Paper No. 6224

# Water supply for the Yeovil district (Sutton Bingham scheme)†

by

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and

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*Discussion*

**Mr G. M. Binnie** (Partner, Messrs Binnie, Deacon & Gourley, Consulting Engineers, London) said that the measures adopted by the Authors to secure watertightness of the reservoir basin had obviously been completely successful, but some of them were rather unusual. He wished to compare them with two examples where, under different conditions, other measures had been adopted.

159. In § 43, the Paper described how, on the left bank, a grout curtain approximately 550 ft long had been constructed, and how this curtain had been reinforced over part of its length by a second curtain 280 ft long. A similar double curtain appeared also to have been constructed over a short length on the right bank. The distance between the two curtains was not given in the Paper, but from Fig. 3, Plate 1, they appeared to be about 8 ft apart.

160. If the grouting materials used for the construction of a grout curtain were not strong enough to prevent erosion occurring in the curtain as a result of internal seepage under the head of the reservoir, there was a case for deliberately making a grout curtain thicker. This applied particularly to the clay-cement grouts often used by French engineers.

161. For a curtain constructed with cement grout as used by the Authors, however, one would expect the grout curtain to be amply strong enough to resist erosion under the reservoir head.

162. For a cement-grout curtain, a single line with holes at, say, 8-ft centres would, in Mr Binnie's opinion, probably be more effective than two lines 8 ft apart with holes at 16-ft centres for watertightness; and while the drilling costs for both alternatives would be much the same, the single line would probably require less cement. From the point of view of both watertightness and economy, therefore, it seemed best when cement was used to keep grout curtains as thin as possible and to concentrate the holes in one line. Why had the Authors used a double curtain?

163. The orthodox solution was to extend both curtains on the axis of a dam far enough into the hillside until either watertight rock was reached or, in exceptional cases, until the length of travel round the end of a cut-off curtain was considered long enough to prevent serious leakage occurring.

164. While the Authors had taken the precaution of constructing orthodox grout curtains some distance into the valley sides at each end, for dealing with the Cornbrash limestone exposed to the reservoir on the left bank, they had constructed what was almost their main grout curtain in an upstream direction where the depths to the base of the Cornbrash limestone were shallower.

† Proc. Instn civ. Engrs, vol. 8, p. 71 (Sept. 1957).

165. The Authors had given an example where the best solution was undoubtedly to construct a cut-off curtain in an upstream direction; but each case had to be considered in relation to the geological conditions prevailing at the site. Sometimes it was not a case of considering merely the watertightness of the reservoir basin, but also the ground-water-tables in the valley sides, when deciding on the best alignment for a grout curtain.

166. As a result of impounding in a reservoir, the water-tables in the valley sides were raised and part of the ground water usually emerged on the downstream side of the dam. This tended to saturate the ground adjacent to one or other flank of the dam and in exceptional cases—for example, if the valley sides were steep and there was a clay material—this could conceivably cause a dangerous slip at one or other of the flanks.

167. After impounding in the recently completed Usk reservoir, for example, both visual inspection and the various pore-pressure gauges had indicated that the dam was watertight, but the ground adjacent to one flank became saturated to an undesirable extent. This had been attributed to ground-water. The valley slope was relatively flat and probably stable in any case, but as a precautionary measure the cut-off curtain had been extended in a downstream direction with a drain above it so as to divert the ground-water clear of the embankment.

168. Another example of a grout curtain aligned in a downstream direction was the right-bank curtain of the Dokan dam being constructed in limestone and dolomite formation in Iraq. Fortunately the dolomite, which was the lower formation, was relatively impermeable at varying depths below the surface and there had been no great difficulty in reaching impermeable rock on the underside of the curtain, but however far and in whatever direction the curtain might have been extended it would have been virtually impossible, except at very great expense, to reach watertight rock at the end of the curtain. In that case also, therefore, it had been decided to extend the grout curtain a sufficient distance downstream to protect the foundations of the arch dam against the effects of leakage and to ensure that any leakage that occurred would emerge clear of the dam foundations.

169. Depending on the circumstances in each case, therefore, the best alignment for a grout curtain could be in almost any direction.

**Mr L. E. Prosser** (The British Hydromechanics Research Association) was particularly interested in the full-scale confirmation of the model tests for predicting the capacity of the spillway. Fig. 19 showed the model results and the confirmatory point obtained from the 14-in. flood-level observation. The determination of the exact values of the coefficient of discharge might seem a rather trivial academic point, but Fig. 20 showed that it had considerable economic significance.

171. In Fig. 20 the profile of the spillway was shown by heavy black lines; there were two curves because the crest level of a portion of the weir had been lowered 6 inches to localize the discharge. Flood levels for normal and catastrophic floods were indicated and after allowing something extra for waves there was a total freeboard of about 6 ft.

172. The engineering design was undoubtedly sound but was this also true of the underlying philosophy? Absolute fool-proof safety had been provided at the expense of building a reservoir high enough and strong enough to hold about 800,000,000 gal of water but which in practice contained only 575,000,000 gal. Anything which would reduce the ullage of about 200,000,000 gal of air was worth serious study.

173. From the point of view of the conventional spillway it was possible to make small gains. The second profile in Fig. 20 was based on work published recently by Wagner[8] which would enable the top water level to be raised about 2 in. for the same

[8] W. E. Wagner, "Morning-glory shaft spillways, determination of pressure-controlled profiles". Proc. Amer. Soc. civ. Engrs, vol. 80. Separate No. 432. April, 1954.

