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8703 DISCUSSION GROUND ENGINEERING GROUP

Geotechnical aspects of Kielder Dam

J. P. Millmore and R. McNicol

Introduction

It was not possible to record all of the geotechnical work at Kielder Dam. We dealt only with the most significant aspects of the site investigations, design, construction and performance of the dam embankment.

49. Attention was drawn to the variations encountered in the glacial till between the site investigation and the dam construction. These variations contributed to a lower factor of safety than anticipated for the stability of the dam.

50. The specification called for a degree of selection of the fill in the embankment. The material with the higher clay content was required in the core of the dam with 'as dug' glacial till in the shoulders. Testing of the soil during the construction of the dam demonstrated that the Contractor was successful in this selection. However, the clay content in the core and blanket was found to be considerably higher than in the soil tested during the site investigation. This increase in clay content was beneficial to the Contractor because the material was the least susceptible to isolated showers of rain. It also provided a very impermeable core. The penalties of this higher clay content were, however, lower strength parameters and higher pore water pressures than anticipated. These factors gave rise to concern over the stability of the dam.

51. Two small stability beams were added at the upstream toe of the embankment. These beams ensured that construction of the dam could be completed safely. Their introduction disrupted the construction programme by only a few days.

52. An economic design was adopted at Kielder Dam but this required a comprehensive system of monitoring behaviour. A more conservative design could have been prepared and this may have avoided such close control. However, the additional cost to the Northumbrian Water Authority would have been substantial.

G. Rocke, *Babtie Shaw and Morton*

The great depth of glacial till that covers the centre of the North Tyne valley at Kielder Dam gave rise to a 'no cut off' type of design, principally because the glacial till was naturally of low permeability. Artesian groundwater conditions existed in the river valley and on the right abutment but not on the left abutment. In fact some boreholes there could not retain drilling water. This abutment, however, bore surface evidence of ancient landslides and this had in the past affected the construction of a mineral railway line which traversed the landslide area. The Kielder overflow channel was carefully sited on the left abutment to

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avoid the unstable ground and consequently acquired a sharp radius bend. Subsequent construction excavations revealed the source of the instability to be weak mudstone beds near the river terrace on the left bank.

54. In October 1981 when impounding of the reservoir was within 4 m of top water level, a final assessment was made of the behaviour of the nine relief wells. Seepage gradients appeared normal but an information gap was recognized downstream of the overflow channel. In that month the reservoir had risen from 176 m OD to 181 m OD for the first time. Concurrently wet areas were noted some 200–300 m downstream of the overflow channel and complaints were received from a tenant farmer at nearby Hawkhope Farm that fresh flows of water were affecting his land. These events gave rise to the installation in boreholes of seven additional piezometers in May–July 1982 on the left hillside downstream of the dam and a further seven piezometers in June 1983 together with hydrogeological studies of the area. Samples of mudstone from the boreholes were laboratory tested in shear box and ring shear apparatus and groundwater analysed for chemical content.

55. The following results were obtained from the investigations.

(*a*) Two fault systems were identified, a down valley series and an oblique series. Some faults tended to promote concentrations of groundwater flows whereas others, sealed by weathered materials, resisted groundwater flows. An intersecting water bearing series converged on Hawkhope Farm rendering it susceptible to artesian outflows. Nearer to the river terraces, a set of faults resisted flows to the river and dramatic head losses were measured across these faults.

(*b*) The borings provided sufficient information to locate the principal sandstone aquifers. One such sandstone had fed artesian water into the excavation for the dam access culvert during its construction in 1976 and caused much softening of the mudstone strata there. Step faulting had lifted this aquifer to levels near Hawkhope Farm where artesian pressures of 7·2 m were measured at one piezometer in 1983.

(*c*) Stability analyses of unstable river terraces downstream of the overflow channel were made using the recently acquired piezometer levels in bedrock there and effective angles of friction of 14° determined from laboratory tests of mudstone in its residual condition. Values of almost unity were obtained at a location where incoming electric power transmission lines were routed.

(*d*) Chemical analyses of reservoir water showed marked differences from analyses of groundwater in piezometers near Hawkhope Farm.

56. The conclusions reached were

(*a*) Impounding of Kielder Reservoir showed no noticeable effects on springs and streams at Hawkhope until the reservoir was within 5 m of the overflow sill level.

(*b*) Natural groundwater flows unable to exit to the reservoir were being diverted to downstream areas where fault systems concentrated the flows towards Hawkhope Farm.

(*c*) Potential instability existed in areas of river terrace 0·5 km downstream of Kielder Dam. Electric power transmission lines traversing these areas had to be re-routed.

(*d*) Initial site investigations for impounding reservoirs should be carried out for some appreciable distance downstream of the dam structure to establish the original groundwater regime.

Dr A. D. M. Penman, *Geotechnical consultant*

The pore pressures found by the Authors were exceptional. The ratio pore pressure/overburden pressure r_u of 10·7 obtained from piezometer B54 (§ 27) is remarkably high. In 1951, I measured r_u values of 1·8 in the fill of Usk Dam[12] but this was due to use of a coarse intake filter that allowed the piezometers to measure the pressure of air in the voids of the fill rather than pore water pressure.

58. The Authors used modern fine filters to ensure measurement of pore water pressure but apparently had some difficulty with the read-out apparatus which used only one electrical pressure transducer to which each filter and its pair of tubes was connected in turn. The action of taking readings was thought to upset the balance between the intake filter and the fill: the method of taking readings may have caused some rise of measured pore pressure.

59. If it is assumed that the measured value of r_u is greater than 1, this may be accounted for by an increase of vertical stress above the overburden value caused by arching. Measured values of vertical stress at the John Martin Dam (Fig. 24)

