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Consolidation, Crusting and Loading of a Soil Slurry at 1 and 100 Gravities

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SUMMARY Experimental results from laboratory and field monitoring of the consolidation, crusting (desiccation) and load carrying behaviour of soil slurries are presented. The laboratory tests were conducted at 1 and 100 gravities. A comparison of the results obtained under the different conditions enabled a more complete picture of the engineering behaviour of a soil slurry, with application to the disposal and rehabilitation of coal mine tailings, to be obtained. Such an understanding would not have been obtained from the results of the individual tests studied in isolation.

1 INTRODUCTION

The engineering behaviour of fine grained mine tailings deposited as a slurry in tailings dams or ponds has, until recently, been of little interest to mine operators or the community at large. The philosophy adopted was one of containment of the tailings slurry with little consideration given to its dewatering, let alone its eventual rehabilitation. Large scale production of coal in Australia, which has escalated over the last two decades, has generated vast quantities of fine grained waste. Whereas the accompanying coarse waste or reject is easily handled, the fine grained waste (predominantly silt and clay sized) is difficult to dewater and hence difficult to handle.

Before existing tailings disposal areas can be rehabilitated there is a need to define the engineering behaviour of the material. This involves gaining an understanding of the processes involved: consolidation, crusting and eventual loading during rehabilitation, so that rehabilitation can be planned and executed on a rational basis.

In the light of the foregoing, field and laboratory studies on coal tailings at a number of mines in the Bundamba District of the West Moreton Coalfields near Ipswich in south-eastern Queensland were commenced in early 1986. This work continues with the assistance of postgraduate student Mr. Peter Morris and technicians Mr. Peter McMillan and Mr. Stephen Taylor. The field studies have necessarily windowed in on just small segments of the life-cycle of a tailings deposit, and have been restricted to investigations of dis-used tailings deposits which are currently undergoing surface crusting and continuing consolidation at a slow rate.

To enable a greater part of the life-cycle of a soil slurry to be studied, a geotechnical centrifuge test at up to 100 gravities was conducted at the Cambridge University Centre. A parallel test using the same soil and model geometry, but at 1 gravity, was also conducted for the purposes of comparison. The paper presents the results of field and laboratory monitoring of New Hope Colliery tailings and of laboratory studies on a kaolin/Gault clay slurry at 1 and 100 gravities.

2 TESTING PROGRAMME

2.1 Field Monitoring and Associated Laboratory Testing

The relevant field monitoring of coal mine tailings included studies of vane shear strength and moisture content profiles with depth, and a trial fill placed on crusted tailings. Associated laboratory testing of relevance included particle relative density (specific gravity), classification, particle size distribution and oedometer testing.

2.2 Centrifuge Testing

Three centrifuge tests were performed, but the results of only one of the tests will be presented and discussed herein. In this test, 40 mm of Gault clay slurry at an initial voids ratio e_o of 2.448 was placed over 200 mm of Speswhite kaolin at an e_o of 3.184. The purpose of the Gault clay was to provide a surface layer of finer pore size than the narrowly graded kaolin (mainly coarse clay sized), capable of supporting the high pore water suctions associated with surface crusting. However, in the test reported a surface covering of water remained over the slurry throughout the test and in any case the Gault clay did not remain on the surface but sank within the kaolin as the gravity level was increased.

The slurry model measured 675 mm long by 197 mm wide and was initially 240 mm in height. It was contained within a heavily reinforced centrifuge strong box fitted with a thick perspex front window. This allowed photographs of the model to be taken during flight. A full description of the Cambridge Geotechnical Centrifuge, on which the package was mounted, and its operation is given in Schofield (1980). The centrifuge has a working radius of about 4 m and can carry packages weighing up to 10 kN at up to 125 gravities. On the centrifuge, model dimensions are scaled up by the gravity level n and drainage time is speeded up by n^2 (due to the model drainage path length being 1/n of that in the prototype). Therefore by testing with a centrifuge the advantages of testing at model scale are combined with prototype stress levels and accelerated drainage times. At 100 gravities the initial prototype depth of slurry is about 24 m and 1 hour at model scale is roughly equivalent to 1 year at prototype scale.

