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The performance of small prestressed concrete units

Ueber die Ausführung kleiner Bauteile aus vorgespanntem Beton

Comportamento de pequenos elementos de betão preesforçado

Comportement des petits éléments en béton précontraint

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Introduction

The design and manufacture of prestressed concrete units is now widely practised and conforms to generally accepted procedures, which, in most countries, are governed by specifications, codes of practice or other authoritative recommendations. These specifications and recommendations have been based on experience and research over a number of years and the purpose of this paper is to extend this background of information relating to the performance of small prestressed concrete units with pretensioned steel.

The structural design of these units is usually required to ensure that cracking of the concrete is avoided under normal working loads and that collapse will not occur under abnormal loading. The first requirement is met by limiting the stresses in the materials arising from the prestress and from normal combinations of the prestress and the working loads after making the necessary allowances for losses of prestress. Security against collapse is obtained by adopting an adequate load factor for the ratio of the total failing load to the total working load for the unit. The results obtained in the investigation described in this paper have a direct bearing on the calculation of cracking and failing moments, and provide some information on variability in behaviour, which may influence the choice of load factor and the technique used in manufacture.

The investigation was carried out in two distinct phases. In the first instance, a number of routine tests were made on several different types of floor joist, in which behaviour under static loading was examined and data were obtained on variations in production. Some of this work was summarised by Masterman [1] in a review of developments in prestressed concrete sponsored by the Ministry of Works; some new and more detailed information on these tests is now given. In the second phase, several series of experments have been made on lines of enquiry suggested in a reassessment of the previous work, and this information is presented for the first time.

Behaviour of the units under load

Three types of prestressed concrete floor joist were tested; each was 12 ft. 9 in. long with the sectional dimensions given in Fig. 1. Some information on their design and manufacture is shown in Table 1. The units were made by the «long-line» system of pretensioning, and release of the wires was effected at between 4 and 6 days after casting. Most of the tests were carried out when the units were between 3 and 9 weeks old, but a few joists were as old as 18 weeks.

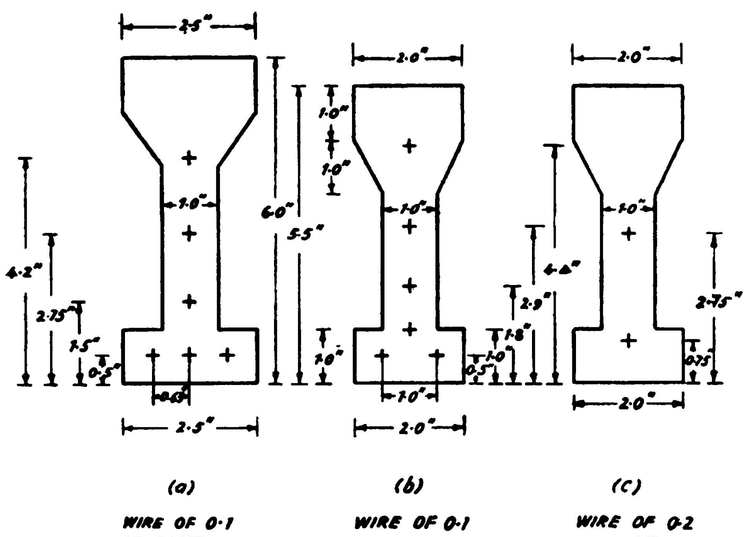

FIG. 1

The joists were tested on a span of 12 ft. 4 $^1/_2$ in. with quarterpoint loading deflections were recorded at mid-span and in some cases the strains in the concrete were measured on gauge lengths of 10 in. or 30 in. at all stages of loading to failure. Each unit exhibited the characteristic behaviour of prestressed concrete under load. For normal working loads, the relationship between deflection and applied load was linear; at higher loading cracking commenced, and the deflection increased at a disproportionately greater rate with increasing load until failure occurred (as shown in Fig. 2). A summary of the results for the total moment at the appearance of the first cracks and at failure is given in Table 2 together with the strengths of the materials at the time of test.

It was found that the total moment at the appearance of the first cracks could be calculated with reasonable agreement with the experimental results when the data in Table 3 were assumed.

TABLE 1

Design and manufacturing details for the floor joists

Type of joist	(a) 6" × 2 1/2"	(b) 5 1/2" × 2"	(c) 5 1/2" × 2"
Diam. of wire: in.	0.104	0.104	0.200
Initial stress in the wires: lb. per sq. in....	182,000	182,000	154,000
Mix proportions by weight of rapid-hardening Portland cement and combined sand and gravel aggregate	1 to 4 1/2	1 to 4 1/2 (2)	1 to 4
Water/cement ratio	0.42 to 0.54	0.45 (2)	0.37
Estimated maximum precompression in the concrete: lb. per sq. in.	1760	1940	2050

(2) Of the 65 joists tested of this type, two were made of concrete with proportions of 1 to 3.5 and a water/cement ratio 0.42 and six were made with concrete with proportions of 1 to 4 and water/cement ratios between 0.41 and 0.45.

TABLE 2

Type of joist	(a) 6" × 2 1/2"	(b) 5 1/2" × 2"	(c) 5 1/2" × 2"
Design moment: lb. in.	21,000	16,000	16,000
Total moment for first cracks: lb. in.			
Mean	37,600	27,500	26,700
Max.	39,600	30,700	30,500
Min.	34,700	22,800	23,500
Coefficient of variation: per cent	—	6.1	—
Total moment at failure: lb. in.			
Mean	63,000	47,800	44,600
Max.	67,600	57,600	49,300
Min.	57,300	35,200	39,300
Coefficient of variation: per cent	—	9.1	—
Crushing strength of 6 in. concrete cubes at the time of test: lb. per sq. in.			
Mean	7,570	7,630	8,300
Max.	9,600	11,400	9,500
Min.	6,050	5,410	8,000
Coefficient of variation: per cent	—	16.3	—
Tensile strength of steel: lb. per sq. in.			
Mean	319,000	326,000	231,000
Max.	325,000	327,000	233,000
Min.	314,000	318,000	229,000
Ultimate mode of failure of the joists	Fracture of the wires or crushing of the concrete	Crushing or shear of the concrete	Crushing of the concrete

TABLE 3

Data assumed for the calculation of cracking moment for the tests

Relaxation of the steel (over-stressed by 5 per cent for 2 minutes)...	5 per cent of initial stress
Modulus of elasticity of the concrete at transfer: lb. per sq. in.	5×10^{6}
Shrinkage of the concrete	150×10^{6}
Creep of the concrete: per lb. per sq. in.	0.2×10^{-6}
Bending tensile strength of the concrete: lb. per sq. in....	800

These figures for the creep and shrinkage of the concrete are half those recommended for pretensioning in the «First Report on Prestressed Concrete», of the Institution of Structural Engineers, [2] and when the losses calculated from these figures are combined with the losses due to the elastic contraction of the concrete and creep of the steel, a total loss of about 17 per cent is obtained for the joists of the type shown in Fig. 1-b. If these joists remain unloaded throughout their life, the loss of prestress may be expected to increase from 17 per cent to between 25 and 30 per cent, and this was confirmed by tests described later in the paper.

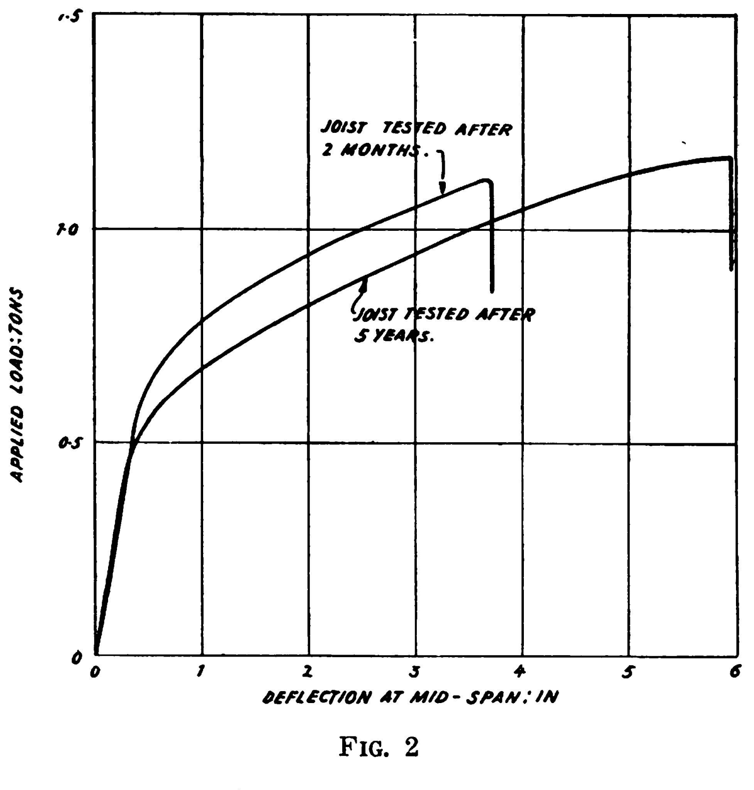

FIG. 2

The calculation of the failing moment, which was based on the following assumptions, gave good agreement with the experimental results. It was assumed that. —

a) the distribution of compressive stress in the concrete was uniform across the compression zone with a value of 0.6 times the cube strength of the concrete;

b) the wires below the neutral axis were stressed to their tensile strength;

c) the maximum compressive strain that could be sustained by the concrete was 0.2 per cent (from this assumption the stress in any wires above the neutral axis at failure could be calculated).

This method of calculation which is very similar to that originally proposed by Abeles [3], gave results for the three types of joists in Figs. 1-a, 1-b and 1-c that under-estimated the failing moments by 8 per cent, 3 per cent and 2 per cent respectively on average for each type.

The maximum compressive strains in the concrete recorded at failure varied between 0.21 and 0.39 per cent, but much of this variation developed during the final phases of the tests, as, at 95 per cent of the failing moment, the maximum recorded strains were between 0.18 and 0.22 per cent. The assumption of a value of 0.2 per cent for estimating the stress in any steel in the compression zone at failure would therefore appear reasonable.

Variations in cracking and failing moments

The results of the tests on the joists of the type shown in Fig. 1-b were sufficient in number to obtain useful information on the likely variations in behaviour of small prestressed concrete units arising from variations in their production. The sixty-five joists tested were drawn from seven different batches each of sixteen joists. In each batch, the joists were cast in timber moulds in groups of four for each concrete mix, and concrete cubes were also cast at the same time as each of the joists for test with the joists. Samples of the steel were also tested. Information was, therefore, available on the cracking and failing moments and on the strength of the materials for each joist tested. The results are to some extent heterogeneous as in a few instances the concrete mix proportions were varied, a few joists were as old as eighteen weeks and from one to sixteen joists were selected from each batch for test. For simplicity, the results have been treated as being drawn from a normal population and the effects of several of the variables in manufacture and test were examined experimentally in further tests which are discussed later.

The coefficients of variation for cracking and failing moment and for cube strength are given in Table 2; the variation in the strength of the steel was very small and is ignored in the examination of these results. Histograms for cracking and failing moment are shown in Fig. 3 and for cube strength in Fig. 4. It may be observed that variation of cracking moment is less than that for failing moment. The coefficient of variation for cube strength of 16.3 per cent is higher than would normally be erpected for good factory production, but this may be explained by the variations in the age and mix proportions of the concrete which have already been mentioned.

The regression of cracking moment (M_c) on cube strength (C_u) was derived and is given by the following equation, which shows a highly significant correlation: —

$$M_c = 0.78\ C_u + 21{,}600 \text{ (lb.in. units)} \tag{1}$$

The form of this expression is such that the whole of the influence of cube strength on the cracking moment can be explained by its influence on the bending tensile strength of the concrete. It must then be assumed that the bending tensile strength is 0.084 C_u, which is equivalent to the tensile stress resulting from the application of a bending moment of 0.78 C_u.

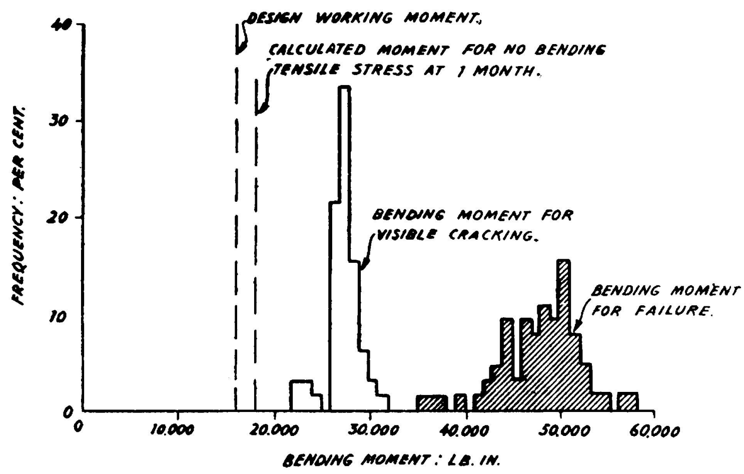

FIG. 3

Two apparently distinct types of failure were experienced in the tests on this particular type of joist: about two-thirds of the joists failed in shear near one of the load points and the remainder by crushing of the concrete near mid-span. A series of subsidiary tests were made on similar joists with central loading on spans of 1 ft., 2 ft., 3 ft., 4 ft., 5 ft., and 7 ft. to determine the effects of the shear conditions on ultimate strength. The results of these tests are shown in Fig. 5, from which it may be seen that the shear conditions did not affect the failing moment when the span was 4 ft. or more, which is equivalent to a span of 8 ft. for quarter-point loading. It would, therefore, appear that in the main tests shear failure only developed at maximum load when crushing of the concrete was imminent, and its appearance in the tests may be neglected in the analysis.

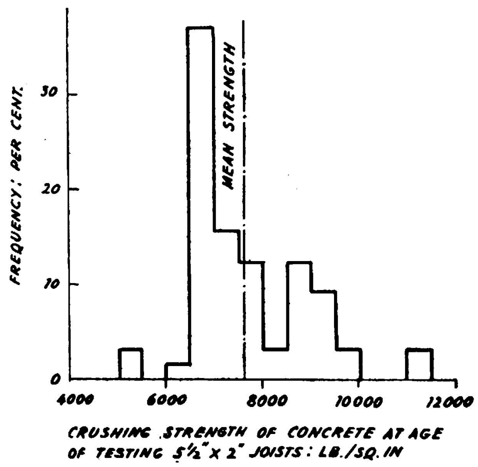

FIG. 4

The regression of failing moment (M_u) on cube strength is given by the following relationship, which also shows a higly significant correlation: —

$$M_u = 1.69\ C_u + 34{,}900 \text{ (lb. in. units)} \qquad (2)$$

from which it may be inferred that the behaviour of the steel has a greater influence on the failing moment than the cube strength of the concrete.

The effect of dimensional errors

The analysis of variance, from which expressions (1) and (2) were derived, are summarised in Table 4, where it may be observed that the residual variances are high with respect to the total variance. Measurements of the dimensions of twelve joists from one batch showed that there were considerable dimensional errors arising from the repeated use of timber moulds. Analysis indicated that the variance in cracking moment and in failing moment for these twelve joists due to these dimensional errors accounted for much of the total variance in each case as shown in Table 4. It is of interest to note that, had the coefficient of variation for cube strength been very much less than the experimental value of 16.3 per cent, it is unlikely that the coefficients of variation for cracking and failing moments could have been less than 5.1 and 8.0 per cent respectively as compared with the experimental values of 6.1 and 9.1 per cent respectively. For the manufacturing conditions under which these joists were produced, greater uniformity could have been obtained by reducing the dimensional errors than by increasing the rigidity of the control of concrete quality. For uniformity in the behaviour of small prestressed concrete units such as these, the use of steel moulds would seem to be essential.

TABLE 4

Effects of variations in cube strength and dimensional errors on variations in cracking and failing moment

Series for the examination of	Effects of variations in cube strength	Effects of dimensional errors
Number of tests in series	65	12
Cracking moment		
(3) Variance of cracking moment	2.8	2.8
Variance due to variations in the quality of the concrete as indicated by the strength of concrete cubes	0.9	
Variance due to dimensional errors ...		1.6
Residual variance...	1.9	1.2
Failing moment		
Variance of failing moment...	19.0	14.1
Variance due to variations in the quality of the concrete as indicated by the strength of concrete cubes	4.5	
Variance due to dimensional errors ...		6.5
Residual variance...	14.5	7.6

Variations in the quality of materials

It was not known to what extent the strength of a concrete cube represented the strength of the concrete in a joist and what variation in strength occurred within the length of the joist. A series of concrete specimens each 12 in. long with a section of 3 in. by 3 in. were made

(3) The units in which variance is expressed in the above table are $(\text{lb. in})^2 \times 10^6$.

in moulds which were of about the same size as those used for the joists. These moulds were made for casting 16 beams of 12 ft. in length in groups of four and were sub-divided to produce eleven specimens in each beam mould, that is a total of 176 specimens. The moulds were mounted on the prestressing bed and filled with concrete, which was consolidated with two shutter-vibrators fixed to each group of four beam moulds in the same manner as for the joists. At or about the age of 28 days, the specimens were tested as beams to obtain the modulus of rupture and the broken halves were then tested between 3 in. by 3 in. steel plates to obtain the crushing strength. Control specimens each 16 in. long with a section of 4 in. by 4 in. were cast in steel moulds and tested in a similar manner. The results of these tests are show graphically in Figs. 6-a and 6-b.

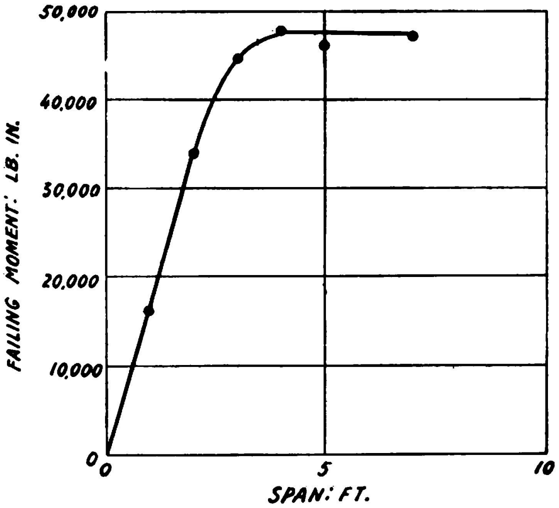

Fig. 5

Analysis of the results indicated that, apart from the differences between mixes, the most significant influence on modulus of rupture and crushing strength was a positional effect which may have been related to the poosition of specimen with respect to the vibrators; specimens at the ends of each beam mould gave low strengths. The coefficients of variation for modulus of rupture and for crushing strength calculated from the mean variances for the specimens in each beam mould were 9.6 and 5.1 per cent respectively, which show that the variation in the strength of the concrete within each joist may have been considerable and would account for part of the residual variances for cracking and failing moments.

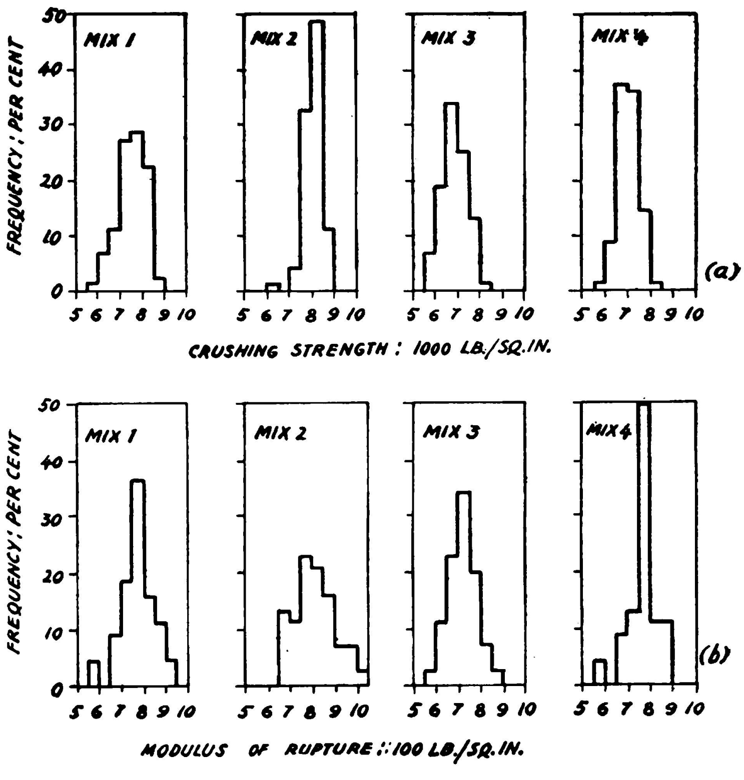

Fig. 6

The mean values for the modulus of rupture for the control specimens and for those cast on the prestressing bed were 781 and 767 lb. per sq. in. respectively. The corresponding values for crushing strength were 6270 and 7400 lb. per sq. in. respectively. These results suggest that the mean crushing strength of the concrete in the joists may have been higher than that of the concrete cubes; as a result, however, of the variation within the joist, this advantage was not reflected in the results for failing moment, except for one or two results that cannot be explained in any other way.

The variation in the quality of the steel in the sixty-five joists tested was too small to be significant in the examination of variations of cracking and failing moments. To obtain some information on the effects of variations in the strength of the steel, a series of eight joists of the type illustrated in Fig. 1-b were made in pairs in a steel mould by the «individual mould» system of pretensioning. For consolidation of the concrete the mould was clamped to a vibrating table. The strength of the concrete and the steel was deliberately varied. Transfer was effected after 5 days and the joists were tested when they were about 28 days old. The mean results for each pair of joists are given in Table 5 with the strengths of the materials. The average difference between the individual results in each pair was 0.6 per cent and the maximum difference was 1.2 per cent.

TABLE 5

Effect of variations in the materials on failing moment

Tensile strength of steel; lb. per sq. in.	Crushing strength	Failing moment of joists; lb. in.	
		Experimental	Calculated
286,000	7,300	44,100	43,400
312,000	8,530	47,500	47,700
273,000	6,430	40,100	40,400
326,000	6,660	43,200	43,100

The regression equation for failing moment on tensile streng th of the steel and cube strength of the concrete was as follows: —

$$M_u = 0.037\ t_u + 2.76\ C_u + 12{,}500 \text{ (lb. in. units)} \qquad (3)$$

This relationship gave a less significant correlation than was obtained for expressions (1) and (2), the correlation being significant at between the 1 and 2 per cent levels. The failing moments calculated from the regression equation are also shown in Table 5. The residual variance for failing moment of these joists was 1.70×10^6 (lb. in. units), which was a very considerable reduction when compared with the residual variances for failing moment in Table 4. It confirms that the use of steel moulds leads to greater uniformity, and also suggests that consolidation of the concrete on the vibration table in the «individual mould» system is superior to the use of fixed shutter-vibrators on the «long-line» system as used in the production of the prestressed concrete joists.

If the failing moment (or the cracking moment) is expressed in terms of the strengths of the materials as in expression (3) in the form: —

$$M_u = X \cdot t_u + Y \cdot C_u + Z \qquad (4)$$

then the variance of failing moment can be expressed in terms of the variances of the strengths of the materials and a residual variance: —

$$\sigma^2_{M_u} = X^2 \, \sigma^2_{t_u} + Y^2 \, \sigma^2_{C_u} + \sigma^2_r \qquad (5)$$

where, σ^2_r is the residual variance of failing moment resulting from variations in production technique. This analysis of variance could be used to simplify the quality control of the manufacture of prestressed concrete units, which is often checked by loading tests on individual units, by indicating the relative importance of the different causes of variation in the strength of the product. If the regression equation at (3) is used to relate the variance in failing moment with the variances in the strengths of the materials, the following relationship is obtained: —

$$\sigma^2_{M_n} = 0.00140 \, \sigma^2_{t_u} + 7.62 \, \sigma^2_{C_u} + 1.70 \times 10^6 \qquad (6)$$

The analysis given here is little more than an example of the method as the different strengths of the materials were obtained artificially and not as a result of sampling from production. It could be shown, however, from (6) that variations in the strength of the steel had a smaller effect on the variation of ultimate strength than variations in the strength of the concrete for this method of manufacture. Under well controlled conditions, the coefficient of variation for the steel is unlikely to exceed 3 per cent and the coefficient of variation for the concrete should not be greater than 10 per cent; expression (6) then leads to the finding that the coefficient of variation for failing moment should not exceed 5 per cent. If greater uniformity were required it could most easily be obtained in this case by improving the control of concrete quality. This method of analysis might also be applied to variations in cracking moment; it was not done in this instance as the data are insufficient to give significant results.

