SOME SOILS ASPECTS OF THE PLOVER COVE MARINE DAM
QUELQUES ASPECTS DES MATERIAUX DU BARRAGE EN MER DE PLOVER COVE

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SYNOPSIS Reference is made to the site investigations, reasons governing the selection of constructional materials, laboratory testing and properties of in-situ and filling materials for the 6800ft long Plover Cove marine dam. The construction of an 80ft high instrumented test mound is discussed. Factors which influenced the design including the method of closure are described in some detail. Soils aspects of problems met during construction are discussed, in particular the characteristics of dredging debris, effects of segregation on the permeability of the core and the nature of segregated lenses of decomposed rock fines. Mention is also made of instrument behaviour and causes for moisture losses in fill compacted at well above optimum water content. The principal lessons learnt at Plover Cove are summarized in the conclusions.

INTRODUCTION

The prominent feature of Hong Kong's Plover Cove water scheme (see fig 1) is the £10m main marine dam (started in 1964) which together with two subsidiary dams has sealed off from the sea a large coastal inlet, the bed of which lies 30-40ft below mean sea level(M.S.L.). The 6800ft long main dam is 100-130ft high and the dredged foundation was formed 70-95ft below M.S.L.-see fig 2&3; this made it necessary to deposit under water some 10m yd^3 of constructional materials which represented over 90% of the total volume incorporated in the dam. The trapped sea water was pumped out and impounding of fresh water in the newly formed reservoir (37000mg useful capacity) began in 1967 before completion of the upper sections of the dam. A full description of the scheme can be found in the paper by Ford & Elliott(1965) whilst some constructional aspects of the dam have been described by Dodd(1966) and Elliott et al (1967).

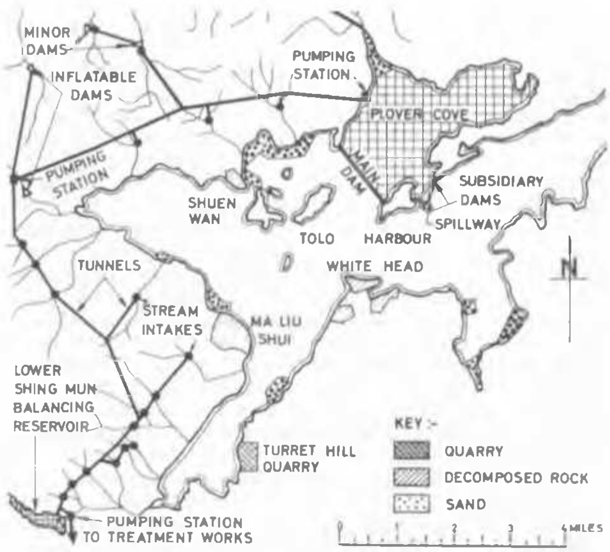

Fig 1 Layout of Plover Cove Water Scheme & Sources of Filling Materials

SITE INVESTIGATIONS

The sub-surface exploration of the foundation and abutments of the Plover Cove main dam site was made with 96 holes involving some

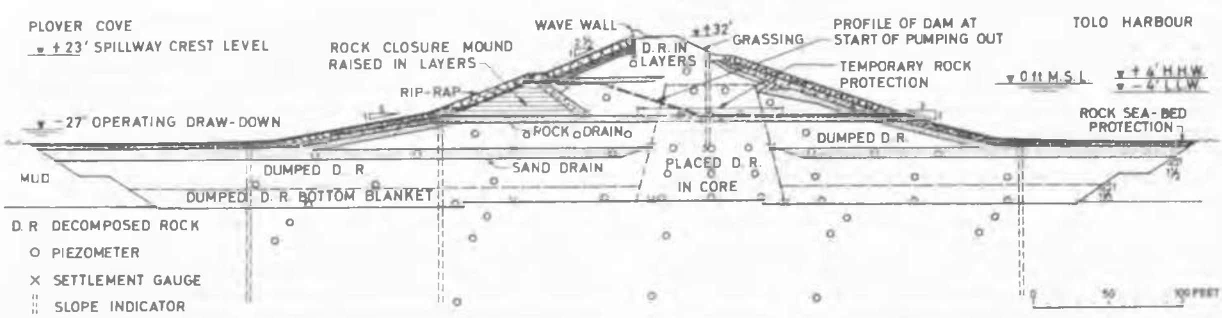

Fig 2 Plover Cove Marine Dam - Typical Cross-Section (Inside Closure Gap)

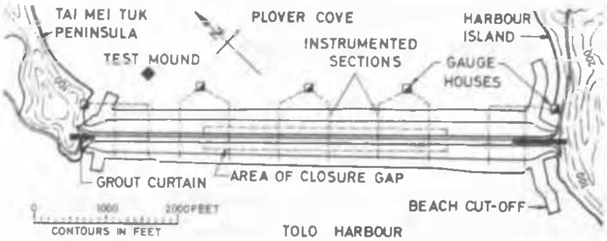

Fig 3 Plover Cove Marine Dam - Plan

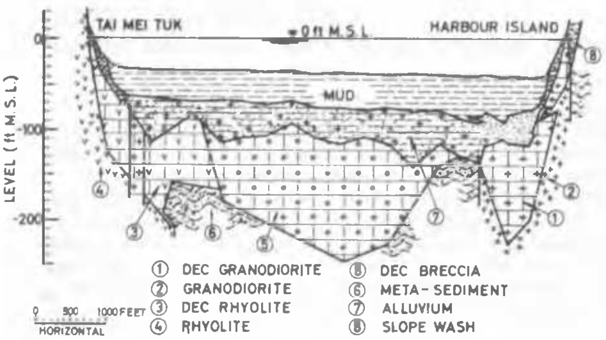

Fig 4 Soil Strata on Centre-Line

6050ft of percussion boring from barge-mounted rigs and 750ft of rock core drilling. These revealed (fig 4) that the bed of Plover Cove consists of recent deposits, viz a 30-50ft thick upper stratum of very soft clay which overlies a 30-50ft thick lower layer of heterogeneous material; these deposits, in turn, overlie the dominant rock formations which surround Plover Cove The borings were augmented by a comprehensive programme of field testing, the highlight of which was the construction of a large-scale test mound (described below). Before construction started, some 1900 hand probings with a Dutch cone penetrometer were made to establish the extent of the soft mud on the dam site; the results of these probings were interpreted with the aid of bore-hole information.