In the centrifuge test the package was first taken up to 10 gravities, in accordance with the normal operating procedure of the centrifuge, and was then taken up to 100 gravities and maintained at that level for the duration of the test. During the first 8 hours at 100 gravities, consolidation of the slurry took place with little change in the water surface level. The slurry was free to drain towards the surface and could drain from the base under an imposed differential head of 30 mm (model scale). Base drainage gave rise to a small lowering of the water surface. After 8 hours surface water was slowly drained from the model and at the same time the differential head on the base drain was increased to maintain some base drainage. From 22.5 hours a series of four vane shear tests was performed inflight. After 24 hours at 100 gravities (27.4 years at prototype scale) a Leighton Buzzard BS 25/52 sand embankment was poured inflight over the surface of the slurry in four roughly equal lifts. This was designed to simulate the placement of fill to rehabilitate a coal tailings deposit. The sand embankment caused gross deformation of the slurry and following construction the dissipation of pore pressure within the slurry was monitored for about 1 hour before the test was stopped.

Instrumentation in the centrifuge test comprised 11 PDCR 81 Druck miniature pore pressure transducers fixed in position at various locations within the model by means of storks attached to the base. Deformation of the slurry was monitored by means of periodic photographs through the perspex window during flight.

2.3 1g Laboratory Testing

The centrifuge test was duplicated at 1 g with the time frame extended to 33 days under a surface cover of water, followed by 67 days in which the watertable was drawn down beneath the surface of the slurry and crusting occurred. The surface was then loaded by two surface footings. Instrumentation included a grid of internal lead shot to enable deformations to be monitored by means of X-radiographs and a number of standpipes to monitor pore pressures.

Associated laboratory testing comprised particle size distribution analyses. Extensive use was made of the considerable data available at Cambridge on Speswhite kaolin and Gault clay.

3 TEST RESULTS AND ANALYSES

3.1 Characterisation of Materials Tested

The characteristics of the materials tested are summarised in Table I.

The low value of G_s for the coal tailings is due to an average coal content of about 40 percent. The Leighton Buzzard BS 25/52 sand used to form the embankment in the centrifuge test had a G_s value of about 2.70 and was uniformly graded about a mean particle size of about 0.4 mm.

3.2 Profiles of Undrained Shear Strength

The profiles of undrained shear strength s_u with depth (in dimensionless form relative to the total depth) for New Hope Colliery tailings, the centrifuge test (from about 22.5 to 24 hours into the test) and the 1g slurry are shown on Figure 1. The tailings were 2.18 m deep and the strength profile shown on Figure 1 is the mean profile based on 34 uncorrected peak vane shear determinations made insitu. The watertable was located at about 1.4 m depth, below which the shear strength was measured to be roughly constant at about 7.5 kPa. The centrifuge test profile is based on four uncorrected peak vane shear determinations made inflight from 22.5 hours into the test. The height of the model at the test location at this stage was about 171 mm (consolidated from about 240 mm initially) and consolidation under self-weight loading was about 85 percent complete. The watertable was located about 37 mm above the surface of the slurry. Based on measured deformations of the 1g slurry at 33 days, it is estimated that its undrained shear strength profile was roughly linear with depth, reaching a value of about 0.4 kPa at the base. The slurry had undergone about 14 mm settlement at this stage and self-weight consolidation was complete. At 100 days the profile shown on Figure 1 was measured by uncorrected peak vane shear determinations. The slurry had undergone about 62 mm settlement (consolidated and crusted from about 240 mm initially), to give a height of about 178 mm and the watertable had passed out of the base of the model. The Gault clay/Speswhite kaolin interface was at a depth of about 30 mm. Below about 60 mm depth the undrained shear strength was measured to be essentially constant at about 1.5 kPa.

