Characteristics of preflexed prestressed concrete beams

by

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and

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Mr J. Ford (Ministry of Transport) wrote that the Authors had accurately predicted the load at which the concrete would crack but the prediction was dependent on the crack appearing under a tensile stress of 450 lb/sq. in. This was low in relation to the working stress permitted by C.P.115 for concrete with a cube strength of 7500 lb/sq. in., and it would be interesting to know whether strains were recorded between the release of the preflexing moment and the appearance of the first crack which confirmed that the stress in the concrete was 450 lb/sq. in. when the crack appeared.

65. The preflexed beams in the test acted as fully composite (i.e. including the precast casing to the bottom flange) up to about 0.29 of the ultimate load. Would the Authors care to comment on the validity of the practice of calculating the stresses in the section at working load on the basis of the section modulus of the full composite section?

Dr S. D. Lash (Department of Civil Engineering, Queen's University, Kingston) wrote that tests of Preflex beams at Queen's University, Kingston, Ontario had led to similar conclusions to those given by Evans and White. Three series of tests had been made during the past ten years.

67. Dowsley, in 1956, tested four small preflexed beams together with two comparable beams not preflexed. The steel beams were of Cor-Ten high tensile steel, and concrete with an ultimate strength of 5500 lb/sq. in. was used to encase the tension flange of the steel beam. Dowsley found that the pre-compression established on release was less than calculations would indicate and that much of this was lost before the beams were finally tested. As a consequence the enhanced stiffness was only in evidence for part of the working load range. In three of the beams no shear connectors were provided and there was some evidence of slip on release.

68. MacDormand, in 1958, studied in more detail the effects of creep and shrinkage using beams of the same size and materials as those tested by Dowsley. Three small beams were tested, of which two were preflexed. A limited number of shear connectors was used on the lower flange; these appeared to prevent slip on release. A substantial amount of shrinkage occurred between release and final testing—this involved shrinkage of the precompressed concrete around the tension flange and shrinkage of the slab placed on the compression flange. Electrical resistance strain gauges on the tension flange of the steel indicated a preflexing stress of 45 kip/sq. in. which was reduced to 17 kip/sq. in. on release. Over a period of 33 days this gradually declined to 8 kip/sq. in. Cracking of the pre-compressed concrete was observed at about 40% of the working load and the resulting loss of stiffness was evident in the load-deflexion graph. Moreover, even at low loads before cracking was observed, the flexural rigidity was only about two-thirds of the expected value.

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FIG. 12: EFFECT OF PREFLEXING ON DEFLECTIONS WITHIN THE WORKING LOAD RANGE: CONCRETE ON TENSION FLANGE ONLY

69. A third series of tests had recently been completed by Kudrencky. In this series larger beams were used and particular care had been taken to prevent slip on both tension and compression flanges. Four beams were tested, three of which were preflexed.

70. The results confirmed the conclusions of the earlier tests. After a month's storage following release, the calculated stresses in the preflexed concrete, based on observed strains in the steel, were tensile in the upper surface, and the maximum compression on the lower surface was reduced to about 600 lb/sq. in. Upon application of load, cracking was observed at about one-third of the working load and by the time the working load was reached relatively little stiffening effect of the preflexed concrete remained.

71. Figs 12 and 13, showing the relative stiffness within the working range of all the beams tested, summarized the tests at Queen's University. Fig. 12 referred to beams with concrete around the tension flange only and Fig. 13 to beams with a compression slab in addition to the preflexing concrete. In only one beam (Dowsley A) was the full stiffness retained throughout the working range. This beam was tested very shortly after release. In all the other beams cracking occurred within the working range and the gain in stiffness at working stiffness was less than half the anticipated value. This conclusion suggested that if preflexing was to be effective, concrete having a very low shrinkage and a very small specific creep should be used. It should also be pointed out that an accidental overload might crack the concrete if it had not already cracked, and hence reduce the stiffness.

72. In calculating the working load it seemed reasonable to ignore preflexing entirely, i.e. not to take into account either the residual stress in the steel or the

FIG. 13: EFFECT OF PREFLEXING ON DEFLECTIONS WITHIN THE WORKING RANGE: CONCRETE ON BOTH FLANGES

TABLE 4: COMPARISON OF CALCULATED AND OBSERVED ULTIMATE MOMENTS

Beams	Preflexed	Slab	Max. M_{obs} Kip. in.	Calc. M Kip. in.	$\frac{M_{obs}}{M_{calc.}}$	Comments
Dowsley A . . .	No	No	429	420*	1.07	Bare beam
B	Yes	No	467	429*	1.11	
C	Yes	Yes	534	529*	1.01	
D	Yes	Yes	512	535*	0.96	
E	No	Yes	515	533*	0.97	
F	Yes	No	298	420*	0.71	
MacDormand 1 .	Yes	No	468	420*	1.11	Lateral buckling occurred
2	Yes	Yes	586	534*	1.10	
3	No	No	469	420*	1.11	
Kudrenicky 1 .	No	No	826	830	1.00	Bare beam
2	No	No	843	830	1.02	
3	Yes	No	816	830	0.98	
4	Yes	Yes	1146	1037	1.11	
5	Yes	Yes	1456	1402	1.04	

* Based on specified yield of 50 k/sq. in.

effect of the prefixed concrete in increasing the section modulus, because the ultimate strength was not altered by prefixing. The ultimate strength might be predicted with reasonable accuracy on the basis of a fully plastic stress distribution. Table 4 compared ultimate moments calculated in this way with the observed moments based on the maximum loads recorded during the test. It would be observed that agreement was good and that there was no significant difference between beams which were prefixed and those which were not.

Mr R. J. Nicholas (Boulton and Paul (Steel Construction) Ltd) wrote that the Authors had been kind enough to acknowledge the part played by Boulton and Paul Ltd, who had been pleased to be of assistance in the project, and in turn acknowledged the interest of the profession in seeking the best approach to Preflex beam design.

74. The Paper had taken a very similar stand to that of the panel of engineers representing the C.P.117 Committee which had examined the Preflex system since the date of the Leeds University tests, and on whose recommendations the present methods of calculation in this country were based. Emphasis was placed on the tensile stresses in the concrete, which was never intended. The Paper drew very definite conclusions from the observations, but in the writer's opinion, these should have been presented with reservations, considering the size of the specimens, the accuracy of measurements outside the laboratory, the performance of the gauges, the uncertainties of values of E_c , variations in temperature, and variations of cross section due to measuring gauges, etc. All of these points had received comment in Mr White's thesis on which the Paper was based.