The effects of the age of the concrete at transfer

In the manufacture of the joists in the earlier phases of the investigation, transfer of the prestress to the concrete was effected at between 4 and 6 days after casting, but no information was available to show the effects of earlier or later imposition of the prestress. Several series of joists of the type shown in Fig. 1-b were made in pairs in a steel mould by the «individual mould» system and release of the wires was effected at different ages. All the joists were tested in the same manner as before, 28 days after casting. The results for one of these series of tests are shown in Fig. 7, where each result for cracking and failing moments represents the mean for two joists. The tensile strength of the steel used in these tests was 286,000 lb. per sq. in.

The age of the concrete at transfer did not have important influence on the observed cracking moment, but this moment was slightly lower when transfer was effected at one day than when it was effected at greater ages. The strength of concrete after one day was between 3,000 and 3,500 lb. per sq. in., which was little more than $1^1/_2$ times the maximum precompression in the concrete. Once the cube strength was more than twice the maximum precompression, the age of the concrete at transfer did not appear to affect the cracking moment, and although greater creep losses would have been expected for the earlier ages they were not excessive.

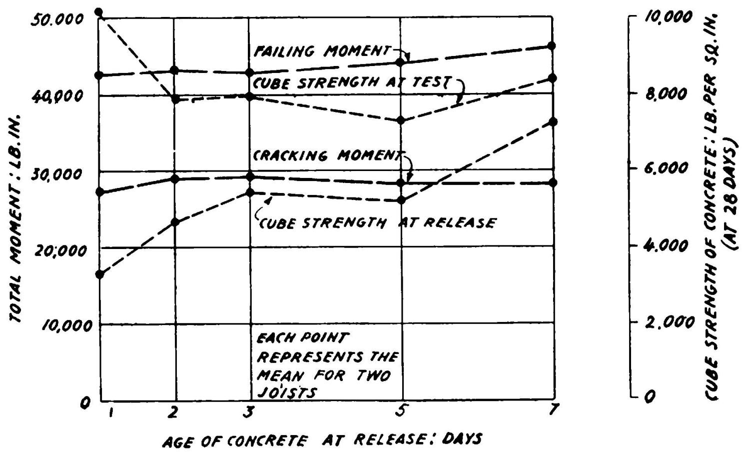

FIG. 7

Within the limits for the age of the concrete at transfer which were examined in these tests, an increase in age at transfer caused a small but distinct increase in the failing moment that was independent of the cube strength of the concrete at the age of test.. All the joists in this series failed finally by crushing of the concrete.

The effects of the age of unit at test

The results of tests on a series of joists of the type shown in Fig 1-b, which were carried out after different periods of storage up to 3 months, were reported by Masterman [1]. Since then further tests have been made on joists from this same series' after periods of 1 year and 5 years of storage in the laboratory. The joists were tested in pairs on a clear span of 12 ft. and were supported at each end on brick piers. Uniform loading of dead weights was applied to the joists through a flexible timber platform. The results of the full series of tests are illustrated in Fig. 8.

A single joist from one of the batches included in the series of sixty-five tests was tested at the age of five years under quarter-point loading on a span of 12 ft. $4\,^1/_2$ in. The result of this test is shown in Fig. 2 in comparison with that for a similar joist tested at the age of 2 months.

The results show that the age of the joists has little effect on the failing moment. The cracking moment, however, appears to increase with age during the first few weeks after stressing, and later decreases after further ageing. The results are insufficient to be conclusive but suggest that the overall loss of prestress for this type of unit may be as much as 30 per cent when they are stored in an unloaded condition for a long period.

Conclusions

The following conclusions were drawn in relation to the performance of small prestressed concrete units with pretensioned steel as a result of the investigation: —

a) Cracking and failing moments for the joists could be calculated with reasonable accuracy on the basis of simple assumptions;

b) Variations in the behaviour of the units may be much greater than those arising from variations in the qualities of the materials, unless particular care is taken to ensure their dimensional accuracy; in this connection, the use of steel moulds is desirable;

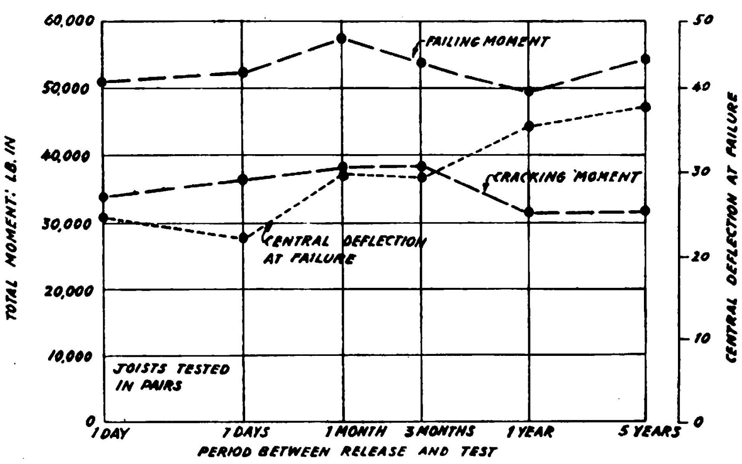

FIG. 8

c) Variations in the cube strength could be related to the consequent variations in the cracking and failing moments;

d) Analysis of the variations in failing moment arising from variations in the strengths of the materials offered a method of facilitating the control of the quality of the product;

e) The age of the concrete at transfer had little effect on subsequent behaviour when the cube strength of the concrete at this stage was greater than twice the maximum precompression in the concrete;

f) The age of the unit at the time of test had little influence on the failing moment, but after 5 years the cracking moment was considerably reduced.

REFERENCES

1. Prestressed concrete developments at the Field Test Unit. Thatched Barn. O. J. Masterman. Struct. Eng., 1950, 28 (10), 246-64, and 1951, 29 (8), 221-6.
2. First report on prestressed concrete; Institution of Structural Engineers, London, September 1951.
3. The use of high strength steel in ordinary reinforced and prestressed concrete beams. P. W. Abeles. International Association for Bridge and Structural Engineering, 4th. Congress, Preliminary Publication, 1952.

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SUMMARY

The paper gives some details of a laboratory investigation of the properties of small prestressed concrete beams, which was based on an examination of their behaviour under static loading. Several different types of unit were tested to failure in routine checks of production on a pilot plant scale. In some of the tests, measurements of the strain in the concrete were made to obtain further information on the process of the development of failure.

One type of unit was tested in sufficient numbers for a study to be made of the sources of variation in behaviour. Significant correlations were established between the strength of the concrete and the bending moment at which cracks appeared and between the strength of the concrete and the bending moment at failure; from these, the effect of variations in the strength of the concrete on the variations in performance was obtained. Experimental examinations of other causes of variations were made to determine the effect on subsequent behaviour of such factors as: —

I variations in the dimensions of the units;

II variations in the quality of the concrete arising from casting techniques;

III variations in the quality of the prestressing steel;

IV the age of the concrete at the time of transfer;

V the age of the unit at the time of test.

These sources of variation are discussed in relation to the factory production of prestressed concrete members generally, and a method of examining the different sources of variation in suggested.

ZUSAMMENFASSUNG

Die Arbeit enthält einige Ausführungen zu einer Laboratoriumsuntersuchung über die Eigenschaften kleiner Balken aus vorgespanntem Beton. Die Untersuchung bezog sich auf das Verhalten bei ruhender Belastung. Verschiedenartige Balkentypen wurden unter laufender Registrierung des Vorgangs bis zum Bruch beansprucht. Bei einzelnen Untersuchungen Es wurden auch folgende Einflüsse untersucht, um die Verschiedenwurden Spannungsmessungen am Beton vorgenommen, um weitere Aufschlüsse über die Bruchbildung zu erhalten.

Von einem Prototyp wurde eine grössere Anzahl untersucht, um die Ursachen für das verschiedenartige Verhalten ermitteln zu können. Es war möglich, bedeutsame Beziehungen zwischen der Betonfestigkeit und den Biegungsmomenten bei der Rissebildung und beim Bruch festzustellen; daraus liessen sich die Auswirkungen aus der Aenderung der Betonfestigkeit auf das Verhalten der Bauteile übertragen.

Es wurden auch folgende Einflüsse untersucht, um die Verschiedenartigkeit des Verhaltens zu erfassen:

I. Aenderung der äussern Abmessungen.

II. Veränderung der Betonqualität im Zusammenhang mit der Betonherstellung.

III. Aenderung der Qualität des Vorspannstahls.

IV. Berücksichtigung des Betonalters beim Aufbringen der Vorspannung.

V. Berücksichtigung des Alters der Betonbalken zur Zeit der Untersuchung.

Diese Ursachen des unterschiedlichen Verhaltens werden im Zusammenhang mit der fabrikmässigen Herstellung vorgespannter Betonteile besprochen; ferner schlägt der Verfasser ein Verfahren vor, mit welchem die verschiedenen Ursachen der Festigkeitsunterschiede geprüft werden können.

RESUMO

O autor dá alguns pormenores acerca de um estudo experimental das propriedades de pequenas vigas de betão preesforçado, baseado no exame do seu comportamento sob cargas estáticas. Ensaiaram-se à rotura vários tipos diferentes de vigas por ocasião de verificações de rotina periódicas de produção de uma fábrica experimental. Em alguns casos mediram-se as deformações do betão de modo a obter pormenores mais completos acerca do modo de rotura.

Um dos tipos de viga foi ensaiado bastantes vezes para se poder elaborar um estudo sobre as causas das diferenças de comportamento que se constataram. Estabeleceram-se relações interessantes entre a resistência do betão e o valor do momento flector correspondente à fissuração por um lado, e entre a resistência do betão e o momento flector correspondente à rotura por outro; destas relações deduziu-se o efeito das variações de resistência do betão sobre as irregularidades do comportamento. Efectuou-se um estudo experimental sobre as outras causas de variação, de modo a determinar o efeito dos factores seguintes sobre o comportamento das vigas:

I Variações das dimensões das vigas;
II Variações de qualidade do betão provenientes das diferentes técnicas de betonagem;
III Variações de qualidade do aço das armaduras;
IV Idade do betão quando da tensão dos cabos;
V Idade da viga quando do ensaio.

O autor discute estas causas de irregularidade em relação às condições de produção em oficina de vigas em betão preesforçado e sugere um método de estudo para as várias causas.

RÉSUMÉ

L'auteur donne quelques détails sur une recherche expérimentale concernant les propriétés de petites poutres en béton précontraint, fondée sur une étude de leur comportement sous des charges satiques. Plusieurs types différents de poutres ont été soumis à des essais de rupture lors de vérifications de routine périodiques de la production d'une usine-pilote. Au cours de certains de ces essais on a mesuré les déformations du béton de manière à étudier de plus près le processus de rupture.

L'un des types de poutre a fait l'object d'un nombre d'essais suffisamment grand pour permettre d'étudier les causes de différences de comportement que l'on avait observées. L'on a établi des relations intéressantes entre la résistance du béton et le moment fléchissant correspondant à la fissuration d'une part, et entre la résistance du béton et le moment fléchissant correspondant à la rupture d'autre part; ces relations ont permis de determiner l'effet des variations de résistance du béton sur l'irrégularité du comportement. Les autres causes d'irrégularité ont également été étudiées de manière à déterminer l'effet des facteurs suivants sur le comportement des poutres:

I Variations des dimensions de la poutre;
II Variations de la qualité du béton dues à des techniques de bétonnage différentes;
III Variations de qualité des aciers de l'armature;
IV L'âge du béton lors de la mise en précontrainte;
V L'âge de la poutre lors de l'essai.

L'auteur discute les causes de variation par rapport aux conditions de production en atelier de poutres en béton précontraint et propose une méthode d'examen de ces différents causes.

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VI a 2

Cuidados a ter na construção das grandes pontes de betão armado

Précautions à prendre au cours de l'exécution des ponts en béton armé de grandes dimensions

Practice of large reinforced concrete bridge construction

Die Berechnung von grossen Brücken in Eisenbeton

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1 – INTRODUÇÃO

Dum modo geral o engenheiro projectista, quer demasiado influenciado pelas operações analíticas ou por processos especiais de cálculo, quer pela sua pouca experiência da construção, dimensiona as obras que concebe sem ter em conta alguns dos importantes esforços que se geram durante as diferentes fases de execução, como se elas fossem por magia construídas de um jacto e só entrassem em funcionamento depois de completamente prontas e descimbradas.

Por outro lado, o empreiteiro, por justificados motivos de interesse material, tem a natural tendência de levar àvante da forma mais económica a construção que lhe foi adjudicada sem se importar com as hipóteses de que o autor do projecto se serviu para base dos seus estudos, para ele até a maior parte das vezes desconhecidas.

Evidentemente que não deverá ser assim.

Logo após algumas horas da primeira betonagem o betão ganha forma própria e começa a ser solicitado. Todos os engenheiros o reconhecem e desde os bancos da escola técnica onde se diplomaram ouviram as razões pelas quais é necessário verificar a estabilidade das construções à medida que estas se vão desenvolvendo, a que não é estranho a variável tempo, pois são, de resto, os próprios regulamentos a exigi-lo.

Todos os constructores sabem também que uma betonagem que não siga um plano devidamente estudado, que um cavalete mais deformável ou uma interrupção dos trabalhos são a causa de uma, por vezes completa, alteração das tensões calculadas.

Ora, se tais factos, de uns e outros conhecidos, não têm importância na maioria das obras correntes, o mesmo não se poderá dizer das construções de grande volume em que, por exemplo, as cargas permanentes provocam estados de tensão bem superiores aos das sobrecargas de serviço ou em que os esforços da retracção diferencial chegam a provocar a própria rotura do material.

Assim, se uma viga contínua for betonada dos apoios para o meio dos vãos poderá acontecer que à medida que a betonagem vai progredindo, vá o cavalete por seu turno sofrendo deformações que levem o betão já endurecido a absorver uma grande parte das cargas que realmente competiram à obra provisória de apoio. Resultará um maior valor para os momentos flectores nos apoios e uma acentuada redução nos vãos relativamente aos que corresponderiam ao cálculo da viga contínua ideal.

Uma outra ordem de betonagem poderá certamente originar alterações em sentido inverso.

Um pilar de grande largura com a betonagem interrompida por algum tempo em certa junta estará sem dúvida predisposto a fissurar no betão mais jovem, perpendicularmente à junta, dado que este contrairá, por efeito de presa, enquanto que o que lhe serviu de apoio já havia contraído na sua maior parte.

Estas considerações são suficientes para só por si justificarem o maior cuidado na elaboração do projecto das grandes obras de betão armado e das disposições que devem ser adoptadas na construção para que exista perfeita correspondência entre a concepção analítica e a materialização dos cálculos. É destes problemas que vamos tentar dar uma ideia através de casos concretos e de que somos responsáveis, total ou parcialmente, pelos defeitos a seguir assinalados.

2 – BETONAGEM E O FUNCIONAMENTO PREMATURO DA CONSTRUÇÃO OU DOS SEUS ELEMENTOS CONSTITUTIVOS

Um elemento estrutural de grandes dimensões não pode ser betonado duma só vez e instantaneamente. Há pois que fraccioná-lo, mesmo na hipótese de betonagem contínua.

No caso particular das pontes — mais geralmente o arco e o tabuleiro vigado — duas técnicas principais podem ser usadas:

betonagem por camadas sobrepostas;

betonagem a toda a altura dos elementos resistentes e em fracções do comprimento.

Ainda outro caso se apresenta: o das estruturas formadas pela associação de vários elementos que não podem ser executados simultaneamente.

2. 1 – Betonagem por camadas

Neste caso, de betonagem por camadas sobrepostas, é necessário garantir que as camadas já endurecidas não entrem em funcionamento prematuro, o que será difícil de conseguir ou, melhor ainda, que possam resistir eficazmente aos esforços gerados.

O *arco* (ou abóboda) executado por camadas sobrepostas (Fig. 1-a) terá que receber as sucessivas camadas sem que as anteriores sejam fechadas Executando primeiramente o intradorso, depois a alma e finalmente o extradorso, mesmo «corrigido» o arco, de forma alguma poderá trabalhar como um único monolito resistente para o seu próprio peso como se ele fosse de material homogéneo e só entrasse em funcionamento depois de completamente executado. Mesmo que não existisse contracção

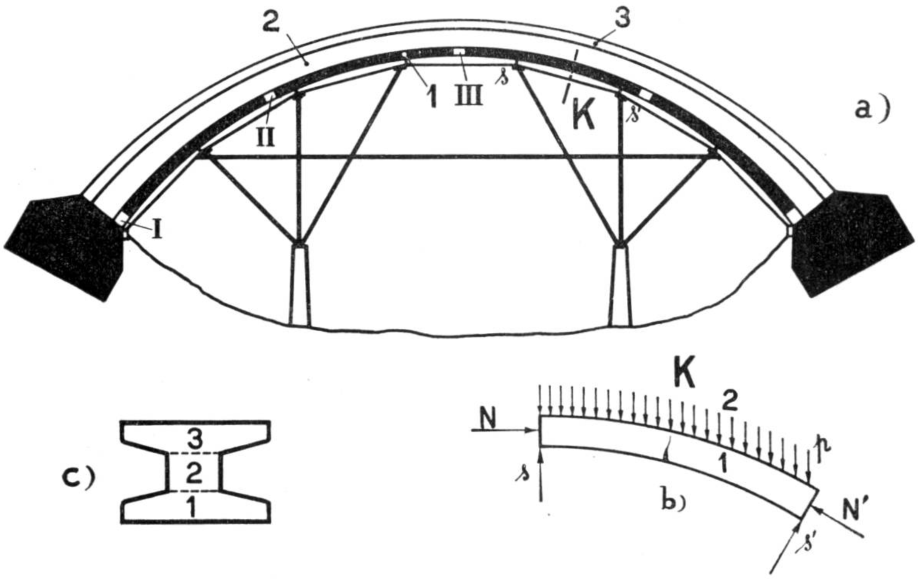

FIG. 1. Esquema dum arco betonado por camadas

diferencial ter-se-ia de admitir que a primeira camada estivesse já solicitada por parte do seu próprio peso e da segunda camada e que estas duas suportariam, em parte, a terceira camada, em condições extremamente difíceis de precisar.

E, se o cimbre tem apoios suficientemente espaçados haverá que garantir que entre esses apoios ss' (Fig. 1-a) a deformação da cambota não provoque, nas débeis camadas endurecidas, flexões exageradas pela actuação do seu próprio peso e em particular pelo peso do betão fresco.

De facto, se fecharmos as juntas I, II, III... antes de lançarmos o betão da 2.ª camada ao executar-se esta passará a primeira a ser solicitada pois é de admitir que o cimbre por ser mais deformável absorva apenas uma pequena fracção da carga do 2.º anel. Por melhor que se pense corrigir os esforços do arco, dado que a correcção não poderá provocar o deslisamento das camadas, há-de fatalmente haver fortes tensões no intradorso em benefício do extradorso, resultando o imperfeito funcionamento da estrutura. Mas se não executarmos as juntas I, II, III... então, não existindo o impulso parcial N e N' (Fig. 1-b) pode dar-se o caso do troço entre escoras s e s' trabalhar à flexão sob a acção do peso p da camada 2, a ponto de romper por não possuir armaduras adequadas. Ao fugir-se dum perigo poder-se-á cair, portanto, num perigo maior.

Seja agora o caso duma *viga contínua* (Fig. 2).

Ao betonar-se a camada 3 sobre as camadas já endurecidas 1 e 2 estaremos a colocar cargas sobre uma viga resistente que evidentemente só transmitirá os seus efeitos ao cavalete depois de partir, dado que uma viga de grande altura tem deformações desprezíveis relativamente

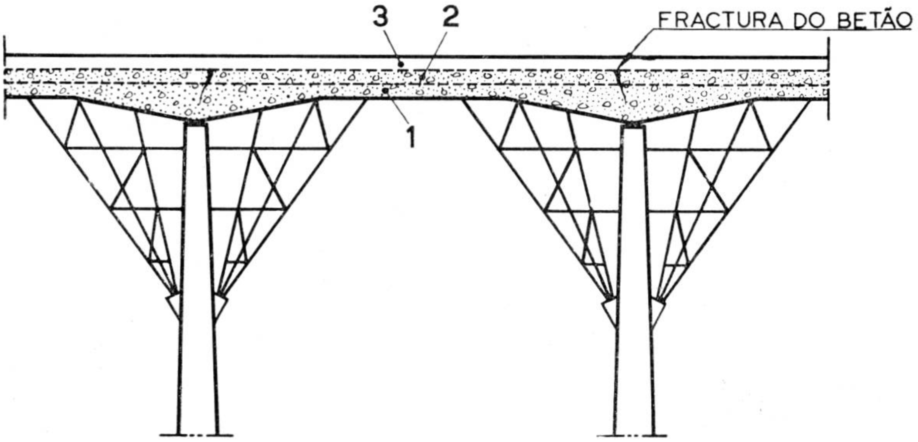

Fig. 2. Viga contínua betonada por camadas

ao cavalete. A meio dos vãos natural é que a viga resista por existirem armaduras suficientes mas nos apoios só não se dará a rotura se nas camadas 1 e 2 existirem fortes armaduras de tracção, o que nem sempre é previsto por quem projecta. Uma destas fendas foi notada na betonagem assim estabelecida no primeiro apoio da ponte do Vale da Ursa, ponte que se representa na Fig. 3. Verificada a deficiência do projecto no que se refere à distribuição das armaduras de tracção nas zonas sobre os apoios e alterado o plano de betonagem de modo a que as camadas de betão envolvessem sempre pelo menos uma fiada de armaduras deixarem de aparecer fendas em todos os outros apoios.

