PHYSICAL AND NUMERICAL MODELLING OF CONSOLIDATION OF MINE TAILINGS

M. Fahey[§] and S.H. Toh[¶]

ABSTRACT

In order to predict the consolidation behaviour of slurried tailings, it is necessary to use large-strain consolidation theory to take account of changes in soil properties during the consolidation process. Such a theory can be incorporated into a numerical model, which has the advantage that it can cope with changes in filling rates, and changes in boundary drainage conditions. The major problem with this type of model is to determine the consolidation parameters for use with the model. The paper summarises the approach adopted at UWA, which makes use of the Geotechnical Centrifuge to derive the model parameters, and also to carry out some limited verification of the model. The paper then describes some results which show what can be obtained with the numerical model, demonstrating the effects of different boundary drainage conditions and different filling rates on density and strength profiles at various times.

1 INTRODUCTION

The proper management of an area used for disposal of slurried tailings, both during and after the active disposal period, requires an understanding of the processes that control the movement of water in the tailings, the rate and final amount of surface settlement, and the changes in the strength profile through the depth of the tailings. This involves understanding the processes of initial sedimentation, self-weight consolidation under various boundary conditions, and formation of surface "crusting" due to the combined effects of water table lowering in the tailings and surface desiccation due to evaporation.

Consolidation of slurried tailings is a process which depends on self-weight stresses, and involves large strains (and hence changes in compressibility and permeability). It is therefore one class of problem which is most ideally studied using a Geotechnical Centrifuge, such as that operated by the Geomechanics Group at the University of Western Australia (an Acutronic model 661). The centrifuge serves two purposes in the tailings consolidation work: tests with simple boundary conditions can be carried out for calibration of the numerical model, and some parametric studies and model verification can be carried out by varying the drainage boundary conditions.

This paper summarises the method of calibrating the numerical model for each new tailings deposit. The model is then used to show the effect of different drainage boundary conditions and rates of tailings disposal on the overall behaviour.

2 CENTRIFUGE MODELLING

Centrifuge modelling is the ideal method of investigating self-weight consolidation of soft soils because it allows the full-scale problem to be properly simulated, but in a much reduced time. When a container of soil is fixed at the end of the rotating arm of a centrifuge, it is subjected to a centripetal acceleration of $Ng = \omega^2 r$, where ω is the angular velocity of the arm, and r is the arm radius. The soil in a centrifuge model has therefore a unit weight ρNg, where ρ is the soil bulk density, and hence a model in the centrifuge has the same stress distribution as a body of soil N times deeper in a $1g$ gravity field. Thus, the centrifuge allows problems to be modelled at a scale of $1/N$ while maintaining true stress similitude.

Because time in consolidation events depends on the square of a characteristic length, consolidation on the centrifuge is N^2 times faster than it takes for the equivalent prototype. Thus, 1 day of consolidation on the centrifuge, at an acceleration of $100g$, is equivalent to 100^2 days (27 years) consolidation at prototype scale.

A description of the centrifuge facility at UWA is provided by Fahey *et al.* (1990). The sample of tailings is contained in a container of rectangular or circular cross section designed to withstand the high stresses of centrifuge testing. The layout of instrumentation for a typical tailings consolidation test is illustrated in Figure 1.

Drainage from the sample during the test can be either just to the top surface (one-way drainage) or to both top and bottom surfaces (two-way drainage). A solenoid valve allows the bottom drainage to be switched on or off at any time. The water table in the bottom drainage layer can be maintained at any level, giving complete control of the base pore pressure during the test. Water that drains to the top surface can be either decanted or left on the surface.

During a centrifuge test, the strength profile can be determined "in-flight" using a cone penetrometer, a miniature shear vane, or a T-bar penetrometer (Stewart and Randolph, 1991). In one case for tests on mineral sands tailings, surface footing tests were also carried out at various stages to assess bearing capacity and foundation stiffness (Toh and Randolph, 1992). The actuators for these types of tests are indicated on Figure 1. As well as permitting vertical loading of the cone or footing, the actuators can also move horizontally, so that cone or

---

[§] Senior Lecturer and [¶] Research Student, Geomechanics Group, The University of Western Australia.

footing tests can be carried out at more than one location without stopping the centrifuge, allowing strength profiles to be determined at different times during the test.

