom, by means of suspension struts, produces tensile stresses in the latter, which are curtailed by compressive stresses caused by the live load in its most unfavourable position. The constructors write that at the bridge of Castelmoron, the tension has always been preponderant so that never any of the suspension struts has ceased to work. At the bridge of Port-D'Agrès, compression subsisted in almost all suspension struts, but of such a small amount that all fear of lateral bending was excluded. The preponderance of tension must forcibly disappear with the increase of span because the compression due to the live loads is proportional to the span, whereas the tension due to the dead weight of the floor is practically constant.

The covering of long suspension struts in order to give them compressive strength, and the necessary arrangements to avoid their lateral bending, involve such complications, which demonstrate clearly, that the parabolic girders are not the most economical solution for large spans.

Due to a particular fact, they are prohibitive beyond a certain limit. As soon as the suspension struts are of great length and form a small angle between themselves, which is easily possible because the base of the triangulation is limited by the span of the longitudinal girders, the expansions and contractions of these bars under the influence of the live load provoke deformations which cannot be followed by top boom and the floor. The moments which result therefrom are of such an order that for spans of 150 m, if not below, preference will be given to the arch which is stable in itself without triangular connection with the roadway.

Recognizing the high degree which the internal hyperstatic character of the system may attain, the designers of the Bridge of Castelmoron have taken care that one of the two suspension struts of each knot may become inactive by the preponderance of compression. The utilization of the effect of the inclination of the remaining bars is interesting if the appearance is not hurt. The moments which are produced in an arch with straight hangers are reduced by the moments $\frac{Ph}{tga}$ produced by the horizontal component (Fig. 20).

PART TWO.

Construction.

The construction of large arched bridges is governed more and more by the study of the methods of execution, among which the scaffolding ranks first, for which without endangering the safety, economical arrangements have to be conceived if the selection of reinforced concrete is to be a judicious solution.

In a first chapter we are giving some details of recent scaffoldings and are describing at some length, due to its novelty that which has been used at La Roche-Guyon.

In a second chapter, we are explaining more briefly the methods of execution recently adopted.

1st. CHAPTER.

Scaffolding.

At all times, bridge designers have tended to treat as a work of art the scaffolding itself on which the elements of construction are placed prior to their acquiring the bearing capacity by their mutual reactions.

Examples are numerous of bridges in masonry where much science and art has been spent on the scaffolding.

In the same way this has to apply for reinforced concrete which has succeded masonry in the solution of larger problems.

In spite of the size of the structure, the Annals of Construction will never separate from the Bridge of Plougastel the reminiscence of the scaffolding which has served to its erection. Engineers will always find there one of the best examples for the application of means offered by nature and of the advantages

