Discussion.

The Author, in introducing his Paper, mentioned that the disadvantage of the system using four jacks instead of the double-action jack was that, naturally, the anchoring-block thus formed had to stand outside the concrete member, instead of being embodied in it; but that was a very small disadvantage, and the system was being used for the construction of roof principals for sheds of 120 feet span.

The tests of the 62-foot test-beam had been very extensive, and, in fact, thousands of readings were registered. Only a few of them could be mentioned in the Paper, but he would be pleased to afford access to the full record of tests. The results obtained showed that in the case of a fully pre-stressed member, that was, a member acting under load without any tensile stress, the actual stresses tallied very closely with the theoretical calculations. In that respect such a member might be placed on a par with a member made of a homogeneous material such as steel. In the case of a heterogeneous material such as reinforced concrete, calculations were usually made on the assumption that the concrete carried no tensile stress; in other words, the calculations corresponded with the state of the member when the tensile zone had been destroyed and the member was approaching failure. Consequently, under the conditions in the neighbourhood of normal load, that was, up to a stage when the concrete did, in fact, withstand a certain tensile stress, the pre-stresses were found to be different from those calculated, and the calculation was really to supply some estimate of the factor of safety. He ventured to draw attention to that fact because it defined the different point of view which had to be adopted when considering calculations relative to ordinary reinforced concrete and pre-stressed concrete. The slight differences which the tests revealed between the measured stresses and those resulting from calculation were due partly to the fact that Young's modulus varied with the magnitude of the stress and partly to the fact that the calculations were based upon the assumption of linear distribution of the stress, plane sections remaining plane during the bending of the member. That assumption, of course, was no more in accordance with reality for a homogeneous material like steel than for a heterogeneous material. The tests proved that the degree of concurrence between the actual and calculated stresses for pre-stressed concrete was quite comparable with that in the case of homogeneous materials. For instance, the strains measured rendered it possible to calculate the values of Young's modulus and to tabulate them in the Paper, and the concurrence in the case of the maximum load was found to be very close, namely, 3,750,000 lb. per square inch as against 3,800,000 lb. per square inch.

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The deflexion of the 62-foot test-beam corresponded with that of a steel girder of equal depth stressed to only 7,000 lb. per square inch, instead of the usual figure, so that the deflexion of the pre-stressed beam was much smaller than that of a steel girder of equal depth and the usual stress, the explanation being that the whole of the section was active as regards the moment of inertia, because there was zero tension on the bottom face of the girder instead of about half-way up. Of course, as soon as the tensile stress became important the deflexion increased, and that was shown very clearly towards the end of the test of the 28-foot beam.

The President said that he was sure that members would agree that the Author's introduction of his Paper had been of a very interesting nature. He believed that that was the first occasion on which the important subject of pre-stressed concrete had been brought before a meeting of The Institution.

Mr. W. J. E. Binnie considered that the British Section of the *Société des Ingénieurs Civils de France* owed a debt of gratitude to the Author, who was, in fact, its founder. The Author had arranged for French civil engineers to visit England from time to time and read Papers for the edification of British civil engineers. Those Papers had always been of considerable interest and had kept British engineers cognisant of what was going on in France. Unfortunately the present circumstances were such as to make the activities of the British Section of the *Société des Ingénieurs Civils de France* extremely limited.

Mr. T. Peirson Frank observed that the Author had made the subject of pre-stressed concrete peculiarly his own. Mr. Frank had not had an opportunity of checking the formulas or of working out the equations given. He had been impressed, however, by the material contained in the Paper, and he thought that the subject was topical, because it bore on the question of economy.

As the Paper was a record of facts, it was not possible to criticize it, but he would like to ask a few questions.

He did not know whether the Author had had experience of pre-stressing by heat methods as well as by the use of jacks; but, if so, he would like to know which method the Author had found to be the more efficacious. Personally he thought it would be the jacks, because they could probably transmit the desired stress a little more accurately.

He would also like to know whether the Author had re-tested any of the pre-stressed members to see whether there was any loss by shrinkage, by creep, or by elastic deformation due to age. In connexion with fairly large engineering works with which he was associated, he had been told that, in certain pre-stressing done on reinforced-concrete girders, a pre-stress of 45,000 lb. per square inch had been applied (by the heat method, incidentally), and on re-testing in May 1940, after about 90 to 100 days, it had been found that there was a reduction to 40,000 lb. per square inch, which was equivalent, in a period of 90 or 100 days, to

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something more than 10 per cent. That made him wonder, when reading the Paper, whether with the passage of years there would be a continuing reduction in the pre-stressing and if a rather different factor of safety would have to be used. Under war conditions, rather different methods had to be adopted in certain types of construction; but, when the war was over, would engineers have to plan for future wars? If so, one would have to visualize a pre-stressed reinforced-concrete beam being cracked sufficiently to allow the pre-stress to be removed almost entirely from the steel members. If that happened, of course, a collapse might occur where otherwise one would not be expected.

Figs. 26 and *27*, pp. 112, 113, *ante*, indicated that the comparison of the measured stress with the calculations was very satisfactory and that, for the heavier loads at all events, the actual stresses were lower than those calculated. He thought that, of all the statements in the Paper, that was probably the most comforting. It was a check, and it showed that the calculations erred a little on the safe side. Perhaps the Author, in his reply, would care to add a word or two more upon the comparison between the calculated stresses and the stresses registered during test.

Dr. Oscar Faber observed that most of his remarks would be in the form of questions which he had not found quite clearly answered when he studied the Paper and had tried to check some of the statements made.

In the first place, in the case of the 64-foot beam made of thirteen precast sections, illustrated in Figs. 4, Plate 1, no tests were described in the Paper, and he would therefore like to ask the Author whether, in fact, that beam had been tested at all. If so, could the results be published? Most of the tests referred to in the Paper related to the 62-foot beam, which was illustrated in Figs. 12, Plate 2. In regard to that beam, it was stated that it was not made of precast sections, but was cast in situ, and that some of the reinforcements went to the end whilst others did not. That emerged clearly from the description, and therefore it was quite clear that the method of pre-stressing described could not have been applied to that beam. He thought it would add to interest in the Paper if the Author would state how the 62-foot beam was, in fact, pre-stressed.

He would like to ask whether the Author had considered how much additional stress occurred on the steel as a result of the loading. Dr. Faber did not think that the point had been mentioned in the Paper. It was quite clear that there would be some additional stress in the steel, because the bottom flange was lengthened and, in some of the beams, the steel was brought up to the neutral axis at the end, so that the steel lay on a curve; and it was clear that as the deflexion increased that curve would become longer. Therefore there must be some additional stress on the steel, and, seeing that the Author started with an initial stress of about 72,000 lb. per square inch, it would be of interest to know what the additional stress might be.

