Precast plate-units for roof construction

by

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Mr P. Lederer (Managing Director, Richard Costain (Construction) Ltd) said that the Paper described a particular application of prestressed precast-concrete plate construction and in presenting it the Author had suggested that its future held interesting possibilities. The discussion might change their views a little, but Mr Lederer wished to emphasize the particular approach which the Author had tried to apply to his work in recent years.

49. Much had been said in the Institution and elsewhere about the need for collaboration between architects and engineers and the difficulties encountered. The importance of achieving harmony between them was well established and only if each brought his special experience, talents and training to bear was it likely that the building or other resulting structure would give satisfaction to its users.

50. Cost was also an element in user-satisfaction, and it was in this field, of the effect on cost of construction problems and their sensible solution, that the Author was contributing. He was trying to apply engineering methods and principles not only to the basic design, the working drawings and the correct use of materials, but to the constructional problems. This was important as there was a barrier there between designer and constructor which should be broken down.

51. Mr Lederer thought it was true that those who, like himself, were industrial members of the profession were much more at fault than the designers. By and large the designers used rigorous methods of analysing their work—mathematical approaches, model tests and so on—and they had every right to expect the constructor partner, very often the contractor, to make his proper contribution when it came to assessing the non-material resources, the resources of labour, plant and staff. Many engineers combined both talents, being designers and also, through personal experience, having a sense of construction problems, but generally it was the contractor who should be able to make an effective, up-to-date contribution to the design process, and it was there, he thought, that they were failing. They should marshal their information in a systematically-recorded manner on all the matters on which, as contractors, they held strong views—productivity of labour, mobility of labour, planning and phasing of operations, the effects of weather and so on. At present they were failing to produce information in this field which could be properly used by the design engineer. If they made progress in that direction on the contractors' side of the industry, considerable advances could be made in engineering design.

Mr A. F. Daldy (Engineer, Building Research Station) asked three questions about the type of construction described in the Paper. The plates were only 2 in. thick and they had two layers of reinforcement in them, so that there could not be very much cover over the reinforcement. His first question was to enquire what measures had to be adopted to waterproof the outside of the plates?

53. Secondly, the Paper dealt with the use of this type of roof for a bakery, which meant that the temperature inside the building would be a great deal higher than the temperature outside on a cold winter night, so that considerable condensation on the inside of the roof was to be expected. What steps were being taken to prevent corrosion from the under-side of the plates?

54. Thirdly, the spans were long, and with straight plates as distinct from the bent ones it looked as though there might be water lying in the valleys. That had been overcome with bent plates, but what was the minimum fall which the Author would recommend for the valleys to prevent the accumulation of rain water?

Mr H. G. Cousins (Consulting Engineer) congratulated the Author on his Paper, which was an excellent one and gave a very complete description of his work, with beautiful illustrations of the details which reflected the class of work with which he had been associated. Mr Cousins had been privileged to see some of the units used, and the accuracy which could be obtained with this sort of method was quite remarkable. Some of the details looked very complicated, but they could be simply controlled when the construction was carried out in the way described. It would not be so simple to carry out the details shown in the illustrations with work done in situ.

56. He was interested in the box beam which supported the V-frames on the bakery, which was similar to one which he had used at Blyth power station for a span of about 120 ft. The section which he had used was somewhat taller and a little narrower, so that it had a greater moment of inertia with less weight. He had used 4-in. thick walls. From the Paper it appeared that the walls for the bakery at Swansea were 6 in. thick which had surprised him a little, especially as the Author tended to go in for rather thinner ones. Mr Cousins had precast his beam in lengths of about 10 ft and then prestressed the units together in a manner very similar to that shown in the Paper. It had been surprising to find that at Swansea this had been entirely done in situ, and he wondered why the Author had not precast the sides and joined them together.

57. The Author was to be congratulated on being an engineer with a feeling for structures, a sort of faith in them. He had probably believed that these V-shaped structures would stand up and had then had to convince somebody that they would. He had remarked on the helpful attitude of the local authority, who believed that they would stand up, and he had produced some calculations which Mr Cousins felt might have followed the Author's feeling for the structure rather than preceded it.

