Paper No. 6575

Selset reservoir: design and construction†

by

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and

Paper No. 6589

Selset reservoir: design and performance of the embankment†

by

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and

Peter Rolfe Vaughan, B.Sc.(Eng.

Joint discussion

Mr A. L. Little (Associate, Binnie, Deacon & Gourley), in opening the discussion, said that the Authors had presented two Papers which were so full of useful data that he felt diffident in introducing yet another topic of discussion. He proposed, however, to say a few words about some problems encountered during the construction of the Ayer Itam dam at Penang, which was now nearing completion.*

109. The area was one of high rainfall, ranging from 100 in. to more than 160 in. per annum. In last year's (1961) construction season the total rainfall was 124 in., but in spite of this they were struggling on. The local rock was granite which underwent in situ weathering under tropical conditions, giving rise to a material having not only a high angle of shearing resistance but apparently a high cohesion as well; at the same time the consolidation coefficient and permeability were rather low.

110. As a result of the high rainfall, not unnaturally, the fill had been wet, and high pore pressures had occurred. The rainfall was fairly uniformly distributed during the year, except that the latter part of the year was rather wetter than the earlier part. A preliminary investigation was made of the borrow areas, and certain assumptions were made about the nature and distribution of materials in them, but unfortunately, when the contractor began his operations in them, it was found that the assumptions were not borne out in practice, and it became necessary to undertake a partial re-design of the dam. Tests which had already been done on the fill were supplemented by further tests and used in the design.

111. Table 5 was a summary of the results of triaxial tests on fill samples. As mentioned by the Authors of the second Paper, there had been an evolution in testing

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† Proc. Instn civ. Engrs, vol. 21, (Feb. 1962) pp. 277–304 and pp. 305–346.
* Embanking completed on 20th March, 1962

TABLE 5.—PENANG, AYER ITAM DAM

Triaxial test results

Consolidated, undrained tests with pore water pressure measurement

Sample	L.L. %	B	c'	ϕ'	Approx. time to failure : hours
US 30	66	0·48 to 0·58	0	36°	2·7 to 4·5
US 41	51	0·57 to 0·58	0	37½°	2·2 to 3·2
DS 22	31	0·11 to 0·13	500	36°	3·2 to 6·0
DS 23	32	0·19 to 0·17	500	36½°	2·4 to 4·8
DS A	36	0·78 to 0·92	300	34°	4·8 to 8·4
DS B	32	0·21 to 0·11	600	34°	2·5 to 3·5
DS 104	39	0·63 to 0·91	0	35°	2·6 to 4·8
		Consolidated, drained			
			c_d	ϕ_d	
DS 111	62	0·58 to 0·72	700	34½°	9·3 to 17·3
		Consolidated, drained with back pressure to produce saturation			
US 43			400	35°	4·5 to 9·2
US 44			500	34°	7·0 to 9·4
US 55			850	37½°	2·5 to 5·4
US 60			400	36°	0·9 to 11·3
US 61			650	39°	1·7 to 2·0
US 95		Min. value of 0·96	650	32°	2·0 to 11·2
US 111	41		0	38°	3·5 to 34·3
DS 113			700	38½°	3·5 to 5·6
DS 127			650	41½°	2·5 to 2·7
DS 150			450	40°	1·7 to 5·2
DS 165	36		0	43°	1·8 to 5·4

technique; the first series of tests were done on partially saturated soils, using the technique of measuring pore water pressure during the tests in order to arrive at effective stress parameters. As time went on they had changed over to consolidated drained tests, and they had saturated the samples by raising the pressure under which the tests were done to dissolve the air in the soil pores. The *B* column gave a clue to the degree of saturation; a minimum value of ·96 was taken as indicating a saturated sample. Fine porous stones had not been used for any of the tests. As a result of these tests they had used design parameters of $c'=500$ lb/ft^2, $\phi'=36°$, but it would be seen that there were some rather considerable variations from those values.

112. Fig. 25 demonstrated his principal point—the difficulty of interpreting individual results. If one plotted all the results on a single sheet one could make some sort of sense out of them, but when one came to look at a single result like this one was in some difficulty. The testing laboratory interpretation was shown by the line going through the origin, with $c_d=0$, $\phi_d=43°$. It would seem more reasonable to take some account of circle 1 and therefore give the material a slight cohesion. This particular line showed $c_d=450$ lb/sq. ft, $\phi_d=40°$. The figure of 450 lb/sq. ft might not seem very large but it was quite important in fact.

113. Fig. 26 was a simplified diagram of the bank, which was provided with horizontal drains like Selset, and he was particularly concerned with the analysis of the

top section on the downstream side. The important effect that c_d had could be observed. For a cohesion of 200 lb/sq. ft the "worst circle" was *BB* (centre B_0) with a factor of safety of 1·50, but if c_d changed to 100, which was not a very big reduction, then the "worst circle" became *AA* (centre A_0) and the factor of safety dropped to 1·15. If one had a c_d of 0 then the "worst circle" was theoretically of infinite radius and was actually along the slope and the factor of safety was 0·5, but along the surface the pore water

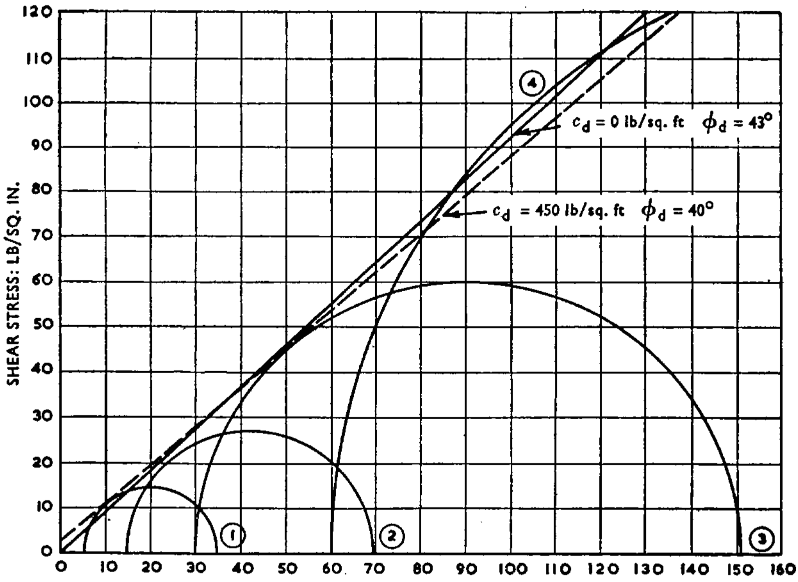
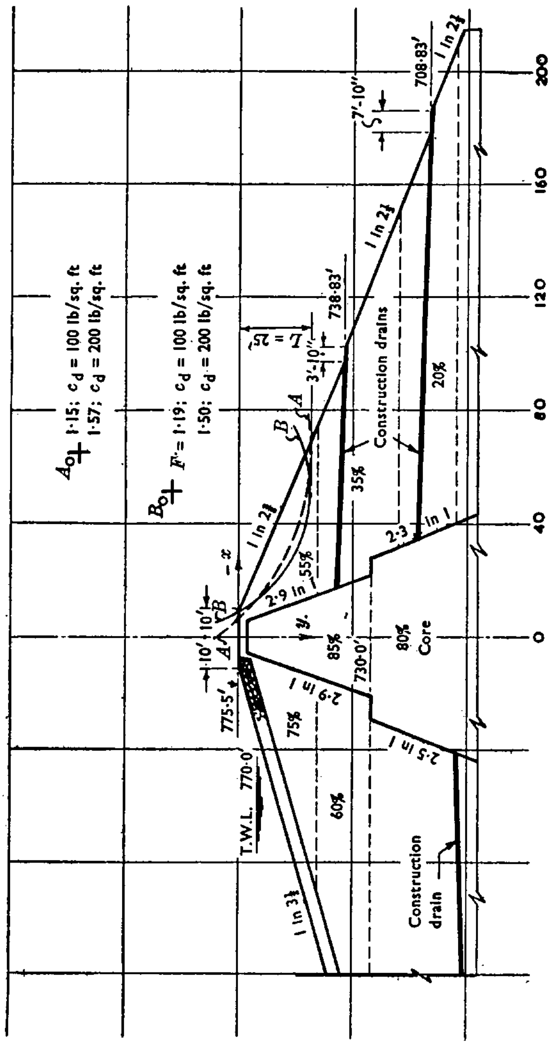


FIG. 25.—INTERPRETATION OF PARAMETERS—DRAINED TEST

pressure was zero, which made the concept rather an artificial one. However, it did indicate that with a very low c' there was a danger of shallow slips. The difficulty about the disproportionate effect of c' did not arise from the method of stability analysis but was a logical conclusion from Coulomb's law, and he hoped that Bishop, Vaughan, and other workers in soil mechanics could give further attention to this very involved problem, which did raise a lot of difficulties for dam designers.

Mr R. J. Court (Agent for Selset Reservoir, Balfour, Beatty & Co. Ltd) said that he would first of all like to comment on the large quantity of masonry which was used on this reservoir construction, altogether a quarter of a million cubic ft of it, which had to be imported and cost a very considerable sum of money. It was used to line the spillway channel, it formed the wave wall and was used for the protection of the upstream slope on the reservoir side. Were the Authors of the first Paper satisfied that these days it was the right material to use on this type of construction? He would have thought that in situ concrete or precast concrete could in most of those instances have been used, rather more economically than the natural stone.

115. It had also to be borne in mind that although the natural stone looked rather delightful, there was a boundary fence around all these reservoirs and the general public had no opportunity of seeing the work at all, and consequently the only people who had



Note:- Assumed pore water pressures r_u: percentage
Downstream shoulder $\phi_d = 36°$

FIG. 26.—SIMPLIFIED CROSS SECTION OF AYER ITAM DAM, PENANG

observed the stone were the Water Board employees, and he did not think they were really all that keen on it, knowing that it cost a lot of money.

116. Reference had been made in the Paper to the method used by the contractors in excavating the cut-off trench and the point should be made that although the method was satisfactory it did have its snags, and in particular this occurred when it was necessary to go down to a deeper bottom than had originally been anticipated. The wells in which the hoists operated were not necessarily deep enough and there was trouble in getting the excavated material away. A short section of the trench was constructed by more conventional methods. The sheeting was horizontal and the excavated material was removed by crane and it might be of interest to know that there was practically nothing to choose between the cost of the two particular methods of excavation of the trench.

117. Reference had been made to the clay core of this dam. They (the contractors) could not find a suitable means of mechanically puddling the core. There was nothing unusual about that. Many contractors had tried before, and all with the same failure. It was just not possible. However, here at Selset the boulder clay used in the construction of the dam itself was pretty impervious in itself. If the engineer were re-designing this particular reservoir would he again specify a puddle clay core? Probably he would not. He would specify a rolled clay core. He had actually done that in the Balderhead reservoir located not very far away from Selset. There was no doubt that the piezometer installations at Selset gave vital information all through the construction of the embankment to the engineer and were of great value to him. However, they (the contractors) felt that they might have got some benefit from the readings which were obtained, and did rather believe that some relaxation of the very rigorous specification for the embankment fill might have been made and so facilitated their work. Actually there was a relaxation. In the third year they were able to place more than the specified height of fill but that was the only relaxation, in that third season. What he wondered was whether the Authors would agree that with the pore water pressures which were shown in Fig. 13 of their Paper they could not have permitted filling with a greater moisture content in the later stages of the actual construction of the embankment.

118. From a contractor's point of view it was interesting to see that the Authors had given considerable information about the incidence of lost time and the effect of rainfall on a construction of this nature. Until one had worked on a project such as this it was extremely difficult to appreciate how little time was actually available in one calendar year for the placing of fill material. The Paper stated that at the end of the 1957 season the contractors, who had experience of what the weather could do to them, had planned to work only 7 shifts out of 11 in 1958, and it was worth while recalling that in actual fact they had not had as long as that for work. They were able to work six shifts in a full week of seven days working whenever they possibly could, and that was equivalent to six out of every 12¾ shifts, since they did not work on Saturdays after four o'clock.

119. Another point which required emphasizing was that work did not cease just when it was raining but there was a tremendous amount of time wasted and lost while one was waiting for the fill to dry out.

Mr Stanley Serota (Managing Director, Foundation Engineering Ltd) said that he would like to enlarge a little upon the subject of the vertical sand drains.

121. It was the Company to which he belonged who carried out the installation of the drains, and he wished to say how very fair the engineers were in permitting them to tender for the job in such a way that if their method of installing the drains did not work, then they would have been entitled to recover all their mobilisation and demobilisation expenses—a very substantial proportion of the whole cost.

122. This was, he thought, a good example of the unusual flexibility of outlook displayed by the Authors of both Papers in their approach to the design and construction of the dam.

123. It was evident from reading the Papers that they were prepared to change their plans according to the way the information came back to them from the piezometer points.

124. His company were, of course, reasonably confident that the method of installing the drains would work, and their principal fears were, firstly, that the alluvial gravel (which appeared to be up to 15 ft thick in places) would resist the pile driving, and, secondly, that the very large boulders known to be in the soft boulder clay might have been immovable. There was a third danger, and that was that these boulders, or the 10 in. cobbles in the gravel, might easily have broken a lot of the shoes, which were made of cast iron.

125. Although from the point of view of making their contract profitable they were protected against a complete failure of the method, there was a very real danger that these difficulties he had just mentioned, while not enough to get them pushed off the job, might easily have been enough to slow it down to an uneconomic level. However, they were quite confident that if the top layer of gravel were loose they should drive through it quite easily, and Dr Bishop had assured him that from his own observations on the site he knew it to be loose. He had said that in excavations it slipped very easily, and the interstices were open and clean. Furthermore, with regard to the clay, they were able to calculate that the largest boulder likely to be encountered would be driven to one side because the clay was so soft, and, of course, this calculation again was dependent on Dr Bishop's observations. It could be said that with those comforting thoughts, and the courage of their belief in Dr Bishop's convictions, they were asked to get on with the job! As it turned out, the driving through the gravel was very easy indeed; as a matter of fact, the records showed only about three or four blows per ft and very few shoes indeed were broken in pushing the boulders aside in the clay, and the job went very well.

Mr J. K. Hunter (Consultant, Sir Alexander Gibb & Partners) said that the Paper by J. and M. F. Kennard gave (p. 303) a table showing a summary of the costs, and it was there stated that the parliamentary estimate prepared in 1952 was £3 million, and that the final cost of the work eight years later was £3·2 million. That £3·2 million, they pointed out, included the sum of £255,000 in respect of the price variation clause, and if one made that deduction it seemed reasonable to conclude that the works were actually carried out within the original parliamentary estimates.

127. Anyone who had had experience in the post-war years of preparing estimates for large civil engineering works—especially those involving techniques which were relatively new to this country—would fully appreciate the difficulty of arriving at a close estimate of the final cost. From that point of view alone the works at Selset represented a very significant achievement especially as it would appear that the working period during which it was possible to place fill had proved to be a good deal less than had been anticipated before the work commenced. He expressed the hope that the consulting engineers and all those others concerned would receive proper credit from the quarter in which it was most important.

128. The summary of cost showed that the sum of £10,000 had been allocated to site exploration and model tests. Having had some experience of site exploration work he found it very difficult to believe that that sum covered all the investigations necessary to provide a sound basis of design. It seemed to him to be a very small sum bearing in mind how little could be bought today for £10,000.

