*Discussion on Paper Nos 6816 and 6841**

Shek Pik submarine pipeline, Hong Kong water supply

by

A. P. Goudy, M.A., A.M.I.C.E.

Shek Pik dam

by

W. J. Carlyle, B.Sc., A.M.I.C.E.

Mr C. D. Crosthwaite (Partner, Freeman, Fox & Partners) confined his remarks to the Paper on the submarine pipeline.

170. He was interested in the failure of the polyurethane coating of the pipes and a number of questions had occurred to him about this. First, he wondered why this coating, which was still a largely experimental material, had been used on a pipeline which, once laid, was inaccessible to further examination. In North Wales his firm had been responsible for what was probably the second largest pipe coating operation of this nature, which had involved coating some 3 miles of 9½ ft i.d. pressure tunnel and ancillary pipeline equipment. The two major factors to be considered were first, as smooth a coating as possible, because friction losses had to be avoided and second, a long life. A coal tar epoxy resin was tried first, which had been moderately successful but serious condensation had been encountered. It had not been possible to dehumidify the air in the working area, and, after discussion with the paint contractor, a modified polyurethane was tried. They had found that this, with an appropriate primer, gave a completely satisfactory coating with the exception of the first length on which the modified coating failed in a very similar manner to that described in the Paper. This was before the right proportions for the materials had been established.

171. The failure at Hong Kong, however, had been complete and Mr Crosthwaite wondered if there had been something wrong in the materials themselves. Although he thought great difficulty must have been experienced with humidity, he wondered if there had been some incompatibility between the self-etching primer (as he assumed that it was) and the subsequent coating. In any event, why had lacquer been used? He regarded this as a coating which, by itself, would only attain a thickness of 0·007–0·010 in. even after five passes, which he would have thought was not sufficient. A coating thickness of 0·020 in. $\pm$0·003 in. was required. They had used a pitch-polyurethane coating, and the pitch admixture had enabled them to put this on with three passes.

172. It might be that the material which had been used at Hong Kong would have been suitable if work had been carried out under controlled conditions. Perhaps an attempt might have been made to control the conditions at Hong Kong, by maintaining the pipes, while they were being coated, at a higher temperature than the ambient moisture.

173. He believed that this material, if used properly, had a tremendous future, but a great deal of thought and care must be devoted to its application.

Mr J. D. Humphreys (Senior Civil Engineer, Balfour, Beatty & Co., Ltd) said with reference to Mr Carlyle's Paper that it was stated in Appendix 3 that the compaction

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test in Zones 1 and 2 was carried out by means of core cutters. There was some evidence that this method tended to give higher values of in situ dry density than, for example, the sand replacement method, although the former was admittedly quicker. Had comparisons between various methods been carried out at Shek Pik and was the Author satisfied that the core cutter did not introduce errors by compressing the less well-compacted material?

175. The construction pore pressures had been forecast (Appendix 3) by assuming a value of $\bar{B}(=\Delta u/\sigma_1)$ and then carrying out Gibson's step-by-step analysis for uniaxial consolidation. Subsequently, constant effective stress ratio tests with a selected value of F of 1·5 had been carried out and these had given significantly lower values of $\bar{B}$. Mr Humphreys understood that the Author was not wholly satisfied with commonly accepted methods of predicting construction pore pressures and might therefore care to consider, and comment on, the method used in the design of Kainji dam, Nigeria. Although this dam incorporated rockfill shoulders, the broad earth core was being constructed using a tropical residual soil similar in several respects to the material at Shek Pik and the method to be described could have a general application to earth dams.

176. Bishop and Henkel[6] described the constant effective stress-ratio test for $\bar{B}$ and Fig. 42 was similar, the symbols used having the same meaning.

177. Fig. 42 could be described in trigonometrical terms by the equation:

$$\frac{1+\sin\phi'_m}{1-\sin\phi'_m} = \frac{c'\cot\phi'+\sigma_1-u}{c'\cot\phi'+\sigma_3-u}$$

from which

$$u = c'\cot\phi'+\sigma_3+\frac{(1-\operatorname{cosec}\phi'_m)}{2}(\sigma_1-\sigma_3). \quad . \quad . \quad . \quad . \quad . \quad (1)$$

178. Incidentally, differentiating equation (1) gave:

$$\Delta u = \Delta\sigma_3+\frac{(1-\operatorname{cosec}\phi'_m)}{2}(\Delta\sigma_1-\Delta\sigma_3)+\frac{(\sigma_1-\sigma_3)}{2}\cos\phi'_m.\operatorname{cosec}^2\phi'_m\Delta\phi'_m. \quad . \quad (2)$$

It was interesting to compare this equation with Skempton's definition of the pore pressure coefficients B and A:

$$\Delta u = B\Delta\sigma_3+BA(\Delta\sigma_1-\Delta\sigma_3). \quad . \quad . \quad . \quad . \quad . \quad . \quad (3)$$

Equation (2) was simply the statement of a property of a diagram such as Fig. 42 and depended upon no soils theory. However, comparison of equations (2) and (3) demonstrated the absence from (3) of a term involving $\Delta\phi'_m$—in effect, changes in F associated no doubt with deformation of the soil. The quantity A, therefore, could not be taken as a constant coefficient, nor could the alternative coefficient $\bar{B}$. This was obviously well known to the designers of Shek Pik dam, as demonstrated by their later tests, but perhaps it was not more generally appreciated. The foregoing explanation did not seem to have been published previously.

179. Equation (1) gave a relationship between the pore pressure u, σ_3, σ_1 and F (since F defined ϕ'_m), c' and ϕ' being taken as constant properties of the material.

180. If, therefore, the results of an undrained triaxial test were to be analysed, σ_3 being a constant for the test, then for any selected value of F there existed unique values of u and σ_1 satisfying equation (1) and it was a simple matter to identify them by plotting, in effect, the stress path through the test. (If the results were taken as applying to the fill in the dam, the value of σ_1 would define the depth of fill up to which the method could safely be applied.)

181. For a selected value of F, it was thus possible to calculate a value of $r_u(=u/\sigma_1$ taking $\sigma_1=\sum\gamma h)$ which would satisfy equation (1) and would be conservative to a depth corresponding to the simultaneous value of σ_1. Repeating the

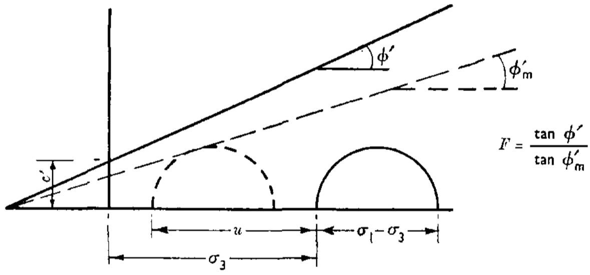


FIG. 42

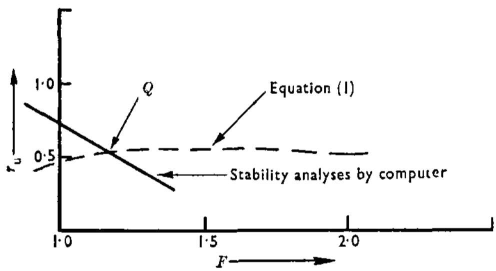


FIG. 43

calculation for different F values then gave a curve of mutually compatible values of r_u and F. Next, a series of stability analyses was carried out in which a range of r_u values was used. F was again plotted against r_u. A typical plot for Kainji dam was shown in Fig. 43.

182. The two curves intersected in point Q which thus defined the only values of F and r_u which both satisfied the simple trigonometrical requirements of equation (1) (which was, after all, only a definition of the terms) and the stability analysis of a particular dam section, F referring to the worst case analysed. In other cases, of course, r_u could be higher or lower but could not then be critical.

183. The actual tests forming the basis of the Kainji calculations had in fact been carried out earlier for more general design purposes and the foregoing theory had to be modified somewhat to take account of the fore-test saturation and consolidation of the test specimens.

184. For this reason only those results were used in which the final moisture content exceeded the initial value, so that the results were practically certain to be over-conservative. Critical failure paths all passed through depths of fill less than indicated by the calculated values of σ_1, so again the calculations were conservative. Typical values of r_u lay in the range 0·3–0·5 with corresponding values of F of about 1·2. Mr Humphreys believed that recorded pore pressures in the dam, at present under construction, would be less than indicated by the calculations.

185. It should perhaps be pointed out that the theoretical basis for this approach was the assumption that the value of F was constant around the postulated shear path —an assumption which was, however, common to most analyses based on pure stress equations.

