Paper No. 6253

Prestressed concrete cylindrical tanks †

by

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Discussion

Mr E. E. H. Bate (Chief Works Engineer, Ministry of Works) said that the comparisons of cost given in the Paper should be very useful.

85. He agreed that casting in complete horizontal rings had been found preferable to casting in vertical panels. It was possible, however, that the difficulty experienced with the first Windscale tank had been due to the gaps between panels being too narrow, making it difficult to prepare the surfaces.

86. The method of preparing joints in liquid-retaining structures was of great importance, and as it seemed reasonable, *prima facie*, to assume that the strongest joint would also be the most watertight, he gave the results of a series of tests on jointed beams.

87. The test specimens were 4-in. × 4-in. × 20-in. beams cast in two portions, with the end of the first portion treated in different ways. Sets of control beams cast in one piece were also made. The beams had been tested over a 16-in. span, the loads being applied at the one-third points, and the results were summarized in Fig. 34.

88. The types of joint preparation had been:—

(i) As struck. Stop-end removed 24 hours after casting and new concrete placed without any joint preparation.
(ii) Exposed aggregate. Stop-end removed 3 hours after casting and exposed surface lightly brushed with water at once to expose the aggregate. This had been to simulate the mist spray treatment described by the Author. The second half had been cast 7 days later.
(iii) Hacking. Stop-end removed after 24 hours and exposed surface lightly hacked with a chisel.

89. Each type of joint had been subjected to different treatments prior to casting the second half of the beam:—

(i) No treatment.
(ii) Joint dry. After 7 days curing in water the specimens had been dried in atmosphere for 7 days.
(iii) Joint wet. The joint had been brushed with a wet brush immediately before placing fresh concrete.
(iv) Cement mortar. A set of the exposed-aggregate type had been given the usual treatment, i.e. covered with a thin layer of cement slurry and ½ in. of cement mortar immediately before placing fresh concrete.

90. The majority of the beams with the exposed-aggregate type of joint had broken not at the joint but well away from it. The average strength of the exposed-aggregate surface-dry beams had been 98% of the corresponding unjointed control beams. Numerous other tests had been carried out on similar lines—some on full-size specimens 14 in. × 10 in. cross-section, and the results followed broadly the pattern shown in Fig. 34. The 14-in. × 10-in. beams had all been tested under single-point loading, and it was of interest that all the exposed-aggregate surface-dry specimens had broken away from the joint, while all those treated with cement mortar had failed at the joint.

† Proc. Instn civ. Engrs, vol. 9, p. 87 (Jan. 1958).

91. In § 45 the Author referred to joints being thoroughly hacked to expose the aggregate. Mr Bate did not like the term "hacking". It suggested a brutal attack, and that in fact was usually done. Hacking of set concrete with a pick or pneumatic tool could be dangerous, particularly if the aggregate was brittle, like flint.

92. On one occasion he had examined carefully random areas of a joint in a reservoir which had been hacked and had found that about 75% of the pieces of large aggregate ($1\frac{1}{2}$ in.) which were exposed were cracked. They could not be pulled away with the fingers, but many could be moved slightly. That was a poor start to making a watertight joint.

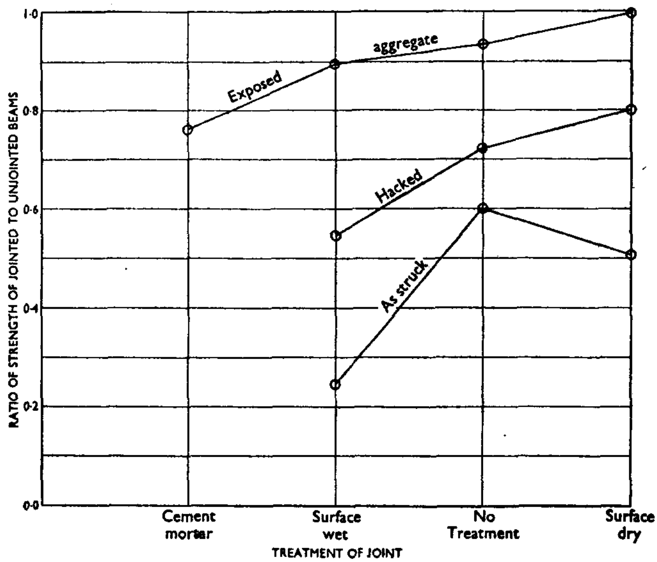

FIG. 34

93. If spraying while the concrete was green was not practicable in any case, it was possible to prepare the hardened surface in a satisfactory manner by a light treatment, but hacking was not the right term to employ for the process.

94. Cleanliness was an essential factor in the production of a tight joint, so that although all the tests appeared to indicate that the best joint was produced by placing the fresh concrete against a dry surface, he would prefer, for the present, to insist on a wash-down of that surface shortly before placing the new concrete.

Mr A. J. Harris (Consulting Engineer) said that the Author had not mentioned the base joint, which consisted of a solid slab of rubber, which offered the best chances of producing a sound watertight job and of relieving all restraint. If this was not used, one other factor which was of importance in considering the base was the shrinkage of the wall. Considering, for instance, the type of joint which the Author had mentioned in which the base was prestressed against a stop, whereby the wall became pin-ended, creep would continue in the wall and the vertical moment in the wall in consequence would be exaggerated with the passage of time. The question of creep—and of shrinkage—bedevilled most attempts at exact calculation of moments and shear forces in walls.