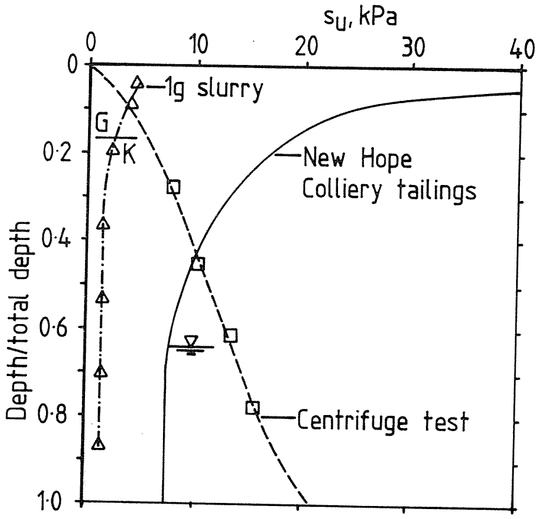

Figure 1. Profiles of undrained shear strength s_u

3.3 Variation of Voids Ratio

Provided that the degree of saturation of a soil slurry can be assumed with reasonable certainty, its void ratio e can be calculated from a measured moisture content w or from measured deformation profiles. The void ratio can also be estimated indirectly from measured s_u profiles. A value for the ratio of s_u to vertical effective stress σ_v' is first required, from which the bulk unit weight γ_b may be calculated (for a known watertable) and hence a value for e (for a soil of known G_s). For normally consolidated soils the ratio s_u/σ_v' was given empirically by Skempton (1957) as

$$s_u/\sigma_v' = 0.11 + 0.0037\ I_p \qquad (1)$$

where I_p is the plasticity index of the soil in percent.

Equation (1) yields values for s_u/σ_v' of 0.22 and 0.24 for Speswhite kaolin and Gault clay,

TABLE 1

CHARACTERISTICS OF MATERIALS TESTED

Characteristic	Material		
	Coal tailings	Speswhite kaolin	Gault clay
particle relative density G_s	1.73 (range 1.63 to 1.85)	2.61	2.71
plasticity index Ip	18 (range 15 to 31)	31	35
liquid limit W_L	43 (range 38 to 51)	69	60
% clay sized (< 0.002 mm)	30	80	50
% silt sized (0.002 to 0.060 mm)	50	20	45
% sand sized (< 0.060 mm)	20	0	5

respectively. Values of s_u/σ_v' for normally consolidated soils may also be determined using critical state soil mechanics principles and either the Cam-Clay or Modified Cam-Clay constitutive laws. The procedure has been set out by, among others, Davies (1981). Applying this procedure to Speswhite kaolin, for which the required critical state parameters are readily available, yields values for s_u/σ_v' of 0.21 and 0.22 for the Cam-Clay and Modified Cam-Clay laws, respectively. Strictly, Skempton's equation and the critical state models were based on isotropically consolidated triaxial test data. Wroth (1984) presented an interpretation of the vane shear test, arriving at an expression for s_u/σ_v' in terms of the vertical and horizontal effective stresses and the plane strain angle of internal friction. There are problems in applying this approach since the horizontal effective stress is very difficult to measure. However, approximate calculations yield similar values for s_u/σ_v' to those obtained above. The result is not very sensitive to the value of the friction angle. For the centrifuge and 1g slurry tests, normally consolidated conditions exist throughout the tests since load is always increasing. For these tests, values for s_u/σ_v' of 0.22 and 0.24 are adopted for the Speswhite kaolin and Gault clay, respectively.

For the crusted New Hope Colliery tailings, it is likely that past re-wetting and re-drying cycles will have made the crust overconsolidated. In a previous paper (Williams and Morris, 1987), it was estimated that the average overconsolidation ratio OCR of the New Hope Colliery tailings was 1.7. Wroth (1984) discussed the various expressions for determining the ratio $(s_u/\sigma_v')_{oc}$ for overconsolidated soil. The expressions were of the form

$$(s_u/\sigma_v')_{oc} = (s_u/\sigma_v')_{nc}\,(OCR)^m \qquad (2)$$

where $(s_u/\sigma_v')_{nc}$ is the value of s_u/σ_v' for the same soil under normally consolidated conditions, and

m is an exponent dependent on soil type and OCR.

