Four Unusual Cores for Fill Dams

by

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SUMMARY. The processed claystone at Mangla Dam, the gravel-sand-silt at Tarbela Dam and the asphaltic concrete and sand bitumen at the High Island Dams are described in this paper.

1 INTRODUCTION

The cores of earth and rockfill dams are usually composed of naturally occurring plastic fine-grained materials, such as silty clay because such materials are more conveniently and economically available. However, situations occur when such is not the case and unconventional solutions must be adopted. This paper will describe four such solutions; namely, the processed claystone at Mangla Dam, the gravel-sand-silt at Tarbela Dam, and the asphaltic concrete and sand bitumen at the High Island Dams.

2 MANGLA DAM PROCESSED CLAYSTONE

Mangla Dam is located in northern Pakistan on the River Jhelum and forms part of the Indus Basin Project; details of which have been described by Binnie et al (Ref. 2). The dam is sited in low foothills but the 33,000 sq. km. catchment is mostly in the high Himalayas. River flows reach a maximum in summer when snow melt and monsoon rains coincide and flows up to 30,000 cumecs have been recorded. Such high flows carry all colloidal clay sizes far away downstream of the dam and the alluvial terraces remaining at the site consist mostly of boulder-gravel-sand sizes with some basins of silt sizes. No plastic fine to coarse grained soils exist in any quantity in the dam area.

The low hills at the dam site consist of fresh water sediments of the Siwalik system of Plio-Pleistocene age which comprise intercalated beds of sandstone, siltstone and claystone of variable thicknesses of up to 30 metres. The sandstones are mostly poorly cemented while the siltstones and claystones are quite hard but a bed containing an appreciable clay fraction shrinks rapidly and develops cracks when exposed to air. These beds then slake rapidly with alternate wetting and drying, producing a shallow slope of debris. At the site, some 1200 to 1800 metres of Siwaliks have been eroded so that the deposits are, in the soil mechanics sense, heavily overconsolidated. The unconfined compression strength and elasticity modulus of insitu materials are of the order of 3000 kN/m^2 and 700,000 kN/m^2 respectively and the material can be classified as a weak to medium strong rock.

The major site works include two spillways, an intake and a power station tailrace; all of which required large excavation containing considerable quantities of claystone. The properties of the claystone were therefore investigated in detail during the site investigation. This included several small trial embankments which demonstrated that the claystone could be readily broken down and compacted. Claystone was therefore accepted for core construction and other proposals such as an impermeable upstream membrane or a silt core were abandoned. The quantity of processed shale required for the core was 8.3 million cu.m. or 12% of the total fill volume.

The cross section of the dam showing the inclined processed claystone core is shown in Fig. 1 and the range of specified core material is shown on Fig. 2. The effective shear strength parameters used for design were 22 kN/m^2 for cohesion and 22 degrees for friction angle.