In view of the compressible nature of the foundation, it soon became clear that a flexible type dam would have to be built using naturally occurring materials. The choice of constructional materials was largely influenced by those which were available within reasonable distance of the dam site - see fig 1.

Owing to the absence of any major rivers, deposits of sand around the coastline of Hong Kong are normally shallow in depth, limited in extent and small in overall volume. The 30 bore-holes in the sand areas indicated that the deposits in Tolo Harbour were no exception and, thus, only minor use of sand fill in the dam could be considered.

The granitic and volcanic soils are formed by tropical (i.e. chemical) weathering of the parent rock types and may extend to depths of about 125ft; these covered the greater part of the area near the dam site and were present in large quantities. However, as products of weathered volcanics are considerably more variable than those of granite and have less favourable engineering properties, borrow areas for the principal soft filling requirements were selected in granite areas where extensive decomposition of the parent rock had taken place. Some 46 trial pits and drill holes were sunk in the principal land borrow areas of Ma Liu Shui (granite), White Head (granite) and Shuen Wan (granodiorite) from which it was concluded that there was ample material to complete the main dam. However, in the detailed site investigation involving 350 trial pits at 150ft centres completed just before excavation started, a 30% (or 2.7m yd^3) shortfall became apparent and the White Head borrow area was extended to include two adjacent areas.

Quarry sites in Hong Kong, from which rock protection and associated filters could be obtained, are comparatively few as only around the coastline and in the bottom of deep narrow valleys is unweathered rock normally found. As neither volcanic nor meta-sedimentary rocks were considered suitable for use in the dam on account of their close jointing, the search for a quarry site was limited to granite areas. Turret Hill, in which 4 drill holes were made, was found to be the most promising source of rock as the depth of overburden was not excessive and joint spacing wide.

TEST MOUND

The 80ft high fully instrumented test mound, shown in fig 5, was constructed at a cost of £90000 on a site 1000ft east of the right abutment using decomposed granite bottom dumped from barges as the principal embanking material. Successful completion of the mound proved that it was possible to form under water an adequately stable and impermeable embankment without any unduly complicated techniques of selection or placing.

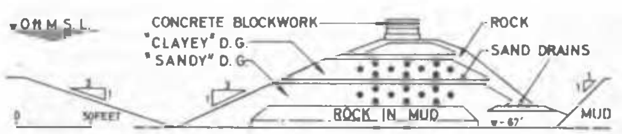

Fig 5 Test Mound - Typical Cross-Section

During construction, 30 combined twin-tube hydraulic piezometer/settlement cells were placed in the mound. Only very small excess pore pressures were observed in the lower "sandy" layer of decomposed granite ($w_p<20$; 32% fines), but immediately after installation of cells in the intermediate "clayey" layer ($w_p>20$; 39% fines), substantial pore pressures in that layer were recorded and these increased throughout further stages of construction. The evaluation of excess pore pressures was complicated by variations in total head caused by tidal fluctuations. In some cases the excess head relative to sea level varied by as much as half the tidal range indicating a "B" value for tidal loading of only 0.5. The "B" value for the continuously applied filling was approximately unity for all piezometers in the "clayey" zone. To eliminate tidal effects continuous ½-hour readings were taken - see fig 6.

The co-efficient of consolidation of material in the "clayey" zone was calculated

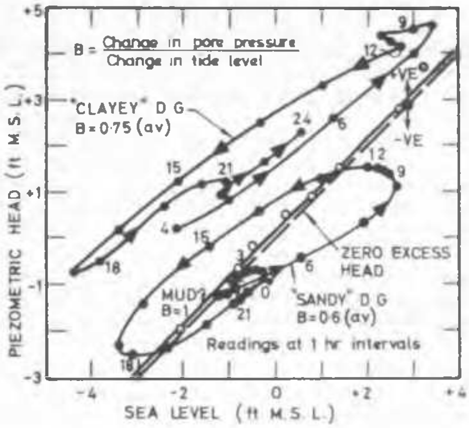

Fig 6 Test Mound - Tidal Effects on Excess Piezometric Head

from pore-pressure dissipation curves and a mean value of $80\times10^{-4}cm^2/s$ was obtained. Permeability tests were also conducted through the piezometer tips and leads; these indicated the permeability of the "clayey" layer to be 0.08×10^{-4}cm/s.

Since it was not possible to measure settlements directly owing to the continuous dumping of material, an indirect method was evolved incorporating an air/water manometer system in which the air/water interface was adjusted to occur within the cell and its position defined by electrodes. Unfortunately many of the cells were displaced during dumping and, as they were no longer approximately vertical, failed to function effectively.

LABORATORY TESTING

General The lab testing was principally undertaken during the original investigations and covered three main categories of soils namely, sea-bed mud, existing foundation materials and filling materials. The zones in which these soils lie on the plasticity chart are shown in fig 7.