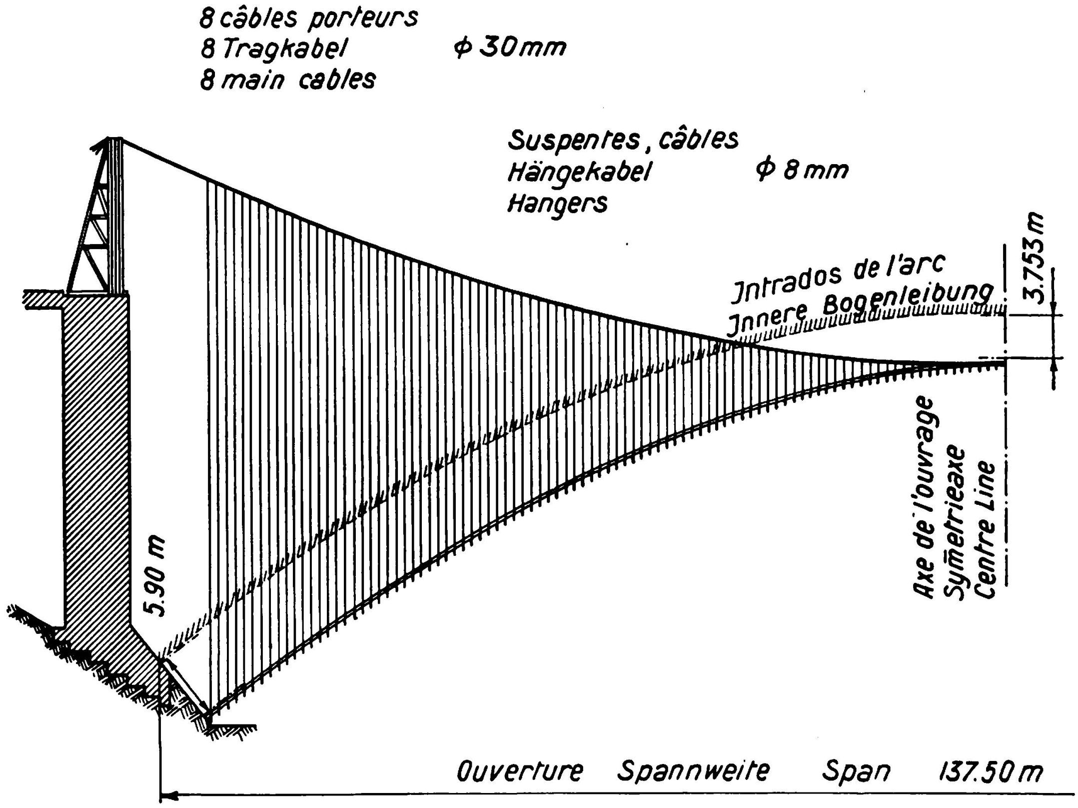

Fig. 21.

La Caille Bridge of 137.50 m span. Erection of false arch work with cables.

which a skilful designer may draw even from elements otherwise hostile to him. Those who have had to study the same project have been impressed by the movement of the tides in this estuary and by the apparent impossibility of supports between the two shores of about 600 m distance. Nevertheless, under the domination of the chief with which docility has the flood not lent itself to carry three times the scaffolding of 150 m span with rising tide, and then to place it again on its supports as the tide went down.

Powerful and expensive installations where necessary indeed, for the utilization of such large natural forces, but they were justified, as they could be used three times. Difficult and delicate manoeuvres had to be carried out and it was necessary to foresee everything in order to work successfully during the interval strictly determined by the tides. But the Enterprise *Limousin*, applying the *Freyssinet* methods, had proved already several times that it did not hesitate to face the most delicate problems of construction.

The situation of the Bridge of Usses (Haute-Savoie), called bridge of the Caille, also deserved that the special arrangements for its scaffolding were studied. The ravine about 150 m deep, did not offer any possibility for the support of the piles of the scaffolding, which necessitated a span of 140 m.

The arrangements studied by Engineer Caquot are a valuable precedent of the solution of constructing scaffoldings over free openings by the aid of suspended cables (Figs. 22 and 23).