75. The idea of prefixion arose from the desire to use high yield steels to their best advantage without the disadvantage of excessive deflexions, and still to maintain the full protection of the casing used to increase the section's inertia. From this point of view, therefore, a Preflex beam was simply a cased steel beam. It should not therefore be considered as a prestressed concrete beam. Admittedly, in order to keep the protective qualities of the concrete a compression was imparted to it by the transfer of strain from the steel encased, as was the case with prestressed concrete. Once that precompression was exhausted, however, behaviour was entirely different. Cracks in prestressed concrete began at the soffit and, once propagated, spread rapidly into the compression zone to hasten the ultimate and final collapse of the beam. With the Preflex system, as was clearly shown in the results of the Authors' and others' tests, the cracks spread slowly as load was applied and did not travel into the compression zone; nor was the beam subject to collapse in the ultimate condition, and some recovery would always occur on removal of load. The strength of the Preflex beam, therefore, was not connected with this concrete.

76. Why was the cracking load so important to the concrete? Advantage had been taken of a $\times 30$ magnification lens to find the first crack of 0.0075 in. The *Recommendations for an International Code of Practice for Reinforced Concrete* (§ 3.311) suggested that the width of cracks normal to the direction of reinforcement should be not greater than 0.0039 in. for aggressive conditions, and 0.0079 in. for normal outside exposure. These levels were reached at about 103% and 140% of the design load of 19.4 tons (Fig. 7). Why should it be a matter for concern that the design load would be indicated at 23.1 tons (i.e. 119%) if outside exposure was acceptable (§ 58)?

77. Prefixion was shown in Table 1 as 16.35 ton/sq. in. In manufacture, beam L2 received a jacking load of 13.75 tons which should have imparted a stress of 19.2 ton/sq. in. to the steel. In fact, from previous trials it was known that the jacks exhibited an error in the lower loads, and jacking was therefore done to achieve the required calculated deflexion; hence the inflated load. It was difficult to imagine, however, where the additional errors had crept in to reduce the stress even further than anticipated. Nevertheless, with complete protection achieved up to a load of 20 tons in practice, how much better the result would have been with full prefixion:

how much better too, in a practical application where the available deck slab might be as much as 5 or 6 ft wide, and where dead loads would ensure a large proportion of the load in place before hardening of the compression zone concrete.

78. Was it worthwhile undertaking complicated calculations to discover stresses not required, when all the strength was required in the steel? In Mr Nicholas's opinion, the test supported the Baes-Lipski approach.

79. Reference had been made to the beam size and its effect on the results. Mr White had been warned that tensions might be high on the top edge of the pre-compressed concrete under working load. If, for example, a load was applied at stage 2 to give a moment equivalent to the release moment at stage 1, it could be shown through the variation of the level of the neutral axis and, therefore of the section moduli, that a compression 1.98 times greater than the release stress would be achieved. In similar circumstances in a beam, for example, 25 in. deep, this factor would be 1.36 or thereabouts. This was one way in which the effect of size on the results might be appreciated, and was important if the results of one test were to be applied directly to other beams.

80. Investigation of some of the results presented several anomalies which could not be explained and which were beyond the scope of this contribution. However, in Table 2 (where the section modulus of the section at stage 8 was known to be 130 cu. in.) the application of a load to return the steel stress at level D to 36 600 lb/sq. in. could be calculated to require a moment of 105 ton-ft. This was equivalent to 178% of the preflexion moment and was out of all proportion to the increase in resistance of the section. Taking the Baes-Lipski figures from Table 1 the factor worked out at 108%. This was probably low, but certainly appeared more realistic. Had the Authors any idea why this should be so?

81. No mention had been made of the additional benefits offered by preflexion. In manufacture, the preloading was applied, released, and reapplied. This first preloading was a stress-relieving process which served to neutralize the residual stress, contained in all rolled steel, and therefore to ensure more nearly elastic behaviour in the steel; it also ensured freedom from the danger of brittle fracture¹⁴ and might well improve the fatigue performance of the steel,¹⁵ already improved by the reduction of the ratio of maximum to minimum stresses in the working range due to the restraint offered by the precompressed concrete. The beam itself was tested under load, and the concrete received its most serious load at the instant of release. All these factors of built-in safety had encouraged the Belgian authorities to allow working stresses in the steel, according to the Baes-Lipski methods, of 18 ton/sq. in. for all structures (including bridges) and their confidence in the protective properties of the concrete had never been misplaced.

82. Some work on creep and shrinkage in Preflex concrete had been undertaken by A. S. G. Bruggeling in Holland¹⁶ who, having found some 20% loss in steel stresses due to these phenomena, had suggested that for complete protection (0.0039 in. crack width) a tensile stress of 850 lb/sq. in. might be permitted providing allowance was made for strain losses of the order of 45×10^{-6} , and providing 2.5% reinforcement, on which he placed great emphasis, was added in the concrete section.

83. As a final note of interest, a calculation by the present recommended elastic method for structures other than bridges on this 8-in. $\times$ 8-in. beam showed that the concrete flange went into slight tension before the dead load was complete, but the range of stress after this stage was such as to ensure protection of the steel (approximately 3.5 ton/sq. in.); the final steel stress under a moment of 40.3 ton-ft (Table 2) gave a stress of approximately 10.5 ton/sq. in.

Mr A. Lipski (Consulting Engineer) wished to answer the criticisms of the methods of calculation which he and the late Professor L. Baes had put forward in *La poutre Préflex.*^{1,2} These could be summarized as follows:

- (a) under-estimation of the effects of shrinkage and creep;
- (b) calculation of inaccurate design stresses, arising from (a), particularly in the tensile zone of the beam;
- (c) conferring to a Preflex beam a cracking load superior to the actual one.

In discussing these points he wished to stress the difference between the problems of safety against failure, and safety against cracking. It was also necessary to agree on the nature and width of cracks to be considered.