With regard to creep and shrinkage, he gathered from the Paper that,

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in the case of the 62-foot beam, to which the initial tests related, the Author had calculated that, if he started with an initial stress of 78,000 lb. per square inch, the stress, after allowance had been made for shrinkage and creep, might be about 57,000 lb. per square inch; but Dr. Faber did not think that it was stated in what period of time that shrinkage and creep had occurred, because he could not find anywhere in the Paper a statement as to the age of the concrete when the test was made. He considered that it was impossible to calculate the effect of shrinkage and creep except in regard to some definite time-period, because the shrinkage and creep went on for years, although most of it occurred during the first year. He had himself made some preliminary calculations, as a result of the study of creep which he had presented to The Institution a few years ago[1], and it appeared quite clearly that as a result of creep and shrinkage the shortening of a piece of concrete under stress might, at the end of a year, be three times as much as the elastic shortening. If that were applied to the present Paper, it would be found that in certain circumstances more than 50 per cent. of the pre-stressing might disappear after a year; and he would like the Author to comment upon that.

With regard to the savings to which reference was made in the Paper, he thought it should be remembered that, when mild steel was replaced by silico-manganese steel of much greater strength, to which much higher stresses were applied, savings in the *quantity of steel* might be effected; but the actual financial saving would be considerably less, because a much more expensive material was employed and the cost of the operations involved in the pre-stressing had also to be taken into account. It was clear that the financial saving would be something very different from the saving of 75 per cent. of steel, and there might be no saving at all.

Reverting to the question of economy, it was stated several times in the Paper that if the beams in question were compared with beams of ordinary design they were found to be much more economical. He did not think that the mixture of the concrete was stated in the Paper; but, assuming that it was 1 : 2 : 4, it seemed to him that in a beam of ordinary design one would presumably be limited to the stress which the Code of Practice would allow to be put on concrete of that character, namely, 950 lb. per square inch; but it was clear from *Figs. 8* that the Author had started with a stress in the concrete of 2,500 lb. per square inch. Therefore it appeared that much of the economy ascribed to pre-stressing was really economy due to stressing concrete to much higher stresses and to the substitution of a much stronger steel for the mild steel which was normally used. He did not understand why it was permissible to stress 1 : 2 : 4 concrete to 2,500 lb. per square inch if only 950 lb. per square inch were allowed in the case of beams of ordinary design. That clearly was economy

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[1] "Plastic Yield, Shrinkage, and other Problems of Concrete and their Effect on Design", Minutes of Proceedings Inst. C.E. vol. 225 (Session 1927–28, Part I), p. 27.

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obtained by a reduction in the factor of safety; and that there was a real reduction in the factor of safety was borne out by the fact that the tested factor of safety on the pre-stressed beams in all cases was about 2·3, whereas on other tests it was generally much higher. Moreover, in all cases the beams apparently failed in compression, so that apparently it was the stress in the concrete which limited the stress in the beams. Dr. Faber considered that that was an indication that the Author was really working on a reduced factor of safety by having very much higher stresses in the concrete. Dr. Faber would undertake, without pre-stressing, to design and test a beam of the same lightness, provided that he were allowed the same high stresses.

A point of particular interest was that the top flange of the 62-foot beam was only 9¼ inches wide; yet the span was 62 feet, and that, of course, gave a ratio of length to breadth of 80 : 1. He would like to ask the Author whether during the test there was any restraint against lateral deflexion. He did not see any indication of such restraint, and, if there were none, it seemed very interesting that one could have a top flange to a beam having the high ratio of length to breadth of 80 : 1 without any lateral support at all. He believed that, according to the London County Council Regulations, it would not be permissible to put any stress at all on that beam. He would like to ask whether there was something very peculiar about the beam, or whether the Regulations might be due for revision in that particular respect.

Mr. C. S. Chettoe considered that the idea of pre-stressing concrete was very attractive, owing to the absence of cracks, to the fact that the whole depth of the section of the beam could be used, to the lightness of the beams, and to the saving in construction-depth. There was also the question of economy, which was perhaps modified to some extent by the factors which Dr. Faber had mentioned. Thanks were due to the Author for the work he had done in pursuing the possibilities of pre-stressed concrete and for his two very interesting Papers.

When the Ministry of Transport wished to augment its stores of emergency bridge material in 1940, it had decided, bearing in mind the factors mentioned above, to construct a number of beams of pre-stressed concrete, with spans ranging from 15 feet to 50 feet, to make up bridges 20 feet wide. The original proposals for the beams were illustrated by cross-sections in the Author's Paper of July 1940. The 50-foot spans were constructed of H-sections, 12 inches wide, and the original design of the other beams was given in that Paper.

Their shape was designed in order to effect greater economy in material. It was intended to tie them together by rods carried through the top surface, but it was felt that as the beams were to be stored for emergency purposes they should have a residual factor; and as they might be useful for railway bridges, it was thought desirable to have a smooth soffit. Therefore the shorter beams, with spans ranging from 15 feet to 40 feet,

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were re-designed and made of the type shown in Figs. 20, Plate 2. That beam was a typical beam of the 28-foot span (all the beams were of that type except the H-beams), and the Southall test, therefore, was a test on a typical beam for one of the bridges.

The beams were constructed on three sites by the method described in the July 1940 Paper, in which the wires were stretched between anchorages about 600–800 feet apart, depending upon the sites and the room available, and three to four rows of beams were laid out. The wires were stretched between the anchorages by means of jacks. Beams were cast around the wires, the pressure of the jacks was released when the beams had hardened, the wires were cut, and the beams were subsequently stored. That method had been adopted before in a case which had been referred to by the Author, in which a very large number of beams were constructed and the results were particularly good; but in the case under discussion some sites had to be found in a hurry and it was necessary to have rather massive anchorages; therefore the cost per beam was higher. The cost depended largely upon the number of beams produced, and when the number was limited it might be better to adopt the alternative method of casting them first and pre-stressing the wires by jacking against their ends.

In designing the beams, it was intended that they should be placed side by side, and that vehicles could, if necessary, run over them without any form of road surface; but in that event there would not be very much distribution between the beams, even in the type of beam finally adopted, where the vertical face of one beam would come up against that of the next beam, as an error in construction might occur, and it did not follow necessarily that the sides of the beams would be in contact. It would be possible to improve distribution by putting down timber or sleeper tracks. Ultimately, when the beams formed part of the permanent structure, it would be possible to have a reinforced-concrete deck with a proper camber, and on that vehicles would run satisfactorily. The original intention was to arrange holes passing right through the deck side by side, through which wires, cables, or rods could pass, the tension being sufficient for the beams to come into contact; but Mr. Chettoe doubted whether even then contact would eventually be obtained, owing to the difficulty of obtaining a perfectly straight face on the side of the beam. Another difficulty was that the beams might have to be used for re-decking skew bridges, in which case the holes would not be opposite one another and could not therefore be used.

Considerable care was required in stacking and shifting the beams. They had to be slung from the ends and must not be supported at all in the middle.