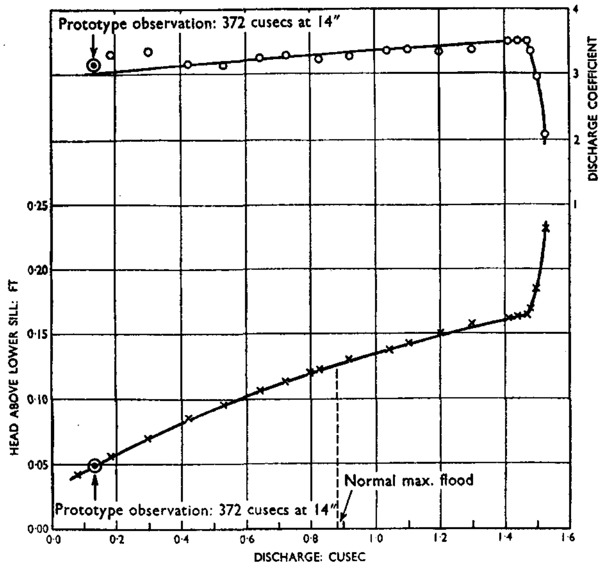


FIG. 19.—CALIBRATION OF OVERFLOW WEIR WITH CIRCULAR OUTLET TUNNEL. MODEL VALUES

flood discharge and maximum water level, representing several days' yield. Another possibility was to query the normally accepted 3-ft design head. This could be reduced as the two-thirds power of an increase in spillway diameter.

174. More important savings could be obtained by the use of syphons, gates, or valves and further research into the design of absolutely safe methods of flow control might give large economic gains.

**Dr H. Q. Golder** (Director, Soil Mechanics, Ltd) recalled that when the first recorded observations of pore pressures in Britain had been made at Knockendon in 1944, he had suggested, very tentatively, that excess pore-water pressure could be the explanation of an observed phenomenon. Later, in 1953 Dr Golder had observed[9]:—

"At the time that work had been carried out, little had been known about the effect of pore-water pressures. The famous case of the San Francisco—Oakland Bay Bridge, in which the pore-water pressure was demonstrated by the water level in a measuring tube standing above groundwater level, was of course well known. However, so far as Mr Golder was aware, the observations made at Knockendon were the first in Great Britain to reveal the water level standing above ground level.

"The first indication of that phenomenon had been a report from an intelligent, well trained, but incredulous foreman that he had left the boring tubes projecting 2 ft above ground level and, during the night, water had risen in them and overflowed. Although rain had fallen in the night that could not account for the quantity of water which had filled the tubes. The Author and Dr Golder immediately proceeded to the site to investigate the report. It had been observed that the foreman had, in fact, reported correctly. Measuring tubes of 3 inches diameter had therefore been inserted in the borehole and the observations reported had been commenced."

[9] J. A. Banks, "Problems in the design and construction of Knockendon Dam". Proc. Instn civ. Engrs, Pt I, vol. 1, p. 423 (July 1952); Discussion, Pt I, vol. 2, p. 199 (Mar. 1953).

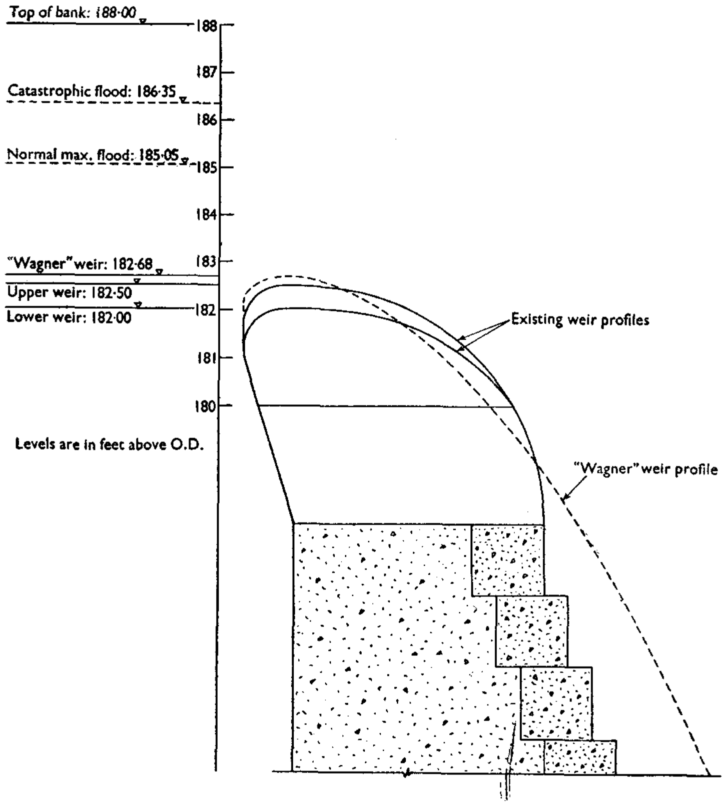


FIG. 20.—COMPARISON OF SPILLWAY PROFILES WITH THE WAGNER WEIR PROFILE

176. The approach at that time had been very tentative. After an interval of 8 years ideas had crystallized and Mr Banks had then written[10]:—

"Observations were made on the water level in the boreholes during the boring operations. In two of the boreholes water rose above ground level when the tubes were left projecting overnight, thus indicating that the water pressure in the pores of the bank material was greater than the static head of water above it. This suggested that the material had not consolidated under the overburden of bank material placed and it was decided to drive pore-pressure tubes so that changes in pore pressure could be observed and noted."

177. At that time, the observations were made simply by measuring the water level in what were then called "stand-pipes". In 1944 Dr Golder had written a report to Mr Banks in which he had said very tentatively:—

"These observations indicate that the hydrostatic pressure in the water in the voids of the bank material was greater than the static head of water above it. In other words, the material had not consolidated under the overburden of bank material. This is not surprising as the bank was placed relatively quickly. Theoretically, these observations could be used in conjunction with equilibrium shear tests on the material

[10] See p. 438 of reference 9.

to determine its existing strength. In the present case, however, it is probable that there is considerable variation in the water pressure conditions through the bank, and it would not be safe to base any calculations on the results of two observations."

178. The important point was that 10 years later, in the Paper now under discussion, the method had become so well established and so well accepted that the Authors merely referred to it, and did not explain it in detail.

