Elephant and Castle shopping centre

by

**E. W. H. Gifford, B.Sc., M.I.C.E., M.I.Struct.E. and A. A. W. Butler,
B.Sc., A.M.I.C.E.**

Mr J. Faber (Messrs Oscar Faber & Partners) supported very strongly the Authors' conclusions that, for this type of central development, the cast in situ flat-plate slab was undoubtedly the cheapest, and the quickest to use. It also achieved the minimum depth of deck construction and allowed for the easiest distribution of mechanical, electrical and sanitary services. It had in addition a satisfactory natural fire resistance.

73. His own firm was at present concerned with the design of the new town centre at Cumbernauld, near Glasgow, which was roughly 600 ft in one direction and would extend to about half a mile in the other direction. Here, too, flat-plate slabs were being used for the six lower deck-levels.

74. The main difference between Cumbernauld and the Elephant and Castle was that whereas the spans at the latter were generally 24 ft, those at Cumbernauld were generally 35 ft and 44 ft. Many different forms of construction, including ordinary concrete beams and slabs, precast concrete, structural steelwork, composite construction, and prestressed concrete had been studied, but for the particular dimensions waffle flat-plate slabs gave the cheapest overall solution to the job.

75. The greatest loads carried by the floors at Cumbernauld were those of the construction of the next floor up, and, indeed, as a result, the structure had already been pre-tested, so to speak, with a good load factor.

76. Fig. 21 showed the ingenious method of formwork which had been devised by the contractors, Messrs Duncan Logan. The rib soffits were formed by steel T-sections in pieces generally about 6 ft long. These were supported on special props arranged at 6 ft centres in the two directions. The indentations were then created by fibreglass pans pushed up from beneath. The advantage of inserting the pans from beneath was that after the concrete had been placed the pans were then free to be released first while the ribs were still supported by the T's.

77. Another feature of this formwork system was that the special props gave double support to the waffle ribs. At the end of each prop there was a bearing plate, which formed its own little piece of soffit shuttering, and the main lengths of the steel T's were then supported separately from the props with independent adjustment. This enabled the rib soffits to be struck earlier than the removal of the props, which were required to support the ribs in between.

78. The sequence of construction therefore was to set up the props and T's; then to place the reinforcements; then to present the fibreglass pans from underneath; and then to pour the concrete. The pans were then removed by a brief shot of compressed air 48 hours later; the rib soffits were struck at seven days, but the props at 6 ft centres in both directions were left in position for 14 days.

79. The Authors were correct in saying that the problem arising from the use of flat-plate slabs was the inflexibility which this could lead to in terms of changed requirements. At Cumbernauld they were trying to meet this point by providing large openings in the floors to a carefully considered layout and then disciplining the tenants to use only these openings. Expansion joints were widened out to give a

FIG. 21: CUMBERNAULD: FORMWORK

10 ft clear width, and these were also available for accommodating new stairways, lifts, and so on. The openings were, of course, slabbed in when not required.

80. Certain problems of deflexion could arise using high tensile steel with long span slabs, particularly with the shrinkage characteristics of the Scottish aggregates. This was especially so where electrical floor heating might subsequently be used by tenants. At Cumbernauld these problems had been eased to some extent by providing cantilever overhangs at all edges, and by working to much lower span/depth ratios than those given in C.P. 114.

81. Shrinkage and creep, of course, also came into these problems of deflexion. At Cumbernauld they had done their best to overcome the problem of shrinkage, by choosing the most suitable aggregate available in the area, by carefully controlling the water/cement ratio to as low a figure as possible, and by taking special precautions to prevent the slabs drying out too rapidly.

82. In considering creep, the slabs had been cambered rather more than might normally be the case, and arrangements were made to ensure that the formwork props were left in place for 14 days.

83. On the 35 ft spans the camber provided had been $1\frac{1}{2}$ in. Of this roughly $\frac{3}{8}$ in. had come out due to dead load deflexion on removal of the props, and a further $\frac{3}{8}$ in. had come out due to creep in the subsequent 18 month period. As far as could

be told, creep was still going on, and it was likely that a total of 1 in. of the camber would come out due to dead load. This would then leave roughly $\frac{1}{2}$ in. to come out as a result of superimposed load, if ever the whole of this were applied; and bearing in mind that the superimposed loads were roughly only half the dead loads, it seemed the provision made had been about right.