58. Mr Cousins had found it interesting to go through the calculations in some detail. He had looked at the assumptions, which was always a good thing to do, and he noticed that the Author started by rotating his structure about the apex at the bottom. That had probably been done for the convenience of working out some calculations, but in working out the rotation due to the torsional action the Author had taken the torsional moment about the apex and used the torsional strength about its centre of gravity. Mr Cousins had thought that this was cheating and so he had checked it. He had thought that if he took moments about the centre of gravity rather than the apex he would be bound to get a worse result, but in fact, without quoting all the figures, the figure of 0.00272 radian went down to 0.002, with a corresponding increase in the stability found in § 41.

59. He wondered why the Author had rotated it about the apex at the bottom when it seemed that it should be rotated about the centre of gravity of the structure, so that he had done a second check, and again the rotation had been less and the stability even more. This had seemed too good to be true, and so he had cut out one leg of the V and applied the same force to the rotation of the one plate about its centre of gravity. That had proved this plate to be infinitely stiff, so that there seemed to be something wrong with the calculations and he thought he had caught the Author out completely.

60. The steps in the reasoning were shown in Fig. 28, from which it would be seen that in the case of the V-shaped unit the Author had assumed a compression force acting in one direction and a balancing tension force acting in the other, but if the rotation was about the centre of gravity both those forces would be acting in the same direction. With a single plate both those forces acting in the same direction against the side of the plate gave a horizontal deflexion of about 20 in. in this case, but the same forces applied to the V-section would give a horizontal movement of about 0.01 in., showing the difference between the stability of the V-shaped unit, which the Author had used, and the single section. Mr Cousins congratulated the Author on his engineering sense, because the thing worked.

FIG. 28

Mr D. Standen (Contract Supervisor, Richard Costain (Construction) Ltd) said he would like to describe the part which he had played during the construction of the folded-plate roof at the bakery at Swansea and to enlarge on the subject of stressing. They had used a single wire stressing jack and each wire had had to be pulled to an extension of $11\frac{1}{2}$ in. A point of interest was that the gauge readings supplied on the jack did not exactly correspond with the measured results, and therefore they used the measured results only and disregarded the reading on the jack.

62. With regard to safety, it was frightening to hear a stressing wire slip. Of the 4000 wires pulled on the contract there had been only two failures, not of the wire, but of the anchorage. To overcome this they had put mesh over the top of the stressing wires at strategic points, so that if a wire failed or an anchorage went, instead of the wire coiling up along the length of the 160-ft stressing-bed it remained in the horizontal plane.

63. The stressing heads could be detensioned by turning back four screws by hand. This had been rather a hard task and had taken anything up to 3 hours with 72 tons of stress on each anchor head, but they had soon devised a method of inserting two 50-ton jacks behind the anchorage head, so reducing the time for detensioning to 20 minutes.

64. For vibrating the thin plate they had first of all tried a poker vibrator, but that had tended to chatter off the steel. They had then used a road rotary vibrator, which tended to overcompact the concrete. They finally decided to use a *Kango* with a rubber attachment at the end, which vibrated the plate and gave a good measure of compaction. The time for concreting one of these plates was about an hour. Curing was by means of hessian, well watered, after the first two hours of concreting, and in frosty weather they also put a polythene-frame screen over the top of the hessian. Of the 180 plates cast during the contract they had had only one failure due to frost action. The compressive strength of the concrete was checked by the Schmidt gun. It was no use applying it to the top edge of the plate; the reading had to be taken in the 2-in. edge. Concrete strengths specified were 6500 lb/sq. in. before release; this was obtained in approximately 16 hours using 'Ciment Fondu'.

65. A typical day's work had been to pour the two plates which had been stressed the day before. In addition to this, the gang also had to cast the in-situ bottom joint of the V, and a further task for the crane crew had been the erection of one V. This constituted a one-day cycle.