85. Design stresses expressed indirectly the measure of safety against ultimate load failure. Apart from this they had no interest for the designer. In the Preflex beam the point of interest was therefore the tensile zone only in steel stress which was calculated on the understanding that the concrete casing did not exist (except for its own weight); having established that this casing could not crush on release, shrinkage and creep were of no importance whatsoever from the point of view of safety against failure. When the steel was considered as though the concrete did not exist, these phenomena were of no importance, as they did not affect safety. Creep and shrinkage did not affect the issue in an ordinary cased steel beam, or in a reinforced concrete beam. They did, however, in a prestressed concrete beam. Here the cracks would open rapidly, once started, and in helping to reduce the area of compression concrete, would influence the ultimate load for failure. This was not the case in a Preflex beam. Mr Lipski did not contest the importance of creep and shrinkage, but was considering it only from the point of view of cracking and eventual stiffness.

86. The problems of safety against cracking and of stiffness were entirely different. Firstly, however, it was necessary to define a crack. It was normal practice now to use a $\times 30$ microscope, but in 1951 he and Professor Baes had used only a hand magnifying glass. It was therefore not surprising that Mr White had found a crack of 0.0005 in. in beam L2 at a smaller load than that at which Mr Lipski had found one about four times as wide (approx. 0.0019 in.). These figures, however, suggested a very good performance on the part of the Preflex beam. Beam L2 was the worst of Mr White's specimens, so how did it really compare with practical requirements?

87. Table 5 showed (as did Fig. 7) a slow crack growth with increase in load, and

TABLE 5

Test load	Total bending moment including moment due to own weight	Total bending moment as a % of theoretical working moment, admitted as being $1.10 \times$ preflexing moment $= 1.10 \times 58.8$ $= 64.68$ ton-ft	Maximum width of cracks (measured)	Measured crack width as a % of width authorized for exterior works
ton	ton-ft	%	in.	%
12.00	4.76 + 40.00 = 44.76	69.2	0.00000	0.0
12.36	4.76 + 41.16 = 45.92	70.9	0.00075	9.5
14.11	4.76 + 46.98 = 51.74	80.0	0.00158	20.0
16.05	4.76 + 53.45 = 58.21	90.0	0.00275	34.8
17.99	4.76 + 59.92 = 64.68	100.0	0.00314	39.7
19.40	4.76 + 64.60 = 69.36	107.0	0.00360	45.5
20.00	4.76 + 66.60 = 71.36	110.3	0.00390	49.3
26.70	4.76 + 88.91 = 93.67	144.8	0.00790	100.0
35.20	4.76 + 117.22 = 121.98	188.5	0.01180	150.0

explained the differences between the tests of 1951 and those now reported. The fact that Professor Evans found a crack at 71% of the working moment and the writer had found one at 165% simply meant that his 'eye' was able to detect a fraction of 1/1000 in., while the original observations were able to detect only 2 to 3/1000 in.

88. The following allowable widths of cracks normal to the direction of the reinforcement had been agreed (*vide* Belgian N.B.N. 15, § 7.921 and *Recommendations for an International Code of Practice for Reinforced Concrete*, § 3.311):

for structural parts exposed to an aggressive atmosphere: 0.1 mm. (0.0039 in.)

for external structural parts exposed to the weather: 0.2 mm (0.0079 in.)

for internal structural parts in a normal atmosphere: 0.3 mm (0.0118 in.).

Only after 110% and 145% of the working moment had beam L2 reached the two most severe of these criteria. Furthermore, the cracks at that stage had produced no practical lack of stiffness in the beam. The factor against cracking, as far as the engineer was concerned, was therefore much better than 1, and this had been confirmed in all tests that had been made. The tests by Messrs Evans and White were a further proof that the Preflex beam was practically uncrackable. Further proof was available from some 10 000 beams already in service, in which no cracks had been detected.

89. Mr Lipski continued that in its fundamental characteristics the Preflex beam was not entirely tied to any other system. Precompressed concrete was common to both Preflex and prestressed concrete beams, but the difference in quantity of reinforcement (10% : 0.5% approximately) led to other consequences of shrinkage and creep. (1) A Preflex beam continued to function like an ordinary cased beam (or heavily reinforced concrete beam) after loss of prestress, the prestressed beam functioned as a lightly reinforced beam. (2) The Preflex beam had at that moment a smaller but sufficient load margin prior to cracking than the prestressed beam. (3) After cracking began, the slow increase in width of cracks in the Preflex beam contrasted favourably with the rapid opening of cracks in a prestressed beam. (4) The margin between appearance of the first cracks and ultimate load was high for Preflex beams. (5) The loss of stiffness in the Preflex beam at the start of cracking, while not negligible, was appreciably less than in prestressed concrete. (6) Unlike prestressed concrete, where collapse was complete after little additional absorption of energy, the Preflex beam could continue to bear load and absorb energy. This was a quality particularly favourable in impact loading. In view of these points it was understandable why the application of common criteria to both systems could be misleading.

90. It was admitted that the Preflex beam suffered a higher absolute loss of stress through creep than a prestressed concrete beam, but the compression before loss occurred was higher and, when all losses had taken place the Preflex beam was in a much more favourable condition than the beam in prestressed concrete. Safety from microscopic cracking might therefore be lower for the Preflex beam but it was higher than in reinforced concrete. Considering the inconvenience entailed in cracking, it could be said that reinforced concrete had a safety factor of about 1, Preflex about 1.5, and prestressed concrete about 1.8 to 2.0. As the width of the crack increased, however, this factor was much more favourable in Preflex beams.

91. So far as design stresses were concerned, the traditional methods for elastic design of reinforced concrete or encased steel were simple and safe, and could be extended to the Preflex beam. The designer was concerned with safety against ultimate failure, and not with the load at which the concrete would crack. As long as, by good practice in manufacture, he could be assured that the concrete would not crack dangerously until well after working load he should be satisfied. As for stiffness, he could use calculations based on the available section at values of E according to the nature and age of concrete, as was good practice in traditional reinforced concrete.

92. In conclusion, after examining the results of the Leeds tests in the light of the characteristics peculiar to the Preflex beam, the writer would suggest that the results were far from invalidating the design procedure and in fact confirmed it. It was an obvious adaptation of the classical calculation of reinforced concrete, which so far as resistance moments were concerned, completely and rightly neglected the phenomena of shrinkage and creep in the concrete. The examination of these phenomena could be confined to the study of deflexions only.