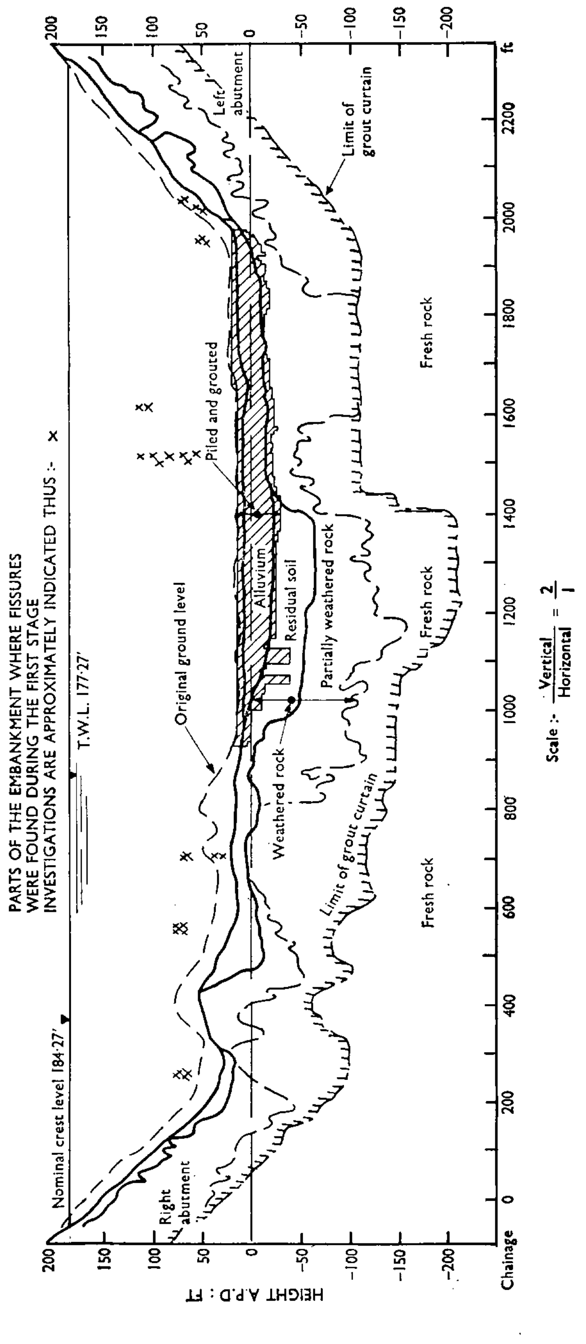


FIG. 44: LONGITUDINAL SECTION OF THE DAM ALONG THE CUT-OFF CENTRE LINE

Mr A. L. Little (Messrs Binnie & Partners) said that the cracking of the core of Shek Pik dam described by Mr Carlyle was a matter of great importance and interest.

187. Fig. 44 was a longitudinal section of the dam showing the cracks discovered in the first stage investigation. The cluster of fissures at chainage 700 had been found in the first stage investigation which the Author described. That was where the trial trenches had been sunk. Two trenches had been dug there and cracks were found which had been filled with grout. There was also an almost horizontal crack, filled with sleeve grout. He had been unable to think of any explanation of the arrangement of the cracks.

188. The Author said that the reservoir had been filled very rapidly in the 1964 season. This must have imposed a very severe test on the dam but it had shown no signs of distress.

189. The cause of the cracking, in quantitative terms at any rate, was still unknown. Many earth dams had undergone much greater deformation than the relatively modest settlement of about 1 ft recorded at Shek Pik in the foundations. At Fort Peck, for example, the settlement had been about 16 ft and although many other phenomena had been observed there, no cracking had been reported.

190. In a paper published in 1963[9] Professor G. A. Leonards and J. Narain had examined data from four earth dams which had developed cracks in service and from one test embankment which had not. They had tested material from all five structures and a sixth material, which they called 'limestone residual clay', which came from southern Indiana and which had been studied in previous work at Purdue University.

191. Table 5 summarized some of the data which had been obtained by Leonards and Narain. There was a considerable range of plasticity, from the non-plastic of Woodcrest and Shell Oil to the extremely plastic limestone clay. However, by excluding the limestone clay, which had been used only in the laboratory tests and not in any of the structures, it would be seen that, where cracking had been observed in the field, the soils involved were all in the low plasticity range. In most cases the fill water content was a good deal lower than the optimum water content, though this was not always so, and in some instances the fill water content was higher. It was notable at Shek Pik that the fill water content was considerably higher than the optimum, and this remained one of the puzzling features.

192. In an endeavour to find out the cause of cracking, Leonards and Narain had made beam tests on prisms of compacted clay, 3 in. wide, $2\frac{3}{4}$ in. deep and $22\frac{1}{8}$ in. long. They coated the prisms with wax to observe the cracks and fitted pins in the sides to observe the movement with a travelling microscope. The strain at which cracking occurred was noted and compared with computed or observed strains in the embankments. Referring to Table 5 it would be seen that the calculated or measured strain was greater than the strain observed in the beam test, and when this occurred cracking took place.

193. There was one important exception. The Woodcrest dam, which according to the theory should have remained uncracked, did in fact crack. The strain in the actual dam (calculated, 0·12%; measured, 0·09%) was considerably less than the strain which caused cracking in the beam (0·20%). It would appear, therefore, that the theory could not be used with any confidence to predict cases where dams would be safe from cracking.

194. The tabulated data also emphasized the fact that it was not possible to rely on plasticity, or the lack of it, or the deformation which took place to decide whether cracking would occur or not. The cracking at Shek Pik remained a mystery, but it suggested that when dams were being constructed with non-plastic material or material of low plasticity the engineer should think very carefully about the problem of cracking in the core; in particular he should pay careful attention to the design of the filters and drainage system.

TABLE 5

Reference	L.L. %	P.I. %	Proctor compaction		Placement		Compaction factor %
			Dry density *per cu. ft*	Opt. *m/c* %	Dry density *per cu. ft*	*m/c* %	
Limestone clay, Indiana	72	45	96	25·9	—	—	—
Portland dam, Colorado	29	8	112	16·3	111	13·5	99
Rector Creek dam, California	38	16	103	19·8	96	18·8	93
Shek Pik dam, Hong Kong	38	12	110	16·1	105	18·5	95
Shell Oil dam, California	Non-plastic		120	11·2	114	12·3	95
Willard dam, test embankment	31	11	110	16·4	105	12·5	95
Woodcrest dam, California	Non-plastic		127	10·2	124	7·2	98

Dr A. W. Bishop (Department of Civil Engineering, Imperial College of Science and Technology) congratulated Mr Carlyle on presenting a very straightforward account of the construction of an interesting and elegant earth dam. As often happened with reports on civil engineering projects, the most interesting and instructive sections were those dealing with unexpected phenomena.

196. It was unexpected to find open cracks at depth in a rolled earth core placed at more than 2% above the optimum water content. It was therefore of interest to see what conditions were known to predispose a rolled earth fill to cracking and whether those or any other special conditions obtained at Shek Pik.

197. For open cracks to occur in rolled fill it was generally necessary for the component of *stress* in one direction to be either tensile or zero. It was not sufficient to have tensile *strains*, because they could be produced in the direction of a compressive stress by the action of a much larger compressive stress at right-angles, and this would not, in general, result in open cracks. The lateral yield in a triaxial compression test was an example of this. It was necessary, therefore, to look for conditions in which tensile stresses might be set up.

198. It was of interest to consider first the simple case of an ideal dam (Fig. 45) which was an elastic structure founded on an elastic foundation having the same Young's modulus. The calculated stresses were positive throughout the whole of the dam and there was no indication of tensile stress. It was therefore necessary to look for tensile stresses to be associated with departures from the simple assumptions made in this analysis, and this was indeed what had been shown by experience.

199. There were three possible classes of departure. First, the problem in general was not one of plane strain, since construction usually took place in a triangular valley and the cross section varied, not necessarily regularly, along the length of the

TABLE 5 (*continued*)

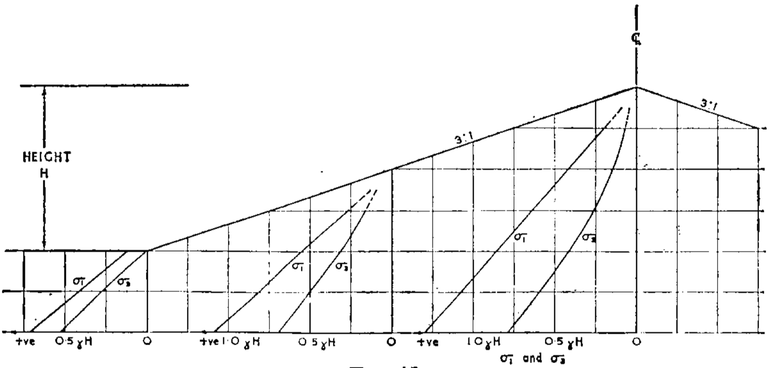
Placement m/c − Opt. m/c	Dam dimensions			Tensile strain at cracking %			Remarks
	Height H ft	Length L ft	L/H	Calculated	Beam test	Field observation	
—	—	—	—	—	0·30	—	Not tested in field
−2·8	35	330	9·5	0·30	0·12	—	Cracked
−1·0	150	900	6	0·29	0·14	0·24	Cracked
+2·4	180	2300	12·8	—	—	—	Cracked
+1·1	54	350	6·5	0·20	0·08	—	Cracked
−3·9	6	800	133	0·17	0·24	—	Not cracked
−3·0	40	1125	28	0·12	0·20	0·09	Cracked

dam. Second, the compressibility of the foundation might differ from that of the fill; and third, the fill itself was not necessarily homogeneous with respect to these properties. These departures could lead to transverse, longitudinal or random cracking.

