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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

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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.

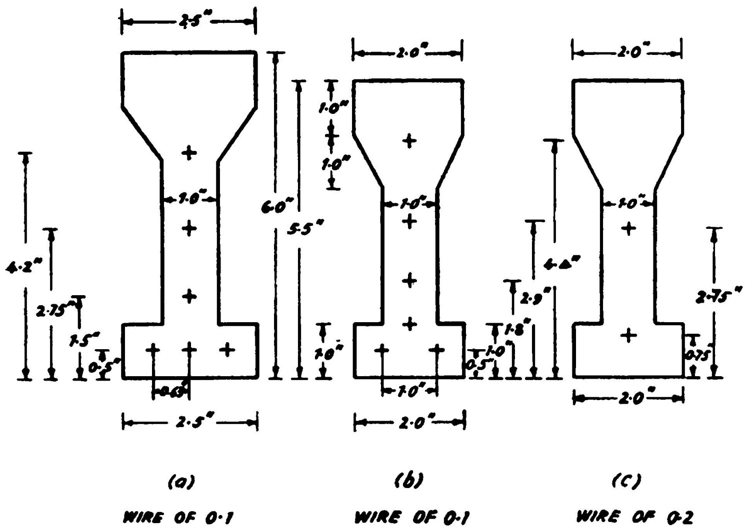

Fig. 1

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 (3)	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.

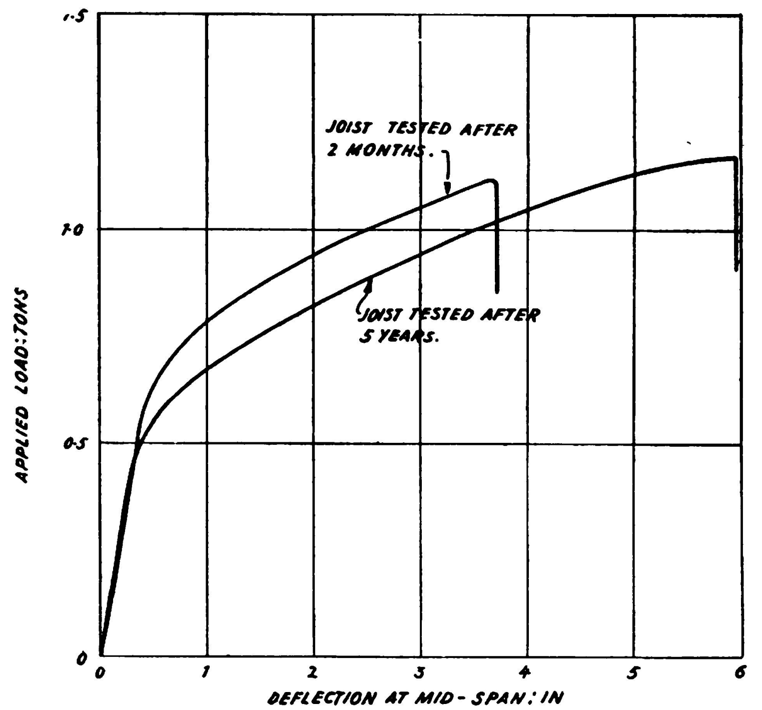

FIG. 2

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: —

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

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.

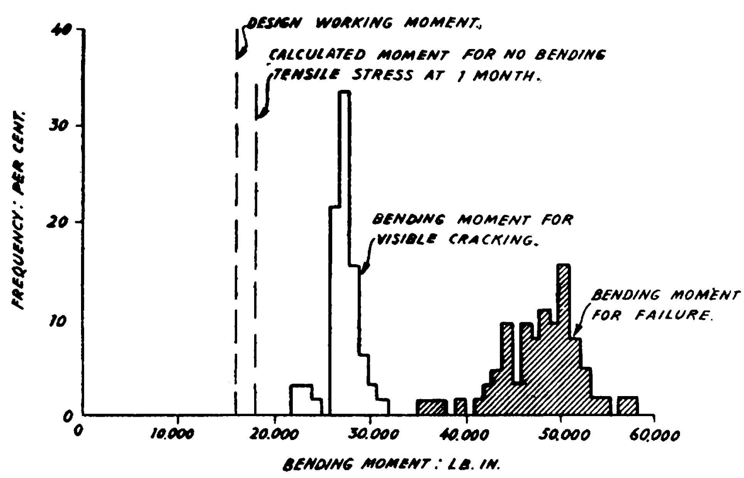

Fig. 3

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.

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

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

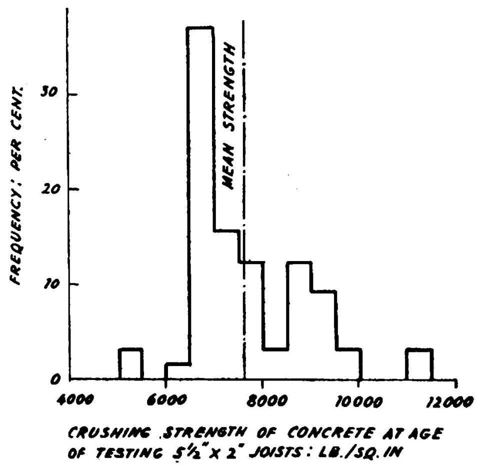

Fig. 4

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.