66. The plant on the site included a 22RB mobile crane with a 40-ft stick, and they had been very lucky in securing from their plant division a German rocket trailer having a very fine suspension. Although the Author was unaware of this, Mr Standen had very nearly lost the first V. They had been trundling it to the point of erection when the trailer slipped down a hidden trench excavation; it had taken 2½ hours to jack the end of the V up 5 ft. He could therefore assure the Author that the V was a very strong structure even as a single unit.

67. The crane time had been about 1 hour to lift each plate and 1½ hours to erect the V, which meant a morning's work on V-plate erection, the rest of the time being spent on normal erection work. The labour force consisted of 16 men: six steel fixers, six labourers, a carpenter, a stresser, and a crane crew of two. They had done the bakery roof in 10 weeks, and after a short pause they did the van loading area in 10 weeks. It should be pointed out that no specialists had been required. The stresser was a locally-employed man who had been sent to one of their subsidiary companies for a week's course and who had been fully able to deal with the stressing problems which occurred during the contract.

68. The 85-ft test unit had proved quite easy to manufacture; the only difficulty had been in transporting it from Swansea to Bridgend, 20 miles on Welsh roads. At roundabouts where they wanted to turn right they had taken the shorter route and not the proper way round. At about 10 miles/h the unit started to oscillate along its length and speed had to be restricted to 8 miles/h, to the consternation of the Welsh road users.

69. At the beginning of the contract it had been suggested that the plates should be cast in the inclined position with a top shutter. In his opinion that was not a practical solution.

70. The site advantages of the flat plate roof construction were that apart from the stressing, no special labour was required; it was easy to control the work and to set up a construction programme, it fully utilized the labour force applied to the task, and because it was a repetitive task it was easy to bonus. These advantages led to good labour relations on the site, quick and efficient work, and a general atmosphere of control which he had felt it an honour to be associated with.

Mr M. Rees (Chief Engineer, Orlit Ltd) congratulated the Author and his firm on their enterprise and on the novelty of the folded-plate type of construction, which gave to the onlooker, to a greater extent, probably, than with any other structural form, a feeling of vigour and strength.

72. One of the chief novelties was the kind of two-stage composite construction employed, in so far as the manufacturing element was not the erection element, which was twice the size of the manufacturing element. This method was not entirely new, however: there was an American precedent which was referred to in the Proceedings of the American Concrete Institute in 1961. (The Russian follow-up had not been heard of yet!) The Americans cast twin plates on the flat with reinforcement connecting one to the other and then raised the common edge of the two plates to form a V and concreted the resulting apex or valley joint in the same way that the Author had done, though with what success it was difficult to tell.

73. The Swansea roof represented another step towards the use of very large precast elements. These large elements brought their own problems as well as their advantages. In the attempt to reduce the weight of the erection element the cross-sections were cut down to the finest possible limits. The question of cover had already been raised in the discussion, and he would add the question of the in-situ concrete covering the stressing anchorages, which, from the cross-sections, appeared to be very thinly covered indeed.

74. Probably the biggest problem in casting long, shallow slabs was that of maintaining an axial prestress. The chief offender in this context was usually the prestressing bed which, though it might be of the self-anchored type and have a length: depth ratio of 25:35, usually sank within a few months to take up a concave arc-like shape. One way of locating the wires centrally was to anchor them every 20 ft or so. That was easily done with shorter units with steel drilled profile plates at the stop-ends, but with longer units the only positive way of maintaining an axial prestress seemed to be to anchor the wires every 20 ft by means of crossheads spanning the width of the prestressing bed and above the level of the concrete.

75. It was not clear whether or not the edge of the plate as it came out was dead straight or not. Perhaps the Author would comment on the point. If it could be assumed that the plate as it came off the prestressing-bed was corrugated, was it necessary to anchor the plates down to the V-shaped cradle as they were laid on the cradle, in order to maintain the straight edge? Any misalignment of this edge would aggravate the buckling problem which had been mentioned, and manufacturing errors could increase the destabilizing torsions so making for buckling, to say nothing of the difficulties in fixing of patent glazing.