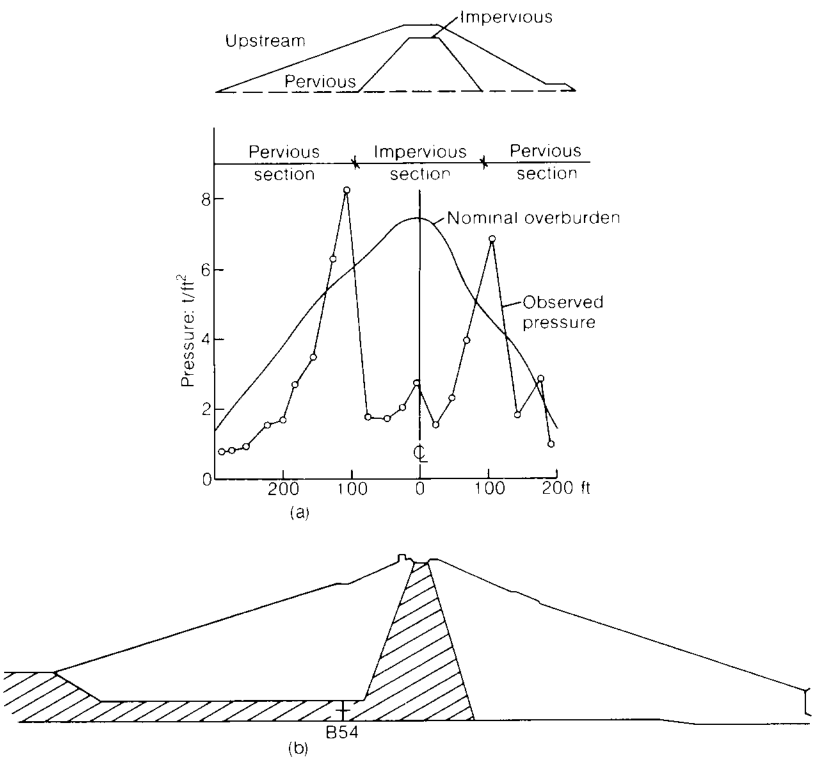


Fig. 24. (a) Vertical stress values at the John Martin Dam; (b) position of piezometer B54 at Kielder

reveal a considerable increase of vertical stress in the fill on either side of the core. Piezometer B54 was in a region where this action could have caused a vertical stress at Kielder greater than the overburden.

60. Drainage layers are important. The high pore pressures measured at Usk, which were confirmed by standpipe piezometers installed in the first year's fill, could have caused failure if the dam had been completed to full height at the proposed rate of construction. The expedient of delaying construction by a year was not acceptable, and to reduce the potential length of drainage path to accelerate drainage layers of gravel and crushed rock were placed over the first and second years' fill. These drainage layers, proposed by Professor Bishop and used for the first time at Usk, were provided as a design expedient in Selset Dam and have since been used extensively e.g. at Derwent, Backwater, Kielder and Carsington reservoirs. Their use enables both core and shoulders to be made from relatively impervious fill.

61. Apparent lack of stability was one problem raised by the Authors. The design section of the dam, with central clay core and upstream internal clay blanket was similar to that of Açu Dam[13] and in some ways similar to Chingford dam.[14] It is therefore not surprising that the factor of safety of the upstream shoulder was monitored carefully.

62. Although measured pore pressures indicated a factor of safety so low that fill placement was stopped when the height was about 7 m below crest level, the movements of the upstream shoulder did not confirm dangerous conditions. The measured horizontal displacement of 100 mm during a 7·5 m rise of fill (§ 34) showed an average rate of movement of 13 mm/m rise of fill. Movements measured at other dams, shown by Table 10, indicated that this rate was not abnormal. Galisteo, Muirhead and the Punchina cofferdam suffered movements that could be regarded as failure conditions. The construction of Derwent Dam was halted because high pore pressures indicated dangerously low factors of safety and a similar situation has arisen with a dam currently nearing completion where, as at Kielder, stability has been assured by the construction of a small berm.

63. The lack of dam movement on impounding is also important. Measurements described by Penman and Charles[15] showed that the horizontal thrust imposed by a clay core on both upstream and downstream shoulders can exceed the horizontal thrust from the reservoir water when the clay is sufficiently soft.

Table 10. Movements measured at different dams

Name	Height: m	Slope: 1 oN	Max. horizontal movement: mm/m height fill
Gepatsch	153	1·5	13
Blowering	112	1·9	10
Llyn Brianne	90	1·75	20
Scammonden	70	1·8	4
Kielder	52	3	13
Galisteo	48	3·2	53
Punchina Cofferdam	45	2	100
Backwater	43	3	3
Derwent	36	2	1
Muirhead	22	2·9	330
Chew Stoke	13	2·5	5

This is a desirable feature because the total pressures within a clay core should exceed the pressure applied by the reservoir water to avoid hydraulic fracture. It is pleasing to see that at Kielder the strength of the core was sufficiently low to allow this to happen, so that the downstream shoulder, already supporting a horizontal thrust greater than that imposed by the reservoir water, was unaffected by impounding, which caused no significant change to the trends of the downstream movement occurring under the clay pressure. A more detailed discussion of this aspect was given by Penman in 1982.[16]

Mr N. J. Ruffle, *Northumbrian Water Authority*

Following Selset in the 1950s and Derwent in the 1960s, this Paper on Kielder can be viewed as the last in a notable series on dams built in the glacial tills of Northumbria. While there are similarities in the composition and behaviour of the clays encountered at these sites, there have also been important differences as shown in Fig. 4. Allowing also for the differing physical characteristics of the various sites, it can be seen why the design and construction of dams of this sort has not become standardized.

65. The transfer of vertical loading from the core to the shoulders has been dealt with by Dr Penman. This transfer was also noted at Selset, where the core was of puddle clay, and it was also noted at Derwent where there was a rolled clay core. It was particularly marked at Kielder with this high value of 1·7 for $\bar{B}$ in the clay blanket. Although the values are not given in the Paper, it appears from Fig. 12 that the value of $\bar{B}$ at piezometer CU51 in the core was correspondingly low, and there is also another piezometer shown in the earlier Paper by Coats and Rocke[2] on the Kielder headworks, CU52 (which is above CU51), which tells the same story. Were the Authors ever concerned that this process of stress transfer would go so far as to prevent the rise in piezometric pressure in the core from rising above reservoir top water level which was desirable to prevent any possibility of hydraulic fracturing?

66. As the embankment rises towards completion, factors of safety fall rapidly and this redistribution of stress takes place. Do the Authors have views on the allowable rate of raising the bank at that time, or has the accumulation of geotechnical experience reached the point where the Contractor can confidently be allowed free rein, subject only to calling a halt if the factor of safety falls too far?

67. Table 5 shows substantial differences in undrained shear strength between undisturbed and remoulded samples. On average there is a 25% difference. How are those differences dealt with in producing Fig. 10(a), which is said to embrace all the results? It is also stated that Fig. 10 shows that quality control was satisfactory. Referring again to Fig. 10(a), where the proportion of results specified to fall between values of 70 and 130 kN/m^2 was 80%, what was the actual percentage achieved?