200. Dealing briefly with these three classes, transverse cracks, which had been dealt with by Leonards and Narain,[9] were found to occur when the longitudinal section was non-uniform (Fig. 46(a) and (b)). Tensile strains occurred in the upper part of the fill over the slopes of a dam placed in a steep-sided valley, and compressive strains occurred above the valley centre. If there was a discontinuity in the slope of the valley (Fig. 46(b)) and the fill was compressible or tended to settle on saturation, the situation was even more acute and cracking was particularly likely to occur. Similarly, if the foundation in the valley floor was compressible or tended to settle on saturation (Fig. 46(c)) there would be a strong tendency for tensile stresses to occur in the slope over the valley side and for transverse cracking to result.

201. A number of cases of failure had been reported, for example by Sherard[10] and by Leonards and Narain.[9] In general the transverse cracks were visible at the surface in this case, and in one dam of this type a complete section had washed out and failure had occurred. This type of crack was generally of the order of ½–1½ in. and corresponded to a surface tensile strain (measured between pegs on the centre line) of something like 0·1–0·2% on the face of the dam.

202. Longitudinal cracks (Fig. 47(a)) occurred in general where the foundation was more compressible than the fill and the foundation under undrained conditions yielded laterally. The tendency to lateral spread in the foundation caused vertical cracks to occur at or near the centre of the dam. A dramatic example of this had occurred after the 1953 floods on the Kent coast in a low bank of rolled chalk on the


HEIGHT
H
3:1
3:1
σ1
σ3
+ve
0·5 γH
O
+ve 1·0 γH
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+ve
1·0 γH
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O
σ1 and σ3

Fig. 45

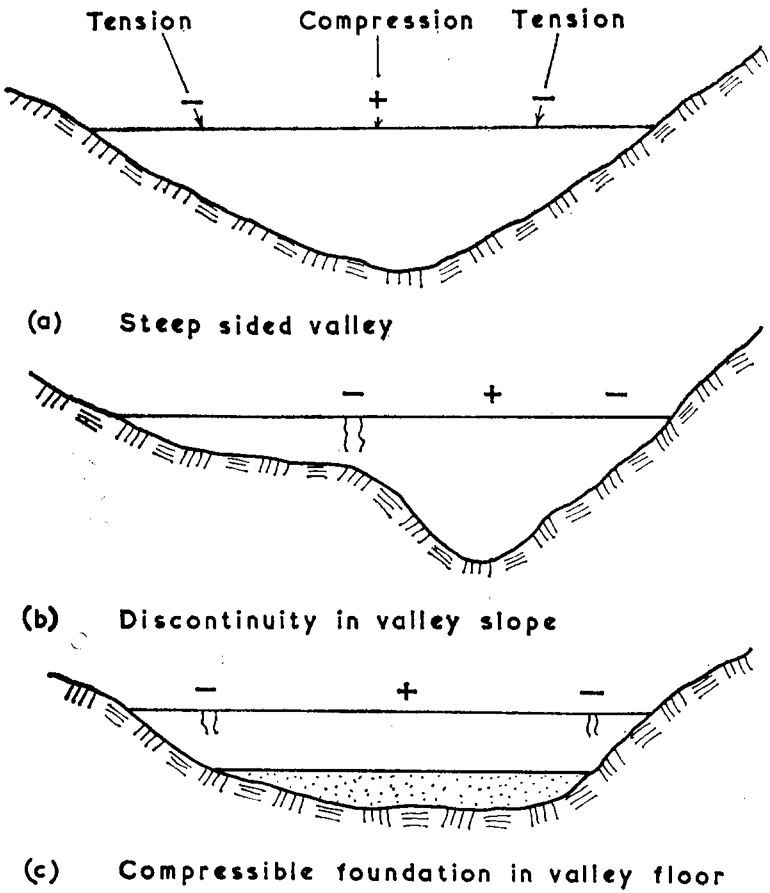

Tension
Compression
Tension
(a) Steep sided valley
(b) Discontinuity in valley slope
(c) Compressible foundation in valley floor

Fig. 46

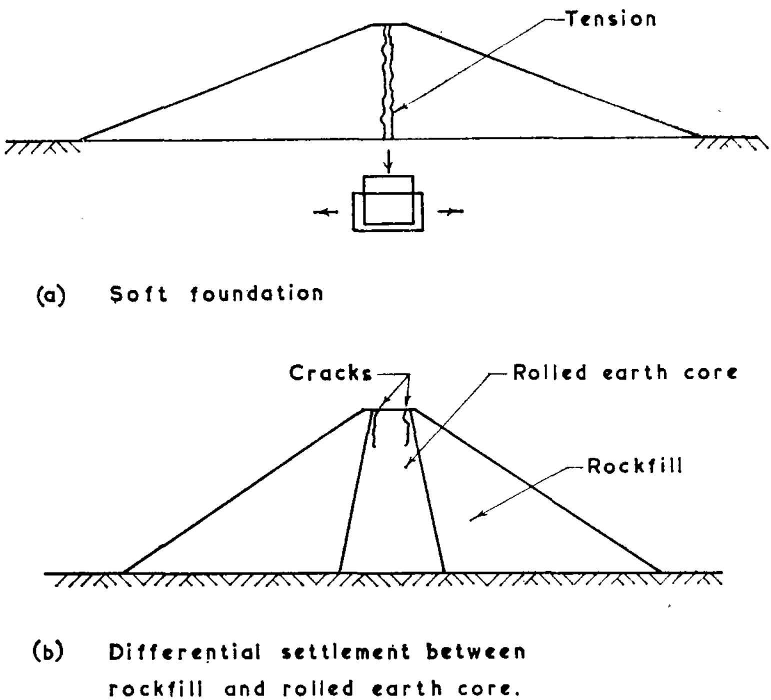


FIG. 47

Southern Region of British Railways on the Birchington marshes.[11] A crack 2–3 ft wide had opened up for a length of 200 ft in chalk fill placed on compressible marshy soil. Similar cracks were a feature of many flood banks placed on east coast and Fenland clays, and the Seven Sisters bank in Canada had failed in several places, apparently due to that cause.

203. Longitudinal cracks had also occurred in at least two dams due to the shell of rock fill or gravel being more compressible than the rolled earth core (Fig. 47(b)).

204. Looking over the history of British dam construction, it was interesting to note that one of the reasons given for the use of a plastic core of puddle clay in earth dams, for so long a traditional feature of British earth dams, had been stated as long ago as 1866 by Jacob,[12] who said that a plastic core was necessary to guard against cracks due to differential settlement in the fill. More recent papers in the 1950's in America showed that there had been a complete cycle of opinion and that after an era of dry fill, engineers had now re-learned the lessons learned by their forefathers, but were expressing the results in more sophisticated language.

205. The core at Shek Pik had been expected to behave as a plastic material. It had been placed at more than 2% above the optimum water content and had been shown to be still about this value. Shrinkage, therefore, was not a likely cause of cracking, but cracking had occurred. It might be that the core should have been put in wetter. There had, however, been several cases on record, including one described by Terzaghi and Lecroix,[13] where a core put in, almost in a puddled clay state, had developed tensile cracks with large deformations, and considerable remedial works had had to be undertaken. Recently the wet core at the Messaure dam in Sweden, described at the Edinburgh Conference in 1964, had developed a cavity

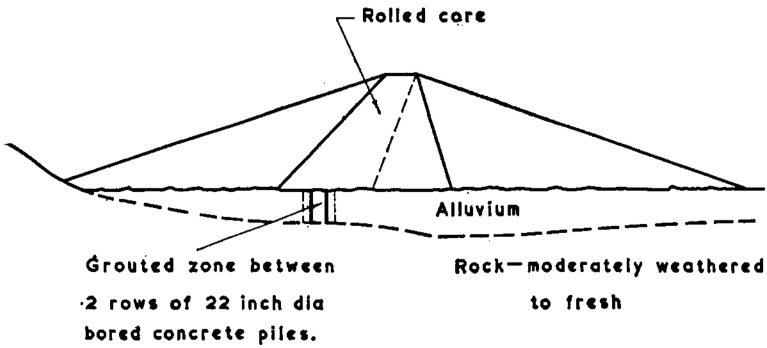


FIG. 48

which required a substantial repair. Wet placement, therefore, was not necessarily the answer.

206. The special features at Shek Pik which appeared to be associated with cracking were threefold. The grouting operations themselves had quite clearly resulted in some crack formation. Heave had been observed during contact grouting, and he gathered from the Author that in the two 50 ft trial trenches, at around chainage 700, horizontal fissures had been found in the fill, filled with the grout being used for the contact grouting. Other fissures filled with the clay–cement grout used subsequently had also been found. It looked as though the grout itself had been causing fissures in the fill. The only crack ever to appear on the surface of the fill was on the left abutment while contact grouting had been going on beneath it. Perhaps the Author would give his views, in retrospect, on the pressures used in the grouting operations.