Surface settlement is monitored using a displacement transducer (LVDT) resting on a small surface bearing plate. The LVDT core is counterweighted using a pulley system to counteract the increase in self-weight with increasing acceleration, thereby preventing it from sinking through the slurry during the early stages of the test while the slurry is still soft. A similar system can be used to monitor settlements at any depth using a plate buried at that depth.

Miniature "Druck" pore pressure transducers are used to monitor the progress of consolidation. By accurately counter-weighting these transducers (in the same manner as with the LVDTs), the transducers settle with the clay specimen during the consolidation process, thereby remaining at the same "material coordinate".

At the end of the test, the sample is dissected to obtain samples for determination of the final density and water content profiles, and to check the final location of pore pressure transducers.

Much of the initial proof testing was carried out using commercially-available kaolin, with a liquid limit of 60 and a plastic limit of 27, but since then the testing has been extended to tailings from the alumina, gold and mineral sands industries. The tailings may be prepared in a slurried state at the same solids content as they are pumped to the disposal area. In some industries, the sand and fines ("slimes") fractions are separated during the extraction process, and some centrifuge tests have been carried out to investigate the options of re-blending these fractions before disposal, compared with deposition in layers.

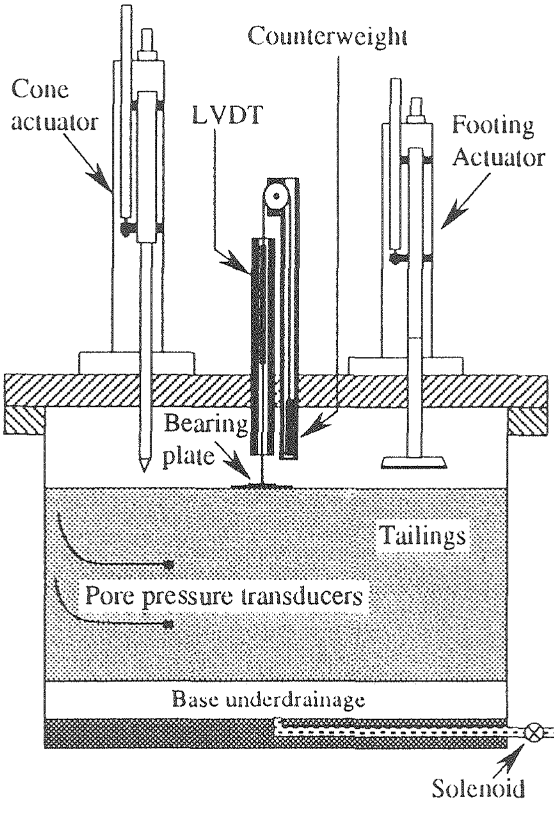


Figure 1. Layout of centrifuge test.

3 NUMERICAL MODELLING

3.1 Large strain consolidation

The consolidation of slurried tailings is a classic example of large strain consolidation – that is, where changes in soil thickness and soil properties (compressibility and permeability in particular) are too large to allow Terzaghi theory to be applied directly. A large-strain consolidation model has been developed by Gibson *et al.* (1967), which allows some of these factors to be taken into account. However, if the finite element or finite difference method is used, there is no need to adopt this type of explicit large-strain formulation, provided an appropriate time step is used in the analysis, self-weight stresses are taken into account, and the geometry and material parameters are updated after each increment. In effect, the small-strain formulation is applied to small increments of consolidation, and the large-strain aspect is dealt with by updating the problem after each increment.

In practice, the numerical approach using the incremental finite element or finite difference method has the advantage that the updating phase after each increment can include addition of fresh tailings at the surface, as would occur in practice during the active life of the disposal area, and changing the boundary drainage conditions such as reduction in permeability or changing the water table in any base filter layers, for example where natural or engineered base drainage is present. Non-homogeneity of the soil skeleton can also be incorporated by grouping the elements into different categories each having different properties.

The assumptions of constant permeability and compressibility in the small strain theory are valid within each increment of consolidation provided the increment size is not loo large. However, some care must be exercised in choosing the size of the time increment, since this affects the stability of the numerical analysis.

In the absence of creep effects, the permeability and compressibility of the soil depend on the void ratio alone. Thus:

$$e = e(\sigma') \quad \text{(1)}$$

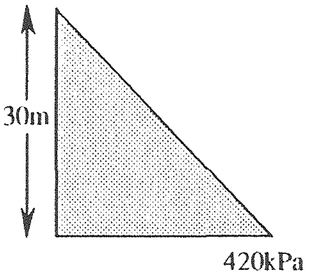
and

$$k = k(e) \quad \text{(2)}$$

The actual relationships used are obtained by the fitting procedures described below.