129. Turning to the second Paper, by Bishop and Vaughan, the Authors had shown that the boulder clay used at Selset was very sensitive to small changes in the water content of the fill. They had pointed out that, with an embankment slope of 4 to 1, a factor of safety of 1·5 to 1 could be reduced to 1, i.e. to the point of incipient failure, by a change of water content of only 1%; as a relative layman in these matters he found that rather startling. A rolled fill dam was most sensitive to the effect of excessive pore water pressure during the construction period. After completion the pressure tended to dissipate, and the factor of safety should therefore increase with time. It might

therefore be argued that, providing other factors remained constant, a properly constructed embankment should become more stable as time went on.

130. The Authors had pointed out, however, that experience at Selset had shown that after the reservoir was filled it was possible for dangerously high uplift pressures to develop on the downstream side of the cut-off, and they went on to point out that such uplift pressures might lead to the eventual collapse of the embankment if provision were not made for their relief. It would seem to follow, therefore, that the continued safety of the dam was to some extent bound up with the future effectiveness of the relief wells provided.

131. Thirty years ago it was common in the design of concrete dams to provide elaborate drainage systems, especially in America: such systems were a functional factor in design. Experience had shown that the drains tended to become choked and to require continued maintenance, and he believed that it was mainly for that reason they were now less favoured than once they were. This made him wonder to what extent the future stability of the Selset dam was dependent on the continued effectiveness of the drainage system.

Mr Hugh H. Dixon (Partner, Howard Humphreys & Sons) said that the word "seepage" in dam engineering had formerly been regarded as rather a dirty word, he thought, but the revolution brought about by Dr Terzaghi, whose conception of a dam was a structure designed to control seepage within tolerable limits and incorporating the features necessary to carry away the seepage in a manner that would not endanger its stability, had made a profound difference.

133. He had intended dealing rather more fully with the puddle clay core but it had been covered already by a previous speaker. However, he had made some calculations, as far as he was able to, from the figures given in the Papers. He had assumed that the permeability of the boulder clay was one-tenth of that in the Usk Dam, and had found that if one took account of the puddle clay core the total seepage through the dam was of the order of 110 gallons per day. If, on the other hand, one omitted the puddle clay core and put in roll fill right across, the flow would have been of the order of 120 gallons per day—he had not the exact figures. This comparison was, he thought, a measure of the significance of the puddle clay, but it might very well have been put there for other reasons.

134. Relief wells had been used in America and, indeed, the water that had been provided by them had been on occasion pumped back into the reservoir—it had been found economical to do just that.

135. The other point he wished to make was that in the instrumentation of a dam a great deal of money was spent in providing pore pressure points and so forth, but as soon as the resident engineers' staff turned their backs on it the equipment went to pot. Only last week he had received a letter from one of his engineers abroad in which he was told that the gauges were all rusted, pointers stuck and so on. He had suspected that something was odd when he got the last readings for the pore pressure measurements!

136. A book written in 1898 regarding Japan, by W. K. Burton, Professor of Sanitary Engineering at Tokio, said with regard to puddle clay cores, " . . . if each layer be thoroughly punned, it will be all right but unless the work be closely watched, it is almost useless. Japanese work officials are inclined to use old men, women and boys for this, but a single horse led by a boy is better than a crowd of stampers!" We have moved a long way since then.

Mr R. H. Cuthbertson (Consulting Engineer) congratulated the Authors of the first Paper not only on the comprehensive way in which they had dealt with the problems covering the whole range of earth dam construction but in giving the Authors of the second Paper the opportunity for a most excellent dissertation on the general problems.

138. His comments were based on experience of the design and construction of two dams within the last five years, built entirely of boulder clay of comparable qualities to Selset.

139. He first wished to comment on the method of excavation of the cut-off trench. This so very often formed a bottleneck in the construction of earth dams under modern conditions. Was it thought that the fixed hoist and monorail method showed any advantage over the more common method of crawler cranes moving about? His firm were now giving their attention to doing a considerable amount of open casting at the top of the trench, particularly where the material could be used on the embankment. Some studies of this had shown that this should be economic in these circumstances up to about 20 ft to 30 ft.

140. The Authors' experience of trying to grout in millstone grit paralleled the experience of Sheffield Corporation in their Broomhead Reservoir Valley, where large quantities of grout were placed with little effect. These works were described in a Paper by the late J. K. Swales, who was responsible for the remedial works, along the same lines as the Authors adopted.

141. The Authors had very properly dealt with the problem of construction period, which very much affected the contractor's interest. He was rather surprised at the amount of interference they had from showers. While he had found that the annual working periods were very similar, his experience had been that placing the fill with scrapers, with a high concentration of plant, one could work during quite rainy weather.

142. The scrapers were continually exposing fresh surfaces of borrow pit and covering the material as it was spread on the embankment.

143. The puddle core may have had some adverse effects in this direction. The dams to which he had referred were both constructed with rolled fill, with only a limited selection of material from the borrow pits, and the advantages to be gained, especially in smaller embankments, with the plant being able to run freely over the core, had to be seen to be believed.

144. As regards the second Paper, the Authors' general appreciations of the problems were in line with his firm's findings in their investigations into the dams to which he had referred, and, speaking as one more concerned with making decisions as to the validity of the various soil design parameters suitable to the circumstances of the site and construction, he much appreciated the Authors' approach. The selection of samples was a very important matter. As a result of recent experience they were now exploring borrow pit areas in very substantial detail, and sinking a large number of holes and sampling every 5 ft. Initially these were examined quite exhaustively, but, in the light of results, one was usually able thereafter to curtail the examinations very much. This procedure gave one very much greater confidence in the parameters produced from tests carried out on samples and mixtures thereof.

145. It was important also, he thought, to investigate borrow pits at different levels in boulder clay. There were quite substantial differences in composition in various horizons. It was generally found that the top layers had a higher moisture content and the lower layers were very much stonier. In selecting for a rolled core that information was essential.

146. The selection of parameters for design always posed a dilemma, and the Authors' observations reflected his (Mr Cuthbertson's) own experience in that direction. Generally his experience of the effect of the application of rigorous analysis had been that flatter slopes, entailing higher costs, had been required. The results in the end appeared to have erred rather much on the safe side, but he thought this was a circumstance which would require to be tolerated until more experience had been gained.

Mr R. C. S. Walters (Partner, Herbert Lapworth Partners) said that there was so much with which he agreed in both these Papers that he could not raise very much in contradiction.

148. For comparison with Fig. 4, Fig. 27* showed a dam trench at Trentabank in

* The illustration is taken from the writer's "Dam geology", Butterworths, London, 1962 p. 72.

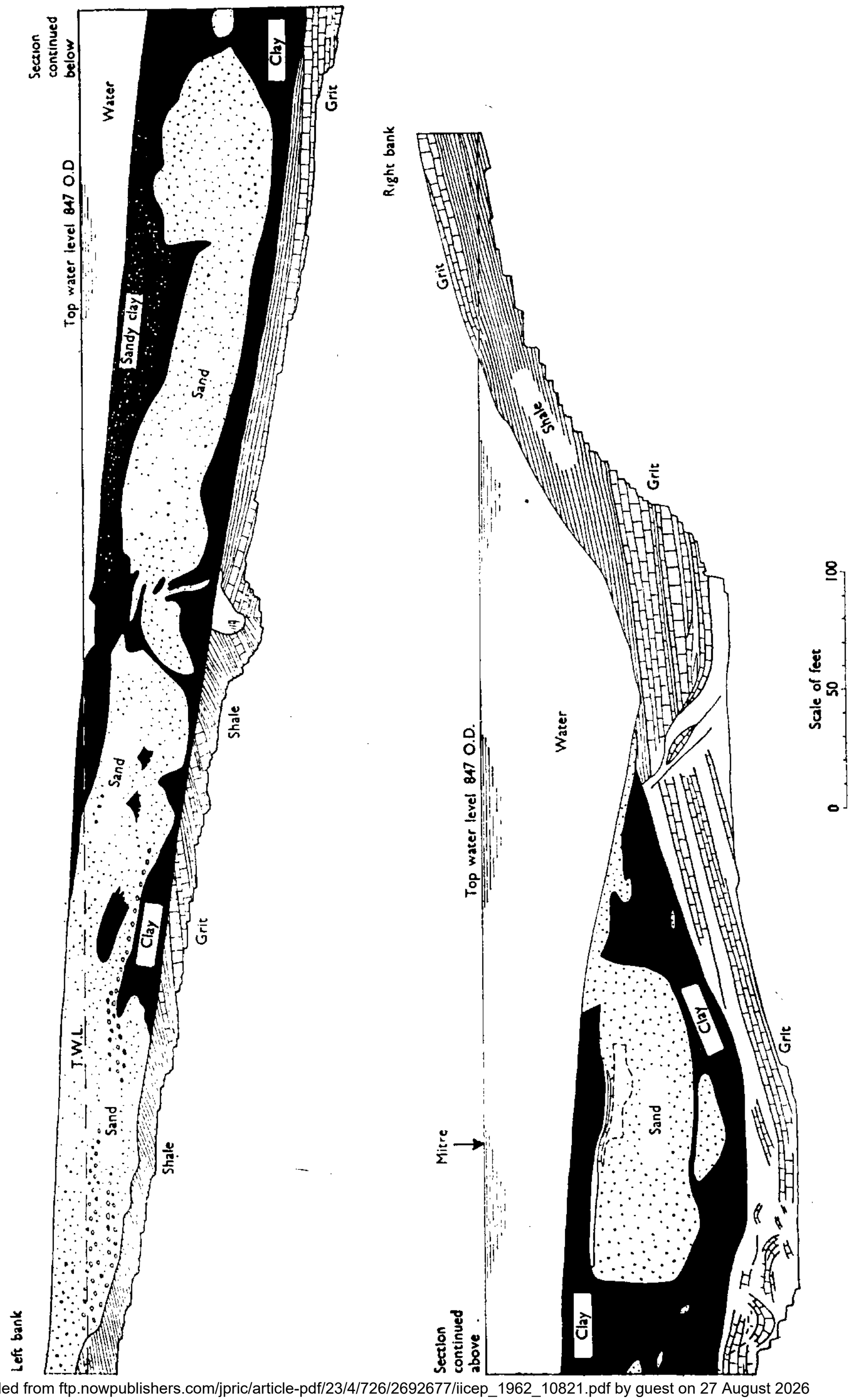


FIG. 27.—THE TRENTABANK DAM (1923–29)

Macclesfield, constructed in 1923–29, impounding 65 ft of water and about 80 ft deep. In those days there were no grout curtains in the gritstones and shales in the Pennines below the Millstone Grits, which were very similar to those at Selset, there was no leakage as far as could be proved after 30 or 40 years, with no grout curtain. With regard to the bank, there was only the section of the trench to go upon, the surface coloured blue on the geological maps; they did not know much about what the bank stood on except through this section of the trench which consisted of large lenticular masses of sand in Boulder Clay. There were no soil mechanics in those days and as far as he remembered there were not very many tests on the particular embankment. The slopes were 2 to 1 on the downstream side and 3 to 1 on the water side. That was just an example of something that had stood, and he was sure that Mr Kennard's dam would stand too.

149. He wished to ask Mr Kennard what was the concrete aggregate, and had he read rightly that the vertical sand drains were filled with limestone, or was it Whinstone as the film said? He imagined that if they were filled with anything like limestone there would be perhaps some clogging. Mr Kennard had also mentioned in § 25, the projecting of the grout tubes, and mentioned also the use of jack hammers. Mr Walters was not clear whether the grout holes which were left by tubes projecting might impede laying the concrete in the trench and whether it would have been better to drill through the concrete for the grouting as he had found on one or two occasions that the projecting tubes interfered with the work.

150. He agreed with the necessity of water-stops in the concrete cut off trench. In one case in his experience they had been omitted and there was some cracking and possibly some seepage (which was not proved).

151. Grouting in stages of 10 ft rather than 25 ft was a desirable precaution, and in these alternating strata of gritstones and shales below the Boulder Clay they should, he thought, be taken in the shorter lifts of 10 ft, even though it was more expensive for grouting in sections of 10 ft rather than 25 ft. The warning of Muirhead was a thing to take to heart, the mechanical analysis of the Muirhead clay and the mechanical analysis at Selset were very similar and nobody could afford, if building a dam of such a material, having such a mechanical analysis, to ignore the fact hereafter. He was very grateful to learn that the 50 ton roller was not apparently successful in the sticky clay. That was very important, because one did not know what sort of roller one was going to put on a clay in the initial stages of a scheme.

152. Finally, with regard to the upturned clay, he had known Oxford Clay "re-deposited" which was discovered by putting the clay into a brick kiln, where it would explode because it had picked up some pebbles.

Dr P. W. Rowe (Reader in Soil Mechanics, Manchester University) congratulated the four Authors on providing such a comprehensive case history of the relation between the calculated and the actual performance of an earth embankment. The Paper by Bishop and Vaughan brought into greater prominence the value of piezometric observations and the need for flexible contractual arrangements if direct advantage were to be gained, by their use. The long-term advantage of such records to the profession was self-evident.

154. Looking at the over-all result in this case it had so happened that the foundation design came out well on the conservative side, with a factor of safety of the order of 2·4, and this arose because the real rate of dissipation of pore pressure was at least three times greater than that predicted by laboratory tests. This ratio implied that if this difficult problem could have been accurately forecast, two-thirds of the sand drain cost might have been saved. It might be useful to reflect on two of a number of possible contributory factors, especially as the use of sand drains tended to be on the increase.

155. The laboratory measurement of the coefficient of consolidation was concerned solely with the rate of pore pressure dissipation by *drainage* under a constant effective stress ratio. But a decrease in pore pressure could arise for two other reasons.

One could be that there was a transfer of applied stress from the clay elsewhere, and the other could arise because, as the effective stress ratio increased during construction, there was a component of volume change which resulted in a decrease in pore pressure. An example of the first cause was shown by the Authors in their Fig. 17, where the pore pressure in the puddle core was remarkably reduced by the transference of applied vertical stress from the compressible puddle to the less yielding adjacent clay core. It was, he believed, the first time this had been shown.

156. In the foundation the stone filling to the drains was likely to have achieved a medium dense state in view of the particle size and method of deposition, coupled with vibrations arising with the extraction of the casing. These were seen in the film.

157. According to Fig. 23 the compressibility of the softened boulder clay exceeded that of even a loose sand by a factor of about 6; consequently 18 in. diameter short sand drains at 10 ft centres would present quite a formidable piling system, so that one would expect a beneficial transference of vertical stress from the clay to these drains during consolidation exactly after the manner observed in the core.

158. Even if the clay compressibility had been as low as that of the drains, due possibly to over-consolidation, one would expect a decrease in the pore pressure ratio for the second cause, just as illustrated in Fig. 12 for the boulder clay fill. Because neither $\bar{B}$ nor C_v were constant throughout the stress range and dissipation range there would appear to be some merit in running model tests to simulate the expected rate of application and to predict the development of pore pressure.