76. This method of construction using large precast elements did not in Mr Rees's view represent a general trend, and he proposed to refer to two separate instances in which use had been made of smaller precast elements. Before doing so, however, he wished to ask the Author two questions. First, did the very interesting cost chart which he had shown include the cost of the main beams or only the cost of the roof plates? Did it include the cost of the edge beams and the supporting columns? It seemed to be shown very clearly by this graph that for a factory of under 75 000 sq. ft the construction would be more economic with smaller units, and 75 000 sq. ft was some factory! Secondly, he would like the Author to elucidate the detail at the bearing of the precast plate.

77. Figs 29 and 30 showed two schemes which were both new developments and which were at present in the development stage. Strictly speaking they were folded membrane roofs rather than folded plate roofs. Fig. 29 showed the pan floor. The basic precast unit was 3 ft 4 in. square in plan and varied in depth from 10 to 12 or 14 in. depending on the overall span of the roof, which could be up to 55 ft. (The little squares were not windows but shear keys.) Erection was on a system of scaffold staging. The precast unit (shown shaded) varied in thickness from 1 in. at

the centre to $1\frac{1}{2}$ in. at the perimeter. The pan was unreinforced and was made by the vacuum process, partly for production reasons and partly to give high early strength, because people walked round on these precast units. A structural screed was then laid over the top and between the units to form beams between the units. The reinforcement was in two directions to form a two-way system of composite diagonal beams.

78. The system shown in Fig. 30 was also in the development stage and also dependent on scaffolding, either fixed or moving, depending on the size of the job. The cost of a roof of that sort was not so sensitive to the total area of the job as the Author's system seemed to be and was possibly more suitable for the smaller job than the Author's system. The basic precast unit was 8 ft in length and had cast into it the diaphragms shown. The system was a little different from the Author's in

FIG. 29: PAN FLOOR OF PRECAST ELEMENTS

so far as the transverse moments in the slab were not taken in 2 in. of reinforced concrete but in the diaphragms themselves, which were reasonably deep. The in-situ connexions made between the units allowed for bond lengths of steel projecting from the precast units. The main tensile reinforcement was either ordinary reinforcement or prestressing cable depending on the span, the size of roof and the constructional sequence. If the whole cross-section looked a little like the Turin Exhibition Hall it was not intended to be in imitation of Nervi, so much as the fact that Nervi's forms had a certain inevitability of logic about them.

FIG. 30: PLATE ROOF OF PRECAST ELEMENTS

FIG. 31(b): GREYHOUND STADIUM, POOLE: DETAILS AND SECTIONS

thing through 45° one would get an upstand beam, tied by a floor slab, the beam being 2 in. wide, 4 ft 2 in. high stressed to 1500 lb/sq. in. and giving a factor of safety of 195. The Code of Practice must be very conservative, or else there was something wrong somewhere. The Author stated that a test had been carried out but did not say that it had been successful. Presumably it was. If so, the result appeared surprising, and it would be most valuable if the Author would go into this intriguing matter further.

86. Mr Jampel then showed slides of the Greyhound Stadium at Poole, Dorset, which was constructed with small elements as described in § 5.

87. Fig. 31 showed cross-sections and a part longitudinal section. The whole structure consisted of precast elements except for the foundations and the columns. The first floor, terrace and staircases consisted of precast beams supporting slab elements. The folded roof, which cantilevered 26 ft was tied by post-tensioned beams, the rear one supporting a lower light-weight flat roof covering the betting offices. The whole of the first floor was enclosed to form betting halls, restaurant, toilets, etc.

FIG. 32: DETAILS OF TYPICAL ROOF PANEL

FIG. 33: ARRANGEMENT OF REINFORCEMENT

88. The roof was constructed of ribbed coffin-shaped units 2 in. thick with diamond-shaped units at the ridge and valley intersection forming permanent shuttering. The reinforcement was placed in the troughs formed by the intersecting units which were then concreted.

89. The roof was designed as a series of inclined Warren girders with common top and bottom chords.

90. Fig. 32 showed details of the coffin-shaped units.

91. Fig. 33 showed the arrangement of the reinforcement.

92. Fig. 34 showed the completed structure prior to felting.

93. The pattern staining visible in Fig. 34 was caused by the reinforcement. It disappeared after felting. The building was constructed in 1960 and the structural cost excluding foundations worked out at approximately 13s. 6d. per sq. ft of covered area.