3.2 Suction

Final rehabilitation of a tailings disposal area requires that the surface of the tailings has achieved sufficient strength to allow access onto the surface for placing surface fill (for surface contouring, or for replacing topsoil). Self-weight consolidation with the water table maintained at the surface for either one-way or two-way drainage produces high strength at depth, but very low strength at the surface. However, lowering the water table can overcome this problem, since it results in negative pore pressure (suction) which increases with height above the water table. Fahey and Toh (1992) illustrated this using the case of a 30 m deep layer of tailings consolidated with a base water table maintained at 10 metres above the base. Assuming an initial unit weight of 14 kN/m3, the initial and final stress states are shown in Figures 2(a) and (b), while Figure 2(c) shows the pore pressures at different stages of consolidation. (Note that this Figure takes no account of density or height changes which occur in reality). The suction above the water table results in significant effective stresses, and hence significant strength, right to the top surface.



Initial pore pressure = initial total stress

(a)

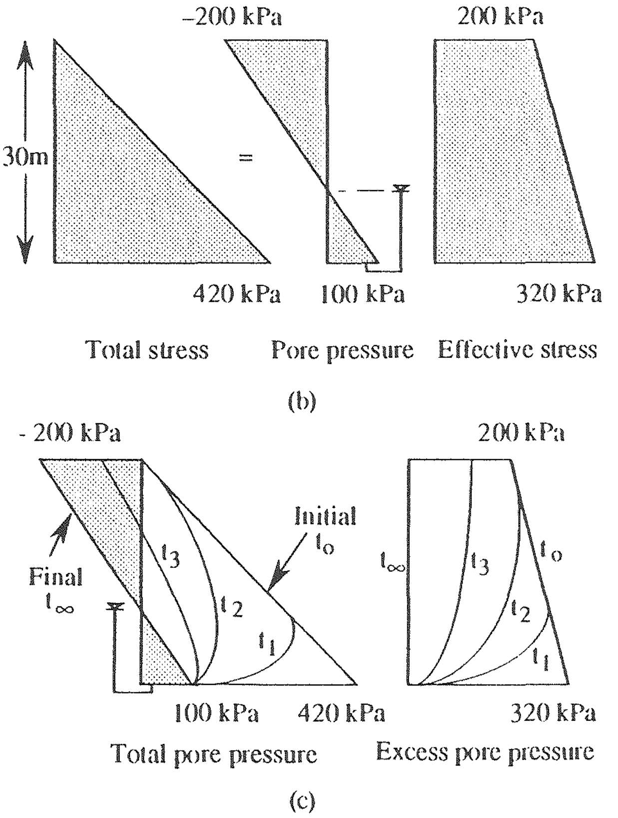


Figure 2. Illustration of process of consolidation from slurry with underdrainage: (a) initial stress and pore pressure; (b) final conditions; (c) schematic representation of pore pressure isochrones during consolidation (after Fahey and Toh, 1992).

Consolidation due to suction cannot continue to increase indefinitely, because eventually the soil starts to de-saturate when the suction at the surface reaches the air entry value for the soil. For any particular soil, this point can be established readily using a shrinkage limit test, since the shrinkage limit corresponds to the point of air entry. A simple version of this test consists of carrying out a standard test for linear shrinkage, but with the addition that the sample in the linear shrinkage mould is measured and weighed at intervals during the drying process. The water content (and hence void ratio) at which shrinkage stops can thereby be established. An example of applying this procedure for kaolin and gold tailings is shown in Figure 3. This information can be incorporated into the model to prevent further suction-induced consolidation after this point is reached.

For the kaolin modelling discussed later in the paper, the suction at the shrinkage limit can be deduced to be very high (certainly more than 1 MPa), which is much higher than can be achieved just by water table lowering in most tailings deposits. However, evaporation from the surface can produce suctions much higher than those required to initiate desaturation. This does not result in further shrinkage, but it can give a very significant increase in shear strength, and hence can play a very important part in rehabilitation of the tailings area.