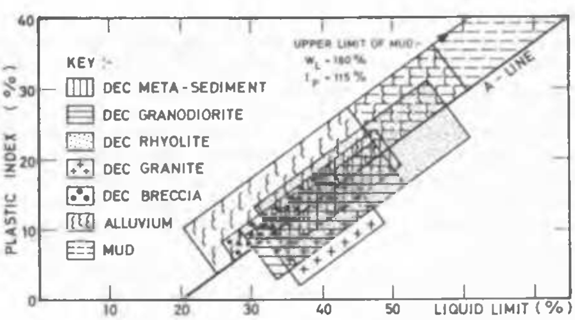

Fig 7 Plasticity Chart for In-situ & Constructional Materials

Permeability results derived from lab falling head tests were, in general, about 10 times greater than those calculated from consolidation tests; no explanation can be offered for this discrepancy. Field permeabilities agreed reasonably well with lab results on samples taken from the test mound; elsewhere, however, field tests were unreliable owing to leakage through casing joints.

In-situ materials The properties of the sea-bed mud, a very soft, sensitive, normally-consolidated marine clay, have been discussed recently by Lumb and Holt(1968). The material, with a liquidity index approaching unity, has a sensitivity of around 5 and is characterized by its high Atterberg limits and compressibility and low strength - see table I. The unconfined shear strengths from normal samples were unsatisfactory owing to sample disturbance but those from 2½in-dia samples obtained with a Swedish foil sampler and in-situ vane tests gave consistent results; these showed that the soft clay gained strength linearly with depth as follows:-

c_u = 35 + 6.75h (h = depth in ft from surface lb/ft^2)

This strength was reasonably consistent with that obtained from Skempton's well-known empirical formula; $c_u = (0.11 + 0.0037 \times I_p)\gamma' h$. A comparison of measured and calculated shear strengths together with index properties for a typical bore-hole is given in fig 8.

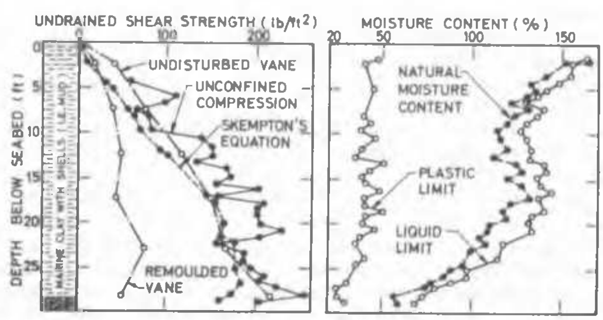

Fig 8 Typical Properties of Sea-Bed Mud

The alluvial deposit is mainly clayey in composition with admixtures of sand and gravel and underlies the soft marine clay; however predominantly sandy zones lie near the abutments. The erratic strength of the material, which has been affected by dessication during its various stages of deposition, is indicative of its estuarine origin. However, mean effective strength parameters increased with depth from about c'=240lb/ft^2, $\phi'=24^o$ just beneath the mud to about c'=410lb/ft^2, ϕ'= 31^o at the surface of the decomposed rock. Likewise in-situ vane shear strengths increased from about 1100 to 1600lb/ft^2 over the same depth (normally about 40ft); these values were about 25% higher than those from undrained triaxial tests. In view of the fairly low measured "c_v" values, settlement of the foundation was not expected to be rapid. Summarized properties of the alluvial deposit are shown in table I.

Decomposed rocks underlying the alluvial deposit consist of in-situ weathered rhyolite, meta-sediments, granodiorite and breccia. It will be noted from table I that the engineering properties of the decomposed rocks are generally good, i.e. high strength, low compressibility and rapid consolidation.

Table I Summarized Soil Test Results (mean values) for In-situ & Constructional Materials

Soil Type	w %	γ lb/ft³	G	w_L %	I_P %	Si-clay %	k x 10^{-7} cm/s (F.H.)	Consolidation Test C_c	e_o	c_vx10^{-4} cm²/s	k x 10^{-7} cm/s	Triaxial Shear Test c lb/ft²	ϕ	c' lb/ft²	ϕ'	K_o
IN-SITU MATERIALS																
MUD	150	35	2.72	160	115	84	3	1.5-	2.7	4-2.5	1.2-0.1	0	12½	0	21	-
	-80	-55		-90	-57			0.9								
ALLUVIAL DEPOSIT																
Clayey	23.2	102	2.70	36	18	-	4.9	0.11	0.63	32-20	0.5-0.3	500	20½	300	27	
Sandy	18.1	109	2.68	-	-	13	170	-	-	-	-	810	27	620	31	-
DECOMPOSED ROCK																
Granodiorite	30.8	89	2.74	42	12	-	89	0.15	0.93	330-310	5.6-3.2	720	22	630	28	-
Rhyolite	21.8	104	2.72	39	17	-	19	0.09	0.76	330-260	3.7-2.3	1070	23	1020	27½	-
Meta-Sediment	19.3	108	2.80	41	18	-	11	0.08	0.64	460-320	2.3-1.9	790	22½	890	24½	-
Breccia	21.6	110	2.74	33	12	-	65	0.10	0.77	150-130	2.1-1.2	1220	24	820	30	-
CONSTRUCTIONAL MATERIALS																
DEC GRANITE - "CLAYEY"																
Lab prepared - total	30.7	92	2.67	92	52	51	60	0.15	0.82	200	7-0.6	-	-	0	32	0.52
Lab prepared - no 36	-	-	-	92	52	91	-	-	-	-	-	60	16½	100	32½	-
Lab prepared - no 200	-	-	-	96	58	100	-	-	-	-	-	70	14	110	30	-
Test mound	20.6	105	-	45	19	35	-	0.10	0.59	37-21	0.8-0.5	-	-	0	33	-
DEC GRANITE - "SANDY"																
Lab prepared - total	32.3	89	2.66	46	10	32	570	0.14	0.86	10000	210-60	1150	18½	240	36½	0.39
Lab prepared - no 36	-	-	-	46	10	64	-	-	-	-	-	-	-	330	38½	-
Lab prepared - no 200	-	-	-	56	12	100	-	-	-	-	-	0	26	0	38½	-
Test mound	20.3	105	-	43	14	25	-	0.07	0.61	480-120	10-3	-	-	0	38½	-
Lab compaction	18.0	108	-	44	13	28	-	-	-	-	-	-	-	-	-	-
DEC GRANITE - DAM																
Dumped underwater	16.3	111	-	31	9	30	-	-	0.50	310-180	-	-	-	500	36½	-
Placed underwater	18.5	106	-	31	9	44	-	-	0.48	44-6	-	-	-	430	32	-
Trapped fines	42.4	78	-	47	20	92	-	-	-	3.5-1.8	0.3-0.1	-	-	0	30½	-
Compacted	12.3	117	-	29	10	30	-	-	-	66-11	-	-	-	430	37	-
DEC GRANODIORITE																
Lab prepared - clayey	43.3	77	2.68	45	16	56	-	0.21	1.16	61-29	7-4	70	16	110	30½	-
Lab prepared - sandy	26.1	98	2.68	30	4	32	-	0.07	0.70	970-520	70-35	390	31	0	38½	-
Lab compaction	20.2	103	-	42	11	40	-	-	-	-	-	-	-	-	-	-
Compacted in dam	18.6	107	-	35	14	40	-	-	0.47	22-9	-	-	-	360	31½	-

