**Zeitschrift:** IABSE reports of the working commissions = Rapports des commissions de travail AIPC = IVBH Berichte der Arbeitskommissionen

**Band:** 6 (1970)

**Artikel:** Prestressing of edge members in long buildings to restrain shrinkage, creep and temperature changes

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**DOI:** https://doi.org/10.5169/seals-7781

II

# Prestressing of Edge Members in Long Buildings to Restrain Shrinkage, Creep and Temperature Changes

Précontrainte des poutres latérales des bâtiments de grande dimensions, pour réduire les effets du retrait, du fluage et de la température

Vorspannung von Randbalken in langen Gebäuden, um Schwinden, Kriechen und Temperaturspannungen entgegen zu wirken

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## 1. GENERAL CONSIDERATIONS

It is well known that temperature changes, shrinkage and creep greatly affect cracking of long reinforced concrete buildings. The development of such cracks cannot be completely excluded without prestressing, though it can be minimised when contraction and expansion joints are provided. This may be further improved by the provision of rigid columns suitably spaced and heavy reinforcement so that all theoretical secondary stresses are taken up by the frame construction. Obviously, even in such a case the development of fine cracks cannot be avoided altogether in non-prestressed constructions, although Rogers[1] claims that "after nine years of operation the structure is virtually crackless". This relates to a bakery at which very large temperature variations apply, and for a length of 520 ft. (158.5 m.) no joints were provided except for the roof. Temperature variations of $\pm$ 60°F ($\pm$ 33°C) were considered in addition to a temperature drop of 15°F (8.4°C) which had to be taken up by the stiffness of the framework, the bending moments of the columns being increased by 200%. As already stated, it is quite impossible to obtain a completely crackless reinforced concrete construction in spite of heavy reinforcement, as the development of fine hair cracks cannot be prevented except by the provision of an effective prestress. The avoidance of cracking becomes more important when precast outer surfaces (usually exposed, even polished) are used which badly show any cracking.

The first mentioned author designed in 1957[2] a construction at which 80 m (240 ft.) long edge beams of a six-storey building were to be prestressed. This related to single-bay composite floors containing precast prestressed beams. Although the thickness of the in-situ concrete was reduced to a minimum it formed in each floor together with the beams and edge members a large monolithic area over the entire building without interior columns and without any expansion or construction joints. The main purpose of prestressing was to avoid cracking. Particulars are described in Section 2.

The magnitude of the prestressing force to offset such tensile stresses which would cause visible cracking would be very large if the losses due to the rigidity of long buildings were fully taken into account. This would be greatly reduced if the prestressing of edge beams could be carried out at the cold period after which only a small further contraction would apply. This is, however, normally impracticable. In practice, the most unfavourable conditions must be taken into account. If e.g. a temperature difference of $\pm 27^{\circ}$ ($15^{\circ}$C) is considered together with an equal amount of shrinkage, this would amount to a contraction, corresponding to a compressive strain of $30 \times 10^{-5}$, if a temperature strain of $10^{-5}$ per $1^{\circ}$C temperature difference is taken into account. For a modulus of elasticity of $E_c = 4.5 \times 10^6$ psi ($3.5 \times 10^5$ kp/cm$^2$)*, a tensile stress of 1350 psi (94.5 kp/cm$^2$) would occur, though heating of the interior introduces further complications. If a tensile stress of, say, 600 psi (42 kp/cm$^2$) is assumed as permissible to avoid visible cracks, it would still be necessary to ensure an effective precompression of 750 psi (52.5 kp/cm$^2$). However, a considerably larger prestressing force would have to be applied at both ends of long edge beams if an effective prestress of the above nature were to be obtained at the centre of the building. The first named author decided on the magnitude of prestress at the first construction, discussed in Section 2, on an arbitrary basis for an average prestress of the edge members of 460 psi (32 kp/cm$^2$). This has proved quite satisfactory. Appreciable compressive stresses are introduced towards the centre of long outer members at temperature rise, when normal elongation is prevented by the rigidity of the construction and thus it may be assumed that the prestress, effective near the ends, is gradually transferred to the centre.