159. Fig. 28 was a model in which the major principle stress, σ_1, could be brought on in the laboratory scaled to the time base in the field as the ratio of 1/scale^2. The major

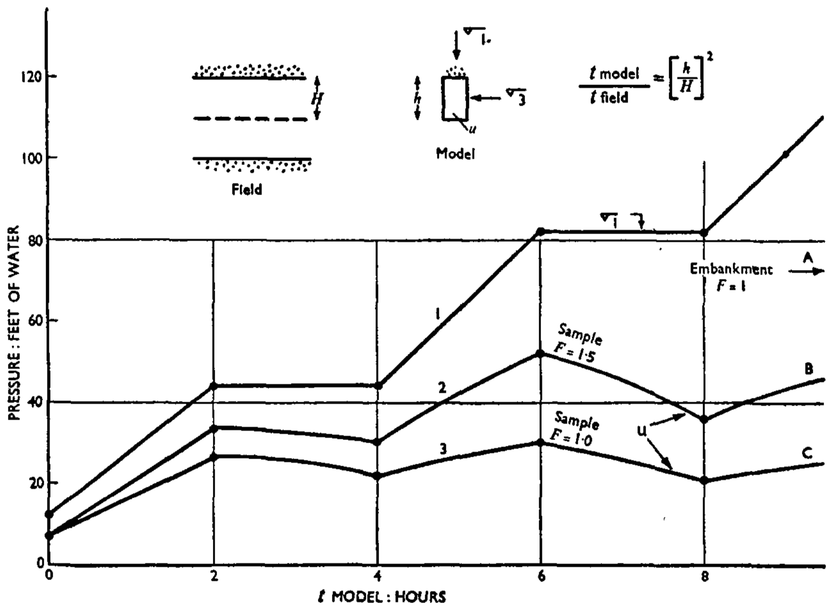


FIG. 28.—SIMULATED MODEL DISSIPATION TEST

principle stress was brought on and arrested to simulate the bank construction, and as the pore pressure both dissipated and built up together, one could arrange the lateral pressure to be such that the ratio of effective stresses maintained a given factor of safety. In graph 2, 1·5 was maintained; in 3 the graph was just held at failure all the way. This test enabled some prediction to be made of the ultimate pore pressure on completion.

It also raised the rather interesting point that evidently when the dam was not failing the pore pressure might be higher than that which would occur if it failed, so that if one coupled pore pressure at safety with $c'-\phi'$ parameters at failure one obtained a slightly different value of the factor of safety from that which would apply if one made an undrained analysis.

160. In conclusion, the use of relief wells to control and recover seepage through rock beneath surface clay formations on sites with an artesian history clearly constituted a very important feature for consideration in the early exploratory work for a reservoir foundation. He wondered whether Mr Kennard had any information regarding the pressure gradient that might be acting now to the relief wells. It would be interesting to know whether, if he were faced again with a similar site, in the after knowledge of the performance of these relief wells he would consider taking the cut-off below the boulder clay.

Mr A. D. M. Penman (Soil Mechanics Division, Building Research Station) said that a very large number of important measurements had been made at Selset but he noticed that one set was not mentioned in the Papers. They were those of the strut load measurements made in the cut-off trench by the contractor. About 34 vibrating-wire load gauges each of 25 tons capacity were installed at a point where the trench was 90 ft deep. He understood that near the surface the strut load increased to about 10 tons and at a depth of 60 ft, 20 tons was measured. He mentioned this work so that other people might remember that it had been done and there were some figures available.

162. The use of piezometers to control the work and maintain stability had been well illustrated by the Papers, and it was obvious that faulty installations could cause expense and be dangerous. Piezometer tips placed in boreholes were particularly susceptible to the fault of an imperfect seal, which would allow leakage up the borehole and cause the piezometer to show a lower pore-pressure than that which existed—a dangerous condition. The importance of a perfect seal had been mentioned in both Papers and he wished to emphasize it again. The effectiveness of the seal could only be measured by dissipation tests from the tips themselves, and these tests could take several days because steady pressure conditions must be established before measurements were made. When programming a piezometer installation due allowance must always be made for these tests when the tips are to be placed in boreholes. One of the serious drawbacks of electrical types of piezometer was the fact that the effectiveness of its seal into the ground cannot be checked.

163. The electrical settlement gauge could be considered as an alternative to the U.S.B.R. Cross-arm Gauge. It consisted of telescoping lengths of alkathene tube, $\frac{3}{4}$ in. and $1\frac{1}{2}$ in. bore, with rolling "O" rings in the joints; placed vertically with 1-ft-square steel plates $\frac{1}{2}$ in. thick with central 2 in. holes placed over them at about 10 ft vertical intervals. The plates were detected by the inductance change of a coil let down the tube. This was the first time the apparatus had been used and the first coil was found to jam easily. A smaller coil, $\frac{1}{2}$ in. dia. $\times$ $2\frac{1}{4}$ in. long, consisting of 6,894 turns of 46 s.w.g. enamelled copper wire on a Tufnol former, and pulled down by 3 brass sausage shaped weights of the same size, worked very well, and showed the most interesting flow movements of the puddle clay described in the Papers, but unfortunately during the second season a stone got into one tube and stuck at a joint 8 ft 6 in. below the surface, and the other tube developed an obstruction 40 ft down. Odd silly things happened: a length of tube was put in with a nipped place in it and a joint was found without any "O" rings: next time a larger tube will be used so that there will be more room for site imperfections. Not that they expected to be able to measure movements in any more puddle cores of course—they are obsolescent!

164. He would like to ask the Authors what had happend to the horizontal strain gauges fitted at Selset at a later stage?

Dr T. K. Chaplin (Lecturer, Graduate School in Foundation Engineering, Birmingham

University) congratulated the Authors of both Papers on giving a very full picture of the work carried out, and particularly the scientific background of the materials used in the dam.

166. He wished to comment on the compressibility of the Selset boulder clay fill, which was plotted in Fig. 4 of Bishop and Vaughan's Paper (p. 313), where the form of the curves suggested that it should be re-plotted in a way which would demonstrate whether the compressibility was being controlled by the granular structure of the fill.

167. Fig. 29 showed the results from Fig. 4, together with some others for lean fills

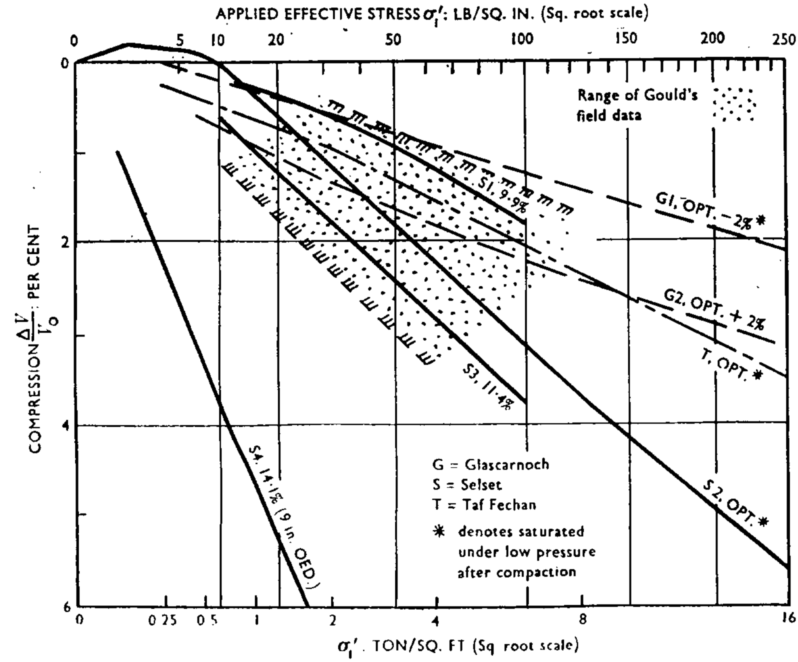


FIG. 29.—COMPRESSIBILITY OF LEAN FALLS FOR BRITISH EARTH DAMS

used in British dams, re-plotted against the square root of the applied pressure (instead of the pressure on an arithmetic scale). For the Selset fill there was a very rapid increase of compressibility with moisture content, and it would be seen that for another dam (Glascarnoch) the difference in fill compressibility between optimum −2% and optimum +2% moisture content was much less, despite the large difference in moisture content. The Glascarnoch fill had a much smaller clay fraction than the Selset boulder clay.

168. The reason for choosing such an unusual form of scale for the pressure in Fig. 29 was that it would show up, as a straight line, the results of almost any laboratory K_0 compression test on sand at normal pressures, where obviously the grain structure controlled the compressibility. Many laboratory tests had demonstrated that in general a granular soil being compressed without friction at zero lateral strain (in a K_0 compression test in the triaxial apparatus) underwent changes of strain which were directly proportional to changes in the square root of the vertical stress.

169. Those who wished to follow up the compressibility of granular soils might like to read his paper on the subject in the Symposium on Granular Soils published by the Midland Soil Mechanics and Foundation Engineering Society[30].

170. In 1953 Gould[31] reported some interesting field observations on the compressibility of various rolled fill dams in the U.S.A. Those results, whose limits were shown in Fig. 29, were given in detail in Fig. 30, which contained all of Gould's curves that showed a decrease of compressibility with increasing pressure. Within the accuracy of replotting Gould's results, it was remarkable how many dams, such as the Anderson Ranch Dam, gave long linear sections of curve when the compression strain

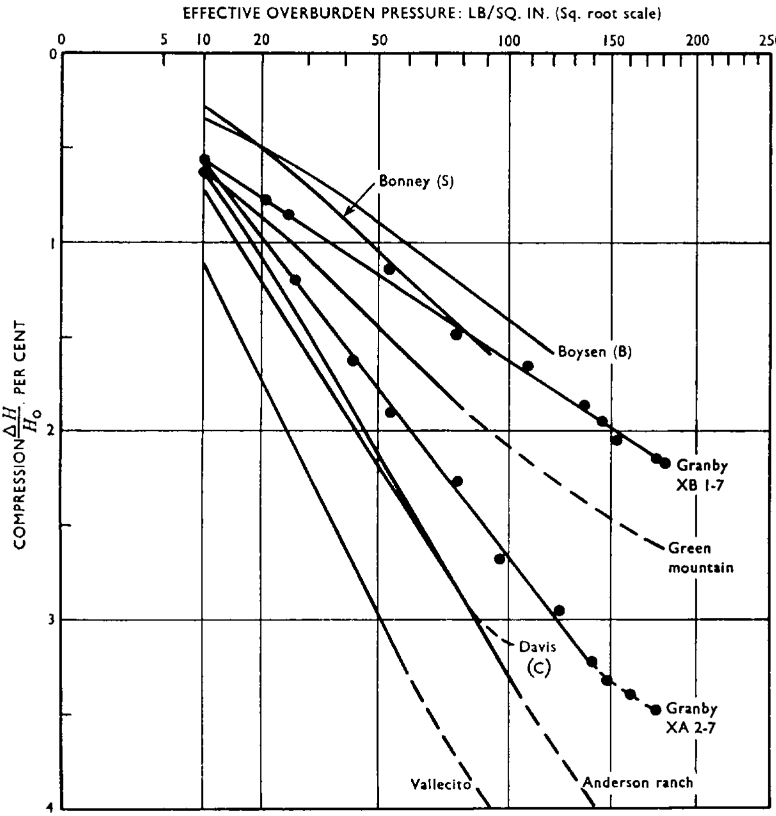


FIG. 30.—FIELD MEASUREMENTS OF COMPRESSIBILITY IN LEAN ROLLED FILL DAMS IN THE U.S.A.

was plotted against the square root of the effective overburden pressure. This indicated that the granular structure of the fills was controlling their compressibilities.

171. The Selset boulder clay showed an extremely rapid increase of compressibility with increase of moisture content near the optimum, even greater than some results quoted by Gould. That was probably due to the differences in Plasticity Index.

172. Gould's main conclusions in his 1953 Paper [31] were that at low pressures the deviation of the moisture content from the optimum was the most important influence on the compressibility at low pressures, and that at high pressures the plasticity of the fines had the greatest influence on the compressibility. It seemed to Dr Chaplin that there must in addition be an interaction effect. The greater the Plasticity Index (or clay fraction) the more one would expect clay films of appreciable thickness to separate

the contacts between the larger particles when the moisture content was above optimum by a given amount.

173. In conclusion, Dr Chaplin wished to emphasise how strongly the granular structure of a lean fill material could affect its mechanical properties, especially its compressibility[32, 33]. There was much more research work to be done before engineers could begin to understand how granular particles of various shapes worked together with clay particles in soils of low Plasticity Index.

Dr G. D. Gilbert (Assistant Engineer, Binnie, Deacon & Gourley) said that in the second Paper the Authors had made several references to the Usk Dam, which was completed in the Autumn of 1954.

175. A paper on the construction of this dam was presented to the Institution in June, 1957, and contained details of the pore water pressure measurements upto 1956[35].

176. Mr Dixon earlier this evening had raised the point of what happened to pore water pressure installations after the Resident Engineer's staff had left. At Usk they had been very fortunate, for the reservoir keeper had maintained a steady set of readings from these gauges, and he wished to show what had been happening since the reservoir was constructed.

177. Fig. 31 showed the reservoir level and the readings for point T7, just below the

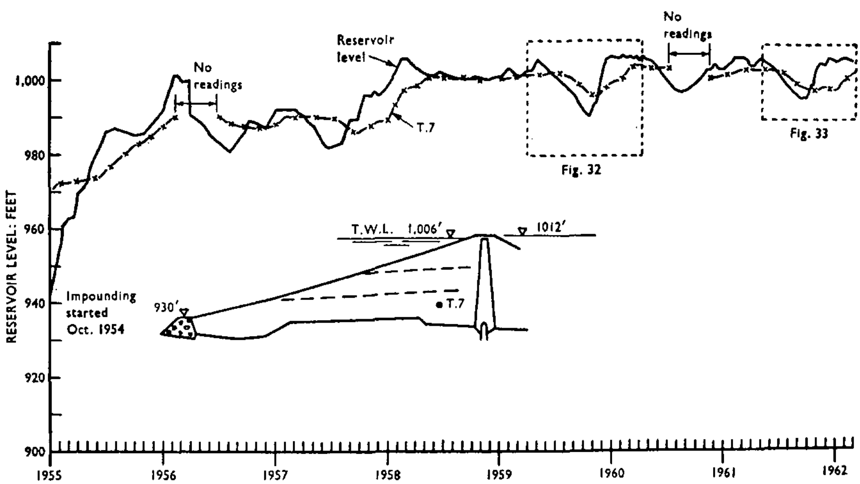


FIG. 31.—TYPICAL UPSTREAM PORE WATER PRESSURE 1955–62

lowest drainage blanket, which were typical of the readings on the upstream side. When the reservoir level went up there was a considerable lag, approximately 2 months, in the reading for the point T7 and on draw-down there was a time lag of approximately 1½ months.

178. Fig. 32 showed the readings from some of the other upstream points for the period April 1959 to March 1960 and the lag in readings can be seen.

179. Fig. 33 brought it up to date to the end of last month (February 1962). These points were continuing to give fairly satisfactory results after almost ten years' service.