However, the granodiorite is less dense, weaker under undrained loading conditions and considerably more permeable and compressible. As has often been noted previously, the Atterberg limits of decomposed rocks lie generally below the A-line (fig 7).

Constructional materials Before the start of construction of the main marine dam, lab testing concentrated on establishing the properties of the principal embanking materials

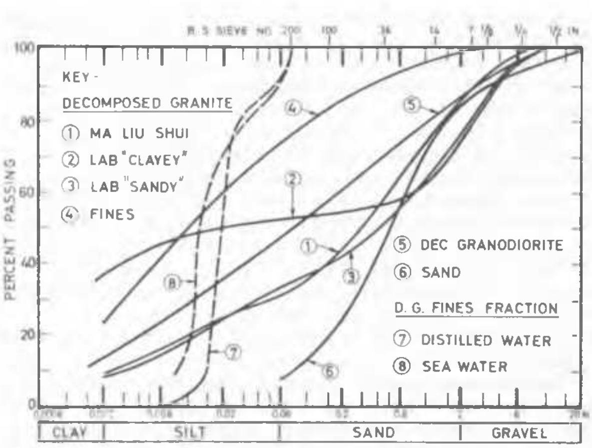

Fig 9 Typical Gradings of Constructional Materials

namely, decomposed granite and granodiorite (for use above water). Grain-size analyses (see fig 9 for typical gradings) were carried out on all samples from the trial pits and an extensive programme of testing was undertaken on material taken from the test mound, from an existing earth/rock fill dam, on 1½ and 4in-dia "sandy" and "clayey" samples (see fig 9) prepared in the loosest obtainable state (to simulate their probable initial state when deposited under water) and on similar samples composed of different fractions of the material (to show the effect of segregation on their engineering properties). A description of this testing has been given by Lamb(1962).

In terms of effective stress, the strength of decomposed granite, whether obtained from the test mound or lab-prepared samples (including tests on samples with coarse fractions omitted), was high and varied from c'=110lb/ft², ϕ'=30° to c'=330lb/ft², ϕ'=38½°. Strengths of samples from the completed dam lay within the same range. Corresponding tests on decomposed granodiorite gave marginally lower values.

Measurements in consolidation stages of triaxial tests showed that both the "sandy" and "clayey" decomposed rock filling were relatively incompressible (generally C_c=0.07 to 0.15); the oedometer method was not considered satisfactory owing to the coarseness of the soil. Tests showed that the rate of consolidation of both decomposed granite and granodiorite would be rapid for "sandy" but fairly slow for "clayey" material; the lab "c_v" of 20 to $200\times10^{-4}cm^2/s$ for the latter compared reasonably well with that from field dissipation tests in the test mound.

The permeability of both types of decomposed rock was of the same order but very variable; "sandy" material was at least 10 times as permeable ($k=0.6\times10^{-4}cm/s$ by falling head method) as the "clayey" material. The in-situ permeability of "sandy" decomposed granite in the test mound, however, was considerably higher indicating extensive pervious zones in the dumped material. A complete summary of the mean properties of the constructional materials is given in table I.

Comparative Atterberg limit tests on samples of decomposed granite and granodiorite used both distilled and sea water. Table II shows that sea water reduced the Atterberg limits contrary to the usual theory. Typical results of comparative hydrometer analyses on the fines fraction of decomposed rock with different suspension liquids are shown in fig 9. The flocs formed with distilled water (without dispersing agent) were clearly larger than when using sea water; again this is contrary to what might be expected. However in neither case are the differences very great; triaxial tests on samples taken from the test mound and later from the dam itself showed no evidence that the engineering properties of filling are worsened by deposition in sea water.

Table II Effect of Sea Water on Limits

Fraction Passing B.S. No. (Type)	Dec Granite w_L D	w_L S	I_P D	I_P S	Dec Granodiorite w_L D	w_L S	I_P D	I_P S
36 (sandy)	46	41	10	4	30	28	4	3
200 (sandy)	56	49	12	11	-	-	-	-
36 (clayey)	92	79	52	50	45	40	16	14
200 (clayey)	96	81	58	49	51	48	17	15

D = Distilled Water : S = Salt Water

DESIGN ASPECTS

Basis of design The reasons for selecting an earth/rockfill dam to seal off Plover Cove have already been discussed.