Mr R. W. Turner (Consulting Engineer) asked for clarification on types (E), (F) and (G) of the structural systems in Fig. 7. The best solution had been the 100% example, a reinforced concrete flat plate, obviously designed according to C.P. 114. The two next best solutions were (E) and (F). What was the method of analysis and design used in the calculations?

85. Basically a two-way spanning slab could be justified only on the old-fashioned, and possibly out-moded, elastic design method. One was not allowed to use the more up-to-date load-factor design method. Would a more profitable scheme have resulted had the yield-line analysis (admissible under the Code) been applied to type (E)?

Mr D. H. Little (Messrs Walter C. Andrews & Partners) said that while he had built some basements, most of his experience with water work had been confined to graving docks. He had never seen a graving dock which did not leak, but the water came through the cracks and joints, not the body of the concrete. His experience was that the quality of the concrete was not all-important in this respect. Most concrete in pre-war docks would have been about a 10:1 mix, yet he had never seen water passing through it, nor did even oil fuel penetrate more than $\frac{1}{8}$ in. into it. However, if reinforcement was not given a cover of more than 2 in. there would be trouble with salt water, so salt water must penetrate that depth, though it did not pass through the body of the concrete.

87. The Authors had specified water-tightness as being where the leaks were not more than one in 4000 sq. ft. From where had they obtained this figure? If there were any leaks outside the specification, and the Client wanted them attended to, did he have to pay for this?

88. If leaks occurred, they would be at construction joints or cracks. One could do one's best to control the cracking, but Mr Little doubted if it could be eliminated entirely. But even if it could, there must be openings at the construction joints, and it seemed to him that these should be treated with a water-bar. If the water-bar were designed properly (which was the consultants' job) and built properly (which was the contractors' job), then if there were leaks, it was the contractors' fault and it was right that they should remedy the situation.

89. On this job the Consultants seemed to think that they could keep the construction joints watertight without water-bars by putting reinforcement through the joints.

90. It was a pity that the Contractors had elected to put every safeguard in—water-bar, plasticisers, etc.—in the hope of keeping it watertight, because, as a consequence, one could never know whether the Consultants were right! How tight would it have been if the water-bars had been omitted?

91. In recent years he had built a tank 400 ft long, 200 ft wide and 18 ft deep, which had had to be filled on the inside with water. All possible methods had been used to exclude the water. When the tank was first filled, there were 400 leaks in an area of 100 000 sq. ft, an average of one in 250 sq. ft! Nearly all the leaks were through the holes by which the back shutter was tied to the front shutter, caused mostly by poor plugging. Three years later the tank was 100% watertight. Unfortunately it was impossible to tell from that example and from the Authors' work, whether there would have been leaks if there had been no water-bars.

92. He was interested in the calculation for the Tube, which was 60 ft away. It had been calculated that it would move $\frac{1}{4}$ in. at one end, and $\frac{1}{8}$ in. at the other. This

was a delightfully fine assessment for such a massive structure, and it was a little unfortunate to have spoiled it by overlooking $\frac{3}{4}$ in.!

Mr F. Walley (Ministry of Public Building and Works) asked the Authors about the ICOS wall. They had been using underpinning and considering the use of sheet piling and then had accepted the Contractors' proposition to use an ICOS wall. Why had they not considered the use of an ICOS wall as a permanent structure?

94. He had found in his experience that one obtained a cheaper structure if one incorporated the ICOS wall as the main wall of the job. There was not much saving if one merely substituted it for steel sheet piling.

95. He had had the problem of building a waterproof concrete basement where the contractors insisted on putting in water-bars and plasticisers. He had usually found that they were led into greater trouble because of leaks at joints where the water-bars had been put in inadequately. In the text there was a reference to leaks at these points. Had the Contractors refused to construct the basement unless these extra details were included? If so, had they paid for the extras?