96. On the question of walls Mr Harris said that prestressed concrete had been in use for 20 years. During that period there had been surprisingly few cases of damage to prestressed concrete structures in service. Such few as had come to his notice had been through corrosion of the high-tensile steel and, with very few exceptions, all those cases of corrosion had occurred in cases where prestressed concrete had been stressed by wires left bare at the time of stressing and later covered with an in-situ concrete covering, either placed or sprayed, which was not itself prestressed. In consequence, there was every technical reason to have the circumferential stressing tendons cast into the thickness of the wall. There was an economic aspect of this. The cost of the screeding of the outside had to be set against the cost of the sheathing and possibly the extra expense of anchorage; in general the cost of the screed outweighed that of the sheathing.

97. Dealing with secondary reinforcement, he suggested that, normally, the secondary reinforcement was of no use whatever if one had a prestressing in both directions, except in so far as it helped to maintain the tendons in their right position during casting.

98. Mr Harris was concerned with the construction of a tank of about 1,200,000 gal capacity. This tank was not yet completed, but the tender figures for the conical roof were of interest. The design provided a series of precast radial ribs with precast slabs resting on them and screed over the top, with reinforcement. The cost of this roof as determined from several tenders was working out at between £3,000 and £4,500 which was considerably less than the £6,000 given in Fig. 32 for a tank of similar size. In view of its convenience in erection and particularly as compared with the difficulties of getting formwork out of the tank when one had completed an in-situ thin shell dome overhead, it appeared to him to be a very encouraging feature.

99. On the total cost of this particular tank, so far as he could tell from the lowest group of tender prices it seemed that he could confirm the Author's statement that the figures given in Fig. 32 were not over-optimistic.

Mr Francis Walley (Senior Structural Engineer, Structural Engineering Branch, Ministry of Works) said that in § 14 the Author stated that a load factor of the order of 2 on the steel was commonly demanded. This was taken from the Draft Code of Practice, and it was completely unreal for prestressed tanks. The concrete stress in a normal-size prestressed tank was relatively small because the thickness of the tank was dictated by practical considerations rather than stress considerations. For a prestress of about 800 lb/sq. in. and an assumed tensile strength of concrete of about 400 lb/sq. in., then the increase in steel stress before cracking occurred in the tank would only be about 5 times 12,000, which was an increase in stress of about 3 tons/sq. in. If this was exceeded all the load was taken on the wire, but at the same time all the water was released from the tank: the tank also was a unique structure in that it could not possibly be overloaded. He therefore saw no reasons for keeping the load factor of 2 on the prestressing wire.

101. The junction of the wall foot and the base slab was the most vulnerable part of a tank structure. What were the Author's views on doming the floor of the tank to produce an outward thrust when the tank was full? This would induce a positive stress at this joint, which the Author stated should be of the order of about at least 100 lb/sq. in. It would obviate any uncertainty which was certainly present in practice as to the amount of compression across that joint because of the friction between the wall of the tank and the floor. It seemed to him to be a means of using the water to create prestress.

102. In § 40 of the Paper, the effect of gunite was referred to as creating a stress in the wall of the tank. Some calculations had shown that for a 30-ft-high tank there was also a stress (in the gunite) from creep and shrinkage in the wall of the order of about 1,000 lb/sq. in.—quite an appreciable stress in the outside covering. He felt that this was part of the answer to the point raised by Mr Harris, that in fact one did get a prestress at least horizontally in the outside covering, and even when the tank was full

there was still a residual compression. That could be shown by calculations, but had anyone measured it either on pipes or in tanks?

103. The method of sealing the flexible juncture was referred to in § 48. One of the epoxy resins, with a hardener, suitably modifying it with one of the thiokols, might be better than those the Author mentioned. Provided the temperature was above 60°F, with a 20% addition of thiokol, the shear strength of that material was about 2,000 lb/sq. in.—it was sometimes referred to as the tensile shear strength—and it had an elongation of about 20%. It would appear to give all the flexibility required in that joint, particularly with alternating wet, hot, and cold conditions; and also, of course, the material from its nature would have all the tenacity required for keeping it on the concrete.

Mr G. D. Allison (Chief Engineer, Preload Ltd) said that the Author's references to continuous wrapping methods of circumferential prestressing appeared to him to give a predominant impression of risk of breakage of the wire. This was certainly not true of present-day practice, as would be evident from statistics of a current project.

105. Sixteen sludge-digestion tanks at Birmingham, with walls 82 ft int. dia. by 31 ft high, were being circumferentially prestressed by wire winding, the wire being stressed by drawing through a die. The quantity of wire required per wall was about 7 tons, and six walls had been completed to date. The average time per wall for the complete prestressing had been 1½ week and on five of the walls, taking nearly 35 tons of wire, no break of the wire had occurred. On the sixth wall, two breaks had occurred owing to an entangled coil of wire jamming up as it was being unwound. The resultant delay had been insignificant.

106. In the earlier days of wire-winding operations, it had been thought desirable to stop the machine every four or five circulations around a wall for the purpose of fixing clamps to the stressed wire to minimize loss in the event of breakage. Today the operation was continuous throughout the length of a standard commercial coil of wire, which, for a 100-ft-dia. tank, meant about twenty circulations of the wall, which could be accomplished in less than ½ hour.