180. In § 35 of the second Paper the Authors stated that "in a saturated clay of low permeability the pore pressure on drawdown was estimated to correspond to a stand pipe level equal to the height of fill above the point considered". This implied that there was virtually no time-lag in the pore pressure response in a saturated fill when the reservoir water level changed and $\bar{B}=1$ for these conditions. How far the Usk embankment

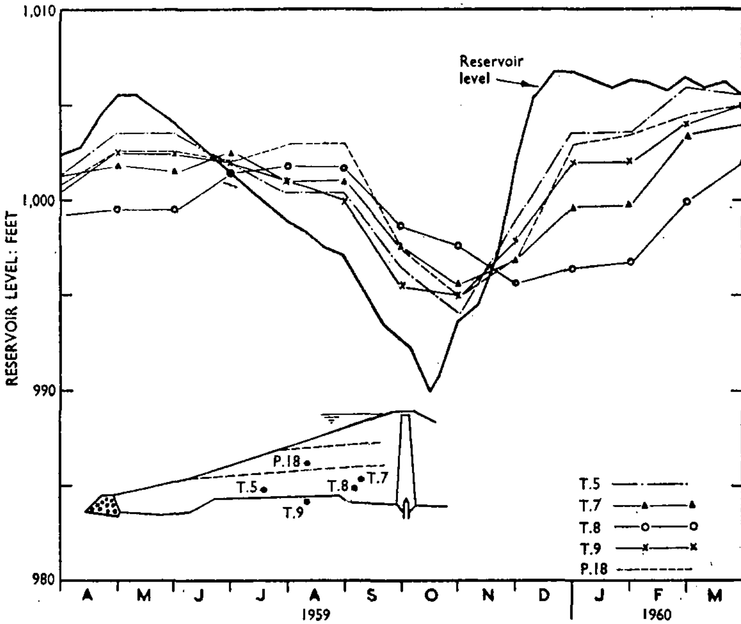


FIG. 32.—DETAIL OF UPSTREAM PORE WATER PRESSURE 1959–60

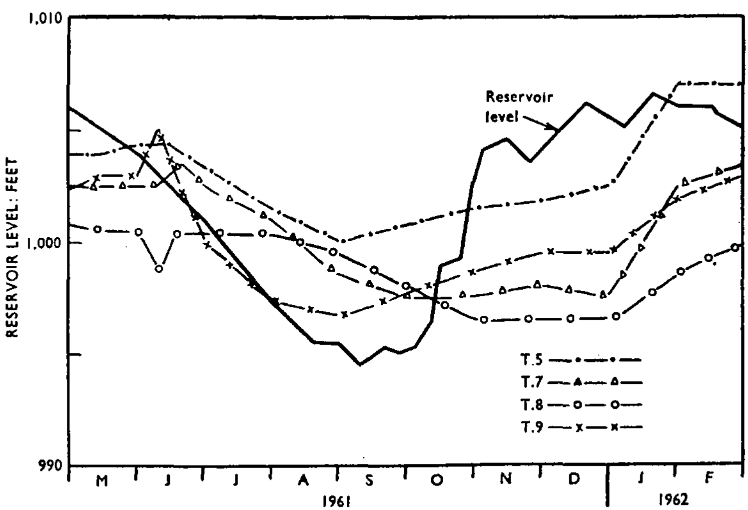


FIG. 33.—DETAIL OF UPSTREAM PORE WATER PRESSURE 1961–62

could be considered to be fully saturated it was difficult to say, but the results just shown did indicate that, despite the presence of drainage mattresses, there might be a considerable lag between the change in reservoir level and change in piezometric level.

181. Similar results were obtained during the controlled drawdown experiment at Glen Shira, and these were, he believed, subsequently confirmed in a second test carried out by the Authors themselves. It would be of interest to have their comments on the results just shown for the Usk Dam in view of the importance of the drawdown condition in the design of earth dams.

Mr T. N. W. Akroyd (Chief Engineer, Constructional Services Ltd) said that he wished to refer to two soil problems raised in the construction of Selset.

183. The first one concerned the control of compaction, possibly the weakest link in present day rolled fill construction. Recently it had been suggested[36] that the best method of compaction control was by controlling the percentage voids and the type of plant to be used. This method was probably more necessary in road construction, where the fill material could vary widely, than it was in rolled fill dam construction, where there was little difference in the soil properties from the various borrow pits. But this method of control had the advantage that the control of compaction was then not tied to the results of the Proctor compaction test.

184. When Proctor introduced his test he stated that it was evolved to reproduce, for most soils, in the laboratory, the results which were obtained with compact equipment in the field; but this had never been so in this country. The test was standardized in this country and re-named the Standard Compaction Test, but the chief difficulty in its application had been to relate it to known amounts of field compaction or plant performance, and yet this should be the very reason for which a laboratory compaction test existed.

185. Mr Akroyd thought his meaning would be clear if reference was made to Table 2 on p. 293 of the first Paper, where it was shown that the in situ density obtained in 1957 was 123 lb/cu. ft, in 1958 121 lb/cu. ft, and in 1959, 125 lb/cu. ft. In 1957 and 1958 the soil was being compacted on the wet side of the Proctor optimum and in 1959 on the dry side, and yet the higher percentages of the Proctor density were obtained with the wetter soil. The conclusion one might draw from this was that either the Proctor test was not a great help in saying what density would be achieved in the field or alternatively the amount of compaction in 1959 was insufficient or the plant was unsuitable. If, however, one determined the amount of compaction by reference to the voids ratio of the compacted soil, then after making allowance for the variation in the specific gravity, the voids ratios indicated that there was little difference in the amount of compaction in each year.

186. It seemed to him that what was important was not the relation of the in situ density to the Proctor compaction test but the percentage voids which existed in the soil after compaction. He noticed in both Papers the constant use of the term "optimum water content". The term "optimum" seemed to suggest that for some particular operation—in this case field compaction—the moisture content used was the best, but in fact it was merely the water content corresponding with the Proctor maximum density. If the modified Aasho test had been used instead, a different "optimum" and a different maximum density would have resulted.

187. His second point related to the pore water pressure measurements, and in particular to Fig. 12 on pp. 297–298. This figure was Fig. 9 in the paper by Bishop, Kennard, and Penman[37]. There appeared to be one or two inconsistencies in these figures. If one took no notice of the smooth curves plotted for 15 April, 1959, 21 September 1958 and 21 June 1959, but instead joined up the actual readings of pore pressure, then a series of hills and dales resulted. This was referred to in § 54 in the first Paper, where it was stated that "local differences of water content and density have a considerable effect in determining the pore pressure distributions in the early stages". It went on to

say that their importance diminished as the height of fill increased. With respect, Mr Akroyd submitted that this was in fact not true.

188. If one considered piezometer point 40 in Fig. 12a, this showed a low value in October, a negative value in April 1958, a low value in September 1957 and a low value in April and June of 1959. In fact it meant that some two years after the initial pore water measurements low values were still being recorded for piezometer 40, and relatively high values were being recorded for piezometer No. 39. It seemed, therefore, that local difference of either water content or density or both, had a noticeable effect in the pore water distribution at all stages.

189. If one turned now to Fig. 12b exactly the same kind of thing happened in regard to piezometer points 44 and 48. In other words the results, as opposed to the curves drawn, indicated the effect of some factor other than hydraulic gradient, on the pore water pressure. Mr Akroyd wrote to add that since the meeting he had discussed Fig. 12 with Dr Bishop, who pointed out that in Fig. 12a the results for piezometer point 46 were average results and therefore did not agree with the results in Fig. 12b. But, Mr Akroyd pointed out, whilst it was a sign of immaturity in the soil mechanics field to be surprised when site test results did not agree with theory, he felt that the use of average results for point 46 alone was open to question. If the results for point 46 from Fig. 12b were used then Fig. 12a becomes Fig. 34. From this figure he felt some difficulty in agreeing with

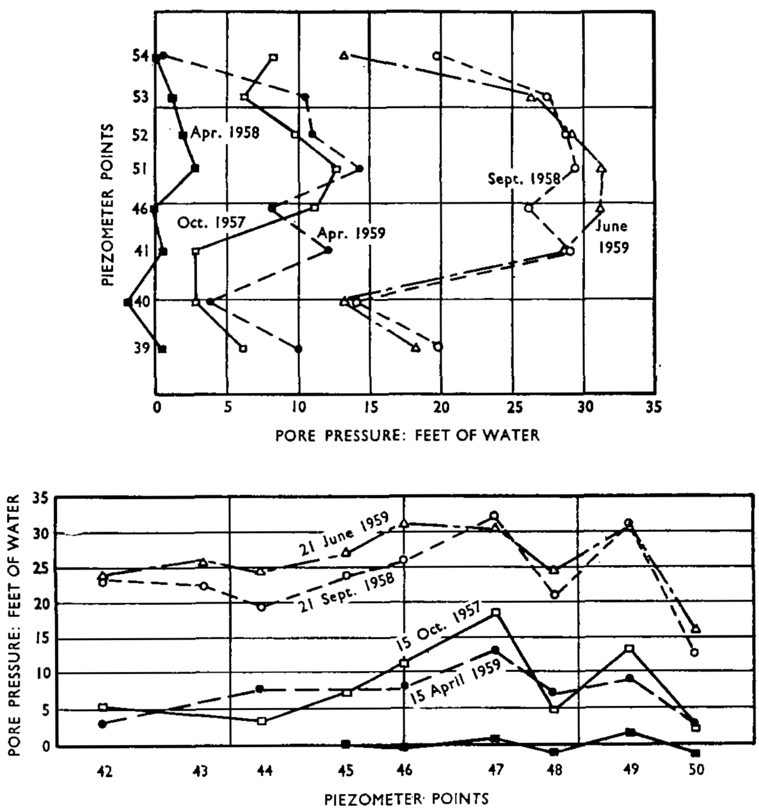


FIG. 34.—DISTRIBUTION OF PORE PRESSURE AT SELSET. LOCAL VARIATIONS IN PRESSURE MAINTAINED DESPITE DISSIPATION WITH TIME

the suggestion that irregularities in pore pressure readings were ironed out with time (§ 54 p. 323).

190. Dr Bishop had drawn attention to the effect of coarse and fine porous stones on the measurement of pore water pressure. This was clearly demonstrated in Table 1b on p. 312, but although he appreciated that coarse stones tended to measure pore air pressure and not pore water pressure, Mr Akroyd did not think this demolished his thesis which was that in a rolled fill dam using uniform material, it may be possible to achieve better construction control if both compaction and pore water pressure were linked to voids ratio. This thesis depends upon the assumption that subsequent pore water pressures are, in rolled fill, directly influenced by the initial voids ratio after compaction. This could not be true if irregularities in the resulting pore pressures were ironed out with time but from Mr Akroyd's interpretation of Fig. 34 it seemed that this was not so.

Mr P. O. Wolf (Reader in Hydrology in the University of London, Department of Civil Engineering, Imperial College of Science and Technology) said that he would touch on two subjects which had not so far been dealt with in the discussion on Messrs Kennard's Paper: hydrology and hydraulic design.

192. The purpose of the construction of the reservoir was to impound water so as to increase the "reliable yield" of the catchment. More such works for water storage in the area would be required. He hoped that the Authors would be entrusted with those developments, and had little doubt that they had made a study of the water resources available both wider in areal extent and more refined in technique than their brief introductory paragraphs might suggest. If considerations of space had led to such undue brevity, Mr Wolf hoped that the Authors would expand their account of a water-resource study which would be of especial interest against a background of the current national conservation efforts.

193. While on the subject of storage and yield, he wished to raise a question suggested by Dr Bishop's introductory remarks on the groundwater flow from the hillside adjacent to the dam. The geological description indicated that there were some permeable strata present. The effective surface storage of the reservoir might be supplemented by underground storage opened up by shafts, adits, or boreholes (both vertical and lateral). That kind of development was most economically done when a contractor had his construction force and plant in the area for the major work of building a dam. The cost of 3000 m.g. of surface storage was approximately £3m, and it would seem likely that more than a 10% increase in storage volume would be obtainable by the expenditure of £300,000 on wells etc., along the sides of the reservoir, enabling the underground reservoir to fill and empty as the surface water level in the reservoir rose and fell, but with a time lag which would be advantageous in periods of drought.

194. Another hydrological problem, that of the estimation of the spillway design flood, appeared from § 61 to have been solved on the basis of the Institution's Floods Report of 1933. That study had been completed by the Authors before, early in 1955, they consulted Mr Wolf on the hydraulics of the spillway. The senior Author had taken a keen interest in the subsequent discussions on the 1933 Floods Report, and it would be of assistance to the current work on design floods if he could indicate whether he would use the same design methods and arrive at the same design discharges as those of § 61 if he had to design the same scheme in 1962.

195. Mr Wolf's second subject, the hydraulic design of the flood spillway, was referred to in §§ 61 and 62 of the Paper. Figs 2, 15 and 17 showed the layout in plan, the general appearance of the inlet end, and a detail of the spillway channel as constructed.

196. The Authors had designed the spillway, on the basis of the best techniques then available, so as to ensure that, at the worst predictable flood discharge,

(*a*) the rise in the reservoir water level would be small,

(*b*) the structure of the dam would be protected against damage from flowing flood water, and

(c) the discharge velocity into the stilling pool would be so small as not to cause any damage to the valley sides.

197. Requirement (a) was based on the economic need to keep the freeboard on the dam to a minimum, and was satisfied by the provision of a weir nearly 400 ft long. Dr Bishop had shown a slide of the unstable slope on the south side of the stilling pool opposite the discharge end of the spillway channel, which illustrated the importance of requirement (c).

198. In collaboration with Professor J. R. D. Francis and with the assistance of Mr R. B. Batty, Mr Wolf had been able, by means of investigations on two scale models, to suggest the hydraulic modifications touched on in § 63. The weir profile had been chosen to give a high discharge coefficient and proved to satisfy requirement (a). The re-alignment of one end of the weir was found to lead to an improvement in the flow pattern downstream. One of the most useful results of the model tests was the discovery that the width of the spillway channel at the upstream end could be reduced by more than 24 ft, with considerable savings in construction costs, without a significant increase in reservoir top water level.

199. The maximum height of the water level reached in the channel was found to be greater than had been calculated. That was due to the simplifying assumptions in the analysis which led to an estimate of the steady mean conditions, whereas on a model instantaneous maxima could be observed. Requirement (a) led to a raising of the under-side of the road bridge over the channel so that, even at peak flow rates, there would be a free discharge without an obstructive orifice action. Requirement (b) led to a corresponding raising of the channel walls so as to ensure that there would not be any spilling out of the concrete-lined canal.

200. Mr Wolf showed a number of slides illustrating the pleasing appearance of the spillway in its setting and the conditions of flow when he had visited the site in September, 1961. The rate of overflow had been a little heavier than that illustrated in Fig. 17, but the effect of the steps in the channel floor was still clearly visible. The model tests showed that, at discharges of several thousand cusecs, the effect of the steps in the floor was negligible. The pilot model used for that part of the investigation (linear scale 1:24, in a straight rectangular channel 24 ft long, 6 in. wide and 7 in. deep) indicated that the steps would have to be of complex and costly design to lead to an appreciable energy dissipation along the channel. The simple steps as shown apparently represented the cheapest form of construction.

201. Several slides of the downstream end of the spillway channel showed the solution to two unexpected difficulties. As originally designed, the spillway channel gradually widened, in plan, and the calculations led to the expectation of a hydraulic jump in the diverging part ("bellmouth"). In the model the flow was found to concentrate in a narrow jet on one side of the "bellmouth" terminating in a jump in the stilling pool and leaving the other side of the "bellmouth" available for a slowly rotating mass of water. The side to which the jet attached itself was unpredictable, although stable, and apparently not connected with the asymmetrical conditions upstream. Several remedies were tried on the model and found to ensure an even distribution of flow and a hydraulic jump in the "bellmouth" The cheapest solution, the "chevron" cross wall developed by Mr Batty, was adopted. At the low flows illustrated in Mr Wolf's slides, the "chevron" wall formed a smooth stilling basin. At high discharges, the wall acted as a deflector which threw the water nearly 30 ft high (Fig. 39*) and let it dissipate its energy as it fell into a larger stilling basin downstream of the "chevron" wall. There the second unexpected difficulty arose. A special study was required to find a shape of stilling basin that would ensure that all along its rim the flood water entered the stilling pool at velocities below 3 ft/sec.

202. Figs 40 and 41 illustrated the main model which consisted of an undistorted reproduction to a scale of 1:100 of a part of the main reservoir, the overflow weir and

* Figs 39–41 are photographs and are placed between pp. 752–753.

side channel, the spillway channel and "bellmouth", and the main part of the stilling pool.

203. The Consulting Engineers gave instruction for the model investigation in March 1955, the main conclusions were available in August, and the final report was ready in October of that year. The great Osborne Reynolds had stated that a model of that type was an indispensable means of proving a hydraulic design. In the present case it was clearly seen to be so.

