The Paper, which was received on 17 August, 1955, is accompanied by eight photographs and seventeen sheets of diagrams and drawings, from which the half-tone page plates, the Figures in the text, and the folding Plates 1 and 2 have been prepared, and by two Appendices.

Discussion

The Authors introduced the Paper with the aid of a series of lantern slides.

Professor A. L. L. Baker (Professor of Concrete Technology, Imperial College of Science and Technology) referred to three significant points in the Paper. First, the specification of the concrete by minimum cube strength at 6,000 lb/sq. in. and the achievement of the contractor of a variation of cube strengths from 7,000–10,000 lb/sq. in., indicated the quality of work possible. That demonstrated the importance of following the recommendations of the Institution's report on "Quality Control" by specifying minimum strength. He hoped that the New Code of Practice would encourage similar standards of quality control in the case of ordinary reinforced concrete, where the higher unit cost of a high-strength mix could be justified.

In Table 2 it was indicated that the cost of a frame for a 51-ft-span workshop building was the same for prestressed concrete as for structural steel. The cost of maintenance painting was not mentioned. Presumably, therefore, it could be concluded that the prestressed concrete design was the most economical. That information was significant.

Professor Baker then showed some slides of recent tests carried out at Imperial College by one of his students, Mr H. C. Visvesvaraya, who was to speak later.

Fig. 21 showed a special jack which had been designed to facilitate the joining together, in full continuity, of precast framework members. The jack was, so far as he knew, the smallest and lightest yet made for its capacity; it could easily be fitted into a small pocket so that two independent precast beams could be joined by a prestressed tie or tendon which followed a fairly flat arc coming close to the top of the beam at the joint and then down again to the anchorage at the jack. That enabled the precast units to be joined together and prestressed continuously in a very simple manner.

It was intended to continue the research and to try to achieve the same advantages of full effective depth around the corners in the end joints of a building frame and a portal frame. There was full effective depth at the support and at mid span in the case shown in Fig. 22. At less than 80% working load there had been no cracks in the test and there had been good agreement with the calculations. Under working load, assumed to be half the ultimate load, there had been a few very fine cracks, the kind of cracks which in normal reinforced concrete construction were not deemed to be serious.

It therefore seemed that there was scope for a type of construction in which the main emphasis was on the ultimate strength, with the full sections of the beam used at the support and at mid-span in order to carry the maximum ultimate load. Such a design, however, might have the slight disadvantage of being slightly highly stressed in regard to tension in the concrete under working load. In many cases that did not seem a serious matter, particularly if cracking would not lead to any serious results if it occurred and particularly if such beams were indoors and under cover.

Before failure occurred in the beam in Fig. 22 a plastic hinge had begun to develop at the support and the bending moment at the support had been practically equal to the bending moment at mid-span. He hoped that later they would be able to report the results of similar tests going around the corner of a portal frame with full effective depth being used. There might be an objection to the possibility of very fine hair cracks occurring in prestressed concrete construction but such cracks had been tolerated for years in ordinary reinforced concrete and it seemed that under certain circumstances there was justification for continuing that practice, particularly since the ultimate strength could be considerably increased and the method of construction facilitated.

Mr Donovan H. Lee (Consulting Engineer) said he agreed with all the principa opinions expressed by the Authors. He suggested that there was not yet a method of framing and prestressing of prestressed buildings which was generally agreed as satisfactory. The difficulties arising from movements in long structures continuously stressed had been rightly emphasized by the Authors, but he agreed there were now a number of successfully prestressed buildings where shortening due to stressing had been avoided. Some designers, he thought, were inclined to forget that to keep down the cost of stressing, the number of stressing operations should be kept to the minimum. Contractors' rates for stressing seen as rates in Bills of Quantities, varied over a wide range and indicated that there was still uncertainty in contractors' experience, or lack of it, as to the cost of stressing.

Mr A. Goldstein (Partner, Messrs Travers Morgan and Partners, Consulting Engineers, London), asked for the Authors' comments on some problems generic to the question of continuous prestressed portals. Referring specifically to a 110-ft-span two-pin portal, he said that if the portal were suitably shaped then, in addition to easing the cable curvature, the moments returned to the corners, which meant that the critical position was frequently at the corners. If the cables were taken right through instead of stopping along the length of the beam, there was some eccentricity to spare at the middle, which meant that the transformation which the Authors had mentioned could be obtained very comfortably.

Further research was still required on the question of transformation. Suppose, for instance, when designing the cables for a portal, it was found that the anchorages at the corners came outside or near the top of the section and transformation was carried out. Then, assuming point anchorages, there must be a certain amount of stress diffusion from the anchorage. What happened in that corner if the stress diffusion left some areas of the corner not stressed to the required degree? Little data on that point was available and the Authors' opinion would be useful.

The Authors had stated that in one of their designs the ultimate load had been reduced by transformation. But some tests carried out by the Cement and Concrete Association, showing transformation of continuous beam cables, did not indicate an effect on the ultimate load. Some tests had also been carried out by the same organization on ultimate load of portals and it would be useful to have those results and the Authors' comments on them. If the tests were representative it seemed that transformation would not affect ultimate load.