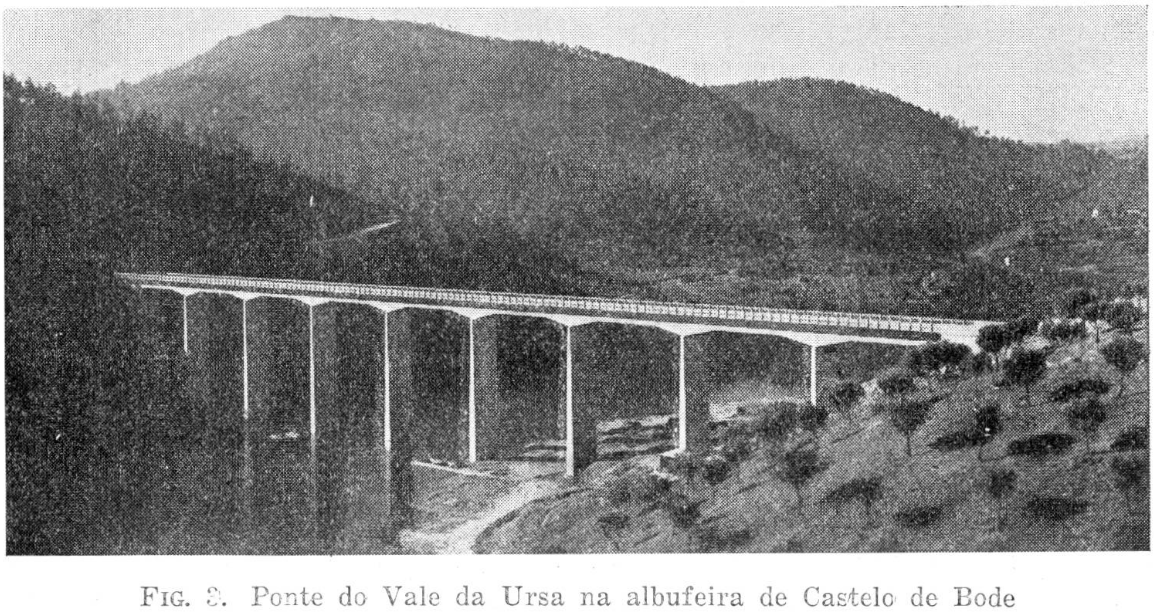

Fig. 3. Ponte do Vale da Ursa na albufeira de Castelo de Bode

2. 2 – **Betonagem a toda a altura dos elementos resistentes**

Também quando se utiliza esta técnica de betonagem não são de desprezar os esforços gerados durante a construção. Assim, no caso dum *arco*, as grandes aduelas, devido aos movimentos do cimbre, funcionam

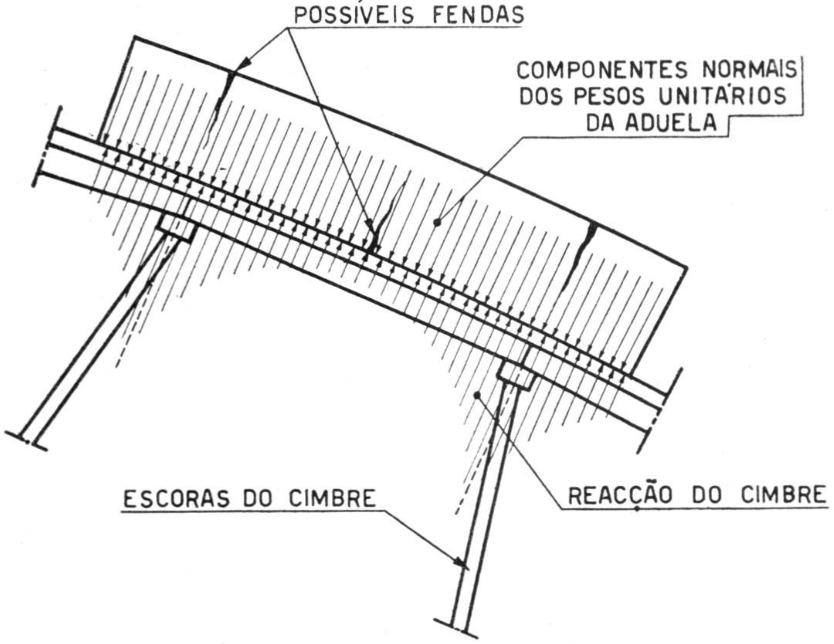

FIG. 4. Esquema duma aduela dum arco betonado a toda a altura das secções

à flexão pelo facto da distribuição das reacções do cimbre não corresponder ponto por ponto ao peso da aduela. É pois possível a existência de fendas como se indicam na Fig. 4, que desaparecerão quando o arco entrar no seu verdadeiro funcionamento. E se o arco é ôco (Fig. 5-a)

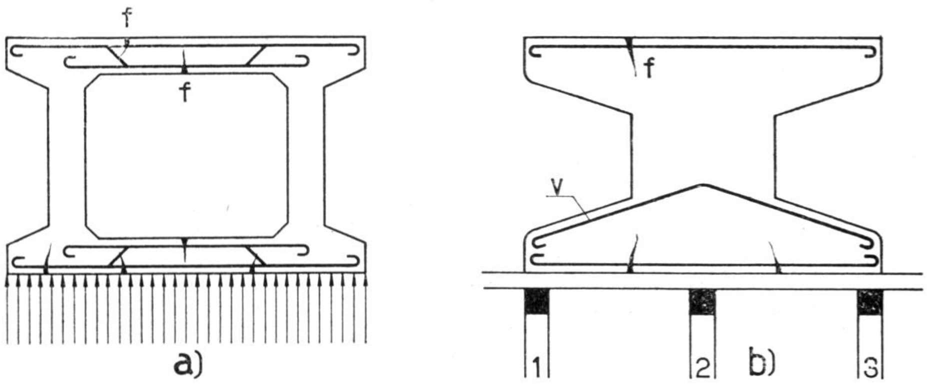

FIG. 5. Esquema das secções dum arco vazado (*a*) e dum arco aligeirado (*b*). Localização das possíveis fendas

ou de secção aligeirada em I, (Fig. 5-b), podem ainda gerar-se esforços na direcção transversal.

Assim, admitindo que a aduela se encontra já executada, pelo facto de não existir impulso que se oponha às forças resultantes do peso e da reacção do cimbre, os banzos, em especial o inferior, serão fortemente solicitados à flexão. Se não se betona ràpidamente toda a aduela ao executar-se o banzo superior já o banzo inferior estará endurecido e então o peso daquele, transmitindo-se apenas pelas almas ao cimbre produzirá neste uma reacção transversal uniforme ou quase uniforme a que o banzo de intradorso do arco terá que resistir por flexão.

Por seu turno o banzo superior, logo a seguir à desmoldagem, ficará solicitado sob a acção do seu próprio peso, com a tendência a romper segundo as linhas f indicadas nas Figs. 5-a e 5-b.

Como se verifica pela Fig. 6 — pormenor das aduelas do arco da ponte da Foz do Sousa indicada na Fig. 7 — a secção transversal do arco é do tipo da Fig. 5-b, apresentando os banzos uma saliência de 1,60 m para cada lado da alma. Para garantir a segurança das aduelas contra tal solicitação foi pois necessário prever as armaduras indicadas.

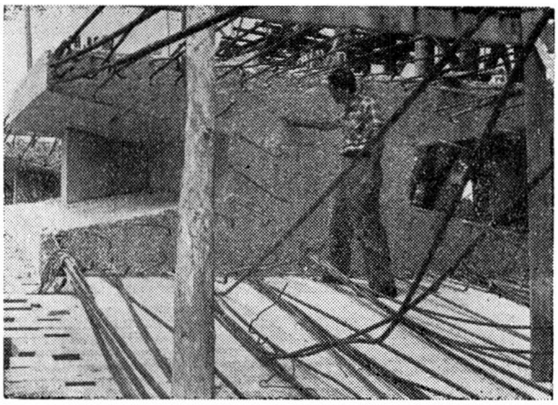

Fig. 6. Pormenor da secção duma aduela do arco da ponte da Foz do Sousa

E se no descimbramento se aliviam primeiramente as cambotas 1 e 3 (Fig. 5-b), serão de admitir esforços no banzo inferior em sentido inverso obrigando por seu turno a empregar as armaduras v. Retirando em primeiro lugar a cambota 2, sob a alma, maiores tensões de tracção surgirão na face inferior do intradorso.

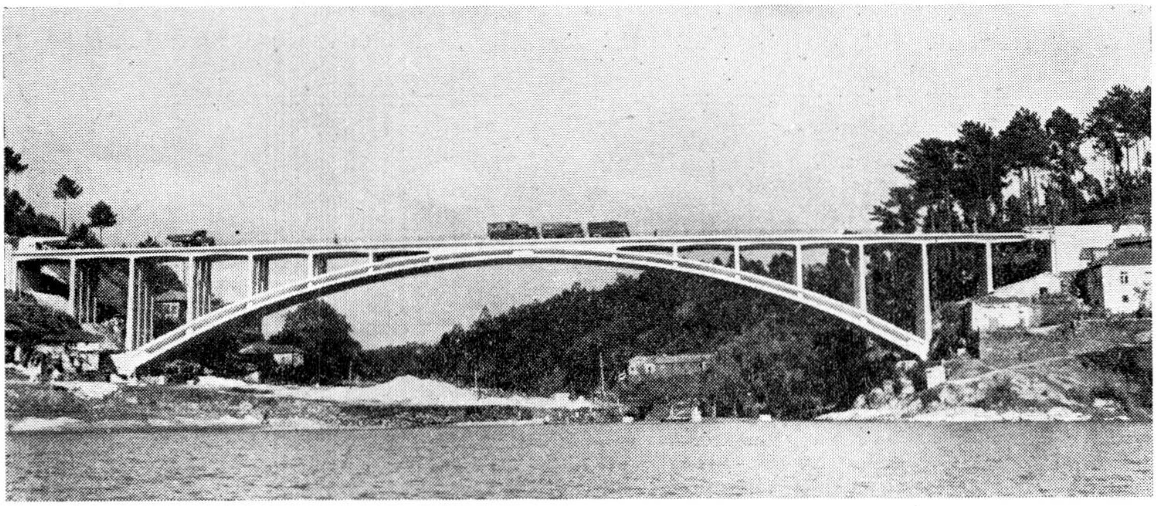

Fig. 7. Ponte da Foz do Sousa — Conjunto

É pois necessário que as secções da estrutura sejam concebidas para resistirem não só ao seu próprio peso mas também que o descimbramento se faça por forma a não obrigar a dispêndio escusado do material para absorver com segurança as tensões acidentalmente desenvolvidas nas diferentes fases de execução.

Procedendo com todos estes cuidados — no projecto e na execução (sem falar no funcionamento longitudinal, por mais conhecido) foi a citada ponte da Foz do Sousa construída sem que se notasse a mais insignificante fenda.

No caso das *vigas contínuas,* com a betonagem por troços a toda a altura, o problema tem ainda alta complexidade.

Para que o funcionamento da viga sob a acção do seu próprio peso corresponda exactamente à hipótese comumente idealizada no projecto torna-se necessário que a viga, uma vez terminada, não esteja já esforçada por deformação dos cavaletes, condição que para ser satisfeita obriga a admitir que a reacção do cavalete seja em cada zona elementar igual ao peso do volume do betão sobre essa zona.

Pràticamente tal hipótese só se consegue deixando pequenos intervalos *i* Fig. 8 entre os diferentes troços 1, 2, 3, 4... e desligando as próprias armaduras o que evidentemente complica a construção.

O que acontecerá então quando não se deixam esses intervalos e se não desligam ou interrompem as armaduras?

Se executarmos a betonagem a partir dos apoios para o meio vão, sempre para um e outro lado do mesmo pilar, é certo que à medida que

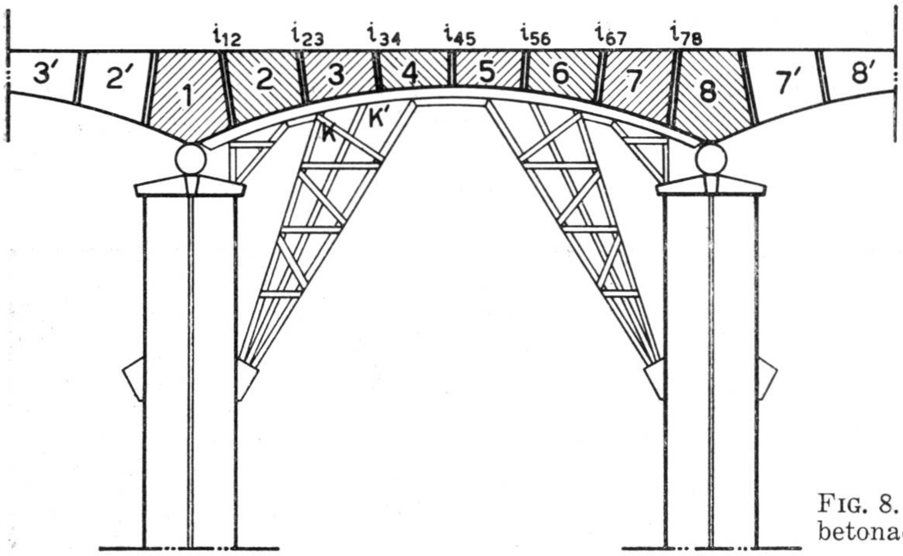

FIG. 8. Viga contínua betonada por aduelas

a betonagem vai progredindo começarão a gerar-se esforços nas duplas consolas sobre cada pilar pois que uma parte do peso do betão informe será absorvido pelo atrito e aderência das juntas de betonagem, pela continuidade dos moldes e do cavalete e das armaduras, em vez de ser integral e localmente transmitido ao cavalete.

Assim, o troço 3, (Fig. 8), deveria descarregar inteiramente sobre a zona KK' mas estando este troço ligado à parte restante já construída 1 e 2, viria para o cavalete apenas uma fracção do peso 3. Prosseguindo por esta ordem com a betonagem a viga contínua passaria a ter momentos flectores nos apoios consideràvelmente maiores do que os correspondentes à viga contínua teórica.

Deixando para o fim as zonas 1 e 8, sobre os pilares, dado que os troços 2 e 7 têm apoios no cavalete consideràvelmente mais rígidos que os centrais 3, 4, 5 e 6, resultará que a viga 2 – 7, já resistente, entrará em funcionamento de peça simplesmente apoiada antes de existir o betão da viga sobre os pilares ou de este betão estar em condições de suportar esforços. Nesta hipótese, aparecerão apenas tenues momentos flectores sobre os apoios.

Só pelo plano de betonagem adoptado será pois possível prever com precisão qual o estado de tensão com que a viga fica depois de descimbrada.

Do exposto e atendendo ao facto de ser sempre ou quase sempre vantajoso reduzir os esforços nos vãos em prejuízo dos apoios deverá a primeira solução ser a adoptada.

Mas é preferivel contar nos cálculos com o momento flector mais conveniente para o vão e para os apoios e estabelecer rótulas provisórias que permitam que a estrutura antes de «fechada» seja isostática. Ao «blocar» os troços que fazem de rótulas a estrutura passará a hiperestática para as solicitações ulteriores, ficando contudo com esforços devidos à carga permanente perfeitamente conhecidos e com a melhor distribuição.

Citaremos a este respeito dois exemplos.

2. 2. 1 – *Caso da Ponte de Coimbra*

Esta ponte é formada por 5 vãos contínuos com uma articulação num vão extremo, apoiada em pilares intermédios pendulares e com os os vãos extremos ligados monolìticamente a montantes articulados na base e constituindo uma superestrutura do tipo pórtico múltiplo.

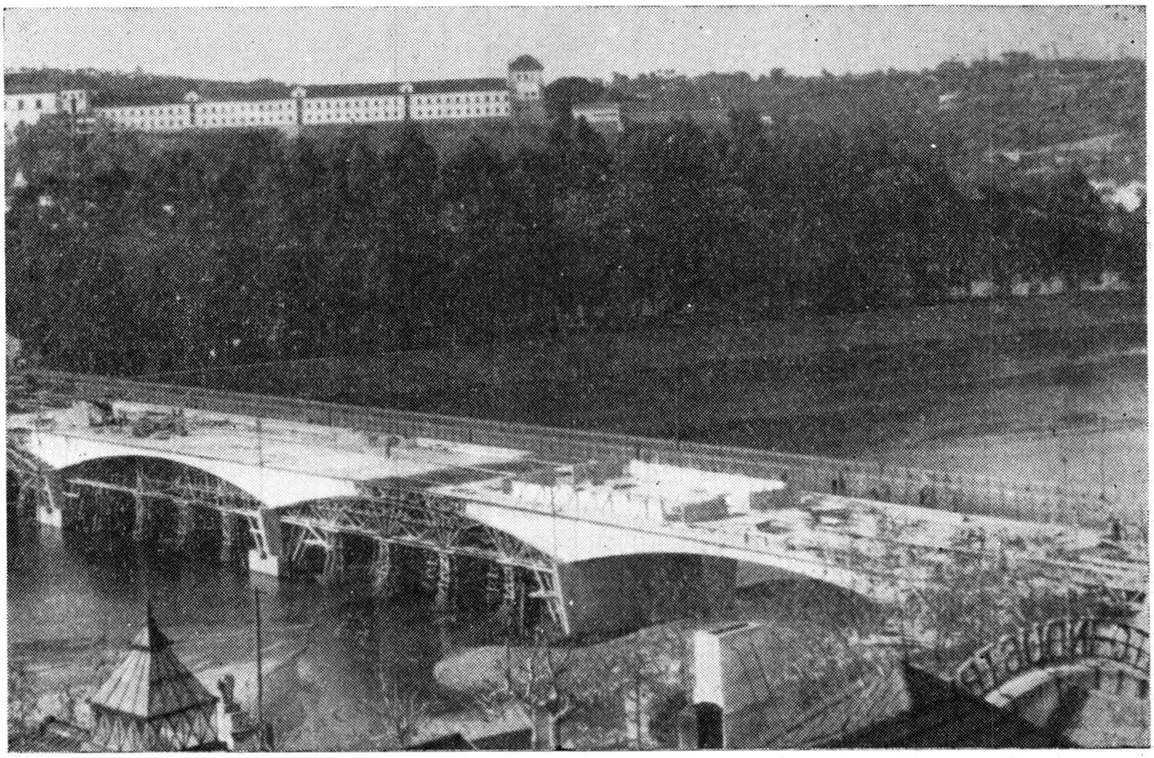

FIG. 9. Ponte de Coimbra — fase de construção — Superestrutura em funcionamento isostático por não estarem fechados os vãos 2.° e 4.°

Para termos a garantia de que a distribuição dos esforços era a desejada e que não se introduziam esforços internos por assentamentos diferenciais das fundações procedeu-se à betonagem do vão central e dos vãos extremos e duma certa parte em consola dos vãos 2.º e 4.º ficando estes por fechar durante tempo suficiente para que se produzissem

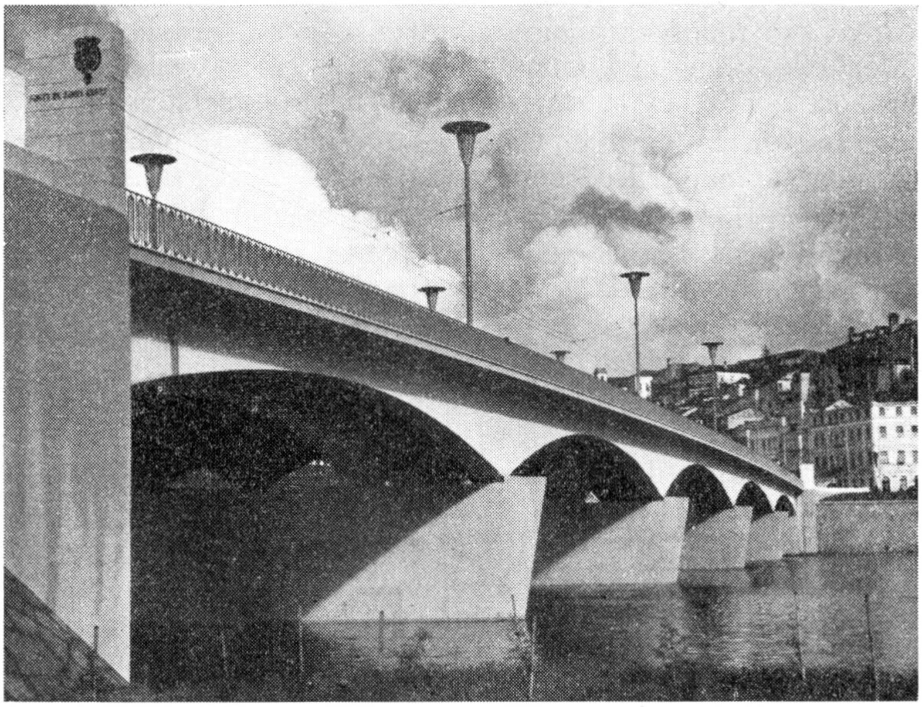

FIG. 10. Ponte de Coimbra (Santa Clara), terminada

os assentamentos dos pilares e se comprovasse por ensaios de carga a indeformabilidade das fundações. Chegou-se mesmo a descimbrar completamente o 1.º vão e a maior parte do 3.º antes de se executarem os pequenos troços centrais de fecho dos vãos 2.º e 4.º, como se vê na Fig. 9.

A correcção dos esforços pode pois fazer-se pela variação do comprimento dos troços em consola, de cargas acidentais colocadas em certas zonas ou com o auxílio de macacos hidráulicos impulsionando pontaletes situados nos vãos ou nas consolas. Utilizou-se esta última técnica para corrigir os esforços com o que se reduziram de cerca de 30 mm as contra-flechas das consolas, como convinha.

Tomando estes cuidados, no projecto e na construção, foi a ponte terminada com pleno êxito. Assinalamos apenas o facto de não termos ligado a importância devida aos deslocamentos durante as diferentes fases da betonagem do que resultou uma razante poligonal curvilínea com ângulos nos topos das consolas e que posteriormente teve que ser corrigido pela pedra de cantaria do focinho da laje dos passeios, correcção que resultou perfeita como se verifica pela Fig. 10.

O estudo das concordâncias dos troços a betonar não pode ser desprezado para que arquitectònicamente nada haja a dizer ou a corrigir. É pois a conclusão que se tira desta última nota.

2. 2. 2 – *Caso das Pontes do Cávado e do Caldo*

Depois da betonagem da ponte do Cávado, na albufeira da Caniçada da Hidro Eléctrica do Cávado, ao serem retirados os taipais laterais das zonas vazadas sobre os pilares, notaram-se umas ligeiras fissuras capilares na parte superior do banzo inferior (Fig. 11). Por essas fissuras se apresentarem em todos os apoios admitiu-se nessa ocasião tratar-se dum efeito sistemático — possìvelmente o resultado duma cedência do cavalete antes que o betão dessas zonas tivesse suficiente resistência.

Na betonagem da ponte do Caldo adoptou-se outro plano de betonagem e reforçou-se a armadura da zona onde se notaram as fissuras na ponte do Cávado com a finalidade de evitar a repetição daquelas fissuras.

Efectuado o descimbramento da ponte do Cávado essas fissuras aumentaram um pouco mais de espessura, não se notando contudo outras.

Imediatamente a seguir a uns dias de intenso calor tropical novas fissuras capilares surgiram em *s* e *m* (Fig. 11).

Revistos os cálculos com a maior atenção concluiu-se estarem pràticamente exactos para as hipóteses formuladas e que é uso considerar, mas não poderia haver dúvidas que algum erro fundamentalmente importante estava em acção.

Analisado o problema sob uma mais alta especulação analítica e experimental — esta com o auxílio de modelos reduzidos de aglomerado de cortiça, de gêsso e de celuloide, planos e espaciais — a par do minucioso exame dos cavaletes e dos planos de betonagem, concluiu-se inequìvocamente terem essas fendas origem:

a) numa importante modificação do diagrama dos momentos devidos à carga permanente, que em vez de ser M M' passou a ser M_1 M_1', (Fig. 11), proveniente da formação de rótulas plásticas na zona vasada dos apoios, devidas ao sistema de betonagem e à deformabilidade dos cavaletes;

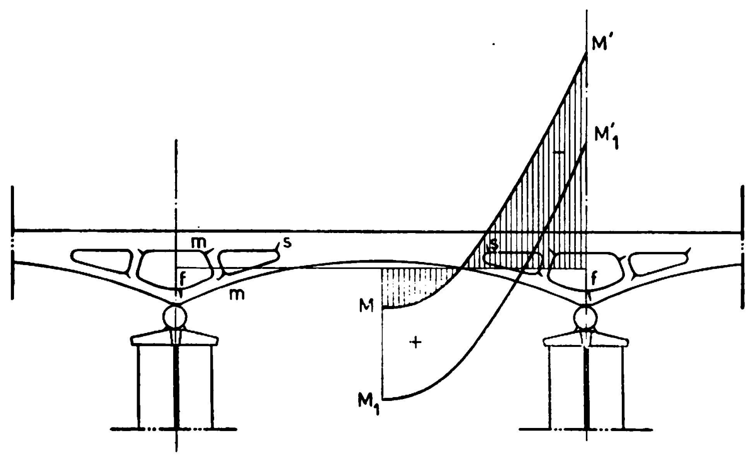

Fig. 11. Esquema duma viga contínua de rótula. Diagramas dos momentos flectores e fissuras

b) na variação de temperatura da laje do tabuleiro, de menor espessura e inteiramente exposta, relativamente às nervuras das vigas, de maior massa e protegidas, quando sobre ela incidiu o intenso calor solar de uns dias de verão o que aumentou ainda mais consideràvelmente o momento positivo, fenómeno que «grosso-modo» se pode computar de intensidade da ordem de grandeza da carga permanente a avaliar pelas flechas observadas em cantilevers construídos, pelos próprios ensaios em modelo reduzido e pelos cálculos analíticos efectuados;

c) na retracção do betão (contracção de presa) e mais particularmente na retracção diferencial das zonas menos armadas para as mais armadas.