107. As for determining the true state of stress of wire wound in this continuous way, there had now been developed a perfectly simple wire-tension gauge which could be applied to the wire very soon after winding, and the stress could be determined to an accuracy of ±5%. There were other methods of constructing a more elaborate tension-indicating device on the machine, but in the case of drawing through a die, if confidence was established in a die of a certain size and shape producing a reasonably constant stress in wire of constant quality and size, then there was no need to take over-frequent tests. Occasional checks were sufficient to demonstrate the state of stress.

108. The methods of applying circumferential prestress to a wall had a very important bearing on the design of the cover coat. Reference was made in the Paper to a cover coat, 2 in. thick, of sprayed mortar reinforced with mesh. Mr Harris had also mentioned the undesirability of any cover coat at all and Mr Walley had taken the opposite view, with which Mr Allison concurred. A properly constructed cover coat used in proper circumstances was perfect protection to high-tensile wire. The question was whether in all cases pneumatic mortar was the right thing to use. In the case of circumferential cables stressed by means of jacks, the Author mentioned that devices such as rollers and bridges were commonly used to relieve the very heavy friction between the cable and the wall. Unfortunately, such devices had contingent disadvantages. Numerous gaps were formed between the wires and the surface of the concrete wall. Such gaps could not be filled with mortar when the cover coat was constructed by a spraying technique and in fact they remained voids occupied by a loose mass of rebound sand. To achieve a minimum 1-in. cover over the lengths of wire farthest displaced from the wall surface it was necessary for the cover coat to be of about 2 in. overall thickness, which necessitated the incorporation of a reinforcing mesh to prevent shrinkage cracking.

109. However, because of the voids behind the wires it was doubtful if mortar spraying was suitable for cover-coat construction in this case. To achieve complete bonding and protection of the wire it would appear to be necessary to construct the cover coat in vibrated fine concrete.

110. Horizontal wires, applies by a continuous-winding technique, lay everywhere in intimate contact with the concrete wall surface and a mortar cover coat, 1¼ in. thick, gave 1-in. cover to the wire. The composite surface of wire and concrete was ideal foi construction of the cover coat by the gunite process, by means of which maximum bond of the mortar to the parent surface was produced. The bond strength of such a cover coat exceeded its inherent shrinkage force and no additional reinforcement was necessary to control shrinkage.

111. On the subject of temperature stresses in walls, Mr Allison fully agreed with the Author on the importance of giving full consideration to these effects. Certain crack failures which had been reported from time to time might have been due to insufficient consideration having been given to temperature and in particular to differential temperature conditions. He was puzzled by the figure in the Paper of 4·5 times the actual temperature gradient across the wall in degrees F, as giving the induced flexural stress. This figure might be appropriate for reinforced concrete but for prestressed concrete, designed to act as a homogeneous material without cracking, the numerical value of the induced flexural stress due to temperature gradient was one half of the product of the modulus of elasticity, the coefficient of linear expansion and the difference in temperature between the wall faces. The answer was 12 or 13 times the actual temperature gradient across the wall in degrees F.

112. The Author and his colleague, Mr Bate, had come down rather heavily in favour of horizontal construction joints. Mr Allison was equally heavily against them and in favour of vertical construction joints. His experience with them on a very large number of tanks of all sizes had been that there was no difficulty in construction, no leakage, and the maximum economy.

113. The apparently practical construction problem of whether to construct a wall in lifts with horizontal construction joints, or in full-height segments with only vertical construction joints, was in reality a question to be considered at the design stage, since the amount of vertical prestress required and the residual flexural tensile stress allowable were related to the nature and disposition of construction joints. The allowable residual tension would be the higher, and therefore the required vertical prestress, less, for full-height construction with no horizontal joints.

114. From the practical aspect, vertical joints had the advantage that it was easy to incorporate in them centrally placed waterbars to guard against any localized defects in the concrete at the joint. If the circumferential prestress had been adequately designed vertical joints were permanently under compression, and the waterbars could be simple and inexpensive, for they were not subject to distortion by movement at the joint.

115. For any normal wall the total length of vertical construction joint was much less than that of horizontal joint. Thus for the 1,000,000-gal tank example used by the Author, with 100-ft int. dia. and 20-ft. height of wall, construction in 5-ft lifts would result in 950 ft of horizontal construction joint, whereas full-height construction in eight segments resulted in only 160 ft of vertical construction joint.

116. Finally, on the subject of protection of circumferential wire, everyone was concerned about the possibility of corrosion should the cover coat, however constructed, fail in some way. It was of great interest to know that manufacturers saw no difficulty whatever in producing and using galvanized high-tensile steel wire, and Mr Allison, as a prestressing specialist, saw no difficulty in applying it. Galvanizing of circumferential wire would be an extremely valuable second line of defence against defective workmanship in the cover coat.

