GROVEHURST DOCK DAM, FOUNDED ON RECENT ALLUVIUM

BARRAGE DU BASSIN DE GROVEHURST FONDE SUR DES ALLUVIONS RECENTES

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SYNOPSIS The dam was constructed under tidal conditions on a soft clay foundation, without dewatering or removal of the clay. Initial closure to normal high tide level was achieved by end tipping, the cross-section being determined by displacement of the alluvium. Vane tests in the foundation were made before, during and after construction, and the position of the fill/alluvium interface was followed by boring. The final profile was designed section-by-section as construction proceeded. The strength reduction by remoulding was followed as the alluvium was displaced and the subsequent increase by thixotropic regain and later the effects of consolidation. The same sequence was studied in the laboratory. Substantial settlements and cracking took place as was expected, but as the dam was extended to the designed profiles the rate of settlement decreased to a small value. It is estimated that the long-term factor of safety will approach 1.5.

INTRODUCTION

It was required to construct at minimum cost a settling lagoon for paper mill effluent from the Kemsley Mills of Bowater United Kingdom Pulp and Paper Mills Ltd. The derelict Grovehurst Dock was available on the tidal Swale channel adjacent to the mill and it was decided to construct a dam across the dock entrance and to strengthen the existing bund walls around the dock, thus totally enclosing a suitable lagoon. The liquor was to be pumped into the lagoon continuously, and to be pumped out to the Swale channel for only two hours at the peak of each tide, thus minimising polution. Solids in the effluent were to be dredged from the bed of the lagoon whenever their accumulation encroached on the required storage. The essence of an economically viable scheme was the construction of the closure dam across the mouth of Grovehurst Dock without dewatering, under the condition of tidal flow in and out of the dock, and without removal of the recent alluvium existing in the foundation on the line of the proposed dam.

The works were carried out on a design and construct basis, between May 1966 and December 1967.

GROUND CONDITIONS AND SHEAR STRENGTH

The foundation conditions for the closure dam consisted of soft silty clays (Recent Alluvium) to a depth of approximately 20 ft, underlain by London Clay. The surface of the Recent Alluvium on the centre line of the dam varied from +6 ft O.D. to -4 ft O.D. and the tidal range, normal springs, is from +9.5 ft O.D. to -5 ft O.D.

The relationship between shear strength and depth was determined by in-situ vane testing before the start of construction, and this relationship is shown on Fig. 1.

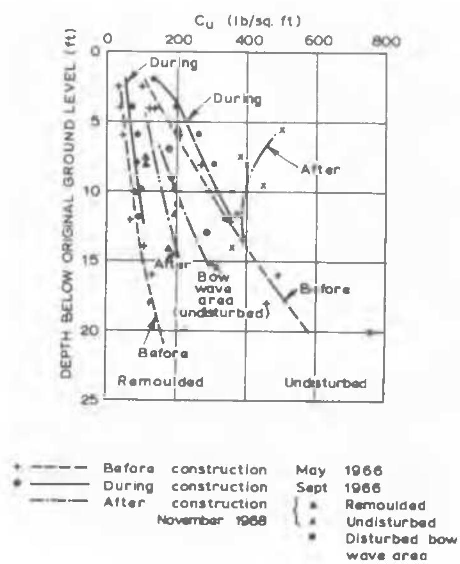

Fig. 1 Relationship between in-situ vane shear strengths and depth before, during and after construction.

It was realised that, for cheap construction, the Recent Alluvium could not be removed mechanically from the foundations and therefore it would be partially displaced by the superimposed fill, suffering remoulding in the process. It was important to know the order of time in which appreciable regain of strength from remoulded towards the original undisturbed values would occur and for this purpose a series of laboratory vane tests were carried out on two separate main samples. An initial undisturbed shear strength reading was followed by remoulding of the samples and then subsequent vane shear readings at logarithmically increasing intervals of time. Separate small divided samples from the two main samples were used for each time interval, and the shear strength results are plotted against log time on Fig. 2. This indicates that the recovery of shear strength in 20 days following construction would be such as to give a total strength at that time of 80% of the undisturbed value, neglecting consolidation effects.