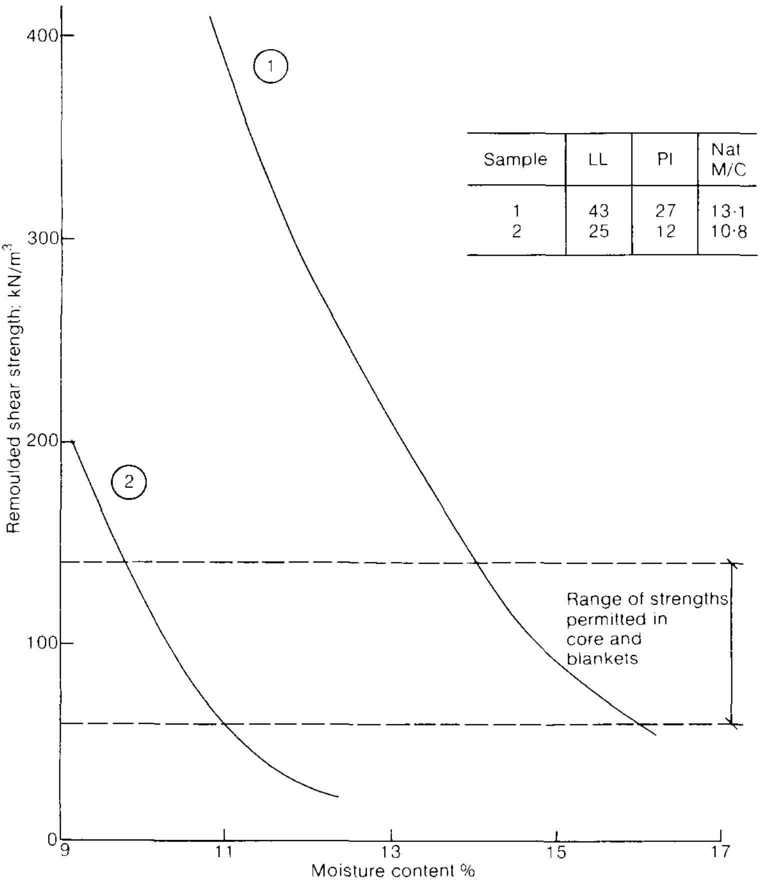
68. The question of damage to instruments, especially those projecting vertically from the bank, has always been a problem. Have the Authors resolved to do anything different on another occasion?

Mr R. Cotton, *Fairclough*

The Authors mentioned in their Paper the difficulty of carrying out a full site investigation of potential borrow pits due to the heavy afforestation. Accordingly, the joint venture instigated an additional investigation at Sandboard Knowe at the very beginning of the job to assist them in the economic planning and proper selection of materials for the core and shoulders of the dam.

70. It was necessary to establish the range of in situ strengths in the borrow pit, and then the relationship between strength and moisture content had to be determined. A wide range of materials existed in Sandboard Knowe. At one end of the spectrum there was wet non-plastic sandy tills, and at the other end very stiff cohesive tills with undrained strengths as high as 350 kN/m.[2]

71. Problems were also met with the thick overburden of soft unusable material. In some cases about 5 m of very soft unusable material had to be taken to tip, and below that some fairly soft glacial till occurred. Fig. 11 shows the differing shear strength moisture content relationships that are present in Sandboard Knowe. Fig. 25 shows typical relationships. Sample 1 is a very sandy till, with a fairly steep shear strength moisture content relationship, and Sample 2 is a much more cohesive till. There is a much wider moisture content range for the core with



Sample	LL	PI	Nat M/C
1	43	27	13·1
2	25	12	10·8

Fig. 25. Relationship between shear strength and moisture content for typical materials in Sandboard Knowe borrow pit

the more cohesive till than there is with the very sandy till, 2% compared with 1·2%. Both these materials were present in Sandboard Knowe mixed together, and it was very difficult to select one from the other.

72. Figure 26 shows the range of in situ strengths in Sandboard Knowe, and the range of moisture contents. Both covered an enormous range, and there was no clear relationship between shear strength and moisture content. Because of these large differences in material type, some means had to be found to select materials best suited for either the core or shoulder of the dam. After a great deal of laboratory testing of materials taken from the borrow pit, the joint venture decided to place the more cohesive tills in the core and to place the dry sandier tills in the shoulders of the dam. All wet sandy tills were discarded as unusable.

73. Since the glacial tills in general had strengths at the upper end or in excess of the upper limit for the core, it became clear that watering would be required for the core and blanket. This very difficult operation was controlled by visually selecting the most uniform material in the borrow pit, and this required moving plant around frequently from spot to spot. The most uniform material was selected, and a large number of shear vane tests were taken on the dam. We would then look at the visual appearance of the traffic material during the passage of earth moving machinery, and the rut depth under the scrapers or dump trucks. Generally a rut depth on the core of about 12 in resulted from a single pass of a large scraper. Finally, the strength would be checked using the triaxial test on remoulded samples.

74. The joint venture had a well equipped laboratory with six technicians. Triaxial testing gear was available, and any of the standard soil testing that was necessary could be done. This large complement was needed to control the 24 hour a day earth moving operation. It is very difficult at night to select materials under floodlights in the borrow pit. However, I think the operation was successful.

75. During 1978 the joint venture became concerned that it would be very difficult to ensure an adequate supply of material for both core and shoulder from Sandboard Knowe of the right quality and, in particular, in the quantities necessary for the 1979 earth moving season. In that particular season there was anticipated the placing of a minimum of $1{\cdot}6 \times 10^6$ m^3 of fill within the embankment, and desirably a volume of the order of $1{\cdot}8 \times 10^6$ m^3. In close co-operation therefore with the Engineer, the joint venture began a detailed investigation of other possible sources, resulting in the selection of Pea Croft borrow pit. However, before exploitation of the borrow pit it was considered prudent to have an independent study of the glacial landforms of the Kielder Valley. This work, carried out by Dr Edward Derbyshire of Keele University, was proven by extensive in-house shell, auger and rotary drilling in both Sandboard Knowe and Pea Croft. In both cases continuous U4 sampling was attempted in order to get a better idea of the in situ undrained shear strengths of the tills. Despite severe weather conditions in the early months of 1979, more drilling was carried out at the borrow areas at this time than had been possible at the design stage.

76. As a result of these investigations, the joint venture decided with the agreement and support of the Engineer and the Employer to proceed with Pea Croft borrow pit, and after the construction of a new $1\frac{1}{2}$ km haul road, the diversion of the River North Tyne and the construction of a small bridge over the Whickhope berm, removal of overburden started in April, enabling filling to the dam to commence at the end of May.