Leonhardt[3] suggests the provision of a limited prestress and states that an average compressive stress of 5-15 kp/cm$^2$ (70-210 psi) should be sufficient. However, even for such an insignificant minimum effective prestress appreciable prestressing forces would have to be applied at the ends to overcome the losses due to rigidity. In the example shown by Leonhardt, it would be interesting to know how large a prestressing force had to be applied at the ends to overcome the friction of the 71 m (232 ft) long wall (fig. 3)

Obviously the provision of a partially prestressed construction is quite satisfactory, either according to Class 2 of FIP-CEB at which visible cracks do not occur (e.g. with a permissible tensile stress as mentioned before); or a construction according to type (ii), as described by the second author in 1950[4] and at the 8th AIBS Congress 1968[5], which was also included in the "First Report on Prestressed Concrete" of the Institution of Structural Engineers 1951. This type (ii) (which unfortunately has been completely ignored in the FIP-CEB classification) presents a fully prestressed construction under normal loading with the possibility that fine hair cracks may develop occasionally under temporary extreme loading; this applies in the present circumstances to severe frost. These cracks would, however, be completely closed under ordinary conditions.

The experience with the first buildings (see Section 2) was very satisfactory in as much as cracks were not noticed and a contraction at the ends of the building of a magnitude of ½ in. (12.5 mm) was noticed when the building was not heated in winter. This indicates that the prestressing force was overcoming the rigidity of the framework. The firm of Consulting Engineers, Jan Bobrowski and Partners, has continued with the provision of prestressing edge beams of long multi-storey buildings. In order to ascertain this, it was considered important to carry out strain measurements. This was first done at a parking garage in Bristol[6] at which photo-elastic stress meters were embedded in the concrete (see Section 3). In another parking garage in Reading the longitudinal non-prestressed, central beam was interrupted by transverse ramps, and in the roof transverse cracks developed which stopped at a distance of about 6 ft. (1.8 m) from the prestressed edge beams, as described in Section 4. Cracking was avoided altogether at another garage in Reading at which also the longitudinal centre beams were prestressed. In another building, at present under construction (Section 5), vibrating wire strain gauges are being embedded in the concrete which should enable the authors to study the gradual develop-

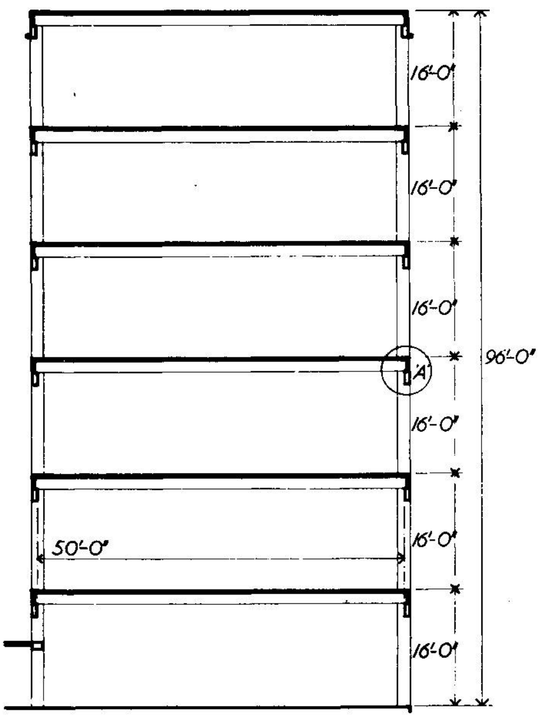
* The stresses are given in psi (lbf/in$^2$) and in kp/cm$^2$ (kgf/cm$^2$) rather than in N/mm$^2$ which fortunately has not yet been introduced internationally.

ment of prestress in the edge beams and the influences of temperature changes; there being thermo couples provided which allow the measurements of the temperature inside the concrete. It is hoped that a more detailed report after temperature movement during the two coming winters can be presented at the Amsterdam Congress 1972.