Mr R. M. Wynne-Edwards (Director, Constructors John Brown Ltd) said that these two excellent Papers were particularly valuable because they had described a full-scale piece of research. Civil Engineering works are nearly all one-off and so ought to be used as a field for full scale research. Valuable contributions to engineering knowledge would result if engineers responsible for new works would treat them as a field for research, decide what information could be usefully obtained, arrange in the contract for appropriate measurements to be made and write down what had been learnt.

Mr N. J. Ruffle (Engineer for New Works, Sunderland and South Shields Water Co.) said that these two Papers had formed a notable contribution to the Proceedings of the Institution, combining, as they did, such valuable data on the developing techniques that were transforming the design and construction of earth dams into highly specialized exercises in applied science. The techniques and the associated processes of thought would in the future allow the utilization of dam sites previously thought to be impracticable. So far, indeed, had this development proceeded, that the Authors, Bishop and Vaughan, had been emboldened in their § 14 to question the criteria applied to the inspection of existing dams. Going further, could it be maintained that inspection should not include a review of the factors of safety? If one accepted that argument it followed, he thought, there would be a need for borings, testing and piezometric measurements on existing dams. Proceeding to the logical conclusion, it meant that the advice of soil mechanics specialists would be required for future inspections.

206. Particularly in respect of uplift pressures one suspected that there were dams in existence with low factors of safety. This need not in itself cause concern, providing that the existing paths of relief, whether man-made or natural, remained open. In that connexion he wished to ask the Authors of the first paper what materials were used for lining the relief wells and whether they saw clogging of the screens, from corrosion or other cause, as a possibility in the future. From similar considerations he queried the wisdom of connecting a 4 in. drain to the head of an 18 in. borehole under the centre of the embankment (§ 18). By what means was the adequacy of this 4 in. drain assessed, and was it considered that it would provide a free outlet for all time? Would the Authors agree that it was preferable, where possible, to site relief wells in accessible positions, for example, near the downstream toe?

207. If seepage paths were long, clogging of the relief installations might not result in significant reductions of discharge. Direct measurement of the piezometric pressures was the reliable indication of uplift, and he wished to know whether such measurements would be a permanent feature of the maintenance of Selset Reservoir.

208. With regard to § 35, he was not clear how the sand drains allowed economies in stone in the upstream toe and in the base blanket. Would the Authors elaborate on that and explain the reason for constructing such a large upstream toe?

209. In § 33 there was reference to segregation of natural sand falling through water. It was true that the various sizes of a graded sand falling through water would travel at differing velocities but, providing the sand was fed into the water at a constant rate, the faster moving coarse particles would catch up the slower moving finer particles that had been fed in earlier, and apart from the few lowest feet at the base of the drain the effect was to produce "the mixture as before". This constant feed rate had to be such that the upward velocity of the displaced water would not wash away the finest particles which it was desired to retain. These principles were being employed at the

present time in the construction of sand drains up to 100 ft in depth for the Derwent Reservoir.

210. Had the Authors considered opening out the spacing of the sand drains towards the toes? The pore pressures set up by the lateral thrust of the puddle core were presumably small under the berms and toe slopes, and the pore pressures due to vertical loading had more than a year longer to dissipate than those under the centre of the bank. Would it not therefore have been logical to adopt larger spacings in those areas?

211. Finally, what was the piezometric pattern in the downstream shoulder of the dam, now that it had presumably stabilized under operational seepage conditions?

The following contributions were received in writing:

Mr S. Rodin (Head of Advisory Section, Central Laboratory, Messrs. George Wimpey & Co., Ltd) wrote that when investigating problems of dams constructed on very soft alluvial foundations, he had found that three foundation factors in particular were most difficult to evaluate with certain confidence as to their reliability.

213. First, if the stresses in the foundations were based on the theory of elasticity, the calculated stresses under the centre of the dam were higher than if a plasticity theory was used, whereas under the toe lower values were obtained from the theory of elasticity. It would be interesting if the Authors could comment on the method used to calculate the stresses in the foundations at Selset.

214. The second factor concerned the determination of the pore pressure coefficient $\bar{B}$. The value of $\bar{B}$ was dependent on the applied stresses and on the stress history of the deposit. The Authors gave a great deal of useful information on the evaluation of $\bar{B}$ for the fill material by means of laboratory tests and also on the values deduced from the field piezometer observations. But they gave no reference to the value of $\bar{B}$ for the foundation deposits. Some idea of $\bar{B}$ could be obtained from the piezometer observations shown in Fig. 22 (p. 339) for piezometer 18, which was situated under the centre of the dam. Some rough calculations based on these observations indicated that $\bar{B}$ ranged from about 0·75 to 0·9 during the three construction seasons. Could the Authors comment on the values of $\bar{B}$ to be deduced from the piezometer observations in the foundation deposits, and how did they compare with the values assumed in the design?

215. The third factor concerned the rate of pore pressure dissipation during construction. His experience was that the results of the laboratory tests should be treated with caution when extrapolating them in relation to the overall drainage conditions of the natural deposit. This was indicated recently when investigating the construction of a small dam located on a thick layer of very soft silty clay. Although it was an alluvial deposit, the clay had no obvious stratification, and the laboratory consolidation characteristics of some horizontally cut specimens were the same as the corresponding vertical specimens. Yet the coefficient of consolidation deduced from field measurements of pore pressure under an embankment section was much higher than the laboratory value, viz:—

	Laboratory c_v	Field c_v
Range:	0·2–2	2–6 sq. ft/month
Average:	0·5	3 sq. ft/month

The ratio between the minimum field and laboratory values was 10, while the ratio between the average values was 6. In this particular case, the factor of safety at the end of construction was approximately 1·0 on the basis of the average laboratory c_v and 1·15 on the basis of the average deduced field c_v.

216. From Fig. 19 (p. 334) the values of c_v deduced from the field measurements of pore pressure varied between 3 and 9 sq. ft/month. It was reasonable to assume that

the pile-driving caused some disturbance to the natural deposit. The effect of disturbance of a soil on its consolidation properties was usually to reduce the coefficient of consolidation. Therefore, the value of c_v for the undisturbed natural deposit might possibly be higher than 3 to 9 sq. ft/month, whereas a value of $c_v = 0 \cdot 7$ sq. ft/month had been used in the design calculations. If the higher values of c_v were used in the design what would be the factor of safety for the case of construction with no sand drains?

217. He could not see any reference by the Authors to a test embankment to check the actual rate of dissipation in the natural deposit. Such a test embankment would have been invaluable as a means of evaluating the efficiency of a sand drain installation prior to the main construction.

Mr W. G. Craig (Taylor Woodrow (Overseas) Ltd, formerly site engineer at Selset, Balfour Beatty & Co. Ltd) wrote that his contribution would amplify the Authors' paragraphs dealing with the construction of the cut-off trench and spillway channel, and describe some of the problems encountered.

219. The method of excavating the cut-off trench described in the Paper was used across the valley floor, a little way up the north hillside and throughout the south side. The timber walings, hung from the surface and strutted by pit props, retained 6-ft-long steel trench sheets against the trench sides. Sheets were inserted in the sandstone and limestone strata only where the rock was loose.

220. In the deepest sections of the trench and up the south hillside, excavation was carried out up to 9 months in advance of the concrete. Inward movements of the trench sides during this period caused considerable difficulty in extracting the sheets immediately prior to concreting.

221. The hoist towers were adapted for hauling up sheets and timber during concreting but the method was not ideal and cranage was invariably resorted to.

222. In the north section of the trench standard 9 in. × 3 in. sheeting with vertical walings was used, excavating on 5-ft faces direct into crane skips. This method showed considerable reductions in the cost of stripping prior to concreting, and was much cleaner and more efficient.

223. During continuous severe weather in the winter of 1957–58, the trench sides became affected by frost as deep as 50-ft and thawing out by braziers was necessary for up to 36 hours before concreting.

224. The trench bottom was sealed with a 12-in. layer of ¾-in. aggregate concrete, the main concrete being 1½-in. gravel aggregate. Despite efforts on mix design, pipe layouts etc., the 6-in. ram pumps refused to pump the ¾-in. aggregate more than 400 ft, although distances up to 1,100 ft were possible with the 1½-in. aggregate. This necessitated keeping access open in winter conditions to both north and south ends of the trench for trucking in the sealer concrete. Savings would have been possible if the trench concrete had been of standard 1½-in. aggregate throughout.

225. Excavation for the Spillweir and Spillway channel and construction of the walls, which were masonry-faced mass concrete, presented no problems of note. An interesting feature of the overflow arose at the junction of the Spillweir and the channel; the large granite slabs forming the steps to the weir joined the curving, battered, masonry-faced abutment to the road bridge.

226. The channel invert, of maximum width 45 ft, was divided into up to 3 strips longitudinally, with transverse joints at up to 30 ft intervals. The slab was 18-in. thick and the specification called for a 1½-in. crushed whinstone aggregate/river sand concrete. Previous experience of pumping this mix into the diversion tunnel works had not been successful at distances over 400 ft.

227. Due to difficulty of access to a channel 2,000 ft long, and plant availability problems, pumping was the obvious solution to concreting the invert slab. In order to avoid more than four pumps, two at the centrally placed batching plant and two re-mix pumps, a maximum pumping distance of 900 ft was necessary.

228. Tests on the effectiveness of additives in the concrete to increase the pumpable

distance were carried out and a non-detergent air entraining agent adopted. Concrete incorporating this agent was approved; cube results showed a density decrease of 2½%, but no variation in strength, compared to the normally gauged concrete. Tight control of quality was exercised at the batching plant where initial entrained air was 5½% by volume. After passing through two 6 in. ram pumps and up to 1,200 ft of pipeline, volume of air in the slab concrete was 3½%.

229. No trouble was encountered in pumping this concrete 800 ft and distances up to 1,000 ft were pumped on occasions. 400 ft was still found to be the maximum for the normal gauge concrete.

230. The texture of the finished surface of the invert was extremely smooth. During the construction period of 3 months no surface cracking or crazing was anywhere evident. It would be interesting if the Authors could comment on the state of the surface now, 3 years after placing.

Mr J. K. Hunter, wrote to add to his original remarks that in §§ 13 and 14 of Bishop & Vaughan's Paper a general reference was made to the factor of safety of embankment dams and the Authors had enquired whether a theoretical margin of 50% (i.e. factor of safety 1·5) was adequate for an important water-retaining structure.

232. The answer to such a question must surely depend on the engineer's judgment as to whether the calculated safety margin reliably reflected the actual state of the embankment, and on the view he took of how this state might change over the years—first, as the result of the natural processes of decay inescapably associated with man-made works, and secondly, as the result of possible neglect by the owners. The latter would be particularly important should the stability of the embankment be dependent, to a material extent, upon the continued functioning of a drainage system.

233. The Authors' reference to the inspection of old embankment dams was very pertinent since no engineer, however experienced, could assess with certainty the margin of safety of an embankment by mere visual inspection.

Mr D. H. Trollope (Reader in Civil Engineering, University of Melbourne) wrote to say that the Authors of the second Paper in their analysis of the failure criteria applicable to the Selset embankment, had adopted the principle that, at failure, the distribution of stresses on the critical surface was given with sufficient accuracy by the "modified slices method" (Bishop § 7).

235. This analysis, in common with all methods that ascribed rigid plastic behaviour to the material concerned, could not indicate the stress and deformation conditions that existed prior to collapse in accordance with the selected mode of failure.

236. With this in mind, Mr Trollope had attempted to develop an alternative approach to the stability problem by considering possible equilibrium body force distributions.

237. There was a considerable weight of evidence, from both field and laboratory, to support the hypothesis that the equilibrium plane stress distributions within an earth or rockfill embankment could be predicted from the statical analysis of the idealized discrete solid model[38].

238. Present interest centred around the reported behaviour of the core and foundation of the Selset embankment and an analysis in terms of the idealized model might be of interest. It would be assumed that the main part of the embankment (i.e. excluding the core) was everywhere of adequate strength.

239. The plane stress-distribution within a triangular wedge could now be expressed in general form in terms of the three stress vectors (σ_x, σ_y, τ_{xy}) at a point as defined in Fig. 35:

$$0 \leqslant x \leqslant y \tan\theta$$

$$\sigma_x = \frac{\gamma y}{1+k}\tan\theta\cot i\left[1-k\cdot\frac{\cos(\theta+i)}{\cos(\theta-i)}\right]^*$$

* These expressions include a correction to the previously published values of σ_x (Ref. 1).

$$\sigma_y = \frac{\gamma}{1+k}\cos\theta\left[\frac{x(1-k)}{\sin\theta}+\frac{2ky\cos i}{\cos(\theta-i)}\right]$$

$$\tau_{xy} = \frac{\gamma x}{1+k}\left[1-k\cdot\frac{\cos(\theta+i)}{\cos(\theta-i)}\right] \quad . \quad . \quad (1)$$

$y\tan\theta \leqslant x \leqslant y\cot i$

$$\sigma_x = \frac{\gamma}{1+k}\sin\theta\cot i\cos i\,(y-x\tan i)\left[\frac{1}{\cos(\theta+i)}-\frac{k}{\cos(\theta-i)}\right]^*$$

$$\sigma_y = \frac{\gamma}{1+k}\cos\theta\cos i\,(y-x\tan i)\left[\frac{1}{\cos(\theta+i)}+\frac{k}{\cos(\theta-i)}\right]$$

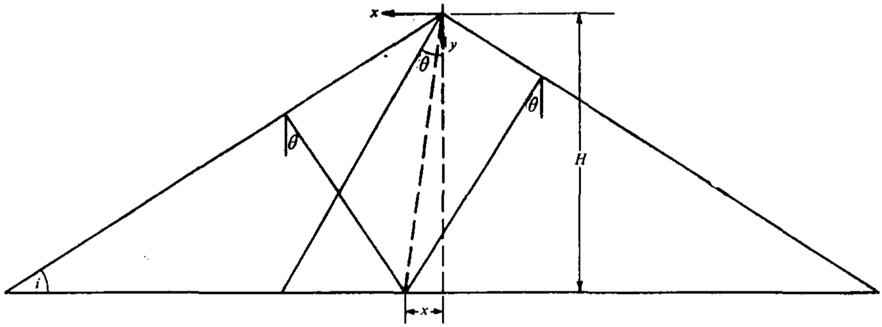


FIG. 35.—FORCE DISTRIBUTION LINES FOR WEDGE SOLUTION

$$\tau_{xy} = \frac{\gamma}{1+k}\sin\theta\cos i\,(y-x\tan i)\left[\frac{1}{\cos(\theta+i)}-\frac{k}{\cos(\theta-i)}\right]$$

There were good reasons for adopting a value of 30° for the distribution angle θ (i.e. the individual units of the model became circular disks) and, for the Selset embankment, a mean value of $\tan i=\frac{1}{4}$ might be taken without significant loss of accuracy. Substituting these values for θ and i in Eqn 1 the limiting systematic stress distributions for $k=1$ (no arching) and $k=0$ (full arching) were shown in Fig. 36.