77. The success of the operation can be judged by the fact that around 2×10^6

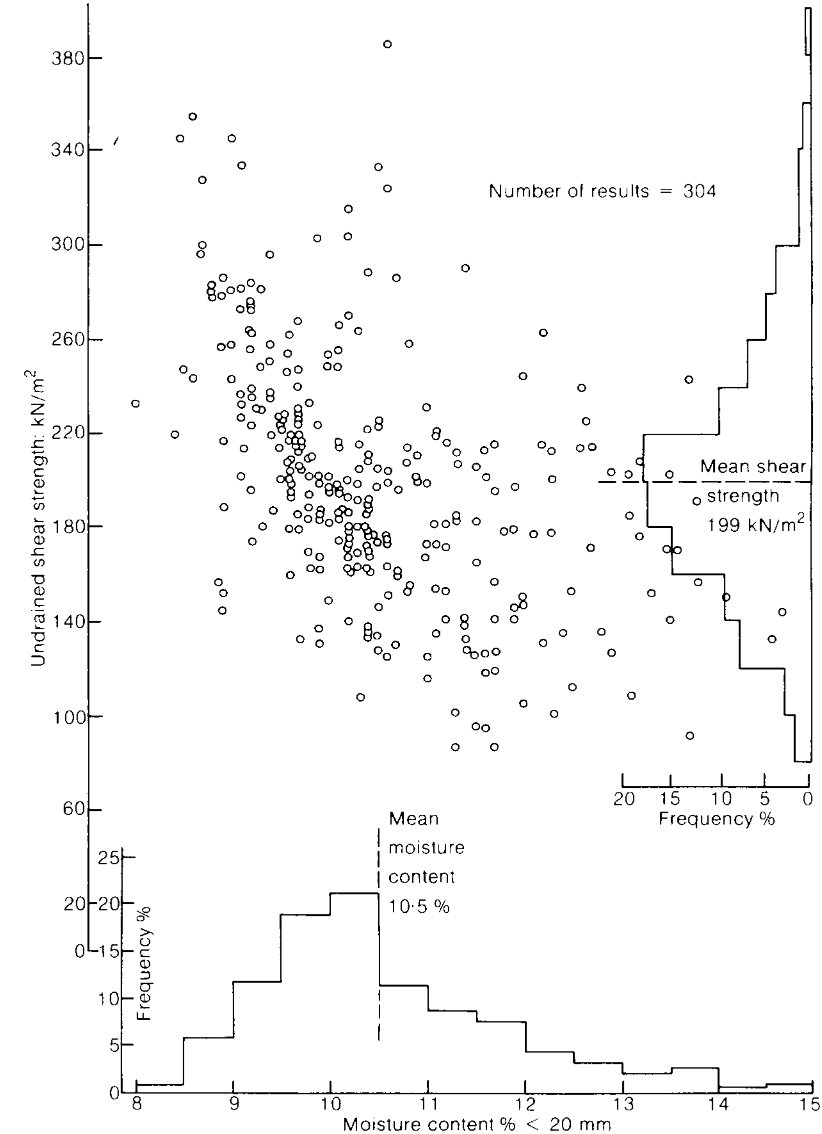


Fig. 26. In situ shear strength against moisture content in Sandboard Knowe borrowpit

m^3 of fill came from Pea Croft, and less than 2% was unusable. This excludes the overburden, which again is about 2 m thick.

78. The glacial tills at Pea Croft were more consistent than those excavated at Sandboard Knowe with a much more uniform shear strength/moisture content relationship as can be seen from comparing Figs 26 and 27. As a result it was

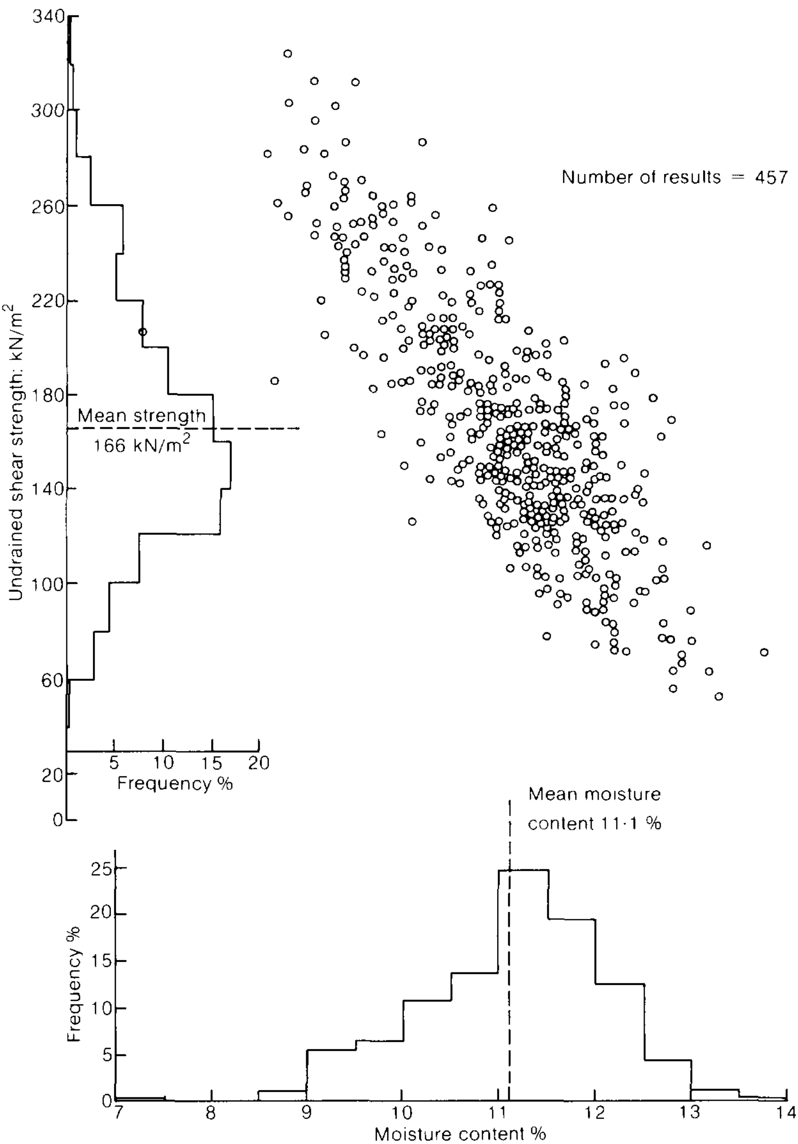


Fig. 27. In situ shear strength against moisture content in Pea Croft borrow pit

possible to use motor scrapers in Pea Croft and thus speed up the operation. Also the control of shear strength was much easier and this and changes in testing method allowed very high rates of compliance to be achieved.