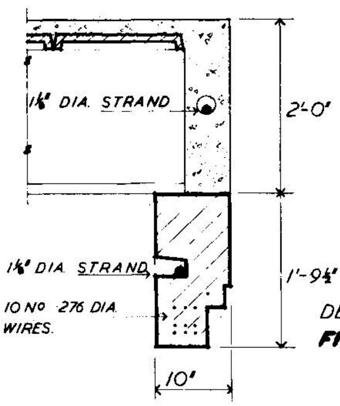
Finally it is pointed out that the design should allow great flexibility in longitudinal direction while still ensuring full stability of the building. Such a solution has been embodied in a design for a proposed building at which some of the rigid corner supports are on rollers.

## 2. WATSON HOUSE, FULHAM, LONDON

As described in paper[2], there are two 6-storey wings (see cross-section Fig. 1), 15 m (51 ft) wide and 30 m (96 ft) high. The 0.6 m (2 ft) deep floors are of composite design, comprising precast pretensioned beams spanning the entire width and an in-situ concrete topping placed on prefabricated concrete shuttering between them. The edge members in each floor (detail A of Fig. 1, as seen in Fig. 2) were prestressed. They are of composite nature, the lower part being precast with pretensioned tendons, the upper part cast in-situ at the same time as the topping to the main floor deck. The precast part which carries the dead weight of the floor contains a chase in which one of the two 19-wire strands 1 1/8 in. (28.7 mm) dia. was placed, the other being located in a duct of the upper part. The minimum breaking force ofthesewas $362 \times 10^3$ lbs ($162 \times 10^3$ kp) the initial prestressing force being 70% of this value and an average effective force of 60% (allowing 40% losses) has been considered, resulting in an average effective prestress of 460 psi (32 kp/cm$^2$), as indicated in Fig. 3, which shows the various stress conditions. Slender reinforced concrete columns supported the beams. Though this design was only experimental it has proved to be quite satisfactory and, as already stated, shortening of approximately 1/2 in. (12.2 mm) occurred at each end over a length of 80 m (260 ft). This would correspond to a temperature reduction (including shrinkage) of 33°C (59°F).




CROSS SECTION OF WATSON HOUSE.
FIG. 1




DETAIL 'A'
FIG. 2

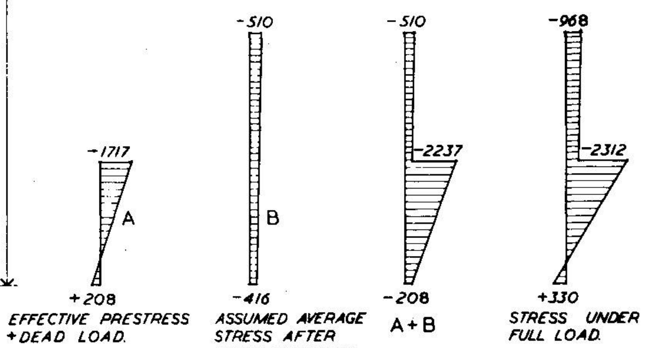



FIG. 3

## 3. CAR PARK AT THE UNICORN HOTEL, BRISTOL

This and the following two car parks, described in Section 4, are based on similar designs inasmuch as precast structural diamond frame wall units formed the outer walls, whereas precast reinforced concrete frames were used for the central spines, the first two car parks containing one central spine and the third car park having two such constructions. In each case long span,composite floors were used similar to the building described before. This is, in the author's view, advantageous from the production point of view because of the reduced number of columns and it is often also more economical. The composite floors were propped in all the designs, which with inverted T units greatly improves the economy.