179. Tribute should be paid to Dr Cooling and his colleagues, who since 1944 had been responsible, directly and indirectly, for the installation of seventeen sets of measurements on dams and banks, both in Britain and abroad. The apparatus was now much more complicated than a single stand-pipe.

180. Dr Golder then showed a series of lantern slides illustrating the placing of the apparatus at Sutton Bingham.

181. The installation of the pore-pressure cells was quite tricky and leaks had to be avoided. The polythene tubing used in the apparatus was now obtainable in lengths of up to 1,000 ft, but the makers had not so far produced it in longer lengths. It was, therefore, possible to place pressure cells 1,000 ft away from the measuring instruments, dig the trench, lay the polythene tube, and then back-fill over it. It was important to know before commencing back-filling that the installation was in working order.

182. Dr Golder said that his colleague, Mr T. G. Clarke, had given him the following quite simple but valuable practical tip. If, when placing the cell, the porous disk was covered with a rubber cover, a water pressure could be applied; the polythene tube could be filled with water and a pressure applied to it. The pressure could be read on the pressure gauge and it could be observed that it did not fall for a period of 2 hours. During this time the trench could be back-filled, but not over the tube until the last moment. When it was quite certain that the circuit was complete and watertight, the rubber cover could be taken off and back-filling completed over the pressure cell. It was then known that the system would work when the bank had been built above it.

**Mr J. H. Abrams** (Area Agent, Messrs Holland & Hannen and Cubitts, Ltd) referred to the placing of the 5-in. concrete protection apron on the 1:3, 1:4, and 1:5 slopes of the dam. Difficulty had been encountered in getting concrete on to these slopes which would give a surface when screeded and which would remain at that slope. The problem had been solved by putting down 4 in. of very dry concrete and making the top 1 in. of similar aggregate/cement ratio but omitting the coarse aggregate and wet enough to be screeded to a surface. Applied when the concrete was green, it had bonded very well and it had been possible to place the concrete and get a good screeded surface on the slopes, in this manner.

184. When planning the job, it had been intended to use a 6-in. concrete-pump to distribute the concrete over the site, but practical tests at the site had shown that it was impossible satisfactorily to pump concrete at the specified ratio of 1:7 by weight. It could, in fact, have been pumped, but the water/cement ratio required had been so high that the strength had been too low. The leanest mix, using local aggregates, which could be pumped without air entrainment was 1: 6·2, and this was uneconomical. The use of "Darex" as an air entrainment agent had proved useful and it had been found possible to reduce the water/cement ratio without reducing the pumpability; its use permitted the pumped mix to be increased to 1: 6·55.

185. The intended method of constructing the tunnel made use of a 12-ft-long steel travelling shutter, which meant that only one section could have been done at a time; but it had not been fast enough. The tunnel had been constructed by "hit and miss" methods. Each 10-ft section of tunnel constructed with the travelling shutter had been followed by a 10-ft gap, which had been later made up by using Acrow-span forms with two metal ribs shaped to the lining of the tunnel, the expanding Acrow forms being slipped in.

186. The Paper referred to the mechanical ramming of the puddle by mainly local labour from Somerset and Dorset. The personnel had not previously encountered clay

puddle. The gang of men detailed to puddle the trench had been provided with suitable boots, but in less than one day the boots were losing their soles.

187. The use of mechanical rammers had been a fortunate expedient, because they had given a fine compaction which could not have been relied upon from the men. The rammers had at first been a failure. When stood up on end, the rammer had buried itself in the clay. Then a harness had been formed by which each rammer had been held about 30° off the vertical, so that the foot did not go right into the clay, and this had acted very well. The Consultants had been somewhat alarmed at the outset, but tests proved that extremely good compaction had been obtained.

188. One pitfall had been in connexion with the pug mill, which had had a circular extrusion with a cheese-cutting wire across the centre to give two semi-circular sections. There was always a certain amount of difficulty in the early stages with a pug mill. The water content in the clay varied as it was dug and stored and the amount of water added at the mill greatly altered the mill's performance. If there was too much water, the whole lot churned round and would not come out; insufficient water caused a solid jam. Various experiments had been made with the extrusion unit. On one occasion when the centre cheese-cutting wire had been left off, the result had been a sausage-shaped circular section and not a semi-circular section. It had been possible to handle these pieces much more easily and to put them in the trench, and they had gone very well until the test had been taken at the end of the shift, when it had been found that the junction of the two circular units was packed with air. However much they had rammed, it had not been possible to get the air out. This section had of course to be removed.

**Mr Julius Kennard** (Edward Sandeman, Kennard & Partners, Consulting Engineers, London) challenged the Authors on the hydrological aspect of the Paper. It was stated in § 2 that "the lowest dry-weather flow recorded was 0·44 m.g.d. in 1946". This represented a flow of a little more than 0·1 cusec/1,000 acres, which was what one would expect as the minimum flow from such a gathering ground.

190. If the gathering ground had been fully reservoired on the basis of 3 consecutive dry years, the reliable yield would be of the order of 4½ m.g.d. for a catchment area of 7,470 acres, an average annual rainfall of 38 in., and a loss of 24 in. in a year of average rainfall. To get such a yield would require much greater storage than the net available capacity of 525,000,000 gal which the Authors had been able to provide. Consequently, there was what was commonly known as an under-reservoired catchment, and it was not so simple a matter to calculate the yield of such an under-reservoired catchment.

191. To arrive at the yield of a fully reservoired catchment was more or less a matter of arithmetic; the main problem was to determine the amount of storage required to enable that yield to be realized. With an under-reservoired catchment the problem was to assess the yield that could be relied upon with a relatively small amount of storage. The Paper gave the figure of 2¾ m.g.d., and Mr Kennard had worked backwards to try and find out how it had been derived. On the basis of a theoretical dry year and an average rainfall of 38 in., the rainfall in the driest year is likely to be 25·3 in. Deducting from that the evaporation losses in the driest year, amounting to 17·9 in., corresponding to a loss of 24 in. in a year of average rainfall, there is left 7·4 in. which could be regarded as the collectable run-off. Over the catchment area in question, that was equivalent to 1,250,000,000 gal.