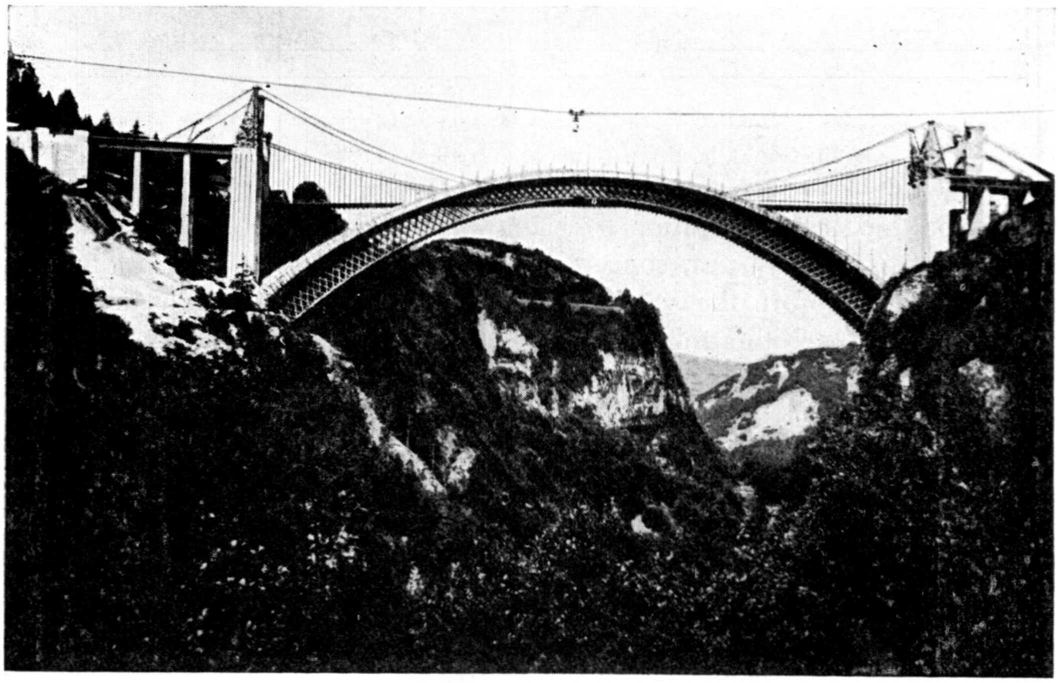

Fig. 22.
La Caille Bridge.

Following the assembly of the first elements of the supporting structure, the strengthening of the centering is made to the intended degree for supporting the first layer of concrete, similar as for a wooden bridge. The necessary rigidity is obtained by multiple framework connecting the booms.

In this solution, there is a judicious application of the suspension bridge in the construction of centerings for large structures; we have the impression that it will always present a favourable solution for large spans which are justly aimed at by reinforced concrete.

The study of the scaffolding of the bridge at La Roche-Guyon has led us to the same conclusions. Even the span of 161 m is the largest which has been bridged over by a type of arch with suspended roadway. This type will always be considered for spanning large rivers where it will rarely be possible to place below the roadway the rise of a large arch.

It would not have been judicous to renounce to supports in the river bed. However, we found the best possible solution by providing only for three piles in the river, spaced at 43.00 m from center to center.

We intended first to have these piles support only the weights of the roadway, of the scaffolding and of the centering, which, after completion of the assembly had to support by themselves the weight of the concrete arch (Figs. 24 and 25).

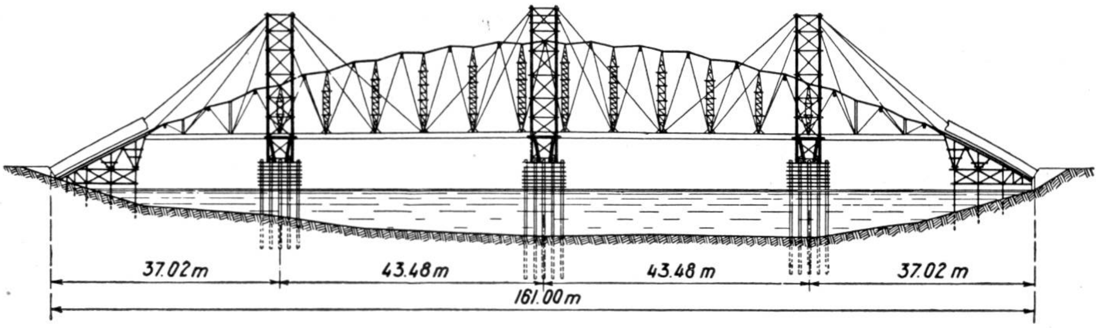

Fig. 23.
Bridge at la Roche-Guyon. Arrangement of staging.

However, for the first application over the largest navigable river of France, we did not want to employ our conceptions without previous verification and we have calculated therefore the scaffolding for supporting the whole dead weight, if by any circumstance this might have been necessary.