207. Much of the cracking, however, had occurred in fill placed in the centre of the valley after the grouting had been completed. The section in Fig. 48 showed that the core rested partly on grouted alluvium between two rows of 22 in. dia. bored concrete pile walls and partly on ungrouted compressible alluvium on each side. This gave a discontinuity: the alluvium directly under the centre line settled about 1 ft, almost all of this occurring during construction—there had been very little afterwards. This marked discontinuity in vertical strain might well have resulted in cracking of the core during placement under the added weight of each layer. The small subsequent settlement would explain why no surface cracking had occurred.

208. The compressibility of the fill was also of interest. The fill itself had an average compressibility of $2{\cdot}1 \times 10^{-3}$ sq. ft/ton. This was less than half the minimum of the U.S.B.R. observations on four dams of residual soil, where the figures varied from 4·5 to 6·5. It seemed to be clear that the soil at Shek Pik was unusual. Sherard *et al.*,[14] in their review of cracking in dams, referred to a cementing action which occurred in partly decomposed soils, and it was possible that this type of hardening had occurred at Shek Pik.

209. The other point to note was the very wide range of compressibility within the fill itself, 0·9–4·3, and it might be this variation rather than the actual compressibility of the fill which was the difficulty. The compressibility of the fill was smaller than that of the alluvium or that of the weathered rock, but not by a very large factor (probably less than the difference between concrete and the rock on which concrete dams were usually founded). The variation within the fill itself, however, was very marked, and this might be responsible for some of the symptoms which had led the Author to diagnose cracking, such as an unusual loss of water from drill holes. Dr Bishop wondered how many engineers would be prepared to check the cores of their dams in this way, and what they would find if they did. He had always been puzzled

by the fact that all the piezometers in the rolled earth fill at the Sasumua dam, upstream of the central drain, had responded and established almost steady-flow conditions within one month of impounding. At Shek Pik the period had been about three months, and if cracking was the cause of the rapid response in that case, it might be suggested that cracking was much more prevalent than had been thought, but in the absence of suitable tests had remained undetected. Where there was an intercepting drain protected by a proper filter there might be no danger. Leakage at Shek Pik was negligible, but the engineering problem was, he thought, one of internal erosion, and in retrospect it might have been advisable to use a thicker filter in a dam of this sort.

210. In conclusion, if the Author had to do the grouting again, would he recommend the same grouting pressures that he had used at Shek Pik?

Mr H. H. Dixon (Partner, Howard Humphreys & Sons) congratulated the two Authors, particularly on dealing with the difficulties which had been encountered.

212. In connexion with Mr Goudy's Paper, Mr Dixon gave a brief description of the problems associated with a pipeline in Libya for which his firm had been responsible.

213. The pipes of 20–22 in. dia. had been shipped to Libya in 12 m lengths with no external protection but lined with plasticized coal tar enamel. A reinforced coal tar wrap was applied on site to the single pipes by rotating them spirally past a stationary wrapping machine. The pipes were then welded in pairs to produce pipes 24 m long and the external wrap completed by hand at the welded joint. The contractor decided on this system because of the difficulty of spinning and handling 24 m lengths and it was interesting to note that this problem was overcome in the case of the Shek Pik pipeline by using a travelling kettle.

214. The main problem on site was the completion of the internal protection at the central weld. Initially the contractor attempted to use a heavy bodied coating (applied cold) compatible with coal-tar enamel, but several coats were necessary to produce a lining free from 'holidays' and it had sometimes taken as much as 5 days for each of the coats to dry. The problem was overcome in the end by devising a method of transporting a hopper of hot enamel inside the pipe and tipping it at a controlled rate while the pipe was rotated and the spread of this internal lining was prevented by means of two hanging shutters. The 24 m pipes were then transported to the laying site and jointed with mechanical couplings.

215. Turning to Mr Carlyle's Paper, he had been struck by the very high rate at which the fill had been placed. It worked out at 570 cu. yd/h. Perhaps the Author would state the total earth-moving equipment used, especially having regard to the fact that the borrow pits had been up to a mile away. To have achieved a fill rate of 570 cu. yd/h in such circumstances showed excellent organization.

216. With regard to the field tests which had been carried out for compaction it had been stated that a sheepsfoot roller and a rubber-tired roller in combination had proved to give the best results. He had been surprised to see that 100% Proctor compaction had not been attained, but perhaps they had not aimed for it. At the Guma dam in Sierra Leone, now under construction, Mr Dixon's firm had carried out field compaction trials on residual soils derived from gabbro but had not been able to obtain the required compaction with any combination of a 20 ton sheepsfoot roller and a 45 ton rubber-tired roller, no matter how many passes were made. A 3¾ ton vibrating roller was available on the site which, surprisingly had given the necessary compaction. They had obtained 100% Proctor compaction with six passes and the permeability was between 1×10^{-7} cm/s and 1×10^{-8} cm/s.

217. The replacement of clay by rock at Guma had helped greatly in ensuring continuity of work during the wet and the dry seasons and it might be useful to design dams in areas having very pronounced wet and dry seasons so that there was a substantial amount of useful work to perform in the wet season and so that the amount of clay to be placed in the dry season was well within the capacity of the plant.

218. The vertical filter at Shek Pik had been placed using draw shuttering and trench excavation. It was, he thought, considerably cheaper and less time-consuming and frustrating to the contractor to allow him to construct it above the surrounding clay fill, with a central coarser material and two flanking finer materials. A greater average thickness was necessary because the materials were dumped in small banks against which the clay fill was placed and compacted.

219. The settlement measurements had been made by the U.S.B.R. type of tube and cross-arm apparatus. Mr Dixon's firm had recently installed some overflow type tips on their dam in Sierra Leone. He could not give a complete report yet but if the system could be perfected it had the great merit that it did not delay the contractor on the surface and it would enable records to be kept of the upstream section of the dam long after operations had ceased.

M. E. Marin (Société Française d'Entreprises de Dragages et de Travaux Publics), referring to Mr Carlyle's Paper, said it was difficult for a contractor to add anything of value to his information. The Contractor had had to deal with three main difficulties: the foundation of the dam, the lack of free draining material, and the compaction of the structure.

221. With modern equipment they had been able to move the muddy material over a wide area, but they had solved the problem only by the practical method of building a network of surface tracks by using a large quantity of alluvium. The lack of free-draining material had been the greatest difficulty, especially in a tropical climate, when there was no hope of handling clayey materials in the rainy season. The borrow-pits for clayey material consisted of material ranging from completely weathered to fresh rock. This had made the planning of the excavation procedure difficult because fresh rock might appear at any moment.

222. Turning to the remarks of the previous speaker, M. Marin referred to the enormous amount of energy used for compaction. Six passes by a 22 metric ton sheepsfoot roller and six passes by a 45 metric ton rubber-tired roller had been necessary on a 6 in. layer to obtain an average of 95·5% of the normal British Standard compaction test, although the moisture content had been very close to the optimum. Increased energy of compaction up to 24 passes had not produced any improvement.

223. In conclusion, he thanked Messrs Binnie & Partners, and especially Mr Carlyle, for their co-operation in dealing with the Contractor's problems.

Mr J. A. Dempsey (Soil Mechanics Ltd) said that the dam at Shek Pik was the first to be constructed with an alluvial cut-off by a British consulting engineer using the technique which had been developed by Soletanche of Paris. Before proceeding with the main work a test length had been constructed on the site of the actual cut-off to judge efficacy, to enable some of the details of the main work to be developed, and to obtain data for estimating costs. This trial length had been formed in a section of the dam shown in Fig. 37 of the Paper at chainage 1100.

225. It consisted of a box formed of the bored piles which penetrated through the alluvial deposits in the valley floor down to the underlying decomposed rock with the object of containing the grout. The alluvium inside the box was treated with clay–bentonite–cement grouts and permeability tests had been carried out in situ. Subsequently the treated material was excavated to look at the position at first hand. Large boulders had been encountered throughout the alluvium and also core-stones in the decomposed granite. These boulders had been one of the chief reasons why a conventional cut-off such as sheet-piling could not be carried out.

226. During the main works, grout was mixed and pumped from a central grouting station and introduced into the ground through sleeved grout pipes. Local clay had been used in the grouts for cheapness, and a clay with a liquid limit of 70% had been obtainable within a few miles of the site. Its quality was improved by the addition of an imported bentonite and ordinary Portland cement was added to rigidify the grout.

227. With regard to the costs referred to in Appendix 4 of the Paper, there had been a considerable amount of site investigation carried out during the course of the works which amounted to £40 000 of the £1·2 million. Also, work valued at £85 000 was performed in investigating and treating the cracking in the core which had been described by the Author and by contributors to the discussion.

Mr M. Waghorn (Managing Director, Coating Contractors Ltd) confined his remarks to Mr Goudy's Paper which he had found of great interest.

229. On the question of the choice of abrasive for preparing the surface, the report of the American National Association of Corrosion Engineers suggested that a non-metallic abrasive was better than chilled iron grit. In the Author's opinion, was the use of the latter in any way a cause of the blistering? There was also the question of the presence of millscale and rust at the base of the blisters. Was not this purely coincidental and in no way the cause of the trouble? It seemed likely that in any sand-blasting operation, however carefully done, traces of millscale and rust would be found.

230. With regard to the tests carried out in the United Kingdom to identify the cause of the failure, he wondered whether the wash primer to which the Author referred had in fact been applied under the same conditions as on the site, where there had probably been very high relative humidity.