Equation (2) has both a theoretical (derived from critical state theory) and an empirical basis. Wroth (1984) reported values for m in the range 0.68 to 0.87, with theoretical values typically of 0.8. Almeida (1984) quoted empirical values for m of 0.67 and 0.76 for Speswhite kaolin and Gault clay, respectively. For the purposes of this paper, a value for m of 0.75 is selected for the New Hope Colliery tailings. On the basis of limited laboratory testing to determine the critical state parameters for the coal tailings, the estimated value for $(s_u/\sigma_v')_{nc}$ is 0.24, giving an average value of 0.36 for $(s_u/\sigma_v')_{oc}$. Lower values are obtained if equation (1) is used. However, since equation (1) is purely empirical and was not based on data for materials of low particle relative density, such as coal tailings, it is not applied here.

The calculated profiles of void ratio e with non-dimensionalised depth for New Hope Colliery tailings, the centrifuge test (from 22.5 to 24 hours), and the 1g slurry at 33 and 100 days are shown on Figure 2. Where it was possible to determine the profile by two independent means, the results agreed to within about 15 per cent and usually to better than 10 per cent, and the profile derived by the most direct means is presented. The New Hope Colliery tailings e profile was based on measured moisture contents, and the centrifuge test and 1g slurry e profiles were based on the measured s_u profiles and adopted s_u/σ_v' values for the materials involved. For all but the uppermost data point of the coal tailings data, the degree of saturation was taken to be 1. The basis for this assumption is recent work by the author and Mr Peter Morris, which indicates that soil remains essentially saturated above the water-table until its moisture content drops below the plastic limit of the soil. The moisture content of the tailings was largely between its plastic and liquid limits and that of the 1g slurry did not drop much below its liquid limit. For the uppermost coal tailings data point, a degree of saturation of 0.6 was assumed, based on recent in situ density test results. For the centrifuge test, the presence of the Gault clay has been largely ignored since it migrated towards the base of the model during spin-up and because it formed in isolated lumps its effect on the behaviour of the model is considered minor. The centrifuge test in particular demonstrates the very marked effect on e of proximity to the top and bottom drainage boundaries. The two profiles for the 1g slurry show the marked effect of full-depth crusting.

3.4 Estimation of Permeability

Various workers have determined empirically based relationships between the coefficient of permeability for vertical flow k_v and the void ratio e. Al-Tabbaa and Wood (1986) found that data for Speswhite kaolin are approximated by the relation

$$k_v = 5.3 \times 10^{-10}\, e^{3.10}\ \mathrm{m.s^{-1}} \qquad (3)$$

Davies (1981) found that at similar e values k_v for Gault clay is about one-tenth of the k_v for Speswhite kaolin. Oedometer tests on 'undisturbed' New Hope Colliery tailings carried out by the author and Mr Peter Morris suggest a relation of the form

$$k_v = a\,(e - c)^b \qquad (4)$$

where a and b are constants dependent on the characteristics of the tailings,

and c is a limiting value for e resulting from the settling out of solid particles to form a lightly cemented cardhouse structure.