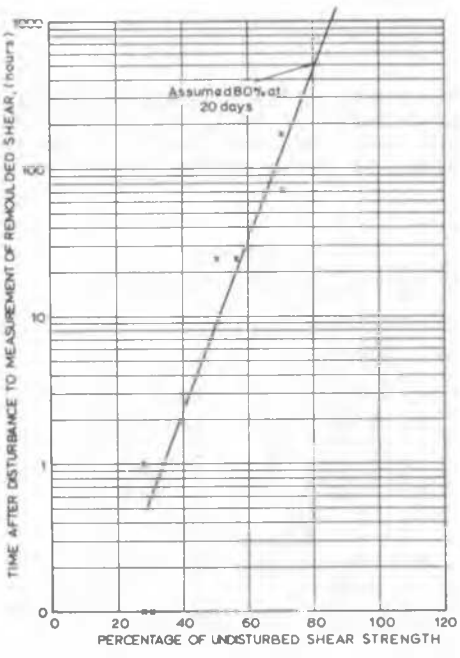

Fig. 2 Laboratory vane tests on Recent Alluvium. Recovery of shear strength after remoulding.

PRINCIPLES OF DESIGN AND CONSTRUCTION

Because of availability of materials the dam design is based essentially on a homogeneous cross-section of London Clay fill. Initially, just sufficient clay fill was to be placed along the line of the dam to effect closure against the tide whilst maintaining a factor of safety of unity, the width of cross section being determined by the displacement of the alluvium. The level of this initial fill was to be just sufficient to prevent overtopping by normal high tides. Subsequent to the closure, the fill could be placed under comparatively controlled conditions, not subject to inundation on the reservoir side, so that better compaction and shear strength could be achieved.

For purposes of sea defence, the final crest level was defined as +18.5 ft O.D. An initial construction level of +20.5 ft O.D. was adopted to allow for consolidation settlement.

The three critical cases for dam stability were end of construction; fully impounded with complete saturation to the seepage line; maximum drawdown. In each case low water was taken on the seaward side. It was decided that an initial factor of safety of 1.2 would be satisfactory for these cases, because of the increase to be expected with time as regain of strength after remoulding and long-term consolidation both take place.

It was not possible to predict accurately the amount of displacement of alluvium which would occur, and therefore the position of the fill/alluvium interface was determined by boring and vane testing as construction proceeded. On the basis of these results the final profile was to be designed section-by-section to give the required factors of safety.

BEHAVIOUR DURING CONSTRUCTION

Filling started with the compaction of London Clay fill in keys excavated in the old sea wall at each abutment and then proceeded by end tipping from both abutments, the initial top working fill level being +12 ft C.D. with a bank width of approximately 50 ft. The dock was closed near the centre at a time of low tide, and although an end position would have been preferred for the closure to minimise the risk of trapping soft material in pockets, no undue difficulty was experienced. Subsequent to the closure, work continued on placing and compacting fill to permit a general raising of the dam.

During and subsequent to the closure, significant amounts of settlement and lateral movement occurred as the ultimate bearing capacity of the remoulded foundation alluvium was exceeded. The alluvium was displaced to the sides of the clay fill, and at all times during the filling operations "bow waves" of displaced alluvium were present on all sides of the fill, often with secondary folding. During spells of dry weather desiccation cracking quickly developed on the displaced alluvium and a stiffer crust resulted. Fig. 3 shows in longitudinal section the amount of displacement of recent alluvium which occurred.

From the start the extensive settlement of the fill was accompanied by the development of cracks, and this was specially noticeable after the closure when superimposed loads were again increased due to the second stage heightening. A prolonged period of intense rain accelerated and spread the formation of cracks at one stage (Fig. 4) and remedial work was carried out rapidly on sealing the cracks to prevent the entry of water and further softening.

Some of the cracking appeared associated with a "graben" type downward movement of a central block of dam, probably due to lack of compatibility of the stress/strain relationship of the clay fill with that of the underlying alluvium. Towards the end of construction there was evidence of a wedge type movement on a large scale.