Mr. Ewart S. Andrews observed that one statement which the Author had made in his earlier Paper, and which he obviously still believed to be true, was that it would be a waste of money to use very strong steel in beams without pre-stressing, because the elastic stretch of the steel would

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cause cracks to occur in the concrete. Mr. Andrews did not believe that that was true, and he would like to see a test carried out on two identical beams, of equal area and of the same quality of steel, one pre-stressed and the other not pre-stressed. He doubted whether the advantage of pre-stressing would pay for the cost of carrying out the pre-stressing.

Dr. Faber had pointed out that the factor of safety in the two beams for which test results were given in the Paper was low. It was rather difficult to ascertain that factor, because the results were given in terms of the superimposed load being so many times the design superimposed load; but Mr. Andrews imagined that the dead load had to be taken into account, and that the apparent figures in the Paper were higher than the real factors of safety.

He would also like to know whether, in the relative areas given for non-pre-stressed steel and pre-stressed steel, the former was of the same quality as the latter, and, if so, what working stress was taken in the calculations. He believed that high-tensile steel had a much higher value as reinforcement than was accorded to it by most of his friends and colleagues, and with high-tensile steel he would be willing to adopt much higher stresses than those in customary use.

The danger of cracking was, of course, a real one. He had no doubt that, with very high stresses, some cracking would occur, but also he had no doubt that with ordinary steel at ordinary stresses there was some cracking. He believed that every reinforced-concrete beam contained cracks, although they could not be seen; and whilst the increased cracking might be bad in exposed positions, he did not think that it would matter in buildings.

Dr. E. C. Smith considered that pre-stressing was one of the greatest advances made in reinforced concrete since its inception.

He had observed that the factor of safety of the 62-foot test-beam was not so high as that of the 29-foot test-beam. Apparently that was due to the flanges of the larger beam being extremely slender, and therefore that beam was not so well suited to resist buckling stresses as was the shorter beam. In practice the roof trusses would be stiffened to some extent by cross-beams or the roof covering, so that they would have an even higher factor of safety than that shown by the test; that was to say, both would have a factor of safety of more than $3\frac{1}{3}$ at a comparatively early age for the concrete to be severely stressed.

It would be interesting, however, to know what precautions against a tendency to buckle were adopted in the pre-stressed beam made up of a large number of precast elements (Figs. 4, Plate 1), especially immediately before the grouting-in of the cable.

Adopting the notation referred to by the Author, Dr. Smith found that the formula for the tension in the steel after elastic deformation was as follows:—

$$f_{so} = \frac{f_{sp'}}{\dfrac{E_s}{E_c}\dfrac{A_s}{A_c}\left(1 + \dfrac{j^2}{i^2}\right) + 1}$$

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where i denotes the radius of gyration of the concrete section, and j denotes the distance of the pre-stressing steel from the centroid of the concrete section. Testing that formula at limiting values for E_c, it would be seen that, if $E_c = 0$, $f_{so} = 0$, and, if $E_c = \infty$, $f_{so} = f_{sp'}$, which obviously were the correct results. Another point was that, in calculating the moment of inertia of the beam section, the area of steel had to be omitted when obtaining stresses due to pre-stressing, but it had to be taken into account when finding the stresses due to the dead and live loads. Pre-stressing could be adapted so that it would produce a concrete virtually as strong in tension as in compression; therefore it offered a wide scope for both large structures and small precast members.

Mr. John Cuerel observed that the work described was perhaps more limited in its scope than some of the Author's earlier work. Nevertheless it was valuable as demonstrating that the behaviour of pre-stressed beams under load approximated closely to that anticipated from purely theoretical considerations. He was not surprised that the beams carried the design loads with an adequate factor of safety, because the general principle was so simple that, with proper care, nothing could go wrong. The interesting point was that the pre-stressing converted the heterogeneous elements into the equivalent of a homogeneous elastic material.

He felt sure that, when certain difficulties had been overcome, the method would find increasing application. The data so far available suggested that it was particularly suitable for pre-cast products, and was not necessarily limited to those of small size. For pre-cast members the use of ultra-high-tensile steel was advisable—and usually essential—on account of the stress-decrease due to shrinkage and creep. Those strains, of course, resulted in a reduction in the tension in the steel; correspondingly the initial tension must be high, as otherwise it would vanish entirely in time. He considered that the necessary use of ultra-high-tensile steel was in itself disadvantageous. Not only did it tend to introduce bond and anchorage troubles—which could be overcome by attention to detail —but also it introduced a certain degree of prejudice, which formed a much greater obstacle than any technical difficulty.

Many engineers considered that a brittle glass-hard high-tensile steel, stressed well up towards its limit, was not an entirely suitable material for the majority of structures. That opinion might be modified when more experience had been gained, but it would certainly carry weight in the earlier stages of development, and would always form a hindrance.

He would suggest that the direction of the development of pre-stressing should, at all events at first, be altered to what he considered to be its finest application, namely, the construction of important structures, such as bridges of 300 feet, 500 feet or 700 feet span, and also, perhaps, the well-designed skyscrapers which he hoped to see after the war. In such structures the use of ultra-high-tensile steel would not be essential technically, and would not be advantageous economically. That resulted both from

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the high proportion of dead load to total load and from the restraint against indefinite creep afforded by non-pre-stressed members. He would propose the use of a medium-high-tensile rolled steel, which could be produced in the form of small rounds, with an ultimate strength of 45–50 tons per square inch and a yield-point of about 55 per cent. of the ultimate strength—say 60,000 lb. per square inch. That steel would have the yield properties of mild steel with only slightly less elongation, and would be readily weldable. It could be produced, he was assured, at a cost only slightly higher than that of mild steel; and a suitable working stress would be of the order of 40,000 lb. per square inch. He considered that the property of weldability was very important when considered in relation to the design of details, to the pre-stressing arrangements, and to anchorage. In connexion with the 62-foot test-beam, a weldable silico-manganese steel had been mentioned by the Author; but, in the absence of further details, he, as a welding man, doubted its ready weldability under field conditions, and also its possession of yield-point properties.

With regard to procedure, during construction the bars to be pre-stressed in both the flange and the web (assuming a large structure and the use of the steel which he had suggested) would be placed in cored or tubed holes, and just before de-centering would be pulled up to about the working stress of 40,000 lb. per square inch. On removal of the supports the dead load, which would amount to 40–60 per cent. of the total, and which would be assumed to be half the total, would tend to produce concrete tensions appropriate to a steel stress of 20,000 lb. per square inch. The concrete surrounding the pre-stressed bars would, of course, still be in compression, and would therefore creep, so that the steel stress would decline. The decrease with time would be measured and plotted and, after a curve had been obtained, the steel would again be pulled up to perhaps a modest over-stress, say, 45,000 lb. per square inch, as determined by the extrapolation of the curve. There would be, of course, a further drop due to the remaining creep, but that would be very small, since the concrete would already be mature; it would already have been loaded for a considerable period, and there would be practically no shrinkage left. The final asymptotic value of the stress could be obtained from the curve, and it would be limited to a figure only slightly less than the working stress, so that when the full load was applied the tension in the concrete would be confined well within the recognized safe tensile value. Therefore all the advantages of full pre-stressing would be derived from the method.