Mr H. Addison (Consultant) said that he spoke from the users' angle. What did the shoppers themselves think about this magnificent building which had been erected for their benefit? He had visited the building to see how it appeared to users, how many shoppers were there and how many shops. How attractive was the shopping centre likely to be?

97. On a more technical level, looking at the shops already occupied and at the empty spaces which would be turned into shops, he had wondered what access was provided for shop keepers. Was there a rear corridor? How were goods brought from the street to the counter without interfering with the normal use of the shops?

98. Was there any main ventilation in the building or was there natural ventilation? No doubt the general spacing of the main columns was enforced by the difficult nature of the ground, but had there been any discussion about the optimum size of the shops? Had potential shopkeepers been consulted? Or had the site conditions been so stringent that there was little choice? At Cumbernauld the spacing of the columns had permitted the shops to be rather larger than in the Elephant building.

Dr R. E. Rowe (Cement and Concrete Association) asked a question about terminology. He had been a little perturbed to read that in studying the various structural actions it had been thought best to adopt a membrane action in the slab. He thought that he might have misread the Paper, because his understanding of membrane action was rather different from that used by the Authors.

100. They had been considering two-way bending of the slab. It would have been interesting to consider the possibility of arching action in a structure of this type, where the effective containment of an internal slab was considerable. This would limit deflexions in a slender slab and would produce an even more economical solution to the problem.

101. Fig. 6 gave a comparison of structural systems. There was a drop on the slab, and Dr Rowe would like to know the result on the economics of the system if that drop were removed. By suitable detailing of the reinforcement in the slab, the particular problem for which the drop was introduced—punching shear—could be dealt with, and the entire shuttering system could have been greatly simplified.

102. His next question might perhaps be directed to Mr Faber, who had mentioned the use of the waffle slab system, which had the same basic characteristics and could develop membrane action. The consideration of this might well have solved many of the problems imposed by deflexion limitation and might have provided a more economical solution than that in fact adopted.

Mr F. S. P. Turner (London Transport) said that the Authors of the Paper were to be congratulated on preparing a Paper with so many interesting features. He had

FIG. 22: VERTICAL MOVEMENT OF TUNNEL CROWN

been mainly concerned with the two Tube tunnels during the progress of the works and he thought that the actual measured tunnel movements would be of interest to members.

104. London Transport had been very concerned about the safety of these tunnels and the trains which occupied them, for from their experience it was known that when such a large area was excavated the ground at the bottom of the excavation would heave and the tunnels would move upwards. It was thought that even if the tunnels did not fracture, the leakage of water into them would be seriously increased. For this reason, the progress of the excavation and the building works had been carefully watched and the tunnels had been continually under observation. This had been done by taking precise levels on the crown of the tunnels and horizontal diameter measurements during the progress of the works.

105. The elastic heave of the tunnels had been calculated as the Paper stated by using Steinbrenner's formula. As shown in Fig. 22 the maximum elastic heave had been calculated to be about $\frac{1}{4}$ in., and to occur in the centre of the site. The amount of excavation removed was equivalent to 1.6 tons/sq. ft. The site was excavated in sections to allow building work to proceed on the completed sections. The building replaced 0.6 tons/sq. ft on the ground except under the tower block which replaced the equivalent weight of all the removed overburden.

106. Steinbrenner's formula was very dependent on the value used for Young's modulus. The one used was obtained from the Shell Building on the South Bank. It was a great pity that no value for Young's modulus had been obtained for this particular site during the soil investigation.

107. The undulating movement of the tunnels outside one end of the excavation should be noted. It was at this end that there were some step-plate junctions which gave an artificial stiffness to the tunnel.

Dr A. J. Dutt (Greater London Council) asked what proportion of wind load was shared by the different service cores for office and shop blocks? Was any wind load resisted by the frame of the office block? It would also be of interest to know what calculations had been made for the flotation of the structure and the intensity of flotation that had been encountered for this building.