Mr K. W. Leung (Lecturer in Civil Engineering, University of Hong Kong) said that he appreciated very much the treatment of the effects of shrinkage and creep of concrete dealt with in the Paper, as against the combined treatment in the other methods mentioned therein.

94. Without the quoted references to hand, it was difficult for him to make direct comments on the other methods but he felt that the use of an equivalent value of $m = m_0(1 + \xi')$ for the computation of stresses in a composite section under condition of plastic flow in the concrete was quite reasonable as long as the effect of shrinkage was not included in the coefficient ξ' , and that the real criterion in the analysis was the assessment of an appropriate value for creep. If ξ' in Froelich's method was taken to be 0.7 with the effect of shrinkage excluded, the resulting figures for creep of the precast concrete would be very close to those derived by the Authors' method with a coefficient of $\xi = 0.1 \times 10^{-6}$. It appeared to him that both methods were satisfactory as a design procedure in the treatment of creep despite their hypothetical differences. As a passing remark, there seemed to be some small calculation errors in the figures +318 and +4011 under Condition 4 in Table 2 and the expression for F_2 in § 16 was presumably meant to be $\xi \cdot E_c \cdot A_c \cdot (f_1 + f_2)/2$.

95. Referring to condition under test load, the use of instantaneous moduli for the analysis of stresses was justified by the fact that the loadings were to a large extent instantaneous in nature, i.e. in all probability the various loadings were not held longer than was absolutely necessary for making measurements. Even if the sub-cracking loads were kept for a long time, it was unlikely that the plastic flow in the precast concrete, running in an opposite direction to that of the creep previously produced through the release of preflex, would be sufficiently great as to call for the use of a modular ratio higher than 1.1 to 1.2 m_0 . In the case of long term sustained loadings, however, there would be a considerable amount of creep taking place in the in situ concrete which must be taken into account.

96. The disadvantage of a preflexed beam lay in the fact that an appreciable portion of the preflexure still remained in the steel joist after release and the prestress in the precast concrete suffered a great deal from shrinkage and creep. This was shown clearly in Table 2 and well reflected in the conclusions given by the Authors. A remedy, it appeared to Mr Leung, was to provide an additional prestress by means of high tensile steel tendons in the pre-compressed zone. This additional prestress might be introduced at different stages and by different arrangements as required.

97. For illustration, let it be supposed that a prestressing force of 50 tons acting along an axis 2.3 in. above the bottom of the precast concrete was applied after Condition 5 in Table 2. Some slight alteration of the precast section would be necessary for the accommodation of the high tensile wires, etc, but, for simplicity of discussion, let all such minor details be ignored and those figures given under Table 2 for the properties of section be unchanged for calculation throughout. The computed results for such an additional prestressing force, the effect of which, as could be seen, raised the cracking load to 21 tons, equivalent to an increase of about 75%, were given in Table 6. The force would also increase the ultimate load. By suitable combination of preflex and prestress, therefore, it seemed to be possible to produce a composite unit more efficient than either the pure preflexed or the pure wire-prestressed beam. Would the Authors express their opinion on this prospect?

TABLE 6

Condition	In situ A: lb/sq. in.		Steel B: lb/sq. in.		In situ C: lb/sq. in.		Precast C: lb/sq. in.		Steel D: lb/sq. in.		Precast E: lb/sq. in.	
	Effect	Total	Effect	Total	Effect	Total	Effect	Total	Effect	Total	Effect	Total
1 to 5. As in Table 2 except for minor changes in 4 . . .				+3 515				+90		-13 585		+2685
5A. Prestressing force 50 tons at 2.3 in. above <i>E</i>			-285				+930		+7370		+1530	
5B. Additional creep of precast concrete . .			-85	+3 145			-210	+810	+2040	-4 175	-340	+3875
6 to 8. Effects as in Table 2		-75		+10 290		-585		+980		-5 045		+3415
9. Test load 21 tons	+2380	+2305	+11 420	+21 710	-1050	-1635	-1400	-420	-14 470	-19 515	-3425	-10
10. Final stresses . .	+2305		+9.70 ton/sq. in.		-1635		-420		-8.72 ton/sq. in.		-10	

Ir. A. S. G. Bruggeling (Consulting Engineer) wished to make a few comments on the characteristics of preflexed beams. The title of the Paper gave the impression that the Authors considered that there was little difference between preflexed beams and prestressed concrete beams. In the writer's opinion this difference was considerable.

99. A prestressed concrete beam was one in which high tensile steel was embedded and the quality of this steel necessitated its protection against corrosion, etc. A Preflex beam, on the other hand consisted primarily of a steel joist surrounded by concrete. The quality of the steel made it unnecessary to protect it any more than the normal steel bars in a reinforced concrete member. Prestressing of concrete beams gave them the necessary carrying capacity. Preflexing of steel joists gave these joists a limited deflexion under full load. Research on the behaviour of Preflex beams had shown that steel joists could be given sufficient rigidity by preflexing. The second point was that the concrete surrounding the lower flange of the steel joist would show limited cracking at full load and this did not influence the rigidity of the construction when there was regular loading and unloading. Figs 9 and 10 showed clearly that the rigidity of the freely supported Preflex beam was nearly twice that of the unpreflexed beam, while Fig. 7 showed that the cracking of the concrete was limited to an acceptable value of $4/1000$ in. at a load of 20 tons.

100. Ir Bruggeling had tested two Preflex beams in order to establish a method of calculating the losses after preflexion due to shrinkage and creep of the concrete. A simple method of calculation was proposed, but at that time no test data were available to assess the losses so they had to be chosen from his experience with prestressed concrete. These arbitrary figures were based on an assumed deformation of the concrete at the soffit of 5×10^{-4} , a value which decreased linearly along a line passing through the intersection of the neutral and y axes. He agreed with the Authors that this method underestimated the effect of shrinkage stresses at the top surface of the lower concrete flange, but in the case of small steel joists only. Preflex beams were normally about 25 to 35 in. deep, so the influence of this method on the loss of stress at the top of the lower concrete flange was not important.