The very low strength of the sea-bed mud meant that it had to be removed completely from the whole length of the dam before embanking operations could begin; the test mound had shown that a satisfactory foundation could not be formed by dumping large-sized rock into the mud. The 1:1½ side slope found stable during dredging for the test mound was adopted where depths of mud did not exceed 35ft. A 20-25ft horizontal berm was added at the mid-point of deeper cuts reducing the effective dredged slope to 1:2. Theoretical analyses confirmed the stability of these slopes.

The three zones of decomposed rock filling in the dam are shown in fig 2. These are a carefully placed core (slightly off-centre to improve stability on draw-down), a blanket bottom dumped over the foundation to prevent seepage under the dam and general filling, also dumped, to support the core. A minimum fines content of 25% was required for both blanket and core. Horizontal sand drains at four levels were incorporated in the general filling to accelerate pore-pressure dissipation and, to facilitate location of possible leakage zones, each drain was isolated at 700ft intervals by decomposed rock barriers. Although a permeability for the core of less than $0.1\times10^{-4}cm/s$ was hoped for, a value of $1\times10^{-4}cm/s$ was considered acceptable.

The risk of cracking in the compacted filling above sea level due to settlement of the underwater embankment and foundation was recognized. Thus a comparatively low density (90% B.S. standard compaction) and high water content (1.15 x optimum) were specified for the compacted filling to ensure that it remained flexible for as long as possible.

Maximum settlements of 3ft in the foundation and 6ft in the dam were predicted of which half and two-thirds, respectively, were expected during construction.

In the light of test mound experience, the steepest fill slopes to be formed under water were 1:3. The external slopes consisted of sand bedding on which fairly thick layers of wide-graded filter rock were to rest. On top of the filter lay the main wave protection, a graded rip-rap of thickness $1.5D_{50}$ (D_{50} = equivalent spherical diameter of the 50% size, by weight, of the grading). A major factor in the choice of rip-rap was its self-healing property and the design itself was based on the recommendations of Burgess & Hicks(1966). A detailed analysis of typhoon wind-generated waves showed that significant damage (greater than 5% of rock displaced) to the rip-rap should not occur more often than once in 75 years.

Fortunately Hong Kong does not lie on the main circum-Pacific earthquake belt some 450 miles to the east. The intensity of the strongest earthquake recorded is estimated at VI on the Modified Mercalli scale and thus ground accelerations of 0.05g were considered in analyses of stability.

Stability during various constructional and operational phases of the dam was analysed principally by the circular slip method with the aid of an English Electric "DEUCE" electronic computer using effective stress parameters. Although it was anticipated that much of the "dumped" (as opposed to "placed") decomposed rock would be "sandy" in nature, a conservative "c_v" value was used in calculating residual pore pressures in the dam.

Table III Design Parameters

Soil Type	γ_d lb/ft³	γ lb/ft³	c' lb/ft²	ϕ'	k $\times10^{-7}$ cm/s	c_v $\times10^{-4}$ cm²/s
EMBANKMENT MATERIALS						
Rockfill	116	134	0	40	-	-
Sand	102	126	0	35	10	-
Dec Granodiorite*	102	124	0	30	0.1	35
Dec Granite-placed	102	126	0	30	1.0	35
Dec Granite-dumped	102	126	0	35	1.0	35
IN-SITU MATERIALS						
Mud	39	89	50	15	0.003	3
Alluvium**top 20ft	101	127	300	25	0.005	20
Alluvium**remainder	101	127	300	30	0.005	50
Alluvium - sandy	107	127	0	32½	10	-
Dec Rock (in-situ)	99	124	900	27½	0.03	300

*Above M.S.L. **Clayey

With the parameters shown in table III, the predicted factors of safety were 1.25 during construction and 1.5 on completion. Wedge analyses were also used to check stability particularly during construction. The design allowed for future limited raising of the dam.

At each abutment, a single line of curtain grout holes at 10ft centres was specified to reduce the effective permeability of the in-situ rock to 0.3×10^{-4}cm/s. Some blanket grout holes at 20ft centres were also necessary to bring the permeability of the contact zone rock under the in-situ decomposed rock or filling near the grout cut-off below 1×10^{-4}cm/s. Cut-off trenches, backfilled with selected material, were planned to isolate the surface beach deposits from sandy foundation material near the abutments.

Closure Final sealing off of Plover Cove from the sea posed an interesting problem. Two means of final closure were studied; the first envisaged using large floating caissons (with built-in sluices) as in recent closures on the Delta Plan in Holland and the second involved uniform raising of a horizontal rock mound. Comparatively cheap and readily accessible rock favoured the latter method.

At the start of final closure, maximum velocities across the 3000ft wide closure gap (sill level -20ft M.S.L.) were expected to be about 1¼ft/s. To check the erodability of the soft filling, flume tests were carried out on representative samples; these tests indicated that no significant scour (<0.1in/h) of either decomposed granite (43% fines) or sand (D_{10}=0.09mm) would occur until water velocities reached about 1.7ft/s.

Model tests at the U.K. Hydraulics Research Station indicated a maximum current of 9.1ft/s at the centre of the closure mound under certain tidal conditions. However, as velocities near the downstream edge of the crest of the mound could exceed this value by 20%, the design velocities were increased accordingly. The stability of rockfill in relation to current velocities was assessed from a U.S. Bureau of Reclamation design chart using as the effective size the minimum specified D_{50} size(17in) of the rock grading.