Throughout the construction period, investigations and tests were carried out which included boreholes, in-situ vane tests and laboratory triaxial and consolidation tests. These were to determine the location of the clay fill/alluvium interface and to obtain appropriate parameters for progressive stability analysis. The relationship of shear strength to depth, during construction, is shown alongside the initial relationship on Fig. 1. There appears to have been a slight increase in strength at upper levels, compared with the before construction values, and this is probably due to rapid consolidation of layers near the underside of the clay fill. As discussed later, clay lumps at the interface are thought to provide a drainage path for consolidation and the "during construction" results of Fig. 1 were

Fig. 4 Cracking after prolonged heavy rain following completion of first stage fill

obtained four months after first loading which is ample time for some consolidation strength gain. Very close to the interface, however, remoulding due to plastic flow was still occurring in September 1966, and so it was necessary to consider low shear strengths for the stability analyses.

When the dam neared completion the latest interface, and parameter information was used to determine the amount of strengthening

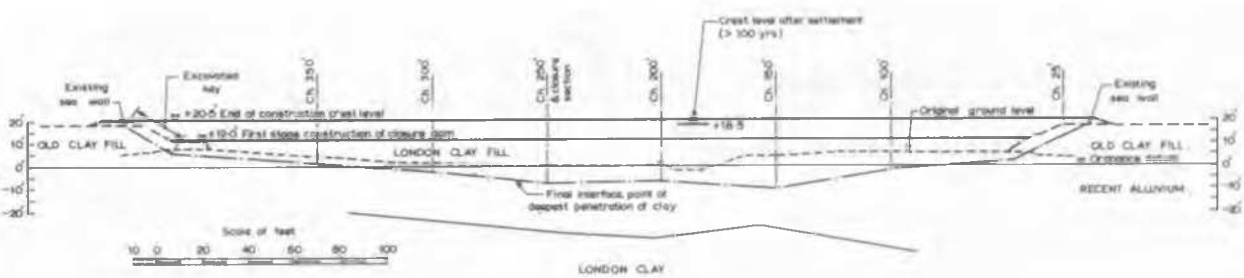

Fig. 3 Longitudinal section through dam and foundation

required for the section as then surveyed, to bring the factor of safety up to 1.2. The strengthening was designed as a 40 ft wide strip of well-compacted clay penetrating to specified minimum depths along the toe of the seaward side berm. The necessary penetration was generally achieved by progressive kneading with tractor shovels. However, at one location the existing fill was too thick to permit the success of this method, and narrow trenches normal to the dam axis were dug and filled successively to achieve the correct penetration of compacted clay.

STABILITY ANALYSES

From the properties of the fill and foundation materials it was apparent that a total stress analysis would be more appropriate than an effective stress analysis in order to ensure compliance with the design criteria.

Because of its sensitivity the recent alluvium is most accurately tested for shear strength by in-situ methods. The depth strength relationship obtained by in-situ vane testing has already been referred to and is shown in Fig. 1. For purposes of adopting design parameters, the recent alluvium in areas of incipient failure planes could be divided into four broad zones characterized physically by the degree of remoulding which had occurred in varying locations. These areas with the undrained shear strength values for the recent alluvium adopted after detailed analysis of the vane test results are shown in Table 1.

The vane tests on which these parameters were based were carried out after first completion of the dam filling to full height but when continuing movements of the main body of the dam and upper foundations were still being observed. Thus the parameters deduced for upper levels of recent alluvium were close to remoulded values with little regain in shear strength, and these were therefore suitable parameters for use in analysis leading to design of the reinforcement measures necessary for stabilising cross-sections to the chosen factor of safety.

In similar manner the London Clay fill could be divided into three main zones as shown in Table 1.

The division in quality of clay fill at +8 ft O.D. is related to the first closure embankment placed in the wet with less control, allowing for subsequent settlement of its original crest level during topping out of the dam. The values for the clay fill above original ground level were based on laboratory undrained triaxial tests, for one set of which samples were cut inclined to the vertical to induce failure on possible pre-disposed planes of weakness which might have been formed by joints between fill layers. This particular sample yielded a c_u value of 1031 lb/sq.ft.

Table 1 summarizes shear strength parameters and densities assumed for the stability analyses.