Finally, he agreed with the Authors entirely about through cables for small sections—putting a uniform prestress in the section. Working on a design stress of 2,000 lb/sq. in., the section was stressed to a uniform stress of 1,000 lb/sq. in. The moments at mid-span would produce a negative stress of 1,000 lb/sq. in., and the moments over the support would occur in the same way, and the section would be only half used.

What was the Authors' opinion of the idea of forming hinges over the supports and putting the cable right through, thereby getting not continuity but something which might be more economical The cables could be placed at the bottom of the core or middle third without causing tension or, if certain tension were accepted at the ends, they could be placed even lower to allow for permanent finishes. The effective moment which could be used if that were done was that due to a stress of 2,000 lb/sq. in. in the example he had chosen, for by placing the cable at the bottom of the core a positive stress of 2,000 lb/sq. in. could be applied at mid-span. There were then the two conditions: in the first case, a prestress of 1,000 lb/sq. in. with bending moment $WL/12$; and in the second case a prestress of 2,000 lb/sq. in. with bending moment $WL/8$ at mid span.

The second method seemed to give certain economies but it had its snags; the rigidity of the continuous beams was lost. Nevertheless, the method had possibilities and Mr Goldstein knew that on one occasion its use had proved on investigation to be more economical than uniformly prestressed multi-span continuous beams.

Mr F. J. Samuely (Consulting Engineer) said that the fact that rigid frames were possible should not be an inducement to use rigid prestressing in all circumstances. For multi-storey buildings, for instance, whenever possible it was better to have a pinned deck

—prestressed concrete which was not rigidly fixed, taking the wind thrust by special arrangements. He had recently undertaken an investigation concerning the wind support for a large one-storey building. It had been found to be cheaper to have a roof deck with columns which were simply leaning against the solid roof. The roof was held up by the gable walls. That was cheaper than a rigid frame.

There were a number of instances where rigid frames had to be used, however, and it should not be forgotten that the designer could alter the bending-moment diagram considerably to suit himself. The Author had shown examples taking the dead load on a beam on two supports and designing the corners accordingly.

One important factor in prestressed concrete was that the cost depended on the difference between maximum and minimum bending moments; it was practically the same whether it was $\pm$ 3,000 lb/sq. in. or plus 6,000 lb/sq. in. and zero. Therefore, by superimposing bending moment on to a corner, without making the corner more expensive, other bending moments could be reduced. In a rigid frame of conventional shape a hinge would not normally be introduced, but Mr Samuely was convinced that if it were prestressed it was worthwhile introducing that hinge frequently, to make certain of a negative moment at the corner. The cost of the construction could be reduced by reducing the cost of the top.

It should not be forgotten that it was always possible to introduce a bending moment artificially by prestressing. Assuming the presence of a prestressing bar, it was possible to have any amount of positive bending moment superimposed on the whole frame or, alternatively, looking at the picture from the other side, it was possible to superimpose any amount of negative moment. Once it was decided to prestress it was possible to alter the bending-moment diagram at will.

In a Paper[6] to the Institution of Structural Engineers a few years ago, Hendry had referred to rigid corners in steel and pointed out that a sharp corner gave very much heavier stresses in compression than those calculated. That could be shown theoretically. Mr Goldstein had been worried about the outer corners of a rigid frame, but it seemed that the same sort of problem arose at the inner corners. If the section were an I-section, the problem could be overcome to a certain extent by completely filling in at the corner to give greater resistance. It was a problem which ought to be investigated and it would be useful if Professor Baker could give some time to it.

Mr J. A. Derrington (Engineer, Sir Robert McAlpine & Sons Ltd) said the Authors had stated that they had found it unwise to draw conclusions about the relative merits of normally reinforced and prestressed concrete, but, in the Paper, he thought, they should have brought out the influence which the relation between the live load and the dead load of a structure had on the choice of the medium of construction. That was made clear, particularly on p. 71 by reference to the 40-ft-span frame, which was cheaper than the normal concrete alternative, even though additional columns might have been introduced into the latter. From his own experience he could recall a three-storey building of roughly the same character, in which the difference in cost between prestressed concrete and normal reinforced concrete was very small, but undoubtedly the introduction of two additional framing columns to the normal reinforced concrete alternative would have reduced that cost by about 30%. The only reason was that whereas one of the structures was a light roof structure, the other was an office building with much heavier live loads.

One could only agree with the Authors' remarks that prestressed columns generally did not pay for themselves, even though by using the magic word "prestressing" the allowable stresses shot up from 1,140 to 2,000 lb/sq. in. for concrete which was common to both structures.

He was intrigued by the novel approach to the design of portals whereby one made a

[6] A. W. Hendry, "An investigation of the stress distribution in steel portal frame knees." *Structural Engineer*, vol. 25, p. 101 (Mar.–Apr. 1947).

full analysis, taking into account the moment of inertia of the sections, and then, by a process of linear transformation, decided what the moment would be at the knee. Considering the total moment as for a simply supported beam and deciding what resistance could be provided at the joint by prestressing forces or by mild-steel or high-tensile-steel reinforcement, would give the same answer a little quicker.

Mr Derrington then showed a number of slides illustrating a building in which prestressing and normal reinforced concrete had been combined economically to give an attractive final appearance.

The Authors had shown most of their columns as hinged and it would be interesting to know the reason. The hinge cost money to produce and it raised complications in erection and construction. It was also rather difficult to guarantee in practice.