117. The Author had rightly emphasized the importance of achieving radial movement of the base of the wall to minimize vertical bending moment. Ideally, the wall

should be able to move freely inwards under the influence of circumferential prestressing, and subsequently move outwards equally freely when the tank was filled, the movements being repeated with emptying and filling of the tank. This ideal was closely approached by the type of wall base joint developed and patented by the Preload Company, U.S.A., and successfully used in America, Britain, and elsewhere. In this joint there is interposed between the wall base and the footing-ring surface a continuous rubber pad of rectangular cross-section so that radial movement of the wall base was accomplished by deformation of the rubber pad rather than sliding action of the wall on the footing. To ensure watertightness a flexible water-bar, of natural or synthetic rubber, or P.V.C., was also inserted in the joint.

118. The rubber pad had a minimum thickness of $\frac{5}{8}$ in. and a width normally $\frac{1}{4}$ to $\frac{1}{3}$ of the wall thickness. Manufacturers had supplied accurate data on the physical properties of such rubber pads so that their resistance to shear deformation under a given intensity of vertical load was known, and therefore the force restraining radial movement of a wall base was accurately known. This force was very much less with the rubber-pad joint than with any other practical type of sliding joint, and consequently vertical bending in the wall was substantially reduced. The negligible restraint offered by the rubber pad also had the important advantage that shrinkage and temperature movements during construction were not resisted, so that risk of cracking before prestressing was virtually eliminated.

Mr J. N. Lowe (Managing Director, Pre-Stressed Concrete Co. Ltd) agreed that the safety factor was not against load but against leakage.

120. Next he turned to the reference to residual stress in § 8. He strongly recommended that the residual stress should be at least 100 lb/sq. in. and, depending upon the material that the tank contained and also whether it was a sliding joint or a fixed joint at the base, even as high as 200 lb/sq. in., so that there could be certainty that all the imponderables of fixing and hinging bases were allowed for.

121. In § 40 there was reference to the protection of steel. He preferred steel to be inside the wall, and thought that the second-best method was to put the steel on the outside with a suitable protective sheathing, perhaps of polyvinyl chloride. In many circumstances he thought there was strong justification for a separate insulating wall outside the tank wall (the separating space could accommodate a walkway). The roof, he felt, should be flat or conical with a low rise/span ratio and should be insulated, but not covered by earth mounding.

122. In § 49 *et seq.*, there was reference to sliding wall tanks. It was preferable that the steel required to take the tank-full water load should take it straight away in the direction of and in the plane of that load (Fig. 35), so reducing the number of imponderables in design.

123. Mr Addison had referred to shrinkage calculations involving Young's modulus and temperature effects, but, Mr Lowe pointed out, someone had to decide on the value of the apparent Young's modulus for concrete. It might be anywhere between $1{\cdot}5 \times 10^6$ and 6×10^6, depending on the condition of loading and of unloading. Surely

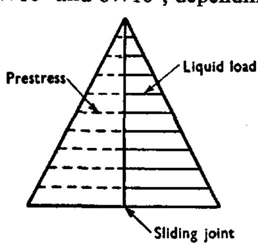

Fig. 35

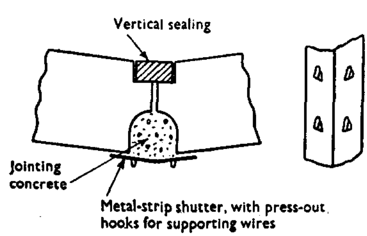

Fig. 36

it was better to adopt a sliding base and so put all one's troubles into one point, namely, the joint at the bottom. In the larger-diameter tanks which the Author had mentioned, sliding joints were almost a *sine qua non*.

124. The insulating wall had to be taken into account early in the design. Primarily the insulating wall was intended for general protection, but the temperature gradient between the contained liquid and the outside atmosphere could be inwards or outwards. If the insulating wall could be made free of in and out movements of the tank wall, then the temperature gradient across the wall could be reduced and so could the quantity of steel required.

125. The advantages of precasting (see § 65) were just as likely to apply to the construction of circular tanks as to any other form of construction. All the usual reasons for considering it were present: first, greater control of concrete quality; secondly, better control of concrete shape; and thirdly, the possible delaying effects of weather were diminished, since there was less in-situ work, and so on. This related particularly to circular tanks; such a tank could be made in precast staves, so that its shape was really polygonal; the angles would form high points which facilitated stressing (in conjunction with the metal pieces shown in Fig. 36). By using precasting in the factory, so ensuring some control over the concrete quality, there was every hope that weeping troubles could be restricted to the joints. Staves would also facilitate the incorporation of vertical prestress if required, and were equally amenable to either post- or pretensioning. The concrete could be tested before it left the factory.

126. The question of setting out was not difficult, and precasting would result in a simple set of centering, struts, etc.

127. The precasting technique also facilitated the possible requirement of lining. Tiles or bitumen, or any other lining material which would withstand the rigours of transporting, could be laid at the bottom of the shuttering instead of being applied in situ in the vertical in all weathers.

128. All the threads of this argument led back to the joint. Having ensured the best concreting by precasting, the next aim was to make the joint waterproof. Fig. 36 showed one way of doing this using vertical metal strips which could be made to serve as shutters and wire-supports and for reducing friction, in cases where outside stressing was to be used, the inside being sealed with vertical sealing strips.

129. Mr Walley had referred to epoxy resins. Mr Lowe had found that if they were applied in temperatures below 60°F, which were common in Britain, they took badly. Furthermore, if hot liquid was contained in the tank, it could have very serious effects on the epoxides over certain temperatures over a certain length of time.