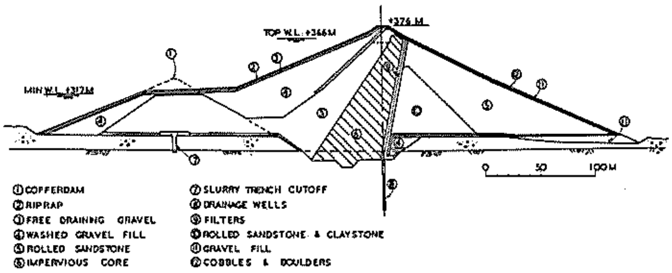

Fig. 1 Mangla Dam

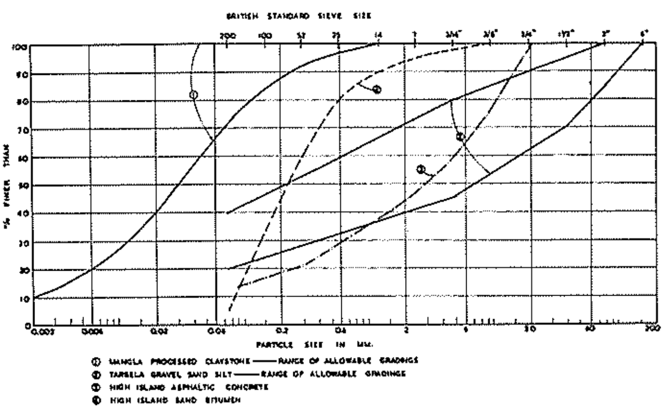

Fig. 2 Gradings of Core Materials

The claystone was removed from required excavation by dozer assisted self-propelled scrapers after the material had been broken up by rippers working to a depth of about 750 mm. Initially the claystone was stockpiled where it was broken down by a 45 tonnes Hough Paydozer; an articulated rubber-tyred tractor fitted with steel grid wheels. The claystone was broken down to a powder matrix with lumps up to 120 mm. in size to which water was added to increase it from the natural moisture content of about 8% to the optimum value of about 15%. For the purposes of laboratory and field testing, the lumps in the fill were considered as discrete particles and were not broken down in such tests as laboratory compaction. As the dam progressed, the pulverising of claystone was carried out in the excavation site at natural moisture content and the necessary water was later added on the dam in conjunction with disc harrowing and mould-board gang plowing. Inspection pits dug in the fill after some time showed that it had become reasonably homogeneous; the moisture content of the lumps having increased while that of the matrix had decreased.

Compaction of the clay was carried out using a roller with five independently suspended rubber-tyred wheels, each with a load of 9 tonnes. The fill dry density obtained was 1.89 tonne/cu.m. which was 100% of the Standard Effort Laboratory Maximum Dry Density (Test 11, BS 1377), while the average fill moisture content was 1% above the optimum moisture content of 15%. The maximum pore pressure response during construction was about 60% of the applied fill load.

The successful filling of the reservoir and operation since early 1967 have confirmed the choice of a processed claystone core.

3 TARBELA DAM GRAVEL-SAND-SILT

Tarbela Dam is located on the Indus River about 130 km. northwest of Mangla, and in a similar topographical situation. Details of the project have been described by Binger (Ref. 1). Like Mangla, the very large flows of the Indus have carried away all colloidal clay material, leaving behind only terraces of boulders-gravel-sand and of silt. The local rocks are of Precambrian and Permian age and are a complex combination of schists, dolerites and limestones.

The only natural impermeable material available for core construction was silt. However, because of the unfavourable experiences elsewhere with non-plastic silt cores when seepage water along differential settlement or construction cracks caused erosion and failure, the silt was modified to render it self-healing. This was accomplished by blending angular gravel and sand sizes to the silt to the specified grading limits shown on Fig. 2 so that they formed a built-in self healing filter to the extent that erosion of silt from a crack would leave behind a material which would act as a filter to prevent further silt erosion. This was demonstrated in laboratory tests carried out in a 400 mm. diameter permeameter containing core material in which a 12 mm. to 3 mm. tapered crack had been artificially formed. Erosion through the crack caused the sides to collapse and choke off further erosion and seepage; the permeability of the material in the crack being determined as only 10 times greater than that of the uncracked core material. The grading limits shown in Fig. 2 are linear rather than the ideal graded C-shape as it was found that the former had less tendency to segregate; an important consideration for a material with a gravel content of about 25%.

The cross section of the dam shown on Fig. 3 incorporates an inclined core for various reasons of which those pertaining to a gravel-sand-silt core are that (a) it results in less differential settlement compared to a vertical core and (b) the lower stress range causes dilation under shear loads and hence advantageous negative pore pressures.