FIG. 34: COMPLETED STADIUM, PRIOR TO FELTING

Mr Francis Walley (Superintending Engineer, Ministry of Works), commenting on the form of the plate, said he did not know whether or not the Author had been aware of the plate form of construction employed by Hoyer in France 20 years ago, using the system of prestressing flat slabs in two sets of directions for barges and boats for use on the Seine and also for the making of pontoons. He had carried the Author's idea further by folding up both ways to form complete boxes in the case of the pontoons and three sides for barges. That was the earliest form of prestressed plate construction of which Mr Walley knew.

95. He would like to ask whether or not prestressing was at all necessary in the type of construction described in the Paper. It had been said that the plates had been stressed and detensioned within 24 hours and it had been mentioned that the

weather was cold, so that presumably the strength of the concrete in that time was not very great compared with the 28-day strength, and also the transmission length in the wires must have been very large. He could imagine that it was a very convenient form of fixing the longitudinal steel, but it would be interesting to know whether or not pretensioning in its strict sense was necessary or desirable.

96. What was the type of contract? Was it a negotiated contract? Had complete freedom of design been given? It would be interesting to know how the cost compared with that of other methods of spanning large areas.

Mr R. J. M. Sutherland (Partner, A. J. & J. D. Harris) wished to clear up two points which he had not understood, though everyone else might have done so. The first was whether or not there was any bituminous material on the top surface. The Author had stated that the slabs were water-tight, but a bituminous surface had been mentioned. It would be interesting to know more about the technique.

98. The Author mentioned fire resistance. What period of fire resistance? In most roofs a definite period was not needed, but Mr Sutherland had recently known a local authority to ask for 2 hours' resistance in a roof. Had any fire tests been made?

99. If the fire resistance was adequate it seemed possible that this flat plate technique might be used for very heavily serviced floors, especially in buildings with central service cores with all the services running one way. The corrugated soffit would often be left exposed.

100. Like other speakers he would be very interested to know the cost. Looking merely at the concrete part, if the cost was 5s. per foot square of building plan that would mean about 20s. per foot cube of concrete. If it was possible to get prestressed concrete 2 in. thick for 20s./cu. ft, including reinforcement, prestressing and everything else, it was very impressive.

The following contribution was received in writing:

Mr J. I. Glanville (Lecturer in Civil Engineering, University of Canterbury, Christchurch, New Zealand) wrote that the Paper was of interest in its description of the effective use of 2 in. thick concrete over large spans, and of the construction and erection techniques involved.

102. Mr Glanville, however, had found it more economical to have the units cast in a V-shaped mould the sides of which could be raised or lowered to give the desired side slopes. The unit was thus ready for erection in one operation. Stripping from the mould was less critical and the continuous pour of concrete provided a trough less susceptible to leakage. On the other hand, although a good-quality stiff concrete could be placed on a slope, a flat pour was undoubtedly simpler. In Manitoba, where sub-zero temperatures during erection are not uncommon and field grouting inconvenient, intermittent welding of embedded light steel and subsequent caulking of the small gap ($\frac{1}{2}$ –1 in.) at the peaks had been used with success on occasion.

103. It might be of interest to the Author to learn that a number of units designed by Mr Glanville were in lightweight concrete. These units, 8 ft between peaks, $2\frac{1}{2}$ in. thick at the peaks to $3\frac{1}{2}$ in. at the trough, with underside slopes of 2:1 and with draped pretensioning steel spanned up to 62 ft with a snow load of 40 lb/sq. ft. Some shorter units when load-tested to failure behaved predictably. After a few years in service the lightweight concrete units showed no visible defects such as excessive deflexion or camber, or cracking.

104. Mr Glanville wondered if it was for architectural reasons that saw-toothed edge beams were used; for, as the Author had pointed out, the conventional diaphragm was not essential for this simple geometric shape. The elimination of the diaphragm left room for additional glazing.

105. The use of the 'deliberately bent plate' by the Author in his fabrication technique provided some interesting food for thought.