Velocities up to 4ft/s were predicted across the sill on which the closure mound rested and so it was decided to replace the intermediate sand drain with graded rock to provide the necessary protection against scour. Near the upstream toe of the dam, velocities up to 3.5ft/s could exist just above the sea-bed mud; further flume tests carried out to check the erodability characteristics of the mud indicated that no significant erosion would occur with currents less than about 6ft/s.

Instrumentation In view of the unusual nature of the project, a comprehensive instrumentation system was planned to give warning of impending failure of the embankment during construction, on draw-down following completion and in any subsequent raising of the dam, to indicate suitable corrective measures should instability appear to be imminent, to check design settlement allowances (in particular, the allowance to be built into the crest of the dam) and to reveal seepage patterns through the embankment and foundation during pumping-out of the reservoir and later operation.

Instrumentation was arranged on 10 sections (see fig 2 & 3) and leads, up to 1600ft in length, led back to 5 major gauge houses, one on each abutment and the other three on steel towers in the reservoir. Some 400 piezometers, mainly of the twin-tube hydraulic type (air entry values 0.2 to 30lb/in^2), 66 settlement gauges (which could function satisfactorily even when tilted) based on a water/air balance system originally developed by the U.K. Building Research Station and 19 Wilson slope indicator (series 200B) installations using plastic casing (to detect both horizontal movements and vertical settlements) were located in the marine embankment. Polythene-coated 0.11in I.D. nylon 11 leads were specified for the piezometer system and larger diameter black nylon for the settlement gauges. A much fuller description of the instrumentation design can be found in the paper by Dunnicliff (1968).

Design changes To speed construction, the contractor suggested combining the bottom and sea-bed drains. As only very small pore pressures had been recorded in the filling, permission was granted.

The contractor was apprehensive about bringing soft filling through the tidal range and elected to construct rock mounds on both sides of the dam within the final profiles, to prevent beaching under wave action. To minimize the quantity of additional rock fill, the reservoir face was steepened up above -17ft M.S.L. from 1:3 to 1:2½; this resulted in heavier and thicker rip-rap wave protection being used on this slope.

A detailed analysis of surge records from local typhoons, using a correlation between surge height and instantaneous barometric pressure, showed that the maximum level to which the sea at Plover Cove could rise, with a return period in excess of 1000 years, was +19ft M.S.L. This resulted in omission of the rock fill above this level on the seaward face of the dam.

EXPERIENCE DURING CONSTRUCTION

Dredging debris To minimize the accumulation of debris during dredging of the main dam trench with bucket and grab dredgers, each lateral cut was limited to 6ft in depth and the final cut was kept to a constant thickness of 3ft. Nevertheless a residue up to 6ft thick of semi-fluid mud, which would endanger the stability of the dam, formed in the bottom of the trench.

A special sampler capable of taking representative samples of this debris at intervals throughout its thickness was devised. Samples indicated that the dredging debris consisted of two layers. The top layer was a dark grey fluid formed by flocculent soil particles (0.006mm apparent size) from in-situ mud dredged at higher levels; the material generally had no shear strength, a liquid limit around 60-100 and liquidity index of 6-9. The bottom layer was often about 2ft thick and comprised a mixture of similar material and small fragments of mud in their original state. This layer was firmer than the top layer and crude experiments in the lab indicated a shear strength of the order of 1-3lb $/ft^2$; its liquid limit was around 50-60 and liquidity index from 3 to 6. Initial efforts to redredge the debris with bucket and grab dredgers and by air lift pumps failed. It was then found that, if the mud was left for 4-6 weeks, it gained sufficient strength by thixotropy and by normal consolidation to be redredged successfully.

Quality control of soft filling The fines content of decomposed rock and sand governed its suitability as under-water filling in specific zones of the dam. Thus it became clear at an early stage that a simple, accurate and rapid method of measuring the fines content of each barge load of material had to be evolved; the normal lab method was obviously unsuitable. The test adopted followed the wet sieving method for fines determination except that the drying and weighing stages were replaced by weighings in a pycnometer. Samples could thus be tested in 15-25 min; comparisons with normal lab tests gave similar results.

Excavated material was generally deposited in large stockpiles near the marine loading installations to ensure that interruptions on the dam site on account of selection requirements were kept to a minimum.

As a check on the methods of construction, many borings, field permeability tests and lab tests were undertaken.

Stability of materials deposited under water The decomposed rock core was normally raised in 15-20ft lifts in a series of 3ft thick layers placed with grabs (up to $22yd^3$ capacity). With careful control, side slopes of 1:4-5 could be achieved; these were flatter than the slopes obtained on the test mound when smaller equipment was used. Thicker layers resulted in slopes as flat as 1:10.

Considerable trimming was needed both to remove sand which had spread onto the core, to prepare the surfaces of dumped decomposed rock to receive sand drains and slope bedding and to form the upper surfaces of these sand zones within specified tolerances.

Very little damage occurred to the rock mound in the last phase of closure (when a maximum field velocity of 9.7ft/s was measured); the water currents packed small-sized material into a tight and stable matrix leaving small projections of the larger pieces of rock. This experience showed that smaller sized rock could have been used, even in the upper zones of the mound. No scour was observed in the filter rock blanket overlying the core and toes of the dam or in the sea-bed mud.

Effect of segregation on permeability of core In spite of careful deposition, there was some segregation of "placed" material. Some fine material was lost into suspension as each bucket-load was lowered through the water and, when deposited on anything other than a horizontal surface, it tended to spread or flow laterally. This resulted in lenses of segregated material deficient in fines. Better placing techniques were evolved to limit lateral spreading and hence segregation. Segregation clearly had a marked effect on the permeability of the core.