The sequence of operations is the following:

Erection of the piles;

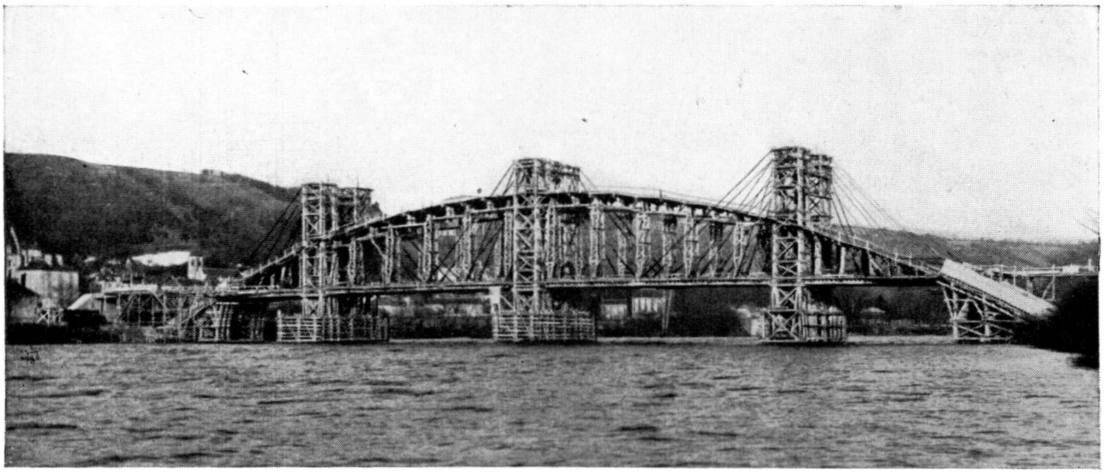

Fig. 24.
Bridge at la Roche-Guyon. View of false arch work.

Advancing the horizontal platform below the reinforced concrete roadway, to be constructed later, simultaneously to both sides of the piles, by using inclined ropes;

Assembly of the centering on this platform.

Previously, the lower parts of the arches up to and above the roadway had been executed by means of closely spaced supports of low height over the slope.

For light arches, whose average heigh of section is hardly $^1/_{80}{}^{th}$ of the span, it is essential to survey very accurately the neutral axis and to ascertain that there will be no deviation of it while carrying out the concrete work.

It was not sufficient to fill out with cement mortar the joints of the timbers, as has been done by Mr. *Caquot* at the Bridge of Usses; it was still necessary to be previously assured of the compression of timbers and of the tightening of the elements of assembly. This compression has been exercised by jacks at the crown.

The dead weight of the roadway and that of the centering to which the former is related allowed to apply a thrust of 280 t. We have given this thrust a maximum of 170 t in order to preserve a safe margin of stability.

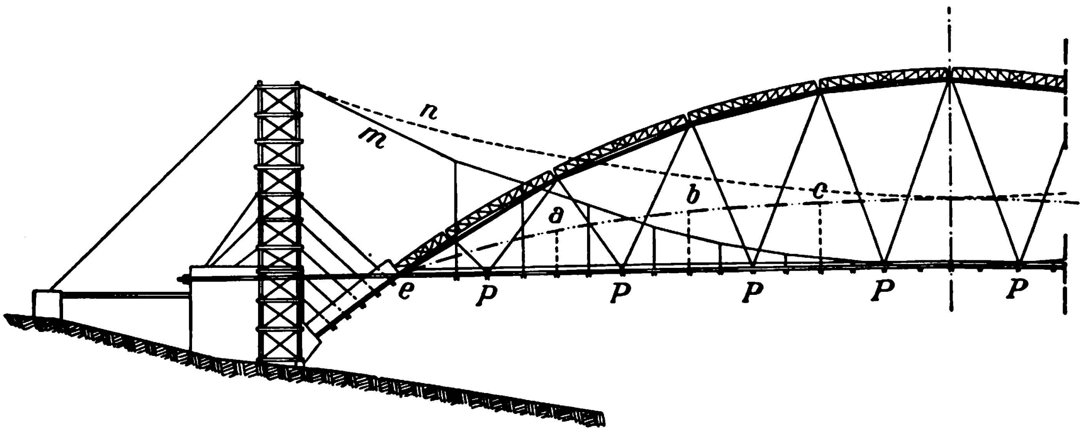

Fig. 25.
Erection of false arch work with cables.