Mr T. Akroyd (Consulting Engineer) said that the Authors had asked for some comment on the question of waterproof basements and contractual responsibilities. It seemed to him that if a contractor undertook to construct a waterproof basement, that was all that was needed. There was no need to define the responsibility for the design and for the construction because these followed automatically from the conditions contract. If there were negligence in the design the matter would, undoubtedly, be taken to arbitration or to the courts, whether or not responsibility for the design was written into the contract. All that was necessary was to agree a contract with a contractor for constructing a waterproof basement. Indeed one could go further and in the contract documents specify the type of concrete and exclude, if necessary, the use of water-bars.

110. With regard to the concrete mix he noted that a mix of 6:1 aggregate/cement ratio had been used; this agreed with Mr Akroyd's investigations which suggested that 6:1 was about the maximum richness consistent with avoiding undue shrinkage or poorly placed concrete. He agreed with Mr Little that water generally did not seep through the concrete, as such, but rather through shrinkage cracks or poorly placed concrete. Although all concrete was slightly impermeable and basements remained dry because the rate of evaporation from the inside was less than the rate of penetration through the concrete, this was not the cause of leaking in basements.

111. Turning to the question of professional engineering responsibility, Mr Akroyd said he assumed that there had been an Architect and a main Contractor and that the work had been carried out under the R.I.B.A. *Conditions of Contract*. If so, then the Engineer could have had no responsibility under the contract. This was because it was impossible to write into the R.I.B.A. *Conditions* that an engineer should have responsibility, simply because Clause 12 of the R.I.B.A. *Conditions* stated that there should be no alteration to the Conditions. Under the R.I.B.A. contract there were only four parties, the client, the contractor, the architect and the quantity surveyor. And yet the engineer was employed directly by the client: he was not an agent of the architect, and he operated on site as an agent of the client. He was accepted on the site as a client's agent because the contractor accepted him. If the contractor said that he took no orders from the engineer, then the engineer must work entirely through the architect. How had the Authors overcome this difficulty in discharging their obligations to the Client?

The following contributions were received in writing:

Mr K. J. Flemons (Sir Frederick Snow and Partners) wished to know whether any thought had been given to the use of the ICOS diaphragm wall as part of the permanent structure once a decision had been made on its use by the main Contractor, as this would have introduced some saving into the basement construction.

113. The ICOS system had been adopted for a three-storey basement car park in the west end of London, with which the writer was connected, and in this case the diaphragm wall served both as the temporary retaining wall and the permanent basement wall. Considerable saving had been experienced in the time spent on the basement construction by the use of this system, which in turn gave corresponding savings in cost.

Mr R. W. Turner (R. J. Crocker and Associates) wrote that it was stated in the Paper that the office floors comprised 9 in. thick H.T. slabs with columns on a 24 ft

FIG. 23: PRESTRESSING FOR FLAT-TYPE FLOOR SLABS

grid. At the discussion on the Paper some mention was made of shear reinforcement over the columns (which unless they exceeded 18 in. square would lead to excessive shear stress in the concrete), and he wished to know what form this reinforcement took, particularly with respect to the recent American Concrete Institution's specification on plate floors, which included a 10 in. lower limit for shear reinforcement.

Mr C. A. Power (Messrs Biggs, Power and Clark, Wellington, N.Z.) commended the Authors on the Paper, particularly for the presentation of the cost analysis of various floor systems. These compared well with a similar analysis carried out by his firm.

116. For some years they had adopted prestressing for flat-type floor slabs and had established the economy of a type not covered in Fig. 7. On a given grid of 24 ft $\times$ 24 ft, a suitable depth for the 120 lb/sq. ft live load would be 7 in. (Fig. 23). The cost per square foot, according to their records, would be 95% of that of item (G) (Fig. 7) and at the same time achieved the following advantages:

- (a) completely flat soffit (elimination of drop panels);
- (b) elimination of deflexions due to elastic flexure, creep and differential shrinkage;
- (c) cable layout which virtually reduced the flat beams of Type F to within the flat soffit, and therefore achieved the same flexibility of use which justified the use of Type F for the major part of the shopping centre structure, without the additional costs otherwise entailed;
- (d) reduction in overall constructional depth of 15%.