130. Finally, Mr Lowe referred to costs. In tanks, as in any other commodity, particularly one which was in demand, there were two ways of getting the cost down. One was by better design and building methods, and the other was by reducing the specification. He was strongly against price-cutting by reducing the specification in any way, whether by cutting down residual stress, or taking the stress in the wires beyond the sensible point, or by reducing the safeguards against corrosion. He cited a £5-million chemical works kept going entirely by £100,000 worth of tanks. To reduce the specification in such a case was hardly sensible.

131. Mr Lowe had made an analysis of costs in connexion with a 1,000,000-gal tank, to decide what should be the diameter in relation to the height, how deep to dig it, and so on. Fig. 37 showed roughly the results of his analysis. It would be seen that the curves came together approximately where the tank diameter was about 130 ft.

132. His last comment on costs was that for very large tanks, flat roofs were distinctly cheaper if columns within the tank were permitted.

Mr A. W. Hill (Deputy Technical Director, Cement and Concrete Association) said that he thought there was an omission in the Paper regarding shrinkage during the concreting stage. The Author referred to the assessment of residual ring compression and limiting vertical flexural tension stresses at values in accordance with Code of Practice requirements; but there should be something in reserve in the design of both horizontal and vertical prestress in order to overcome the effects of shrinkage, since if cracks occurred they would have to be closed up at the outset.

134. The arguments for and against the provision of vertical joints were not likely to be conclusive. The essential condition with all joints was that the face should be treated to expose the aggregate, and with the horizontal joint this was very simple with a fine water spray. With horizontal joints, however, it was difficult to maintain a straight alignment for the vertical prestressing steel, and unless a substantial gas barrel was used, the friction losses were likely to be heavy. With vertical sections each cast unit could take up part of its shrinkage before the adjacent sections were concreted, so almost halving the total shrinkage.

135. If the base slab was very thin it was important that it should be kept thoroughly wet. In certain circumstances alternate wetting and drying could be more detrimental than no curing at all.

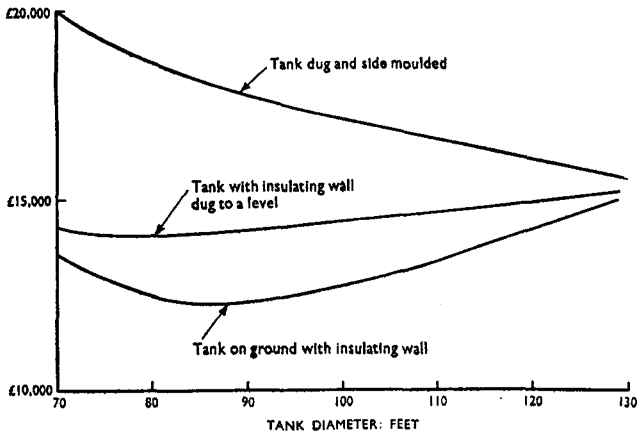

FIG. 37.—COST CURVES FOR 1,000,000-GAL TANK, SHOWING INFLUENCE OF DIAMETER/HEIGHT RATIO AND EFFECT OF DIGGING TO GIVE A REQUIRED LIQUID LEVEL

136. After about 5 years in service, the tank at Crawley mentioned in § 62 had no longer been required for water-retaining purposes, and had been handed over to the local farmer as a store. A doorway had been cut near the bottom of the tank, and from the resulting concrete and steel some specimens for test had been obtained after 5 years in use. No evidence of corrosion was obtained. Tensile tests on six samples of wire gave values ranging from 2·85 tons to 2·76 tons, representing ultimate stresses of from 112 to 110·2 tons/sq. in. These results were rather higher than those of the steel when it went into the job, due possibly to the extra work by the winding and stressing operations. From the concrete which had been cut out—which was entirely of pneumatic mortar—two cubes, of nominally 3 in. side, had been cut. These had had to be capped, and loads had been applied in one case perpendicularly and in the other case parallel to the bedding plane. The specimens had been tested wet. The compressive strength had been calculated on the area of the smaller of the two faces in contact and had been 5,850 lb/sq. in. for the specimen with the load applied perpendicular to the bedding plane and 8,150 lb/sq. in. for the specimen with the load parallel to the bedding plane. These results had at first seemed rather conflicting, but he had heard that other engineers had obtained similar experiences.

The following contributions were submitted in writing.

Dr K. Hajnal-Kónyi (Consulting Engineer) referred to § 14 and observed that, generally, adequate margin must be allowed for in design against the limit of serviceability and against failure by collapse. These two conditions had to be considered separately, since a factor of safety against reaching the limit of serviceability by itself did not mean anything with regard to failure and *vice versa*. The limit of serviceability might coincide with failure or a structure might have a high factor of safety against failure yet the limit of serviceability might be lower than the working load.

138. Both these extremes had to be avoided in practical design. In the first case the failure risk was increased since the structure might fail without any warning. In the second case it would not be suitable for doing the job for which it had been designed.

139. Prestressed concrete cylindrical tanks represented an exceptionally favourable case to which these general considerations did not apply. First, the deadweight did not enter into the effect of ring forces at all and was favourable for the vertical stresses. Secondly, the design load could never be exceeded. Thirdly, should the tank become unserviceable and leak the load would be automatically reduced and the tank would not fail. Indeed, failure was inconceivable and repair of leakage should always be possible at a much lower cost than a replacement of the whole structure. That was one of the advantages of prestressed concrete cylindrical tanks as against welded steel tanks, in which local failure under working load might occur and might have serious consequences.