192. On the assumption that during the drought period, during which the reservoir was constantly falling, only 10% of the year's run-off, equivalent to 125,000,000 gal, was available for distribution in addition to the 525,000,000 gal of storage, a total of 650,000,000 gal available for distribution during the drought period was arrived at. Taking this period as 8 months, i.e., 240 days, one arrived apparently at the Authors' yield figure of 2·71 m.g.d. This meant, however, that the run-off of 125,000,000 gal in 8 months was the equivalent of ½ m.g.d., which was about the same as the 0·44 m.g.d. experienced by the Authors in 1946 as the lowest dry-weather flow recorded. Mr

Kennard accepted this latter figure but not the average of ½ m.g.d. over 240 days. If one expected an average over 240 days of no more than 0·3 cusec/1,000 acres, which was still low, the run-off figure of 125,000,000 gal would be three times as much. There would therefore be 250,000,000 gal of additional water to distribute, and this would add a further 1 m.g.d. to the yield over the drought period of 240 days.

193. In that case, the reliable yield of the catchment area and reservoir was more of the order of 3½–3¾ m.g.d., and when the ½ m.g.d. compensation water was deducted there still remained no less than 3–3¼ m.g.d. for supply, which was substantially more than the figure of 2¼ m.g.d. for which the Authors had allowed. They were indeed lucky if the ½ m.g.d. compensation water had been agreed on that basis. Did they have further information concerning run-offs which would completely upset Mr Kennard's argument?

194. The question of grouting had already been fully covered. It appeared that from the quantities of grout which had been injected, however, the Cornbrash limestone seemed to have taken no more than the average of the other strata into which grout had been injected. Mr Kennard had reached that conclusion after trying to interpret the figures in the Paper. Did the Authors consider it reasonable and fair?

195. A figure of £2/sq. yd for the cost of the grout curtain had been given; it was a very valuable figure to have. It had been confirmed in other cases, although some engineers had found the cost of grout curtains to be higher in certain circumstances.

196. He congratulated the Authors on their courage in ignoring the Institution Flood Committee's Report. It was only right that one should have regard to lowland catchment areas and make some deduction from the figures given in the Flood Committee's Report, based on upland areas. What was the quantity of rainfall that caused the flooding which occurred in December 1956 referred to in the Paper?

**Mr A. D. M. Penman** (Soil Mechanics Division, Building Research Station) emphasized that the purpose of installing the pore-pressure apparatus at Sutton Bingham had been to check the stability of the dam during construction. Once the dam had been completed and the reservoir filled for the first time the further pore-pressure measurements would provide information about the condition of the fill and it was hoped that eventually the steady state could be observed.

198. It was fortunate that at Sutton Bingham two of the piezometer tips were in the upstream side of the dam, because in earlier work on other dams, the Building Research Station had not been allowed to carry the small leads for the tips through the puddle core. At Sutton Bingham a long loop had been formed in the four 5-mm-dia. polythene tubes where they passed through the puddle core, to give a long seal between the tubes and the puddle clay, and no leakage was anticipated.

199. The gauge house and piezometer apparatus had been removed before the reservoir was opened, and the Authors had kindly agreed to build a manhole round the ends of the tubes, where they came out of the bank, so that subsequent pore-pressure measurements could be made. Mr Penman had been to the dam during the past week to take readings from the tips, and the measured piezometric levels are shown at N in

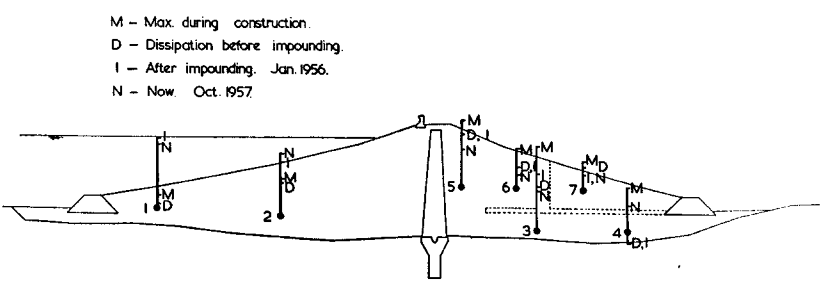


FIG. 21.—PORE PRESSURES

Fig. 21, together with the maximum value measured during construction (M), the level to which the pore-pressure had fallen immediately before impounding (D), and the value which was observed when the reservoir was filled (I).

200. The Paper stated that the most impervious material—the Forest Marble—had been placed nearest the puddle core, with the more pervious fill forming the toes of the dam. This construction was reflected in the pore-pressure changes which have occurred. At tip No. 1, the small constructional pressures had dissipated quickly and the piezometric level had followed the reservoir level closely during impounding: the current value agreed with the reservoir level which had been about 2 ft below T.W.L. for a few weeks. At tip No. 2, however, the constructional pressure was much greater and the dissipation less. The water pressure on the bank from the reservoir when it was filled caused an almost equal increase in piezometric level, which had continued to rise and would reach the reservoir level if the flow downstream was small, although it looked as if it would take a number of years before equilibrium was established.

201. On the downstream side, tip No. 5 had shown a fairly high constructional pressure, which was still dissipating slowly, suggesting that the material was fairly impervious. The other downstream tips had behaved in a similar way, although tip No. 3 had shown an increase in pressure during impounding. Tip No. 4 had caused the Authors some concern because the pore pressure had reached a maximum during construction and, during the waiting period, had fallen to a negative value. The Authors had commented on the over-consolidated nature of the Forest Marble, which might have caused swelling of the fill which was placed above tip No. 4 and developed a negative pore pressure. It could probably also be explained, however, from the fact that the fill was perhaps pervious and the low pore pressures might mean that the water had been able to drain to the bottom of the excavation at that time before the reservoir had been filled.