Estas diferentes acções, sendo do mesmo sinal, somam os seus efeitos e dada a pequena percentagem de armaduras dessas zonas (pois o seu funcionamento previsto no cálculo era especialmente de compressão) as fendas deveriam realmente formar-se como se constatou na supracitada análise.

Qual a solução a adoptar para dar à estrutura o estado de tensão inicialmente previsto ou o mais aconselhável?

Depois de aturado e cuidadoso estudo e perante o espanto dos operários das obras, resolvemos cortar alternadamente os vãos os quais apenas ficaram ligados pelas armaturas inferiores. Deixaram pois de existir momentos positivos nos vãos constatando-se nos novos cantilevers formados (em substituição da viga contínua) as deformações correspondentes à eliminação dos momentos positivos.

Uma vez cortado o betão e o ferro dessas zonas e ainda corrigido o estado de solicitação com o auxílio de cargas adicionais nos extremos das consolas, soldaram-se a electrogéneo as armaduras cortadas. Retiradas essas cargas encheram-se de betão as zonas que se havia demolido. Desta forma aumentaram-se de 10 a 15 % os momentos flectores sobre os apoios (aumento que se eliminará para o efeito da variação diferencial de temperatura) o que não apresenta qualquer perigo para as estruturas mas em contra-partida puseram-se em funcionamento óptimo aquelas secções que temporàriamente foram esforçadas além do que era permitido.

É pois de concluir que um tabuleiro contínuo merece toda a atenção para que de qualquer modo não haja — por razões construtivas — uma profunda alteração na distribuição dos esforços inicialmente calculados. E dada a grande dificuldade de evitar tais incovenientes só um caminho seguro existe: considerar, no projecto e na construção, para a carga permanente, articulações provisórias que obriguem a ponte a funcionar em condições de isostaticidade.

Mas se por uma razão qualquer o mal já está feito, que não exista pelo menos o receio de lhe dar o remédio já indicado: cortar a ponte em certas zonas onde deveria ser nulo o momento flector ou onde mais convenha e voltar depois a ligá-la novamente, soldando as armaduras e refazendo o betão demolido.

2. 3 – *Betonagem duma estrutura monolítica constituída por vários elementos*

Quando a superestrutura é formada por vários elementos constitutivos que sejam obrigados a um funcionamento de conjuto também o plano de betonagem pode ter fundamental importância. É necessário que ao «fechar» um certo elemento ou parte de elemento não existam esforços nos restantes elementos da «malha» que se completa, particularmente se esta é triangulada e que possam ser prejudiciais em virtude das cargas que faltam não poderem já contribuir para os anular ou reduzir, como sucede nos bow-string, vigas Langer, Nielsen, e em tipos particulares de arco de tímpanos vazados.

Seja o caso da ponte em arco de tabuleiro superior de tímpanos vazados — arco esbelto e tabuleiro rígido, como se adoptou na *ponte de Barca d'Alva* (Fig. 12).

Este tipo de ponte é particularmente vantajoso por permitir a construção com um cimbre muito ligeiro pelo facto dos muretes e do tabuleiro,

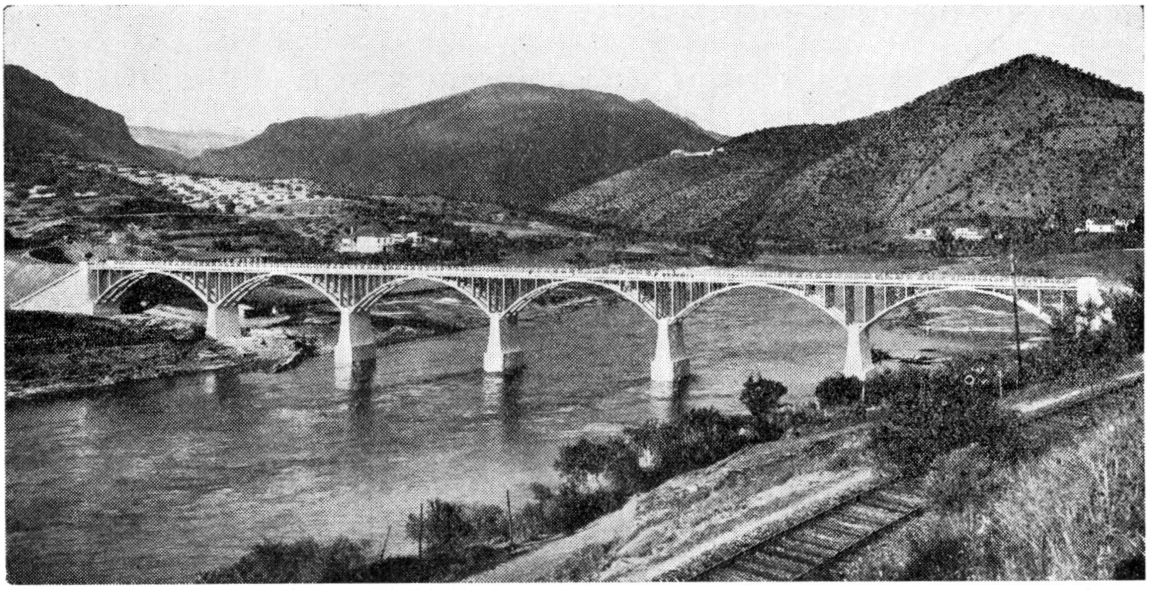

FIG. 12. Ponte de Barca d'Alva, do tipo arco esbelto e tabuleiro rígido

que mais pesam, poderem executar-se já com o arco a servir de cavalete, desde que se distribuam convenientemente as zonas de betonagem.

De qualquer modo, suponhamos que se executa o tabuleiro pela ordem indicada na Fig. 13 começando pela zona do fecho. Quando estiverem executados os troços 1, 2, 3, 4, 5 e 6, o arco funciona como elemento isolado apenas com uma perturbação na zona do fecho (visto que o betão aderindo ao do arco já trabalha em conjunto para as cargas 2, 3, 4, 5 e 6.

Mas, algumas horas depois de se betonarem as zonas 7 e 8, por exemplo, já o arco tem os seus movimentos condicionados ao tabuleiro nas zonas entre os troços 2 e 3. Então, as cargas adicionais 9 e 10, de algumas dezenas de toneladas, não poderão colaborar integralmente nesses troços do arco visto ser agora o tabuleiro a absorver a maior parte dos momentos flectores.

Se no projecto se havia admitido — como geralmente sucede — que a directriz do arco é o antifunicular do carga permanente, o momento que foi para o tabuleiro nas zonas entre 2 e 3 proveniente das tais cargas 9 e

10 fará imensa falta ao arco por não permitir a centragem da linha das pressões, por hipótese admitida nos cálculos.

Em conclusão, a execução da obra tem que corresponder ao cálculo ou este ao processo de execução. No presente caso, em que se adoptou

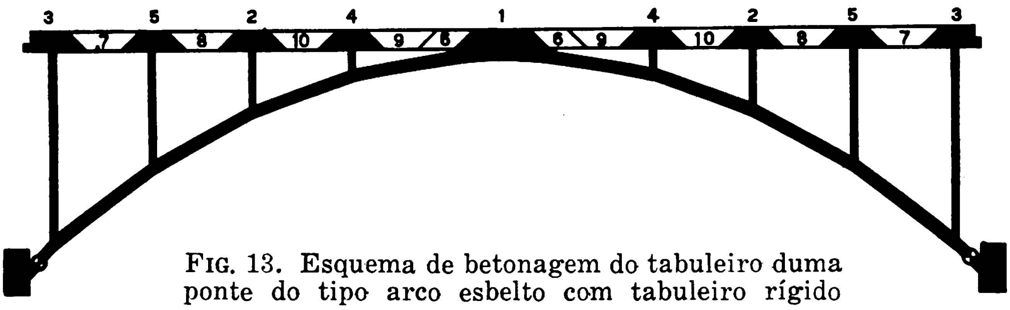

FIG. 13. Esquema de betonagem do tabuleiro duma ponte do tipo arco esbelto com tabuleiro rígido

para directriz o antifunicular da carga permanente dever-se-á «fechar» simultaneamente todos os troços 7, 8, 9 e 10, quer por camadas quer preferivelmente a toda a altura em fracções da largura.

3 - A CONTRACÇÃO DIFERENCIAL E OS SEUS EFEITOS NAS CONSTRUÇÕES MONOLÍTICAS

O betão é um material com «vida própria» de características bem conhecidas, função do tempo, para a mesma génese. Além das características mecânicas as mais importantes são a *contracção de presa,* dum modo geral prejudicial e a *fluência,* fenómeno de auto-adoptação ao mínimo esforço, quase sempre favorável.

Sendo pois a contracção uma função do tempo, traduzida por uma curva O, a, b, c, d com a configuração da Fig. 14, resulta que, se sobre um certo betão já resistente de *n* dias de idade, executarmos um novo troço de betão este não pode contrair livremente segundo a curva O' a' b' c' d'.

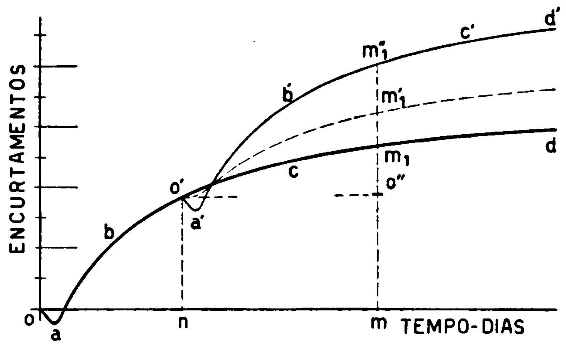

FIG. 14. Variação da contracção do betão com a idade — Contracção diferencial

Suponhamos que não existe a fluência e que os betões resistem, o novo às tracções e o mais idoso às compressões (este sem dúvida resistirá). Ao fim de m dias a junta de betonagem do betão velho encurtou de m m'_1 e o novo betão que devia ter encurtado se fosse livre de m''_1 o" encurtou apenas m'_1 o", isto é, ficou submetido a uma tensão correspondente a m''_1 m'_1. Gerar-se-ão, portanto, na superfície de betonagem tensões de escorregamento e perpendicularmente tracções no betão mais jovem e compressões correspondentes ao encurtamento adicional m'_1 m_1 no betão mais idoso.

Estas tensões raramente provocam a rotura tanto mais que são atenuadas pela fluência, principalmente a do jovem betão. Contudo, tratando-se de juntas de betonagem muito extensas — no caso das barragens, dos elementos de pontes muito largas — tal rotura poderá surgir se não se adoptam disposições construtivas adequadas como é, com efeito, suficientemente conhecido dos engenheiros da especialidade.

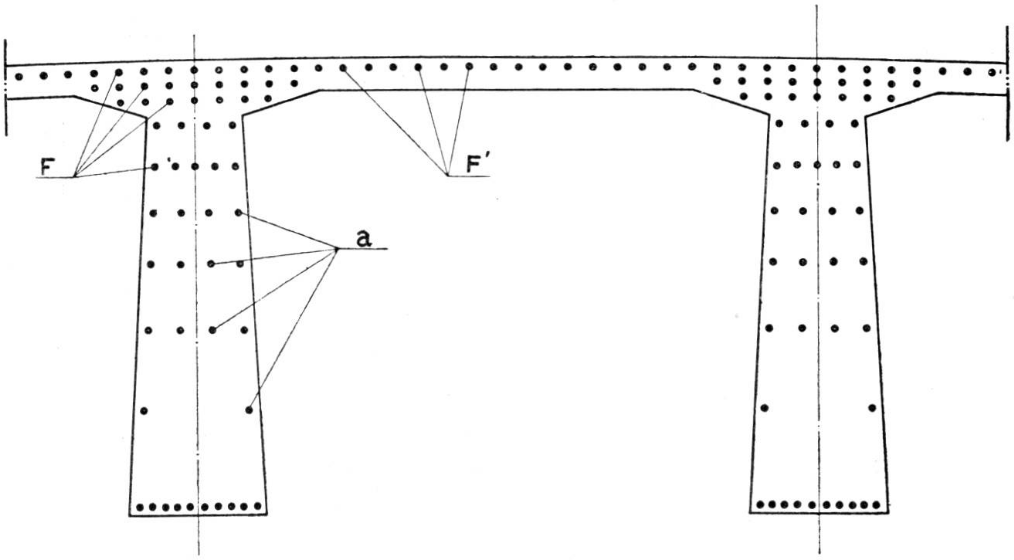

FIG. 15. Esquema da distribuição das armaduras nas zonas dos apoios das vigas contínuas, em «T»

Um outro aspecto do problema diz respeito à diferença da contracção verificada entre as zonas com grande percentagem de armaduras e das zonas fracamente armadas, embora da mesma idade, efeito nem sempre devidamente tido em conta nos projectos, fenómeno importante, sem dúvida, para as peças de grande comprimento e de apreciável altura das secções transversais ou de grande largura com armadura concentrada em restritas zonas.

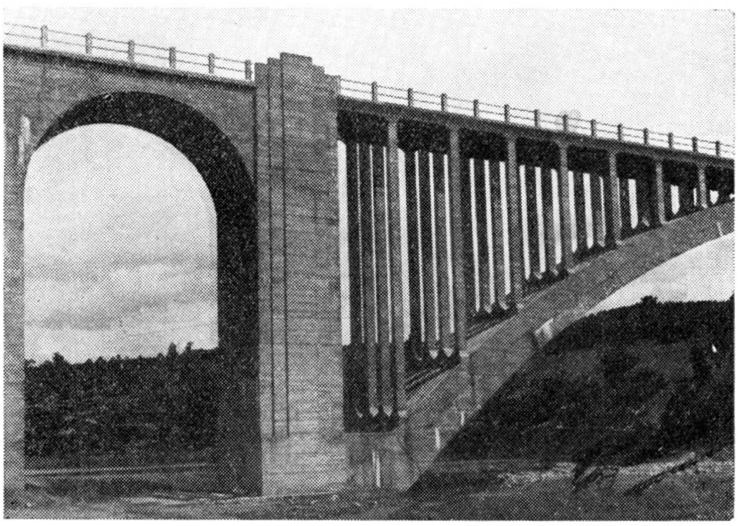

FIG. 16. Ponte em arco, betonado por aneis

Referimo-nos às vigas altas armadas à tracção e à compressão (para ter em conta a alteração dos esforços com as posições das sobrecargas) com reduzidas armaduras nas almas ou a grandes vãos de laje dos tabuleiros vigados nas zonas dos momentos negativos das vigas "T". É pois necessário colocar armaduras *a* nas almas (Fig. 15) e distribuir as armaduras de tracção das vigas, pela alma F e pela laje, F'. Se assim não está indicado nos desenhos deverá o construtor chamar a atenção do autor do projecto ou das entidades da fiscalização dos perigos que poderão advir.

Tais dispositivos para reduzir os efeitos da contracção diferencial consistem, no primeiro caso, na alternância, das betonagens, no endentamento das camadas, no emprego de varões de travamento, caixas, saliências ou simples pedras entre os dois betões, na adopção de armaduras longitudinais colocadas nos troços mais jovens paralelamente às juntas de betonagem e próximo destas, etc., além dos aconselháveis tratamentos dos betões (mantê-los húmidos, empregar a refrigeração) e do emprego de cimentos especiais como os de baixo ou moderado calor de hidratação. No segundo, além dos cuidados já enumerados, na distribuição racional das armaduras pela secção da peça.

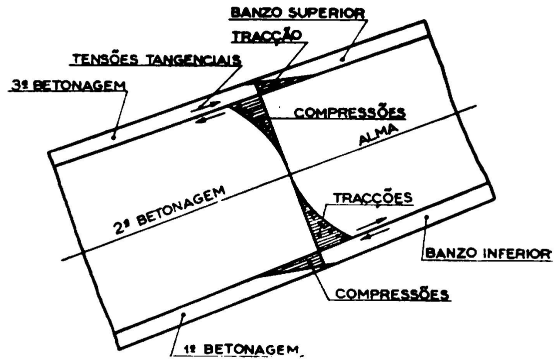

FIG. 17. Tensões devidas à contracção diferencial num arco betonado por aneis

No caso das pontes o fenómeno surge mais particularmente nos elementos seguintes:

a) Nas juntas de ligação das «camadas» dos arcos quando se adopta este tipo de construção, o que se considera altamente defeituoso mesmo quando se endentam as camadas, semelhantemente à clássica técnica usada nas abóbodas de cantaria de pedra natural.

Na Fig. 16 duma ponte onde foi seguida esta técnica a 2.ª camada desligou da 1.ª pelo menos em certas zonas como se constata pelas próprias escorrências calcáreas das juntas.

Entre as várias camadas dum arco construído por anéis desenvolvem-se tensões que esquemàticamente se indicam na Fig. 17 e que se sobrepõem às do funcionamento «arco».

Um caso semelhante ao da construção dos arcos por camadas é o dos muros de tímpano sobre as abóbodas.

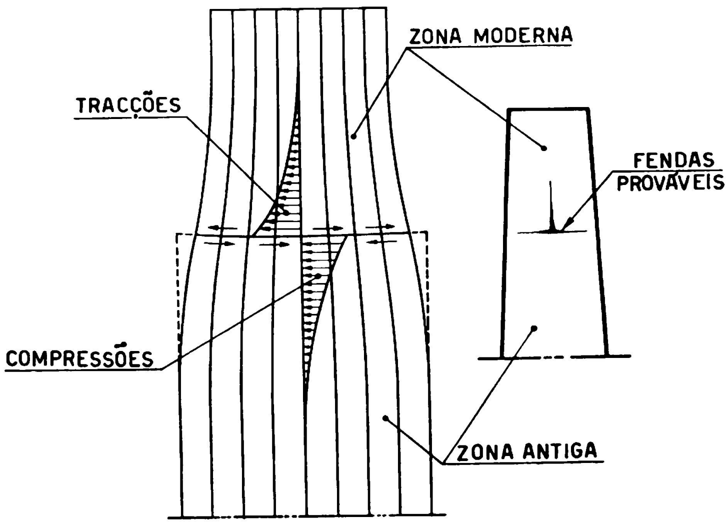

FIG. 18. Tensões devidas à contracção diferencial num pilar. Fendas prováveis

b) Nas juntas de betonagem dum mesmo elemento quando betonado simultâneamente em toda a área das secções normais à directriz, tal como nos grandes pilares ou na construção de arcos por aduelas.

Neste caso, o problema não tem a gravidade do caso anterior, à excepção dos depósitos para líquidos, porque mesmo que se dê a rotura da junta, por deslizamento, ela continuará a trabalhar em boas condições por estarem em geral em funcionamento de compressão. O fenómeno pode esquematizar-se na Fig. 18. O troço mais antigo na ligação ao moderno é obrigado a comprimir-se e este a dilatar donde resulta, na junta, um estado de tensão tangencial e nas secções paralelas à directriz um estado de tensão normal, de tracção no betão novo e de compressão no betão velho.

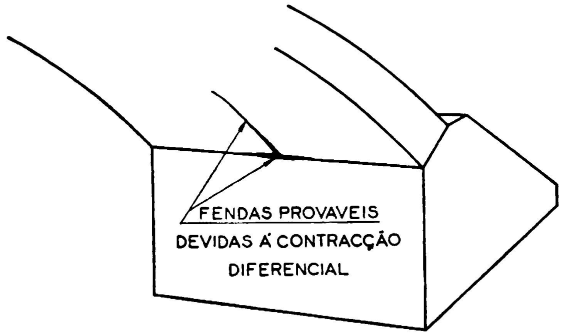

FIG. 19. Efeitos da contracção diferencial na ligação duma abóboda aos encontros

Se a superfície tem uma grande área, da ordem dos 100 m², ou um grande lado, 20 ou 30 m, quase certo é dar-se a rotura quando o intervalo nas betonagens vai além de 15 a 30 dias se não se tomam as disposições atrás indicadas. A rotura dá-se como indica a Fig. 18 sendo numerosíssimos os exemplos que se poderiam apresentar.

O caso mais característico é o da ligação das abóbodas largas aos maciços dos encontros (Fig. 19). Estes são construídos em geral com uns meses de avanço de modo que o betão da abóboda tende a contrair sobre um maciço de grande volume, já quase completamente contraído.

Se a abóboda não possui armaduras transversais suficientes é pràticamente inevitável a rotura, para larguras de abóboda superiores a 20 m. Constatamos uma rotura deste tipo na Passagem Superior da Cruz das Oliveiras na Auto-estrada Lisboa-Estádio, apenas com 15 m de largura de abóboda, a qual tem muito reduzida armadura transversal.

Não é pois possível, sem perigo, projectar uma larga abóboda de betão simples, pois que a esses efeitos adicionam-se os das sobrecargas e dos impulsos dos muros de tímpano.

4 - A DESIGUALDADE DE TEMPERATURAS ENTRE AS DIVERSAS FIBRAS DO MATERIAL DAS SECÇÕES

Pode dizer-se que é corrente, no cálculo de pontes, ter-se em conta os efeitos das variações de temperatura uniforme, quando as estruturas são hiperestáticas. Mais raramente se consideram as variações de temperatura desigual dos elementos componentes de uma obra e só muito excepcionalmente se tem em conta a diferença de temperatura entre as diversas fibras do material das secções transversais das estruturas.

Este último efeito é, contudo, de alta importância em certas obras directamente expostas às radiações solares. As mais afectadas são, sem dúvida, os tabuleiros de pontes (na construção civil os terraços) do tipo hiperestático (lajes contínuas ou lajes nervuradas hiperestáticas) como é do conhecimento geral. Todos os engenheiros civis sabem calcular estes efeitos pelo menos para certas hipóteses particulares como a da variação linear da temperatura nas secções transversais.

Mas o que nem sempre se tem presente é a ordem de grandeza das tensões geradas e muito especialmente o «sentido» dos esforços gerados.

Nos tabuleiros do tipo viga contínua, laje nervurada ou laje sem nervuras, para fraca camada protectora isolante como é a dos pavimentos modernos das pontes (betão betuminoso de 3 a 5 cm de espessura), a variação de temperatura pode atingir, no nosso clima 10 a 20° C, consoante a região. Do meio dia às 3 da tarde natural é verificar-se em certos dias de verão tais variações de temperatura no sentido positivo, quer dizer, encontrar-se a face superior a 40° C e a face inferior a 25° C, o que não sucede em sentido inverso. De facto ao desaparecer o sol essa diferença estará pràticamente reduzida a zero não fazendo o fresco da noite com que exista uma diferença negativa de mais do que $^1/_4$ ou $^1/_5$ daquele valor.

O que sucederá pois?

Se chamarmos positivos os momentos flectores que geram tracções nas fibras inferiores somos levados a concluir que os efeitos do sol sobre os tabuleiros contínuos de pontes provocam momentos flectores positivos acentualmente mais intensos do que momentos negativos. Haverá portanto um apreciável aumento de tensões nas zonas dos vãos, tensões que serão em muitos casos da ordem das provocadas pelas cargas permanentes com uma correspondente diminuição de esforços nas zonas dos apoios.

Em conclusão, se tais efeitos não foram considerados no projecto deverá o construtor ao elaborar o plano de betonagem, tomar o cuidado de executar a superestructura de modo que ela fique com momentos positivos menores do que previu o cálculo tanto mais que os efeitos da fluência são geralmente do mesmo sinal da variação diferencial de temperatura. Foi esta razão pela qual, no exemplo atrás citado das pontes do Cávado e do Caldo, decidimos anular totalmente os momentos a meio do vão provenientes da carga permanente à custa do corte temporário dos tabuleiros.

RESUMO

No presente artigo indicam-se os cuidados a ter na construção das grandes pontes de betão armado tendo em vista a eliminação total ou parcial dos esforços secundários desenvolvidos nas estruturas durante as suas diferentes fases de execução em particular provenientes do sistema de betonagem e em função das deformações dos cavaletes e da contracção diferencial do betão.

Analisa-se, também, o fenómeno da alteração da distribuição dos esforços principais desenvolvidos nas estruturas proveniente das mesmas causas e, ainda, da desigualdade de temperatura entre as diversas fibras do material.