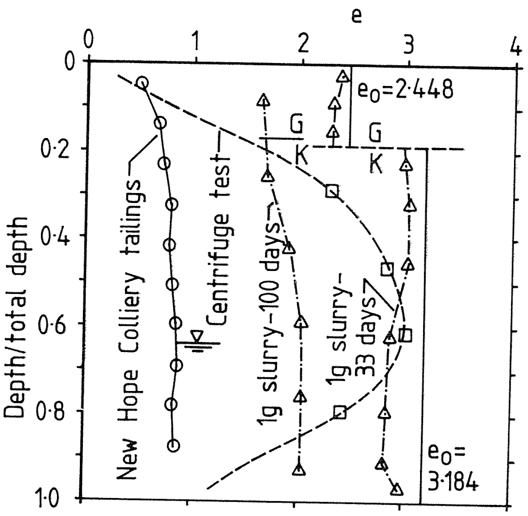

Figure 2. Profiles of void ratio e

Limited testing to date suggests values of 2.6×10^{-6}, 7.5 and 0.4 for a, b and c, respectively. Profiles of k_v based on the above relations are shown on Figure 3, which highlights the very dramatic drop-off in permeability towards drainage boundaries and even more so towards crusted surfaces.

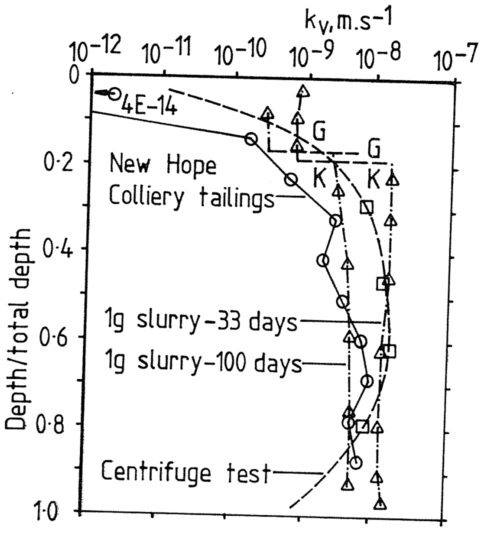

Figure 3. Profiles of permeability k_v

3.5 Estimation of Pore Water Suctions

In the tests described, pore water suctions were not measured, but they may be estimated for the cases in which they occurred by means of the s_u/σ_v' ratio. This is done for the crusted New Hope Colliery tailings on Figure 4 and for the partially crusted 1g slurry after 100 days on Figure 5. The magnitude of the pore water suction is given by the difference between the profile of vertical effective stress σ_v' for self-weight effects alone and that obtained from the s_u/σ_v' ratio which includes the effect of suction. For the coal tailings, an average $(s_u/\sigma_v')_{oc}$ ratio of 0.36 is seen to give convergence of the two profiles at about the level of the watertable to give roughly zero suction at that level. Towards the surface, the pore water suction approaches 200 kPa. In reality, the OCR would vary with depth, approaching a maximum value towards the surface. This would produce higher $(s_u/\sigma_v')_{oc}$ ratios towards the surface, indicating lower near surface suctions than those shown on Figure 4. For the 1g slurry, the suction approaches about 20 kPa towards the surface and has a substantial value over the entire depth.