Another point to which he wished to refer was the emphasis laid by the Author upon the care and experience required in carrying out pre-stressed work. As many members knew, the development of high-grade normally-designed reinforced concrete had suffered considerably from the prevalent view that any type of work could be carried out largely by unskilled labour, with only a sprinkling of trained men and the minimum of

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direction and supervision, whilst reliance was far too often placed on men learning the job as it went along. He would urge the importance of creating organizations experienced in design as well as in construction, and equipped with the necessary specialist staff for carrying out the work.

Mr. C. E. Reynolds stated that a primary principle of M. Freyssinet's pre-stressing practice was that the pre-stressing force was so balanced with relation to the bending moment due to normal working load that no tension was developed in the concrete, whilst, if tensile stresses should be developed, they were so low that no cracking was produced. Conditions could be easily visualized under which temporary or even permanent tensile stresses sufficiently high to produce cracking could occur. For example, it was considered that the usual factor of safety adopted in the case of normal structures would cover accidental overloading. Apart from accidental overloading there was intentional overloading. For instance, when a structure was designed for London, and pre-stressing was specified for it, it was probable that the District Surveyor would require to be satisfied in regard to the strength of the members, not only by calculation, but also by test loads, and the usual requirement was the imposition of a 50-per-cent. overload. If such an overload were applied to the stress conditions illustrated in *Figs. 25*, a calculated tensile stress approaching 600 lb. per square inch might arise, that was to say, a condition slightly more severe than the conditions shown in *Figs. 28*; and, although cracking had not occurred in the Author's test-beam at that calculated stress, Mr. Reynolds considered that the stress would undoubtedly be sufficiently high to crack everyday concretes.

A further cause of tensile stress might be shrinkage. The coefficient of $\frac{0{\cdot}25}{1{,}000}$, assumed in the Author's calculations, was undoubtedly a reasonable figure for the first stage, as had been proved by the tests, and it was also, doubtless, reasonable for the ultimate shrinkage in structures wherein shrinkage was partially unrestrained, or in reservoirs or similar works where the concrete was in a more or less constant state of dampness. In a fully-restrained member, however, an ultimate shrinkage coefficient of almost double the stated value might not be unreasonable, in which case an additional tensile stress exceeding 200 or 300 lb. per square inch might be produced in the concrete, and that stress, either alone or combined with one or other of the tensile forces mentioned, might conceivably cause cracking. Once a crack had formed, the homogeneous nature of the section was destroyed. Unless conditions were favourable to autogenous healing in course of time, the concrete could no longer take tensile stresses, and therefore the tensile forces would be transferred to the steel, which, already highly stressed by the pre-stressing operation, would be called upon to carry still further stress. He would like to ask whether, in the Author's opinion, with a pre-stress of 35 tons per square inch, there was sufficient margin below the yield-point stress of 40 tons per square inch to take up any extra tension due to any or all of the causes

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that he had tried to describe, or to similar causes, bearing in mind that the actual stress in the steel after pre-stressing, as shown by the tests, was higher than that allowed in the calculations, and also remembering that an additional tensile stress of possibly 5 tons per square inch, calculated from the figures given in the Paper, was induced in the steel to correspond with the reduction in the compressive stress on the concrete from 1,200 lb. per square inch to zero.

Further, was it feasible, in the Author's opinion, to resist such accidental additional tensions, or even some of the tensions due to the normal working load, by the inclusion of additional unstretched reinforcement? Mr. Reynolds knew that the principle of introducing unstretched reinforcement was the underlying idea put forward by Dr. Paul Abeles[1]. Dr. K. W. Mautner[2] had already published in some detail his views on such proposals, and had expressed his general disapproval of them.

Regarding the margin between the stress in the pre-stressed steel and the elastic limit of that material, it was of interest to determine what was the magnitude of that stress with the concrete uncracked. Adopting values similar to those given in the Paper, the initial pre-stress was about 35 tons per square inch. The test results of one of the beams showed that shrinkage, creep, and elastic deformation caused a reduction of only 4·7 tons per square inch for normal load, increasing ultimately to 5·7 tons per square inch; the maximum additional stress due to elongation under normal load alone could be shown to be about 5 tons per square inch. Thus, without any excess loads and with uncracked concrete, in round figures, the actual stress might be $35 - 5 + 5 = 35$ tons per square inch. If loading in excess of the normal were applied, further stress would be added, and if the concrete should crack, still further stress would be imposed on the steel, which already had but a small margin below the yield-point stress of 40 tons per square inch.

Another small point relating to the steel stress was whether, when calculating the reduced pre-stressing force, some allowance should not be made for friction on the mild-steel collar or other retaining device, where the pre-stressed wires fanned out to the anchoring device as in Figs. 3, Plate 1. Although the angular change of direction was small (about 7 degrees), the friction resulting from the bearing force on the steel collar might reduce the applied pre-stressing force by anything up to 5 per cent. It was true that that was low compared with the calculated reduction (about 27 per cent.) due to shrinkage, creep, and elastic deformation. It might be that the difference between the calculated and actual stress reduction due to creep, etc., allowed sufficient margin to cover the reduction due to the friction mentioned. He raised that point because he had examined sketches of stretching devices wherein the stretched wires were bent through a much sharper angle than in the Author's apparatus.

[1] "Saving Reinforcement by Pre-stressing." *Concrete Constr. Engng.*, vol. 35, p. 3; July 1940.
[2] "Saving Reinforcement by Pre-stressing." *Ibid.*, vol. 36, p. 73; Feb. 1941.

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With regard to the grouping of the bars in the beam section in *Figs. 5*, as the test-beam was one-third the size of the actual beam, the vertical space between the superimposed layers of wires in the bottom flange of the actual beam would be $3 \times 0{\cdot}118$ inches, or less than $\frac{3}{8}$ inch for bars about $\frac{5}{8}$ inch in diameter. That was less than the $\frac{1}{2}$ inch which was regarded as the minimum allowable clearance in present-day practice, and he would like to be assured that, in spite of the vibratory process adopted, the concrete was well packed around the reinforcement and was entirely free from voids among the closely-packed bars. It had to be remembered that in the test-beam the space was even smaller—less than $\frac{1}{8}$ inch. Some such assurance appeared to be important, since pre-stressing produced high compressive stresses on the concrete around the bars.

He considered that the use of pre-cast sections combined with pre-stressing formed a noteworthy contribution to the standardization of construction which must inevitably take place after the war.