231. Mr Waghorn assumed that what the Author referred to as a wash primer was probably a polyvinyl butyral based material and he questioned whether it was phenolic modified, because these products were notorious for their water-sensitivity when in a single coat.

232. The Author mentioned that an epoxy coat used on the weld margin had not shown signs of blistering. Had the same primer been used underneath this epoxy coating?

233. On the question of the relationship of the dry film thickness to the peak-to-trough depth of the blast pattern, 2½–3 times the peak-to-trough depth was usually accepted as a reasonable dry-film thickness for the coating. In the situation at Shek Pik, with Class 1 conditions and very limited access to the pipe, the upper limit of that thickness would seem desirable, and the acceptance of a coating thickness of 0·008 in. seemed to be a little on the low side.

234. Had the Consultants considered the use of a coating method which would involve the surface preparation of the pipes by blast cleaning and priming using what was now known as a prefabrication primer and by coating the pipe and afterwards using the plugging technique used in America and to a less extent in this country? It would seem that this would have avoided condensation on the primer, which he believed was the likely cause of the problem, because the American technique involved the passing of a plug of water miscible ketone prior to the paint plug.

235. There were pitch–polyurethane coatings which had the approval of the Water Research Association for use with potable water which had a much better application tolerance. Mr Waghorn asked if it would have been wise to use something of this kind and with greater coating thickness.

Mr Berry expressed particular interest in the submarine pipeline aspect of the work and referred to what had struck him as two particularly difficult problems when considering a similar type of work elsewhere. There was the question of the pre-dredging trench in areas where there was considerable cross-current and the question of whether the trench would stay open until a pipe had been laid in it. Perhaps Mr Goudy would say something about this. Presumably by over-dredging it would be possible to do something, but it was a question of very careful judgement as to the rate of progress.

237. The second question was that of corrosion. He imagined that the Author had given serious thought to the problem of how to judge the estimated length of life

of a pipe in sea-water, and wondered how its life could be determined when the pipe was completely inaccessible.

Mr W. Fox (Kier Ltd), speaking as a representative of the Contractors who had been engaged on the submarine pipeline, said that the first thing that had had to be decided was how deep and how wide the trench should be dug. The American company which sponsored the Consortium had had considerable experience of this, and it was on their judgement that the trench had been dug approximately 3 ft deeper than had been required and something in the region of 20 ft wider than the actual overall width of the pipes. This judgement proved sound because the material had stood on the side of the bank and when checked by echo-sounder there had appeared to be very little movement. The sides of the trench, so far as it was possible to tell, had stood without collapsing for a period of nine or ten months. The bottom of the trench had in it, so far as could be ascertained, a slurry (presumably the sea-bed mud) which was continually agitated by the movement of the tide, and in which the pipes floated. This was one of the factors which had been of great assistance in moving the pipes and keeping their centres approximately 10 ft apart.

239. This had been the first occasion on which pipes had been pulled in a pair, and a major question had been whether the pipes would stay apart or whether there would be a tendency for them to come together or overlap. The pipes had originally been held apart on the towing end by a sliding former. This had not been altogether successful and was replaced by an RSJ. The bridles had been passed through holes in the RSJ and had maintained the towing heads at 10 ft centres. The inherent stiffness of the pipe and the slurry in the bottom of the trench had been the two factors which maintained the pipe centres without trouble. This was checked by using the echo-sounder. A tug was fitted with an echo-sounder and calibrated for engine speed. Thus by straightforward measurement on the trace it was possible to determine the distance apart of the pipes to the nearest foot.

240. One of the problems with this type of operation was to keep the pipes on the sea bed. It was essential to keep the negative buoyancy of the pipe within very close limits. At Shek Pik it had been decided to concrete the pipes in shutters. The pipes had arrived at Hong Kong in 27 ft lengths, and the pipes were weighed individually after unloading. Each pipe was then placed in store and eventually joined with two others to make an 81 ft length. This pipe was then wrapped and reweighed and the circumference measured in six places. By selecting pipes of suitable weights and average diameter it had been possible to alter the width at the top of the shutters in order to achieve the buoyancy that was required. A pipe with a negative buoyancy of 4 lb/ft run with a spread of about 2 lb was produced, which had made it possible to lay the pipes in 60 ft of water without trouble.

241. The pipe laying was based on a rate of six joints a day, the limit being set by the problems of the internal lining. There had been very little trouble in welding the pipes and making the jointing, and the whole barge operation had run smoothly.

242. The job had been a particularly happy one and the relations between contractors and consultants had at all times been most amicable.

The following contributions were received in writing:

Mr P. C. Edwards (Engineer, Harris & Sutherland), referring to § 112 on compaction of the fill, and the subsequent discussion on compaction plant, suggested that there would appear to be no specification for compaction that was ever maintained in the field; also, that many of the 'standard tests' were of doubtful value except as very crude 'controls' of consistency. In his opinion it must eventually be realized that the best specification should state the desired strength, density or permeability required in the completed work leaving open the question of how this result should be achieved, so that contractors' proposals could be considered by, and agreed with, the engineer.

244. It was revealed during the Symposium on Chalk in Earthworks[15] held at the Institution that the Ministry of Transport were about to revise their 'standard specification' on a 'method' basis. Would the Authors agree that the Institution should collaborate with the Federation of Civil Engineering Contractors and agree some standard principles for compaction specifications?

Mr A. W. Shilston (Consulting Engineer) referring to Paper 6816, observed that whilst his interest in submarine pipelines veered more towards the disposal of sewage or sewage effluent through long sea outfalls, the ultimate function of the conduit when laid did not greatly affect the appreciation that had to be made as to constructional feasibility in any particular situation.

246. A long submarine outfall would generally warrant a more detailed site investigation than would be necessary in a scheme similar to that described by the Author. However, in § 5 Mr Goudy did refer to chemical and bacteriological analyses having been undertaken as part of the maritime survey and Mr Shilston enquired whether those were carried out by nominated organizations or by agencies at the complete discretion of the site investigation contractor? In fact, were the analyses done locally? It would be of interest to learn of the bacteriological, temperature, salinity and other chemical conditions actually met and what significance was attached to these in the design appreciation.

247. The structural design of submarine pipelines was dominated by the construction technique adopted, which was in itself a highly specialized operation. In many instances it would be unwise for the employer's professional advisers to have set ideas on procedure to be adopted and it was generally desirable to allow wide discretion to tendering contractors to put forward proposals which reflected their particular style of expertize. That had, however, the disadvantage of delaying the completion of the final design in view of the particular intimacy of relationship as between design and construction.

248. Could the Author give some further indication of the terms of reference included in the tender invitation and confirm, or otherwise, that the specification was generally functional in character? Had conditions of contract been prescribed, of the Institution form or the international variant of it, or had terms been negotiated with the successful contractor? No doubt more than one tender had been received and it would be instructive to learn whether other interested contractors had put forward methods of pipe laying significantly different from those adopted by the successful contractor.

249. If Mr Shilston's previous remarks were generally relevant, one would conclude that the supply of the steel pipes and their preparation had been a matter directly within the main contractor's administration and not the subject of pre-order. However, §§ 80 and 81 left one in some doubt as to whether that was so and perhaps the Author could indicate if a sub-division of contracts was implied. In view of the considerable amount of work which had to be done at Hong Kong on infilling sections of the internal lining together with the execution of the whole of the external coating protection, would it not have been feasible and advantageous to have had all the coating work done at site? Did the Author consider on reflexion whether pickling of the steel would have been desirable in its initial surface preparation, and if so whether in fact that would have been ruled out on grounds of lack of available facilities or of cost? Design and specification attitudes for overseas work were probably more often determined by considerations of what one could do most effectively with the resources available rather than what one would like to do.

250. The reference in § 41 to the flash-set concrete placed around the pipe lines at the welded joint connexion between the strings made one apprehensive, at first sight, in view of the extremely short period of time that elapsed between the placing of the concrete infilling on the barge and the subsequent immersion of the joint. Presumably it was argued that the concrete lining was not submitted to axial loading and was therefore non-structural, so the brief hardening period was of no moment.

Was the Author, from observation, entirely satisfied that infilling without any surrounding local wrapped protection, was in no sense a weak link in the system?

251. With reference to Mr Carlyle's Paper, Mr Shilston asked whether the Author would comment on the formulation of the contract for the construction of the Shek Pik dam, particularly in the following aspects.

(*a*) Was it framed in the general style of a British measure and value type contract? If not what were the salient differences?
(*b*) What were the nationalities of the tendering contractors?
(*c*) Was there any direct financial incentive to the contractor to finish the works within the stated construction period, in view of the critical position in Hong Kong?
(*d*) Did the contract allow for pricing on a fixed price or price fluctuation basis?
(*e*) Since a separate contract had been placed outside the main contract for Zone 4 sand, was selection of the remaining materials in the dam controlled by inserting prime cost rates in the Bill? If not, was the decision to place the separate contract for the sand fully justified in the event?