With reference to the continuous prestressing of a framed building, the stress which was applied when spread over the entire floor and beam construction generally worked out fairly low and the resultant movement was much less than that expected from temperature expansion and contraction.

Mr Derrington found prestressing had to justify with multi-storey office blocks in normal circumstances. He had worked out some figures and had found that the cost of providing 100,000 lb. prestress force for a 40-ft span was between 9*s* and 12*s* per foot run. That force could be resisted by high-tensile steel without prestressing for between 7*s* and 9*s* per foot run, depending on the type of high-tensile steel used.

An attractive way of blending the two materials was to use the long-line fully bonded prestressed unit with cast-in-situ slab, and the cost of providing that force then dropped to between 4*s* 6*d* and 5*s* per foot run. Continuity could be provided if necessary or desirable by the use of mild-steel or high-tensile-steel bars and the cost was much less than when introducing short prestressing cables, which raised those figures to between 16*s* and 20*s* per foot run.

Mr E. W. H. Gifford (Consulting Engineer) referred to Guyon's statement of 2 or 3 years ago that prestressed concrete would not become a complete framework material until it could achieve continuity. The way in which that had been done in Great Britain had been due in no small part to the work of Mr Walley and his colleagues.

The assumption of composite action between an in-situ floor and a simple beam presented no particular problem but obviously, with continuity, that floor coming into tension at reversal of moment changed the value of I for the section at that point. The exact point where the change occurred was rather difficult to establish. Although it was not necessary to split hairs over it, it would be interesting to know whether, in the tests which had been carried out, the Authors had discovered whether that point was significant or not.

It had been pleasant to see from the Paper that two-stage prestressing had been used to a larger extent. In the light of later experience would the Authors have considered that in the Kilburn project? The use of two stages seemed very attractive for two reasons. First, in the average prestressed frame, because of the virtues of prestressing of being able to reduce the effective dead-load moment going round the corner, they were not concerned to the same extent with reducing the mid-span moment by having very stiff columns. If stressing were done in two stages, the first cable, which had no effect on the corner, took up a large part of the shortening of the beam. The second system of cables was available for putting in the heavy live-load moment in the span and taking care of the live-load moment going round the corner of the frame. The dead load had been virtually stressed out by the primary cable.

The Authors had mentioned the difficulty of longitudinal stressing particularly in long buildings and they had mentioned a certain figure of elastic shortening in long buildings. If the edge beams of a building were to be precast and some form of joint made between the beams and the columns they had to take into account not only the elastic shortening of the beam but also the effect of shrinkage of the mortar joint. Shrinkage could be offset by using very dry caulking, but 3 years ago he had had an interesting experience with ten 33-ft crane beams where a 330-ft cable had been used as a secondary, with mortar pressure

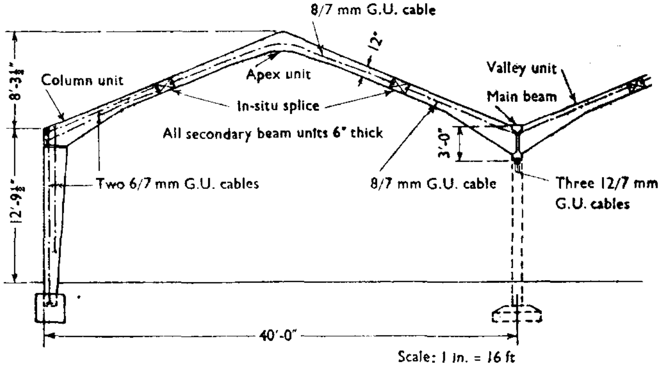
8/7 mm G.U. cable
12"
8'-3½"
Column unit
Apex unit
Valley unit
Main beam
In-situ splice
All secondary beam units 6" thick
3'-0"
12'-9½"
Two 6/7 mm G.U. cables
8/7 mm G.U. cable
Three 12/7 mm G.U. cables
40'-0"
Scale: 1 in. = 16 ft

FIG. 23

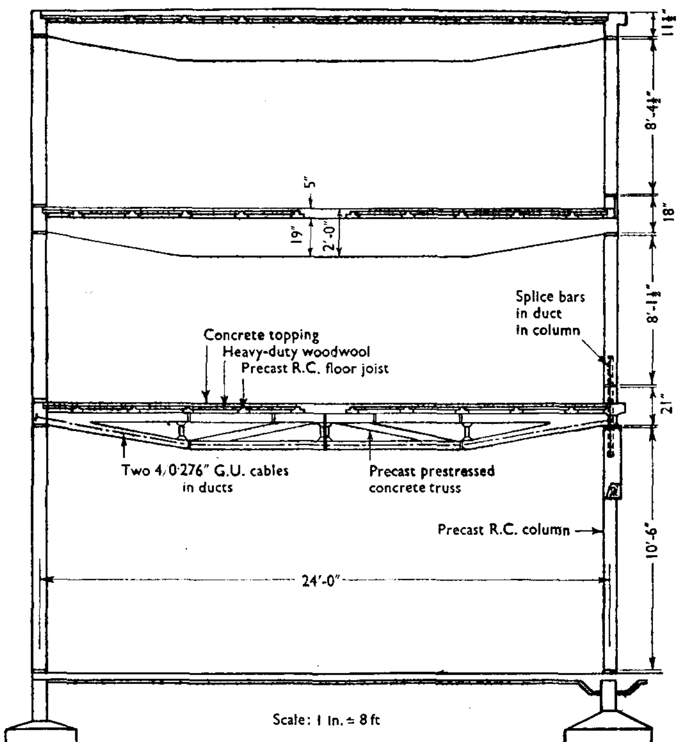
11½"
8'-4½"
5"
18"
19"
2'-0"
8'-1½"
Splice bars in duct in column
Concrete topping
Heavy-duty woodwool
Precast R.C. floor joist
21"
Two 4/0·276" G.U. cables in ducts
Precast prestressed concrete truss
Precast R.C. column
10'-6"
24'-0"
Scale: 1 in. = 8 ft