The effect of this pressure is beneficial in several ways:

a) First, it produces compression in the timbers below the arches which surmount the tension, resulting from moments due to the considerable wind loads. We have thus been able to calculate the centering as a girder of 130 m span, between the intersections of the roadway with the arches, below which the full reinforced concrete soffit of the arch acts as wind bracing.

   The frame members of this girder were formed by ties crossing at 45 degrees. This saved us the trouble of calculating the piles for horizontal thrusts and enabled us to construct them lighter or even to omit them.

b) The braces connecting the centering with the roadway are stressed by tension, according to the weight of the roadway which the imposed thrust tends to lift. Fig. 22 shows that these tension members form with the centering and the platform a parabolic beam of 130.00 m span. They are made of round steel in order to expose the smallest possible surface to the wind; their inaptitude to resist to the least compressive stress demands that they be always stretched and that the initial stress be always larger than the compressive stress due to uneven concreting operations. Most frequently, the fraction of the weight of the roadway, which is to be carried will suffice; if not, additional ballast will be needed.

For reasons of precaution, due to very first application of this method, we have erected additional stays spaced 8.00 m apart which are capable of supporting and transmitting to the ropes, attached to the piles, the whole load, if this should be necessary. The previously applied compression permitted us to reduce the number of stays and thus to diminish the surface exposed to the wind. The line of the timbers is broken and forms projecting angles in the middle of the interval between stays. The previously applied compression caused forces directed upwards which are larger than the weight of arch of 5000 kg/m length.

The concave angle above the stays establishes a uniform compressive stress which necessitates strengthening of these stays. This is preferrable to doubling the number of stays with all their framework which would have offered a large surface to the wind, making it too difficult to establish stability with the horizontal forces, considering the height of 40 m above the point of fixation.

By this arrangement, the span of the timbers has been reduced to 4.00 m instead of 8.00 m and this has enabled us to construct these elements of current sections without using composed girders which, regarding the wind, would have had the same inconvenience as the augmentation of the stays.

Our assumptions have been well confirmed. The whole composition of centering, roadway and braces acted together as a girder of 130 m span. The maximum deformation observed during concrete operations has been 8 mm and yet it was unnecessary to concrete the arch by fragments, uniformly distributed over the length; the concrete-work has been done in a continuous manner, beginning at the two extremities and working up towards the crown. It is of importance to mention that we proceeded by successive elements, as will be seen further on.

Another advantage of the placing of jacks at the crown of the centering was that of lowering the latter by simple releasing of these jacks.

The results obtained in this first application show that it would be possible, as we had intended first, to reduce for large spans the number of piles and even to omit them completely, if the situation of the site permits to do so.

As we have pointed out, the Bridge of Caille is already an example of this reduction, whereby a cable has been used for the erection. If this method necessitates the erection of too high piles on the river banks, the centering can be put in place also with a smaller rise, as shown by the dotted line of Fig. 26, either by means of a platform suspended on a cable "m" or by means of a special cable "n". The centering is then lifted by compression, applied by jacks at the extremities "e". During this lifting operation, made possible by the hinges a, b, c the weight of the roadway gives the necessary stability for the maintenance of the regularity of the centering. It is sufficient to regulate the movement of these hinges a, b, c in proportion to the length of the ordinates of the centering. We do not intend to discuss this question further, which refers to mechanics and for which there are several practical solutions available.

At a bridge with roadway above, the regulation of the movement of the same points could be obtained by anchorages in the soil of the valley or in the bottom of the river bed.

During assembly operations, the compression of the centering will always be maintained sufficient by increasing the weights P, if necessary, to compensate the wind loads.

After completed assembly, the addition of the strengthening braces of the centering will establish rigidity instead of the hinges a, b, c.

The construction at La Roche-Guyon has proved that these ideas may be realized with all necessary security. We are convinced that large, rigid centerings can be constructed economically, opening large perspectives for the construction of large reinforced concrete structures.

2nd CHAPTER.

Methods of execution.