101. In these tests he had found certain facts which might be of interest.

102. The influence of the longitudinal reinforcement in the precast concrete on the magnitude of the losses due to shrinkage and creep of the concrete was very important. The compressive stress in this reinforcement increased from 15 400 lb/sq. in. to 20 600 lb/sq. in. over a period of three months. On the other hand, the shrinkage and creep of the precast concrete was slowed down by this reinforcement. In that time that part of the force C' (Fig. 4) in the precast concrete carried by the longitudinal reinforcement increased from 16% to 27%. Simultaneously, the force C' decreased by 20% of the original value directly after release. In these tests the percentage of longitudinal reinforcement in the precast concrete was 2.5 and in the Authors' 1.0. It followed that the losses due to shrinkage and creep in the Author's tests were higher than those found by Mr Bruggeling. If his results were introduced into the computations in the Paper the cracking load was found to be 15.1 tons.

103. The tests made it clear that the quality of the precast concrete, especially with respect to the creep behaviour, was very important. If the concrete was not creep-resistant, the losses were more important than those found in the test mentioned above.

104. To obtain the cracking load a deformation of 4.5×10^{-4} was introduced into the concrete at the soffit while the tensile strength was taken as zero. (The percentage of longitudinal reinforcement in this precast concrete must be 2.0 or more.)

105. Cracking of $4/1000$ in. maximum width in the precast concrete had no noticeable influence on the rigidity of the construction. Limited cracking of the concrete at full load was therefore permissible.

106. When the Preflex member was unloaded the cracks in the precast concrete closed. This phenomenon was the same as for prestressed concrete.

107. The agreement between the tests carried out at Leeds and in Mr Bruggeling's laboratory clearly showed the potentialities of the preflexing technique, especially when the results of research under various conditions were carefully annotated and analysed.

Mr R. J. Blake (Senior Assistant Engineer, Messrs Scott & Wilson, Kirkpatrick & Partners) wrote that as design engineer responsible in part for the introduction of preflex beams to the United Kingdom, he had worked with the Authors during the preparatory stage of the tests described in the Paper. Difficulties were encountered in producing beams of a size small enough to fit into the available laboratory facilities, and since this point was relevant to the conclusions drawn, perhaps some mention of them ought to be made.

109. When preflexing shallow steel beams it was always difficult to obtain adequate pre-compression at the upper edge of the tension flange casing, and because the steel beams for testing could only be eight inches deep the difficulty was greater than usual. Because the neutral axis of the combined steel beam and pre-cast casing lay just above the top of the casing, release of preflexion caused very little compression at the top of the pre-cast casing of the test beams. A further complication was caused by the introduction of studs passing through the casing.

110. When any preflexed beam was completed by the addition of concrete to the upper flange, usually in the form of a deck or floor slab, the neutral axis moved upwards, away from the pre-cast casing. Loading applied to the completed beam could, therefore, cause a greater tensile strain at the top of the pre-cast casing and sometimes this was greater than the compressive strain induced at this plane by preflexion. In practice this difficulty was compensated by the large proportion of the total load which acted before the beam and top slab worked in combination. Live load, which remained to be carried by the completed beam, was often less than half of the total load, but for test beams the loading corresponding to live load was usually much greater than half of the total load, and in consequence the possibility of tensile strains occurring in the casing were enhanced when compared with ordinary beams.

111. These difficulties were referred to in the Elstree Test Beam Report, and it would be most useful if the Authors would give their reasons for not mentioning them in the Paper.

112. In § 58, as one of their concluding remarks, the Authors implied that the design load might be calculated using a stress of 18 ton/sq. in. when the preflexion stress was given as 16.35 ton/sq. in. This was entirely contrary to the principles set down by Messrs Baes and Lipsky in their technical writings on the preflexion of steel beams, wherein they stated that the preflexion stress must be equal to or greater than the highest design stress, the stresses referred to being those in the tension flange of the steel joist. Since this principle had always been accepted as a basic rule to obtain satisfactory preflexion, perhaps the Authors could enlarge upon their remark.

113. By suggesting that the concrete stresses rather than the steel stresses should control the design of preflexed beams, the Authors seemed to overlook the fact that the pre-compressed concrete casing contributed nothing to the strength of the beam. The purpose of this casing was twofold: it served to protect the steel, and was a means of improving the behaviour of the steel beam within its working range. This improvement was mentioned by the Authors.

114. It could not be too strongly emphasized that the strength of a preflexed beam was provided by the steel beam, and that when combined with a floor slab it was the steel beam and the slab that determined the load-carrying capacity of the composite beam. If the pre-cast casing was removed, the beam would still be capable of carrying the full design load; the behaviour of the beam in carrying its working load, however, would be that of any ordinary steel beam.

115. The methods of calculation suggested by Messrs Baes and Lipsky were

intended to provide a means of calculating stresses in the steel, and effective modular ratios were used in a similar manner to those adopted for reinforced concrete design. For the calculations made prior to the tests conducted at Elstree, the Baes-Lipsky method was adopted. Subsequently a comparison between calculated and recorded steel stresses showed excellent agreement.

116. Alternative methods of calculation had been suggested in various countries, including the United Kingdom, and some were in use. It was interesting to note that the original Baes-Lipsky method of approximate calculation could universally be applied for selecting the size of the steel beam and pre-cast casing, irrespective of whatever 'exact' method was used to calculate stresses.

117. The Authors might agree that the preflexed beam should not be regarded as a prestressed-concrete beam. If so, it would be interesting to have their reaction to a suggestion that, whilst it was important to ensure that the pre-cast concrete casing did not crack when the beam was under load, if such cracking were to occur its consequences would only be those found with reinforced concrete beams.

118. Perhaps they would also agree that the beams preflexed for these tests, whilst serving the purpose of providing information relating to bond characteristics could not be taken as completely representative of practical beams and that the conclusions should be read with this in mind.

Mr B. A. Haseltine and Mr Saffery (Jenkins & Potter, Consulting Engineers) felt that it was not sufficiently emphasized in the Paper that the calculations and results shown related to a young beam, test-loaded almost instantaneously; in the long term the effects of shrinkage and creep would be more pronounced, and indeed could almost negate any advantages gained from the preflexion.

120. The Authors had used a shrinkage value of 2.5×10^{-4} and 2×10^{-4} for the precast and in situ sections respectively. Normally, even with the best concrete, a value of 3×10^{-4} would be used. The combined effect of altering these values when used in the formulae in the Paper left the cracking load only slightly reduced, although unequal alterations reduced the load considerably.