Consolidated drained triaxial compression

Table 1 Parameters of soils

Zone	Field Vane Test c_u (lb/sq.ft)					γ bulk	γ sat
	No. of tests	Max.	Min.	Mean	Value for Design	lb/cu.ft	lb/cu.ft
Clay fill above 8 ft O.D.		lab. quick undrained			1500	120	122
Clay fill below +8 ft O.D. to O.G.L.					1000	115	118
Clay fill below original G.L. and in toe trench	9	440	125	250	230	100	100
Disturbed recent alluvium at shallow depths below centre of dam	16	415	50	190	140	100	100
Disturbed clay fill or recent alluvium at toe of dam, surface	3	100	50	75	75	100	100
Recent alluvium at greater depths, central section of dam	22	470	100	270	270		
Disturbed recent alluvium at greater depth at toe of dam	6	170	110	130	130		

tests were carried out on undisturbed samples of the recent alluvium yielding values c' = 290 lb/sq.ft, ϕ' = 22° and c' = 0 lb/sq.ft. ϕ' = 28° 30'. However, effective stress analysis was found not to be a governing criterion for design.

For the total stress slope stability analysis two basic methods were applied. These were a slip circle method according to Bishop and the wedge analysis. As was to be expected with a flat dam of narrow crest width on foundations of low shear strength, the wedge analysis proved the more critical and was later used exclusively.

The final series of analyses were based on a detailed site investigation carried out immediately after completion of the main clay filling. Sections at 50 ft. centres along the centre line of the dam were each investigated by seven boreholes located in the following critical positions; centre line of dam; at toes of each upper slope; at middle and outer edge of each berm.

This site investigation provided detailed information concerning the position of the interface between the Recent Alluvium and clay fill and included determination of undrained shear strengths by vane testing. The results of the vane tests were used to determine the chosen parameters for the weak material as described above. Undisturbed samples of clay fill taken from the boreholes were used for laboratory determination of its parameters.

The closure area at chainage 250 ft was subjected to an extended testing programme including vane tests for foundation alluvium in each borehole and in two additional boreholes outside the toes of the dam, in the "bow wave" area of displaced alluvium.

Each of the sections as surveyed and investigated was analysed for both seaward and reservoir slopes. The seaward analysis was carried out for unsaturated (end of construction) and saturated cases, the latter assuming full saturation to top reservoir water level of +14.8 ft O.D. with no water against the sea face. First results for the sections as surveyed showed unacceptably low factors of safety and at this stage a trench backfilled with sound clay was incorporated as necessary at the toe in the analysis to yield the required factor of safety, with saturated conditions.

The reservoir side slope was similarly analysed for unsaturated and saturated cases, the latter including the effects of drawdown from maximum water level +14.8 ft to minimum retention level +11.8 ft. The results for this condition for the sections as surveyed were satisfactory and thus no toes strengthening was necessary on the reservoir side.

Fig. 5 shows a typical cross-section as surveyed and analysed at this stage indicating the location of the investigational holes, the parameters adopted for the various zones, the incorporation of the trench filled with sound clay at the riverside toe to improve stability on that side, the position of the most critical wedges and the method of analysis. A similar section and method were used for each of the cross sections at 50 ft intervals.

Because of the difficulty of constructing the strengthening trench backfilled with sound clay in the soft conditions existing at the toe, the final trench was not always identical with that designed in this series of analyses. However, in each case the toe trench, as constructed, was surveyed by