79. Tests carried out between ourselves and Sandberg, for the core using remoulded tests, are shown in Fig. 28. Similarly, the results for the shoulder of the

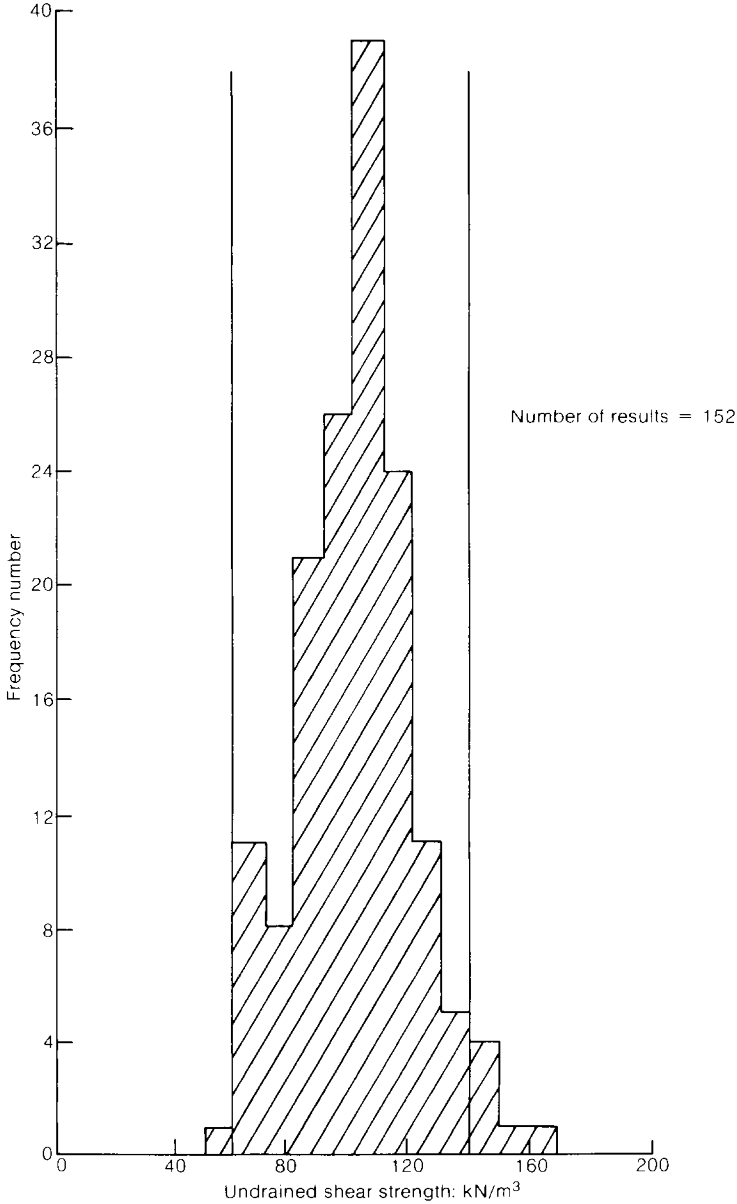


Fig. 28. Undrained shear strengths for core material excavated from Pea Croft borrow pit between 16 August 1979 and the end of 1980

dam, for Pea Croft material excavated and placed in 1981, show almost 100% compliance with the specification (Fig. 29).

80. I was responsible to the Project Manager for the control of all geotechnical matters connected with the selection of materials and the construction of the dam. From the very beginning the joint venture established a large laboratory that ran in parallel with that of the Engineer, which enabled the joint venture to plan comprehensively their material selection and embankment construction. While at the beginning it would perhaps be true to say that the Engineer wondered at the need for such a dual laboratory system, there can be no doubt that the joint venture testing programmes that were ultimately developed were enormously successful.

Mr D. J. Coats, *Babtie Shaw and Morton*

Referring to the control testing of placed fill during construction, Table 4 shows that 4604 shear strength tests, almost equally divided between the core and the shoulder, were carried out over three years between June 1977 and July 1980; but in fact active placing was done only for about 90 weeks during that period due to the short construction season. Testing averaged one test for every 900 m^3 placed, over 50 tests per week, and this does not include the tests done on the placed fill which Mr Cotton mentioned, or the tests carried out in the borrow pit. Is this order of testing to be expected on earth embankments?

82. Table 4 shows that although unconfined compression tests and plate bearing tests were attempted, it was not possible to use these extensively. The reason for attempting them at all was to obtain early results, and in the case of the plate bearing tests to avoid handling samples and preparing them, and by testing a relatively large volume in situ to minimize the effects of the local variations in moisture content, particularly when water spraying was being used.

83. The unconfined compression tests were unsatisfactory as the sand lenses in the samples caused early failure, and construction traffic and plate bearing tests are apparently incompatible. If the rate of testing used at Kielder is required, a simple in situ test, confirmed as necessary by occasional triaxial testing for comparison, must be identified. It must be acceptable to both Contractor and Engineer. The situation is now very far away from the heel indentation test and tyre rutting depths, which have also been referred to as subjective and open to challenge; but what else is suggested? Something could be produced that could give a continuous indication of the shear strengths in the material when placed, and something that is also acceptable to all concerned and agreed within the industry.

84. The importance of fast communication of results from site to designer must be emphasized. Information tends to creep single file and be ponderously processed through lengthy organizational conduits. There is also a desire to double check and beautify each report so that it will stand the scrutiny of the ages. The standard of reports received from Kielder was good, but there was no doubt about the benefit of Mr Millmore in the design office keeping in close touch with Mr McNicol and others on site, so that unhealthy trends were identified without delay. Mr Millmore's presentation showed he had visited the site sufficiently to be aware of the site conditions.