Fig. 4 shows the elevation of the car park adjacent to the Unicorn Hotel, Bristol. The main element in the external walls is the precast "double diamond" unit, mentioned before, which incorporated an integral ring beam, as indicated in Fig. 5. Each upper ring beam contains five post-tensioned tendons of 1 1/8 in. (28.5 mm) dia. At the first floor level there are six such tendons provided. Each of the tendons was tensioned to 70% of its strength. The precast "double diamond" units have steel shoes with locating studs, which were removed before stressing. There was little friction at prestressing since the steel shoes were lubricated with graphite and only afterwards connected by welding. Finally they were protected by a 1 in. (25.4 mm) thick layer of Pyrok (a cementitious gypsum mixture). A relatively high prestress was introduced in the precast concrete, since prestressing was carried out, before the in-situ concrete was added.

The floor construction, already described, is seen in Fig. 7. External support at ground level is obtained by five V-shaped members 40 ft. (12 m)

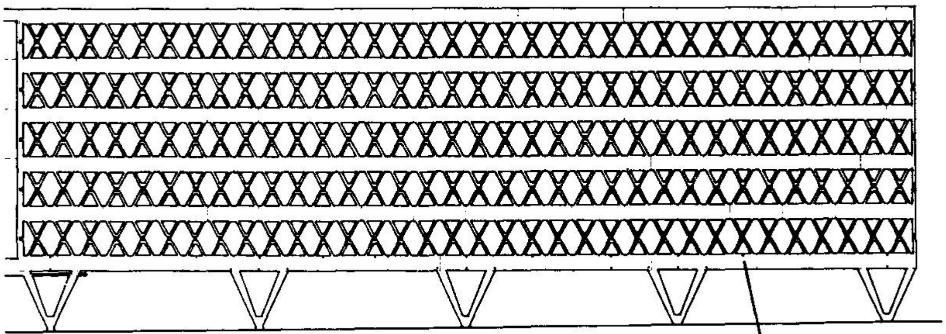
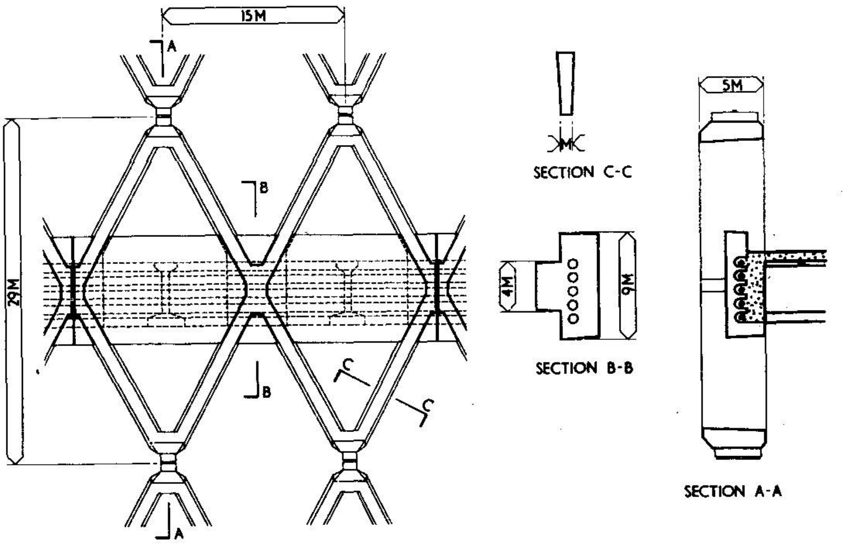
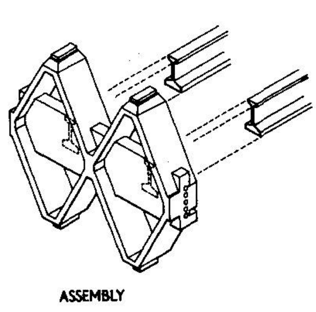


FIG. 4. ELEVATION OF CARPARK – UNICORN HOTEL, BRISTOL

PHOTOELASTIC STRESSMETERS IN EDGE BEAM
FIG. 6




ELEVATION
DOUBLE DIAMOND UNIT

FIG. 5.