Mr J. H. Fleming (Sir M. MacDonald & Partners) wrote that both Authors were to be congratulated upon the amount of useful information included in their Papers. The cost figures, however, could have been made more valuable by a reference to the quantities involved, so that the reader did not have to hunt through the text and work out the sums for himself. On the pipeline it appeared that the cost of manufacture and delivery of the steel pipe was £155 per ton, and the internal protection 5*s* 8*d* per sq. ft, while pipelaying, including the external wrapping and concrete coating, was £30 per linear ft.

253. Similarly, at the dam, the diaphragms of continuous bored piles apparently cost £3 per sq. ft and the overall cost of the grouted curtain was also about £3 per sq. ft. Unfortunately in § 167 the quantities of the various classes of fill material were omitted, but if the dam as a whole contained 6·3 million cu. yd (see § 103) then the total cost worked out at about 10*s* 6*d* per cu. yd of fill. Could the Authors include some analysis of the cost figures in their reply, so that the Paper could be made more useful for reference and for comparison with other projects?

254. The cracking of the dam core seemed to have been a source of worry, although it was not clear what the cost of the remedial measures amounted to. From the grading curves in Fig. 23 the residual soil appeared to have similar properties to moraine material which had been successfully placed by the 'Swedish wet-compaction' method, although the percentage of clay was somewhat higher. Since it appeared that pore pressures in this material were dissipated very rapidly after compaction, did the Author consider that the cracking could have been avoided, and a satisfactory core achieved, by using a moisture content between 5 and 10% greater than the Proctor optimum? Of course the consequent low strength of the core, while enabling it to adjust to differential settlement, might have involved a disproportionate increase in cost if more material was needed in the shoulders.

Mr J. E. Mitchell (Senior Engineer, Sir M. MacDonald & Partners) wrote that undersea pipelines and dams 180 ft in height on deep alluvial foundations were not everyday events in British water engineering practice. The Authors were to be congratulated on interesting and informative Papers.

256. He was particularly interested in the cut-off grouting and the ingenious methods adopted to obtain an effective curtain. However, the high cost of this section of the work, and the dislocation and delay to the embanking programme caused by the grouting operations, should perhaps prompt some rethinking on this aspect.

257. Recently, the writer's firm had been connected with two dams in the Middle East, both just under 30 m in height, on deep alluvial foundations. The sites were adjacent to the Rift Valley, and the bed rock below the alluvium was faulted. At an

early stage in the design of the dams the provision of cut-off arrangements had been given careful thought. Partial cut-offs had been rejected, since flow nets, even based on assumed anisotropy of foundation permeabilities, showed that the leakage could be considerable—possibly as large as the incoming flow. Grouted cut-offs had been rejected on grounds of cost, and also, in view of the disturbed foundation geology, because a flexible system that could accommodate itself to possible differential settlement of the foundations was thought desirable. Since the reservoir areas were relatively small, and a large proportion of the areas were already covered by a reasonably impermeable soil layer, it had been decided to blanket the whole area. Field experiments showed that the permeability of existing soil deposits was of the order of 10^{-5} cm/s, and laboratory experiments showed that, when compacted at optimum moisture content under laboratory conditions, this figure was as low as 10^{-7}. Hence it was proposed to place the blanket to the same specification as the main impermeable fill, and to scarify, water and roll the in situ sections. Calculations based on the method outlined by Kisch[16] showed that the seepage from the reservoir area was likely to be insignificant. The estimated cost of such a blanket was a fraction of that of a grouted cut-off in this case. Provision had been made for relief wells downstream of the dam.

258. It was thought that this method may have had some application to cases where a grouted cut-off was considered desirable, but the delays caused by grouting were inacceptable, in that the cut-off might be grouted in some convenient position upstream of the dam, and connected to the main dam by such a blanket. The method also had the advantage of reducing the differential settlement below the dam that has been put forward as a possible cause of the cracks in the core at Shek Pik. However, the blanket needed careful maintenance, particularly where the stream was fast flowing and bearing considerable bed load.

259. The unit costs obtained by uniting Table 2 with Appendix 4, showed little economic benefit in using dredged sand in preference to the 20 ft layer of rock mooted by the Author in § 98 for the upstream shoulder. Were there engineering reasons which weighed the scales in favour of sand?

260. It was noted that the upstream protection was changed from concrete slabbing to rip-rap. Did this indicate a preference for rip-rap on the grounds of performance or was the decision taken for economic reasons?

261. The Authors' reaction if faced with the design of a dam on similar foundations and with similar core material would be instructive. Would they rely on post-construction investigation (and possible similar remedial measures) to assess the extent of any cracking in the core, or would they adopt an upstream filter or sand layer to try to make the core self-sealing if cracking occurred?

The Author of the Paper on the submarine pipeline thanked contributors to the discussion for the interest shown in the work in Hong Kong.

263. Replying to Mr Crosthwaite the Author said it was hoped that the use of the polyurethane lining would overcome objections to the more conventional materials described in § 13. The particular material used was tested for a year before coating operations commenced and these were carried out by the manufacturer who had previously used the same product, without trouble, on a variety of applications throughout the world. The Author considered that a great deal of thought and care had been devoted to its application. When the Paper was written there was no indication that the lining was discontinuous, but since then flakes of the lining material had emerged from the pipeline.

264. There was no evidence of any undesirable materials being formed by reaction between elements in the primer and the isocyanate material in the lacquer. The Author agreed that undesirable materials such as free phosphoric acid, alcohols and/or solvents could be left under the lacquer if the primer was applied too soon after mixing and/or if insufficient time was allowed for reaction and drying. Experimental

work had indicated that drying time was not significant down to intervals well below those employed at Singapore.

265. Although blowers were employed to control the humidity during lining it must be accepted, in view of atmospheric conditions in Singapore, that there may have been some slight surface moisture. Tests have shown that the primer was not unduly sensitive. The presence of moisture in the lacquer would act as an accelerator and provide a more rapid cure, but with the evolution of carbon dioxide gas during curing. No special significance should be attached to the word 'lacquer'. It was the manufacturer's terminology for the material used to coat the steel.

266. Synthetic resin coating thicknesses of 0·007–0·010 in. were normal practice in the chemical industry for more severe conditions than those appertaining to this pipeline. They have been providing effective protection for many years to vessels for the storage of beer, fruit juice and other liquids with a low pH value.

267. The Author agreed that thicknesses of about 0·020 in. were required for pitch-polyurethane coatings but these coatings were not available in 1959 when the decision on lining material was made. He felt that it was unfair to dismiss the thinner coating whilst the reason for the blistering remained undiscovered.

268. Mr Crosthwaite had suggested that it would be more suitable for the coating to be applied at a higher temperature than ambient. This was, in fact, what the manufacturer would have liked to have done, but the facilities for stoving 30 in. diameter pipes were not available at the time. The lining of some smaller pipes, used elsewhere on the Shek Pik scheme, which had been cured by a stoving process, was in good condition.

269. The Author thanked Mr Dixon for his contribution and appreciated his problem of making good the internal lining at pipe joints. Freedom of movement for operators would be much more restricted in a 20 in. diameter pipe than in the 30 in. diameter pipes used on the Shek Pik pipeline.

270. Mr Waghorn had asked several pertinent questions. The Author agreed with him that the use of a non-metallic abrasive for preparing the steel surface was unlikely to have eliminated blistering since the mill scale and oxidation products referred to in § 74 would, presumably, still have been present. Expert examination of microphotographs of the steel surface had indicated nothing unusual.

271. Tests carried out in the U.K. included the preparation of samples under humid conditions as experienced on site. In these tests, plates were coated with excess material brought back from Singapore and Hong Kong but the coating could not be induced to blister either when the plates were exposed to the atmosphere or submerged under pressure.

272. The primer used was polyvinyl butyral based but was not phenolic modified.

273. Mr Waghorn's question about the use of an epoxy at joints referred to the coating of the joints on the 48 in. diameter pipeline between the treatment works and the start of the submarine pipeline (see Fig. 1) which had not been covered when blistering was first discovered. The steel was grit blasted and coated with a three-coat epoxy in accordance with the manufacturer's instructions, but blistering was seen shortly after the pipeline had been filled and tested. Microphotographs showed that the thickness of coating was three times the shot-blasting profile and a continuous adequate thickness of material had been applied.

274. The plugging technique for lining the pipeline was considered but was thought to be impracticable for a pipe with a diameter of 30 in. Subsequent experience had indicated that the technique was often good for pipes up to 6 in. in diameter but had not yet been perfected for larger pipes. However, if this method had been used, there was a danger that the internal surface might have corroded in the long interval before pipelaying could be completed, and that the time lag between priming and coating may have led to poor adhesion.

275. Mr Berry's question about the size of trench required to ensure that it stayed open until the pipes were laid had been answered by Mr Fox. The Author added

that conditions were not quite the same in the sandy material at the bottom of the Lamma Channel where side slopes of 1 in 3 were required. Cross currents at sea-bed level were not significant, the maximum being about 1 ft/s, but it was the variation in current between the surface and sea-bed that caused difficulty in positioning the grab for accurate dredging. Some guidance on the behaviour of a trench in the particular material encountered had been given to the Contractor by observations on a short length dredged during the tender period.