240. It would be noted that, for these stress systems to be developed, the average shear strength on the centre line—the location of the puddle core—was required to be

$$\tfrac{1}{2}(0{\cdot}88-0{\cdot}29)\,\gamma H \approx 5{,}200 \text{ lb/sq. ft } (k=1) \quad . \quad . \quad . \quad . \quad (2)$$

or

$$\tfrac{1}{2}(2{\cdot}30-0{\cdot}00)\,\gamma H \approx 20{,}000 \text{ lb/sq. ft } (k=0)$$

These values were many times the average shear strength of the puddle core at Selset (330 lb/sq. ft), and further, the maximum permissible principal stress difference (660 lb/sq. ft) was only a very small fraction ($\sim$4%) of γH [135 × 130 lb/sq. ft]. For the purpose of this analysis, therefore, one might write $\sigma_{y(x=0)}=\sigma_{x(x=0)}$. Equating the expressions for σ_y and σ_x (Eqn 1) and putting $x=0$ one found that $k=0{\cdot}67$ and, theoretically

$$\sigma_y = \sigma_x = 0{\cdot}7\gamma y \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

241. The observed values of the major principal stress in the core (σ_1)† might be computed from equation 3 and Fig. 17 of the Paper i.e.

$$u = u_0+\bar{B}\Delta\sigma_1$$

and from Fig. 17

$$u_0 \approx -15 \text{ ft of water } (P_1 \text{ and } P_2)$$
$$u_0 \approx -8 \text{ ft of water } (P_3)$$

† It was implied that $\sigma_1=\sigma_x=\sigma_y$ at $x=0$.

again from Fig. 17

$$u_{max} = 33 \text{ ft of water } (P_1 \text{ and } P_2)$$
$$u_{max} = 25 \text{ ft of water } (P_3)$$

242. Assuming $\bar{B}=0{\cdot}8$ at the maximum recorded pore pressure—a value that would appear to be reasonable in the circumstances

$$\left.\begin{aligned} \Delta\sigma_1 &= \frac{33+15}{0{\cdot}8\times 92} = 0{\cdot}65\ (\gamma y)\ (P_1 \text{ and } P_2) \\ \Delta\sigma_1 &= \frac{25+8}{0{\cdot}8+54} = 0{\cdot}76\ (\gamma y)\ (P_3) \end{aligned}\right\} \quad . \quad . \quad . \quad . \quad (4)$$

These values were close to the predicted value of $0{\cdot}7\gamma y$ (Eqn 3).

243. The above analysis was obviously only approximate. In particular, no account had been taken of the increase in shear strength which would follow the dissipation of pore pressure in the core corresponding to a value of $\bar{B}=0{\cdot}8$, although this factor was not likely to be of major significance.

244. If now, however, account was taken of full pore pressure dissipation ($\bar{B}\rightarrow 0$) one might obtain some interesting results.

245. Adopting the effective stress parameters $c'=0$; $\phi'=24°$ for the core (Fig. 36) there were two possible relationships between σ_x and σ_y according to whether one assumed $\sigma_x>$ or $<\sigma_y$ (c.f. Rankine active and passive earth pressures).
Thus

$$\sigma_x = \sigma_y \frac{1+\sin\phi'}{1-\sin\phi'} (\sigma_x > \sigma_y) \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (5)$$

or

$$\sigma_x = \sigma_y \frac{1-\sin\phi'}{1+\sin\phi'} (\sigma_y > \sigma_x).$$

Then if

$$\sigma_x > \sigma_y; \quad k = 0{\cdot}39 \text{ and } \sigma_{x(x=0)} = 1{\cdot}18\gamma y.$$

or if . . . . (6)

$$\sigma_x < \sigma_y; \quad k = 0{\cdot}94 \text{ and } \sigma_{x(x=0)} = 0{\cdot}36\gamma y.$$

The worst case for the foundation was that given by $\sigma_x > \sigma_y$ as the increased lateral thrust must have been resisted by shear along the base of the embankment.

246. With $\sigma_{x(x=0)}=1{\cdot}18\gamma y$ (Eqn 6) the average shear resistance required in the foundation was given by

$$\tfrac{1}{2}\times 1{\cdot}18\gamma H^2 = 4H\times C_u \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (7)$$

whence $$C_u \approx 18 \text{ lb/sq. in.}$$

Alternatively, assuming full dissipation of pore pressure in the foundation and $c_f'=0$; $\phi_f'=27°$ (Fig. 36), the factor of safety against shear along the base of the embankment was given by

$$\tfrac{1}{2}\ 1{\cdot}18\gamma H^2 = \tfrac{1}{2}\gamma H.4H \tan\frac{\phi'_f}{F_f} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

whence $$F_f = 1{\cdot}8.$$

247. If, however, the shear strength of the core (or the dam material) was *underestimated* then the value of $\sigma_{x(x=0)}$ could increase, with consequent increase in the shear stress along the foundation. The theoretical maximum value of $\sigma_{x(x=0)}$ was that when $k=0$. For the Selset embankment this maximum value was $4/\sqrt{3}\ \gamma H$ and the corresponding values of C_u and F_f were

$$C_u = 35 \text{ lb/sq. in.}$$
$$F_f = 0{\cdot}9$$

248. It should be noted, however, that this value of $F_f<1$ did not, necessarily, imply

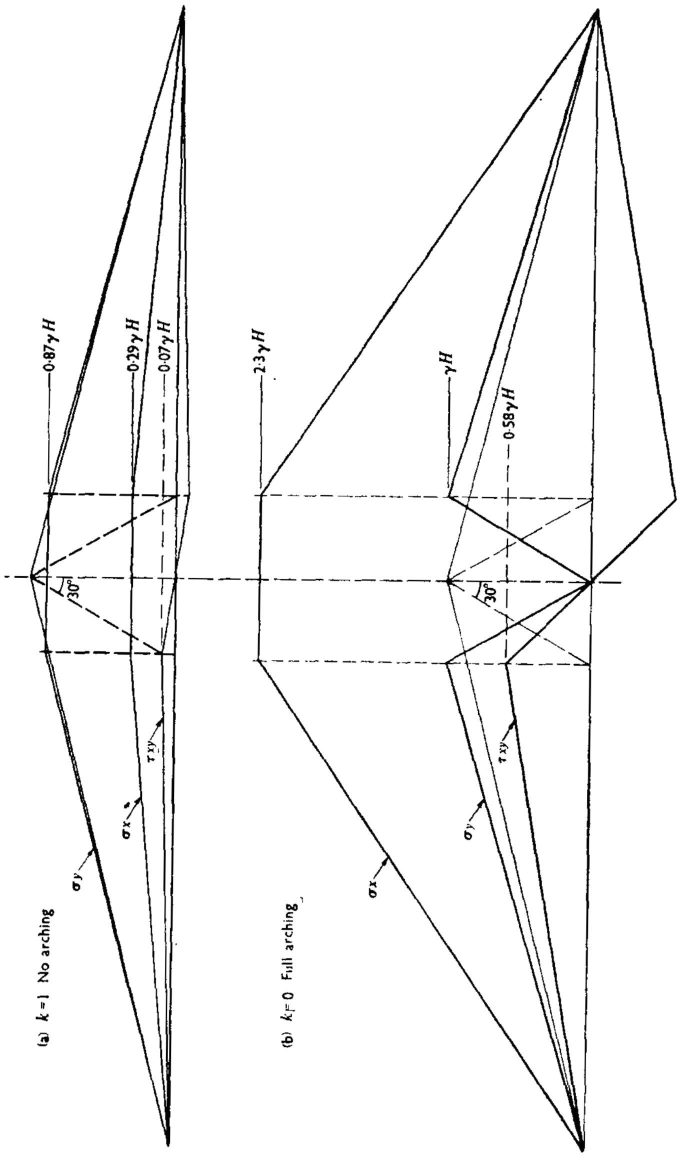


FIG. 36.—LIMITING STRESS CONDITIONS FOR SELSET EMBANKMENT

FIG. 39.—VIEW FROM THE EAST OF A DISCHARGE CORRESPONDING TO 13,000 CUSECS THROUGH THE BELLMOUTH CONSTRUCTED TO THE RECOMMENDED DESIGN. HYDRAULIC JUMP RISES ABOVE THE CHANNEL WALLS ONLY AT THE CENTRE

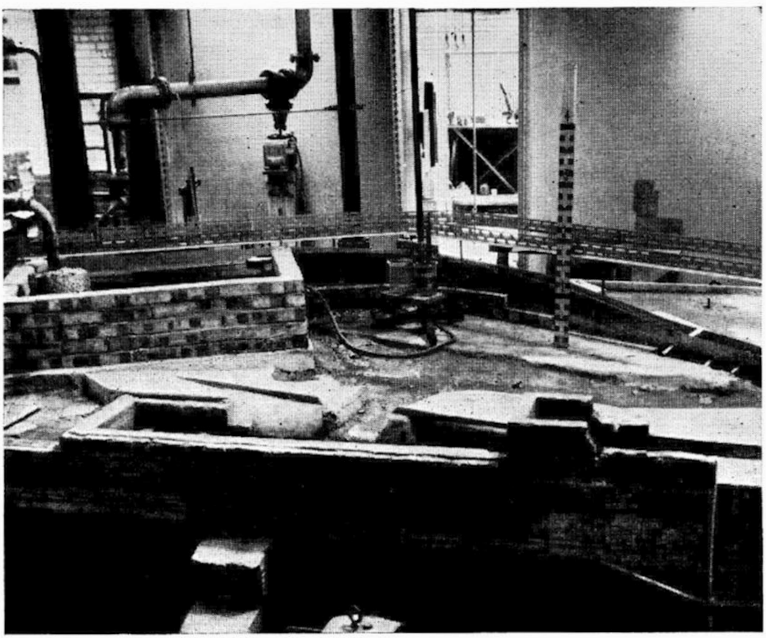


FIG. 40.—VIEW OF THE NORTH-WEST END OF THE MODEL SHOWING THE PART OF THE RESERVOIR REPRODUCED, THE WEIR, SIDE CHANNEL, BEND AND CHUTE

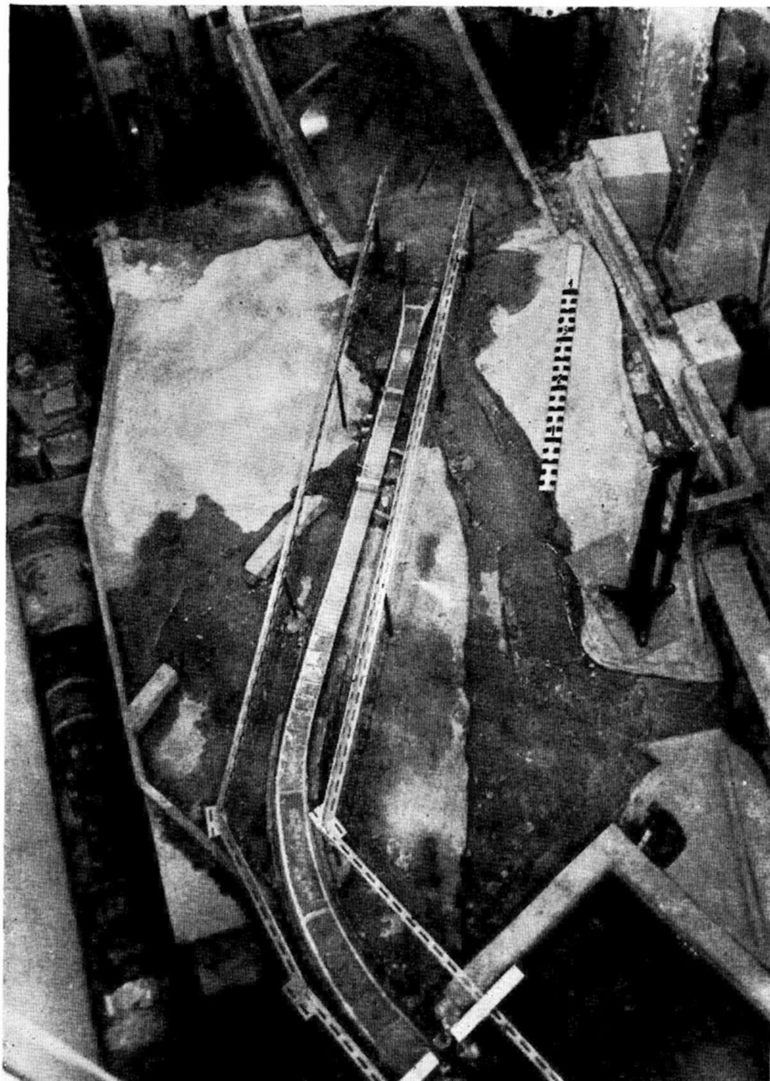


FIG. 41.—PLAN VIEW OF THE SOUTH-EAST END OF THE MODEL SHOWING THE BEND, CHUTE, BELLMOUTH, APRON, STILLING POOL AND HORSE-SHOE WEIR

catastrophic failure of the embankment. Provided the materials concerned exhibited plastic properties then lateral yield would occur, thus reducing the lateral thrust below the $k=0$ value. If the material in the foundation were brittle, however, catastrophic failure could occur.

249. Based on the foregoing analysis, a logical explanation of the behaviour during construction of an embankment, such as that at Selset, could be given.

250. At an early stage, when the fill still had the characteristics of a very wide embankment, shear stresses would be developed near the centre line to such an extent that the full plastic resistance of the core material would be developed.

251. At this stage $\sigma_{\mathrm{y}} > \sigma_{\mathrm{x}}$ and

$$\sigma_{\mathrm{y}} - \sigma_{\mathrm{x}} = 2C_{\mathrm{u_c}} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

where $C_{\mathrm{u_c}}$ denoted the undrained shear strength of the core material.

252. If it was now assumed that the fill was placed in such a manner that drainage was inhibited ($\bar{B}=1$) then for the remainder of the construction period a state of partial arching would exist to an extent governed by the limiting shear strength of the core (Eqn (9)). It would follow that the relatively large "settlements" of the type recorded at Selset during placement of the fill (Fig. 16a, § 59), were in effect large plastic strains arising from yielding of the core.

253. A consequence of this condition was that, for equilibrium, adequate shear strength must be developed along the base of the embankment. At Selset the required average shear strength in the foundation at the end of construction would be 10·6 lb/sq. in.

254. If the available shear resistance in the foundation did not reach this value then failure by spreading would follow. In any case, lateral yield either in the foundation or within the embankment, must occur in order to develop the required shear resistance.

255. It was this lateral yield that permitted the large vertical strains to develop in the core as referred to above. The occurrence of lateral yield was referred to by the Authors in the footnote to p. 326.

256. To turn now to the influence of drainage of the core, it had been noted that theoretically, the lateral stress ($\sigma_{\mathrm{x(x=0)}}$) might either increase or decrease with consequent variations in the vertical stress σ_{y} (Eqn 5).

257. An increase in lateral stress was associated with the further development of arching stresses. This would occur following downward movement in the core owing to consolidation settlement in the core and/or the foundation. It was to be expected therefore that post-construction settlement would induce further lateral yield. While this was not, of itself, critical in relation to the overall stability of the embankment it might have undesirable effects on any structures erected in the vicinity.

258. Concerning the alternative possibility, that of a reduction in σ_{x}, it appeared that this was the most critical factor with regard to catastrophic failure of the embankment. However, unless there was some mechanism whereby the shear strength of the foundation (or of the embankment) was reduced below that developed at the end of construction, no failure was likely. It was there that the effect of uplift or under seepage became critical and the Authors rightly emphasized this factor (§ 98). The concern expressed by Casagrande[29] was timely in this regard.

259. For the Selset embankment, under conditions of full lateral stress relief on the core, the critical value of r_{u} due to under-seepage was 0·85. If the average pore pressure across the downstream half of the dam reached this value then both core and foundation were in limit equilibrium and catastrophic failure would follow. As the Authors pointed out, however, undesirable local failure might occur before this average value was reached. The importance of adequate foundation drainage in dams of this type could not, therefore, be overemphasized.

260. The construction of an embankment such as that at Selset might therefore be called, in some respects, an exercise in "controlled failure". There were many examples in soil and rock engineering where this principle applied.