The permeability of the core was measured (or estimated) by three different techniques namely, by rapid determinations with a special hand-operated 3¼in-dia penetration probe, from field tests on 20-30ft sections in bore-holes and from determinations of void ratio and fines content of samples taken from the dam. In the last method, permeabilities were estimated from two families of curves showing relationships between void ratio, fines content and permeability for lab prepared samples of decomposed rock; these curves are shown in fig 10. A description of the techniques used can be found in the paper by Holt (1967).

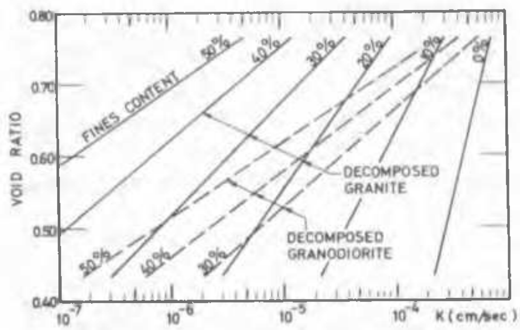

Fig 10 Permeability/Void Ratio/Fines Curves

In spite of progressive improvements to the permeability probe, results with this instrument were erratic owing to inadequate sealing around the collar, smear effects on the porous tip and its very local sphere of influence; this method was abandoned as soon as the filling was far enough advanced to allow large-scale bore-hole tests to be carried out.

Permeabilities measured in the large-scale bore-hole tests (see fig 11) indicated that the average "k" of the material placed under water (mainly dec granite) was 1.0×10^{-4}cm/s. This value is higher than that estimated (0.2×10^{-4}cm/s) for samples taken in the same

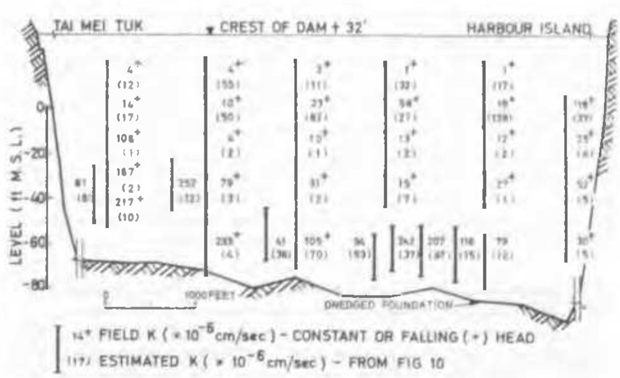

Fig 11 Bore-Hole Permeability Tests

bore-holes and from probe tests (0.08×10^{-4}cm/s) but corresponds to the design value. The difference is likely to be due to anisotropy in the decomposed rock placed under water. Bore-hole tests in the more homogeneous compacted fill above sea level showed much lower permeabilities, namely 2.3×10^{-7}cm/s(av). As would be expected this value was somewhat lower than that estimated from fig 10 (4.3×10^{-7}cm/s), the curves in this figure representing the upper limit of permeability. Fluctuations in sea level complicated interpretation of the results of the field tests. Apparent permeabilities were as much as 2-3 times greater at high tide than at low tide depending on the datum used (see fig 12); the reason for this is not fully understood but it may have been due to air trapped in the pores of the fill.

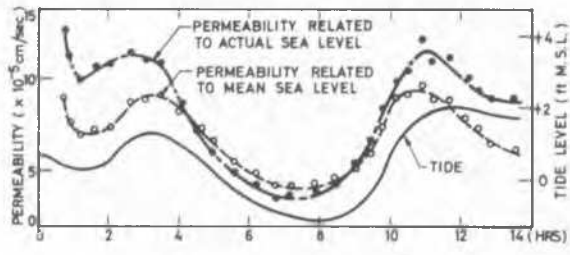

Fig 12 Tidal Effects on Permeability

Decomposed rock fines After the tempo of work increased, it became evident that much of the fines from material bottom dumped from barges was going into suspension in flocs and drifting, partly with the tides and partly by gravity, into the deeper areas of unfilled trench. At first these deposits had negligible shear strength and a liquidity index of 4-6. They generally accumulated to depths of a few feet but reached 30ft in the deepest section of the trench. The self weight of the flocs, small as it was, eventually caused collapse of the flocculent structure at the bottom of the layer and produced a material recognisable as a soil having a finite shear strength.

Although the fines had a fairly high effective angle of internal friction ($30\frac{1}{2}^{o}$), their consolidation characteristics were poor (c_v=2.6×10^{-4}cm^2/s). A layer up to $2\frac{1}{2}$ft thick only could be tolerated if high construction pore pressures were to be avoided. Excess fines were removed by a combination of dredging, displacement, and absorption (by granular material) techniques. A typical grading of the fines is shown in fig 9 and summarized test results in table I.

Tests on decomposed granite before and after deposition, showed that up to 12% (dry weight) of the material dumped could be lost into suspension. This loss decreased as fines contents increased above about 20%.

Instrumentation Instruments in the foundation and embankment indicated generally satisfactory functioning of the dam during construction, pumping out and subsequent refilling of the reservoir. However, in spite of care to protect the instrumentation, leads to about half the piezometers and a third of the settlement gauges were broken by dragging anchors and dredger buckets.

No excess pore pressures were recorded in the under-water shoulders of the dam. Even in the core, excess pore pressures seldom exceeded 10ft(max 24ft) and usually dissipated within a few days. The rapid response of piezometers during pumping-out and refilling of the reservoir should be noted in fig 13. Excess pore pressures were recorded in the fill (mainly dec granodiorite) compacted above water level. Although "c_v" values derived from pore-pressure dissipation curves (see fig 14) averaged 10×10^{-4}cm^2/s or only about one third the design value; there was no instability of the fill.