202. Now that the reservoir was full and a period of 18 months had elapsed in which some rain had fallen, the basin might have filled up and the pore-pressure reading of a week ago perhaps indicated that it was at about the level of the drain.

**Mr A. P. Lambert** (of John Mowlem & Co. Ltd) requested the Authors to expand the information given in §§ 58 and 59 concerning compaction of the bank during construction. It appeared from § 59 that the compaction of the clay material had been carried out entirely by the construction traffic, and § 58 stated that "Site tests on it taken from time to time proved that consolidation was satisfactory". How had the site tests been taken, and what was the criterion by which they had been judged to be satisfactory?

204. Mr Lambert asked this because he was interested in a dam which was just about to be built. It was to be 70 ft high—considerably higher than the one at Sutton Bingham. There was to be no puddle-core wall. The dam was being built more or less of clay and it was expected that the compaction obtained from the construction traffic alone would be inadequate. The consultants were, in fact, asking for extremely powerful and heavy compacting equipment to be provided.

205. Sections 58 and 59 of the Paper, if expanded a little, would enable one to assess what standard of compaction in clay material could be obtained simply from the excavating plant.

**Mr W. J. Dyer** (Chief Assistant Engineer, Messrs T. & C. Hawksley, Consulting Engineers, London) asked if there was any special joint between the tunnel and the overflow shaft. The concrete structure was very substantial and was apparently on a form of clay foundation. Was there any likelihood of a differential settlement, which would affect the overflow shaft and the tunnel, and had any steps been taken to deal with it?

**Mr R. T. A. Bolitho** (of Cornwall, Ontario, Canada) observed that should it be necessary to increase the storage in the reservoir, the over-flow of 3 ft required to discharge the design flood over the free weir crest might be utilized. If automatic gates were installed to maintain a constant top water level the capacity of the Sutton Bingham reservoir would be increased by more than 20% at a moderate cost. If two such gates were installed, each with the required maximum discharge when open, there need be no anxiety lest jamming of a gate lead to overtopping of the dam. One suitable type of gate had been described by Thorn[11] but others were available.

208. The Authors observed in § 58 that the Forest Marble clay produced filling material nearly as good as puddle clay; what was the permeability of this material? Had the Authors considered the elimination of the puddle clay in favour of a zoned-fill-type dam?

209. The exposed area of the permeable cornbrash limestone on the west bank appeared from Fig. 3 to be limited in extent; had the Authors considered sealing the surface with clay instead of the grouting carried out?

**Mr P. O. Wolf** (Reader in Hydrology, Dept of Civil Engineering, Imperial College of Science and Technology) observed that the Paper contained a wide range of information but the economy of the Authors' style had led to statements in the Paper which might be misunderstood by less experienced engineers—for example, the Floods Committee's estimate of a "normal maximum flood" of 4,500 cusecs, "and double this amount for a 'catastrophic' flood". It had to be understood that every engineer was asked by the Floods Committee to use his best judgement in every case (as in § 98 the Authors had shown that they had done at Sutton Bingham), and the Floods Committee never suggested in their Interim Report of 1933 that the multiplier 2 was exact, or even approaching adequacy in all cases.

211. Section 104, on the capacity of the bellmouth overflow, was related to that point. It was well known that the control of the discharge through such a device was at the circular weir crest until the water level inside the bellmouth had risen above the crest level and the control of the discharge passed to the tunnel downstream. Mr Wolf thought that the figures obtained from a perspex scale model would, in the present case, be near the actual discharge capacity, and that the maximum flood capacity of the arrangement was not much more than the 5,000 cusecs mentioned, with no appreciable increase even with a rise of several feet in the upstream water level.

**Mr Nikhileswar Sanyal** (Senior Assistant Professor of Civil Engineering, Government Engineering College, Jubbulpore, India) referred to §§ 83–87 and asked what type of clay (considered in terms of laboratory tests) the Authors considered ideal for puddle?

213. How had the upstream and downstream slopes of the dam (§ 22) been arrived at?

214. What was the yard-stick for permissible seepage (§ 30) from an impounding storage reservoir with an earth dam?

215. Mr Sanyal noted from Figs 8 and 9 that the pore pressure in the upstream embankment had risen considerably during impounding. Since rising of pore pressure was accompanied by reduction of shearing resistance, what precautions should be taken during impounding to enhance the dam's safety against failure?

216. Reading §§ 129 and 139 together, it could be seen that provision had been made to add activated carbon or lime for removal of taste and odour, presumably because the Authors anticipated algal growth in the impounding reservoir. Without a micro-biological study as to the species of algae growing in the reservoir how had it been decided that activated carbon or lime would meet the situation?

217. To what extent had the prechlorination been successful in controlling the growth of algae in settling tanks, in view of the fact that prechlorination had some inherent draw-backs for this type of work? Mr Sanyal advocated "charcoal blanketing" for control of algae in settling tanks.

[11] R. B. Thorn, "The design, fabrication, and erection of automatic radial sluice gates". Proc. Instn civ. Engrs, vol. 6, p. 126 (Jan. 1957).

218. Could the Authors suggest a relation between the rate of flow through a sand filter and the depth of the sand?

219. Had the Authors observed residual chlorine at the extreme ends of the distributing system? If it was very low, what provision had been made to maintain a suitable residual there? This was important in a district water-supply system, where long lengths of mains were involved.

**Mr J. A. Banks** (Partner, Babtie, Shaw and Morton, Consulting Engineers) observed that the fact that the water supplies in the district had been derived from so many springs and wells and that no large impounding reservoir had been constructed on the Forest Marble suggested to him that hitherto engineers had been doubtful of the feasibility of constructing a watertight reservoir in the prevailing geological conditions. It would seem that Sutton Bingham reservoir was an example of important engineering works of great benefit to a community, and one which the engineers had been able to undertake with confidence because of the advance in knowledge derived from soil mechanics.