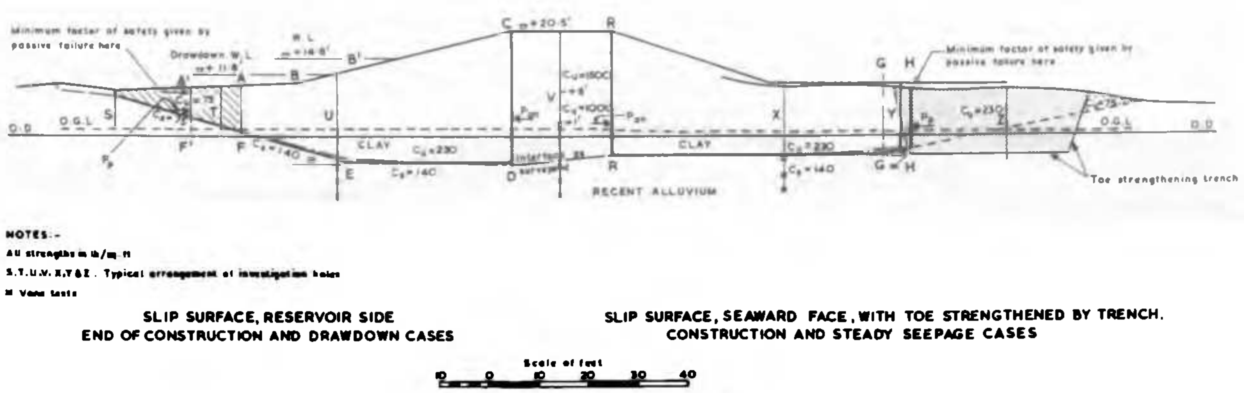

Fig. 5 Cross section at chainage 200 ft as surveyed, investigated and analysed at end of construction

augering upon completion, and in each case the section finally constructed was shown to be at least equivalent to the proposal, from the point of view of stability.

The lowest end of construction factor of safety obtained for all sections as finally modified by the constructed toe trench was 1.20, just equal to the minimum considered acceptable. The parameters assumed for this analysis were somewhat conservative, as may be seen from the range of results quoted above, and as the shear strength of the alluvium will increase further with time the factor of safety will increase also. Fig. 6 shows the detailed design cross section of the dam with slope protection and filters.

SETTLEMENT AND CONSOLIDATION STRENGTH GAIN

During the initial stages of placing the embankment fill the recent alluvium in the foundation became overstressed and plastic flow of the foundation material took place. This resulted in displacement of the foundation material by the fill. During investigations carried out throughout the construction period to determine the location of the interface, a maximum vertical displacement of the interface below original ground level of 12 ft was recorded (Fig. 3). No theoretical analysis of the behaviour at this stage appears realistic as the conditions were highly variable and indeterminate.

Following the stabilising of the dam, plastic flow virtually ceased and the foundations then became subject both to thixotropic regain of strength after remoulding, and to normal consolidation processes resulting in long-term settlement and strength gain. It was possible to predict the long-term settlements on the basis of parameters measured in some detail (16 samples and oedometer tests) for a site at Tilbury, where the Thames recent alluvium is very similar. The design parameters deduced from these results were confirmed by a single set of tests for Grovehurst Dock and the values adopted were:

$$c_v = 4.2 \text{ sq.ft/yr}$$

$$\text{and } m_v = 0.135 \text{ sq.ft/ton}$$

A simple one-dimensional consolidation analysis on this basis predicts a total settlement varying between 2.6 ft at chainage 100 ft completed after 280 years and 1.9 ft completed after 140 years at chainage 150 ft. A plot of the settlement curves for chainage 100 ft based on this analysis is shown on Fig. 7.

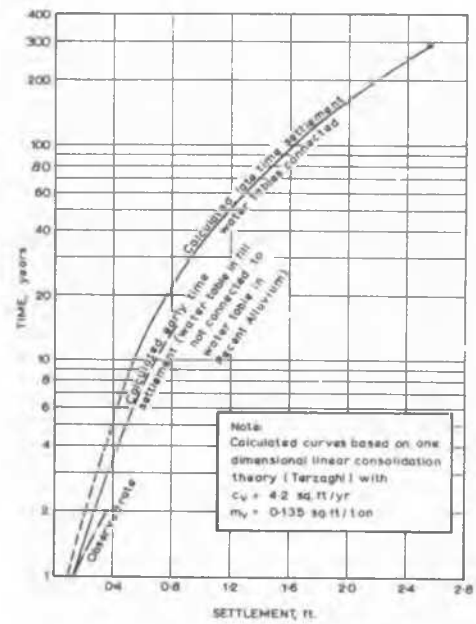

Fig. 7 Theoretical and observed settlement/time curves, chainage 100 ft.