140. It did not therefore seem reasonable to demand "a load factor of the order of 2 against failure", i.e. to restrict "the maximum tensile stress in the horizontal prestressing cables under service conditions" to "half the ultimate strength of the wire". If serviceability was secured, the consideration of safety against failure was entirely superfluous.

141. On the other hand, serviceability was affected not only by loading and unloading, but also by the loss of prestressing force due to friction, shrinkage and creep of the concrete, and relaxation of the wires, and by changes of temperature.

142. The effect of loading could be assessed with accuracy only if the boundary conditions at the junction of the wall and the base slab were clear. The ideal solution, requiring the minimum amount of vertical prestressing, would be a sliding joint without any friction. That was a purely hypothetical case which could not be realized. The practical choice was therefore between a hinged joint which was secured against sliding and a semi-sliding joint. If sliding was completely prevented the lower range of the wall near the joint could not be prestressed at all, since there was no stress without strain. A semi-sliding joint, with a pre-determined and limited movement equal to say half the theoretical movement without friction which could safely be achieved in spite of friction, appeared to be therefore a fair compromise and the relatively best solution.

143. The loss of prestress by friction along the tendon, particularly with tendons inside the wall, caused not only a variation of the ring forces as shown in Fig. 9, but also a deviation from the true cylindrical shape with subsequent variation of the vertical moments.

144. The greatest uncertainty in the design assumptions was the effect of shrinkage and changes of temperature. While the effect of a change of temperature of the liquid could be assessed with a reasonable degree of accuracy, the influence of the sun on the south and west sides in the case of exposed tanks, could hardly be estimated and set up a very complicated system of stresses which were unevenly distributed. It was therefore necessary to distinguish between mounded and unmounded tanks and provide more residual compression and less residual tension in the former than in the latter.

145. In order to obtain reliable numerical data for design it would be very desirable to construct a few tanks with strain gauges (say of the vibrating-wire type) built in at critical places on which readings would have to be taken under varying service conditions (tank empty, full, partially filled, etc.) for several years.

Mr Samuel McConnel (Consulting Engineer, Nairobi) had been responsible for the construction of many normal cylindrical tanks up to 1,000,000 gal capacity. Up to 100,000 gal capacity it had been found economical to construct the walls of brickwork, coursed masonry, or concrete blocks, dependent on local conditions, laying the necessary reinforcement in the horizontal joints and providing the normal type of sliding base with bitumen seal in a groove between the inside of the wall and the concrete floor. This type of tank required no shuttering and was useful where skilled labour was difficult to find. Although there must be considerable friction between the base of the wall and the floor which was coated with bitumen he had never had any trouble with cracks in the wall due to a restraining bending moment. It was essential that the bitumen joint should have a priming coat that would adhere to the concrete surface and the flexible bitumen seal should completely adhere to the priming coat.

147. Most engineers were convinced of the economy of the type of tanks described up to at least 8,000,000 gal capacity, if site conditions were favourable. Whether that was correct for the smaller sizes was doubtful, for a 750,000-gal covered circular reservoir recently constructed by Mr McConnel in Kenya had cost £15,000, which appeared to be the cost of a comparable prestressed tank in Great Britain according to Fig. 32.

148. Mr McConnel drew attention to the construction by the Kenya P.W.D. of three reservoirs of this type at Mombasa. There were three of 6,000,000 gal capacity, each of which cost £55,000 and were provided with concrete roofs. In view of the much increased costs at Mombasa that seemed to be a notable achievement—probably accounted for in part by the fact that the floor was dished to a much greater extent than shown in any of the Author's illustrations. Many service reservoirs in the United States and Australia had been built with a flat base, then 1-in-2 slopes to just below ground level, then a small berm, and then vertical walls. This led to very great economy in walling at the sacrifice of some additional roofing and flooring. Mr McConnel's attention had been drawn to this by the late Allen Hazen, and the method of construction did not always appear to receive the attention it merited in the construction of large cylindrical and rectangular tanks. Owing to the cost of roof shuttering, domed and toric vaults were by no means popular and consideration should be given to flat prestressed precast concrete roofs which could be laid at great speed with little or no shuttering or scaffolding; the columns could also be precast on the reservoir floor as had already been done in many cases.

149. The Author had very ably summarized the economic advantage of this type of construction, but it was probable that the ultimate development would be the use of precast panels as used in the 7½-million-gal tanks at Sasolburg, South Africa. A detailed description of these had been given by Doanides.[5]

150. Mr McConnel had seen many elevated tanks, including hundreds of the Intze type in all parts of France, but had never seen them protected against extremes of temperature with cork or lightweight concrete as described in § 68. With the advent of prestressing it was unlikely that any more elevated cylindrical steel tanks with an outside reinforced concrete wall would be built. Such tanks were not uncommon in the British Isles, but were practically unknown elsewhere.