221. The Authors had referred to the importance of not losing much water by leakage because the reservoir was a small one in a large gathering ground. Mr Banks suggested that the circumstances would have been much less favourable had it been a large reservoir in a small gathering ground, giving limited opportunity for making good loss by run-off. The fact that there had been very little water encountered in the cut-off trench might indicate that the substrata was less pervious than the geological situation would seem to suggest. This was supported by the intensity of the pressure grouting. From the particulars given in § 43 and by reference to Fig. 4 he assessed the intensity of grouting in other than local areas as about 12 lb/sq. ft of screened area. This was little more than one-fifth of the intensity which he had found was required in a sandstone formation and about one-third of what was needed in volcanic ash with whinstone intrusions.

222. Since the cut-off trench was almost entirely in clay, there might have been some justification for filling the trench with puddled clay. With clay available adjacent to the dam and no suitable aggregate on the site, the engineers must have been faced with a somewhat difficult decision, but they had been right in adopting concrete filling for the cut-off trench. Regarding the form of the concrete at the junction with the puddle clay, Mr Banks had adopted the keyed groove and the concrete "nose" with equal success. The former had particular application when the top of the concrete filling was within the trench, i.e., below the stripped surface level.

223. Referring to § 22 it was stated that triaxial tests had been made on the embankment material, and that the suitability of the material had been fully confirmed. However, the decision to adopt slopes as flat as 1 in 6 for an embankment 44 ft high suggested that the material had been less favourable than might have been desired. Could the Authors give particulars of the index properties, compaction characteristics, and mechanical analyses of the materials used? It was remarked in § 58 that consolidation of the embankment material had been satisfactory. Mr Banks had found that with good compaction subsequent settlement by consolidation, even over a long period, was surprisingly small, and the allowance of $\frac{3}{8}$ in/ft of height would seem to be ample.

224. The pore-pressure results were very interesting. They revealed somewhat higher pressures than had been recorded in boulder clay during construction of Knockendon reservoir[12] but it was not possible to make an effective comparison without some information about the water content of the material as placed. Would the Authors give some particulars of this?

225. What was the estimated saving (either ratio or percentage) by adopting the bellmouth-type overflow and tunnel in preference to an open weir and channel? In a

[12] J. A. Banks, "Problems in the design and construction of Knockendon Dam". Proc. Instn civ. Engrs, Pt I, vol. 1, p. 423 (July 1952).

scheme which he was at present designing, and in former schemes investigated, the indications had been that there was little difference in cost either way, and in these circumstances his tendency had been to favour the orthodox spillway and channel. The combination of valve tower and overflow on somewhat similar lines to that adopted for Sutton Bingham had been investigated in a preliminary way and the indications were that the discharge of flood-waters through the tunnel or culvert was largely controlled by the hydraulic conditions at the bottom of the shaft. Was the safe maximum capacity of the tunnel as determined by the perspex model controlled by pressurized conditions at the base of the overflow shaft?

226. The Authors considered that a fairly steep gradient in the tunnel was desirable, it being felt that reservoir tunnels tended to settle in the course of time. Had they any data to support their belief? The gradient in this case—1 in 100—had probably been determined by site levels or hydraulic requirements; if it had been adopted to counter settlement then it would be disturbing to have to anticipate settlement such as would require a gradient of this order for correction. If such circumstances did exist, then there might have been a case for the adoption of a culvert with some form of flexible joints rather than a rigidly-lined tunnel.

**The Authors,** in reply, referred to Mr Binnie's comments on the spacing of the grouting holes. The day-to-day work was governed to a large extent by the advice of the cementation sub-contractor and the 8-ft spacing mentioned in the Paper had been taken as the basis because it had suited the spacing of the contractors' timber strutting in the trench. It had been convenient to have the grout pipes clear of the struts. Where the ground had been considered to be particularly disturbed, the spacing had been reduced to 4 ft and in the better ground it had been increased to 16 ft. It had been quite obvious by the coupling-up between holes if the grouting was passing freely or not.

228. The double line had been used only on a short section on the centre-line of the dam where there was a considerable fault or throwing-down of the Oxford Clay and the Cornbrash on the right bank. A second line had also been put in on the wing grouting upstream, with the main object of testing the efficiency of the first line. French engineers had adopted the double line, e.g., at the Ghrib dam, Algeria.

229. The answer to the question concerning the upstream grout curtain as opposed to a trench was that at that stage a depth of 74 ft had been reached on the left bank and it would have been necessary to carry on the trench at that depth for a considerable distance before meeting the outcrop of the Cornbrash. It might, of course, have been possible to have half the length grouted and half in open trench, but it had seemed to be more consistent to use grouting throughout.

230. Replying to Mr Prosser, the Authors said that they had been very interested in trying to tie up the actual flow discharge of the weir with the diagram from the model tests. This was always difficult because of wave action. It was difficult to find a period when the water level was so still that it was not influencing the discharge. On the Birmingham reservoirs, for example, wave action could cause a discharge of about 100 m.g.d. The Authors had been fortunate in getting one or two readings, but they could not get any of the very high flows because the measuring weir downstream was used as a check and the flume quickly became drowned owing to the choking of the river channel on ground further downstream and outside the control of the local authority.

231. The only difficulties experienced with the pore-pressure-measurement installation had arisen from freezing at the recording end. The equipment had been filled with air-free distilled water, which probably froze more easily than anything else. At first a number of gauges had frozen and the ends of the polythene tubes had split. This was the main point which needed to be overcome, either by burying the recording equipment well underground in a manhole, for example, or by devising suitable anti-freeze mixture. At a later stage of the work, a reasonably satisfactory anti-freeze mixture had become available.

232. One of the main difficulties from the contractor's viewpoint had been the placing of the puddled clay and obtaining suitable labour. There were not many scientific tests that could be applied on the site which compared with the experience of seeing a clay in a state which could be placed reasonably well. Normally, what might be called the "finger" test was used. Taking, for instance, a piece of clay from the mill of about the size of a brick, if it was possible to place one's forefinger straight in it up to the hilt reasonably easily, it was just about right for puddle.