3.3 Shear strength

The consolidation model does not give shear strengths directly. However, the shear strength can be deduced using the Cam Clay model, and this has been incorporated into the program. This allows the shear strength distribution with depth to be calculated at any stage. The ability to carry out strength measurement on the centrifuge at any stage of the test allows the model predictions of strength to be verified, at least qualitatively. The most convenient strength measurements are carried out using continuous penetration

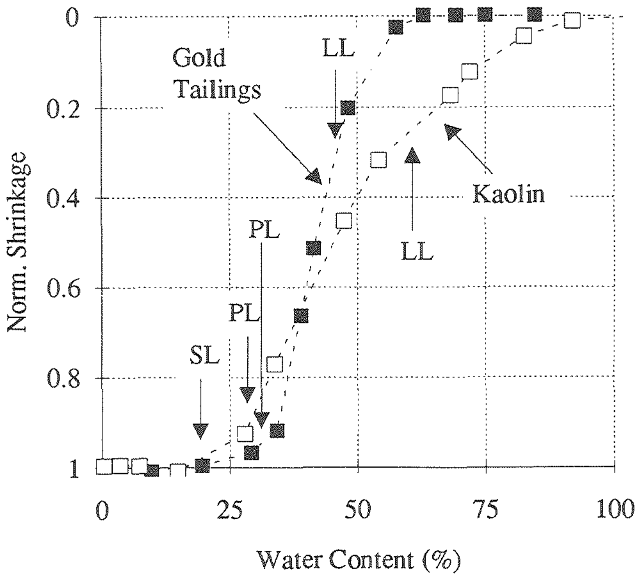


Figure 3. Shrinkage limit measurements on gold tailings and kaolin.

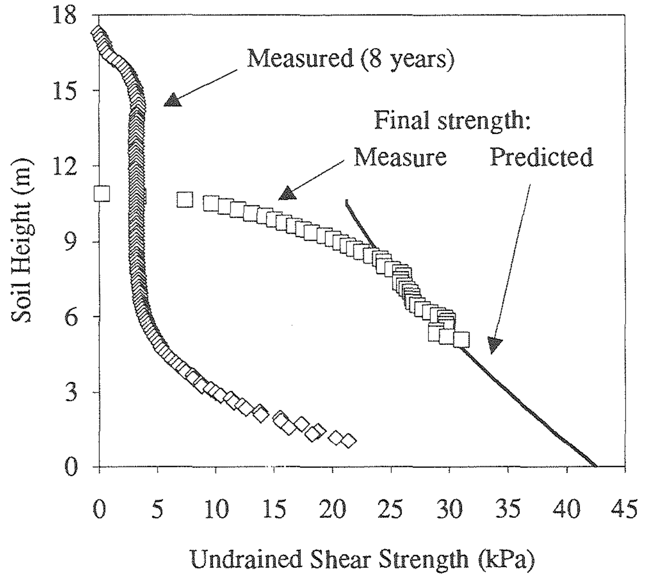


Figure 4. Strength profiles measured with T–bar penetrometer, and final calculated strength profile.

devices, namely the cone penetrometer and the T-bar penetrometer (Stewart and Randolph, 1991). Derivation of strength from the former requires the use of an empirical factor N_q, but for the latter a closed form solution allows the strength to be determined directly.

Figure 4 shows an example of a test in which the T–bar penetrometer was used to measure strengths is discussed by Fahey and Toh (1992). In this Figure, the symbols represent the measured strengths at two different stages of the test, and the line shows the final calculated strength profile. The agreement between the final profiles is good, except right at the surface, where the analysis for the T–bar penetrometer may not be appropriate.

Strength profiling using the cone penetrometer was also used in centrifuge tests on mineral sands tailings in which surface evaporation was not prevented during the test (Randolph *et al.*, 1991). In this case, strengths in the desiccated crust equivalent to q_c values of over 10 MPa were obtained as a result of the evaporation-induced suctions.

3.4 Calibration of numerical model

The numerical model developed at UWA has great flexibility, and can be used to study the effects of varying filling rates, decanting free water appearing on the surface during consolidation, changing drainage boundary conditions, etc. Programs to analyse large-strain consolidation problems already exist elsewhere. However, all such programs require input parameters, and some means of verifying the predictions, if they are to be used with confidence in practice.