Ilustra-se o artigo com os casos concretos:

a) Arco abatido do tipo aligeirado (ponte da Foz do Sousa).

b) Pórtico especial com vigas de altura muito variável (ponte de Coimbra).

c) Arco múltiplo esbelto de tabuleiro rígido (ponte de Barca d'Alva).

d) Viga contínua de alma vasada sobre pilares ôcos de alvenaria (pontes do Cávado e do Caldo sobre a albufeira da Caniçada, da Hidro Eléctrica do Cávado).

RÉSUMÉ

L'auteur décrit les précautions à prendre lors de la construction des ponts en béton armé de grandes dimensions en vue de l'élimination totale ou partielle des efforts secondaires qui prennent naissance au cours des différentes phases d'exécution; il traite plus particulièrement le cas des efforts secondaires dûs au système de bétonnage adopté et en fonction des déformations des supports de coffrages et du retrait du béton.

Il étudie également le phénomène de la modification de la distribution des contraintes principales dues aux mêmes causes et aux différences de température entre les différentes fibres du matériau.

Il cite les exemples pratiques suivants:

a) Arc surbaissé allégé (Pont de Foz do Sousa).

b) Portique spécial à poutre de hauteur très variable (Pont de Coimbra).

c) Arc multiple mince à tablier rigide (Pont de Barca d'Alva).

d) Poutre continue en caisson sur piliers creux en maçonnerie (Ponts du Cávado et du Caldo sur la retenue de Caniçada de l'aménagement Hydro-Electrique du Cávado).

SUMMARY

The author describes pratical methods applied to large reinforced concrete bridge construction in order to eliminate in part, or totally, secondary efforts developped in the course of construction; the author deals more particularily with those secondary efforts which are caused by the concrete laying methods adopted and depending on the trestles' deformations and shrinkage of concrete.

Modification of the repartition of a bridge's principal stresses due to the same factors and to differences of temperature between the various fibers of the concrete is also dealt with.

The following practical examples are mentionned:

a) Lightened flat arch (Foz do Sousa Bridge).

b) Special frame structure with large variations of beam heigth (Coimbra bridge).

c) Multiple thin arch with rigid platform (Barca d'Alva bridge).

d) Continuous box web beam on hollow masonry columns (Cávado and Caldo bridges on the Caniçada empoundment lake of the Cávado Hydro-Electric Scheme).

ZUSAMMENFASSUNG

In der vorliegenden Arbeit werden die Massnahmen gezeigt, welche bei der Konstruktion von grossen Brücken in Eisenbeton getroffen werden müssen, um die Sekundärkräfte im Bauwerk vollständig oder teilweise auszuschalten. Diese treten während den verschiedenen Phasen des Baues auf, insbesondere infolge des Betoniervorganges, der Formänderungen des Lehrgerüsts und des Schwindens von Beton.

Untersucht wird ferner die Aenderung der Hauptbeanspruchungen, welche aus denselben Gründen und infolge der Temperaturunterschiede in den verschiedenen Teilen des Bauwerks auftritt.

Die Arbeit wird illustriert durch folgende konkrete Fälle:

a) Flacher Bogen in augeflöster Bauweise (Bücke von Foz do Sousa).

b) Rahmenbrücke mit veränderlicher Querschnittshöhe der Balken (Brücke von Coimbra).

c) Mehrfacher schlanker Bogen mit Versteifungsträger (Brücke von Barca d'Alva).

d) Durchlaufender Balken mit unterbrochenem Steg über Hohlpfeilern aus Mauerwerk (Brücken von Cávado und Caldo über den Caniçada-Stausee des Kraftwerks Cávado).

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Slide-forms in concrete bridge column construction

Gleitschalungen für Betonsäulen bei Brückenbauten

Cofragens deslisantes para a construção de pilares de pontes

Coffrages glissants pour la construction de piliers de ponts

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Bromma

For high bridge columns of concrete the costs for forms and scaffolding for shuttering and casting in situ are rather high compared with the costs for the reinforcing steel and the concrete which are the main components of the construction. The support-columns on concrete arches with large spans have to be made as slender as possible which makes the use of concrete of a high quality necessary. A great deal is expected also in regard to the look of the columns. Previously used, conventional timber or steel shuttering for columns hamper the handling of the concrete in the narrow form and for the erection of the shuttering a comparatively large number of skilled hands is needed.

Through the simplified method for slide-form concreting — the Concretor System — developed in Sweden during the last decade, large savings in material and labour costs can be made, and at the same time the concrete will be easily accessible for handling, whereby the possibilities of making high quality concrete probably are better than by any other method. A profitable use of this new method, however, is subject to a suitable design of the construction and a comparatively large height of the columns.

The first bridge building in Sweden at which slide-forms were used would be the railway bridge over the river Vindelälven up north, constructed in the years 1950 to 1952 by Nya Asfalt AB for the Royal Board of Swedish State Railways. This bridge comprises an arch span, 112 metres of length (fig. 1) and approach viaducts, supported by pairs of columns with circular cross section, spaced 14.35 metres (fig. 6). The height of the columns varies from 9.7 to 22.8 metres. The total length of the bridge is 242 metres

When in late autumn 1952 the work was started, heavy increases in prices, caused by the war in Korea, prevailed. It was, therefore, to the advantage of the client as well as of the contractor to keep the quantity of material to be used for temporary constructions as low as possible. The contractor, together with AB Byggförbättringar of Stockholm who originated the Concretor Method, made out a working programme for the use of slide-forms. The programme was approved by the client and all the bridge columns, except the shortest ones at both sides of the crown of the arch span, were constructed by the Concretor Method. The forms were of sheet steel and of a height of 1.2 metres (fig. 3). The jacking equipment consists of a number of hydraulic jacks, here four, on which the form equipment is suspended. These jacks are connected through oil pipes with a small electric powered oil pump. When the pump is started oil is pressed to the jacks which all raise simultaneously. When needed the jacks may be operated individually, thus allowing for adjustment in case of deviation from the vertical line.

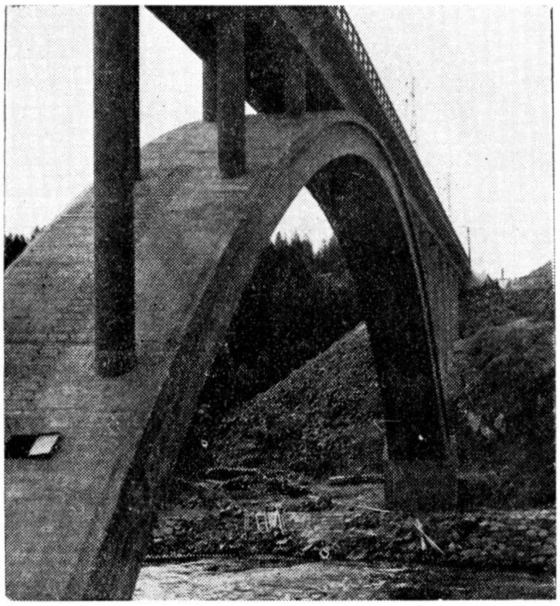

FIG. 1. Railway bridge over the river Vindelälven in North Sweden

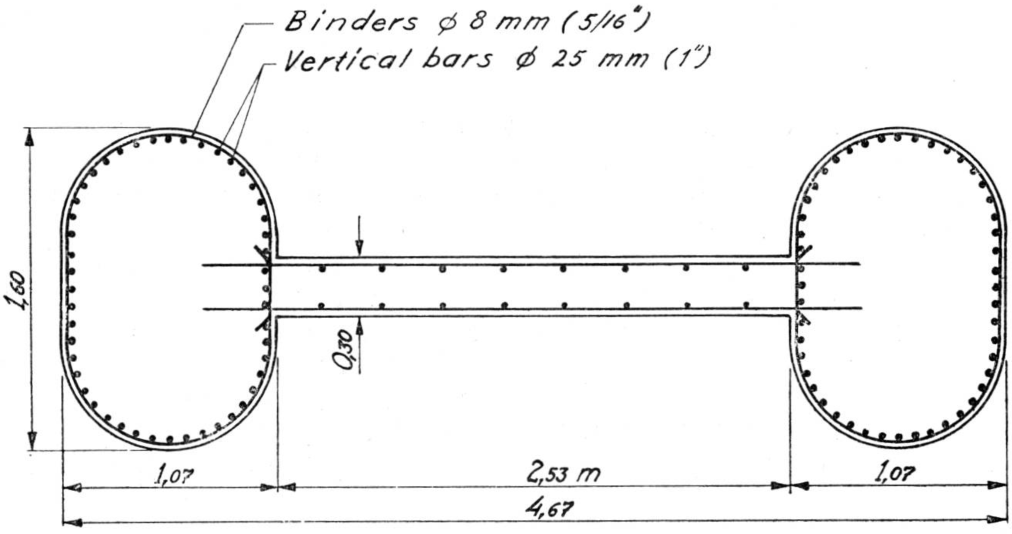

FIG. 2. Cross section of an abutment column

Each jack «climbs» on a smooth steel rod 25 millimetres of diameter and each raise is 25 millimetres.

The lifting speed depends amongst others on the growing strength of the concrete, and was in this case 3 to 4 metres per 24 hours. The quality of the concrete used for columns and all other parts of the concrete construction, with the exception of foundations and abutments was K 400 (min. ultimate strength 400 kg/cm^2 or 5700 lb/in^2 after 28 days). The reinforcing steel consists of deformed bars of the quality Ks 40 with a yield ratio of 40 kg/mm^2 (57000 lb/in^2).

The columns on the arch abutments (fig. 2) are heavier than the rest of the columns. On account of the new method the sharp angles between the columns proper and the connecting wall have to be rounded, which merely adds to the structural strength. Furthermore, the binders, being placed in the course of the pouring, need have two splices, making each unit consist of two U-shaped parts instead of one oval.

For constructive reasons the vertical reinforcement of the columns has to be made using as long bars and as few splices as possible. The maximum length of the bars is 10 metres. It was therefore, necessary to build, at the side of a column to be constructed, an auxiliary scaffold fitted with extending fixtures for keeping the reinforcing bars in position. The steel scaffolding used will be seen from fig. 4 and 5. If it is possible to limit the length of the reinforcing bars to 5 to 6 metres they may be kept in position by fitting similar fixtures to vertical masts, attached to the slideform. The reinforcing bar positioning fixtures will then follow the upward movement of the slide form, and no scaffolding will be needed at the side of the column.

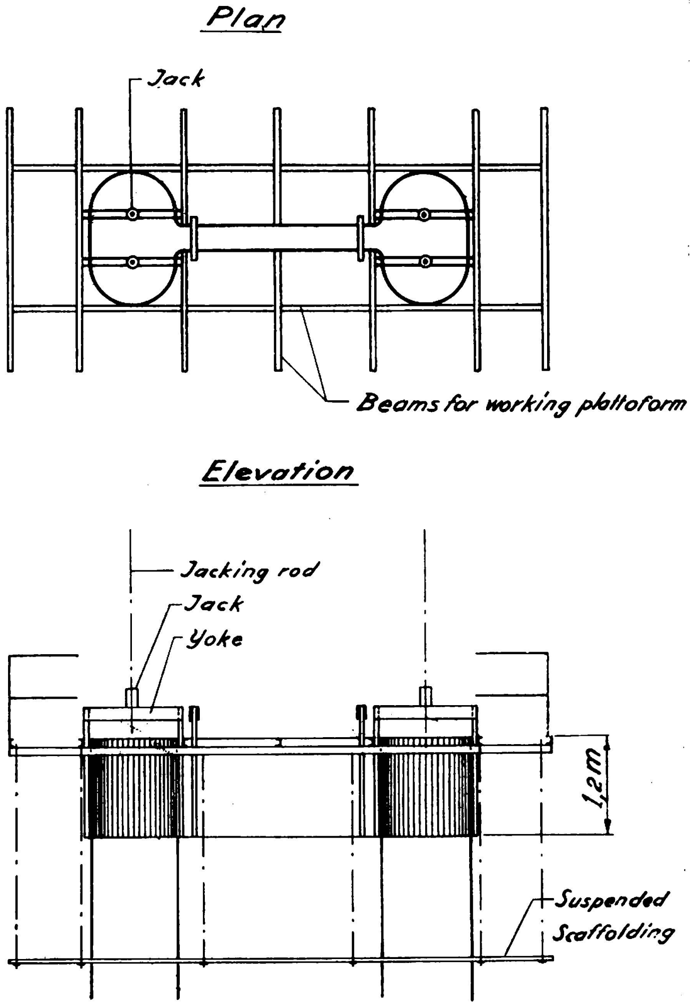

FIG. 3. Slide-form for abutment column

When the first column was cast the night temperature was rather low and calcium chloride was added to the concrete so that the lifting speed would not have to be reduced too much. When the slideform had been raised somewhat above the position shown on fig. 6 a suspended scaffolding was attached as indicated on fig. 3 to make possible inspection of the slide-form itself and of the consistency of the concrete underneath the slide-form. The suspended scaffolding is used also for finishing the concrete surface. Great demands are made on the appearance of the

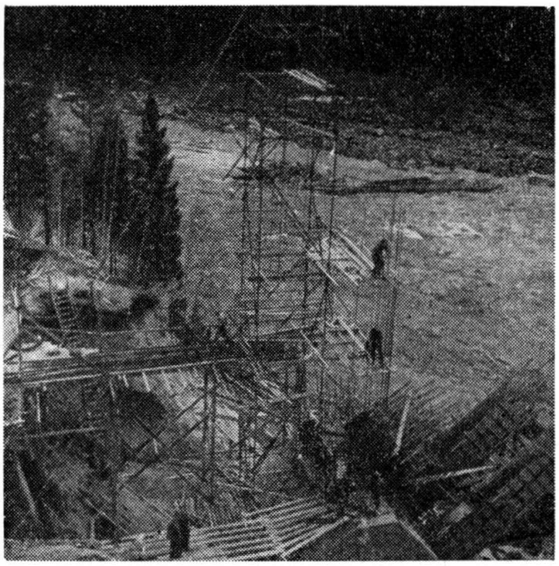

FIG. 4. Auxiliary steel tubular scaffolding at an abutment column. The assembly of the sliding form is in progress

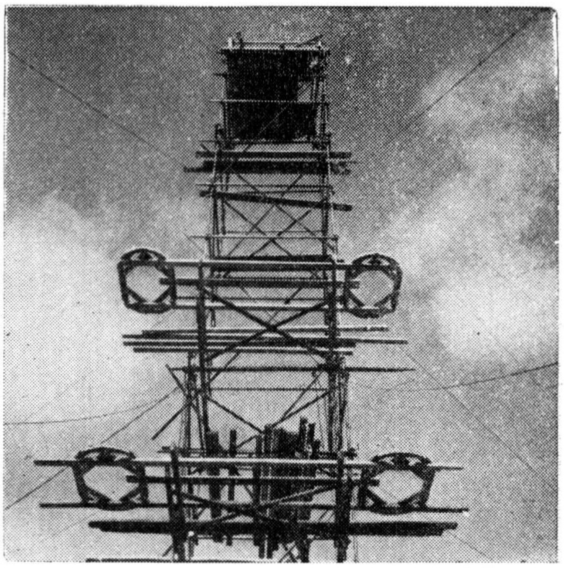

FIG. 5. Auxiliary scaffolding with fixtures for the reinforcement, seen from below

concrete surfaces of bridges and therefore the aftercasting treatment is of main importance. During the period following the casting all slide-formed surfaces are getting flamy or striped depending upon the age and the humidity of the concrete. If the after-casting treatment is properly done by skilled workmen these flaws normally will disappear when the concrete has dried up. A slight rubbing of the surface with a broom should be sufficient, whereas the rubbing with felt is not recommended. The after-casting treatment of course includes water spraying and protection of fresh concrete surfaces, amongst others, against insolation.

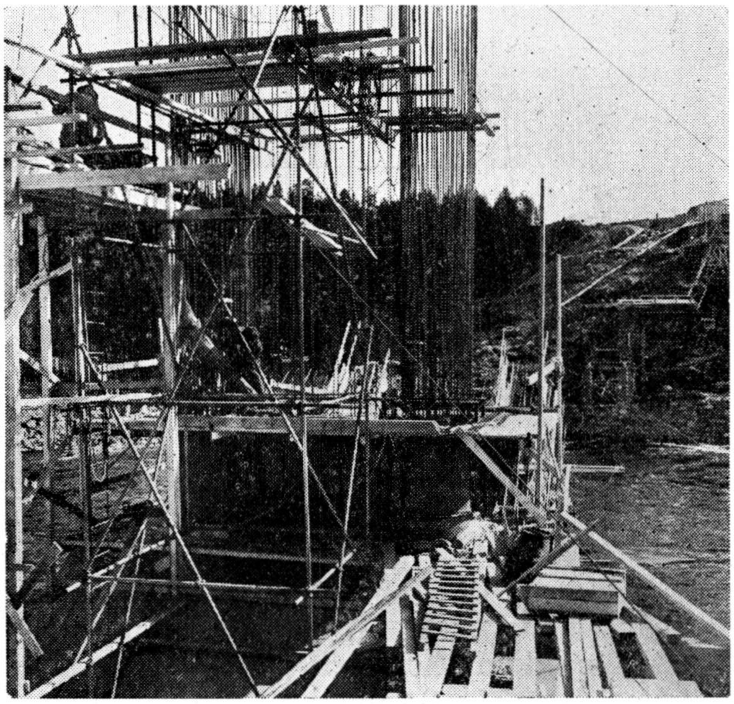

FIG. 6. Concreting of an abutment column by means of a sliding form. The raising of the form has recently begun. Photo. Eric Rosenberg

The rest of the columns are circular and placed in pairs. The viaduct columns are 1.0 metre and the columns on the arch are 0.9 metres of diameter. Slide-forms and working platform used for these columns were similar to the equipment used for the abutment columns. Consequently the slide-forms for a pair of columns were connected through the working platform. In these columns too the binders were made in halves.

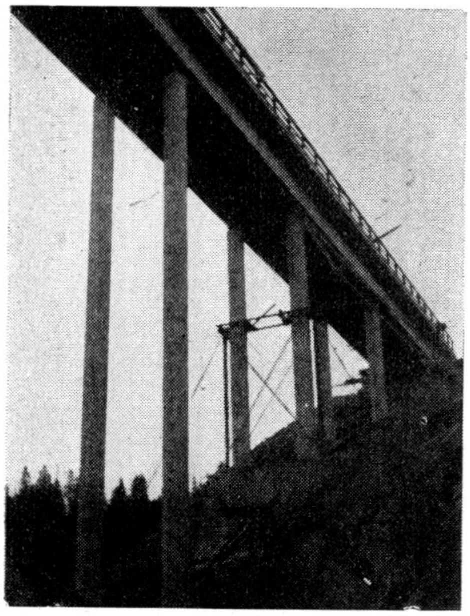

FIG. 7. Guying of a column

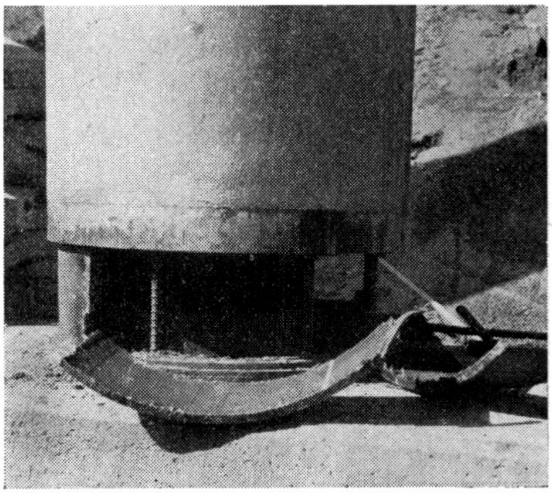

FIG. 8. Provisional arrangement used for a row of columns

During the construction work the columns are fixed only in the foundation, and the longer columns therefore are exposed to buckling until they are fixed in the bridge deck. Until the concrete is strong enough the columns are even susceptible to wind stress. Several columns, therefore, had to be guyed during the casting and the guying had to remain until the bridge deck was concreted. Before the critical height of the columns was reached a guy bridge as shown on fig. 7, was mounted right below the suspended scaffolding. The guy bridge consists of rings around the columns connected by a steel frame. The guy bridge was supported by guys resting on the foundation and braced by steel rods with tightening nuts at the end brackets.

To make possible adjustments made necessary by eventual future sinking, columns not having foundations on rock were supported on the foundation by bearings of steel.

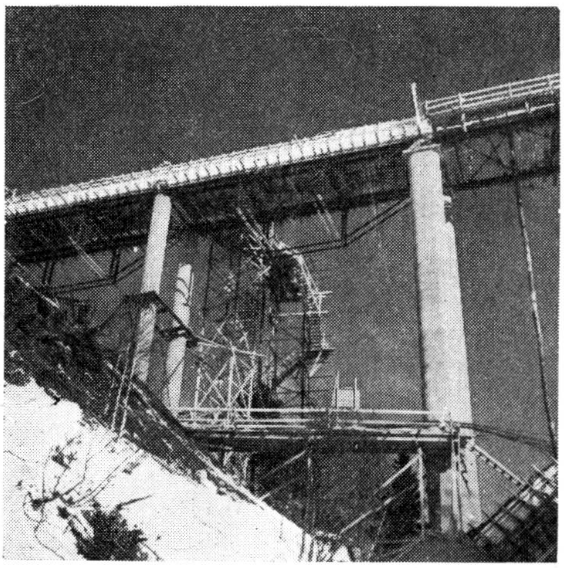

FIG. 9. Steel trusses between columns, concreted by means of slide-forms. The hoist and tower at the concrete mixing plant is seen in the background

When casting these columns the foundation and the columns were provisionally connected by means of reinforcing irons passing through holes in the plates of the bearing, and a supporting ring of steel was inserted between the plates. After concreting the bridge deck the provisional parts were removed (fig. 8).

In order to fully utilize the rationalization implied in the use of slide forms, conventional scaffolding close to, or between, the columns

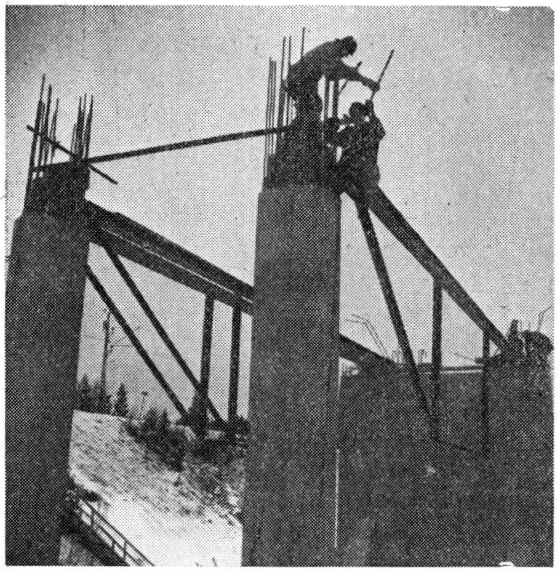

FIG. 10. Mounting of steel trusses on the top of a column

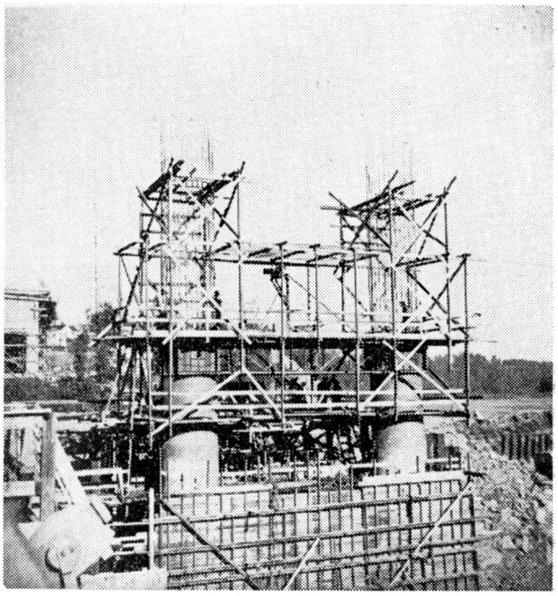

FIG. 11. Columns under construction for a highway bridge across the river Indalsäven at Bergeforsen, Sweden

for carrying the form work for the superstructure under construction should, if possible, be avoided. At the construction of the bridge here described, the space of 14.35 metres between the viaduct columns was overbridged by steel trusses (fig. 9). The steel trusses were placed on sand boxes, located in cupels at the top of the columns (fig. 10). The area of the remaining part of the cross section of the top of the column was sufficient for taking the full bridge deck and traffic load.