FIG. 24

pads. The whole was stressed in one operation. He had been suspicious of what would happen and had left the end columns free. There had been nearly ½-in. shortening or push-in on each end column, which was very nearly twice the elastic shortening.

Fig. 23 showed part of a five-span, five-bay multi-pitch roof. The secondary beams were at 15-ft centres supported by 60-ft-span main beams. There were two spans, each of 60 ft, for each bay. The secondary beam valley unit formed a diaphragm to the main beam itself. In some ways it was a little crude from the technical point of view but it had the advantage of economy on the site and in the drawing office.

The apex units had been prestressed in the factory, as had been the valley units and the external corner units. Reinforced concrete splices had been used at the points of minimum bending moment. The main beams had been cast in 15-ft sections with the valley unit stressed in between, using temporary false work. The main beam had been stressed with two cables as primaries and there was one secondary cable passing continuously through the building on the same system as used at Sighthill. The column had a precast cap and a mortar pressure pad for 3,500 lb/sq. in. was placed on each side. The cable transferred the prestress and at the same time performed the useful function of holding the beam on to the column.

There would be no point in carrying out single-stage prestressing on the 60-ft-span beam in that case because in such a two-span system there was no economy in making full continuity. There would be full shortening of the beams by all cables, which would be embarrassing to the outside columns.

Fig. 24 showed a frame building, although not in the sense used in the Paper. There was a prestressed truss of precast sections joined together with two tendons passing through the bottom boom. They were divided over the column so that splice bars passed through a hole in the end of the truss and down into the 6-in. × 6-in. precast reinforced concrete columns. Obviously the frame did not carry wind, which was carried in the way which Mr Samuely had described—by the floor acting as horizontal beam.

It was a school building and for its particular size and function it was a remarkably cheap job. The floor acted continuously with the truss and had been placed on it after the truss had been erected and before grouting. Only the live load was carried round the corner of the frame and because of the form of the truss the live-load moment at that point was very small.

Fig. 25 showed another small building which worked out cheaply. The main beam was a prestressed composite but there was a soffit unit similar to the type described by Mr Samuely[7] some years ago. The inverted U was stressed first with the primary wires along the bottom, and those were anchored. The work was done on a scaffold and the precast reinforced concrete column was placed against the end of the soffit system. The secondary cables followed the path indicated to the anchorage in the outer column itself. Wind was not carried on those columns. By so proportioning the beam and the column the moment passing round was extremely small and quite a small prestressing force was required in the second cable to offset it. It had been so arranged that that prestressing force was sufficient to balance the dead-load bending moment at mid-span, so that the live-load moment in the column was the only concern and the shortening effect of the relatively small force was almost negligible on that span. The roof was a normal three-pin frame.

Mr Gifford then turned to the application of temporary hinges to multi-span beams and particularly to systems of unequal span. The Authors might agree that on multi-span buildings equal spans in prestressed concrete tended to be a little uneconomic, and it seemed that in many factories and similar buildings a system of spans of the type more familiar in bridge work would be useful. He hoped shortly to have the opportunity to work out the details. He sketched a three-bay frame in which the outside spans carried

[7] F. J. Samuely, "Some Recent Experience in Composite Precast and In-Situ Concrete Construction, with Particular Reference to Prestressing." Proc. Instn Civ. Engrs, Part III, vol. 1, p. 222 (Aug. 1952).

the central span as a suspended span for dead-load condition only. The floor beams, which would be of precast construction, later to be covered with the floor slab, were placed so that the outer-span beams, with their overhang into the centre bay carried nearly 80% of the dead load of the system. Pressure pads were situated at points of negative moment. A cable could be introduced running through the pressure pads into the required positions at the back and over the top section. The cable could be stressed and the joint would be filled at the bottom. That placed the cables exactly in the position where they were needed and avoided any unfortunate results from secondary moments. It also seemed to give a simple method of erection. It was obviously designed for a job