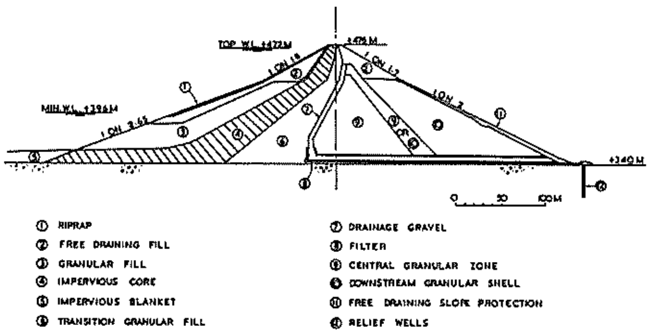

Fig. 3 Tarbela Dam

The main and subsidiary dams of the project required 29 million cubic metres of blended core material which represented 16% of the total fill volume. The silt and alluvium borrow areas from which the core and part of the other zones were won were located about 10 km. from the main dam and a large processing plant near the borrow area and a system of conveyor belts with a maximum capacity of 14,400 t/hour were provided to transport the material to the dam.

The average material placed in the core was midway between the grading limits shown on Fig. 2. Triaxial shear tests were carried out on prototype material in apparatus taking 600 mm. dia. by 1500 mm. high specimens, as well as on 150 mm. diameter specimens of modelled gradings. The tests showed behaviour similar to granular material in that the tangent to the Mohr's circles decreased with increasing normal stress. The results confirmed the design effective angle of friction (consolidated undrained condition) of 34 degrees up to a normal stress of 1000 kN/m^2 and 22 degrees thereafter. Permeability tests carried out on prototype material in the 600 mm. triaxial apparatus indicated K values of about 5 X 10^{-6} cm/sec.

The core material was placed in 300 mm. layers and compacted with either a 100 t pneumatic tyred roller or a 10 t (deadweight) vibrating roller. The insitu density achieved was 2.28 t/cu.m. which was 97% of the Modified Effort Laboratory Maximum Dry Density (Test 12, BS 1377); a fill moisture content of 6.5%, which approximates Modified Effort Optimum Moisture Content being used.

4 HIGH ISLAND ASPHALTIC CONCRETE

The High Island Water Scheme Reservoir will be formed by constructing two rockfill dams at either end of the narrow sea channel between High Island and the mainland peninsula of Sai Kung. Details of the scheme have been described by Vail (Ref. 6). Because of their height of 100 metres, the dams will be founded on bedrock which, in the channel, is about 30 m. below sea level. To enable the construction to proceed in the dry, low cofferdams have been built on the seabed on both sides of both main dams.

The local rock is volcanic rhyolite which has such a thin soil cover that there is insufficient material of acceptable plasticity to form a natural earth core. Earthfill is available outside the catchment but this would involve long hauls and the formation of unsightly borrow areas which are considered unsatisfactory in a small area such as Hong Kong. Alternate cores were therefore considered. Deck type membranes of either concrete or asphaltic concrete were rejected because the need to store water as the dams rose would not allow the usual practice of forming the membrane after the dam was built and partly settled. A flexibly jointed concrete core was rejected because of the uncertainty of the joints. An asphaltic concrete core was finally adopted (Fig. 4), making the cores in the High Island dams the highest built so far and about 60% higher than in the next highest Wiehl dam.