The cupels were filled out with concrete afterwards. Compared with conventional scaffolding a further advantage of these steel trusses will be that they make possible to compute deformations with a high grade of accuracy.

When this bridge building had been completed a new contract for a similar bridge over the river Indalsälven at Ragunda was placed with Nya Asfalt AB. With exception for the differences in regard to foundations and the length of the columns, the main difference between these bridges is that the Ragunda Bridge has one additional viaduct span at each side of the river. In view of the good result obtained in the construction of the Vindelälven Bridge the client, the Royal Board of Swedish State Railways, again agreed to the suggestion that slide-forms should be used for the casting of the columns and the slide-form equipment previously used was used once more.

Owing to delays one of the abutment columns had to be concreted in the winter. Before starting the casting a heating shelter was built around and to the full height of the column. Hot air was blown in at the base of the shelter. Because the extending reinforcing irons will cool down the concrete just poured it is, unfortunately, not possible to use

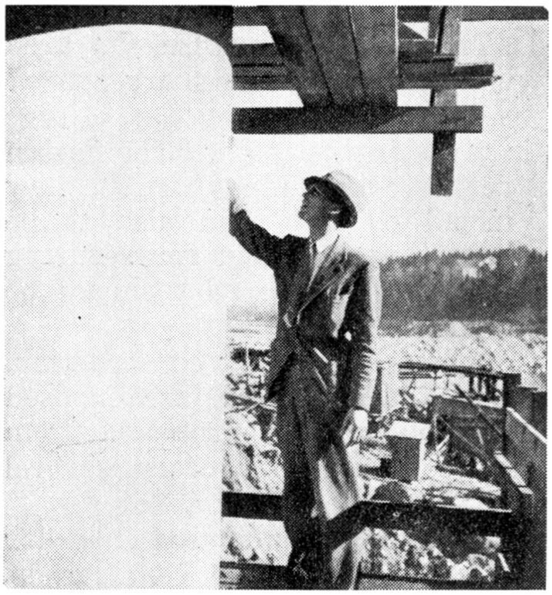

FIG. 12. Examination of the concrete surface of a column at Bergeforsen

FIG. 13. Concrete viaduct at Bergeforsen, Sweden

the simpler method of hanging tarpaulins from the working platform of the slide-form equipment. Great attention and foresight is called for at such a casting in wintertime.

The same method, as described above, was used also in the construction of the highway bridge over the river Indalsälven at Bergeforsen, which was built in the years 1953-1954. The columns of this bridge are 2.0 metres of diameter, and the quality of concrete used was K 300. Fig. 11 shows a pair or columns under construction and fig. 12 a detail of the concrete surface. Even here a pair of columns had to be cast at a low temperature.

The columns of a concrete approach viaduct (fig. 13) placed in rows of three, were also constructed with the aid of slide-forms. The columns are 0.7 metres of diameter. All columns in one row were cast at the same time from a common working platform.

Here too, there were two lifting jacks in use on each column. Because of the small diameter the columns will be susceptible to eccentric loads on the working platform. When constructing similar groups of columns by the aid of slide-forms columns less than 0.65 – 0.70 metres of diameter, therefore, would not be recommended.

SUMMARY

For high bridge columns of concrete the costs for forms and scaffolding are rather high. Through slide-form concreting large savings in material and labour costs can be made and the concrete will be easily accessible to handling.

For a concrete railway bridge, constructed in North Sweden from 1950 to 1952, all support-columns were concreted by means of slide-forms in accordance with the Concretor method. The bridge comprises an arch span, 112 m in length and approach viaducts, whose piers are spaced 14.35 m.

A description of the sliding forms is given in the paper. The design of the abutment columns was modified so as to be adapted to the new method of construction. The binders in all columns had also to be modified. It was necessary to build an auxiliary scaffold for securing the vertical reinforcement bars in position, since these bars should be as long as possible.

The columns are fixed only in the foundations during construction. Therefore, the columns, which are of considerable length, are exposed to buckling in the course of construction, and are not protected from buckling until they are fixed to the bridge deck. For this reason, several columns had to be guyed during the slide casting.

An account is given of the provisional arrangements used in order to render possible the concreting of columns with steel bearings at the base.

Steel trusses resting on the tops of the columns carried the form for the superstructure. These trusses contributed to the utilisation of the possibilities of rationalising the construction produce, since conventional scaffolding was entirely dispensed with in this manner.

For another similar railway bridge, constructed from 1953 to 1955 the columns are concreted by means of slide-forms. One of the columns of this bridge was built in the winter, then the temperature was permanently below 0° C.

The same method has also been used for the columns of a highway bridge.

ZUSAMMENFASSUNG

Die Gerüst- und Schalungskosten bei längern Brückensäulen sind ausgesprochen hoch. Bei Anwendung der Gleitschalung lassen sich bedeutende Material- und Arbeitskosten einsparen; auch wird die Zugänglichkeit für die Betonbearbeitung verbessert.

Bei einer in Nordschweden in den Jahren 1950-52 gebauten Eisenbahnbrücke aus Stahlbeton wurden alle Stütz-Säulen mit Hilfe von Gleitschalungen nach dem Concretorsystem betoniert. Die von einem Bogen überspannte Mittelöffnung dieser Brücke ist 112 m lang, und die Stützen der seitlichen Anschlussviadukte liegen je 14,35 m auseinander.

Die Bemessung der Widerlager-Säulen wurde der neuen Betoniermethode angepasst. Auch die Bügel wurden entsprechend ausgebildet. Um die möglichst langen Vertikaleisen in ihrer richtigen Lage festzuhalten, musste ein besonderes Hilfsgerüst erstellt werden.

Während des Baues sind die ausgesprochen langen Säulen nur am untern Ende eingespannt; demnach sind sie dem Knicken ausgesetzt, bis sie mit der Fahrbahnplatte verbunden werden können. Aus diesem Grunde mussten verschiedene Säulen während des Betonierens seitlich abgestützt werden.

Es folgt eine Beschreibung über die Einrichtungen, die das Betonieren von Säulen mit Stahllagern am untern Säulenende ermöglichten.

Ueber den Säulenköpfen aufliegende Stahltragwerke dienten zur Unterstützung der Fahrbahnschalung und hatten wesentlichen Anteil an der Verwirklichung dieser rationelleren Bauweise.

Bei einer ähnlich gearteten, im 1955 fertiggestellten Eisenbahnbrücke werden die Säulen ebenfalls mit Hilfe der Gleitschalung betoniert. Eine dieser Säulen wurde im Winter erstellt, als die Temperatur ständig unter 0° C lag .

Das gleiche Verfahren kam auch bei Strassenbrücken-Säulen zur Anwendung.

RESUMO

O custo dos andaimes e das cofragens dos pilares de pontes de betão de grande altura é bastante elevado. O emprego de cofragens deslizantes não só permite realizar importantes economias de material e mão de obra como também facilita a manutenção do betão.

Na construção de uma ponte de caminho de ferro no Norte da Suécia, de 1950 a 1952, betonaram-se todos os pilares de apoio com o auxílio de cofragens deslizantes segundo o método «Concretor». A referida ponte compõe-se de um arco de 112 m. de vão e dois viadutos marginais cujos pilares têm uma espaçamento de 14,35 m.

O autor descreve as cofragens deslizantes. O projecto dos pilares dos encontros foi modificado de modo a adaptar-se ao referido método de construção. Os estribos também foram modificados em todos os pilares. Tornou-se ainda necessário construir um andaime auxiliar para suportar as armaduras verticais que deviam ser o mais compridas possível.

Durante a construção, os pilares só estavam ligados às fundações, ficando portanto, devido à sua grande altura, submetidos à encurvadura até ao momento de virem ligar ao tabuleiro. Por esta razão, alguns pilares tiveram de ser suportados durante a betonagem.

O autor relata as disposições provisórias adoptadas para tornar possível a betonagem dos pilares com articulações metálicas na base.

Vigas metálicas trianguladas, apoiadas no topo dos pilares, suportavam a cofragem da superestrutura. Essas vigas contribuiram para o aproveitamento das possibilidades de racionalização do método de construção, pois permitiram dispensar completamente os andaimes habituais.

Na construção de outra ponte semelhante executada de 1953 a 1955 foram os pilares também betonados com cofragens deslizantes. Um dos

pilares foi inteiramente betonado durante o inverno com temperaturas permanentemente negativas.

O mesmo método foi também utilizado para construir os pilares de uma ponte de estrada.

RÉSUMÉ

Le prix des échaffaudages et des coffrages des piliers de ponts en béton de grande hauteur est assez elevé. L'emploi de coffrages glissants permet non seulement une sérieuse économie de matériaux et de main d'oeuvre mais encore rend la manutention du béton plus aisée.

Lors de la construction d'un pont-rail dans le Nord de la Suède, de 1950 à 1952, tous les piliers d'appui ont été bétonnés à l'aide de coffrages glissants selon la méthode «Concretor». Ce pont comprend un arc de 112 m de portée et des viaducs d'approche dont les piliers sont espacés de 14.35 m.

L'auteur décrit les coffrages glissants. Le projet des piliers des naissances a été modifié afin de pouvoir les adapter à la nouvelle méthode de construction. Les étriers de tous les piliers ont également dû être modifiés. Il a été nécessaire de construire un échafaudage auxiliaire pour guider les armatures verticales qui devaient être aussi longues que possible.

Pendant la construction, les piliers n'étaient fixés qu'aux fondations, ce qui, du fait de leur grande hauteur, les soummettait au flambement jusqu'au moment de les relier au tablier. Pour cette raison, certains piliers ont dû être étayés pendant le bétonnage.

L'auteur décrit les dispositions provisoires adoptées pour rendre possible le bétonnage des piliers munis d'articulations métalliques à leur base.

Des poutres métalliques triangulées, appuyées sur le sommet des piliers, soutenaient les coffrages de la superstructure. Ces poutes ont contribué à l'utilisation des possibilités de rationalisation du mode de construction en permettant d'éviter entièrement les échafaudages conventionnels.

Des coffrages glissants ont été également utilisés dans la construction d'un autre pont-rail éxécuté de 1953 à 1955. L'un des piliers a été completement bétonné en plein hiver par des températures constamment en dessous de zéro.

La même méthode a également été employée pour la construction des piliers d'un pont-route.

VIa4

Berechnung der Stahlbeton Fertigteil-Konstruktionen verbunden an Ort und Stelle

Cálculo de estruturas de betão armado com elementos prefabricados, ligados no local da obra

Calcul des constructions en béton armé exécutées en éléments préfabriqués montés sur place

Calculation of reinforced concrete structures incorporating precast elements erected at the site

BÉLA GOSCHY

Budapest

1.) ***Einleitung.***

Die Wirkungsweise der an Ort und Stelle verbundenen Stahlbeton-Fertigteil-Systeme unterscheidet sich von den monolythischen Stahlbeton-Konstruktionen und dadurch wird die Statik und Bemessung der Stahlbeton-Verbundkonstruktionen wesentlich beeinflusst. Ausserdem müssen noch die Zeitveränderlichkeit, die plastischen Verformungen (Kriechen und Schwinden) des Betons, sowie die Montage dieser Konstruktionen bei den Rechnungen beachtet werden. Die Untersuchung dieses Verhaltens ist das Ziel der vorliegenden Arbeit.

Die Stahlbeton-Fertigteilbauten können ohne Gerüst, als selbsttragende Tragwerke ausgebildet werden. In diesem Falle wird das Gesamtgewicht (Fertigteil- und Ortsbetongewicht) allein von dem *Fertigteil* getragen. Sie können aber völlig oder nur teilweise auf Gerüst gebaut werden, in diesem Falle wird das Gesamtgewicht, einschliesslich die Nutzlast von dem *Verbund-System* getragen. Die Rechnungsverfahren werden auf diese zwei Grundfälle ausgerichtet.

Der mit dem Ortsbeton zusammenwirkende Fertigteilbalken bildet innerlich ein statisch unbestimmtes System. Bei der Vermittlung der unbekannten statischen Grössen -wie es bei den statisch unbestimmten Stahlbetonsystemen üblich ist- rechnen wir mit dem homogenen Betonquerschnitt.

2.) ***Bezeichnungen.***

$F_1,\ J_1,\ W_1,\ i_1^2 = \dfrac{J_1}{F_1}$	Querschnittsgrössen des Fertigteils
$F_2,\ J_2,\ W_2,\ i_2^2 = \dfrac{J_2}{F_2}$	Querschnittsgrössen des Ortsbetonteils
$P_o,\ M_o$	äussere Druckkraft, Biegungsmoment
$P_1,\ M_1,\ N_o$	Last- und Momentenanteil des Fertigteils
$P_2,\ M_2,\ N_o$	Last- und Momentenanteil des Ortsbetonteils
$P_B,\ M_B$	Bruchkraft, Bruchmoment
$\varphi_1,\ \varphi_{1,t};\ \varphi_{1\infty}$	Kriechzahl des Fertigteils
$\varphi_2,\ \varphi_{2,t};\ \varphi_{2\infty}$	Kriechzahl des Ortsbetons
$k_1,\ k_2$	Prismenfestigkeit des Betons
σ_f	Fliessgrenze des Stahls
$\sigma_{10},\ \sigma_{20}$	zulässige Betondruckspannung bei mittigem Druck
$\sigma_{1b},\ \sigma_{2b}$	zulässige Betondruckspannung bei reiner Biegung.

3.) ***Einfluss des Kriechens und des Schwindens.***

Die Fertigbauteile und die an Ort und Stelle betonierten Trägerteile kriechen nach verschiedenen Gesetzen, die von dem Betonalter und der Betongüte, wie auch von dem Beginn der Belastung abhängig sind.

Der Stahbeton-Fertigteil beginnt mit Δt früher zu kriechen als der Ortsbeton, nach folgendem Gesetz:

$$\varphi_1,\ t_1 = \varphi_{1\infty}\left(1 - e^{-t_1}\right) \tag{1}$$

Die Kriechlinie des Ortsbetons hat die folgende Form:

$$\varphi_2,\ t_2 = \varphi_{2\infty}\left(1 - e^{-t_2}\right) \tag{2}$$

Die beiden Zeitpunkte stehen in folgendem Zusammenhang:

$$t_1 = t_2 + \Delta t$$

Wenn wir den Zeitpunkt der Wirkung der ständigen Last mit t_{10} bzw. t_{20} bezeichnen, so kriechen die einzelnen Trägerteile nach:

$$\bar{\varphi}_1,\ t_1 = \varphi_1,\ t_1 - \varphi_1,\ t_{10} = \varphi_{1\infty}\left(e^{-t_{10}} - e^{-t_1}\right) \tag{3}$$

$$\bar{\varphi}_2,\ t_2 = \varphi_2,\ t_2 - \varphi_2,\ t_{20} = \varphi_{2\infty}\left(e^{-t_{20}} - e^{-t_2}\right) \quad (4)$$

Da zwischen den beiden Funktionen eine Affinität besteht, kann man die Kriechfunktion des Fertigteils als Funktion des Ortsbetonteils ausdrücken:

$$t_{10} = t_{20} + \Delta t;\ \nu = \frac{\varphi_{1\infty}}{\varphi_{2\infty}} \text{ und } \mathcal{K} = e^{-\Delta t}$$

Die Funktionen des Kriechens:

$$\bar{\varphi}_1,\ t_1 = \nu\mathcal{K}\ \varphi_{2\infty}\left(e^{-t_{20}} - e^{-t_2}\right) = \nu\mathcal{K}\ \bar{\varphi}_2,\ t_2 \quad (5)$$

$$\bar{\varphi}_2,\ t_2 = \varphi_{2\infty}\left(e^{-t_{20}} - e^{-t_2}\right) \quad (6)$$

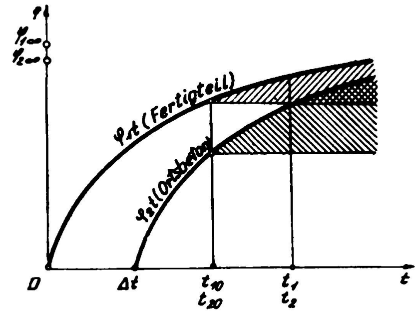

ABB. 1. Kriechkurven der Trägerteile. Die Belastung beginnt zur Zeit t_{10} bzw. t_{20}

Bei der Berücksichtigung des Kriechens kann man zwei Sonderfälle unterscheiden:

a.) Ist der Fertigteil nicht mehr kriechfähig, so erleidet er unter der ständigen Last keine Verformungen.
In diesem Falle ist $\Delta t = \infty$ und $\bar{\varphi}_1,\ t_1 = 0$

$$\bar{\varphi}_2,\ t_2 = \varphi_{2\infty}\left(e^{-t_{20}} - e^{-t_2}\right)$$

b.) Wird der Ortsbeton sofort nach dem Erhärten belastet, dann ist $t_{10} = \Delta t$ und $t_{20} = 0$, und die Kriechfunktionen:

$$\bar{\varphi}_1,\ t_1 = \nu\mathcal{K}\ \varphi_{2\infty}\left(1 - e^{-t_2}\right) = \nu\mathcal{K}\ \bar{\varphi}_2,\ t_2 \quad (5\text{-a.})$$

$$\bar{\varphi}_2,\ t_2 = \varphi_{2\infty}\left(1 - e^{-t_2}\right) \quad (6\text{-a.})$$

Bei der Berücksichtigung des Kriechens werden wir die Aenderung des E_b -als unbedeutende- vernachlässigen.
Das Schwinden ist eine dem Kriechen affine Erscheinung; so können wir die Schwindfunktion proportional der Kriechfunktion auffassen. Das Verhältnis zwischen den beiden Funktionen ist durch $\frac{\varepsilon_\infty}{\varphi_\infty}$ gegeben, wo $\varepsilon_\infty = \omega\ T^o$ ist. Das Schwindmass wird wegen der Schwindbewehrung vermindert angenommen.
Die Schwindfunktion des Fertigteilträgers:

$$\bar{\varepsilon}_1,\ t_1 = \nu\mathcal{K}\frac{\varepsilon_\infty}{\varphi_{1\infty}}\ \bar{\varphi}_2,\ t_2 = \mathcal{K}\ \bar{\varepsilon}_2,\ t_2 \quad (7)$$

Die Schwindfunktion des Ortsbetonträgerteils:

$$\bar{\varepsilon}_{2,\,t_2} = \frac{\varepsilon_\infty}{\varphi_{2\infty}}\, \bar{\varphi}_{2,\,t_2} \tag{8}$$

4.) Die Bestimmung der inneren Kräfte des Verbundquerschnittes.

Bei der Berechnung der Trägerteile nehmen wir einen vollkommenen Verbund an.

4.1 *Mittig gedrückte Verbundquerschnitte.*

4.11 Die Kraft P_0 greift nur am Fertigteil an. In dem Zeitpunkte $t = 0$ wirkt auf den Fertigteilträger die Teilkraft, $P_1 = P_0$ und auf den Ortsbetonträger die Teilkraft $P_2 = 0$.

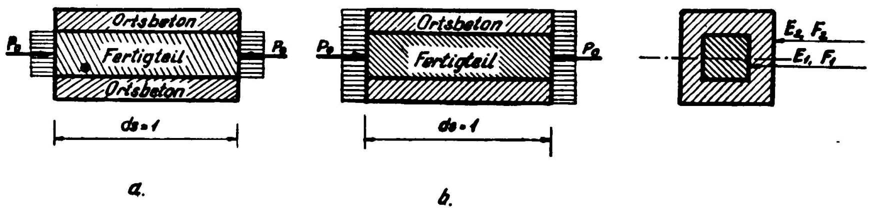

ABB. 2. Zusammengesetzter und an Ort und Stelle verbundener Druckstab. Abb. 3-a. Die Last P_0 wird nur von dem Fertigteil getragen. Abb. 3-b. Die Last P_0 wird von dem Verbundträger getragen

4.12 Infolge der Wirkung der Dauerlast P_0 treten plastische Verformungen auf. Dis Kräfteumlagerung im Verbundquerschnitt bestimmen wir durch die Auflösung des Verbundes zwischen dem Fertigteil und Ortsbeton und durch die Einführung der unbekannten Kraft X_t. Dann können wir die zwischen den zwei Teilen etstehende relative Verschiebung auf eine Säulenlänge $ds = 1$, zwischen dem Zeitpunkt t bis zu $t + dt$, als Null annehmen.

Die Formänderungsgleichung ist:

$$X_t\left(a_{11}^{1}\,\frac{d\varphi_1}{dt_2} + a_{11}^{2}\,\frac{d\varphi_2}{dt_2}\right) + \frac{dX_t}{dt_2}\left(a_{11}^{1} + a_{11}^{2}\right) + a_{10}^{1}\,\frac{d\varphi_1}{dt_2} = 0$$

Die einzelnen Werte:

$$a_{11} = a_{11}^{1} + a_{10}^{2} = \frac{1}{E_1 F_1} + \frac{1}{E_2 F_2}$$

$$a_{10} = a_{10}^{1} + a_{10}^{2} = \frac{P_0}{E_1 F_1} + 0$$

Die Gleichung wird mit $dt_2/d\varphi_2$ multipliziert; dann durch die Einsetzung von $\bar{\varphi}_{1},\, t_1 = \nu\mathcal{K}\, \bar{\varphi}_{2},\, t_2$ bekommen wir:

$$X_t\left(\nu\mathcal{K} a_{11}^{1} + a_{11}^{2}\right) + \frac{dX_t}{d\varphi 2}\left(a_{11}^{1} + a_{11}^{2}\right) + \nu\mathcal{K} a_{10}^{1} = 0$$

Die Einführung der folgenden Beziehungen:

$$\alpha = \frac{\nu \mathscr{K} a_{11}^{1} + a_{11}^{2}}{a_{11}}; \quad \beta = \frac{\nu \mathscr{K} a_{10}^{1} + a_{10}^{2}}{a_{11}} = -\overline{X}$$

$$\overline{X} = \nu \mathscr{K} P_o \frac{E_2 F_2}{E_1 F_1 + E_2 F_2}$$

gibt $\dfrac{dX_t}{d\varphi_2} + \alpha X_t - \overline{X} = 0$ (9)

Die Lösung der Gleichung (9):

a.) Wenn $t_{20} \neq 0$ (mit Schonzeit)

$$X_t = \frac{\overline{X}}{\alpha}\left(1 - \lambda\, e^{-\alpha \varphi_{2\infty}}\right) \tag{9-a.}$$

$$\lambda = e^{+\alpha \varphi t_{20}}$$

b.) Wenn $t_{20} = 0$ (ohne Schonzeit)

$$X_t = \frac{\overline{X}}{\alpha}\left(1 - e^{-\alpha \varphi_{2\infty}}\right) \tag{9-b.}$$

Zur Zeit $t = \infty$ entsteht im Ortsbeton die Druckkraft $P_{2t} = X_t$ und in dem Fertigteilträger die Druckkraft $P_{1t} = P_o - X_t$.