85. Whether the berms described in §§ 38 and 39 were essential will never be known; but the factors suggesting their need were quickly recognized, and they allowed a subsequent rapid rate of construction and avoided unnecessary interruptions. The first berm was anticipated, and was put in at the end of a construc-

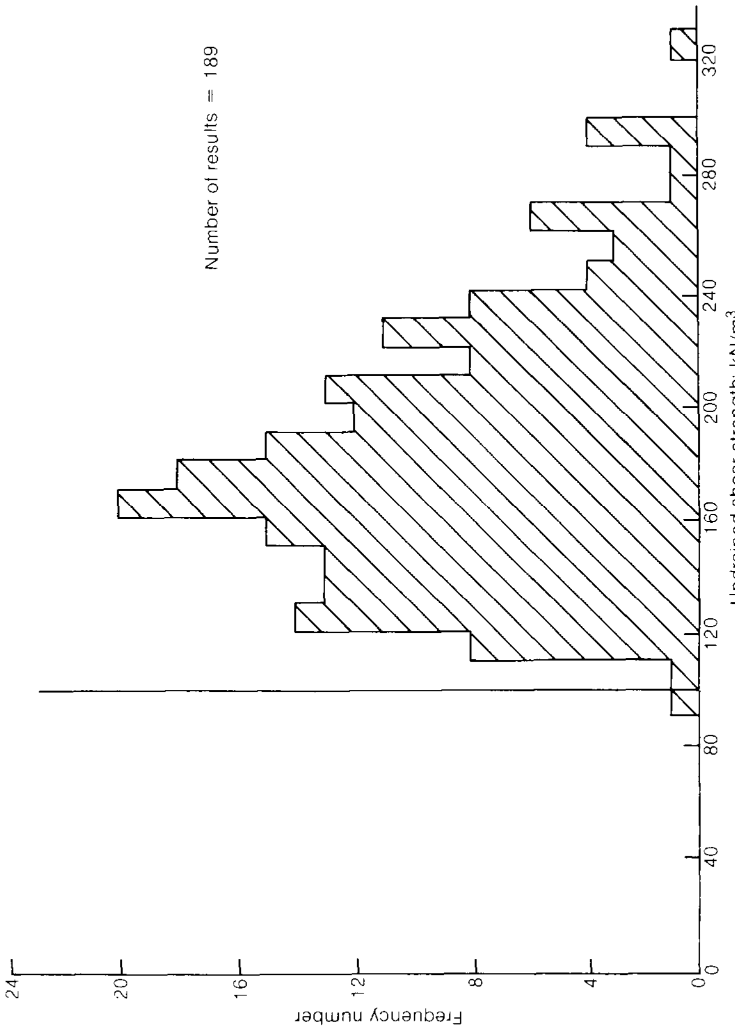


Fig. 29. Undrained shear strengths for shoulder material excavated from Pea Croft borrow pit in 1980

tion season, causing no delay. The second one was put in because difficulty was foreseen, and was completed in a matter of days so that delay was minimal.

Mr M. F. Kennard, *Rofe Kennard and Lapworth*

My main point refers to § 37 and the fact that the pore pressure increases had reduced the factor of safety so considerably. A drop from 2·5 to 1·4 in only one week is very considerable and very surprising. How was that controlled in practice? Were continuing stability analyses being carried out so that information was available within a day of it occurring, or were there certain limits on pore pressure rises on certain piezometers being used as a means of control?

87. That there was no specified rate of construction is surprising, because if the design is based on certain pore pressures at certain times of construction based on consolidation, I would have expected some limit to be specified. This was certainly the case for the Selset Dam where limits were imposed and worked very successfully.

88. The critical slip surface was shown to pass through the core, the base blanket and the cofferdam, which are all in rolled clay to the core specification. Would the Authors now consider it necessary to have such a thick base blanket, and to use clay for all the cofferdam?

89. Mr Rocke's contribution regarding water in the hillside reminds me of similar circumstances at both Selset and Borderhead dams, also in the north east of England. In both cases there was an increase in groundwater conditions in one abutment, and it was also concluded it was not due to reservoir water, but to redistribution of the groundwater because of the reservoir upstream of the locations. He also mentioned the use of water analyses that showed that the water was not from the reservoir. I have also tried this, and have had great difficulty in getting satisfactory results, because it is no longer reservoir water that is being measured when it comes downstream. It has become groundwater, and considerable changes from reservoir water would have been expected.

90. What was the reasoning behind the positioning of the inclinometers? Putting an inclinometer in the centre of an earth dam is inviting trouble during construction, and I suggest that as much information could be had on the movement along a potential slip surface if it were nearer the upstream or downstream toe, which would be only in a very short length of fill.

91. The Authors also brought up the tubes ahead of the fill. Another method which has been used is to bury each length, dig down and extend it so that the amount protruding is considerably reduced.

92. The movements shown in Fig. 18 showed that the instrument house at the downstream toe hardly moved at all, yet the downstream face of the higher magnetic probe extensometer system moved very much more. Was the lack of relative movement due to the massive nature of the lower instrument house?

Mr C. J. Sammons

With reference to control testing of the undrained shear strength of the clay core fill, I understand that various methods were experimented with during construction and that it was some time before a test procedure was agreed upon. The criterion eventually adopted required that the average triaxial remoulded shear strength of five samples taken from an area should be within specification. This averaging allowed considerable local variation in strength. Were you quite satisfied with this control test procedure? If so, might you have been unnecessarily penalizing the Contractor during the first year and a half's fill placing when stricter limits were used? I understand that high rates of fill rejection caused the earthworks sub-contractor serious problems. Could the method of control testing have been sorted out before construction began and have been more clearly specified in the Contract?

94. Mr Coats said he would like to find a field index test that could be used alongside triaxial testing to help control placing the sort of clay fill found at Kielder. The TRRL's 'Moisture Condition Test' could be tried. The test is simple and quick. I recently specified its use to control stiffness of a gravelly clay core fill and this worked well.

95. Why was the arrangement of a vertical core with an upstream clay blanket chosen? Was it to separate the small amount of grouting you had to carry out from the main construction work? Did you consider using an upstream sloping core? It was also surprising that a filtered drainage interceptor wall was not provided downstream of the core.

96. Dr Penman suggested monitoring the surface movement of earthfill dams during construction as a method to assess their stability. Whether this can safely be done depends on the type of clay used to construct the dam and on foundation conditions. The fill material at Kielder has a low clay fraction and thus behaves in a non-brittle manner. The foundations are also relatively strong. If the factor of safety of the dam had approached unity adding further fill on top would just have caused some balancing outward movement of the toe. The main consequence is likely to have been some disruption of the construction programme. Movement measurement could certainly have been used to monitor stability. However, it would not be safe to rely on measurement of surface movements where a dam was built of a more plastic brittle fill, such as London clay, or where weak foundations were involved. They might give little warning of sudden major shear failure.