In fact, it is possible to do this calibration and verification using a combination of laboratory consolidation tests and observations of the performance of full-scale disposal areas, but the centrifuge offers the most direct method. However, the strategy adopted at UWA is to use the geotechnical centrifuge both for the model verification stage, and to provide soil parameters for each new tailings problem being investigated, as described by Fahey and Toh (1992). The centrifuge test is thus not merely a simulation of field behaviour but may also be utilised to obtain the parameters required for a numerical model.

In the numerical analysis, relationships between voids ratio and effective stress (e–σ') and permeability and void ratio (k–e) for the slurry are required to solve the large strain consolidation problem. In the proposed procedure, the e–σ' relationship is obtained from the water content distribution with depth through the slurry after the completion of consolidation on the centrifuge.

There are (at least) two methods to determine the k–e relationship. The most direct method is by laboratory measurement using either the constant head or the falling head methods. These techniques, presented by Al-Tabbaa and Wood (1987), and Aiban and Znidarcic (1989), have shown good k–e relationship for voids ratios greater than about 2. However, most slurried tailings have higher initial voids ratios, typically greater than 3. These laboratory techniques are not suitable for such soft soils due to the consolidation effect of the seepage stresses which occur during the permeability test

The k–e relationship can conveniently be obtained using a back-analysis method, where the k–e relationship is varied to obtain the best fit between numerical prediction and behaviour observed in either full-scale or centrifuge tests. This relationship can then be used to predict subsequent tests in order to verify both the numerical analysis and the k–e relationship.

In the methodology outlined by Fahey and Toh (1992), centrifuge tests with relatively simple configurations are required to provide the basic parameters, and the numerical model is then used to carry out the parametric study of the disposal options. A variation of the procedure is to use a test with the simplest drainage (drainage to the top surface only, for example) to provide the model parameters, and then use the numerical model with these parameters to predict the behaviour in other tests with different drainage boundary conditions (two-way drainage, with or without lowering of the base water table, for example). For the kaolin used in the numerical analysis described later in this paper, the relationships derived using this procedure were:

$$e = 3.15(\sigma')^{-0.162} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

where σ' is in kPa, and

$$k \text{ (m/day)} = 4.32 \times 10^{-5}\left(\frac{e^{4.25}}{1+e}\right) \quad \ldots\ldots\ldots\ldots\ldots (4)$$

The model was then used to predict the behaviour in tests with two-way drainage, with and without water table lowering in the base drainage layer, showing good agreement between the predicted and observed behaviour.

Of course it is recognised that this procedure provides only limited verification of the model and the parameters used. True verification is possible only by comparing the predictions with the observed behaviour in a full-scale problem. The approach described here has been used for a number of projects in Western Australia for the alumina, gold and mineral sands industries. However, to date only one such project presented the opportunity to compare the predictions to the behaviour observed in a fully instrumented full-scale tailings deposit. Predictions in this case can be termed "Class B" predictions (i.e. made after the event, but with the predictions being made before the predictor had access to the results of the full-scale test). Final analysis of these results is still being carried out, and the results will be presented in a forthcoming paper (Fahey *et al.*, 1993). However, it appears at this stage that good agreement can be obtained using the proposed approach.

3.5 Limitations of the model

Apart from the geometry and material property changes which occur in a full-scale tailings deposit, two other very important aspects of real behaviour are not allowed for in the simple scheme outlined above. These are:

- Lowering the water table results in suctions which increase with distance above the water table, as outlined above. However, as suction increases, the stage may be reached where de-saturation starts to occur, so that the simple relationship between pore water pressure and effective stress no longer holds. The suction corresponding to this point can be established using a simple shrinkage limit test, as outlined above. In reality, since the shrinkage limit is generally lower than the plastic limit, and since the effective stress at the plastic limit is generally of the order of 800 kPa (see Wood, 1990), the effective stress corresponding to the shrinkage limit would be higher than 800 kPa.
- In many parts of Australia, the latter stages of consolidation of the soil near the surface may be dominated by evaporation, which will give higher strengths than predicted in a simple consolidation theory. Thus, the approach is conservative, especially in areas of high net evaporation, especially where disposal is in thin layers with some time being left before disposal of subsequent layers for evaporation to have an effect. During the early stages of evaporation, the rate of evaporation is equal to the pan evaporation rate, and this is currently being incorporated into the model as an option for the boundary condition at the soil surface. More sophisticated models will be required for the later stages of evaporation where the rate falls below the pan evaporation rate.