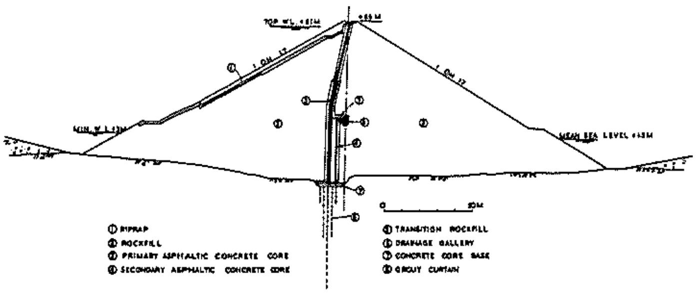

Fig. 4 High Island Dams

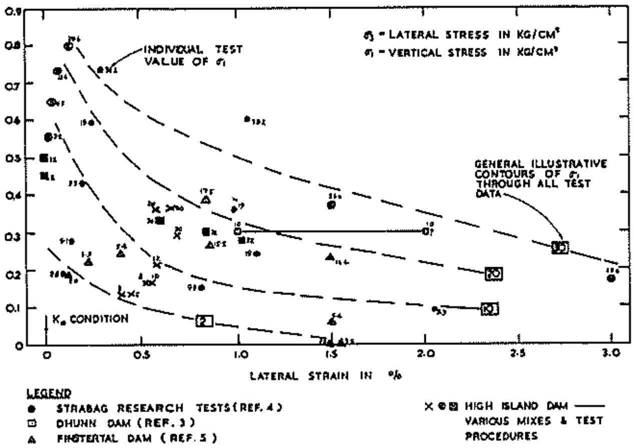

Fig. 5 Asphaltic Concrete Stress-Strain Behaviour

The asphaltic concrete core consists of a well graded mixture of coarse and fine aggregate, sand and filler with 6.3% bitumen, the combined grading of which is shown on Fig. 2. It is similar to that used for road surfacing. Asphaltic concrete is a plastic material with creep characteristics, and a laboratory investigation of these properties was carried out using specialised triaxial apparatus developed by Strabag, the head of the consortium of sub-contractors now placing the High Island cores. The apparatus has been described by Rienossl (Ref. 4) and enables long term loadings at constant temperatures (20°C was adopted for High Island) to be applied. The usual testing programme is to determine the ratio of lateral to vertical stresses for the core at zero lateral strain (the K_o condition) and then to allow lateral strain to occur and remeasure the stress ratio. Typical results obtained for the High Island core material and for cores in several European dams are presented on Figure 5, which shows that K_o increases with increasing vertical stress to quite a high value of 0.8, but that this drops considerably as even small horizontal strain is allowed. Despite differences in location, similar ideal graded asphaltic concrete was adopted for the various dams and this similarity is reflected in the test results so that general contours of stress ratio to lateral strain at various vertical stresses can be drawn through all data on Fig. 5.

Compatibility of stress-strain behaviour between the core and the rockfill shoulders is of prime importance. The well graded angular rockfill at High Island has an angle of friction of about 40° and a K_o value therefore of 0.35 although it may be higher in practice due to the lateral stresses imposed during compaction. This is less than that of the asphaltic concrete core for vertical stresses in excess of about 500 kN/m². Some lateral yielding of the lower core will therefore occur but Fig. 5 shows that this will be tolerably small before equilibrium is reached. The high density of the core will mean that the rockfill will settle more than the core and therefore a hung up condition in the core will not occur.

The primary core is stepped in width from 1.2 m. at the bottom to 0.8 m. at the top while the secondary core, designed to intercept any seepage through the primary core, is 0.6 m. wide. At a K value of 10^{-9} cm/sec., the core is to all intents, impermeable.

The core is placed in 20 cm. layers which are compacted at the same time as the narrow supporting zone of transition rockfill using specialised equipment developed and patented by Strabag; the required core density being between 1 and 3% air voids (the solid density is about 2.44 t/cu.m.).

Core placing at the West Dam commenced in October 1974 and is still in progress.

5 HIGH ISLAND SAND BITUMEN

Water tightness was achieved at 3 of the 4 cofferdams at High Island by using hydraulic fill between rockfill bunds. However, space limitations at the fourth site, the East Sea Cofferdam, precluded this design and an alternate arrangement was sought. The cofferdam is located in 15 m. of water exposed to typhoon waves of 10 m. design significant height and rapid construction was necessary. Steel sheet piling, caissons and a concrete diaphragm were considered and rejected because of the time required and the large boulders in the seabed.