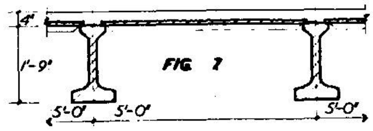



FIG. 7

apart each carrying approximately 1,700,000 lbf (770,000 kgf); each arm containing three tendons of 12 wires 0.276 in. (7 mm) dia., anchored in the foundation pile cap and carried through the first floor ring beam. Post-tensioning of these V-shaped supports was carried out after completion of the first floor. All external structural elements are of white concrete with Capstone aggregate.

This system is highly statically indeterminate and was designed by the first named author by analogy to rigid walls and anchorage zones which allowed dimensioning of the "double diamond" element. It was subsequently checked by Professor Z.S. Makowski and Professor J.R. Hussey, who developed a computer programme based on statical indeterminacy of 362, as stated in paper[6], requiring 990 simultaneous equations. The building was self supporting during erection, and each stage was analysed separately.

In view of the complexity of the design it was decided to check the actual stresses by means of photo-elastic stress meters. These are bi-axial glass gauges which, when bonded to the material subsequently subjected to stress, allow the magnitude and directions of the applied principal strains to be obtained from photo-elastic fringe patterns visible on the gauge if it is observed with polarised light. Fifty such stress meters, produced by Horstman Ltd., Bath, England, were embedded in diagonal and horizontal precast concrete and in-situ concrete members at first and second floor levels and two at the bases of "V-shaped" columns. Principal particulars of positioning are shown in Fig. 6. This construction was built in 1965 and the stresses were assessed for the fully erected building under dead load only, based on the strains from the photo-elastic cylindrical plugs. The stresses obtained for the bases of the "V-columns" agreed very well with the calculated values. Most of the compressive stresses, obtained from the stress-plugs inserted in precast concrete, are in some agreement with the calculated stresses. Some are as close as 94% of the calculated values, others fall down to 60%, whereas in two cases the difference is even greater. This seems to indicate that there were secondary stresses and/or friction influences at prestressing, in spite of the provision of sliding. Most of the stress plugs inserted in the in-situ concrete did not function satisfactorily, probably in consequence of imperfect bond. A number of plugs did either not work at all or gave unreliable results particularly when in tension. It is claimed by the manufacturer of the stress plugs that, in the meantime, improvements have been made which should give better results in future.

In March and August 1970 further strain measurements were made. The temperature in March was + 42°F (+5.5°C) and in August + 68°F (20°C) respectively. A number of stress meters indicated variations in the stress level at several points along the length of the girder, relevant to the ambient temperature of concrete.

Generally the stress meters located in the compressive zone of the beam, near the supports, reflected the restrained expansion of the girder by the definite increase in the compressive stress.