121. The effect of long-term creep was more important. The Authors had used a value for the creep co-efficient of 0.1×10^{-6} $\xi/\text{lb sq. in.}$ for the precast concrete and nil for that in situ to fit in with conditions at the test age; the normally accepted value over a long period was 0.3×10^{-6} $\xi/\text{lb sq. in.}$ Incidentally, the formula in § 16 was incorrect; δA_c and A_c should be added to the left and right hand sides respectively to make F_2 the force.

122. At a specific creep of approximately 0.15×10^{-6} the effect of creep on the cracking load was reversed. Below this value, as in the Paper, creep increased the cracking load, but above it the load was reduced. When a value of 0.23×10^{-6} was taken for the precast concrete the cracking load decreased to between 7 and 8 tons; with a specific creep of 0.3×10^{-6} the preflex was almost eliminated and the cracking load was very small. Similarly the addition of creep to the in situ concrete had the effect of decreasing the cracking load and increasing the deflexion.

123. Turning to the load capacity of the beams it was interesting to see that the Authors confirmed that the ultimate load capacity was not increased by preflexion. Thus, on a load factor design where deflexions did not rule, there was no advantage to be gained for the extra cost involved.

124. In addition to the unpreflexed beam which was tested it would have been interesting if a further test had been carried out on a straightforward composite beam of the same section, as it was unlikely that anyone would bother to precast the bottom flange concrete onto a steel beam unless they were preflexing it. Shrinkage and creep on this precast concrete masked the comparisons between a preflexed beam and a straightforward composite one. A normal composite design on the section gave a design load of 10.3 tons using good quality concrete and $m=12$.

125. The criterion for safe load from the tests had been set at a limiting deflexion

of span/600. CP 117, Part 1, specified the same deflexion criterion as B.S.449, that is, the live load deflexion should not exceed span/360. It was true that some of the 'live' load applied in the test would in fact, be dead load in an actual structure.

126. Whilst a reduction in the limiting deflexion would appear to increase the load capacity of both beams, the effects of creep must be allowed for, and, in any event, stress, and not deflexion, was more often the limitation for design load with composite construction. It was misleading to indicate (Fig. 10) the same theoretical deflexion line for an unpreflexed beam as for a prefixed one. The calculation appeared to have been based on an uncracked section which was not true of the unpreflexed beam; the impression was given that only the prefixed beam followed the forecast deflexion and that the unpreflexed one behaved in a much less reliable way.

127. Before experimental results such as these could be applied to design, long-term tests were needed. Had the Authors any such tests planned?

Mr R. G. Tyler (Road Research Laboratory) wrote that as part of its programme of research into the effect of creep and shrinkage in bridges, the Road Research Laboratory was carrying out tests on four Preflex beams in collaboration with Glamorgan County Council. These beams were to be used for the bridge over the River Ogmore on the Bridgend Inner By-pass. A comparison of the results with those given in the Paper might be of interest. Two of the beams were 90 ft long and the other two 54 ft long; the steel joints used were 36 in. deep. The purpose of the tests was to ascertain the stress loss in the precast concrete.

129. The beams were prefixed and released in the period 27 October to 9 December, 1964 and had been left in the casting yard pending transportation to the site in the late summer of 1965. Strain readings had been taken using Maihak surface vibrating wire gauges on the steel and similar gauges of R.R.L. type buried in the concrete. On release, the stresses at the critical section at the top of the precast concrete were 1045 and 982 lb/sq. in. respectively for the two larger beams and 1660 and 1380 lb/sq. in. for the two smaller beams. When the last results were taken in April 1965 these stresses had fallen to 760, 620, 1200, and 870 lb/sq. in. respectively. These stress values appeared to be generally above those quoted by the Authors in Table 2 for the same critical section. The main reason for this appeared to be connected with the fact that the beams tested by the Laboratory were larger than those tested by the Authors. As a consequence of this, the ratio of the depth of the precast concrete to the depth of the joist was one third for the 90-ft beams and 4/9ths for the 54-ft beams, whereas for the beams tested by the Authors the ratio was about 1:2. The stress difference across the depth of the concrete for the beams tested by the Laboratory was, therefore, smaller and the stress at the top of the precast concrete correspondingly larger. The Authors' conclusion (§ 57) that the precast concrete cracked long before the Baes-Lipski design load was attained seemed less likely to be true for the larger Preflex beams.

130. Fig. 14 showed the measured strains for one of the 90-ft beams tested by the laboratory. Strain diagram (a) was the prefixion diagram arising from the moment in the joist alone while diagrams (b), (c), and (d) gave the change in the strains from the time of release up to the time the readings were taken. The net strain in the composite beams at any time was the sum of (a) and the appropriate release diagram.

131. The steady shift of the neutral axis of the composite section with the passage of time was to be noted. It was apparent that should the concrete fail or suffer infinite creep then the neutral axis would return to the neutral axis of the steel joist alone.

132. From the positions of the neutral axes on the release diagrams the effective modular ratio, m , might be calculated. This modular ratio was based on the effective modulus of elasticity E_c up to the particular time considered, i.e. the ratio of the stress calculated from the strain readings to the total strain measured at that time. The modulus of elasticity for steel was found to be 30×10^6 lb/sq. in. from tests on samples.

FIG. 14: STRAINS IN 90-FT LONG PREFLEX BEAM

133. The coefficient ϵ' in the formula $m = m_0(1 + \epsilon')$ which was used in the Baes-Lipski method and which was mentioned by the Authors (§ 11) might be calculated using the value m_0 at release for the four beams as shown in Table 7.

134. It was clear that if the final value of ϵ' was to be 0.50 as had been suggested by Dischinger (§ 11) then the beams must be taken to the site within about two weeks after release and built into the bridge immediately. In this way the relief of the compression in the bottom flange caused by the dead weight of the in situ concrete and the consequent creep recovery should keep the value of ϵ' down to 0.5; otherwise the losses would be greater, as implied by the Authors.

135. As more losses were likely to occur in the beams tested by the Laboratory prior to their delivery to the site, it had been clearly demonstrated that, in general,

the completion of bridge abutments should coincide as nearly as possible with the release of the Preflex beams. In this way the losses would be kept to a minimum and the efficiency of the system assured.