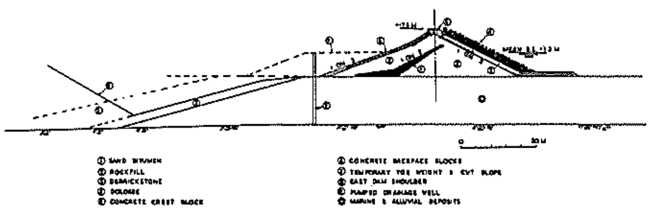

Fig. 6 High Island East Sea Cofferdam

Sand bitumen was chosen for the core and blanket zone, despite its high cost, because it enabled a relatively narrow cofferdam (Fig. 6) to be built quickly and by straightforward methods. Sand bitumen has been little used outside Europe but Bitumarin, one partner in the consortium of sub-contractors carrying out the various bitumen works at High Island, had considerable experience with the material.

A marine sand from within the site area and whose average grading is shown on Fig. 2 was used for the work. Laboratory tests performed on sand bitumen mixes within the range from 2 to 6% bitumen established that 4% bitumen was the optimum content. At this bitumen content, the material was friable, did not ball up and was not sticky but yet did not separate into individual sand grains. A low permeability was not sought but rather an ability to loosely bind the sand grains together and allow water to pass through without erosion occurring. With this property, it was possible to place sand bitumen directly against coarse rockfill without the need for intervening filters.

Laboratory tests carried out in various laboratory apparatus including a 100 mm. dia. specimen triaxial cell, a 100 mm. dia. permeameter and a 600 mm. dia. oedometer showed that sand bitumen at the field density of 1.44 t/cu.m. had a permeability of between 0.5 and 2×10^{-3} cm/sec. which was two and one half to ten times lower than that of the natural sand at the same density which was 5×10^{-3} cm/sec. (This natural sand permeability is intermediate between 10×10^{-3} cm/sec. at the loose density of 1.39 t/cu.m. and 2×10^{-3} cm/sec. at the maximum density of 1.65 t/cu.m.) Samples behaving as if fully saturated (namely $\bar{B} = 1$) were found to have a saturation by moisture content determination of only 60% which suggested that many bitumen enclosed, unconnected air voids were present and influencing the low permeability; the variability of these connections causing difficulty in reproducing results. Tests carried out in the 600 mm. oedometer to simulate piping of the sand bitumen into the rockfill bund showed that piping (defined as a tenfold increase in permeability) occurred at a hydraulic gradient of 13 when 1½" aggregate was placed directly against the sand bitumen on the exit face and 7 when 6" aggregate was so placed. Consolidation tests performed in the oedometer showed that sand bitumen placed at density of 1.44 t/cu.m. would settle only some 3% under the maximum imposed rockfill load of 250 kN/m^2.

The sand bitumen was mixed in a conventional road asphalt plant at a temperature between 160 and 200°C and then stockpiled for several weeks to allow the material to cool to the dumping temperature of about 90°C.

The sand bitumen was placed in the core by trucks running on the initial rockfill wedge and end dumping down the slope. The sand bitumen in the blanket was dumped from bottom opening barges in the form of 250 t windrows which were wrapped in light wire mesh to minimise separation in deep water. No compaction was performed with either method; the average density of the truck dumped core being 1.35 t/cu.m. and that of the barge dumped blanket being 1.44 t/cu.m.

6 SUMMARY

The four cases described in this paper demonstrate that necessity is the mother of invention. It is likely that solutions of this kind may be even more applicable than natural earthfill cores in some situations.

7 ACKNOWLEDGEMENTS

Binnie & Partners of London were the Consulting Engineers for the Mangla and High Island Dams, the Owners being respectively the Pakistan Water and Power Development Authority and the Hong Kong Government. Tippets, Abbett, McCarthy and Stratton of New York City were the Consulting Engineers on Tarbela Dam, the Owner being the Pakistan Water and Power Development Authority.

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