⁂ **Dr. E. Probst** observed that in addition to effecting a certain economy in the use of steel, the pre-stressing of reinforced-concrete structural members rendered it possible to obtain more than ordinary safety against cracking, owing to the low tensile strength of cement and, consequently, of concrete. The first modest attempt to utilize the pre-stressing of concrete to prevent its cracking in the tension parts of a reinforced-concrete arch, was made about 30 years ago. At the same time, tests on pre-stressed reinforced-concrete beams showed that cracks appeared at loads about 50 per cent. higher than those which caused cracking in non-pre-stressed beams of the same type.

The practicability of the idea of pre-stressing depended entirely upon the effectiveness of the devices used in the process and upon the anchoring of the steel reinforcement. In any case wide experience and great care were necessary in elaborate pre-stressing operations such as those depicted in the Paper.

Research-tests, in principle, should be made, as far as possible, on test-specimens made of the same concrete, and with the same size and distribution of the steel, as used in the structure, and conclusions drawn from the tests should be limited to the conditions of the test-specimens.

The Author, in his conclusions, had made a comparison of the calculated stresses with those measured at the test-beams. To ascertain the more or less divergent results of theory and measurement, certain assumptions had to be made. In his report of the tests made at Southall in 1940 the Author had referred to E_c, the theoretical modulus of elasticity of concrete in bending-compression, which had to be determined from the measured deflexions.

In the equation $\delta = M/E_cI$, the deflexion δ was measured, and the bending moment M could be easily determined at any section of the beam for every load. The moment of inertia, I, of a reinforced-concrete cross-

⁂ This contribution was submitted in writing.

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section was practically the same as for a homogeneous material, so long as no cracks resulted from loading. Up to the cracking-load the deflexions were extremely small, as was shown in Table III, but they increased after cracking. Moreover, in order to obtain exact measurements in field tests, the effect of the sun's rays and of wind should not be neglected. The values of E_c, as determined from the equation $E_c = M/\delta I$, should therefore be used with care.

After the formation of cracks, the value of I depended upon the position of the neutral axis of the cross-section, which moved as the load increased, and upon the value of the modular ratio m. That rendered it rather difficult to ascertain exact values of E_c from the measured deflexions.

Carefully-measured deflexions, however, enabled the appearance of the first cracks to be discovered; in that respect Table II and the final conclusion of the Paper were instructive. The centre deflexions in the load-interval between the gauge-pressure of 390 and 640 lb. per square inch had been measured as 0·2 inch and 0·42 inch respectively, whereas the measured deflexion at a gauge-pressure of 890 lb. per square inch increased to 0·80 inch, or about twice as much as at the preceding interval. That indicated that cracking must have occurred during the second interval. The record of the tests stated, indeed, that the first cracks became visible at a gauge-pressure of 890 lb. per square inch.

Accordingly, in the final conclusions, the ratio deflexion : span increased more rapidly during the phase when cracking occurred, and, naturally, more rapidly still in the phase near the breaking load.

The Author, in reply, agreed with Mr. Peirson Frank that heat might be utilized for pre-stressing. Centuries ago a very moderate pre-stressing of vaults was obtained through the cooling of tie-rods heated previous to fixing. Lately, heat had been used for pre-stressing pressure-pipes by winding up heated hooping; but it was necessary to use special alloy steel, so as not to affect too much the quality of the steel as regards modulus, elongation, etc. The method was not applicable for pre-stressing with a final stress of, say, 50 tons per square inch, as with hard-drawn wire, for the following reason: in order to obtain such a final stress, an elongation of approximately $\frac{1}{190}$ to $\frac{1}{200}$ was necessary, to allow for deformation, shrinkage, and creep. That would require heating to about 900° F., a temperature which would affect considerably the quality of hard-drawn steel.

Therefore, irrespective of the difficulties to be overcome for a suitable anchoring, heat pre-stressing was limited to low stresses, with consequent greater relative loss in the final stress. Moreover, it did not lend itself to accurate control, whereas the use of jacks enabled elongation and stress to be controlled at each stage, and, as described in the Paper, a safe combination of stretching and anchoring to be achieved.

The question of the loss of pre-stress in course of time was certainly very important. The age of the 62-foot beam at test was 22–30 days. The average test-cube strength at 28 days was 5,600 lb. per square inch.

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The above-mentioned age did not allow of direct comparison with ordinary reinforced-concrete beams, as the Freyssinet process of vibrating, compressing, and heating resulted in a compressive strength for the concrete of about 3,000 lb. per square inch, 6 hours, at most, after completion of each sectional element of about 6 feet length.

On the other hand, the 28-foot Southall test-beam was concreted in the ordinary way, without compression or heating; it was 1 month old when tested.

For the 62-foot beam the losses of elastic deformation, initial shrinkage (for 30 days), and first creep under load could be estimated from the data given in the Paper (62-foot beam).

For the normal load $P = 3{,}531$ lb.; $M = 179{\cdot}5$ ton-feet.

The theoretical stresses for an assumed decrease from $f_{sp} = 78{,}000$ lb. per square inch, to $f_{so} = 63{,}500$ lb. per square inch were (see p. 101, *ante*):

$$f_{cs} = +1{,}645 \text{ lb. per square inch}; \; f_{ci} = +221 \text{ lb. per square inch.}$$

According to the measured strains, they would be:

$$f_{cs} = +1{,}500 \text{ lb. per square inch}; \; f_{ci} = +420 \text{ lb. per square inch.}$$

The formula on p. 97, *ante,* was based upon the assumption of linear distribution of stresses, which was not in strict accordance with facts: consequently it was not far from the truth to state that there was practically no loss for f_{cs}. The figure for f_{ci} corresponded to $f_{so} = 69{,}000$ lb. per square inch. Consequently, the first loss for normal load was of the order of 9,000 lb. per square inch.

As regards the inclined principal stresses, a similar comparison indicated that the stress had fallen to $f_{so} = 71{,}000$ lb. per square inch in direction 15 and to $f_{so} = 61{,}500$ in direction 16 (the result in the latter case being rather doubtful).

The results after 32 days under the maximum test-load (for measurement) were stated on p. 107, *ante.* Their indications were less favourable than the reality, the modulus of elasticity in tension being unknown, and important tensile stresses being already developed under that maximum test-load.

After all tests had been carried out up to the maximum load during 32 days, the beam was maintained under two-thirds of the normal load, that was, under 577 lb. per foot-run, during 18 months. That load was selected because, under it, the compressive stresses in the upper and the lower flanges were approximately equal. Two exactly similar beams had been constructed and tested in parallel during the initial period; one beam was then tested until failure in a Research Institute, whilst the other was utilized for further observations during 18 months, including recording of the variations of atmospheric temperature, and control of the level of the supports. Thus that long-time creep test was carried out under the most exacting conditions.