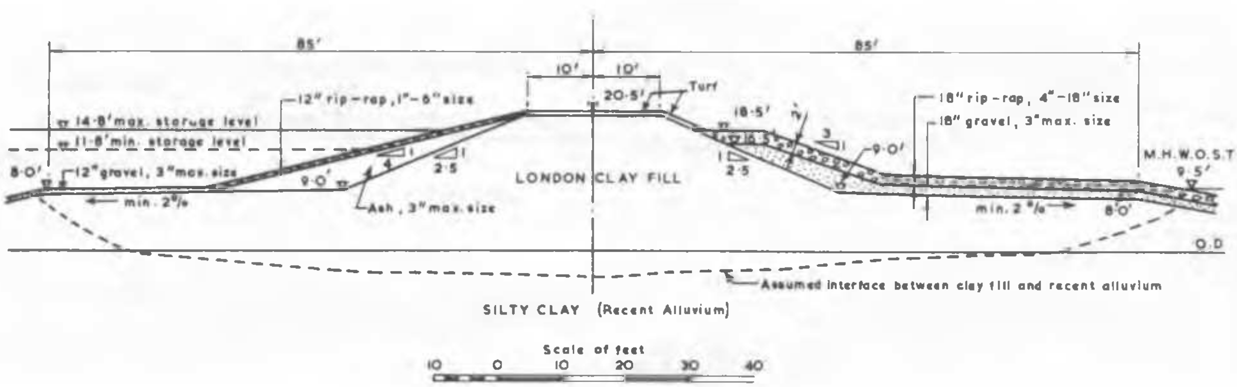

Fig. 6 Typical cross section

Settlement plates were used during construction of the dam, but due to the very large vertical and horizontal displacements which took place as plastic flow occurred the plates and their markers became inclined and displaced and therefore had to be discounted. However, directly stability was achieved permanent markers were established on the crest and berms of the dam, and readings on these have been proceeding at regular intervals. A plot of these results is also shown on Fig. 7. It will be seen that almost two years after completion of the dam, settlement against time continues steadily at a rate approximately 50% in excess of the calculated rate for this period. However, the uncertainties involved in the one-dimensional analysis are so considerable, particularly regarding drainage conditions, that this amount of agreement appears satisfactory. For the analysis, single drainage was assumed upwards to a drainage layer at the base of the clay fill, it being considered that the softened lumps of placed clay in this area would have sufficient voids to form a drainage layer. It was also assumed that downward drainage would not be significant. but this latter assumption has not been investigated exhaustively. However, it is likely that the observed rate of settlement results from a horizontal permeability greater than the vertical.

Early in the loading period, there was a perched water table in the clay fill separated from the ground water table, and for these conditions the fully consolidated effective stress in the foundations would be greater than that corresponding to the condition which develops later when the water tables become connected. Thus early and late time settlement curves are shown on Fig. 7, without any attempt at a firm prediction of the time of transfer from one curve to the other.

Further vane testing was carried out in November 1968, two years after completion of the dam, and the results are plotted on Fig. 1. There has been appreciable gain of strength due to consolidation immediately below the interface, supporting the hypothesis of an effective drainage layer there. Remoulded strengths also show a gain in the same depth range. Further results of interest were obtained in the "bow wave" area immediately downstream of the seaward side toe of the dam. This is an area of maximum disturbance where 75 lb/sq.ft was the maximum shear strength permissible for design analysis. The strengths remain below the normal depth/strength relationship before construction, but show at least double the minimum design strengths for the area. All evidence points to an appreciable improvement in factor of safety during the early service life of the dam.

WINDMILL CREEK DAM

Some thirteen years prior to the construction of Grovehurst Dock Dam, a dam was built by a similar method across Windmill Creek on the Island of Sheppey. The foundation conditions, the cross section of the dam, and the method of construction were all similar to those pertaining at Grovehurst. Thus Windmill Creek Dam formed a suitable basis for confirmation of predictions of future performance of Grovehurst Dock Dam, especially with regard to the thixotropic regain of shear strength and the consolidation performance of the recent alluvium foundations.

Three boreholes were therefore put down at Windmill Creek in order to permit a series of vane tests in the foundation alluvium, using the same field technique as for the Grovehurst Dock Dam site investigation.