The Author, in reply, expressed his gratitude for the interest in the Paper which had been shown by the large number of questions asked and suggestions made.

107. Several speakers had raised the question of cover to the reinforcement in the V-plate units. The plates were 2 in. thick, the stressing steel was 0.276 in. in diameter and the lateral reinforcement was $\frac{1}{4}$ in. in diameter top and bottom. The three layers of reinforcement came to slightly over $\frac{3}{4}$ in. giving $\frac{7}{8}$ in. cover to the main steel and $\frac{1}{2}$ -in. cover to the secondary steel. The minimum cover to the post-tensioned anchorages was $\frac{3}{4}$ in. This was within the Codes of Practice, but only of course for internal conditions. In order to comply with the regulations of the Code it was therefore necessary to have a protective coating on the outer surfaces of the plates. This was achieved with two coats of bitumen on the van loading roof and three layers of roofing felt over the bakery.

108. Mr Daldy had asked about water-proofing. There were two types of roof. The roof of the bakery had valley joints which were plain reinforced and it was accepted that the joints would crack as permitted in the conventional reinforced-concrete theory. The felting on this part of the roof was therefore relied upon to make it watertight. The roof of the van-loading area was, on the other hand, post-tensioned. This had been done because they had been given permission to experiment and they wished to develop a watertight roof. A V-plate unit unpainted with bitumen and with a post-tensioned valley joint had been tested by blocking off its ends and filling it with water. This had been left for two years and there were no visible signs of any water having leaked through the plates or valley joints. Thus the bituminous coating was not required so far as the watertightness on the van-loading roof was concerned.

109. In answer to Mr Daldy's last question, the valley fall was 1 $\frac{1}{2}$ %.

110. The roof of the bakery was insulated by a $\frac{3}{4}$ -in. layer of expanded polystyrene placed between the roofing felt and the concrete. The Author has calculated the thermal transmittance or U value of the roof and found this to be 0.25, which was well within the 0.3 permitted by the Thermal Insulation Act of 1957. However, condensation had been anticipated on the underside of the bakery roof and it has been treated with an anti-condensation paint to prevent corrosion.

111. Mr Cousins had raised some interesting points relating to the design of the roof. The calculations for the box beams had taken into account the fact that the weight of the roof was applied eccentrically through saw-tooth projections onto the walls, and also that during erection there were times when there were units on one side and not on the other. The Author felt that the 6-in. thickness of the walls was justified when the above secondary effects were considered.

112. Mr Cousins had asked why the beam had not been precast. They had in a sense precast it. They had used a shutter which moved along, casting each section against its adjacent section. The Author has always considered that with conventional precast construction, one paid first for making the joints and then again for joining them up. It was therefore advantageous, wherever possible, to cast concrete against concrete so that not only was an accurate connexion made but also it was free of charge.

113. The Author was pleased that Mr Cousins' calculations had shown that the V-plate units were in fact more stable against buckling than his own calculations had shown. However, the centre of rotation of a section corresponded to the centre area of a section only when the section had two axes of symmetry. The centre of rotation of a section was about the centre of shear and for an angle section this was at the intersection of the two plates. The rotation therefore should be taken about this point and it could be seen that this has been correctly accounted for in the calculations given in the Paper.

114. In reply to Mr Jampel, it was true to say that the principles in the buckling analysis given in the Paper could be applied irrespective of the angle of inclination of the V-section. However, Mr Jampel was incorrect in comparing the results obtained in the Paper and, in particular the symmetrical case, to an L-beam since the conditions existing in this case were entirely different. First the loading was not applied on the axis of symmetry and secondly the plates did not act identically. By applying the same bending moment to the section, it could be shown that an L-beam having leaves 50-in. long and 2-in. thick was unstable, which was in accordance with the Code of Practice recommendations.

115. With regard to the calculations being pre- or post-construction, the set of calculations given in the Paper was post-construction. However, they had done similar calculations using a method of successive approximations at the early design stages and also calculations based on the Euler formula. The results of the calculated deflexions were later found to agree favourably with tests on a typical unit filled with water.