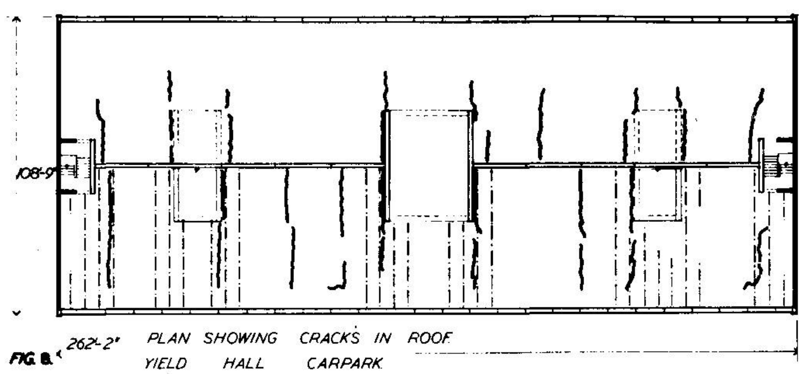



FIG. 8. PLAN SHOWING CRACKS IN ROOF YIELD HALL CARPARK

## 4. CAR PARK, YIELD HALL, READING

The construction of this car park is similar to that in Bristol, except that instead of the V-shaped columns, single columns were provided at 5 ft. (1.50 m) centres. Moreover, in this two-bay building of similar width to the Bristol design, the arrangement of the ramps was different and the centre spine beam was interrupted. As already stated in Section 1, transverse cracks developed in the roof construction, as indicated in Fig. 8. These cracks extended across the building and terminated at a distance of about 6 ft. (1.8 m) from the prestressed edge beams. This clearly shows the efficacy of prestressing of the selected construction. The car park was completed in 1966.

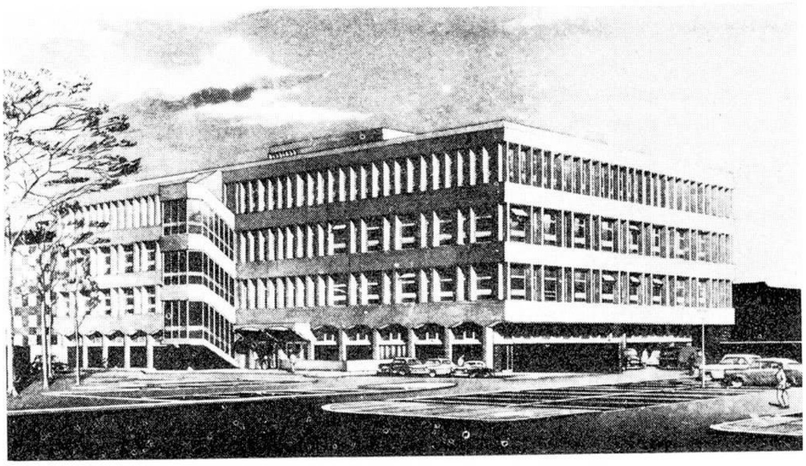
Based on this experience the centre beams of a second car park in Reading, having three bays, were prestressed. Otherwise, the construction is in principle the same. This building, which was completed in 1968, does not show any transverse cracks.

## 5. OFFICE BUILDING, WESTMINSTER, LONDON, 1970

This is a four-storey building, 192 ft. (58 m) long and 108 ft. (32.8 m) wide, as seen in the photograph, Fig. 9, whereas Fig. 10 shows a typical plan. There are two staircases at the ends and the walls enclosing them are provided with sliding joints to facilitate prestressing in longitudinal direction. The external walls are precast, and all surfaces have exposed polished concrete. The precast ground floor columns have cantilevers and are 12 ft. (3.6 m) apart. They are separated from each other, as indicated in the picture, Fig. 9. The columns are connected with the upper floors by reinforcement so placed as to give minimum stiffness in longitudinal direction, but appreciable stiffness transversely. The upper floors contain precast load bearing box frames which are connected to the edge beams to achieve a similar effect to that at first floor. The floor construction is similar to that of the previous buildings with the difference that the prestressed beams of 21 in. (0.53 m) depth are 4 ft. (1.2 m) apart. There are two outer bays of approximately 50 ft. (15 m) span. Along the spine, precast reinforced concrete H-frames are provided 20 ft. (6 m) apart. Cross-section of the edge beams are seen in Fig. 11: "A" refers to a normal case at which the in-situ slab is connected with the edge beam. It contains two tendons, each of them comprising two cable of four No. 0.6 in. (15.3 mm) dia. strands. Section "C" refers to a part adjacent to the staircase (where a strain gauge has been placed) without slab, whereas the horizontal Section "B" indicates that the joints between adjacent precast parts remains open for 6 in. (15 mm), which means that the prestress at the joints is greater than within the edge beam of relatively large section. This design had been completed before the car parks at Reading became operational, however, it was still possible to incorporate the post-tensioning of the transverse edge beams in full appreciation of its advantages clearly demonstrated in these car parks described in the preceding section.