276. Mr Berry's next question referred to the life of the pipeline. The Author felt that all precautions—internal lining, wrapping, concrete surround, cathodic protection, deep burial and water treatment control—had been taken to make the life as long as possible although he hesitated to indicate a definite number of years. The failure of the internal lining and the discharge of flakes of the lining material indicated that there were now places where the pipe surface was unprotected. Test plates have been inserted in the pipeline to observe the rate of corrosion of steel with different water treatments, and it was intended to install measurement probes to monitor the rate of corrosion. The addition of sodium silicate to the water at the treatment works might cause a protective film to be built up on the exposed steel.

277. Mr Fox had described the importance of weight control in wrapping and concreting pipes. The Author added that BS534 permitted tolerances of -4 to $+16$ lb/ft in the weight of the steel shell. As the length of freely suspended pipes was 900 ft when the laybarge was pipelaying in a depth of water of 60 ft, increases in negative buoyancy would have caused the maximum allowable bending moments and stresses to be exceeded.

278. The chemical and bacteriological tests mentioned by Mr Shilston had been undertaken to assess the effect of the sea-bed material on the concrete pipes surround. Samples had been analysed by the Government chemist in Hong Kong and by a laboratory nominated by the Consulting Engineers in London. It was concluded from the analyses that attack by bacteria or carbon dioxide in the pertaining alkaline conditions was most unlikely. Similarly, it was thought that no boring animals could 'breathe' in the depth of backfill specified. A cement containing fly-ash had been specified to provide additional resistance against deterioration caused by sulphates.

279. The results given in Table 6 had been obtained from the analyses.

TABLE 6

Sea-bed material	
Loss on ignition . . .	2·19–7·3%
Sulphates (SO_4). . . .	0·023–0·070%
pH value	7·6–8·2
Sulphate reducing bacteria.	100–1300/g
Sea-bed water	
pH	8·1–8·6
Specific gravity at 15°C .	1·023–1·025

280. Measurement of temperature, which was subject to seasonal changes throughout the year, and salinity in which variations were caused by the river discharge in the Canton estuary had not been measured as part of the site investigation programme. Measurements have been made and published by Chau and Abesser.[17]

281. In answer to Mr Shilston the Author confirmed that the specification was generally functional in character but adequately covered the work required under the contract. The tender documents included two bills of quantities, one suitable for the laybarge technique and the other for a pulling operation. As indicated in § 20, tenderers had submitted the bill most appropriate to the method in which they specialized, with supporting calculations.

282. The contract had been let under the Hong Kong Government's standard conditions of contract which were based on the international conditions. A special clause required tenderers to include for the provision of a performance bond for £1 000 000 as a safeguard against failure to succeed in laying the pipeline. Six tenders had been received. Alternative proposals for pipelaying included the use of a laybarge with a ladder to support the pipes after submergence to enable pipelaying to continue across the Lamma Channel, pulling pipe-strings and floating pipe-strings out to sea where they would have been jointed on the pipeline alignment before being sunk. A jetting device had been proposed as an alternative to dredging by two tenderers.

283. The supply of steel pipes and their internal lining were the subject of separate supply contracts placed by the Hong Kong Government. The pipelaying contractor was required to insure the pipes from the time he took delivery until the completed pipeline was successfully pressure tested. The possibility of establishing a pipe-rolling mill in Hong Kong was investigated and considered to be uneconomical as the internal lining had to be applied before the pipelaying contractor set up his site organization to ensure a supply of pipes and it was decided that this was best done in the pipe manufacturer's factory where certain necessary facilities were available.

284. Pickling of the steel and surface inspection had been considered but the additional cost did not seem to be justified.

285. The difficulties in placing the concrete at tie-in joints were threefold—use of fresh high-alumina cement from air-tight tins, clean tools, and ensuring hand mixing was not continued after the initial set (about 5 min). Before launching the pipes the concrete had been tested for hardness with a hammer as it had to carry the weight of the pipe over the launching rollers. After leaving the barge the concrete had fulfilled its primary function of protecting the wrapping during launching.

286. The Author disagreed with Mr Fleming's calculation of costs and prepared the following summary.

Supply Contracts	
Steel pipes, supply and delivery	£77 per ton
Internal lining	3*s* 2*d* per sq. ft
Pipelaying Contract	
Pipelaying	£14 per ft
Cathodic protection	£9400
Cross-connexion chamber	£56 000

287. **The Author,** in his introduction to the Paper on Shek Pik Dam, said that since the Paper was written the reservoir had been filled and performance data had become available to confirm the prognostication in § 146.

288. The first heavy rain had fallen in June 1964, and the reservoir rose from 70 ft to 90 ft in 2 days. Further rain at the end of August including one fall of 11½ in. in 12 h had caused the reservoir to rise from 110 ft to 180 ft (2½ ft above spillway level) in 34 days. The flow from the foundation drains in the alluvium and the embankment filters was proportional to reservoir level but had been only 0·16 cusec. at full reservoir.

289. Following the filling of the reservoir, the settlement of the foundation had been found to be negligible. The compression of the fill under the berm at level 144 ft had been negligible but the settlement of the crest had reached 0·2 ft at chainage 1470. The cross-arm apparatus at this point had shown that the settlement was the result of compression of the core of the dam in the zone between levels 80 ft and 110 ft where the cracks had been most frequent and in which the largest quantities of test water and grout had been injected (see Fig. 49).

290. The progressive saturation of the core had been observed from readings of piezometers and standpipes. The simple flow net, which was shown in Fig. 50, had

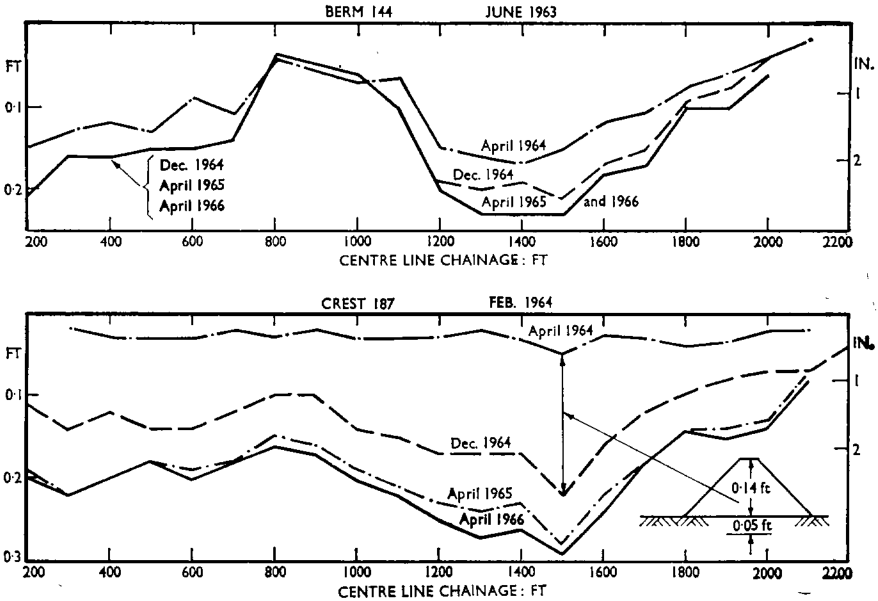
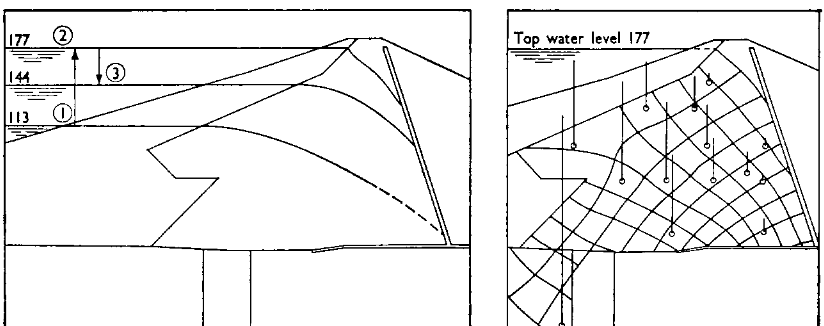


FIG. 49: SHEK PIK DAM SURFACE SETTLEMENT



PHREATIC LINES FOR FIRST FILLING AND DRAW-DOWN IN RESERVOIR

FLOW NET — FULL RESERVOIR

FIG. 50: CROSS SECTIONS THROUGH THE CORE OF THE SHEK PIK DAM

been derived from the observations. The rate of saturation had been faster than would have been expected from a soil of low permeability and could be due to a network of small cracks remaining in the core. From the flow net it could be deduced that the permeability was fairly uniform.

291. There had been no opportunity in the Paper to congratulate the Contractors on their work on the dam. Despite the difficulties and the fundamental changes in design which had had to be made, the work had been completed ahead of schedule and the reservoir had been impounded 1 year earlier than required in the contract. This had been at a time of critical water famine in Hong Kong when 12 mgd were being imported in oil tankers.

292. Replying to Mr Humphreys the Author said that for a constant value of ϕ_m Mr Humphreys's test gave the results of the test which had been described by Bishop and Henkel[5] more readily for the actual factor of safety (F) of the design.