The first interesting point was that, very soon, the influence of the variations of atmospheric temperature obliterated that of the increment of shrinkage and creep. The second point was that the variation of the

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modulus due to increase in shrinkage and creep at the end of the 18 months corresponded to a loss of 15,900 lb. per square inch, being only 20·4 per cent. of the initial stress, whereas allowance had been made for a loss of 21,000 lb. per square inch, or 27 per cent.

It was tolerably certain that shrinkage and creep after 20 months would hardly increase the 20·4-per-cent. ratio.

Further convincing information concerning those presumable losses was supplied by tests of the high-pressure pipe referred to in M. Freyssinet's lecture of March, 1936[1]. That test-pipe had an internal diameter of 17 inches and a thickness of $1\frac{3}{8}$ inch. It withstood an internal water-pressure of 1,300 lb. per square inch before fissure-leakage started. In order to obtain that remarkable result, the concrete had been pre-compressed by hard-steel hooping embedded in the concrete up to:

$$\text{compression} = \text{tension} = 1{,}300 \text{ lb. per square inch} \times \tfrac{17}{2} \text{ inch.}$$
$$= \text{approximately } 11{,}000 \text{ lb. per inch-run.}$$

Consequently the compressive stress of the concrete was:

$$f_o = \frac{11{,}000}{1{\cdot}376} = 8{,}000 \text{ lb. per square inch.}$$

It might be assumed that concrete of that high quality would withstand a tensile stress of about 500 lb. per square inch before leakage, so that the minimum pre-compression must have been 7,500 lb. per square inch.

One of the pipes was retained for testing at the age of 12 months, when it was found that the first leakage occurred when the water-pressure attained 1,070 lb. per square inch, which meant a loss of slightly less than 18 per cent. That was substantially of the same order of magnitude as the percentage of loss for the 62-foot beam; and it would be conceded readily that the creep under a pressure of about 7,500 lb. per square inch, maintained during a full year, was a maximum occurring only under severe test conditions, and was not likely to occur in actual practice.

The creep was a function not only of load and time, but also, and to an even greater extent, of the capillary constitution of the concrete[2]. That explained the importance of the processes elaborated by M. Freyssinet for producing very high-grade concrete able to withstand, without injury, compression much higher than 7,500 lb. per square inch.

Two identical beams were made at Southall at the same time, one of which was tested in May 1940, when 1 month old, whilst the other was kept in an open yard, unprotected against weather, to be tested when 12 or 13 months old. The Author hoped to describe the results of those further tests in due course.

In order to have an adequate margin against all losses, it was advisable to use very high-grade tensile steel, that was, hard-drawn wire. The

[1] *Loc. cit.* See also T. J. Gueritte, "A New Technique of Concrete Construction", Trans. Liverpool Engineering Society, vol. 58, 1937, p. 125.

[2] E. Freyssinet, "*Une Revolution dans les techniques du beton.*" *Librairie de 'Enseignement Technique.* Paris, 1936.

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formulas for losses given by Dr. Mautner in the Appendix to the Paper of July 1940, allowed generally for shrinkage and creep $\frac{0{\cdot}75}{1{,}000}$ and, for all losses together, from 25 to 30 per cent. of the preliminary stress; and, according to experience acquired during the past few years, that was decidedly on the safe side. It meant a decrease from the preliminary stress, f_{sp}, of 70 tons per square inch to about 50 tons per square inch. With that kind of steel (B.S.S. 300) it would be possible, in specially unfavourable cases, to design for a margin of 40 per cent., thus allowing for shrinkage plus creep $\frac{1{\cdot}26}{1{,}000}$, or about five times the normal shrinkage after 28 days. That statement should allay the fears expressed by Dr. Faber. It had been made clear in the Paper that whereas, for the 62-foot beams constructed in 1936, the initial stress was only 78,000 lb. per square inch, the present practice was to pre-stress to about 154,000 lb. per square inch. Details of that beam were given so that the developments of pre-stressing could be followed from the early stages, and data registered; but the question had to be considered under its present aspects.

The Author doubted whether the factor of safety of the 62-foot beam, namely, 2·3 at failure, was so low as Dr. Faber thought. It should be remembered that the beam was an exact model of roof trusses of 183 feet span. A roof designed for the maximum load which it would ever be called upon to withstand might reasonably have a rather lower factor of safety than, for instance, a bridge; the 28-foot beam, which was an actual bridge-beam, failed at 2·92 times the calculated bending moment, which corresponded to 3·35 times the moment due to the superimposed load; similarly, the beam of 16 feet 3 inches effective span described in the Paper of July 1940, which was an actual beam for heavy storage, failed at 2·95 times the calculated bending moment.

The really important point appeared to be the intentional or accidental increase of the superimposed load. Considering a reinforced-concrete girder for which the relation between the total bending moment, M_T, the moment due to the dead-weight, M_{DW}, and that due to the superimposed load, M_{SL}, was:

$$M_T = 0{\cdot}4M_{DW} + 0{\cdot}6M_{SL};$$

then, for a factor of safety of 2·5,

$$2{\cdot}5M_T = 1{\cdot}0M_{DW} + 1{\cdot}5M_{SL}.$$

With a pre-stressed girder, which had much smaller dead-weight, the relation might be

$$M_T = 0{\cdot}25M_{DW} + 0{\cdot}75M_{SL},$$

and for the same factor of safety, namely, 2·5,

$$2{\cdot}5M = 0{\cdot}625M_{DW} + 1{\cdot}875M_{SL}.$$

Therefore, the margin for the increase of superimposed load, for the same factor of safety, would be 25 per cent. higher.

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The 62-foot beam carried, before failure, 13·5 times its deadweight. An ordinary reinforced-concrete beam for normal load carried usually 2·65 times its deadweight, which was about double that of the equivalent pre-stressed beam. It would carry, with the same factor of safety, only $2{\cdot}3 \times 2{\cdot}65 = 6{\cdot}1$ times its deadweight.

Safety against cracks was always greater for pre-stressed concrete, as all tensile stresses for normal load were eliminated, whilst those for higher loads were reduced considerably. In an ordinary reinforced-concrete beam with 2 per cent. of reinforcement, the tensile stress under normal load was 600 lb. per square inch, or more, whereas it was zero in a pre-stressed concrete beam. For instance, with the 28-foot beam, with a factor of safety 2·92 times the calculated bending moment, the well-defined cracks which had formed up to twice the calculated moment disappeared entirely when the load was removed, whilst under a test load equal to 2·85 times the bending moment, or almost under failure conditions, the non-elastic deflexion was only 24 per cent. of the total deflexion.

A similar state of affairs was observed for the 16-foot-3-inch span beam (July 1940 Paper), which failed at 2·95 times the bending moment: up to the loading corresponding to 2·7 times the moment, all cracks disappeared when the load was removed.

In an ordinary reinforced-concrete beam, so near failure, the wide cracks did not close on removal of the load.