Site investigation results from the time of construction of Windmill Creek Dam were available, including vane testing carried out prior to that construction in May 1953, and during construction in May 1954. The best-fit line depth/strength profiles for 1953 (before construction), for 1954 (during construction) and for 1966 are plotted together on Fig. 8. For each date two lines are drawn, a full one for undisturbed shear strength and a broken one for remoulded shear strength. In each case the

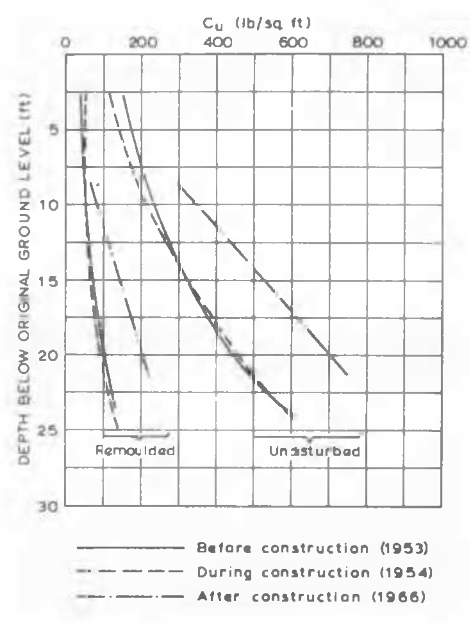

Fig. 8 Windmill Creek Dam. Depth strength relationships by in-situ vane tests

remoulded shear strength was measured at the same level as the undisturbed shear strength but in the remoulded case the vane was turned six times and then allowed to stand for five minutes before the shear determination. The best-fit lines were obtained by taking average values of individual strength determinations for particular levels. The before construction undisturbed shear strengths are generally below the equivalent values for Grovehurst Dock Dam as shown on Fig. 1. Comparing the 1953 and 1954 undisturbed shear strength profiles for Windmill Creek Dam, it can be seen that at levels within 10 ft of the original ground level there is a decrease of approximately 20% of initial shear strength after remoulding of the upper levels of recent alluvium by the construction works. This is in reasonable agreement with the laboratory results to assess the rate of recovery as shown in Fig. 2. At greater depth no significant effect of remoulding by construction is apparent and it may be inferred that the disturbing effect did not reach great depths. As would be expected, there is no significant difference between the 1953 and 1954 remoulded shear strength profiles.

The shear strength profile for 1966 shows a significant increase in both undisturbed and remoulded shear strengths. This increase is due both to thixotropic recovery of shear strength with time after disturbance and to the consolidation effect of higher overburden pressures applied since completion of the dam. It is interesting to compare the shape of this depth/strength relationship at a late stage of consolidation with that of the 1968 results for Grovehurst Dock Dam shown on Fig. 1 where conditions of only partial consolidation strength gain exist.

The favourable effect of continuing stability and loading with time was confirmed by field observation at Windmill Creek Dam, and to a lesser extent at Grovehurst, as being in good accord with predictions based on laboratory results.

CONCLUSIONS

The economical method of construction of Grovehurst Dock Dam, accepting initial plastic flow of the foundation material and cracking of the placed fill, has proved feasible. Operation of the dam throughout its first year has been satisfactory and observed settlements are reasonably close to predictions. Fig. 9 shows a photograph of the dam after a year's service. A close watch will be kept on future settlement, and should the crest level approach +18.5 ft O.D., the design minimum, before the rate of settlement becomes negligible, then additional fill will be placed as necessary.

As the main materials and factors influencing the stability at Grovehurst Dock Dam are very similar to those of Windmill Creek Dam it appears reasonable to extrapolate the stability results for Grovehurst Dock Dam to the extent confirmed by the field measurements at Windmill Creek. On the basis of such an extrapolation the factors of safety for Grovehurst Dock Dam may be expected to rise to approximately 1.5 over the next thirteen years.

ACKNOWLEDGEMENTS

Grovehurst Dock Dam was constructed by John Mowlem & Co. Ltd., to the design of Soil Mechanics Ltd.

The paper is presented by permission of John Mowlem & Co. Ltd. and Bowaters United Kingdom Pulp and Paper Mills Ltd.

Fig. 9 Completed dam and discharge pipeline, from reservoir side.