Mr A. W. Shilston (Consulting Engineer) enquired whether feasibility studies on lowering the ground-water level had formed part of the site investigation. In the context of § 43 of the Paper, what was the division of responsibility as between Consultant and Contractor on the choice of the dewatering method actually employed? Were stability problems other than those relating to flotation risk contemplated, having regard to the grading analyses of the gravel? If so, what precautions were taken in the design of the wellpoint system to prevent instability of the gravel, and what control was exercised during the site operations?

118. The Authors' remarks in § 36 regarding the efficacy of water-stops were interesting and Mr Shilston invited further comment, even of a speculative nature, as to their doubtful efficacy. Unless very accurately positioned, 6 in. stops tended to be on the short side; if they were going to be used, a 9 in. minimum length was perhaps to be preferred.

119. In water-retaining or -excluding structures, there were at first sight objections on the grounds of the extra cost involved, to the practice of discontinuous casting of concrete wall panels, using either the alternate bay or the gap system, to mitigate the effects of concrete shrinkage. If greater reliance could be placed on water-stops to achieve their appointed task, there seemed to be a case for placing walls in high lifts in continuous panel lengths with a water-bar across the vertical construction joint. The introduction of the water-stop which effected a joint seal on the water contact face of the concrete was an interesting development in furtherance of that aim. With the reduction in length of construction joint, the possibility of then using both a surface and a central water-stop was worth canvassing, although the inclusion of the former would militate against the use of the grouting technique on a subsequent occasion in the event of leakage at the construction joint.

120. In § 63, the Authors stated that basement wall panels were cast in 30 ft bays with 10 ft lift heights. Were the panels cast continuously or alternatively, and was the lift height exclusively the Contractor's choice or was that criterion a limiting one imposed on him by the specification? Was a maximum delay period between the placing of adjacent lifts specified?

121. To gain a better overall concept of the work at the Elephant and Castle shopping centre, could an approximate indication be given of the relationship between the civil engineering, building and services components of the job cost? Mr Shilston had speculated on the problems likely to arise in a scheme of this kind executed under one omnibus contract: he imagined that in such an event an R.I.B.A. *Form of Contract* with quantities would be adopted. The keynote of the Authors' Paper was the fact that they were describing the *civil engineering* aspect of a large building development; mention was made of the Resident and Assistant Resident Engineers. There were difficulties in identifying 'the Engineer' and 'the Resident Engineer' in an R.I.B.A. *Form of Contract*. The engineering content of buildings was increasing and the days were disappearing, particularly in the larger developments, when the architect's clerk of works was responsible for supervising all site operations but only within the very limited mandate that he derived from the R.I.B.A. Form.

122. Could the Authors say how those problems of legal terms of reference were resolved? Alternatively could they sketch the pattern of interlocking relations necessary, in their view, to accomplish the complete design, supervision and construction of a similar type of scheme, referring broadly to a suitable form of contract procedure?

The Authors, in replying to Mr Faber, said it had been pleasant to hear from another engineer who appeared to be working in exactly the same direction as themselves. They had followed the same line of thought as Mr Faber and had incorporated the waffle slabs on a large project with a column spacing of 20–25 ft with considerable success; they had also proposed this method for much larger spans on other work, notably hospitals. The waffle slab, the plain flat slab, the hollow tile floor and the B.R.C. hollow mould floor could all be classed as variations on a basic theme. They all had great economic advantages but each type had a specialized application so that one had to be fully aware of the limitations in order to obtain the best results from each one.

124. For example, the waffle slab was excellent for larger spans because the dead load had to be decreased, but it could not be used for buildings where a high fire-resistance standard was required. It could not be satisfactorily plastered and was often used with a suspended ceiling. In a building where fire-resistance requirements were low, the combination of suspended ceiling and waffle-slab floor was excellent. The services could easily be run anywhere within the ceiling void and because of the nature of the design, holes for conduits or pipes could be punched in the floor at any time during or after construction. This gave the services engineer great freedom.