293. If F (point Q, Fig. 43) for the trial design was found to be lower than required then the design would have to be changed and a new plot of F against r_u would have to be made. Thus, in the end, the same processes would have to be performed by both methods to achieve a pore pressure ratio prediction and the required factor of safety for a trial design. The actual triaxial test suggested by Mr Humphreys was somewhat easier to carry out since σ_3 was constant throughout the test. The method was satisfactory for the undrained condition but did not help in the prediction of pore pressure after partial consolidation, which was the fundamental problem in the use of effective stress analyses.

294. With regard to the use of core cutters for density measurements in the bank, control tests with sand or water bladder replacement and with nucleonic density meters had indicated that the method was satisfactory.

295. Mr Little had referred to the work of Leonards and Narain[9] who simulated the embankment strains which would lead to cracking in test beams of soil. In the dams which had been examined, the cracks had been observed on the surface, and the strains had been calculated from crest settlement. In the case of Shek Pik cracks had not been evident at the surface and the crest settlement after completion had been negligible. The Author agreed with Mr Little that in soils of moderate to low plasticity cracking could not be predicted on theoretical grounds but it should be anticipated in the design of the core and the protective filters and drains. It was evident that soils of high plasticity would have relatively low permeability and high construction pore pressures. The resulting low shear strength would inhibit cracking despite stresses imposed by differential compressibility in the foundation or the fill. Thus, observation of the core pore pressures, together with stress measurements, might be the most useful indicator of possible cracking.

296. Professor Bishop had dealt with the question of higher fill water content which was raised by Mr Fleming; the Author agreed that non-plastic soil could not be made plastic in an embankment by increasing placement water content because rapid consolidation would restore the *status quo*. The wet placement method had been adopted where weather conditions had inhibited normal compaction by rolling, and, in the construction of the Swedish and Norwegian dams, rolling at normal water contents had been preferred. The change of wet filling had been made as the weather deteriorated later in the season.

297. The Author would not recommend grouting the completely-weathered to highly-weathered rock in the abutments of a dam such as Shek Pik. It would certainly be more efficient to excavate a normal cut-off trench in that zone. Where the weathered-rock was overlain by deep deposits of drift or alluvium (chainage 900 ft to 1400 ft, Fig. 37) the work could be completed before embankment construction was commenced and the necessary high pressures would not be damaging.

298. Mr Dixon had been surprised that the compaction had not resulted in fill densities 100% of Proctor maximum. It should be pointed out that the average water content was 2½% wet of Proctor optimum. At that point on the curve, Proctor density was reduced so that field densities were 98% of Proctor. The densities had been perfectly satisfactory and the filling rate had never outstripped the capacity of the compaction plant.

299. The peak monthly rate of filling had reached 450 000 cu. yd which had been disposed as follows:

residual soil	300 000 cu. yd
sand	80 000 cu. yd
rock fill	70 000 cu. yd.

The plant which had been used was listed below.

Residual soil

300. A team of two tandem and two single 300 cu. yd le Toumean scrapers and three teams of face shovels with rear dump trucks had been used. Two shovels were 2 cu. yd and one 3 cu. yd capacity. Each shovel had been equipped with sufficient trucks to give a truck-to-bucket volume ratio of 50. Five trucks were K.W. Dart 26 cu. yd capacity; 16 trucks were Berliet 18 cu. yd capacity. In addition there had been a large number of 6 cu. yd trucks for use on difficult roads and congested areas.

301. To maintain the filling rate the following service plant had been required.

(*a*) Borrow pits:

1 wheeled dozer Michigan 280 for two excavators in borrow pit;
1 dozer Cat. D.9 for loading scrapers;
1 dozer Cat. D.9 for ripping in scraper borrow area as required.

(*b*) Spreading:

Each Cat. D.8 dozer spread fill in 9 in. layers at 400 cu. yd/h from the rear dumps and 600 cu. yd/h from scrapers.

(*c*) Compaction:

2 sheeps foot rollers, 22 ton ballasted;
2 rubber tyred rollers, 45 ton ballasted, with necessary haulage tractors Cat. D.8 and Continental Cat. D.8.

(*d*) Watering and road maintenance:

4 tankers, 1000 gal capacity, for watering fill and borrow pit roads;
2 patrol girders for road maintenance.

Sand

302. (*a*) Loading and hauling:

3 cutter suction dredges;
6 barges, 600 cu. yd capacity;
4 tugs, 300 hp.
The cycle time for barges was 20 h.

(*b*) Unloading:

3 excavators with 2½ cu. yd grab buckets

(*c*) Filling:

1 No. 3 cu. yd excavator fitted with 4 cu. yd clamshell bucket;
10 rear dump trucks, 12 cu. yd capacity;
1 Michigan 280 rubber tyred dozer spreading;
1 Cat. D.8 dozer compacting.

Rockfill and rip-rap

303. Rock drilling had been done by crawler wagon drills making 3 in. diameter holes. Loading had been done initially by two traxcavators, Cat. 995H, but these had been replaced by a 3 cu. yd face shovel when it had been possible to spare it from the residual soil excavation. Rear dump trucks were 18 cu. yd Berliet from the residual soil fleet.

304. Mr Edwards, in his reference to § 112 on the compaction of the sand fill, had implied that the specification for control by measurement of the relative density had not been implemented; this was not so.

305. The Author disagreed with the suggestion that the Institution should produce standard principles for compaction specifications in so far as these might relate to fills for embankment dams. The designer of the dam was the person best fitted to specify the compaction of fill so that it would have the desired properties. He might choose to do this so that it would have the desired properties, or he might choose to

do this by specifying the plant and amount of compaction or the density of the fill against a control; in any event he would not wish to be bound by a standard specification.

306. During the construction of recent fill dams for which the Author's firm had been Engineers, it had been found that measured density compared against some standard was the most reliable control for compaction. Routine tests had been made of other soil properties such as shear strength, compressibility, permeability and index properties. These tests had been made as a check on the design criteria and had often been subject to considerable variation while giving satisfactory results on average for the whole bank.

307. In certain cases strength measurements had been proposed as a field control on filling. For example, it had been suggested that the specification for a projected clay fill on Weald clay at Cuckmere should place both an upper and lower limit on unconfined shear strength of the fill in order to maintain the relative flexibility of fill and foundation. The Author understood that this had been done at Walton reservoir for the Metropolitan Water Board. Compaction was not usually a critical factor in dam construction and relations between engineers and contractors were generally agreeable on this part of the work.

308. Mr Shilston had asked about the contracts for the dam.

(*a*) The contract for the cut-off had been awarded on a cost-plus-fee basis. The contract for the dam had been of typical measure and value form.

(*b*) Tenders had been received from Hong Kong, U.S.A., France, Japan, Sweden and Malaya.

(*c*) There had been no direct financial incentive to finish the works within the stated construction period.

(*d*) The contract for the dam had not allowed for variation in price of labour or materials.

(*e*) Sand for the upstream shoulder of the dam had been supplied to a stockpile at Shek Pik as a replacement for the alluvium in the borrow pit which had been found to be unsuitable. Thus, the dam Contractor had had a similar operation to perform subject to the variation of haul distance and access.

The award of a separate Contract for the supply of sand to the stockpile had been justified by the success of the operation.

309. Mr Fleming had asked about the cost of the remedial grouting in the core; that had amounted to £85 000, but the drilling and grouting equipment was available in the site. Costs of the cut-off grouting were as follows.

	Cost/sq. ft
Piled and grouted curtain in alluvium . .	£12·4
Curtain in weathered rock	£7·75
Curtain in fresh rock	£1·40

310. The final costs of the dam construction had been settled and the overall cost of the work had been £4·24 million or 13·3*s*/cu. yd over all types of fill and ancillary works but excluding the grouted cut-off. The actual costs of fill were as follows.

	Cost/cu. yd
Residual soil	4*s* 5*d*
Alluvium	16*s* 5*d*
Sand	20*s* 6*d*
Filter material	24*s* 6*d*
Rock fill and rip-rap . .	19*s* 6*d*

311. Mr Mitchell's use of blankets in place of central grout screens would be limited to special sites where the reservoir area was rather small and where there was

no danger of uplift on draw-down. It would be of interest to know what protected the blanket from drying and cracking.

312. Recently the Author had been faced with the problem of making a design for a dam of residual soil which had similar properties to that of Shek Pik and foundations of variable compressibility. Alternative designs with an upstream deck of asphaltic concrete or an upstream sloping core had been considered and the latter had been selected primarily on the grounds of cost. Both designs had been considered to be able to withstand cracking, the former by the flexibility of the deck and the latter by the thickness of the upstream deck and by the disposition of the drains and filters.

313. At Shek Pik rip-rap had been preferred for service reasons. Sand had been preferred for use in the upstream shoulder for the following reasons.

(i) It could be hauled and placed in the wet season when access for rockfill would be impossible.

(ii) At the time the decision had to be made, the Contractor's quoted rates for guaranteeing the required amount of rockfill had been much higher than the rate at which he had been supplying rip-rap.

In the event the two wet seasons had exceptionally low rainfalls and the quarry had been better than anticipated; thus the Contractor had been able to supply rockfill at about the same price as the total cost of sand filling.

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