Dr. Faber, in his comment that at the start the stress in the concrete was 2,500 lb. per square inch and, consequently, exceeded that allowed by the Code of Practice for reinforced concrete, appeared to have failed to consider two points. The first was that pre-stressed concrete was, in essence, quite alien to reinforced concrete, and the Code of Practice was not applicable. The second was that, in any case, the Code dealt with stresses under normal load, and not with stresses which might develop during construction. The 2,500 lb. per square inch, or other high stresses developed under the initial pre-stressing, occurred during construction, and were reduced considerably when the member was subjected to loading. For instance, in the 62-foot beam, when the jacks were released, when the dead weight came into play, and the elastic deformation took place, the 2,500-lb.-per-square-inch initial compressive stress fell at once to something like 2,000 lb. per square inch.

Moreover, that initial stage (without any superimposed load, shrinkage, or creep) constituted a complete test for each beam, as the compressive stress prevailing at that time would never recur under the action of the load. That ensured absolute safety.

The initial stress in that 62-foot beam could be exceptionally high, because it was a roof truss. Consequently the compressive stress set up in the lower flange by the pre-stressing would decrease very rapidly under the action of the permanent superimposed load of the roof-slabs.

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The stress under normal load was reduced to about 1,500 lb. per square inch, which was quite normal for vibrated and compressed concrete of that kind (1 : 1½ : 2½ mix).

Naturally the stress in the steel increased when the bending moment due to the load increased. But as there was considerable decrease of initial pre-stress due to shrinkage and creep, the subsequent increase due to load brought it up less than half-way toward the initial value in most cases. Thus, for the 28-foot beam the original pre-stress was 68·7 tons per square inch (see p. 108, *ante*), the calculated final stress, f_{so}, without load was 51·8 tons per square inch, and the stress under normal load at midspan was 56·3 tons per square inch.

For hard-drawn wire, the 0·1-per-cent. proof-stress (as no real yield-point existed for such steel) lay between 85 and 90 per cent. of the breaking stress, say, 85 tons per square inch, as mentioned on p. 108, *ante*; but in ascertaining the factor of safety, it had to be borne in mind that, for each increment of 100 per cent. of load, so long as the section was homogenous, the tests showed that there was an increase of only 4·5 tons per square inch for the steel stress. After fissuration due to considerable increase in loading, the lever arm for the steel increased slowly.

The position as regards the factor of safety of the steel was, therefore, quite different from that obtaining in a steel structure, wherein the steel stress increased almost proportionately with the total load; it differed also from that existing in a reinforced-concrete beam, wherein the increase of steel stress was far more rapid in the sections where the tensile zone was destroyed.

That was one reason (among many others) why the recent proposals, mentioned by Mr. Reynolds, to add unstretched steel and depart from the homogeneity of the section, would merely interfere with the advantages of the Freyssinet process, as had been demonstrated by Dr. Mautner[1].

The remarkable elasticity of the tested beams up to the immediate approach to failure indicated plainly that the steel was not overstressed. This fact is evidenced also by *Fig. 19*. Immediately after rupture, both halves of the beam at once cambered upwards, thus showing that pre-compression was still acting.

In considering questions of stresses, it might not be out of place to refer to the fact that, on the Continent, the admissible compressive stress of concrete was one-third of the strength of test-cubes at 28 days. The reasons for that were set out clearly by Dr. Mautner in 1936[2]. The reason for admitting a higher compressive stress for pre-stressed beams than for reinforced-concrete beams was that the narrower limitation for reinforced concrete did not arise from the desire to secure for the concrete a factor

[1] "Saving Reinforcement in Prestressing." *Concrete Constr. Engng.*, vol. 36 (1941), p. 73 (Feb. 1941).

[2] "The Elimination of Tension in Concrete, and the Use of High Tensile Steel by the Freyssinet Method." Final Report of the 2nd Congress of the International Association for Bridge and Structural Engineering. Berlin, 1939.

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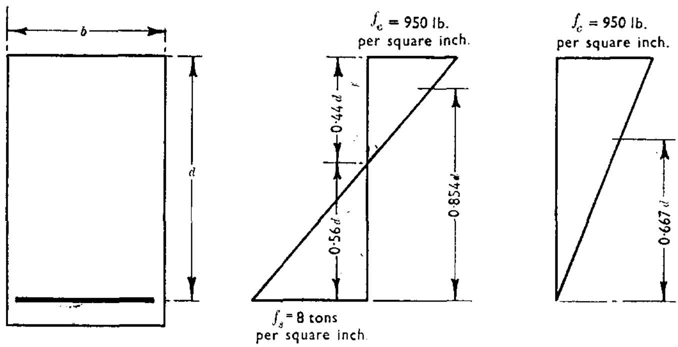
of safety more than twice as high as that for the steel, but to limit indirectly the stresses in the tension zone, and to avoid massing too much steel there. That reason disappeared for the homogeneous section of a pre-stressed member, where tensile stresses were excluded for both bending and shear.

Although such higher permissible stress had been intentionally allowed on the Continent, applications in Great Britain were designed for a maximum compressive stress of 1,250 lb. per square inch, to which no objection should be taken for a 1 : $1\frac{1}{2}$: $2\frac{1}{2}$ mix.

The economy realized by pre-stressed concrete was not due, as suggested by Dr. Faber, to the adoption of higher stresses for concrete, namely, 2,500 lb. per square inch. As had been previously explained, the concrete stress under normal load for the 62-foot beam was not 2,500 lb. per square inch, but only 1,500 lb. per square inch, as measured in the tests. The economy was due to the fact that the materials were utilized better, as was shown by the simplified example illustrated in

Figs. 29.



Figs. 29. Comparing two beams of the same scantling, $b \times d$, with equal f_c (950 lb. per square inch), one in reinforced concrete, and the other in pre-stressed concrete: for the reinforced-concrete beam, the bending moment which could be withstood was :

$$M_1 = \frac{950 \times 0{\cdot}445d}{2} \times 0{\cdot}854d = 181d^2,$$

whilst for the pre-stressed concrete beam it was :

$$M_2 = \frac{950 \times d}{2} \times 0{\cdot}667d = 317d^2$$

Consequently, with equal compressive stress the bending moment which the pre-stressed beam might withstand was 75 per cent. greater.

Naturally, in practice, the saving in steel was not all profit. Experience indicated that the cost of stretched and fixed hard-drawn steel was about 2·5 times that of mild steel, but the section of steel required was only one-sixth of that for normal steel. That left a margin in favour of pre-stressed design, to which had to be added that resulting from the saving

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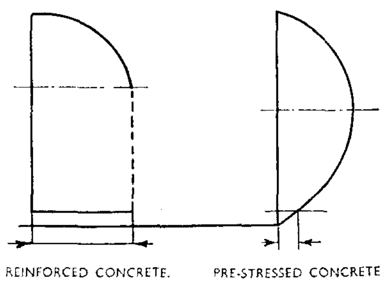
in concrete; the latter was important, and led to a further saving from the point of view of the supports and of the foundations. Recent applications had proved decidedly the economy of pre-stressed concrete in suitable cases. Obviously, there was no need to resort to it in many cases.