125. Mr Little had commented on his experience in jetty construction but construction over water, such as jetty or bridge, was in an entirely different class from the construction described in the Paper, being nowhere near as stereotyped. In those conditions precast concrete came much more into its own. The difficulty of supporting formwork for in-situ concrete and allied work over water usually put up the cost of in-situ work considerably. There had been an enormous boom in recent years in the use of precast concrete in building, much of which was due to serious research, but there was also an element of fashion involved, and care must be exercised before accepting the general idea that precast concrete was always the right method. The Authors did not think that the conditions which were required for this to be true obtained yet, although it was generally thought that they might obtain in 10 years' time. Labour conditions were still such that the problems presented by a particular job had to be analysed very carefully before a decision could be taken on the structural medium.

126. Mr Walley had asked why the ICOS wall had not been incorporated as part of the permanent structure. The chief reason was that the depth of the wall had been about 30 ft. It was a cantilever wall with very high water and earth pressures and it required either a very large base in the form provided by the Authors or base fixity

by penetration which the strength of the soil did not allow. In a typical sheet-piled wall the problem of overturning would be overcome by tying the wall back at the top, but in this case it had not been possible to use a tie. However, the ICOS wall should be used wherever possible as Mr Walley had suggested. A classic example of this use was the Milan Underground where an ICOS wall had been put down each side of the street, a top slab cast, and the ground between excavated. The wall was finally supported at top and bottom by transverse slabs. In the case under discussion the ICOS wall had been used in substitution for bored piles. These were proposed instead of sheet piling because there was not the necessary headroom to drive the latter.

127. Dr Rowe had suggested that the use of the term 'membrane action' might not have been altogether accurate. On the other hand, the Authors would be interested to hear how to achieve two-way bending in a large slab without considerable membrane action taking place. Dr Rowe was correct in saying, when referring to membrane action in the two-way flat-beam system, that if membrane action could be fully utilized an even cheaper design could be achieved. In fact, the Authors had talked to Dr Wood of the Building Research Station about this, but it had not proved possible, in practice, to effect any economy over conventional design methods.

128. It would have been possible, as suggested by Dr Rowe, to make the central slab substantially thinner, but for the fire-resistance requirements, and they could probably have left the reinforcement out of this slab altogether and it would still have been satisfactory. Unfortunately, the by-laws and the local authorities had to be satisfied, and in the rush of preparing a design of this type, the time was short for calculations of this nature.

129. It had been suggested that the drop panel could have been removed from the flat-slab floor. Their immediate reaction to this was that if they had done so the floor would have slid down the columns. Special shear connectors might have obviated this, but the cost would have been greater than any other savings which would have been achieved. The code method, based on empirical studies, when applied to a flat-slab floor with drops, gave an extremely efficient and economical design and one which it was difficult to better by other means.

130. The problem of accommodating shear in a very thin slab was often met with, particularly in office blocks, and the form of shear connector for the office block devised by the Authors had been unsatisfactory from some points of view. There had been an absence of information and they felt that some thorough and extensive research into the problem of column-head shear connectors would be valuable. Until recently, very little had been available.

131. With reference to Mr Turner's query about the design method for the two-way flat-beam floor, the floors had been designed by the ordinary code methods. The two-way beam system had been designed by analysing the beams in the conventional way and the all-important part of the floor—the slab—had been designed by the code method for two-way spanning slabs. The results achieved had been cheaper than those achieved by any other way. The yield-line design method which gave results comparable in cost with the conventional flat-slab design had been tried, but the Authors felt that the conventional design was superior to the yield-line method to some extent in that the distribution of the steel corresponded more realistically to the structural action of the floor.

132. A question had been raised about the wind loading on the office block. Wind forces had been transmitted to ground via the floors acting as horizontal girders through the stair walls and lift shafts. No attempt had been made to carry these forces via the columns.

133. Mr Turner had asked about the value obtained for Young's modulus for the soil of the site. The value used had been determined from a hand-carved sample obtained with some difficulty and expense from a special pit sunk for the purpose. This method was considered to be a great deal more accurate than anything that

could be obtained from normal borings. To have obtained an equivalent sample from the Elephant and Castle site would have been prohibitively expensive and the value already obtained was accurate enough. Nevertheless, it would have been interesting to have had their own sample. The decision was a matter of economy.