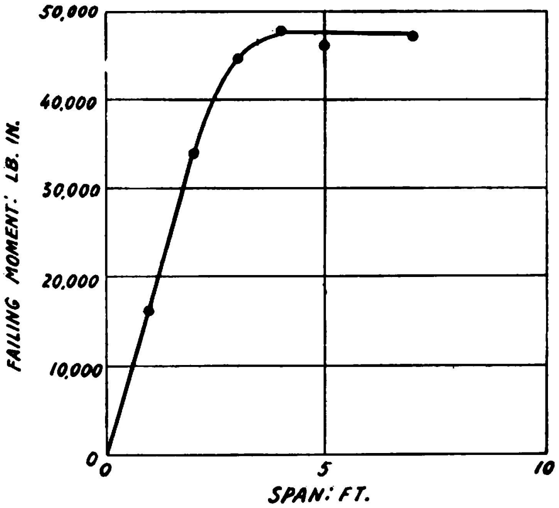

FIG. 5

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.

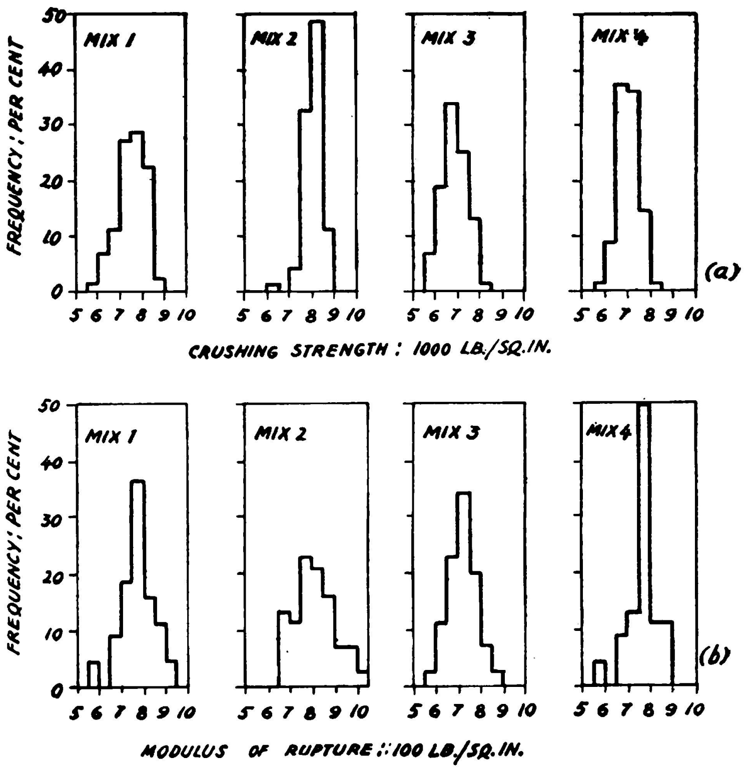

FIG. 6

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: —

$$M_u = X \,.\, t_u + Y \,.\, C_u + Z \tag{4}$$

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

$$\sigma^2_{M_u} = X.^2 \, \sigma_{t_u}{}^2 + Y.^2 \, \sigma_{C_u}{}^2 + \sigma^2_r \tag{5}$$

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: —

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

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.

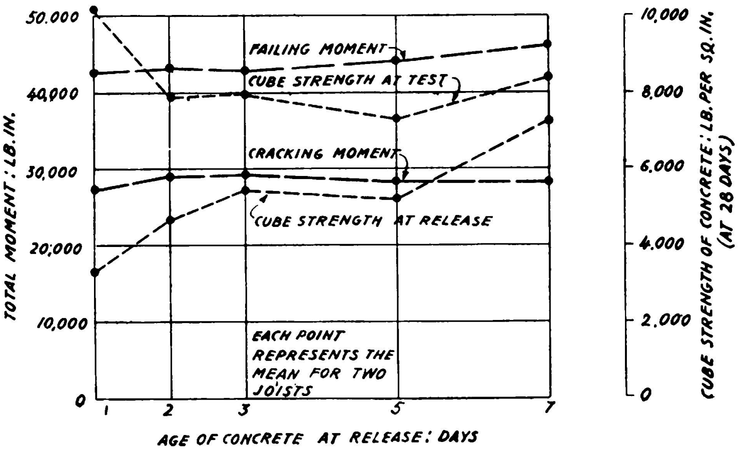

FIG. 7

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;

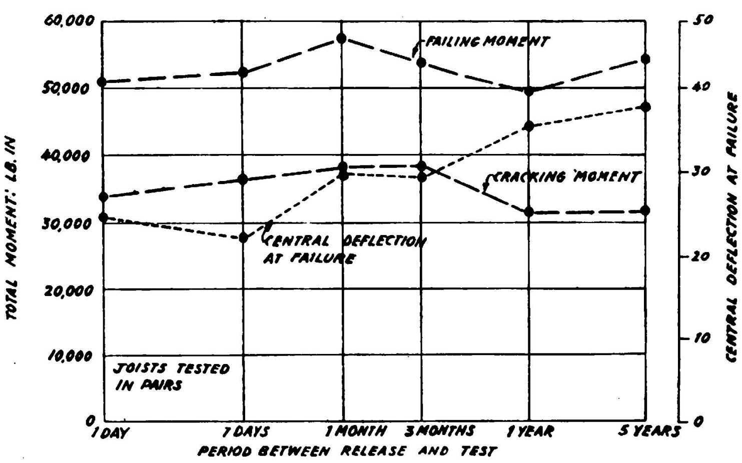

FIG. 8

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.

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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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