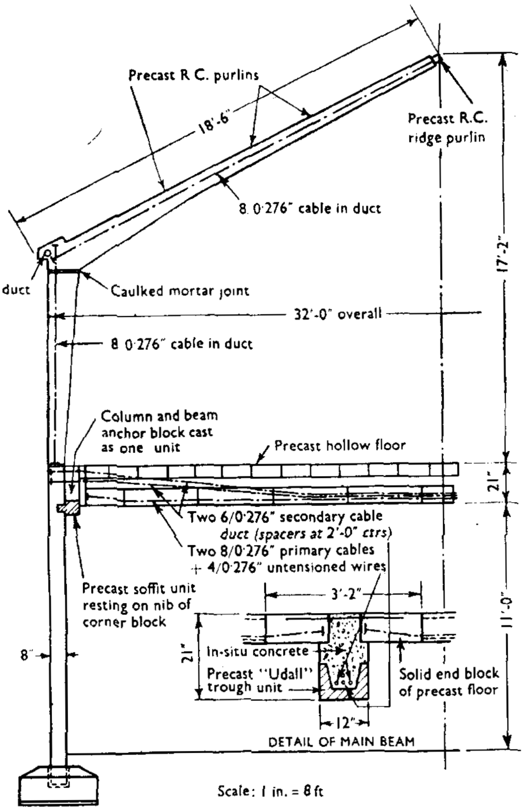

Fig. 25

where framing would be used in any case and the whole work could go through with no interruption through the secondary stressing, which could follow with the erection.

After the meeting, Mr Gifford wrote that he had examined that proposal in detail on a scheme for beams of 44–63–44-ft span at 20-ft centres. The very heavy live load of 300 lb/sq. ft produced such large reversals of moment in the end spans that continuity was obviously not economical under those conditions. But the use of the suspended span for live and dead loads appeared to be satisfactory. He expected that the factory would be built with that type of beam.

Mr J. N. Lowe (Pre-stressed Concrete Co. Ltd) said that because of the existing economic circumstances in Great Britain generally, engineers were spending a great deal of time and energy on the design of structures for buildings. It was important to remember that in building, more than in civil engineering, the design was closely associated with architecture, quantity surveying, and probably R.I.B.A. general conditions.

The work of the engineer might be associated with only 25–40% of the total cost of the work. Nevertheless it should be realized that the engineer's work could greatly affect the remaining cost and it was not always the cheapest structure that produced the cheapest total job. In particular, the effects of installation of services should be borne in mind.

First, the engineer and the architect would choose the medium in which they would work and then select the method of using that medium. The Authors had presented a useful summary for helping the architect and engineer to make that choice.

Mr Lowe's firm had followed much the same path of development as used by the Authors, starting with a frame which was basically beams on load-bearing walls and following through with work not unlike Sighthill. The next step was to continuous frames, leading to the method of supporting floors on pin-jointed columns, staying the building against horizontal deflexions by anchoring the floors (which then acted as horizontal beams) to suitable and convenient vertical cantilevers such as stairwells, lift wells, and internal or external walls. The company had used that method satisfactorily for millions of square feet. They were now, however, once again examining the usefulness of beams on load-bearing walls.

On p. 74 there was a very pointed reference to how much one part of a structure acted with another part. Tests had indicated that it was difficult to prevent interaction between all parts of a structure, and the engineer had to examine that question very carefully.

On pp. 75 and 76, the Authors had mentioned the importance of carefully planned erection procedure. Before embarking upon a complicated erection plan the engineer should satisfy himself that unforeseen delay in one item of the erection procedure did not involve accumulative delay to the whole job. The engineer should avoid the possibility of having to recalculate an erection procedure during the work to reduce such accumulative delay.

Well-planned erection procedure could possibly save materials, but it should be borne in mind that continuous design did not always do so—particularly if the design was not according to ultimate loading—for it might increase labour costs and might also tie the hands of the contractor more than the engineer or architect for the job would wish.

On p. 79 the Authors had suggested that the prestressed bending-moment diagram was always worthwhile drawing. That was a vital point. It was absolutely essential for an engineer to remember that prestressing was the application of an external force. Without that realization he could never work satisfactorily in the medium.

Turning to p. 81 Mr Lowe was not sure that the simple grid of columns and beams was the cheapest method on a cubic-foot basis—always bearing in mind that the structure was but part of the cost of a building.

On the other hand he agreed strongly with the comment on p. 83 that there were advantages in uniformly stressing a member. The engineer should never be goaded into liking the pretty technical solution in building. His answer affected the cost of the work of architects and consulting engineers.

Summarizing, Mr Lowe said that the decision to be reached was whether to frame or not to frame. Any choice made now between load-bearing walls, fixed building frames, and pinned building frames, might not be a final one, for the ratio of cost of labour to materials was changing rapidly and what was right today might be wrong tomorrow.

Mr H. C. Visvesvaraya (Civil Engineering Department, Imperial College of Science and Technology) said that Professor A. L. L. Baker, at the beginning of the discussion, had already indicated their general approach to the problem. The continuous beams tested were 30 ft 8 in. long on two spans each of 15 ft with third-point loads. The continuity was established by short internal ties at the support, anchored within the depth of the beams. On pp. 77 and 78 the Authors had indicated some of the disadvantages of the capping cable system as used in France. There were of course many more difficulties with them, all of which had been solved by the suggested new method of construction. Further, certain additional advantages had been gained. The details would be published shortly. The beams tested so far had been pretensioned prestressed units joined together, as indicated in Fig. 26a. The resultant line of pressure was indicated by the dotted line. There was an unavoidable discontinuity or sudden jump in the line of pressure at the anchorages of the internal tie but that should not cause any worry because in general those anchorage points could be located in the zone of contraflexure where the "cable zone" was widest and most suited to deal with such discontinuities. The condition of the line of pressure could be improved when curved cables were used in the span such as in post-tensioned units, cast-in-situ or precast, as indicated in Fig. 26b. It might be relevant to