Mr Cockshaw, *Fairclough*

There was a dyke at Plashetts that was clearly identified in the contract, and the contract clearly pointed out that aggregates could be obtained from there. The joint venture produced all its coarse aggregates from there. Problems with flakiness arose and the manufacture of 10 mm aggregate had to be suspended and the aggregate was afterwards imported. The coarse aggregates, however, were mainly produced from there. A significant amount of intermediate layer drained material was produced from the river bed, and also, as an unexpected by-product, good quality sand.

98. As contractors, one of our problems was that of having a limited number of vehicles allowed on the public road. This was looked at very carefully so that volumes of material could be controlled very effectively. There was a lot of information on site investigation and it was through its use that the opportunities developed. The joint testing took some time to get established, but worked very well indeed, and there was a close working relationship between the Engineer and ourselves.

99. Pea Croft gave maximum flexibility at a time when maximum flexibility and high outputs were required, and the Engineer and the Employer worked very closely with us in developing that major area. It should be remembered that at the beginning it was very heavily forested, and it was very difficult to do site investigation. We had the benefit of having a clear site to go in and take additional holes, which of course we did together.

100. Possible damage to instrumentation by contractors is a very difficult problem, particularly when working round the clock, and damage does take place. There must be a balance between the number of instruments that are placed in the dam, and the difficulty that they automatically create in trying to do the job properly and effectively. Preserving them would perhaps be easier if there were less of them. Reliability is the crucial factor. There is a great deal of instrumentation and although I accept that some of them did not work very well, I believe there are a large number of instruments still working effectively. The monitoring that has gone on both during construction and after the job was finished bears testimony to the fact that what is happening at Kielder Dam is known.

Mr Millmore and Mr McNicol

Dr Penman and *Mr Ruffle* both refer to the high pore pressures measured in the clay core and internal blanket during the construction of Kielder Dam. These pressures are also related to other issues raised during discussion.

102. The possibility of measuring pore air pressure in the piezometers was referred to by *Dr Penman*. The modern ceramic tips to which he referred were used to overcome such problems at Kielder and the piezometers were de-aired when necessary. More than 70 m head was measured at piezometer B54 and at this pressure of water, any air was likely to be fully dissolved. We did not experience any difficulty with the read-out apparatus. The calibration of all equipment was regularly checked to account for minor drifting that occurs on all such equipment. We consider that a reading frequency of once every two days was sufficient to restore equilibrium of the piezometer/fill system. The rate of earth filling made it unnecessary to read the piezometers more frequently.

103. Most of the piezometers in the core and blanket shown in Fig. 30 indicated high pressures. However, the pore pressure at piezometer B54 was the highest measured in any piezometer in the embankment. Table 11 summarizes the maximum pore pressure values for all piezometers in the core and blanket during the final earth-filling season. From this table it can be seen that the pore pressure ratio r in piezometer B54 was 0·8, not 1·7, as indicated by Dr Penman. Paper 8703 stated that it was the ratio of increase in pore pressure to increase in weight of earth fill (B) which reached 1·7. As *Mr Ruffle* indicated, the B values measured at piezometers CU51 and CU52 were not as high as in piezometer B54. However, the pore water pressures in both piezometers were high and CU51 was only slightly lower than the adjacent B54.

104. *Dr Penman* and *Mr Ruffle* both agreed that the B values greater than 1·0 in the blanket were most likely a consequence of the stress transfer within the dam. The pattern of stress transfer at Kielder Dam can be seen from the piezometric profile shown in Fig. 20. The difference between this profile and that at John Martin Dam described by Dr Penman is most likely related to the internal geometry and soil parameters at both sites. The piezometric values in all piezometers within Kielder Dam were plotted continually and estimates of the final pore pressures within the dam were made regularly. On the basis of these, there was never concern that the stress transfer would be such that the pore pressures within the core would be lower than the reservoir top water level. Hence there was no concern about hydraulic fracture of the core. These predictions of the final piezometric values were also used in the monitoring of the stability of the dam during construction.

105. The thickness of the clay blanket no doubt did contribute to the high pore

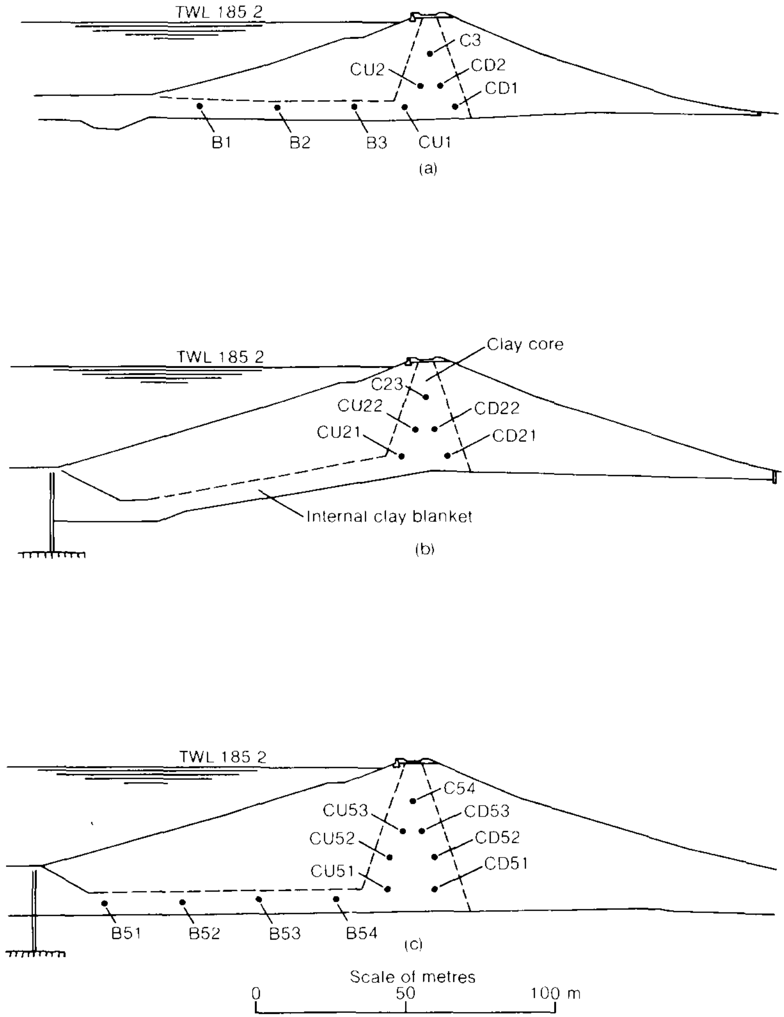


Fig. 30. Cross-section of Kielder Dam showing the core and blanket piezometers: (a) south end of dam; (b) intermediate section (c) river valley

pressures as suggested by *Mr Kennard*. The blanket thickness was initially selected as 6 m on the basis of laboratory test results. In situ parameters subsequently dictated a thinner blanket of 5 m. We did not wish to incorporate a rolled clay blanket any thinner that 5 m since this formed part of the impervious zone in the dam. With hindsight, it may have been appropriate to construct the clay blanket a little drier than the core material by specifying a higher strength for the soil. This would have produced lower pore pressures.