For a member under compression by wires invariably fixed at the ends of the member and highly stretched, no danger of buckling existed: the slightest deviation from the axis would cause a corresponding deviation of the wires or cables, which would give rise immediately to a stabilizing counteraction on the part of the stretched wires.

For the top flange, the provision of the slab and transverse members of the complete structure guarded against danger in that respect, and for the tests guiding stiffeners were provided.

It was true that the steel reinforcement was set out in the lower flange much more closely than was usual in reinforced-concrete construction. That was permissible for pre-stressed concrete owing to the intense vibration, compression, and heating resorted to for the construction of

Figs. 30.



the 62-foot beam, as described in the Paper of July 1940. No voids were discovered during the thorough examination following the tests.

It should be remembered that the use of hard-drawn wire, as at present used, stretched to about 154,000 lb. per square inch, instead of 78,000 lb. per square inch, as for the 62-foot beam of 1936, increased appreciably the space between the wires. As regards the applications in situ, with cables, the question did not arise at all. In considering that point, it should also be remembered that for prestressed concrete the question of adhesion between concrete and steel, as it presented itself for reinforced concrete, did not exist, because, for the homogeneous section there was only an almost negligible shear force acting on the steel, as illustrated in *Figs. 30.*

No special tests had been made of the 64-foot beam composed of thirteen precast elements, mentioned by Mr. Chettoe, as the principle of a beam made of small elements had already been tested on the beam described in the March 1938 Paper, which was built in separate consecutive elements of about 5 feet each—the length of the mould. Each section hardened completely in a few hours (by the vibration–compression–heating process devised by M. Freyssinet) before the next element was cast. The

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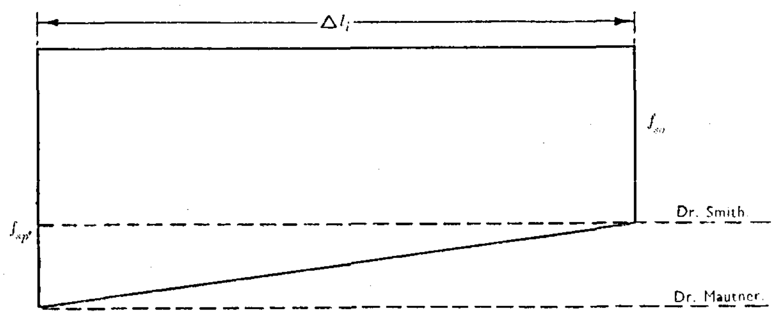
same method was used for the construction of the bridge girders of 108 feet span described in the July 1940 Paper, and gave full satisfaction.

The pre-stressing of the shorter bars of the 62-foot beam was described in the July 1940 Paper (pp. 15–16).

With regard to the angle of the anchor blocks and of curved cables, it was true that the angular change of direction of the wires should be taken into account for assessing the degree of pre-stressing; that factor had to be considered also in estimating the anchoring force. Experienced designers knew that the angle had to be much sharper than had been deemed sufficient and that the decrease of the cable force had to be calculated. Very valuable information, in that respect, had been obtained in connexion with the construction of a 32,000-ton press made of pre-stressed concrete in France; the question was there of paramount importance.

About 25 years ago the Author had been inclined to share Mr. Andrews' views in regard to the advantage offered by high-tensile steel for rein-

Fig. 31.



forced concrete; but whilst excessive cracking might not be dangerous inside ordinary buildings, it became dangerous in factories, reservoirs, bunkers, river and sea work, etc.; and eventually the Author came to the conclusion that high-tensile steel was not advisable, without pre-stressing. Comparative tests of beams with non-stretched high-tensile steel had been carried out frequently, and reference to the voluminous report of the International Association for Bridge and Structural Engineering would appear to settle the question.

The formula for the tension in the steel after elastic deformation, f_{so}, given by Dr. Smith, was quite correct on the assumption that the total deformation Δl_i of the lower fibre corresponded to the final stress of the steel (*Fig. 31*).

Dr. Mautner's formula calculated the total deformation of the lower fibre under the preliminary stress, the stress falling gradually from $f_{sp'}$ to f_{so} during the shortening. The difference of the results, being a sum

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of differences of the second order, was very small. For instance, for the 28-foot beam, Dr. Smith's formula gave $F_{so} = 0{\cdot}872 f_{sp'}$, whereas Dr. Mautner's gave $f_{so} = 0{\cdot}853 f_{sp'}$. The difference was only 1·9 per cent., Dr. Mautner's result being rather on the safe side.

From the numerous tests of welds made in connexion with the construction of the 62-foot test-beams, it appeared that the welds were very sound. That point, however, was now devoid of importance, because the use of hard-drawn wire of great length, as employed for pre-stressed work, rendered welding unnecessary.

Unreasonable prejudice retarded progress. A good deal had been directed against reinforced concrete about 45 years ago, but was eventually overcome. Doubtless similar prejudice would apply to pre-stressed concrete owing to its use of hard-drawn wire: but, with regard to bond, far from being disadvantageous, high-tensile steel was a boon because, owing to the small diameter of the wires, and their swelling during the initial shrinkage and creep, the adhesion was so great that special anchoring-hooks were unnecessary, as had been explained in the July 1940 Paper.

Mr. Cuerel was correct in saying that pre-stressed concrete appeared to be particularly adapted to the construction of structures of great span, such as bridges. In fact, owing to its great lightness, as exemplified in a comparison of the section of the 62-foot beam, with that of a steel beam, it permitted, for instance, the construction of beams of far longer span than was practicable with reinforced concrete.

The method advocated by Mr. Cuerel for utilizing medium-high-tensile, instead of very-high-tensile steel, and stretching in two stages, the second stage taking place after de-centering, appeared to be similar to that already tried on the Continent. In the case of a bridge of about 200 feet span, the medium-hard-steel bars were to be re-pulled after some creep and shrinkage had taken place. An important plant of jacks, which were to remain permanently in the structure, were provided: but it happened that the internal friction had become so great that the re-pulling could not be effected properly, and the body of the bridge could not be raised; the serious sagging which had taken place could not be reduced, and that caused considerable trouble and expense. It is not correct to assume that creep would be greatly diminished when the dead load came into play, because the concrete in the pre-stressed zone in the middle section was not heavily compressed. It made very little difference to the further creep whether it was due to compressive stress of the concrete of the top or of the bottom part.

Dr. Probst's comments on the question of cracking were interesting. The early attempts at pre-stressing referred to by him were probably those made by Lund and Koenen, and others, which failed, as previously explained, because the low tensile stresses adopted disappeared entirely in course of time. The suggestion that research-tests should be made on full-size specimens conformed with M. Freyssinet's own views, within practical possibilities.

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