We are dealing here only with those methods which are important for the construction of large spans and which are in relation to the scaffolding, but we are not touching the improvements of the methods for the putting in place of reinforcement bars and concrete, which apply generally to all other structures.

For reasons of economy as well as for safety, the centerings should be loaded progressively, in a manner, improving their stability by the strength of the elements carried out first. For a long time, if not since the very beginning, masonry bridges have always been constructed by succesive layers. The low permissible stresses of this material enable the neglect of the increase of stress in the lower layers, due to the weight of the upper ones.

The bridge of Caille with its span of 140 m is a splendid example of this same process, because it means very much even to design a centering of this span for a load of nearly 13 t/m length, and it was advisable to include in the bearing system the lower boom of the rectangular section of the girder, forming the first ring. In order to exclude any increase of stress, the designers have applied a very ingenious idea of chief engineer Baticle. The concrete was poured between reinforced concrete key-stones, 0.18 m thick, prepared in advance, the mass of which was about one third of the total quantity. As in every layer only the poured concrete is subject to shrinkage, whereas the key-stones were pre-shrunk, it was sufficient to distribute them in the 3 layers, such as to provoke angular deformations by the shortening of the concrete rings, releasing the lower intrados. This method requiring an exact calculation has proved very efficient; no perturbation whatsoever has been noticed in this mass of concrete which was not reinforced.

Mr. *Freyssinet* has used the same method of concreting by layers at the Bridge of Plougastel. Compensation has been effected by jacks at the crown, after lowering of the centering of each arch. An estimate of the distribution of load between the scaffolding and the first layer of concrete has served to determine the contractions to be applied.

All those who are studying still larger problems, such as spans of 300.00 m or more, which produce stresses in the reinforced concrete, which can be faced easily at the present state of the manufacture and application of cement, will have to limit the cost of scaffolding, if this system is to be able to compete successfully with other solutions, particularly with suspension bridges. Without ignoring the merits of the specialists of this construction, it will be recognized that reinforced concrete represents the most judicious solution if the condition of the site allows an easy taking care of the thrust. According to our own opinion, this will only be possible by dividing the execution of large arches into separate elements for which the weight per linear meter be reduced to that amount, compatible with the assurance of cohesion between the successive elements and their own stiffness, prior to the joining.

By this we mean not only the division in separate layers, but also in elements obtained by vertical joints. This latter case is particularly apt for the splitting up of booms of large width, for plate-webbed arches as well as for trussed arches, under the supposition that the hight of section justifies the latter system.

By working with separate layers, the procedure is such, as to apply pressure onto them by means of jacks, simultaneously with the progressive lowering of the centering (first layer 1, then 1 and 2, then 1, 2 and 3 etc.). In this way, additional forces are introduced which can be compensated after complete lowering of the centering as it was done at the Bridge of La Roche-Guyon.

Construction by way of vertical separations does not require these compensations. At Roche-Guyon, we have used successfully methods of measurement (fixed marks and jacks) for controlling the influence of concreting by layers and the results of the compensations made.

This was necessary in view of the general application of this procedure for large spans; it was far from being superfluous as light arches with section of small height were concerned for which a displacement of the pressure line would have occasioned a considerable increase of the moreover high working stresses.

At every partial lowering of the centering, the forces produced by the jacks and the movements of the arch have been observed. At the final lowering of the centering the proper equilibrium of the forces of the free arch was established, by introduction at the crown of a thrust conventiently placed out of centre in order to obtain no displacement of the pressure line. In this way, "the neutral state" of the arch (without any appreciable bending moment) was reached i. e., the state for which all internal moments, due to the mutual influence of the separately loaded vaulting rings are exactly compensated; we have noticed a perfect concordance with the calculations and besides, with what degree of precision this procedure can be executed, because, once the neutral state being reached, the application of an additional thrust of only 1 t at the centre of gravity of the crown section is sufficient to produce an opening of the joint of 14.4 mm and a rise of the crown of 29 mm.

Finally, in all these investigations the coefficient of elasticity was determined without ambiguity; it was found to be from 2.10 to 2.2×10^6 t/m^2 which confirms the results obtained at Qued Chiffa.