106. The rate of movement of the upstream face referred to by *Dr Penman* may

Table 11. Maximum pore pressures in core and blanket piezometers—final earth filling season (1980)

Piezometer	Max pore pressure: *u* m	Max pore pressure ratio *r*	Max pore pressure ratio *B*
South end of dam			
B1	175	0·6	*
B2	182	0·7	*
B3	200	0·8	1·4
CU1	200	0·7	0·9
CU2	186	0·5	0·7
CD1	185	0·4	0·6
CD2	192	0·6	0·9
C3	192	0·7	0·8
Intermediate section			
CU21	199	0·8	0·8
CU22	190	0·7	0·4
CD21	198	0·7	0·7
CD22	190	0·5	0·6
C23	F	F	F
River valley			
B51	164	0.5	*
B52	187	0.8	*
B53	190	0·7	*
B54	209	0·8	1·7
CU51	205	0·8	0·8
CU52	195	0·7	0·6
CU53	188	0·6	0·6
CD51	202	0·7	0·8
CD52	196	0·7	0·8
CD53	181	0·4	0·5
C54	187	0·4	0·6

Reservoir Top Water Level = 185·2 m AOD
u = Pore water pressure in metres AOD
* = Indeterminate—increase in *u* without any direct fill increase
F = Faulty instrument

not have been abnormal. However, the larger internal movements within the dam together with the high pore pressures warned of a reducing factor of safety against shear failure. It may be that, as indicated by *Mr Sammons*, measurements of movement at the upstream face may not be appropriate with the Kielder glacial till. Inclinometer I10 became blocked at the estimated position of the potential slip surface (Fig. 15). An inclinometer near the upstream toe would have proved extremely useful in measuring movement as indicated by *Mr Kennard*. However, such a position for an inclinometer would have penetrated the impervious clay blanket which clearly was not desirable. The movement in the downstream shoulder was monitored more extensively because the problem of creating leakage paths did not apply. *Mr Kennard* wondered whether the massive nature of the instrument house may have affected downstream tow movements. However, Fig. 17 showed that the progressive reduction in downstream movement was fairly uniform along the base of shoulder. This was most likely due to the resistance to movement offered by the foundation of the dam.

107. Control of the rate of construction of a dam embankment is common practice as indicated by *Mr Ruffle* and *Mr Kennard*. The design of the embankment allowed for a fast rate of construction. It was decided that at Kielder the method of controlling construction would be to carefully monitor the embankment behaviour and to impose restrictions on construction only if this proved to be necessary. This method proved successful at Kielder.

108. The monitoring of the dam behaviour during construction did involve continual analysis of the stability as suggested by *Mr Kennard*. At the peak period of construction an assessment was made every two days. A programmable calculator was used in this analysis of the potential noncircular slip. The analysis was completed within about 15 minutes of obtaining the piezometer readings.

109. The testing of the undisturbed shear strength of the fill material attracted considerable interest during the discussion as it did on site. The difference between the undisturbed and remoulded strengths referred to by *Mr Ruffle* was examined in detail. An analysis of a larger number of strength tests than was shown (in Table 5) indicated that generally the shear strength results of remoulded samples were only marginally higher than undisturbed samples. This was attributed to the loss in moisture during the remoulding process. No adjustment was therefore made to the test results in Fig. 10 to account for differences in the method of testing. This figure included the results of all material tested on Kielder Dam. It was not a reflection of the earth fill in the completed embankment because material that was outside the Specification Limits was reworked until it complied with the Specification. Taking this into account, the test results of all material in the embankment were within the outer specified limits and 75% of the results lay within the '80% limits' of 90 to 130 kN/m.

110. *Mr Coats* referred to the frequency of testing and to the techniques adopted at Kielder Dam. The Authors would concur with his view that there is a need for development in methods of site testing of soils to provide quick, reliable results. The moisture condition test referred to by *Mr Sammons* may prove useful in this aspect, but with the shear strength specification and the wide range of glacial till at Kielder it would require extensive calibration before it could be used with confidence. Mr Sammons refers to the various methods of testing, adopted on the embankment. The test methods were constantly reviewed since no control test was found to be ideal in producing quick reliable results. In the same way, the Contractor constantly reviewed his method of winning and placing fill material. With this diligent approach it is only to be expected that a much lower percentage of test failures would be realized in the later stages of construction. The test procedures used in the initial stages of the embankment construction were unconfined compression tests. As indicated by *Mr Coats*, the samples in these tests failed prematurely. Therefore, soil, which in reality was too strong for the core and blanket, appeared to have satisfactory test results. The failure rate in the early stages of the embankment should therefore have been higher than measured and not lower as suggested by *Mr Sammons*. The limitations of the various test methods adopted at Kielder demonstrated that it is not prudent to specify the method of strength testing in the contract documents.

111. Investigations into the groundwater pattern on the north hillside downstream of Kielder Dam were being conducted at the time of preparing Paper 8703 (see § 41). *Mr Rocke* provided the details of the outcome of this investigation.

112. The arrangement of the vertical core and upstream clay blanket was referred to in § 14 and was described in more detail by Coats and Rocke.[2] One reason for not incorporating an upstream sloping core mentioned by *Mr Sammons* was the increased risk of a slip surface developing. A sloping core would have provided a shorter potential slip surface in core material and therefore a lower factor of safety. A filtered interceptor chimney drain downstream of the core was not incorporated since this type of drain would have encouraged drying of the core material, increasing the risk of hydraulic fracture.

113. *Mr Cotton* demonstrated the extensive geotechnical investigation and testing conducted to meet the construction requirements of the Contractor. This work included an assessment of the variations in two borrow areas comprising two adjacent drumlins of glacial till. Mr Cotton highlighted the importance to a Contractor of detailed site investigations and sound interpretation of results.

114. *Mr Cockshaw* remarked on the close working relationship between the Contractor and the Engineer. This co-operation was not only on the soil testing and earthworks, but took place in every aspect of the Contract.

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