4.13 Die Kraft P_o greift den Verbundquerschnitt an. Auf Grund der elastischen Formänderungen wird zur Zeit $t = 0$ der Fertigteilträger mit dem Kraftanteil P_1 und der Ortsbetonträger mit P_2 belastet:

$$P_1 = P_o \frac{E_1 F_1}{E_1 F_1 + E_2 F_2}; \quad P_2 = P_o \frac{E_2 F_2}{E_1 F_1 + E_2 F_2}$$

4.14 Infolge der Dauerbelastung P_o entstehen plastische Verformungen, sowohl in Fertigteilträger wie auch im Ortsbetonträger.
Die Kraft X_t kann man nach dem Verfahren 4.12 bestimmen.
Das Belastungsglied in Funktion der Zeit:

$$a_{10}^{2} \frac{d\varphi_2}{dt_2} - a_{10}^{1} \frac{d\varphi_1}{dt_2} = \frac{P_2}{E_2 F_2} \frac{d\varphi_2}{dt_2} - \frac{P_1}{E_1 F_1} \frac{d\varphi_1}{dt_2} = \frac{P_o}{E_1 F_1 + E_2 F_2} (1 - \nu \mathscr{K}) \frac{d\varphi_2}{dt_2}$$

$$\beta = \frac{a_{10}^{2} - \nu \mathscr{K} a_{10}^{1}}{a_{11}} = -\overline{X}; \quad \overline{X} = (1 - \nu \mathscr{K}) \frac{P_o}{E_1 F_1 + E_2 F_2} : \frac{1}{E_1 F_1} + \frac{1}{E_2 F_2}$$

Die Lösung der Gleichung gibt für X_t folgende Werte:
a.) Wenn $t_{20} \neq 0$ (mit Schonzeit)

$$X_t = \frac{\overline{\overline{X}}}{\alpha}\left(1 - \lambda\, e^{-\alpha \varphi_{2\infty}}\right)$$

b.) Wenn $t_{20} = 0$ (ohne Schonzeit)

$$X_t = \frac{\overline{X}}{\alpha}\left(1 - e^{-\alpha \varphi_{2\infty}}\right)$$

Zur Zeit $t = \infty$ wird der Fertigteilträger mit $P_{1t} = P_1 + X_t$, und der Ortsbetonträger mit $P_{2t} = P_2 - X_t$ belastet.
4.15 Die Berechnung des Schwindens:

Das relative Schwinden zwischen den Trägern im Zeitpunkte $t = \infty$ ist $\varepsilon = \varepsilon_\infty \left(1 - e^{-t_{10}}\right)$, wenn der Altersunterschied der zwei Trägerteile $\Delta t = t_{10}$ ist.
Die Schwindkraft wird durch das Kriechen wesentlich vermindert und im Zeitpunkte $t = \infty$ erreicht sie den Wert nach der Gleichung:

$$X a_{11} + a_{10} = 0\,; \qquad a_{11}^{1} + a_{11}^{2} = \frac{1}{E_1 F_1} + \frac{1}{E_2 F_2}$$

$$a_{10} = \varepsilon_\infty \left(1 - e^{-t_{10}}\right) = \varepsilon_\infty \left(1 - e^{-\Delta t}\right);\ X = \varepsilon_\infty\,(1 - \mathcal{K})\,;\ \frac{E_1 F_1 \times E_2 F_2}{E_1 F_1 + E_2 F_2}$$

$$X_t = \frac{X}{\alpha \varphi_{2\infty}}\left(1 - e^{-\alpha \varphi_{2\infty}}\right) \tag{10}$$

4.2 *Biegung des zusammengesetzten Balkens.*

Bei Biegung des zusammengesetzten Balkens werden zwei Fälle untersucht:

a.) die Schwerlinien der beiden Querschnitte fallen zusammen,
b.) die Schwerlinien der beiden Querschnitte sind verschieden.

Die Untersuchung der Träger des Falles *a.)* können wir nach Punkt 4.1 ausführen, wenn wir in die Formeln statt P, M und statt F, J einsetzen.
Untersuchung der Träger von Punkt *b.)*:
4.21 Das Biegungsmoment M_o belastet den Verbundträger.

Die Berechnung der Innenkräfte zur Zeit $t = 0$, können wir in folgenden Schritten durchführen (Bezeichnungen nach Abb. 3):

a.) Berechnung der Null-Linie des Verbundträgers:

$$a_2 = a\,\frac{E_1 F_1}{E_1 F_1 + E_2 F_2}\,; \qquad a_1 = a - a_2$$

b.) Bestimmung der Kernpunkte unter der Voraussetzung, dass die Spannungen in der neutralen Achse gleich Null sind.

$$b_2 = \frac{i_2^2}{a_2}\,; \qquad b_1 = \frac{i_1^2}{a_1}$$

c.) Berechnung des Hebelarmes des inneren Kräftepaares:

$$c = a + b_2 + b_1$$

d.) Bestimmung der inneren Kräfte: $N_o = M_o / c$; $M_1 = N_o\, b_1$;

$$M_2 = N_o\, b_2$$

e.) Bemessung der Trägerteile. Im Besitz der Normalkräfte und Biegungsmoment wird der Stahlbeton-Fertigteil als ausmittig gezogener und der Ortsbetonträgerteil als ausmittig gedrückter Balken bemessen.

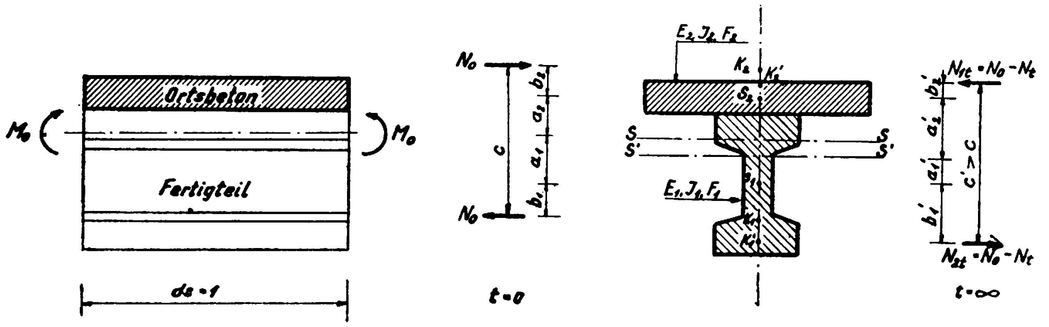

Abb. 3. Innere Kräfte in einem Stahlbeton-Fertigteilträger mit Druckplatte zur Zeit $t = 0$ und $t = \infty$

4.22 Die plastischen Verformungen verursachen eine Umlagerung der inneren Kräfte. Die Veränderung der Normalkraft wird aus der Formänderungsgleichung gerechnet:

$$N_t \left(\frac{1}{E_1 F_1} \frac{d\varphi_1}{dt_2} + \frac{1}{E_2 F_2} \frac{d\varphi_2}{dt_2} \right) + \frac{dN_t}{dt_2} \left(\frac{1}{E_1 F_1} + \frac{1}{E_2 F_2} \right) =$$

$$= N_o \left(\frac{1}{E_1 F_1} \frac{d\varphi_1}{dt_2} + \frac{1}{E_2 F_2} \frac{d\varphi_2}{dt_2} \right) - \frac{M_1\, a}{E_1 J_1} \frac{d\varphi_1}{dt_2}$$

Mit $N_o = \dfrac{M_o}{c}$ und $M_1 = N_o b_1$

Die Lösung der Differenzialgleichung:

$$N_t = \left(1 - \lambda\, e^{-\alpha\, \varphi_{2\infty}}\right) \frac{\beta}{\alpha} N_o \qquad (11)$$

Wenn $\alpha = \dfrac{\nu \mathcal{K} a_{11}^{1} + a_{11}^{2}}{a_{11}^{1} + a_{11}^{2}}$; $\beta = \alpha - \alpha'$; $\alpha' = \dfrac{\nu \mathcal{K} a_{10}^{1}}{a_{11}^{1} + a_{11}^{2}}$;

$$a_{10}^{1} = \frac{ab_1}{E_1 J_1}; \quad \beta = (1 - \nu \mathcal{K}) \frac{E_1 F_1}{E_1 F_1 + E_2 F_2}$$

Zur Zeit $t = \infty$ wird die Normalkraft: $N_{1t} = N_{2t} = N_o - N_t$

Da das Biegungsmoment M_o unverändert bleibt, wächst der Hebelarm von c zu c'. Die Grösse des c' ist durch die Beziehung gegeben:

$$c' N_{1t} = c N_o = M_o \text{ und davon } c' = \frac{M_o}{N_{1t}}$$

Infolge des Kriechens verschiebt sich die Null-Linie des zusammengesetzten Querschnittes in Richtung des Fertigteils und die Lage der Kernpunke ändert sich nach Abb. 3. Die Umlagerung der charakteristischen Punkte des Querschnittes kann man leicht bestimmen aus den Beziehungen:

$$a + b'_1 + b'_2 = c' \tag{12}$$

$$a = a'_1 + a'_2 \tag{13}$$

$$a'_1 \, b'_1 = i_1^2 \tag{14}$$

$$a'_2 \, b'_2 = i_2^2 \tag{15}$$

Zur Zeit $t = \infty$ sind also die inneren Kräfte:

$$N_{1t} = N_{2t}; \quad M_{1t} = N_{1t} \, b'_1 \text{ und } \quad M_{2t} = N_{1t} b'_2$$

4.23 Die Last wird nur von dem Stahlbeton-Fertigteil getragen. Die Belastung des Stahlbeton-Fertigteiles wird zur Zeit $t = 0$ $M_1 = M_o$ und der an Ort und Stelle betonierte Trägerteil wird unbelastet, $M_2 = 0$.

4.24 Um die inneren Kräfte infolge plastischer Verformungen zu bestimmen, lösen wir die Differenzialgleichung:

$$N_t \left(\frac{1}{E_1 F_1} \frac{d\varphi_1}{dt_2} + \frac{1}{E_2 F_2} \frac{d\varphi_2}{dt_2} \right) + \frac{dN_t}{dt_2} \left(\frac{1}{E_1 F_1} + \frac{1}{E_2 F_2} \right) =$$

$$= \frac{M_o \, a}{E_1 J_1} \frac{d\varphi_1}{dt_2} = \frac{N_o \, b_1 \, a}{E_1 J_1} \frac{d\varphi_1}{dt_2}; \quad N_o = \frac{M_o}{b_1}$$

wird $$N_t = \left(1 - \ell\, e^{-\alpha \varphi_{2\infty}}\right) \frac{\beta}{\alpha} N_o \tag{16}$$

$$\beta = \nu \mathcal{K} \text{ aus } \beta = \alpha' = \frac{\nu \mathcal{K} a_{10}^{1}}{a_{11}^{1} + a_{11}^{2}}; \quad a_{10}^{1} = \frac{ab_1}{E_1 J_1}$$

Die Kraftanteile des Verbundquerschnittes zur Zeit $t = \infty$ werden: im Ortsbeton die Druckkraft N_t und das Moment $M_{2t} = + N_t b_2$; im Fertigteil die Zugkraft N_t und das Moment $M_{1t} = M_o - N_t (a + b_2)$.

4.25 Infolge Schwindens des Betons entsteht laut Punkt 4.15 eine Normalkraft X und ein Moment Xa. Die Randspannungen werden dann im Stahlbeton-Fertigteil:

$$\sigma_1 = - \frac{X}{F_1} + \frac{Xa}{cF_1} + \frac{Xab_1}{c\,W_1} \tag{17}$$

und im Ortsbeton:

$$\sigma_2 = + \frac{X}{F_2} - \frac{Xa}{cF_2} - \frac{Xab_2}{c\,W_2} \tag{18}$$

Die Schwindspannungen werden durch die plastischen Verformungen wesentlich vermindert (Gl. 10) und zur Zeit $t = \infty$ erreichen sie die Werte:

$$\sigma_1 = - \frac{X_t}{F_1} + \frac{M_t}{cF_1} + \frac{M_t\,b_1}{c\,W_1} \tag{19}$$

$$\sigma_{2t} = + \frac{X_t}{F_2} - \frac{M_t}{cF_2} - \frac{M_t\,b_2}{c\,W_2} \tag{20}$$

wo $X_t = \frac{X}{\alpha\varphi_{2\infty}} \left(1 - e^{-\alpha\varphi_{2\infty}}\right)$ und $M_t = X_t\,a$.

5.) Bruchberechnung zusammengesetzter Stahlbeton-Konstruktionen. Bruchsicherheit.

5. 1 Mittig belastete Säulen (ohne Knickgefahr).

Die Bruchlast eines symmetrisch zusammengesetzten und auf mittigen Druck beanspruchten Stabes wird durch das bekannte Additionsgesetz berechnet. Die Bruchlast des zusammengesetzten Stabes ist gleich der Summe der Bruchlasten der einzelnen Stäbe.

$$P_B = P_{1B} + P_{2B}, \quad \text{wo } P_{1B} = k_1\,F_{1b} + \sigma_t\,F_{1s} \tag{21}$$

$$P_{2B} = k_2\,F_{2b} + \sigma_f\,F_{2s} \tag{22}$$

Ferner werden wir nur mit dem Betonquerschnitt rechnen; dementsprechend sind die Bruchlasten:

$$P_{1B} = k_1\,F_1\,; \quad P_{2B} = k_2\,F_2$$

Der Druckstab wird auf Grund der zulässigen Last P_o bemessen.

$$P_o = P_{10} + P_{20} = \frac{k_1}{n}\,F_1 + \frac{k_2}{n}\,F_2 = \sigma_{10}\,F_1 + \sigma_{20}\,F_2$$

Die Bruchsicherheit ist: $n = \dfrac{P_B}{P_o}$.

Belasten wir nun den Fertigteil mit einer Druckkraft P_1, werden infolgedessen die Spannungen im Fertigteil: $\sigma_1 = \dfrac{P_1}{F_1}$.

Bestimmen wir in diesem Zustande die Drucklast P_o' des Verbundquerschnittes sodass infolge der Lasten P_o' und P_1 die Spannungen im Fertigteil den zulässigen Wert σ_{10} erreichen sollen. Zur Vereinfachung der Berechnung nehmen wir an, dass der Beton ein idealplastischer Baustoff ist Wegen $P_o' = P_{10}' + P_{20}'$, ist

$$\sigma_1 + \frac{P_o'}{F_1} \cdot \frac{E_1 F_1}{E_1 F_1 + E_2 F_2} = \sigma_{10}, \text{ oder}$$

$$P_1 + P_o' \cdot \frac{E_1 F_1}{E_1 F_1 + E_2 F_2} = P_{10}; \quad P_o' = (P_{10} - P_1) \frac{E_1 F_1 + E_2 F_2}{E_1 F_1}$$

Um die zulässige Last P_o des Verbundquerschnittes zu erreichen, steigern wir die Last mit ΔP. Der Wert von ΔP hängt von der zulässigen Spannung σ_{20} des an Ort und Stelle betonierten Trägerteiles ab, — und sie ist proportional mit der Vorbelastung P_1.

$$\Delta P + P_o' \cdot \frac{E_2 F_2}{E_1 F_1 + E_2 F_2} = P_{20};$$

$$\Delta P = P_{20} - \frac{E_2 F_2}{E_1 F_1} (P_{10} - P_1)$$

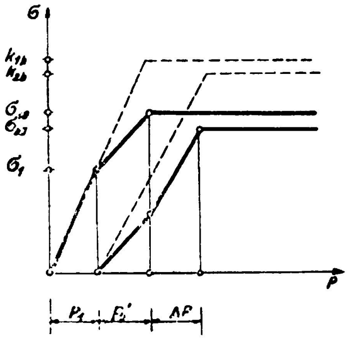

ABB. 4. Verlauf der Druckspannungen in einem zusammengesetzten Querschnitt mit Vorbelastung P_1

Die Bruchsicherheit in dem auf zulässige Spannungen bemessenen Querschnitt wird nun:

$$n' = n \left(1 + \frac{\Delta P}{P_b}\right) \tag{23}$$

5.2 Bruchberechnung des auf Biegung beanspruchten Verbundträgers, wenn die Trägerteile dieselbe Schwerlinie haben.

Das Betonmoment des Verbundquerschnittes ist bei reiner Biegung im Bruchzustand: $M_B = M_{1B} + M_{2B}$; nach den Versuchergebnissen:

$$M_{1B} = k_1 b_1 h_1{}^2/3 \tag{24}$$ $$M_{2B} = k_2 b_2 h_2{}^2/3 \tag{25}$$

Um die Grössenordnung der Sicherheit zu bestimmen, werden wir ein solches Bemessungsverfahren benutzen, das die Elastizitätstheorie und die Bruchtheorie vereinigt. Zu diesem Zwecke werden wir die Versuchsergebnisse der Bruchtheorie auf die Elastizitätstheorie übertragen.

Das Verfahren ist n-frei und auch bei homogenen Querschnitten brauchbar. Die Richtigkeit des Verfahrens wurde bei Rechteckquerschnitten in vollem Masse bewiesen (1).

Da wir die Rechnungen auf den homogenen Betonquerschnitt beziehen, wird das Bruchmoment beider Trägerteile:

$$M_{1B} = K_1 W_1 ; \qquad M_{2B} = K_2 W_2$$

Rechteckquerschnitte haben den Formfaktor k/K laut den Versuchen von der Grössenordnung 0,5-0,7.

Das zulässige Betonmoment ist:

$$M_o = M_{10} + M_{20} = \frac{K_1}{n} W_1 + \frac{K_2}{n} W_2 = \sigma_{1b} W_1 + \sigma_{2b} W_2$$

Der Sicherheitsfaktor ist: $n = \dfrac{M_B}{M_o}$.

Wenn der Stahlbeton-Fertigteil eine Vorbelastung M_1 bekommen hat, kann man wie im Punkt 5.1 den Wert ΔM bestimmen.

$$\Delta M = M_{20} - \frac{E_2 J_2}{E_1 J_1} (M_{10} - M_1)$$

Und so erhalten wir die Bruchsicherheit des Verbundträgers, bemessen auf Grund der Elastizitätstheorie:

$$n' = n \left(1 + \frac{\Delta M}{M_b} \right) \tag{26}$$

Bemerkung: Die Bruchsicherheit ändert sich, wenn wir beim Bemessen den Einfluss des Schwindens und Kriechens auch in Rechnung setzen.

5.3 Bruchberechnung des auf Biegung beanspruchten Verbundträgers, wenn die Trägerteile verschiedene Schwerlinien haben. Bruchsicherheit.

Nach der Plastizitätstheorie ist der Additionssatz auch für das Bruchmoment gültig. Das Bruchmoment des Stahlbetonverbund-Plattenbalkens ist, wenn $x = 0.5\ h$ und v (die Plattendicke) $< x$, und b_1 konstant ist:

$$M_B = M_{1B} + M_{2B} = k_1 b_1 \left(\frac{h}{2} - v \right) \frac{3h - 2v}{4} + k_2 b_2 v \left(h - \frac{v}{2} \right)$$

$$\text{wenn } h = h_1 + v \text{ und } v = \alpha\, h_1; \quad h = h_1 (1 + \alpha)$$

$$M_{1B} = k_1 b_1 h_1^2 \left(\frac{3 - 2\alpha - \alpha^2}{8} \right); \quad M_{2B} = k_2 b_2 v^2 \left(\frac{1 + 0.5\alpha}{\alpha} \right)$$

(1) Hognestad, E.: Univ. of Illinois Bull. Series n°. 399, vol. 49, n°. 22.

Da die zulässigen Spannungen $\sigma_{10} = \frac{k_1}{n}$; $\sigma_{20} = \frac{k_2}{n}$ sind, die zulässigen Momente:

$$M_{10} = \sigma_{10}\, b_1\, h_1^2 \left(\frac{3 - 2\alpha - \alpha^2}{8}\right) \tag{27}$$

$$M_{20} = \sigma_{20}\, b_2\, v^2 \left(\frac{1 + 0.5\,\alpha}{\alpha}\right) \tag{28}$$

Die Berechnung wird an ausmittig belasteten Querschnitten gezeigt. Da zwischen der Normalkraft N_B und dem Moment M_B im Bruchzustand ein geradliniger Zusammenhang besteht, können wir das Gesetz in die Form bringen:

$$\frac{N}{N_{1B}} + \frac{M}{M_{1B}} = 1\,; \quad \frac{N}{N_{2B}} + \frac{M}{M_{2B}} = 1$$

$$\frac{N}{\sigma_{10}\, F_1} + \frac{M}{\sigma_{1b}\, W_1} = 1\,; \quad \frac{N}{\sigma_{20}\, F_2} + \frac{M}{\sigma_{2b}\, W_2} = 1$$

Wenn $\frac{\sigma_{10}}{\sigma_{1b}} = \frac{\sigma_{20}}{\sigma_{2b}} = 0{,}5$ dann

$$M_{10} = \sigma_{1b}\, W_1 \frac{1}{1 + \frac{h_1}{3b_1}} = \sigma_{1b} \frac{b_1\, h_1^2}{6} \left(\frac{1}{1 + \frac{h_1}{3b_1}}\right) \tag{29}$$

$$M_{20} = \sigma_{2b}\, W_2 \frac{1}{1 + \frac{v}{3b_2}} = \sigma_{2b} \frac{b_2\, v^2}{6} \left(\frac{1}{1 + \frac{v}{3b_2}}\right) \tag{30}$$

Die Bedingung, dass die Biegungsmomente aus der Gl. (27, 29) und (28, 30) gleich sein müssen, gibt den Wert des Formfaktors β_1 und β_2

$$\beta_1 = \frac{\sigma_{10}}{\sigma_{1b}} = \frac{4}{3}\, \frac{1}{3 - 2\alpha - \alpha^2}\, \frac{1}{1 + \frac{h_1}{3b_1}}$$

$$\beta_2 = \frac{\sigma_{20}}{\sigma_{2b}} = \frac{\alpha}{6\,(1 + 0.5\,\alpha)}\, \frac{1}{1 + \frac{v}{3b_2}}$$

Die auf Grund der Elastizitätstheorie bemessenen Träger sind nur bis zum Erreichen der zulässigen Randspannungen in Anspruch genommen. Die Randspannungen:

$$\sigma_{1b} = \frac{\sigma_{10}}{\beta_1} \text{ und } \sigma_{2b} = \frac{\sigma_{20}}{\beta_2}$$

Wenn der Fertigteil infolge Montierung mit M_1 vorbelastet ist, kann die zusätzliche Beanspruchung ΔM in der folgenden Form ausgedrückt werden:

$$\Delta M = M_{20} - \frac{4}{3}(M_{10} - M_1)\frac{b_2 + \frac{W_2}{F_2}}{b_1 + \frac{W_1}{F_1}} = M_{20} - \frac{4}{3}(M_{10} - M_1)\frac{b_2 + \frac{v}{6}}{b_1 + \frac{h_1}{6}}$$

Wenn $b_1 = \frac{h_1}{6}$ ist, so bleibt die Randlinie des Fertigteilträgers spannungslos. In diesem Falle steht die Null-Linie des Verbundträgers in der Grenzlinie der zwei Querschnitte. Dann ist $b_2 = \frac{v}{6}$ und somit $\Delta M = M_{20}$. Wenn $b_1 > \frac{h_1}{6}$ ist, liegt also die Null-Linie des Verbundträgers in dem Fertigteil, so ist $\Delta M < M_{20}$.

Wenn $b_1 < \frac{h_1}{6}$, liegt also die Null-Linie in der Druckplatte, ändert sich das Vorzeichen des zweiten Gliedes und ΔM wird $\Delta M > M_{20}$. Die Zugspannungen mindern den Wert der Vorbelastung im Steg.

Die Werte M_{10} und M_{20} werden mit Hilfe der Formfaktoren aus M_{1b} und M_{2b} gerechnet, und die Bruchmomente:

$$M_{1B} = n\, M_{10}\,; \qquad M_{2B} = n\, M_{20}\,.$$

6.) *Statisch unbestimmte Stahlbeton-Verbundkonstrukionen.*

Die statisch unbestimmten Stahlbeton-Verbundkonstruktionen kann man mit den bekannten Methoden lösen. Im Rahmen dieser Arbeit werden einige Gesichtspunkte zur Vereinfachung der Berechnung angegeben.