233. Mr Kennard's point concerning storage and yield was debatable and Mr Walton agreed that possibly a little more could be obtained from the yield than had been allowed for. The following figures, which Mr Walters had provided, were the basis for the original calculations. Using the Deacon diagram for run-off of 38-in. rainfall, losses of 24 in. had been allowed, leaving 14 in. p.a. collectable. According to the diagram, this would yield 360 g.p.d./acre, which, multiplied by the 7,470 acres of the catchment, gave about $2\frac{3}{4}$ m.g.d. This was admittedly over the fringe area on the diagram, or towards the left-hand side, which was in the range of the two consecutive driest years. On the basis of the net storage of 525,000,000 gal, $2\frac{3}{4}$ m.g.d. was equivalent to about 190 days' supply. The high dry-weather flows on which some of the figures were based were 0·1 cusec/day per 1,000 acres or 0·125 cusec/week or 0·25 cusec/month. Concerning the question of the 24-in. losses, it should be mentioned that the report on the assessment of compensation water for a 42-in. rainfall listed 25·86 in. as the loss.

234. What Mr Kennard had said about the quantity of grout accepted in supposedly non-porous areas was true. Quite a considerable quantity of grout had been accepted by the Forest Marble. This was probably in the porous limestone layers; there were, in fact, some limestone layers in it. The pressures had had to be reduced in the floor of the valley to prevent the cement travelling long distances and coming out on the surface. It had also been found that the Oxford Clay on the right bank accepted a considerable quantity of grout. The exact reason for this was not known.

235. The 22-in. discharge over the weir, which was the largest yet recorded, was equivalent, so far as could be assessed from the model chart, to about 1,000 cusecs. The flood had occurred after a very severe storm centred over Weymouth, where a rainfall of about 7 in. had been recorded in 24 hours. Speaking from memory, the rainfall on the catchment area had been about half of that over the same period; so that a rainfall of about $3\frac{1}{2}$ in. or possibly a little more in 24 hours had produced the flood of 1,000 cusecs.

236. Mr Penman had drawn attention to one of the points of anxiety concerning piezometer measurements: i.e., the danger or otherwise of carrying the polythene tubes through the puddle core and the possibility of leakage being set up. They were of course, very small tubes. They had been laid in a trench, which had been first lined with puddle clay. More clay had been placed on top of the tubes and the whole had been heeled in, so that the tubes were embedded reasonably well in puddle clay for a length of at least 200 ft; there was not much chance of leakage over the whole of that distance.

237. In reply to Mr Lambert, site tests for gauging the compaction of the embankment fill had been made as followed.

238. Proctor tests using standard apparatus described in B.S.1377 had been carried out on the material from the borrow-pits to find the variation in density with moisture content. The results had been plotted to find the maximum dry density and optimum moisture content.

239. The material could be divided into two groups, a brown sandy clay with maximum dry density of 102 lb/cu. ft with 23% moisture content; and a blue clay with maximum dry density of 104·5 lb/cu. ft with 19·5% moisture content.

240. The density of material as placed in the enbankment had been found by levelling a small area, digging a hole about 4 in. deep, and weighing the material excavated, the volume of the hole being found by means of a sand bottle.

241. The relative compaction was then:

$$\frac{\text{density}}{\text{max. density on Proctor curve}} \times 100\%$$

This figure was only *relative* to an empirical figure, so percentages higher than 100 were possible. In fact, the average relative compaction for all the tests had exceeded 100%, but if two unduly high results were eliminated, an average of 98·2% for the remaining tests was obtained.

242. The Authors considered that an average of 95% compaction might be acceptable.

243. Mr Dyer had questioned the joint between the tunnel and the overflow shaft. The concrete had been finished off in a flush joint and type A "Expandite" water-stop embedded right round it. It was, therefore, completely flexible, and to date there had undoubtedly been some movement. The same type of joint had been made between the tunnel segments upstream of the tunnel core. There were four such joints between the overflow shaft and the centre-line of the dam. Definite movement had taken place on those sections, the tendency being for the overflow shaft to settle slightly relative to the concrete in the trench in the centre-line. In some cases a joint of the tunnel had opened at the bottom and closed at the top, and the next joint had done the reverse. It had undoubtedly been a very wise move to make the joints completely flexible.

244. Various minor costs might be of interest. The model, which had proved very successful, had cost only about £400. The cost of the aerial survey, with 2-ft contours of the whole of the reservoir basin, had been £360; and of the pore-pressure measurements, £533. There had been queries from time to time about the cost of removing the unsuitable area of alluvium in the bottom of the reservoir and replacing it with better material. The cost had been slightly less than £23,000.

245. The use of sand drains to relieve the pressure was another method of dealing with this type of condition and had been used successfully at Usk and also at Chewstoke and Selset reservoirs. There would not apparently have been a great deal of difference in the cost had this method been used at Sutton Bingham, where the opinion of the Authors had been that the depth was not excessive and that it was more satisfactory to remove the bad material and replace it.

246. The Authors agreed with Mr Bolitho that flood gates or a siphon would increase the capacity of the reservoir at little extra cost but these expedients were by no means standard practice for English domestic water-supply systems. For hydro-electric schemes, particularly with concrete dams and fully electric automation available with skilled electric operatives, they would agree, but for single earthen embankment in a populated area with one or two unskilled electric operators such as the reservoir keeper or treatment plant attendant, they would not consider adopting this procedure. They were not, however, prejudiced because recently they had added a siphon for 4,000 cusecs on an earthen embankment for about £10,000—cheap for an additional storage of 200 million gallons.

247. The variable nature of the Forest Marble had not been apparent until opened out, for it was in a considerably faulted area masked by a thickness of alluvium. There were limestone and gritstone seams in the formation and the use of proper puddled clay had been considered essential, supported by the fact afterwards that even the natural Oxford Clay accepted grout.