136. The adoption of suitable values for ϵ' depending on the lapse of time before the use of the beams would appear to be the simplest, yet a quite sufficient way of calculating the losses arising from creep and shrinkage. The free shrinkages measured by the Laboratory in six 14-in. $\times$ 6-in. $\times$ 6-in. companion specimens cast and stored with the beams on the site were 20, 35, 30, 65, 115, and 115×10^{-6} in./in. from the day after casting up to April 1965. The maximum values were of the same order as those measured at other sites by the Laboratory.¹⁸ Bearing in mind that the shrinkage of the mass of concrete in the beam flanges was likely to be less than the shrinkage of the

TABLE 7: BEAM C 8, 90' SPAN

Date:	16.11.64 (Release)	17.11.64	17.4.65
m	5.39	7.04	9.27
ϵ'	0	0.31	0.72
E_0 lb/sq. in. . . .	5.57×10^6	4.26×10^6	3.28×10^6

Beam C 13, 90' span

Date:	23.11.64 (Release)	9.12.64	17.4.65
m	6.42	8.36	12.28
ϵ'	0	0.30	0.91
E_0 lb/sq. in. . . .	4.67×10^6	3.59×10^6	2.45×10^6

Beam E2E, 54' span

Date:	8.12.64 (Release)	14.12.64	17.4.65
m	5.17	6.85	7.96
ϵ'	0	0.33	0.54
E_0 lb/sq. in. . . .	5.80×10^6	4.38×10^6	3.77×10^6

Beam E2F, 54' span

Date:	9.12.64 (Release)	14.12.64	17.4.65
m	6.85	9.78	14.7
ϵ'	0	0.43	1.45
E_0 lb/sq. in. . . .	4.38×10^6	3.07×10^6	2.04×10^6

companion specimens it was doubtful whether a separate calculation for the effect of shrinkage was justified. Taking the most unfavourable results for the Laboratory's tests, i.e., those for beam E2F, it would appear that $\epsilon' = 1$ when the delay between release and use in the bridge was about 4 months. The full results of the Laboratory's tests were to be published in the form of a Road Research Laboratory Report.

The Authors, in reply, were very interested in the diversity of opinions expressed in the discussion and in observations made about the suggested method of analysis.

138. This method of analysis was put forward by the Authors when it was realized that the standard method of design up to the time of the tests could not account for the behaviour of the test beams under the combined effects of shrinkage and creep.

139. The Authors wished to state that a tensile strength (c_t) of -450 lb/sq. in. for the precast concrete was used in the calculations for the following reasons:

- (1) it served as a basis for comparing the different methods of analysis mentioned in the Paper;
- (2) research had shown c_t to vary between $1/1.3$ and $1/1.6$ of c_r (13); c_r was shown by experiment to be of the order of 600 to 650 lb/sq. in. thus making c_t of the order of 450 lb/sq. in.
- (3) shrinkage cracks had been observed on the upper surface of the precast casing *before* release. These cracks closed on release but this meant that the concrete was cracked to start with.

140. It would be futile to confuse creep strains with 'instantaneous' strains (occurring during the test) as Mr Ford suggested, e.g. Fig. 4 gave the strain at the top of the precast casing two months after release as $+24 \times 10^{-5}$ units (including creep strain). There was no record of this strain for zero test load at the time of the test (i.e., 4 months after release) due to the loss of the Demec targets, but it must have been greater than $+24 \times 10^{-5}$. The change in strain due to the observed cracking load was only -11×10^{-5} (instantaneous strain) and the estimated change in strain due to the weight of the in situ concrete, spreader beam, shuttering, etc., was of the order of -14×10^{-5} . This showed that although the nett strain at the top of the precast casing was greater than $(24 - 11 - 14) \times 10^{-5}$, i.e. greater than -1×10^{-5} , the concrete had already cracked.

141. If by 'working load' Mr Ford meant the Baes-Lipski design load then the precast concrete would most certainly be cracked through and it would be unrealistic to calculate stresses on the basis of the section modulus of the full composite section due to loss in the moment of inertia.

142. The Authors were interested to know that similar tests, carried out by Canadian research workers under laboratory conditions, confirmed their conclusions.

143. The joist used by Dowsley⁷ was a 16 in. $\times$ 11 in. R.S.J. (16 WF96 section). This was twice the depth of that used by the Authors and results obtained were similar. It would seem that the size of the R.S.J. did not improve matters as much as Mr Blake had inferred. Losses due to shrinkage and creep were higher than the Baes-Lipski analysis made out.

144. In § 72, Dr Lash stated that the ultimate strength might be predicted on the basis of a fully plastic distribution. The Authors found that when the ultimate load of 42.3 tons was reached in the case of beam L2, failure occurred through crushing of the in situ concrete slab and the load fell rapidly to around 29 to 30 tons, where it remained constant. Attempts to increase the load resulted in increasing deflexions due to plastic behaviour of the joist. On the basis of a yield stress of 25 tons/sq. in. and a plastic modulus of 49 cu. in. the applied load W worked out at approximately 30 tons, which agreed with the experimental value.

145. Mr Nicholas described the Preflex system as the use of '... high yield steels to their best advantage, without the disadvantage of excessive deflexions ... casing used to increase sections' inertia'. These were surely the aims of prestressed concrete as well, where high tensile wires were used to improve deflexion characteristics by preserving the moment of inertia of the uncracked section. His further statement that '... it should not be considered as a prestressed beam', seemed entirely to contradict his previous statements.

146. Mention was also made of the slow progress of cracks as compared with those in a prestressed beam which hastened to ultimate collapse. This was obviously because the Preflex beam had a large cross-sectional area of steel (9 to 10% of the area of concrete) unlike the conventional prestressed beam which was only lightly reinforced. It was well known that cracks in concrete beams tended to widen after repeated cycles of loading. What may have been a crack of the order of 0.0039 in. at 103% of the Baes-Lipski design load could well have been doubled after several cycles of loading and unloading. It was a dangerous practice to define design loads by allowable widths of cracks as implied by Mr Nicholas.

147. Regarding jacking loads and imparted stress, the Authors would always prefer to rely on stresses computed from strains in the steel (after the release of residual stresses) as in their opinion strains measured on the R.S.J. with a Demec gauge gave rise to far more reliable stress indications than those computed from hydraulic pressure gauge readings. In this connexion the Demec gauge was found to be superior to the usual electrical resistance gauge as the former was less liable to drift over long periods of time and under the effects of temperature changes, humidity, etc.