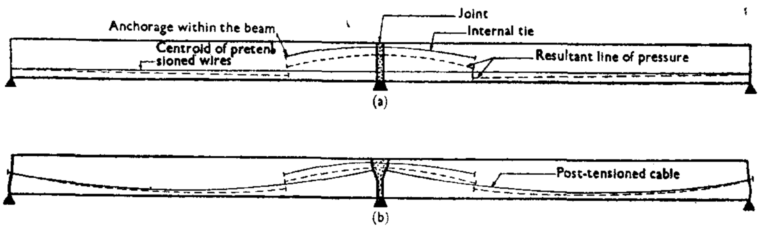

FIG. 26

point out that the invariance of ultimate load on prestressed continuous beams with linearly transformed cables shown by Guyon and Dr Morice had serious limitations and those limitations and the consequent drawbacks in any new way of approach to ultimate load design should be borne in mind.

Very little consideration seemed to have been given to joints although they had been used on many occasions. Joints should receive greater attention in prestressed concrete construction than in reinforced concrete construction because realization of high working load meant prevention of cracking at those loads, which in turn meant obtaining the maximum resistance against tension. In the case of normal reinforced concrete the quality of the concrete used was not generally high and hence the modulus of rupture of a well-made joint would not be very different from that of the adjoining concrete; whereas in the case of prestressed concrete the concrete adjoining the joints might have a modulus of rupture of the order of 600 to 1,000 lb/sq. in. or even more; all that would be wasted unless the joint itself could take a high tensile stress.

In the case of tests carried out at Imperial College, Professor Baker had mentioned the minute cracks which had occurred at about 80% of the working load (taken as half the ultimate load). If cracking was considered to be the criterion for working load the load factor would be about 2·3. The cracking occurred at a place where it could be tolerated in a normal structure, such as the buildings under discussion, because from the point of view of aesthetics it was at the support and did not matter; from the point of view of corrosion of steel there would normally be the column and hence the cracks were unlikely to form at that load.

The Authors had indicated on p. 86 in Appendix I that the load factor for the frame was 5·8. That gave an indication how uneconomical a structure might be from an ultimate-load point of view when the designs were based only on elastic theories.

He did not agree with the Authors that uniform prestressing in building frames was a solution of the problems which were being faced today.

He entirely agreed, however, with their suggestion that the prestress-moment diagrams and secondary-moment diagrams should always be plotted. He found the influence-coefficient method of analysis to be the easiest though the Authors preferred the theorem of three moments. The advantage of the influence-coefficient method was, he thought, that it looked upon the structure as a whole and also that very little extra labour was required to check by both elastic and plastic methods of design.

The Authors had mentioned on p. 71 that 2,700 lb/sq. in. compression and 600 lb/sq. in. tension had been allowed at working loads without any mild-steel reinforcement and that no trouble had been experienced. That, he thought, should serve to stimulate confidence that so long as the structure had the required factor of safety in regard to the ultimate load, those high stresses could be allowed without any harm at working loads. However, the cube strength of the concrete in that case had not been mentioned; what was it?

On p. 83 the Authors had stated "it is true that on the ultimate basis . . ., yet permanent cracks in a prestressed structure would be undesirable." Mr Visvesvaraya wished to qualify that statement by saying that "as long as these permanent cracks are not wider than those permitted in corresponding ordinary reinforced concrete structures."

Mr G. O. Kee (Assistant Engineer, Donovan H. Lee, Consulting Engineer) said that ultimately the ideal design would be for fully prestressed frames, but that stage had not yet been reached. Each material had to be used to its best individual advantage with the present knowledge Cost was generally the deciding factor and a prestressed member with in-situ concrete, mild-steel reinforced, seemed at present to be the most economical solution in most cases. He showed a slide illustrating a large structure of composite construction in the United States, covering 6 acres, where the beams were prestressed and designed for continuity by the use of mild steel across the supports. The in-situ deck was designed as a car park. The precast members had been cast and erected within 75 days.

Dr P. B. Morice (Head of the Structures Department of the Research and Development Division of the Cement and Concrete Association) said the Paper represented the first considered discussion of the application of continuity to prestressed concrete building structures in the United Kingdom. So far as he could see, continuity in beams became advantageous in the design-load range only when the dead weight was too great to allow a direct design method to be applied. In some cases the moment variations in continuous beams could be as much as 15% greater than those in the corresponding simple beam system. Probably a similar effect occurred in some forms of building frame.

He could not understand why the Authors had restricted the ends of their beams in simple pin-jointed frames to a state of no rotation, particularly where they used leg prestressing, since they could use, with some advantage, other concordant profiles which bent legs or allowed them to rotate. Also, he could not see why the small eccentricity variations required to eliminate elastic, creep, and shrinkage shortening were not applied.

Mr Goldstein had mentioned the effects of linear transformation on ultimate load. With full moment redistribution linear transformations theoretically had a very small effect, if any, on the ultimate strength of the structure as a whole. Dr Morice had carried out tests on both continuous beams and portal frames in which transformations had varied the strength ratios at critical sections from 1 : 1 to as much as 20 : 1, and the ultimate strength had remained constant. However, in mentioning those results he pointed out that at the moment they referred to structures in which centre-span point loading had been applied, but full moment redistribution might not occur, for example, in the case of sway in frames.