For the Bridge at Castelmoron, mentioned above, Inspector General Mr. *Mesnager*, Consulting Engineer of Messrs. Christiani & Nielsen has reached the release of the centering in another way.

The concreting of the parabolic girder has been preceded by the placing, on the centering, of separate elements of a core, prepared in advance. For the section at the crown of 1.00 m $\times$ 1.20 m, the core section amounted to 0.55 m $\times$ 0.80 m.

We conceive that the designers have not been preoccupied by the increase of stress which has been inflicted on this element by the weight of the concrete, because the lateral reinforcing has given it additional strength. There is no inconvenience of the core of a cross section being stressed higher, since it is contracted by the material which envelopes it; furthermore, the increase of stress does not correspond to the whole weight of the concrete, as the centering and the core were acting together.

Conclusions.

I. — *With regard to the strength of material:*

Even at the present state of the manufacture of cements and the current methods of mixing concrete, by observing the rules and regulations governing granulometry and vibration, it is possible to consider spans up to 800 m for reinforced concrete bridges.

II. — *With regard to calculations:*

The present methods are precise enough to obtain for the whole length of the arch the maximum ligthness, compatible with the stability for which a diagram similar to that of Fig. 10 will give the size. It will be easy to verify the stability by multyplying the live load by an increasing coefficient: for example 2, and by ascertaining that the pressure line remains inside of the core, whereby the reinforcements are stressed only to the permissible limit.

The degree of indeterminateness of the coefficient of elasticity is low by the use of vibrated concrete of good granulometric composition.

The calculations are very rapid, due to the rules of similitude, and the tentative tests at the beginning are abolished, since a simple rule allows the determination of the average section for any working coefficient adopted.

III. — *With regard to construction:*

The realization of large spans finds its greatest difficulty in the study of the scaffoldings and their assembly. The relative expenditures can be kept in a non-prohibitive limit by a reduction of weight, to be applied to the scaffolding. This is obtained by the concreting in layers, for which the centering is lowered successively by the means of jacks. Verification has been made of the possibility of rigorously compensating, for a plate-webbed arch, the influence of a layer of arch on the one below. This compensation is not necessary by concreting in sections, devided by vertical joints.

IV. — The comparison of the arch, whose moments of inertia decrease from crown to springings with arches, whose moments of inertia are constant and with arches, whose moments of inertia increase from crown to springings, is to the advantage of the first mentioned: especially for large rise-span ratios, due to the small average section and the reduced thrust on the abutments.

The application of statically determined systems for large spans is not favourable, because of the difficulties of the hinges. With regard to a particular arrangement, made possible by the position of the hinges, we may add that for spans beyond 150 m, it is no longer advisable to compensate the bending moments by a triangular framework between the arches and the roadway; spans of this size should be bridged over by arches stable in themselves, either plate-webbed or trussed.

V. — It is economical to arrive at high strengths by mean of lateral reinforcement. This principle may be applied to its extreme limit for large spans; the longitudinal reinforcement is judiciously supressed or simply reduced to tie rods for the spiral reinforcements and the stirrups.

At the Bridge of La Roche-Guyon, their percentage weight has already been reduced to 0.5, that of the spiral reinforcements and the stirrups together being 1.3.

This reduction will dissipate all fears as to the increased compressive stress of the reinforcement, due to shrinkage and to the influence of the slow compressions of concrete, not fully known up to now.

A capital advantage of the lateral reinforcement is the high uniformity of strength.

Summary.

The first part of the Author's paper refers to the calculation of wide span reinforced concrete bridges. Various factors are to be considered for statically indeterminate arches, such as: the veriation of the moment of inertia, the shape of cross section, the span-rise ratio and the working stress. The Author studies the limits of spans and gives a comparison between different arches and the bridge of Roche-Guyon of which the results of calculation are stated. He describes further a number of three-hinged arch bridges recently built in France.

The second part of the paper deals with the execution and the false arch work of bridges, in particular the false arch work employed for the Roche-Guyon bridge. He gives a short description of systems recently employed.