134. Mr Little had asked about the calculations for the uplift of the tube tunnels. They had calculated what happened in the middle of the tube as well as at the end and it had been at the middle where the movement had been $\frac{1}{4}$ in. more than calculated. At the ends the calculations had been accurate. Mr Little had also dealt with water penetration through the walls of a graving dock. The mix used at the Elephant and Castle had been 1:6, which was rather lean. The Authors thought that this was about the right balance, providing enough cement to produce the least shrinkage in the walls without making the walls permeable. A rich mix would be impermeable but would shrink, and a lean mix would not shrink but would be permeable.

135. Many of the shops using basement storage sold television sets and similar goods. Some of these goods were stored against the wall where there were 12 in. of concrete with 19 ft of water on the other side. This was very different from a graving dock where walls tended to be thick and where a certain number of leaks could be tolerated. The Authors in this case were interested in reducing the permeability of the concrete to a minimum because of this specialized storage.

136. The problem of shrinkage cracks in the walls was often associated with the location of construction joints and the Authors had been interested in the discussion about whether or not to have water-bars. In this case, however, it had been shown that the shrinkage cracks tended to be almost independent of construction joints. The reinforcement was carried through to give an adequate distribution of the shrinkage forces through the joints and then the shrinkage cracks occurred at random. Some occurred quite close to the construction joints, and the existence of construction joints seemed to make little difference to whether or not the crack occurred.

137. If there were a water-bar in the joint, it might or might not make the joint more watertight, but would not stop shrinkage cracks opening up 2 ft away. Except in the case of a few water-bars which were adrift and leaked, most of the leakage in this case had come through small shrinkage cracks. It had, however, been extremely small, far less than in most ordinary basement construction which relied almost entirely on membrane waterproofing. The Contractors had elected to use the water-bars because they did not wish to be responsible for the water-tightness unless these were used. The specification had been drawn up so as to allow the Contractors to introduce variations if they considered these were necessary.

138. Some suggestion had been made about providing a 'possible contingencies' sum for grouting, but this produced a certain vagueness about responsibility. The R.I.B.A. *Form of Contract* made the contractor responsible for workmanship and the architect responsible for design, but when a crack occurred the difficulty was to decide whether it had been caused by workmanship or by design.

139. Regarding engineers' responsibility, it was true that the R.I.B.A. contract made certain provision. The engineer could be nominated by the architect as his agent. On this job the problem had been overcome because the civil engineering work had to be finally certified by the Architect on the Engineers' recommendations. If the Engineers had not recommended the work for certification there would have been some argument between the Architect and the Contractor, which neither of them wished, so the Engineers' position on those grounds alone was fairly strong. The position was odd and quite extraordinary legally, but by a certain amount of acceptance of these things on both sides of the industry, one was able to operate with only occasional friction.

140. The access to the job had been by a ramp which ran down to the basement leading to the underground truck tunnel which gave access on both sides into the

stores. The goods were brought up through lifts into the shops. At the end of the truck tunnel there was a turntable.

141. This was an unusual type of shopping centre and history showed that in a number of places in the world where such centres had been tried, lettings were rather slow, particularly above the ground floor.

FIG. 24: SHEAR REINFORCEMENTS

142. In replying to Mr Turner details of the shear reinforcements used were shown in Fig. 24. The design principles used, in the absence of reliable research information, had to be conservative, and were as follows:

- (a) the average shear stress as defined by C.P. 114 for flat slabs was limited to 130 lb/sq. in.;
- (b) the cranked bars were designed so as to support the whole of the shear;
- (c) the lower bars were intended to provide dowel action; and were also designed to carry the whole of the shear.

143. The Authors thanked Mr Power for his favourable comments on the Paper, and noted with interest his experience in prestressing flat-plate floors in New Zealand. They felt, however, from experience in England, that this technique would only be applicable to specialized projects. Because of the increased complexity of prestressing over normal reinforcement, a cost penalty was incurred which outweighed the saving due to the reduction in the quantity of materials.