116. Messrs Rees, Jampel, Walley, and Sutherland had raised the question of the cost of the roof. It was unfortunately impossible for the Author to give a direct answer, since the cost was very dependent upon the varying requirements. However, the Author was able to say that the manufacturing cost of V-plates could be achieved at 5s./sq. ft and the total cost of manufacturing, assembling, and erecting at 11s./sq. ft. It would be noted that these costs did not include for supporting columns, edge beams, or the Contractor's profit. The Author considered that this type of construction was economical from 40 000 sq. ft, and that 75 000 sq. ft, which was the figure mentioned by Mr Rees, was approximately the roof area where minimum cost could be obtained under conditions of mass production. If a standard unit was made up which could be used for a number of different contracts a 75 000 sq. ft roof area was not unrealistic.

117. It had been interesting to learn of Mr Rees' and Mr Jampel's systems of roof construction. However, what the Author had tried to express in his Paper was that, up to a certain span, it was worthwhile to employ large-size units especially when speed of construction was in mind. When small elements were used he suggested that they should be very small, as they were quicker and easier to handle. He did not believe in the half-size elements for which a crane was needed, transport was awkward and temporary propping and staging was still necessary.

118. The Author was aware of the work done with folded plates in America and had seen it some years ago, but it appeared from advertisements in the A.C.I. Journal that this type of construction was started after the Swansea roof had in fact been completed.

119. With regard to the straightness of the stressing bed, the Author did not think that this was a major problem, since the variation of 1 in. on a length of 65 ft could easily be adjusted during the assembly in the cradle. Problems would arise if the variation was such that the wires projected out of the unit. They had had no experience of this happening. The edges of the plates for the V-units used at Swansea were straight, and although a slight camber was given to these units when *being assembled in the cradle to allow for creep deflexion, the resulting edges of the assembled units themselves had proved to be better than that normally experienced with an insitu construction of the same thickness and length.* On the other hand, the edges of the plates for the butterfly units were curved, but their self-weight was sufficient to hold their ends down in the cradle which itself had a significant curvature.

120. A hollow rubber bearing had been given to the V-plate units since they did not want any bending moment to be introduced from the plates into the valley joints. The rubber was lightly reinforced to prevent it squashing and it was found to work satisfactorily.

121. In reply to Mr Jampel, the Author apologized for an error in the Paper, which stated that the V-plate units could be glazed. W-plate units had been devised as a glazing version of the V-roof.

122. He had introduced the box beam in the van-loading area instead of spanning the 100 ft with a V-roof directly for a number of reasons. At a very early stage it had been suggested that the service ducts should run through the box beams into the bakery from the boiler house, which would have been a convenient way of doing it. Also they had wanted one type of unit for the entire roof to keep manufacturing costs to a minimum.

123. As had been stated in the Paper, there were two requirements affecting the size of the valley joint. One of the requirements was dependent upon the fact that the post-tensioning force for the valley was proportional to its area. Thus in determining the area of the valley joint a balance had to be made between a practical level of prestress and an adequate bending modulus for the section.

124. In reply to Mr Walley, the prestressing wires were detensioned when the concrete had reached 6000 lb/sq. in. The reason for applying prestress into the plates was primarily to accommodate the temporary conditions during erection. The secondary effect was to make the plates watertight. Using a different method of construction it would have been possible to avoid the use of prestressing but the system adopted was considered to be the most economical.

125. The contract, which included the design and construction, was what was usually referred to as a 'package deal'.

126. The fire resistance provided by the roof was approximately one hour, but no fire tests had been made, since this was not a requirement of the Local Authorities.

127. Mr Glanville had asked why the saw-tooth edge beams had been adopted. The answer to this was that the V-units were themselves stable and a diaphragm tie was not therefore required. The Author was most interested to learn of the work that Mr Glanville has been carrying out in New Zealand and thought that the method of construction was very ambitious.

CORRIGENDA

In the original Paper, pages 332-333: § 36: A factor of 144 to convert sq. ft to sq. in. should be inserted; § 41: The figure 0.0025 should read 0.025 as in § 40.