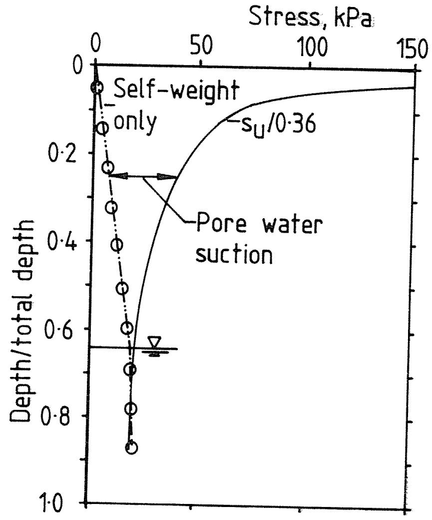

Figure 4. Estimated suctions in crusted New Hope Colliery tailings

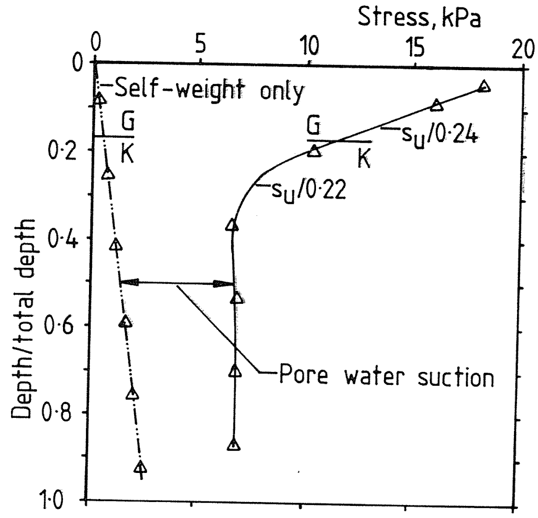

Figure 5. Estimated suctions in partially crusted 1g slurry at 100 days

3.6 Bearing Capacity

The results of a trial fill on the New Hope Colliery tailings are presented in detail in Williams and Morris (1987). A 2 m height of loosely dumped coarse reject was placed on the tailings, exerting a bearing pressure of 27.5 kPa. This was well short of the bearing capacity of the tailings and resulted in a maximum settlement of about 40 mm (less than 2 per cent of the tailings thickness). The bearing capacity of the tailings was estimated using the method of Brown and Meyerhof (1969), by considering punching failure of a stiff layer of finite thickness overlying a semi-infinite soft layer. This approach gave an estimated bearing capacity of 64.3 kPa, suggesting a factor of safety against failure of about 2.3. A consideration by the author of the bearing capacity of other tailings deposits in the West Moreton Coalfields suggests that the strength of the underlying soft layer governs bearing capacity.

In the centrifuge test, the sand embankment was poured through water on to a soil with zero strength at its surface. The strength increased roughly in proportion to depth to reach a maximum value of about 21 kPa at the base. Significant bearing capacity could be mobilised only through gross deformation of the soft near-surface soil. After four lifts, the maximum height of embankment had penetrated into the model up to 67 mm and exerted a bearing pressure of up to 106 kPa. This localised bearing pressure was about 7 times the pre-existing average value of s_u supporting it, allowing for no strengthening with time between lifts (about 4 min. in total at model scale or 28 days at prototype scale). In fact, little dissipation of pore water pressures occurred between lifts except close to drainage boundaries, indicating little load-induced strengthening. The maximum value of the bearing capacity factor N_c was therefore about 7. Applying this value to the average strength of the soft underlying layer of coal tailings suggests a bearing capacity of about 70 kPa.

The footing tests on the 1g slurry model gave bearing capacities of about 10 kPa, about 7 times the average strength of the soft underlying layer. The method of Brown and Meyerhof (1969) gave bearing capacities about 50 per cent lower than those measured.

4. DISCUSSION AND CONCLUSIONS

4.1 Factors Influencing Soil Slurry Behaviour

The behaviour of a soil slurry is strongly dependent on its permeability, which is itself a function of the void ratio, particle size distribution, plasticity and chemical composition of the slurry. For a given soil, k_v will be governed by e. In close proximity to drainage boundaries, e will reduce during consolidation with k_v reducing at a faster rate. This will inhibit the drainage of regions further from the boundary. A reduction in e driven by crusting will also lead to a dramatic reduction in k_v. An even more dramatic reduction in k_v is known to occur once desaturation sets in.

The bearing capacity of a partially consolidated and partially crusted soil slurry appears to be governed by the strength of the underlying soft layer, with a bearing capacity factor of about 7 being appropriate.

4.2 Implications for Coal Tailings Disposal and Rehabilitation

Substantial consolidation of coal tailings slurries can rarely be achieved in practice. Often there is no base drainage or the base rapidly becomes clogged with fines and the early formation of a surface crust inhibits further consolidation at depth. To take advantage of evaporation, it is necessary to deposit the tailings in relatively thin layers and allow them to crust. Lateral drainage through permeable containment bunds may assist dewatering. To achieve enhanced bearing capacity, it is vital that dewatering of the tailings at depth be facilitated.

5. ACKNOWLEDGEMENTS

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