261. It should be remembered, however, that the analysis on which the preceding argument was based assumed that the materials concerned exhibited full plastic behaviour and it might be reasonably assumed that the soil at Selset fulfilled this condition.

262. Where, however, the fill and/or foundation was in the unsaturated state, a condition that might apply in embankments placed dry of optimium, the material might behave with brittle failure characteristics. In this case the deformation would be associated with severe cracking within the embankment. Casagrande[39] had drawn attention to the occurrence of this cracking at the centre line of a number of dams.

263. It was perhaps pertinent at this point to acknowledge the outstanding foresight of Professor Arthur Casagrande in relation to the problems of earth dams. In focusing attention on cracking in earth dams and the control of under-seepage he had anticipated two of the major problems associated with the need to build higher and higher embankments.

264. Mr Trollope would also like to express his appreciation of the clear and precise presentation of the Paper by the Authors. This clarity greatly simplified the preparation of this contribution.

265. In conclusion Mr Trollope would wish to restate his plea for increased measurements of total earth pressure in addition to pore pressure in earth dams. It was surely unscientific to measure, on a practical scale, only one part of such an important factor as effective stress.

The Authors of the first Paper, **Mr Julius Kennard and Mr M. F. Kennard,** in reply, wished to state that their remarks should be considered in conjunction with the reply of Dr A. W. Bishop and Mr P. R. Vaughan as some of the points raised had been considered by the Authors of both Papers.

267. The puddle core, referred to by Mr Court and Mr Dixon was preferred to a rolled clay core when the dam was designed in 1954, because there were doubts whether there might be stratification of the rolled clay leading to increased horizontal permeability and whether the effect of heavy rainfall might cause clay to be washed out of some layers leading to zones with higher sand and stone contents, which would also cause increased permeability locally. The experience of the rolled boulder clay at Selset was satisfactory and has led to a rolled clay core being specified for the adjacent Balderhead Reservoir now under construction. The measurements of coefficient of permeability of the puddle clay showed that the rolled clay had in fact slightly lower permeability than the puddle.

268. Turning to the cut-off trench, Mr Cuthbertson had asked whether the hoist and monorail method of excavating the cut-off trench showed any advantage over the usual method of using cranes. Mr Court and Mr Craig had both added to the description of this method and it appeared that the more common method of cranes was generally to be preferred due to its greater flexibility. In reply to Dr Rowe, the Authors considered that in the light of knowledge now available there appeared no need to take the cut-off trench through the deep boulder clay. In the valley bottom, however, at the site of one trial pit, no boulder clay was found to overlie the gritstone.

269. Several speakers had mentioned the possibility of deterioration of the efficiency of the relief wells. This point had not been overlooked in the design and the wells were made 12-in.-dia. so as to allow for subsequent drilling inside if this was found to be necessary in the future. In one well, a sample piece of casing had been lowered into the water and could be removed for inspection. No deterioration of this sample or diminution of the flows had been experienced so far. The 4-in.-dia. drain from the 18-in.-dia. de-watering borehole, mentioned by Mr Ruffle was considered to be adequate for the flow to be catered for. A design based on relief of uplift meant that a number of piezometer readings must be read regularly, and a few of the Bourdon gauges were being changed to mercury manometers so that they could be read easily in the future.

270. Several points were raised regarding the sand drains, and in reply to Mr Walters, the filling of the drains consisted of crushed whinstone. Mr Ruffle asked how the sand

drains resulted in economies in the upstream toe and base blanket. These arose because had the clay been removed, the base blanket would have had to be placed over a larger area and the stone toe would either have been longer or deeper. If the upstream toe had not been constructed of the shape designed the tunnel forebay would have had to be positioned further upstream, thus increasing the length of the tunnel. The method of filling sand drains with sand at a constant rate would not be suitable for drains where a casing had to be used and extracted. Because of the uncertainties in the performance of the sand drains, as considered at the design stage, it was not thought desirable to widen the spacing towards the toes. The construction of a trial bank, as suggested by Mr Rodin, was not considered as the time involved would have meant that the advantage of using sand drains to avoid any delay would have been lost. The cost of such an embankment, which could not be within the seat of the dam, would be appreciable. Mr Serota's remarks on the sand drain contract and the Authors' approach to it were appreciated.

271. The siting of relief wells, discussed by Mr Ruffle, depended on the layout of the embankment and ancilliary works, and it was agreed that relief wells should generally be in accessible positions near the downstream toe. This was done in all cases except for the de-watering borehole referred to in § 18. This hole was allowed to act as a relief hole, as, if it had been plugged, it was possible that the plug might not have been completely effective and uncontrolled seepage under the seat of the embankment might have occurred. Dr Rowe had asked about the pressure gradient to the relief wells. The latest levels were in the following table.

TABLE 5.—LATEST WATER LEVEL RECORDS

Date	Point	Water level
19.7.62	P. 19	940·3 ft. O.D.
	P. 20	931·7 ft. O.D.
	B. 23	930·8 ft. O.D.
	W. 8	952·2 ft. O.D.

272. The amount of £10,000 for site exploration costs which Mr Hunter thought very small was in fact only the cost of the initial exploration contract to decide whether the site was a suitable one. A considerably larger amount was expended during the period of the main contract and was included in the "other work" item. The Parliamentary estimate of £3 million was not to be compared directly with the £3·2 million given for the cost of the work, because as stated in § 62 there should be added the cost of land, fees, supervision, etc.

273. The use of masonry, as raised by Mr Court, was decided upon because three other reservoirs belonging to the Water Board had all been constructed with masonry over 50 years ago, and it was the Board's wish to use masonry on this project. There was no doubt that the use of insitu or pre-cast concrete would have been cheaper.

274. Mr Penman had asked what had happened to the horizontal strain gauges. These had comprised piano wires attached to buried plates in the embankment and led out horizontally through polythene tubing. There had been delay in placing these gauges and in addition, it was found that the piano wire having been coiled, rested against the tubing and did not give consistent readings. After a period the wires became rusted, notwithstanding oiling on installation. The difficulties mentioned by Mr Penman in the settlement gauges were expected to be overcome in the design now to be used at Balderhead Reservoir, where in conjunction with Mr Penman, these gauges were being made of larger and more rigid tubing.

275. The difficulties due to wet weather had been referred to by Mr Court and the Authors considered that this was one of the most important aspects of the programming

of and tendering for large clay embankments today, and it was of interest to note that it was one of the first research subjects approved by the Civil Engineering Research Council. Mr Cuthbertson had stated that he was surprised at the amount of interference experienced from showers. His experience elsewhere, that scrapers could work during quite rainy weather was appreciated by the Authors, but as stated in § 45, the use of scrapers was prohibited at Selset, as this would have led to wetter material being placed in the embankment. An increase in moisture content would have required a flatter slope and thus it was considered desirable to place the fill as dry as possible.

276. The use of fill with a greater moisture content, was raised by Mr Court from the contractor's point of view, but in general work was stopped in wet weather because the rubber-tired plant was unable to continue to run on the bank without puddling of the clay and causing considerable difficulty, necessitating the removal of the puddled material from the surface. No moisture content limits were specified; the plant in fact set the limit. Had the material been wetter it was doubtful if the relaxation in the rate of construction in the third season would have been possible.

277. Mr Walters' Fig. 27 of the Trentabank trench was a very interesting example of an apparently successful cut-off in unfavourable strata. A few standpipes in the permeable strata would, however, provide useful information on uplift pressures!

278. The grouting was carried out in 10-ft stages, through pipes left in the concrete. The concreting was carried out by pumping and the pipes did not form an obstruction.

279. Mr Wolf had suggested that the considerations of space had led to a curtailing of the information on the hydrology. This was so as the hydrological investigation had in fact represented a very large amount of work. The Selset Reservoir did not fully utilize the whole of the Lune and Balder catchments but the subsequent construction of Balderhead Reservoir, which was designed as a regulating reservoir to provide water in the river Tees for pumping out in the middle reaches, would make full economic use of the catchment areas. The use of relief wells has shown that some additional underground water can be obtained, not only at this site but probably at other sites in the Lune and Balder valleys.

280. In reply to Mr Wolf's question regarding the estimation of the spillway design flood in the light of the subsequent discussions on the Institution's Flood Report of 1933, the Authors considered that as the catastrophic design flood of 13,000 cusecs represented an inflow of 2 to 2·5 times the normal maximum flood inflow, the recently published discussion had not shown this figure to be too low for such a spillway. To arrive at the design flood other methods were also used including the senior Author's "average height of catchment" formula. With an open side channel spillway the estimation of a catastrophic flood was not as critical as with a closed design such as a bellmouth and tunnel. The open spillway channel would carry a flow of 13,000 cusecs entirely within the retaining walls, even allowing for waves.

281. The inspection of old dams, especially under the Reservoirs (Safety Provisions) Act, mentioned by Mr Hunter and Mr Ruffle, was a topic of increasing importance especially as a large number of dams were now of considerable age. The Authors agreed that more than a superficial look at the embankment was required and in certain cases a soil mechanics investigation would be justified. Any instrumentation provided at a dam should be considered and the observations analysed at such an inspection.

282. The Authors were grateful to Mr Craig for his comments describing the construction of the spillway works. No deterioration of the surface of the invert has occurred in three years.

283. Mr Akroyd considered that the percentage air void method of compaction control was preferable to the "percentage Proctor" method used. The air void method had been described as being preferable when the moisture content varied considerably from the optimum. It was known that for this particular fill, the moisture content would not vary appreciably. The air void method had been developed primarily to reduce settlement of road embankments which was a different problem from that of earth dams, where settlement could be accepted. The 1959 figures had to be considered in the light

of Table 1A of Bishop and Vaughan's Paper. In 1959 a large proportion of the fill consisted of the more plastic material from the North borrow pit, which had a lower optimum dry density.

The Authors of the second Paper, **Dr A. W. Bishop and Mr P. R. Vaughan,** in reply, said that not only had much useful data been contributed in the Discussion but that most of the more intractable problems of earth dam design had been raised as well. Even without the excuse of lack of space it was difficult to give an adequate reply.

285. Mr Little had drawn attention to the difficulty which often arose in evaluating the cohesion intercept from the results of standard triaxial compression tests and to the consequent uncertainty in determining the factor of safety against shallow slips. To reduce this difficulty differences of less than 1 lb/sq. in. between the cell pressure and the back pressure required to cause saturation were currently being used in drained tests at Imperial College. This gave even lower effective stresses than were indicated by circle 1 of Mr Little's Fig. 25, but a differential mercury manometer was necessary to measure the small pressure differences[40]. It had been found that the failure envelope was not necessarily a straight line near the origin.

286. The Authors felt that Mr Little was perhaps rather pessimistic in Fig. 26 and § 113 in assuming zero pore water pressure at the surface in conjunction with a pore pressure ratio $r_u = 0{\cdot}55$ throughout the adjacent fill. This implied upward flow right to the surface of the fill with free water running down it. Freshly placed rolled fill had a negative pore water pressure at the surface, particularly where subjected to shear stresses. Though rainfall and redistribution of pore pressure during consolidation might entirely destroy the negative pressures, field measurements of pore pressure at shallow depths in the slopes would be of great value in establishing whether this actually occurred in practice.

287. At Selset the value of the cohesion intercept had proved to be particularly significant in the analysis of stability on drawdown, when the pore pressure at the surface would tend to zero and not to a negative value. Values of factor of safety calculated for various assumptions were given in Table 6. For most of the fill in the upper part of the slope the value of ϕ' was of the order of 30° and hence the margin of safety was adequate under the most severe operating conditions even if c' were zero, subject to the field value of $\bar{B}$ not differing greatly from unity. If higher pore pressures actually occurred, then even the relatively flat slopes at Selset had a lower margin of safety against drawdown failure than was desirable.

TABLE 6

c': lb/sq. ft	ϕ'	Factor of Safety F	
		(a) Without stone facing and with no drainage	(b) With drainage blankets and stone facing, and allowing for calculated drainage during drawdown
0	26·5°	0·87	1·13
0	30·0°	1·01	1·31 (c)
160	26·5°	1·26	1·45
160	30·0°	1·42	1·63 (c)

Notes: (*a*) Obtained from stability coefficients with $H = 54$ ft, $r_u = 0{\cdot}46$ (i.e. $\bar{B} = 1$ during drawdown).

(*b*) Obtained from analysis of drawdown of 45 ft at 2 ft/day with $\bar{B} = 1$ and $c_v = 3{\cdot}0$ sq. ft/month.

(*c*) Obtained by extrapolation.

288. The Authors did not feel that Mr Court was right in describing as a "very rigorous specification" one which only called for a dry density of 95% of the Proctor optimum density (after allowing for stone content) and put no upper limit on water content other than that implied by the need to obtain this density.

289. The relaxation in the rate of placing, from 45 ft to 63 ft in one season, made to permit completion on time, had in any case led to the pore pressures illustrated in Fig. 13 of the Authors' Paper and to the minimum acceptable factor of safety for a short time around completion (1·25–1·50). Had higher pore pressures been set up, completion would have probably been delayed until the following season.

290. Mr Serota's remarks showed that scientific reasoning might be as valuable to the contractor in determining the means to be used as it was necessary to the engineer in preparing the design.

291. Drainage was the obvious and now well established method of controlling uplift pressures in foundation strata, but the Authors agreed with Mr Hunter that if one accepted this, then the stability of the dam depended on the continued effectiveness of the drainage system. This also applied, however, to many of the other features of a dam whose effectiveness was less easily measured or remedied.

292. At Selset permanent provision was made for the observation of uplift as a direct standpipe level at five points downstream of the centre line. The yield of the relief wells could also be measured quite readily. In addition, as described in the accompanying reply by Julius Kennard and M. F. Kennard, the relief wells were placed outside the embankment so that they could be redrilled or augmented by new wells if this ever became necessary. Only the borehole used for de-watering the cut-off trench, and subsequently connected to the drainage system, was inaccessible but its contribution to the total flow was small. In general the Authors were strongly against placing relief wells beneath the embankment unless maintenance was possible from an adit or shaft.

293. It should be added that although the relief wells were necessary to give a factor of safety in excess of 1·5 and to prevent unsightly springs downstream of the dam, calculations indicated that their efficiency would have to be greatly reduced before even local failure of the dam became likely.

294. The Authors agreed with Mr Dixon that a more realistic view of seepage had resulted in recent years from a fuller appreciation of the work, in particular, of Dr Terzaghi and Professor Casagrande. Observations at Selset and at Balderhead had indicated that "solid rock" even including the grouted areas and the beds of shale, had an average permeability of the order 10^{-5} cm/sec., and was therefore relatively permeable compared with the bank fill and puddle core at 10^{-8} cm/sec.

295. Mr Dixon's experiences of the neglect of instruments were unfortunately not unique. Much valuable information had been lost as a result of failure to continue accurate readings of pore pressure and settlement after construction. The effects of damp, frost and neglect could be minimized by using mercury manometers in plastic tubing instead of Bourdon gauges, and the necessity of de-airing the system could be greatly reduced by using nylon tubing and very fine grain ceramic piezometer tips. However, unless either the engineer, or the client or some central authority such as the Building Research Station, accepted responsibility for taking the readings and publishing them, the loss of information would continue.

296. Mr Cuthbertson had emphasized the value of a thorough borrow pit investigation. If this could be made before the job was put out to tender it would certainly smooth the path of both the engineer and contractor, and of their respective advisors. It should also lead to a more efficient use of the available materials than was sometimes possible at present. The Authors would, however, like to see some flexibility retained in all earth moving contracts to permit modifications to be made when the actual performance of the contractor's plant on the various materials under prevailing weather conditions became apparent.