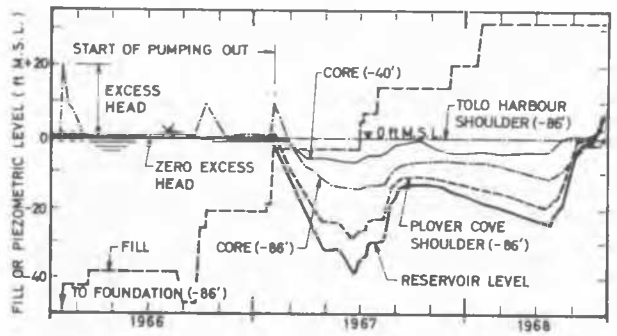

Fig 13 Typical Pore-Pressure Observations

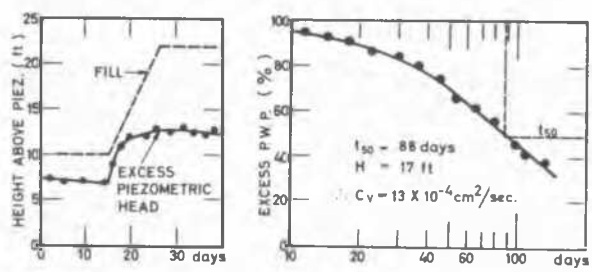

Fig 14 Compacted Fill - Typical Build-up & Excess Pore Pressure Dissipation

During construction, settlements at foundation level generally amounted to 2-3ft, i.e. quicker than predicted. Observed settlements in the dam 30ft above the base have reached nearly 3ft relative to the foundation compared with the predicted 6ft for the full height of the embankment. The plots in fig 15 show that the response of gauges to each load increment was very rapid; this would be

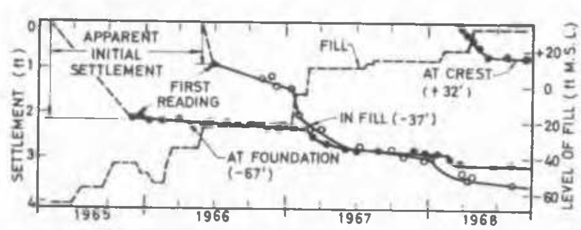

Fig 15 Typical Settlement Curves

expected from the quick pore-pressure dissipation in nearby piezometers.

In general, seepage flow through the foundation and embankment followed the expected patterns.

The slope indicator installations were closely watched during initial draw-down and refilling of the reservoir. Only small horizontal movements were recorded, mostly under 3in and seldom over 6in; these were hardly significant as the sensitivity of measurement did not exceed about 1in(horiz) in 40ft(vert).

Compacted fill In order that the soft filling (normally dec granodiorite) would meet the specification referred to previously, it was compacted with bulldozers in 12in loose layers much wetter than necessary; as a result the dozers' tracks formed ruts some 12in deep. The mean placement water content was 1.36 x optimum (20.8%) and density 93% B.S. standard compaction (103lb/ft^3).

Although the compacted fill was covered with 2ft of loose soil during breaks in construction, it lost much of its moisture by a combination of consolidation, natural drainage and evaporation (and associated shrinkage). Loss of moisture by consolidation would not be large where the surcharge is small. Fill above the phreatic surface and adequately protected against evaporation will drain by gravity until an equilibrium water content, maintained by capillary forces, is reached. This condition was checked with lab soil suction tests, the results being shown in fig 16a; there was close agreement except in the surface zone affected by evaporation. The influence of evaporation was also checked and lab results agreed closely with field experience. Fig 16b shows that the effect of evaporation on water contents is by no means confined to the surface but will, in time, extend to substantial depths.

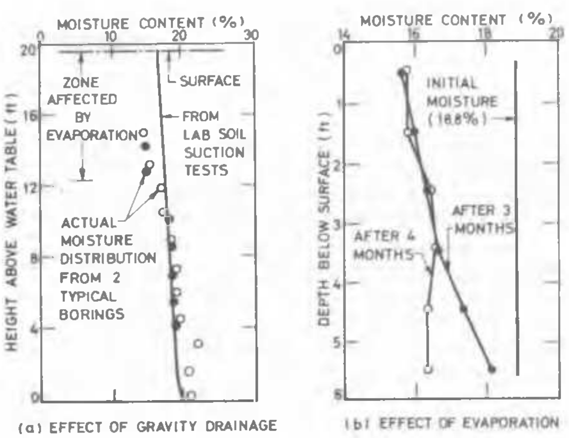

Fig 16 Loss of Moisture in Compacted Fill

CONCLUSIONS

The Plover Cove marine dam has paved the way to a new approach to building earth dams and will clearly be the forerunner of many similar structures throughout the world. Underwater earth dam construction techniques are very different from conventional methods and give rise to unusual problems. The principal lessons learnt at Plover Cove are:-

i Extensive and costly site investigations, including large-scale field tests, are fully justified.

ii Refined methods are needed when dredging thick deposits of soft clay in deep water to minimize accumulations of debris.

iii A simple cross-section is essential to avoid complicated placing techniques.

iv Decomposed granite is a good fill material for under-water earth dam construction. However, attention must be paid to controlling segregation, in particular in the core, and to preventing thick deposits of segregated fines.

v A horizontal rock mound brought up uniformly over a long protected sill is a practicable method of effecting initial closure of a coastal inlet.

vi Rock mounds are desirable to protect soft filling in the tidal range.

vii Comprehensive instrumentation is needed to monitor the behaviour of under-water dams during construction.

ACKNOWLEDGEMENTS

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