248. With regard to sealing the outcrop of the Cornbrash with clay, the Cornbrash was itself exposed to the skies and would be water-bearing, so that when the water in the reservoir was lowered any water pressure in the Cornbrash would push the clay sealing into the reservoir. A similar problem had arisen with the Sautet Dam in France and in the digging and treatment of the slopes of the Donzère-Mondragon and Montélimar canals, driven through permeable material making it impossible to keep the canal-water entirely in or the ground-water entirely out.

249. The Authors agreed with Mr Wolf that the last line of the Floods Committee report suggested that for an upland area the normal flood might be doubled for a

catastrophic flood. They had estimated what was considered to be a reasonable low-land flood for the Yeo at Sutton Bingham and had doubled it. As Dr Semenza had implied,[13] an engineer was called upon to exercise some intuition on occasions!

250. The maximum capacity of the bellmouth overflow when gorged was about 5,000 cusecs, which was the capacity of the tunnel. Since this was considered to be a catastrophic flood, the capacity for lower flows was more efficiently obtained by increasing the slope of the tunnel.

251. In reply to Mr Sanyal, there was no hard and fast specification for puddled clay; the "finger test" (§ 232) was the most satisfactory indication.

252. The upstream and downstream slopes, had been arrived at by analogy from experience elsewhere coupled with the fact that there were undoubtedly good, bad, and indifferent materials on the site confirmed by some of the shear-test analyses, which were equally variable, and slip circle analyses through the bad material which had ultimately been taken out. The flat slopes certainly seemed to have been justified but the Authors would say that it was very difficult to be too precise in deciding before opening out the foundations in such a heterogeneous material.

253. For domestic water supply permeable leakage depended primarily on safety (geological conditions) and sufficiency to meet the maximum demands in droughts. In this case leakage through the Cornbrash might not have endangered the structure but the supply. In hydro-electric schemes, leakage might not be very material: every case must be judged on its own merits; there was no yardstick.

254. Upstream pore-pressures would naturally rise and fall with the water level, since the material would be sufficiently granular to stand up to the flat slopes either wet or dry. An excessive rise in pore-pressure on the down-stream side, however, would necessitate slowing down construction and/or inserting horizontal layers of granular material.

255. There was still too little known to be able to *anticipate* exact biological conditions. Since biological conditions might arise suddenly and apparatus was cheap, it was best to provide the latter. The Authors were not familiar with the advantages of "charcoal blanketing". The pre-chlorination was very successful.

256. The rate of flow through 3 ft of sand in the sand filter was 80 to 100 gal/hour/sq. ft. No residual chlorine had been observed at the ends of the distribution system which was a long network of mains taken off the service reservoir at Coker Hill. However, after the works had been in use for about 12 months some specimens of *Asellus aquaticus* had been recovered from certain consumers' taps in the distribution system. These had probably been long resident in the old mains and had retreated before the spread of chlorine through the system.

257. The Authors were grateful to Mr Banks for bringing back memories of early difficulties and decisions which had had to be made, such as the impact of soil mechanics on the behaviour of the Forest Marble and the clay (with its limestones and grit seams), which was harder than anticipated with the considerable (for this area) thickness of very permeable Cornbrash.

258. The left-bank Cornbrash would not collect sufficient water in itself to show much in the trench but when it was to be submerged with 40 ft of water in direct contact with it, it might absorb more than the dry-weather flow between a large and a small gathering ground; hence the Authors' nervousness regarding the smallness of the reservoir and the attendant calculations to ensure $2\frac{3}{4}$ m.g.d. with a reservoir as watertight as possible.

259. Regarding the overall density of the grout curtain, it was gratifying to learn that the quantity of grout used was not high compared with other formations in Mr Banks's experience. For the matter of the junction of the concrete in the trench with the puddle

[13] Carlo Semenza, "The most recent dams by the Società Adriaticà di Elettricità (S.A.D.E.) in the Eastern Alps". Proc. Instn civ. Engrs, Pt I, vol. 1, p. 508. See pp. 512, 558.

in the embankment, "nose" versus "groove", the Authors felt, as Mr Banks has said, that the groove was preferable below ground level; also having regard to settlement hereafter of the bank, the groove was more likely to retain the puddle whereas a nose might tend to separate it.

260. Index properties of materials were varied; the following could be taken as typical:—

| | |
|---|---|
| Average liquid limit . . . . . . . . . . . . . | 31 to 44% |
| Average plastic limit . . . . . . . . . . . . . | 11 to 18% |
| Maximum dry density . . . . . . . . . . . . | 110 to 115 lb/cu. ft. |

261. With regard to compaction, the settlement allowance was generous. In the first 2 years after completion of the wave wall, the maximum settlement recorded at any station was a little more than 3 in. The curve did not look unpleasing.

262. The average moisture content of banking material was 21·3%, the maximum 29%, and the minimum 16·7%. The material, however, was extremely variable.

263. The estimated saving in adopting the bellmouth was approximately £10,000; a large tunnel had been necessary for construction.

264. It would be safe to say that for all flows up to about 1,200 cusecs there would always be air in the tunnel and up the shaft. Thereafter the total head loss estimated was $1{\cdot}74V^2/2g$—the velocity head in the outlet tunnel made up of:—

| | |
|---|---|
| Loss in overflow shaft and tunnel entrance | $= 0{\cdot}55V^2/2g$ |
| Loss due to pipe friction in outlet tunnel | $= 0{\cdot}42V^2/2g$ |
| Loss of kinetic energy at outlet of tunnel | $= 0{\cdot}77V^2/2g$ |

265. The Authors had had experience of settlement in a tunnel after about 10 years, shown up by an unsightly puddle of water in the tunnel which had a very flat gradient. The gradient of the tunnel had been increased to suit the site; any increase in slope was considered to be also an advantage for helping discharges at all ordinary flows.