Since such a design is very complex with regard to determining the inherent rigidity, causing loss of prestressing, more reliable strain measurements were introduced to investigate the development of the prestress in the various edge beams and to study the effects of temperature changes.

Vibrating strain gauges were embedded in the edge beams; generally, three gauges were provided at each edge beam (one approximately at the centre and one each about 20 ft. away from the ends). At the roof two more strain gauges are being provided at the two longitudinal edge beams at intermediate positions. These gauges were developed by the Building Research Station and are manufactured by Deakin Instrumentation Ltd., Walton-on-Thames. The type is a pre-tensioned transducer and allows measurements of internal slowly varying strains (e.g. due to temperature change) in the concrete. The gauges and the measuring equipment, including that for determining the internal temperatures by means of thermo couples, were provided by the Building Research Station, which is co-



FIG·9

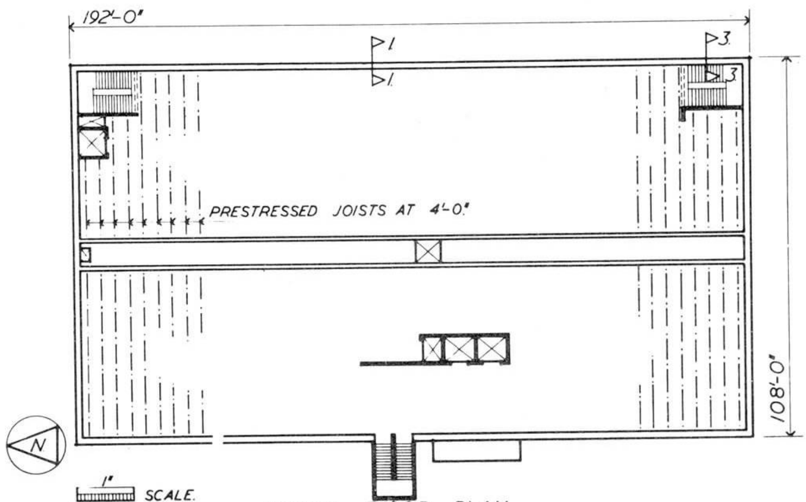
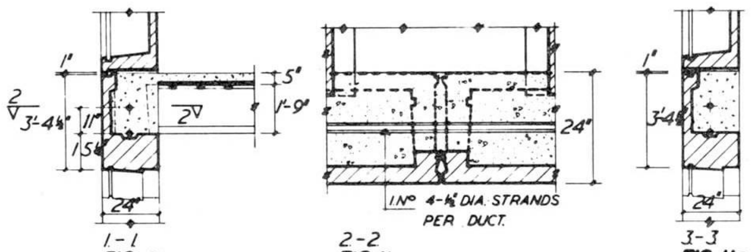



FIG. 10. TYPICAL FLOOR PLAN. OFFICE BUILDING. WESTMINSTER.




1-1 FIG 11A. 2-2 FIG 11B. 3-3 FIG 11C.

operating in these investigations, and the assistance of Mr D.W. Bryden-Smith is gratefully acknowledged.

At the time of writing this paper, a limited number of measurements only have been made: however, they have already revealed some interesting results. For example, in the east facade edge beam of the ground floor, the gauges at 21 ft distance from the ends showed after prestressing compressive strains of $26 \times 10^{-6}$ and $41 \times 10^{-6}$ respectively, whereas near the centre the strain was only $15 \times 10^{-6}$. The difference near the two ends can be explained by the fact that the second strain relates to a rectangular section ("C" in Fig. 11) whereas at the first ("A" in Fig. 11) the slab of the floor co-operates and takes up a part of the prestress. These strains would correspond to respective stresses of 117 and 184 psi (i.e. 8 and 12.9 kp/cm$^2$) at the ends and 67 psi (4.7 kp/cm$^2$) at the centre for $E_{83} = 4.5 \times 10^6$ psi ($315 \times 10^3$ kp/cm$^2$). The nominal compressive stresses for the entire cross section of the longitudinal edge beams was at prestressing 270 psi (19 kp/cm$^2$) at the ends. It should be noted that the edge beams carrying the ground floor were propped at the time which must have contributed to the losses, and it will be interesting to ascertain the changes after removal of the props, and of course proper allowance will also have to be made for proportion of prestress taken by heavy reinforcing bars in the first floor beams which is particularly concentrated towards the corners of the building. A later measurement at higher temperature shows an appreciable increase in the compressive strain at the centre, elongation being hindered by columns.