Mr R. W. Pearson (Senior Structural Engineer, Chief Structural Engineer's Branch, Ministry of Works) said he had been directly concerned with the Sighthill Stationery Office store at the erection end. The shortening of beams when stressed across the full width of the building, especially with a multi-storey building, was, he was convinced, of great importance, particularly where the columns were very stiff. In the case of Sighthill the secondary beam had been stressed across the full width of the building—120 ft—and the shortening which had taken place during that secondary prestressing was about $\frac{3}{8}$ in. in 120 ft. There was a very stiff column and the effect on the column would have been serious if they had not permitted the beams to move relatively to the column. The main beams were simply supported and rested on a bracket. While the secondary beams had been stressed, the main beams had slid on the bracket a distance of about half the total shortening. The main beams were jacked off the column, thus permitting the column to spring back and allowing the rest of the movement to take place.

That movement had taken place with only a second cable, which in fact had fewer wires than the primary prestressing cable in those beams. Subsequently further movement would take place due to creep and shrinkage. It had been interesting to hear mention of shrinkage in the joints and the design of joints for connecting precast members would be an interesting subject for a Paper.

The form of the joint was of great importance because of the cost. The question was whether such joints could be filled with mortar or concrete, and whether the cable which passed across them could be sheathed, at a reasonable cost.

*** **Mr J. S. Shipway** (Assistant Civil Engineer, Messrs Maunsell, Posford, and Pavry, Consulting Engineers), referring to the Stationery Office Building, asked if the stresses of 2,700 lb/sq. in. compression and 600 lb/sq. in. tension were the initial stresses in the beams before anticipated losses of prestress due to creep and shrinkage occurred, or the final working-load stresses. If the former, what were the stresses under working load and what percentage loss in prestress had been allowed for?

Longitudinal prestressing had been mentioned in the Paper and in the discussion, but Mr Shipway was not clear as to its precise purpose in the case when the longitudinal beams were pure tie-beams. Was it designed merely to hold the whole structure firmly together and if so, was the amount of prestress required determined by calculation or by engineering judgement? Was the longitudinal prestressing of the whole floor, as in the examples quoted at Le Havre and in America intended to have a function similar to the transverse prestressing of bridge decks?

There was no mention in the Paper of any difficulties arising in the design of end blocks, and it would seem that they no longer presented any problems. Was it becoming quite usual now to design the reinforcement for end blocks and such points as portal knees more by judgement and experience than by actual detailed calculation? Where the tendons passed over the supports for continuity and were anchored in the respective beams, was there any special reinforcement required to take care of the high local stresses at those points, and if so was the amount required calculated for, or otherwise? Could the Authors supply a detail for the reinforcement, if any, at one of those points?

Mr Walley, replying on behalf of Mr Adams and himself, referred first to Professor Baker's remarks on concrete strength and the correlation between design cube strength asked for and that obtained. That was still largely in the hands of the man who put the concrete into the job. The Ministry had a section for advising contractors on the possible best mix to use on a job, but in the end they were left in the hands of the man doing it. The relatively high concrete strengths quoted had been obtained without elaborate or expensive precautions.

He agreed with Professor Baker that probably the ultimate strength was more important than the stresses at normal design load, provided that under normal working conditions the deflexion and general behaviour of the structure were satisfactory. Figs 21 and 22 showing the method used in the Imperial College experiments were interesting. He was not sure that he could agree entirely to cracking over supports if it occurred under the real load which the building was called upon to carry. His only worry was with corrosion. He was not sure that enough was yet known about corrosion in prestressing wire and rods but he had no qualms about the cracking from the theoretical or any other point of view.

*** This contribution was submitted in writing after the closure of the oral discussion. —Sec.

It was true, as Mr Lee had said, that prestressing was not the only answer. The fact that prestressing was a successful medium for a given structure did not mean that it would necessarily be successful or cheap even in a similar problem. Each case had to be considered on its merits and an unbiased view given to the architect or whoever required the information. He hoped that he had not given the impression that there was no other method but prestressing; in fact, he had been at pains to say that it was in reality a very small part of any large building programme although it was a desirable method in certain well-defined cases.

Mr Goldstein had asked what happened at the knee: everyone would like to know the answer to that question. The stresses at that point were extremely high. With normal distribution on the anchorage there was a zone of high compression spreading out from the distribution plates with possibly idle concrete at the corner. In the slide shown by Mr Adams a mild-steel cage had been put at that point; the size of the rods and their disposition being determined by experience. That was the method adopted on a great many engineering problems, except that the experience was hidden in a Code of Practice. On one of the portals there had been trouble on the corner with cracking and crushing of the concrete. It had been found that the contractor had omitted some of the rods in the cage at that corner. A portion had had to be cut away for the steel to be inserted, and after that there had been no trouble.

He agreed with Mr Goldstein and Dr Morice on the fact that linear transformation did not necessarily affect ultimate load conditions. Both Mr Guyon's and Dr Morice's experiments confirmed that in certain cases that was so. Nevertheless, it would be reassuring to have a theoretical basis for it.

Mr Goldstein had discussed whether uniform stressing was better than putting the cable at the lower core point. The second method had been used at Sighthill but it had the disadvantage of lack of continuity. In his Introduction he had apologized for having mentioned ancient buildings—Sighthill was now in that category, though not yet an ancient monument! His apology had been unnecessary since many of the speakers had described the same devices as those used at Sighthill in connexion with other structures.