Mr D. H. Lacey (Director, Kottler & Heron, Ltd, civil engineering contractors) remarked that there was insufficient emphasis in the Paper on recent improvements in design, materials, and construction, as applied to prestressed concrete generally. The faults mentioned by the Author had been eradicated quite consistently. In particular, difficulties of making the basal joint leakproof had been removed by the use of the arrangement shown in Fig. 30 which was in regular use and was proving completely effective. Modifications made in the manufacture of steel wire for die drawing had contributed to the elimination of all but the very occasional break when stressing with the external wrapping machine. Furthermore the regular use of fresh wire reduced the risk of breakage due to rust. The cost of wire lost through breakages was negligible.

152. On the question of horizontal stress actually present in the wire or cables,

[5] P. J. Doanides, "Some notes on precast prestressed tanks". Trans. S. Afr. Instn civ. Engrs, vol. 5 (1955), p. 207.

referred to in § 37, with external wrapping, however applied, it was now practicable and in fact regular practice to measure the stress during stressing and on completion. On the contrary if the wires or cables were in ducts that was virtually impossible. An assessment of the stress in ducts could be made by: (a) measuring the elongation of the wire at the jack, any calculation being dependent on the E-value of the steel, which was variable; (b) readings on the jack, which were subject to error; and (c) calculations based only on empirical formulae. Furthermore a wire in a duct might break, throwing a proportionately greater load on remaining wires. With external wrapping therefore an engineer was in a far better position to know how the structure was behaving.

153. The Author had stated (§ 1) that leakage, where it occurred arose more frequently from defects in detail or workmanship than from the direct result of inadequate design. The simplicity of the prestressed concrete tank, a large proportion of which was constructed in virtually mass concrete, and particularly when it was externally wrapped by machine and the only grout tubes were straight, promoted the construction of a watertight and more reliable finished structure compared with one built in reinforced concrete where the risk of faults due to inferior workmanship and/or design might be greater. The workmanship of a prestressed tank was tested during construction to a greater load than could ever be placed on it under working conditions. There was, therefore, an excellent opportunity of assessing its quality before ever it was put into service.

Mr F. C. Greenfield (Senior Engineering Assistant, Messrs Brandt & O'Dell, Consulting Engineers) observed that the Paper would be especially welcome to water engineers who, though they might not possess the specialized knowledge of a prestressed concrete designer, must nevertheless assume the overall responsibility for their reservoirs and water towers. This required them to keep abreast of developments in any new material which might, in certain instances, possess advantages over traditional ones. There seemed little doubt that the circular prestressed tank had come to stay and with the apparent success of such a variety of examples mentioned by the Author, it would have to be given full consideration in future. Certainly, in these times of unstable steel delivery, the quantities involved compared very favourably with those in a conventionally reinforced structure, and it was undoubtedly a more economical method of solving the problem than the grotesque mass concrete structure to which engineers had often been forced to return in recent years.

155. Apart from other considerations, every encouragement should be given to a process which could minimize or even eliminate one or more sources of tension in a concrete water-retaining structure. Would the Author say if complete elimination of flexural and direct tension was ever a practical possibility? Referring to § 12, could it be generally accepted that cracks caused by prestressing would subsequently completely close on filling with the liquid? Also, if mild-steel reinforcement was used to resist the tension caused by ring prestressing, should it not be regarded as a "structural" measure rather than a "cracking" precaution? Should the permissible stress in the mild steel be limited to approximately 150 lb/sq. in. times the modular ratio? The Author had mentioned a load factor for design but no factor of safety against cracking.

156. A further advantage of a prestressed tank was the greater practicable depth of liquid, depths of up to about 40 ft apparently being quite common. It would be interesting to know if this figure had ever been approached in reinforced concrete. Disadvantages seemed to be: the difficulty of forming an earth mound on a shell roof, which was particularly affected by solar heat; the difficulties of providing access, vent pipes, and glazed openings (for occasional light) in the shell roof; and the inconvenience of having to take pipes through the floor (for example, the inlet pipe would probably have to have a low point which would silt up unless drained).

157. Would the Author say if it was advisable to design the roof of a prestressed concrete tank as a "water retaining" member in view of condensation on the underside? Perhaps that was particularly important with a shell roof in view of its sensitiveness to temperature changes.

158. It was interesting to note in the examples that the largest (9,000,000-gal capacity tanks at Sasolburg, S. Africa) had been constructed of precast concrete units (were these successful?) and also that a water tower of 2,000,000-gal capacity had been built at Dallas, Texas. (Surely this has no counterpart in reinforced concrete.)

159. The Author did not seem to have mentioned curing, which was such a vital aspect of reinforced concrete water-retaining structures and presumably of prestressed tanks. Were any of the examples asphalted? One wondered if the asphalt lining was now out of date in any case.

160. Could the Author provide a cost/capacity curve for a traditional tank?

161. The New Code of Practice would no doubt be awaited eagerly by all concerned with liquid retaining structures.

Mr A. H. Beckett (Partner, Sir Bruce White, Wolfe Barry & Partners, Consulting Engineers) referred to the section on economics, which offered a means of quickly discovering the cheapest form of construction for a given tank capacity. The construction of the tank at Dallas, Texas, whereby the walls were seated on a rubber ring seemed particularly attractive from the design point of view, since that form of construction neatly side-tracked the controversial question of load distribution between sides and bottom. Could the Author say if the system had proved trouble-free in service and how frequently the joint was subjected to movement by filling and emptying the tank?