## 6. CONCLUSIONS

Experience described in the foregoing, has shown that the development of cracks in long multi-storey buildings without expansion and/or contraction joints can be satisfactorily controlled by prestressing. This is preferably applied to longitudinal edge beams and, in wide buildings, to longitudinal spine beams and transverse edge beams. Provisions should be made to allow contraction of the beams during prestressing to ensure compatibility between design assumption and actual behaviour of the building construction. The maximum advantages are possible if only precast parts of edge beams are prestressed, as in the car parks described above, before in-situ concrete is added. Moreover, if the precast edge beams are made of lightweight concrete their efficiency could be almost doubled in view of its lower Young's Modulus, thus using to advantage what is normally regarded as an inherent drawback of lightweight concrete.

REFERENCES

1. P. Rogers: "Temperature Changes on Reinforced Frame of Bakery Structure", Presented at the 1970 Annual Convention, ACI, New York, April 11-17.
2. A. J. Leggett and J. Bobrowski: "New Premises for the North Thames Gas Board, Fulham", Paper No. 6486, Proc. Inst. of Civ. Eng., Vol. 20, September 1961, pp. 85-106.
3. F. Leonhardt: "Konstruktive Massnahmen zur Verringerung schädlicher Einflüsse von Kriechen, Schwinden und Temperature", IABSE Symposium Madrid, 1970, p. 148.
4. P. W. Abeles: "Further Notes on Principles and Design of Prestressed Concrete", Civ. Eng. & Publ. Works, Vol. 45, July and Oct. 1950, pp. 443-445, pp. 657-662.
5. P. W. Abeles: "The Practical Application of Partial Prestressing", IABSE, 8th Congress, New York, Prepared Discussion IVb, pp. 913-924.
6. J. Bobrowski, J.S. Bernatek and B.K. Bardhan-Roy: "Use of Photo-elastic Stressmeters in the Unicorn Hotel Car Park, Prince Street, Bristol, to be published.

SUMMARY

In this paper the advantage of prestressing edge beams of long multistorey buildings are discussed so as to avoid cracking due to contraction. This includes description of five examples. Specific considerations are necessary at the design stage to permit longitudinal deformation at prestressing in order to reduce the losses of prestress due to rigidity of the building. Strain measurements at two of the buildings are briefly described, which are necessary to clarify the great complexity of the problem.

ZUSAMMENFASSUNG

In dem vorliegenden Artikel werden die Vorteile von Vorspannung langer Randbalken in mehrstöckigen Gebäuden, um Rissbildung infolge Verkürzung zu verhindern, diskutiert und 5 Beispiele beschrieben. Besondere Ueberlegungen beim Entwurf sind nötig, um Verluste der Vorspannkraft infolge der Steifheit der Konstruktion zu verhindern. Dehnungsmessungen an zwei der Gebäude werden kurz beschrieben, die nötig sind, um die Kompliziertheit des Problems klarzulegen.

RESUME

On expose dans le présent article les avantages de la précontrainte des poutres latérales dans les bâtiments à plusieurs étages, dans le but d'éviter la formation des fissures dues à la contraction. On décrit ensuite 5 exemples. Certaines considérations sont nécessaires au stade du projet, afin de permettre les déformations longitudinales provoquées par la précontrainte, qui doivent réduire les pertes dues à la rigidité de la structure. On présente rapidement les mesures d'allongement sur deux des bâtiments, ceci pour essayer de clarifier ce problème complexe.