6. 1 Die Verformungsgleichungen eines unbestimmten Tragwerkes im Zeitpunkt $t = 0$, sind:

$$a_{11}\, X_1 + a_{21}\, X_2 + \ldots\ldots\ldots\ldots\ldots + a_{10} = 0.$$

$$a_{21}\, X_1 + a_{22}\, X_2 + \ldots\ldots\ldots\ldots\ldots + a_{20} = 0.\ , \text{ usw.}$$

Bei der Ausrechnung der einzelnen Beiwerte werden wir nur die Verformungen eines Trägerteils in Betracht nehmen. Z. B.: Beim Verbundquerschnitt aus der Abb. 3. können die Verformungsglieder auf die Ortsbeton-Druckplatte bezogen werden. In diesem Falle werden die Momentenflächen der virtuellen Kräfte $X_i = 1$, nur auf die Schwerlinie des Ortsbetons bezogen. Diese Tatsache hat nur bei Trägern mit polygonalen oder krummlinigen Achsen (Rahmen und Bogen) Bedeutung, bei Trägern mit geraden Achsen (Durchlaufträger) ist die Beziehungsachse gleichgültig.

Dementsprechend die Belastungsglieder:

$$a_{ik} = \int_0^s \frac{M_i\, M_{2k}}{E_2\, J_2}\, ds + \int_0^s \frac{N_i\, N_{2k}}{E_2\, F_2}\, ds$$

$$a_{io} = \int_0^s \frac{M_i\, M_{20}}{E_2\, J_2}\, ds + \int_0^s \frac{N_i\, N_{20}}{E_2\, F_2}\, ds$$

M_i und M_{2k} sind die Momente der virtuellen Kräfte $X_i = 1$ und $X_k = 1$, bezogen auf die Schwerlinie des Ortsbetonträgers, M_{20} ist das Aussenmomentanteil am statisch bestimmten Träger. Z. B.: bei Durchlaufträger (Verdrehung an einem Ende des Trägers).

$$a_{ik} = \int_0^l \frac{M_i\, M_{2k}}{E_2\, J_2}\, ds = \int_0^l \frac{M_i\, M_k\, b_2}{E_2\, J_2\, c}\, ds$$

$$a_{io} = \int_0^l \frac{M_i\, M_{20}}{E_2\, J_2}\, ds = \int_0^l \frac{M_i\, M_o\, b_2}{E_2\, J_2\, c}\, ds$$

6.2 Die Belastungsglieder des unbestimmten Tragwerkes in der Folge der Zeit werden:

$$a_{ikt} = \int_0^s \frac{M_i\, M_{2kt}}{E_2\, J_2}\, ds + \int_0^s \frac{N_i\, N_{2kt}}{E_2\, J_2}\, ds$$

Wenn $\frac{b'_2}{b_2} \cong 1$ und $\nu = 1$, der Wert des auf den Ortsbeton bezogenen Momentes ist.

$$M_{2kt} = N_{2kt}\, b'_2 \cong N_{2kt}\, b_2 = M_k \frac{b_2}{c} \left(1 - \lambda\, e^{-\alpha \varphi_{2t}}\right) \frac{\beta}{\alpha}$$

$$a_{ikt} = \int_0^s \frac{M_i\, M_k}{E_2\, J_2} \frac{b_2}{c} \left(1 - \lambda\, e^{-\alpha \varphi_{2t}}\right) \frac{\beta}{\alpha}\, ds$$

$$a_{iot} = \int_0^s \frac{M_i M_o}{E_2 J_2} \frac{b_2}{c} \left(1 - \lambda e^{-\alpha \varphi_{2t}}\right) \frac{\beta}{\alpha} ds$$

Die Formänderungsgleichung zwischen den Zeitpunkten t und t + dt:

$$a_{11} \frac{d \varphi_2}{dt_2} X_{1t} + a_{11} \frac{dX_{1t}}{dt_2} + a_{12} \frac{d \varphi_2}{dt_2} X_{2t} +$$

$$+ a_{12} \frac{dX_{2t}}{dt_2} + \ldots\ldots\ldots\ldots\ldots + a_{10} \frac{d \varphi_2}{dt_2} = O .$$

Wo z. B.:

$$a_{ii} \frac{d \varphi_2}{dt_2} = \int_0^s \frac{M_i M_i}{E_2 J_2} \frac{b_2}{c} \lambda \alpha e^{-\alpha \varphi_{2t}} \frac{\beta}{\alpha} \frac{d \varphi_2}{dt_2} ds$$

6. 3 Bruch des statisch unbestimmten Tragwerkes.

Das statisch unbestimmte Tragwerk wird auf Grund der Elastostatik bemessen, mit Rücksicht auf die Bauweise oder Montage und Ueberlagerung der inneren Kräfte.

Bei der Untersuchung der statisch bestimmten Träger im Gebrauchszustande, haben wir darauf hingewiesen, dass die Berechnung der Vorbelastung, des Kriechens und des Schwindens in manchen Fällen, zu einem höheren Sicherheitsgrad führt. Bei statisch unbestimmten Tragwerken ist auch die unbestimmte Grösse eine Funktion der Zeit. Bei freigebauten Teilen, wo die gesamte Last von dem statisch bestimmten Fertigteil getragen wird, entstehen nach dem Verbund negative Momente und die positiven Momente vermindern sich. Bei auf Rüstung gebauten Tragwerken geht das umgekehrt, an der Stelle der frischen Verbindung werden die negativen Momente vermindert.

Die Berücksichtigung dieser Beanspruchungen und Spannungsumlagerungen beeinflusst aber den Bruchzustand des Trägers nicht, wenn wir den Bruch des Trägers mit der Bildung der Fliessgelenke identisch nehmen.

7.) *Schlusswort.*

Die Bedeutung der Stahlbeton-Verbundkonstruktionen wird von Tag zu Tag grösser infolge der rapiden Entwicklung der Fertigteil-Bauten. Das Bestreben, ein durch Versuche bestätigtes Bemessungsverfahren zu entwickeln, ist von wirtschaftlicher Bedeutung. Durch die Versuche wird es möglich sein, mehrere Fragen zu beleuchten, z. B.: die Frage des Verbundes der zwei Betonteile, der Schubsicherheit, des Einflusses der Zeit, der Bemessung des Plattenbalkens und der Schubbewährung, usw.

Die versuchsmässige Auswertung obengenannter Fragen ist im Gange und die Ergebnisse werden im Schlussbericht veröffentlicht.

ZUSAMMENFASSUNG

Der Zweck der vorliegenden Arbeit ist, einen Ueberblick der Berechnung der Stahlbeton-Fertigteilkonstruktionen -die nachträglich an Ort und Stelle verbunden sind- zu geben. Es wird die Bestimmung der Teilkräfte einzelner Trägerteile, bei verschiedenen Lastfällen untersucht.

Die statisch bestimmten Tragwerke werden in zwei Gruppen analysiert: Systeme mit gleicher Schwerachse und Systeme mit verschiedenen Schwerachsen. Da beim Brückenbau und Hochbau der letztere Fall immer häufiger angewendet wird, wollen wir ein vereinfachtes Rechnungsverfahren zeigen, womit die Spannungsumlagerungen leicht ermittelt werden können.

Im Rahmen der Arbeit wird der Einfluss des Kriechens und des Schwindens, -hauptsächlich mit Rücksicht auf die Schonzeit und den Zeitalterunterschied beider Trägerteile-, in Betracht genommen.

Die Frage der Bemessung der Stahlbeton-Verbundkonstruktionen wird ebenfalls behandelt. Es wird ein Bemessungsverfahren vorgeschlagen, das die Ergebnisse der elastischen und plastischen Theorien vereinigt und das durch zahlreiche Versuche bestätigt wurde. Auf dieser Grundlage kann der Sicherheitsfaktor beim Bruch der nach der Elastizitätstheorie bemessenen Verbundsysteme leicht ausgerechnet werden. Die Berechnung des unbestimmten Verbundsystemes wird auch behandelt, besonders mit Rücksicht auf den Einfluss der Zeitveränderlichkeit auf die einzelnen Belastungsglieder.

RESUMO

O autor dá uma ideia geral do cálculo das estruturas de betão armado com elementos prefabricados montados no local da obra e analisa a determinação dos esforços nas vigas para diversos casos de carga.

As estruturas isostáticas dividem-se em dois grupos: as co-axiais e as de eixo distinto. Sendo o último o caso mais corrente nos edifícios e nas pontes, o autor expõe um método de cálculo simples que permite determinar fàcilmente a repartição das tensões em estruturas desse tipo.

O autor também considera o efeito da fluência e da contracção do betão, especialmente no que diz respeito à diferença de idade de dois elementos de uma viga.

Estuda-se igualmente o caso das construções compostas de betão armado. O autor propõe um método de cálculo que combina os resultados das teorias elástica e plástica e que foi verificado em numerosas experiências.

Este método permite determinar fàcilmente o coeficiente de segurança à rotura de sistemas compostos calculados pela teoria da elasticidade.

O cálculo dos sistemas compostos indeterminados também é discutido, considerando particularmente a influência das variações das cargas elementares com o tempo.

RÉSUMÉ

L'auteur donne un aperçu général du calcul des constructions en béton armé executées en éléments pré-fabriqués montés sur place. Il étudie le determination des efforts dans les poutres pour divers cas de charge.

Les structures isostatiques peuvent se diviser en deux groupes: les structures co-axiales et celles à axes distincts. Les dernières étant plus courantes dans le bâtiment et les ponts, l'auteur expose une méthode de calcul simple qui permet de determiner facilement la repartition des contraintes dans des constructions de ce genre.

L'auteur considère aussi l'influence du fluage et du retrait, surtout en ce qui concerne deux éléments d'une même poutre, d'âges diférents.

Les cas des structures composées en béton armé est également étudié. L'auteur propose une méthode de calcul qui combine les résultats des théories élastique et plastique et qui a été vérifiée par de nombreuses expériences. Cette méthode permet de determiner facilement la sécurité à la rupture de systèmes composés calculés par la théorie de l'élasticité. Le calcul des systèmes composés indeterminés est aussi traité, en considérant surtout l'influence de la variation des charges élémetaires avec le temps.

SUMMARY

The author gives a general survey of the calculation methods used in reinforced concrete structures incorporating precast units assembled at the site. He discusses the determination of forces in the members for various types of load.

Statically determinate structures are divided into two groups: coaxial systems and systems with distinct axis. Owing to the latter being more commonly found in buildings and bridges, the author describes a method of analysis through which stress repartition is easily determined for that type of structure.

The author also considers the influence of concrete creep and shrinkage, as related to the differences of age between two precast elements.

Composite reinforced concrete structures are also considered. A calculation method is proposed which is based upon the results of the elastic and plastic theories and has been verified by a number of tests. This method enables to easily determine the failure safety coefficient of composite structures calculated by the theory of elasticity. Indeterminate composite structure calculation is discussed, especially regarding the influence of the variation of load elements with time factor.

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The vibration of precast concrete elements for post-stressed bridge members

Des Vibrieren von Beton-Fertigteilen für nachträglich zusammengespannte Brückenglieder

Vibração de peças de betão prefabricadas para elementos de ponte post-esforçados

Vibration d'éléments préfabriqués pour membrures de pont précontraintes sur place

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Bridges contructed of post-stressed concrete units have certain advantages over other types of construction. The more important being the small amount of maintenance required and the possibility of manufacture at a convenient site, reducing the erection work to placing the units on a skeleton scaffold and stressing the bars or wires linking them together. These advantages are especially important in areas of rapid development such as West Africa, and areas of high rainfall requiring much maintenance of steel structures.

Concrete of high strength is always specified for this type of construction and the usual method of obtaining this high strength is to use a mix of low water content compacted with the aid of vibration. Vibration may be applied by: —

1. Vibrating tables.
2. Shutter vibrators.
3. Immersion vibrators.

All these may be used for compacting precast post-tensioned units, the method chosen depending on the weight of the casting and its dimensions. Vibrating tables are usually limited in the weight of concrete which may be compacted upon them; this is less important in the case of electro-magnetic tables since any number of these may be placed side by side to share the load and to run in phase with each other.

Mechanically driven tables owing to their inability to run in phase with controlled amplitude cannot be used in this way. Immersion and shutter vibrators are used in sufficient numbers to achieve compaction without consideration of phase difference.

The requirements are a casting which is completely homogeneous with uniform strength and modulus of elasticity throughout. Unfortunately these are rarely achieved, it is therefore proposed to examine the problem and discuss its solution.

The problem approximates to that of obtaining uniform strength throughout the casting. It has been shown elsewhere (1) that for vibrated concrete there is a minimum acceleration below which compaction does not take place. It should be emphasised that this minimum acceleration is the acceleration of the concrete and not that of the vibrating unit or mould. The minimum value of acceleration for compaction is of the order $2\,{}^1/_2\,g$ to $3\,g$ dependent upon the frequency of the vibration and the workability of the mix. It may be stated therefore that to achieve full compaction throughout the casting the acceleration of every point within the casting must be equal to or exceed the minimum value.

In a large casting the acceleration at any point varies according to the distance of the point from the source of vibration and to the proximity of the side of the mould (2). Thus the lowest value of acceleration occurs near the centre of a casting vibrating on a vibrating table. It might be argued that if the acceleration at this point is greater than $3\,g$ then all would be well. In this connection it is necessary to consider a further point. An increase in acceleration results in a reduction in the time of vibration necessary to achiéve compaction, therefore those parts of a casting having the highest acceleration will be compacted before those with a lower acceleration. It can readily be seen therefore that vibration should be continued for such time as is necessary for full compaction at the lowest acceleration.

The concrete adjacent to the sides of the mould and on the surface of the casting will have a greater acceleration than that at the centre and will be compacted first, its appearance will not then be a true indication of the completeness of vibration of all parts of the casting. If no account is taken of the foregoing then a casting may result in which the outer layers are fully compacted whilst the core is not. The conditions are analogous to a case hardened steel and the casting will not fulfill the requirements stated previously because its strength will vary. The much publicised danger of OVER VIBRATION must be considered in this context. If a mix is suitable for vibration then after full compaction has taken place no further increase in strength will result however long the period of vibration nor will it be in any way adversely affected. The troubles of overvibration and segregation only result from a badly designed mix.

Concrete mixes must be designed for vibration, the ordinary methods of design for hand placing being inadequate and in some cases harmful when applied to concrete which is to be vibrated. In addition to the segregation a further problem must be borne in mind when designing a mix, namely «Rotational instability» (3). The phenomenon is caused by one part of the mould having a slightly higher acceleration than

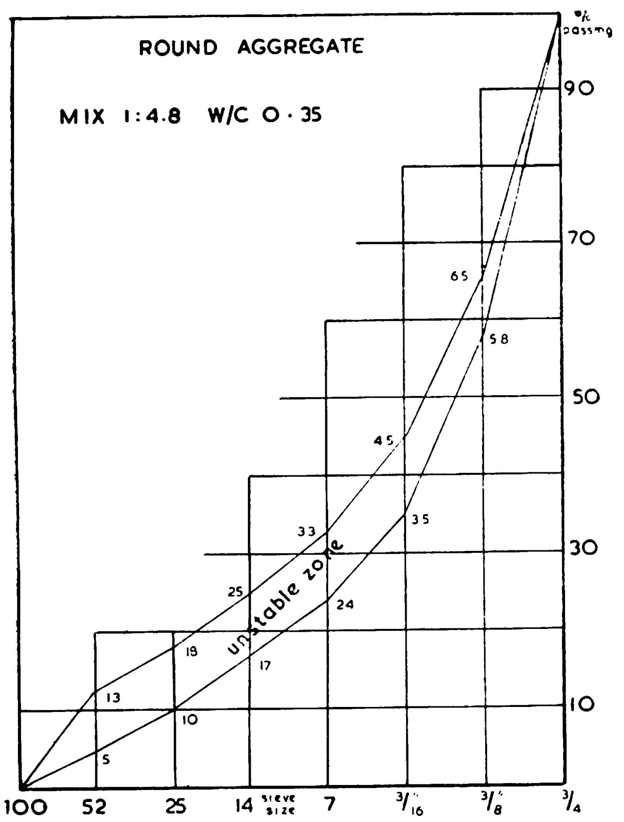

FIG. 1. Limit curve for rotational instability at accelerations up to 18 g with rounded river gravel

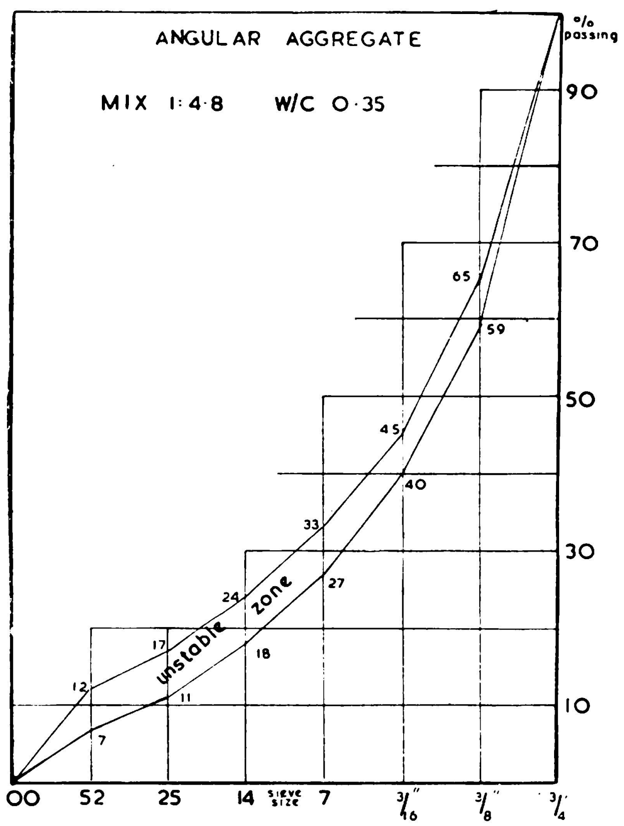

FIG. 2. Limit curve for rotational instability at accelerations up to 18 g with angular granite aggregate

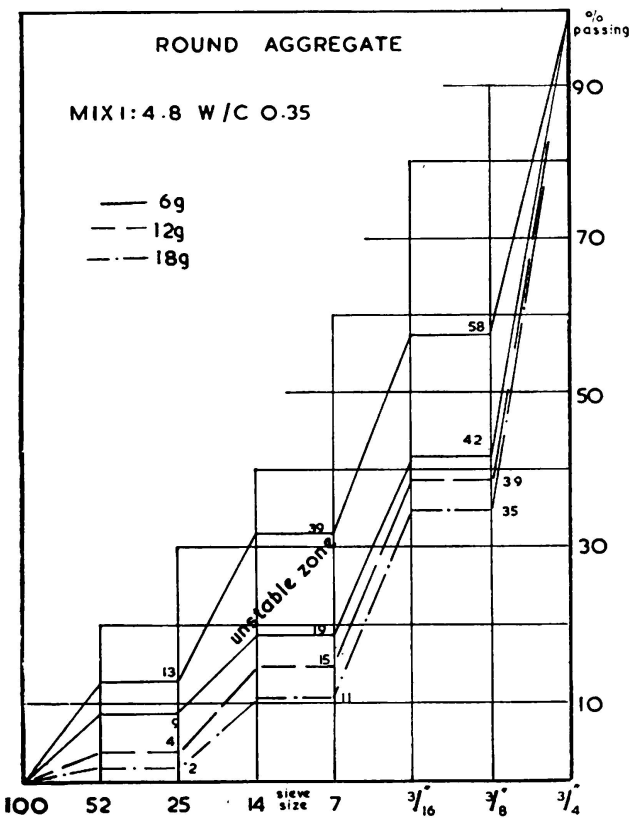

FIG. 3. Limit curve for rotational instability at accelerations up to 18 g with gap graded rounded river gravel

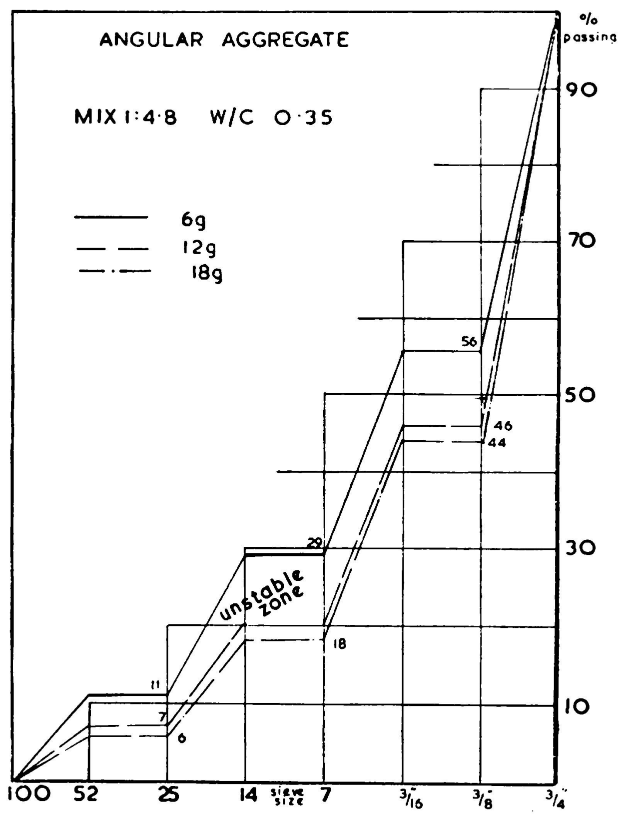

FIG. 4. Limit curve for rotational instability at accelerations up to 18 g with gap graded angular granite aggregate

other parts, in practice this, it will be agreed, is inevitable. This variation of acceleration results in a couple tending to rotate the concrete mix about a horizontal axis. If rotation does take place one part of the plastic concrete rises above the level of the remainder and a depression occurs diametrically opposite. The concrete rotates at a rate of several revolutions per minute sucking in air at the depression and forcing air hubbles into the body of the casting; compaction is never achieved and this may result in the reduction of strength by 50 % without any surface indication on the resulting casting. This phenomenon is also caused by unsuitable grading, thus the grading of a vibrated mix should be considered more closely than that for hand placed concrete. Besides considering ultimate density its internal frictional resistance must be high to eliminate Rotational instability; this may be accomplished by using the minimum quantity of fine sand i. e. that passing a N° 52 B. S. sieve.

It has frequently been claimed as an advantage of post tensioned concrete that every component is tested whilst stressing takes place. This is only partly true since only if the factor of safety of the concrete is reduced below unity does failure take place. If the strength is reduced, as it may well be, by 30 % by any of the foregoing then the stress factor of safety may be reduced from say 3 to 2 and test cubes or cylinders will not indicate such reductions since they are small enough for full compaction to be achieved under all save the most extreme conditions of instability or segregation.

The difficulties have now been described. What are the remedies?

I. Ensure that adequate power for vibration is applied to obtain an acceleration at any point in a casting greater than 3 g. A rough guide to the power required is to allow 1 watt per pound of concrete plus the energy required to vibrate the table mould etc. without any concrete (4).

II. Vibration must be continued for a sufficient time for full compaction of the whole casting. Say 120 % of the time at which the emission of bubbles from the surface ceases.

III. Design the concrete mix for vibration using fig. 1, 2, 3, or 4.

REFERENCES

1. *Concrete Building and Concrete Products.* September, 1953.
2. *The Engineer.* 26th February, 1954.
3. *Concrete and Constructional Engineering.* February, 1953.
4. *Magazine of Concrete Research,* n° 15.

SUMMARY

This paper considers the problems encountered in the manufacture of precast units for post tensioned bridge members of high strength with the aid of vibration. The desiderata are stated together with difficulties and the means of overcoming them to achieve the desired results.

ZUSAMMENFASSUNG

Die Arbeit behandelt die Probleme bei der Herstellung von vibrierten Fertigteilen, die später zu Brückenteilen zusammengespannt werden. Die auftretenden Schwierigkeiten sowie die Möglichkeiten zu deren Überwindung und somit zur Erreichung der gewünschten Ergebnisse werden beschrieben.

RESUMO

O autor considera os problemas relativos à fabricação, com o emprego de vibração, de elementos préfabricados para vigas de pontes de grande resistência preesforçadas no local. Indica também as condições a que devem obedecer, bem como as dificuldades, e meios de as resolver, de modo a obter os resultados procurados.

RÉSUMÉ

L'auteur considère les problemes posés par la fabrication, à l'aide de la vibration, d'éléments préfabriqués pour poutres de ponts précontraintes sur place. Il décrit les desidérata ainsi que les difficultés, et les moyens de les résoudre, pour obtenir les résultats recherchés.

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