148. Mr Nicholas referred to the hypothetical case of returning the steel stress at level D to 36 600 lb/sq. in. with an applied moment of 105 ton ft. According to the Authors' calculation the application of the cracking load of 12.1 tons resulted in a stress of 23 018 lb/sq. in. in the steel. This stress was computed using the value of 130 cu. in. for the section modulus, but Mr Nicholas should not have used the same modulus with an applied moment of 105 ton ft, caused by a load of 31.5 tons, as the concrete at this stage was severely cracked and the 'uncracked' section modulus did not apply. A glance at Fig. 8 would indicate cracking to approximately 6 in. above the soffit. The section modulus would thus be considerably reduced and the moment required to return the stress at level D to 36 000 lb/sq. in. would also be reduced. The Authors did not accept the Baes-Lipski analysis as being a true indication of the stresses in the section, as it was based on 'effective' rather than 'instantaneous' moduli, even where the applied load was concerned.

149. Mr Lipski seemed to have overcome the problem of shrinkage and creep by disregarding the precast casing (§ 85). This did not seem to agree with his original theory as set out in 'La Poutre Preflex'.^{1,2} In the Authors' opinion this casing was of considerable importance as its performance under shrinkage and creep governed the preflexion stresses in the joist. Mr Lipski's other comments were closely related to those of Mr Nicholas, which were dealt with above.

150. The Authors thanked Messrs Leung, Haseltine, and Saffery for pointing out the omission of A_c in the expression for F_2 in § 16 and would hasten to assure them that the factor A_c had however not been omitted from the calculations from which Table 2 had been prepared. Regarding the figures of +318 and +4011, these had been checked and found to be correct.

151. The Authors believed that beams had been constructed along the lines suggested by Mr Leung by Bolton and Paul, Norwich, except that the joist had not been preflexed. Pretensioned wires surrounding the lower steel flange were used.

152. It would be possible to attach cable ducts to the lower steel flange and to cast them into the precast concrete. These could then be post-tensioned before the in situ concrete was added to make up for losses in prestress. Care would have to be taken to ensure that stresses after release were not excessive. This could be checked

by the upward deflexion of the joist. Mr Tyler's suggestion that the beams to be used within two weeks of release was a more economical solution.

153. It would be difficult to comment on the results of Ir. Bruggelings tests without a knowledge of the dimensions of the specimens, properties of the concrete, instrumentation, etc. Very little relief could be obtained by increasing the percentage of the secondary steel in the precast casing, as this would only have the effect of distributing the shrinkage cracks more evenly throughout the concrete, not eradicating them. The Authors had used a percentage of 1.5 and not 1.0 for this reinforcement, as Ir Bruggeling had suggested; this was not far removed from the figure of 2 he had recommended.

154. The Authors agreed with Mr Blake regarding the difficulty of preflexing small shallow beams such as the ones tested and ensuring adequate compression at the top of the precast casing. However, although due allowance had been made for this shortcoming, it was still not possible to reconcile the behaviour of the test specimens with the Baes-Lipski design procedure. The effect of studs passing through the precast casing was negligible. This was amply proved by the fact that several of the initial cracks occurred between studs and not at sections where the presence of studs had reduced the sectional area.

155. In § 110 Mr Blake raised an interesting point regarding strain at the top of the precast casing. The section moduli at this level, with and without the in situ topping, were 224.5 cu. in. and 444 cu. in. respectively. The former would be even less in practice. Hence, it was true to say that the applied loading during the test would influence the strain at this level more than the live load would in practice. Messrs Haseltine and Saffery had made a similar point when they referred to the parallel case of limiting deflexions in § 125.

156. It was regretted that the Authors misrepresented the principle regarding the relationship between the highest design stress in the steel and the preflexion stress (§ 112) as set out in the technical writings of Messrs Baes and Lipski. However, this did not in any way affect the comparison of the various design procedures.

157. The Author's appreciated the remarks made by Messrs Haseltine and Saffery about shrinkage and creep and agreed that, in the long term, the effect of preflexing would be lost completely. Mr Tyler had suggested keeping the storage period down to a minimum to reduce the effect of creep.

158. In § 126 Messrs Haseltine and Saffery were correct concerning the theoretical deflexion line in Fig. 10, which was based on the uncracked section. Fig. 10 was not intended to emphasize this aspect of the unpreflexed beam but to compare it with the preflexed beams regarding deflexion, 'hysteresis', and the large residual deflexions after release of loading.

159. No long-term tests had been planned, but the results of Mr Tyler's tests could also be considered to show how E_0 and m varied with time. If m_0 was the modular ratio just after release, then according to Table 7, m , the modular ratio after approximately 4 to 5 months was 1.72, 1.91, 1.54, and 2.54 times m_0 for the four specimens tested and not 1.5 m_0 , the figure used in the Baes-Lipski analysis. The Authors generally agreed with Mr Tyler's remarks but had observed shrinkage to be much higher than that noted by Mr Tyler. Over a period of approximately 4 months the free shrinkage of concrete prisms cast, stored and cured with the beams were 5×10^{-4} , 2.1×10^{-4} , and 2.5×10^{-4} while that of the in situ concrete was 1.9×10^{-4} , 2.2×10^{-4} , and 2.8×10^{-4} over a period of only 15 days.

REFERENCES

14. WELLS A. A. Influence of residual stresses and metallurgical changes on low stress brittle fracture in welded steel plates. *Weldg Res.*, 24 (4), 183s-191s.
15. HARRISON J. E. Further fatigue tests on fillet welded specimens subject to prior overloading. *B.W.R.A. Report, Brit. Wldg J.*, 1965, 12 (May), 255-258.

16. BRUGGELING A. S. G. *De Spanningsverdeling in enn Preflex Constructie*. N.S.M., Alphen aan den Rijn, 1964.
17. BRUGGELING A. S. G. The stress-distribution in preflex beams. *Heron*, 1965, **13** (1), 1-27.
18. KERENSKY O. A. and LITTLE G. Medway Bridge: design. *Proc. Instn civ. Engrs*, 1964, **29** (Sept.), 19-100. Discussion: *ibid.*, 1965, **31** (June, 160-204).