297. Mr Cuthbertson had found that the application of rigorous analysis led to the

use of flatter slopes. This did not necessarily mean that current practice had become over-cautious; it could also mean that at some stage of construction or operation the older cross-sections to which he referred had a margin of safety considered inadequate by current standards.

298. In this connexion Mr Hunter had stated in §§ 231–232 that whether or not a given factor of safety was adequate must depend on the engineer's judgement. The Authors did feel, however, that there should be some more general agreement about the minimum standards to be called for in this respect, which was no less important than, for example, that of spillway capacity, about which the Institution's Flood Committee had published reports.

299. Mr Walters had given an interesting example of a dam founded on a pervious substratum in which the omission of a grout curtain had led to no apparent problems downstream. The effect of impounding on the downstream water table must have been considerable and it was just in such circumstances that a few standpipes would do so much to clarify the margin of safety against uplift.

300. Dr Rowe had drawn attention to some of the factors which might influence the magnitude of pore pressures set up during construction. It was necessary at the design stage to keep a sense of perspective about the relative importance of these and many other factors. It was clear from Fig. 37 that at Selset the extent to which the actual value of coefficient of consolidation in the field agreed with the value obtained from laboratory tests on small samples was of over-riding importance. Undisturbed samples of the boulder clay foundation were difficult to obtain and the limited data had therefore been interpreted rather cautiously.

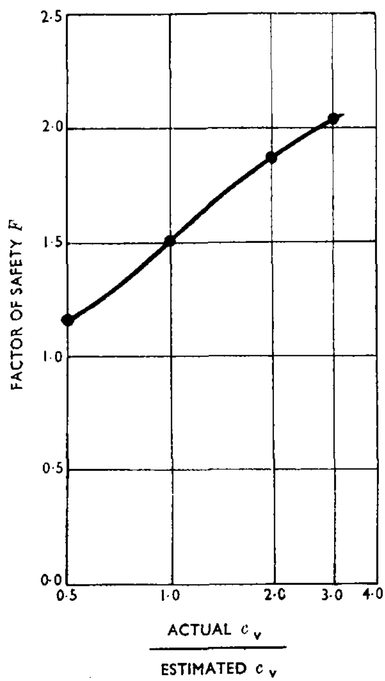


FIG. 37.—VARIATION OF FACTOR OF SAFETY F FROM NOMINAL VALUE OF 1·5 WITH CHANGE IN COEFFICIENT OF CONSOLIDATION c_v IN THE FOUNDATION

301. Had the average value of c_v in the field fallen below the estimated value due, for example, to unrepresentative sampling, it was clear from Fig. 37 that the effect on the factor of safety might have jeopardized the whole scheme.

302. The possible saving in the cost of the sand drain installation envisaged by Dr Rowe would have amounted only to between 1% and 2% of the cost of the reservoir. The necessary additional data could only have been obtained by loading at least one fairly extensive trial area for a season. This was out of the question at Selset but might be an economical procedure elsewhere if the sand drain installation formed a larger percentage of the total cost.

303. Fig. 37 showed that a threefold increase in the value of c_v would raise the factor of safety against foundation failure in the soft clay from 1·5 to about 2·0, other things being equal. The increase to 2·4 referred to in § 77 reflected two additional gains; the percentage success in driving the sand drains was more than the 75% assumed in the design (§ 69) and lower pore pressures were recorded in the puddle core and the zones of fill through which this particular slip surface would have to pass (Fig. 20). Mr Little's experience at Ayer Itam showed that such a favourable combination of circumstances could not always be relied on.

304. Both Dr Rowe and Mr Rodin had raised the question of the pore pressure which would have been set up in the foundation under undrained conditions and of the influence on this of the changes in stress distribution which occurred during construction. In Table 7 values were presented for three locations. The coefficient of consolidation calculated from the observed decrease in pore pressure during the shut-down periods had been used to calculate the amount by which the pore pressure increase during the construction season had been reduced by drainage.

305. The results were expressed in terms of the ratio of Δu_c the undrained increase in pore water pressure thus obtained, to Δh the column of soil vertically above the point considered, expressed as an equivalent head of water. Table 7 showed that this ratio lay in general within the range $1{\cdot}0 \pm 0{\cdot}10$ except for two extreme reading values of 1·16 and 0·86. The increase in pore pressure continued, however, after the fill had been completed above the point considered, due to the spread of load from the remaining fill being placed towards the centre of the embankment. The overall ratios at the end of construction worked out, perhaps fortuitously, at the same value of 1·11. It was of interest to note that the corresponding value calculated at the design stage for the Chew Stoke dam was 1·12 (Ref. 6 of Paper No. 6589).

306. This value reflected the fact that the assumption that the major principal stress was vertical or was equal to the vertical head of soil became less accurate as the embankment grew from a wide almost uniform load to an approximately triangular section. However, the most important phases of consolidation in the foundation occurred in the first two seasons when the simple approximation was accurate enough. During these stages there was no evidence that the effect of shear was as marked as in Dr Rowe's test, nor that significant load transfer onto the sand-drains occurred. This was probably due to the compressibility of the crushed whinstone, which became saturated only after placing, together with the possibility of slight lateral yield. In any case a very low "upper limit" was placed on load transfer by the low resistance to penetration of the clay fill above the sand drains, even if they had been rigid.

307. The Authors agreed that a large drop in pore pressure could occur as shear failure was approached under undrained conditions. However, where the factor of safety did not drop below 1·5 during construction, this was unlikely to increase the apparent coefficient of consolidation by more than the 50% referred to in § 80 and for normally consolidated soft clays might have little effect.

308. Mr Penman had added some useful data about the field observations. The Authors would like to say how much they had valued his co-operation on the site.

309. The Authors agreed with Dr Chaplin that there was still much to be learned about the relationships between the mechanical properties of soils and index properties such as plasticity index, etc.

TABLE 7

Point No.	First Season				Second Season				Third Season					$\frac{\Sigma\Delta u_c}{\Sigma\Delta h}$ (1)
	Δu	Δu_c	Δh	$\frac{\Delta u_c}{\Delta h}$	Δu	Δu_c	Δh	$\frac{\Delta u_c}{\Delta h}$	Δt	Δu	Δu_c	Δh	$\frac{\Delta u_c}{\Delta h}$	
14	31·5	46·6	49	0·95	56·5	87·1	86	1·01	1. 2. 2.	16·0 1·0 −7·0	18·3 12·1 4·6	17. (55) (70)	1·07 (0·19) (0·06)	1·11
17	38·0	51·1	53	0·97	56·3	84·7	87	0·97	1. 2. 2.	14·0 8·0 −9·0	16·5 21·9 2·2	19 (55) (70)	0·87 (0·35) (0·03)	1·11
18	51·5	60·4	57	1·06	61·8	89·6	83	1·08	2. 2. 1.	45·6 20·0 −1·0	53·4 45·4 17·6	46. 53. (40)	1·16 0·86 (0·44)	1·11

Δu = Measured change in pore pressure (ft of water).
Δu_c = Change in pore pressure compensated for drainage (ft of water).
Δh = Change in overburden pressure (equivalent ft of water with $\gamma = 135$ lb/cu. ft). Bracketed figures denote change in height of dam when not above point (expressed as equivalent ft of water).
Δt = Successive time intervals during third season over which changes considered (months).

Note: (1) $\Sigma\Delta h$ refers only to fill above the point considered.

310. The Authors welcomed the valuable records which Dr Gilbert had produced and hoped that other Water Authorities would follow Swansea's example. Although instrument lag appeared to offer the simplest explanation of the delayed response to water level changes, calculations showed that this explanation could not in fact be sustained even if gas completely filled all the connecting tubes. The results therefore indicated that the upstream fill was still not fully saturated. Whether it would ever saturate fully was uncertain. It was possible that gas detected in the piezometers at Usk during construction resulted from the decomposition of the organic content of the fill, and would continue to be produced during use. It would be of great value to continue these observations during a major drawdown so that their implications from a stability point of view could be more clearly established.

311. The limited data available to the Authors suggested that this lag, though apparent on several dams, did not always occur.

312. The Authors agreed with Mr Akroyd that a certain amount of re-thinking was needed about the control of compaction on earth dams. They did not, however, feel that this would lead to the abandonment of the Proctor (or Standard Compaction) test. It had a place with the other index tests such as the Atterberg Limits. It provided a valuable means of comparing the probable placement conditions of various soil types. It was not apparent how soils as different in their optimum dry densities as the boulder clay at Selset (121 to 128 lb/cu. ft) and, for example, the residual soil at the Sasumua dam (62 to 72 lb/cu. ft) could be discussed on the basis of void ratio or percentage voids alone (§ 186) especially when the Sasumua soil showed, if anything, the more favourable mechanical properties.

313. Mr Akroyd might have viewed more sympathetically the conclusions drawn from the group of piezometers Nos 39–54 if there had been space in the text for the plan of the test area illustrated in Fig. 38. To avoid short circuiting of the pressure differences through the disturbed soil in the 2-ft-deep trenches containing the polythene tubes to the B.R.S.-type piezometers, the locations had to be spread in plan away from the intersecting vertical and horizontal lines desired. Variations in the properties of the

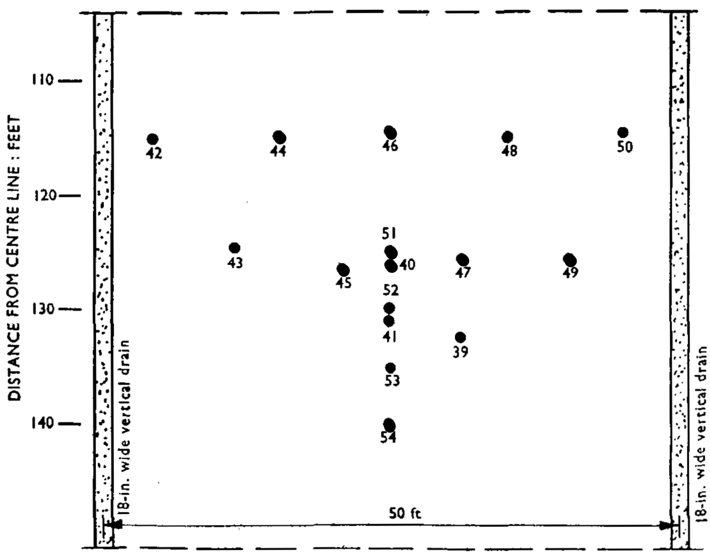


FIG. 38.—PLAN OF PIEZOMETER GROUP 39–54

fill laterally, in which direction relatively little redistribution of pore water appeared to occur, therefore to some extent obscured the effects of the redistribution vertically, in which direction drainage mainly took place.

314. Points 45 and 47 were averaged in which 46 as they were nearer in plan to the "vertical" group under consideration in Fig. 12*a* of Paper No. 6575. Points 39 and 40 remained anomalous, but both were very close to the drainage blanket and 9 ft apart horizontally. The possibility that two sets of connexions had been crossed out of such large total could not be ruled out.

315. However, in September 1958 and June 1959, when the higher pressures minimized trouble due to the presence of undissolved air, the departures from the isochrones expected in a homogeneous fill were certainly diminished in importance, which was all that § 54 of the first Paper had stated.

316. The Authors had not made a detailed study of ways of augmenting average yield by relief wells, but they felt that the additional adits and boreholes envisaged by Mr Wolf might show a larger average yield if placed downstream to collect the natural valley inflow deflected by the presence of the reservoir and to intercept any seepage. The high natural water table appeared to offer little scope for augmenting storage upstream.

317. Most of Mr Ruffle's questions had been answered in replies to Mr Hunter and Dr Rowe. The Authors appreciated his support for their concern about the criteria to be applied to the inspection of existing dams.

318. Mr Rodin had asked about stress calculations used in the foundation at Selset and about the value of $\bar{B}$. Much of the answer was contained in the reply to Dr Rowe. Since the rate of consolidation was the major uncertainty and the coefficient of consolidation was lowest in the first season (Fig. 19), the problem was analysed as a uniform vertical load, to which the bank approximated at that stage, with $\bar{B}=1$. The actual values of pore pressure ratio were given in Table 7.

319. Even if the value of c_v could have been raised at the design stage from 0·7 to, say 5·0 sq. ft/month, the factor of safety would have been less than 1·0 without sand drains unless the soft clay were less than 12 ft in depth. It was known from trial pits that the soft clay was deeper than this in some places. A detailed investigation of its extent would have been costly and time consuming, whereas the driving records of the sand drains automatically provided this information.

320. Mr Rodin had provided more evidence that the field value of c_v might considerably exceed the laboratory value, though in his example the actual effect on the factor of safety was less than at Selset (Fig. 37). It was not clear, however, whether the field results confirmed the conclusion drawn from the laboratory results, that the horizontal and vertical values of c_v were the same. Even without sand drains lateral flow had an important influence on rate of consolidation under loads of limited extent.

321. Dr Trollope had shown that the systematic arching theory[38] led to results which were in qualitative agreement with the behaviour of the plastic core. The Authors were not clear that it would have been of value at the design stage in making a quantitative estimate of the overall factor of safety against sliding, since the parameter k, denoting the extent of arching, could apparently lie anywhere between 0 and 1, and was difficult to relate to the shear strength and width of the core.

322. There was, indeed, a fundamental difference between the meaning given to the term "factor of safety" by Dr Trollope on the one hand, and the Authors and most other contributors to the discussion on the other. This allowed Dr Trollope to refer to § 260 to the construction of the Selset embankment as an "exercise in 'controlled failure' ", while Mr Cuthbertson and Dr Rowe were referring to the design having erred on the safe side.

323. The numerical values of factor of safety given by the Authors referred to plastic yield along a complete slip surface. Dr Trollope referred in § 246 to a factor of safety of 1·8 along the base of the embankment when the shear resistance of the core was fully mobilized (§ 245) implying a factor of safety of 1·0 in that zone. How far the bank was from complete rupture then depended on the relative weighting of the

two terms. The procedure could not be defended logically since even base shear was not uniformly mobilized.

324. Dr Trollope had rightly drawn attention to the fact that an analysis based on plastic behaviour might not be applicable where one or more of the zones showed brittle failure characteristics. This was particularly important where large strains might be expected as, for example in rolled fill embankments on soft clay foundations or subject to seismic displacements.

325. Dr Trollope appeared in §§ 244 and 252 to be using the parameter $\bar{B}$ to denote the amount of drainage rather than the influence of degree of saturation and stress ratio on the pore pressure set up under undrained conditions[23]. This could be very misleading.

326. Post-construction settlements did not seem to lead to appreciable further lateral yield, as predicted by Dr Trollope in § 257, either at Selset or at Chew Stoke[24], where site conditions had made measurement easier.

327. The Authors would support Dr Trollope's plea for field measurements of total stress. Instruments were in fact being installed for this purpose in the Balderhead embankment. Current practice was not, however, as unscientific as was implied. As far as the determination of the overall stability of fairly homogeneous embankments was concerned the average total stress could be determined with reasonable accuracy from statical considerations, while the pore pressure ratio could lie anywhere between 0 and 1 depending on placement conditions and drainage etc., and had therefore to be checked by field measurements.

328. In conclusion the Authors agreed with Mr Wynne-Edwards about the need for using more fully the opportunities presented by civil engineering works for obtaining much-wanted data. Sir Benjamin Baker[41] had drawn attention in the Institution's Proceedings in 1881 to the "present want of experimental data" about "some of the elements affecting the stability of earthworks", and the stage had certainly not yet been reached 80 years later when the situation could be viewed with complacency.

329. The detailed analysis of the field results was mainly carried out by Mr Vaughan while holding a D.S.I.R. Research Studentship and the Authors were glad to acknowledge the Department's assistance.

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