Mr Samuely had raised an important point which was not appreciated by people who were not very familiar with prestressing. It was that the bending-moment diagram could be varied in an infinite number of ways and the prestressing forces could be used to influence the bending-moment diagram into the desired shape for the structure. By placing cables relative to some hinge that alteration could be made to give the portal structure the requisite shape. The placing of pins in a portal structure, as Mr Samuely had described, was also an important device in framing.

Mr Derrington had complained that nothing had been said about the influence of the relation between the live load and the dead load. Had the Authors tried to weigh all the pros and cons the Paper would have been extremely lengthy. Prestressing certainly became a possibility worth considering whenever the dead loads formed the major portion of the loads to be carried. In the various roof structures shown by the Authors the major load was the self-load and the external loads were essentially very small by comparison.

He did not agree with Mr Derrington on the detailed calculations in connexion with the design of portals; he did not think the calculation was exactly the same whether the moment was put on the knee with prestressing tendons or reinforcement. Redundant reactions were being introduced which were not being introduced when taking the continuity moment with mild-steel rods. There were two ways of dealing with the problem: one was to avoid rotation and secondary moments; in the other the rotation and the moment induced in the legs were calculated and allowances made. The Authors could not agree with Mr Derrington that a linear transformation altered the knee moment. They had assumed that it had no effect at all on the moments. It did not matter very much which way was used. The slides showing composite action between prestressed members and in situ concrete were extremely interesting and confirmed the Authors' feeling that it was a very promising form of multi-storey construction.

Mr Gifford had asked whether they thought two-stage prestressing always an advantage.

Where dead load and live load were very different there were significant theoretical advantages in using two-stage prestressing but a question to be considered was that asked by Mr Lee—whether the equipment was brought back to the job or left on the job in idleness. All such factors had to be taken into account in deciding whether to use one-stage or two-stage prestressing.

Mr Gifford had also raised the question of elastic shortening and the subsequent movement of long beams. It was not clear where the movement came from when it was equal to twice the elastic shortening when stressed. That would be expected to happen later as a result of creep and shrinkage but not immediately. In the Paper the Authors had said that more work should be done on the problem so that they might feel more confident in carrying out long sequences of stressing and so eliminating intermediate anchorages. It was a convenient form of construction because the anchorage devices were external and accessible, which was not always the case with internal jacking arrangements.

His drawings illustrating the system of supports at points of contraflexure had been used many times, for instance many years ago on crane gantries in Orleans. In that case continuity had been established over the supports.

Mr Lowe had outlined the three choices of building frames. The third was becoming more and more popular. It was a useful way of looking at the problem because in so many cases the moment from wind and sway was carried twice over—in the frame and the diaphragm walls, which made it impossible for the frame to act if it wanted to. There was no reason that the wind moments should not be carried, thus pleasing the architect by reducing all the column sizes.

Mr Lowe's remarks on erection procedure were all-important. When introducing prestressing forces it was possible to juggle in many ways but the result could be to make it very difficult for the site. It was necessary to go back to something simple, and the Authors had appealed for that in the Paper.

Mr Visvesvaraya had given some interesting comments on continuity and on the effective centre of the prestressing forces. That could be quite remote from the position of the prestressing tendons. He had drawn attention to the dead load factor found by calculation on one of the jobs in the Paper, and that sort of thing arose from time to time when adhering rigidly to the working-load conditions. On the other hand, with a load factor of 2 only might it not behave very well at working-load conditions. The load factor of 5·8 related to the centre of the beam after linear transformation. The true load factor was about 3 in both beams and columns. In that case the working load and not ultimate strength was used for design. The average 28-day cube strength at Sighthill was just over 7,000 lb/sq. in.

The Authors could not agree that cracks as wide as those acceptable in reinforced concrete should be allowed.

Replying to Mr Lee, Mr Walley said he was sure that composite construction was a powerful tool when designing continuous buildings. He agreed with Dr Morice that continuous beams were not always advantageous; Sighthill was a case in point.

He thanked Mr Pearson for his comments on Sighthill; the views on the additional movement which could take place were important. That had not been so important at Sighthill because it was a completely glazed building, but it would become important where diaphragm walls, which were solid, were introduced, since they tried to stop the building from moving. Provided it could move he was not worried but once they tried to stop it from moving different considerations arose.

In answer to Mr Shipway, the stresses quoted for Sighthill were erection stresses. The stresses under working-load conditions were 2,000 lb/sq. in. (compression) and for zero (tension). The percentage loss of prestress assured was 15. The Authors agreed that where longitudinal beams were merely tie-beams there was no need to prestress. That was the case on the Kilburn building. Where, however, intermediate columns were omitted it became a necessity. The longitudinal stressing of the floors in Le Havre and America was, as far as the Authors knew, the primary stressing. The detailing of the end blocks was often done by eye provided there was nothing unusual in the disposition of the anchorages, but the Authors would hesitate to recommend that method to anyone who had not done a detailed calculation of such a block. It would not perhaps be wise to supply a detail of a particular end block since it would not necessarily apply to another job.

The closing date for Correspondence on the foregoing Paper was 15 March, 1956. No contribution received later than that date will be printed in the Proceedings. —Sec.