163. The Author had stated that in some cases mild-steel reinforcement, not prestressed, was used to supplement the high-tensile prestressed steel. Generally concrete shrank when curing and that set up compression in the mild steel which had to be balanced against internal tensile stresses in the concrete. The higher the percentage of mild steel the greater was the concrete tensile stress. Before the mild steel could act in tension, as presumably the designers intended, the tensile stresses in the concrete would be increased beyond the ultimate, when cracks would develop, making a leaky structure. Prestressing reinforcement reversed those internal stresses and offered a complete answer to the shrinkage problem, which might well be the reason why prestressed concrete tanks did not leak. Could the Author say if the partial introduction of mild steel had, in his experience, been a cause of leakage?

Mr Creasy, in reply, said that it was gratifying to record that the discussion had ranged over so many aspects of construction and design; and equally important to record that opinion seemed generally divided on the more controversial questions.

165. The pros and cons of casting the wall with vertical or horizontal joints had been hotly debated. In the Author's opinion these formed the most likely potential cause of leakage and on this account the horizontal joint was to be favoured since this was very much the easier to make in a sound manner.

166. Likewise, the objection to the precast-stave form of construction appeared to lie in the increased number of site joints to be made, all of which were a potential source of weakness.

167. Reference had been made to the forming of the joint between wall and base slab by means of a continuous rubber ring as shown in Fig. 30. This form of detail was patented and used in conjunction with one particular system of construction which tended to withdraw the continuous pad from the widest application. It was understood that the pad was somewhat expensive but it was offset in cost by the reduction in vertical prestress. This latter was required in part to deal with the effect of partial winding and could not with safety be entirely eliminated. Tanks constructed in this manner had proved satisfactory in service.

168. There seemed to be considerable advantages in making a semi-sliding joint of the type recommended in the Paper whereby the wall was drawn into a predetermined mid-position and pinned. The base of the wall was then prestressed and the vertical moments were comparable with that required in any case for the end winding effect. For the storage of cold water the mastic seal was quite satisfactory. For hot liquids the

seal must be elastic and strongly adhere to the concrete. The polysulphide rubbers had been found very satisfactory for this purpose.

169. The semi-sliding type of joint had the advantage in design of eliminating many of the imponderables. In other cases the design must be based on probable envelope conditions and over-exact refinements were not justified having regard to the many factors of uncertainty involved.

170. Where the cables were placed inside the wall they were more easily protected but construction would be somewhat more troublesome than if placed outside the wall. The opposing claims might perhaps be rationalized by placing the cables outside the wall in normal atmosphere and inside the wall where the atmosphere was specially malignant, as in heavy industrial areas or near the sea. Where the wire was placed outside the wall a covering of pneumatic mortar had proved satisfactory in all normal atmospheres. Where polyvinyl sheathing was used as a protection for the cables, the sheathing tore when the wires were tensioned.

171. The cost of the formwork for an in-situ roof could be considerably reduced by initially casting and supporting the crown and suitable intermediate rings, using a moving form to cast the in-filling panels. Precast rib-and-slab roof was an attractive alternative solution.

172. The initial and long-term measurement of stresses in the wire and in the concrete was an exercise well worth trying, but such field measurements tended generally to give unsatisfactory results.

173. The Author was pleased to note the improvements in continuous winding operations. The wire was tensioned by drawing through a die and it was not clear how galvanized wire could be successfully used in such a case.

174. The residual tension of 150 lb/sq. in. on the inside was chosen as the near practical limit of strength in tension of the high-quality concrete used. It was not anticipated that the concrete would crack at this stress. Additional prestress could, of course, be induced for any case in which this figure was thought to be too high. Mild-steel reinforcement was used at the foot of the wall where the induced moment was relatively small and at a low stress in tension.

175. There seemed to be no justification for designing the roof as a water-retaining member. The cracking was not generally so severe as to allow the condensation to cause excessive corrosion and in special cases suitable protection could be cheaply provided to the underside.

176. In respect of Fig. 35, it should be noted, as explained in § 9, that the horizontal prestress provided must be directly equal to the ring tension of the liquid load on the basis of complete freedom of the wall-foot, irrespective of the actual conditions of base restraint.

177. The Author agreed that the water tank was an example of a structure where the actual load in practice could exceed the design load by only a small margin and endorsed the recommendation that the load factor could be safely reduced to 1·5.

178. The cost figures quoted in the Paper were for unmounded ground-level tanks and approximately 20% was added to the net cost of the prestressed concrete structure to include for site stripping, excavation, contract preliminaries, and trimming for fittings. On the same basis the cost of the 4-million-gallon open tank should be increased to £35,000. The cost of a 4-million-gallon closed tank with fixed wall-foot would be increased by approximately 10% as compared with a tank having a sliding foot. These costs were lower than those which had been examined for traditional reinforced concrete tanks.

179. The storage tank must be more than merely structurally stable and even minor cracking was a measure of failure. This explained the importance of workmanship in the construction of the tanks, including the quality of the formwork and the care required in placing and curing the concrete, with major emphasis on the preparation of the joints. Few tank structures ever failed from structural instability, but failures could follow from defects of workmanship. This placed all the greater responsibility on the designer to provide a structure of form and detail which assisted a successful construction.