4 APPLICATION OF NUMERICAL MODEL

4.1 Effect of drainage boundary condition

In earlier papers (particularly Fahey and Toh, 1992, Toh and Fahey, 1991 and Toh, Fahey and Kitamura, 1991), measured pore pressure, density and strength profiles in centrifuge tests with various drainage boundary conditions were compared to predictions using the numerical model. In this paper, the types of predictions which can be made using the numerical model will be illustrated. This type of parametric study would typically be undertaken in the early stages of design of a disposal area.

The cases considered are summarised in Table 1. In all cases, the voids ratio on deposition was 3.0. The soil parameters used are those for pure kaolin, obtained as outlined above. The variables considered were:

- The base drainage condition: Case 1 had an impermeable base, while all the other cases had a permeable base, but with the water table in the base drainage layer maintained at different heights above the base;
- Treatment of water which drained to the surface: In Case 4, the water draining to the top surface was removed immediately (decanted), but in all the other cases, it was allowed to pond on the surface. In the final phase of consolidation where all drainage is towards the base (Cases 2, 3, 5, 6 and 7), this ponded water has to be drawn down through the soil profile before final equilibrium can be reached, thereby adding to the total consolidation time.
- Time to deposit the tailings: In Cases 5 and 6, the tailings were deposited at a uniform rate over periods of 10 years and 5 years, respectively. (In fact, tailings were added in monthly intervals, with a depth of 0.167 m and 0.333 m being added per month, respectively). For all other cases, the tailings were deposited instantaneously to a full initial depth of 20 m.

Case	Base head (m)	Surface water	Time to deposit 20 m
1	Impermeable	Ponded	Instantaneous
2	18	Ponded	Instantaneous
3	7	Ponded	Instantaneous
4	7	Decanted	Instantaneous
5	0	Ponded	10 years
6	0	Ponded	5 years
7	0	Ponded	Instantaneous

Table 1. Summary of cases considered in parametric study.

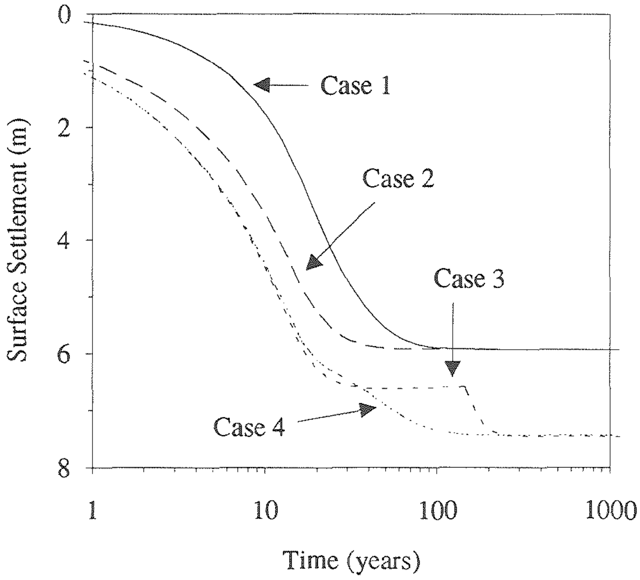


Figure 5. Surface settlements, Cases 1–4.

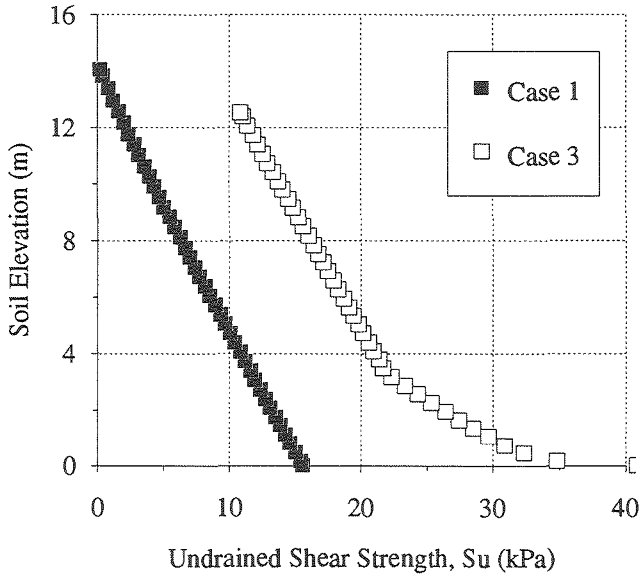


Figure 6. Final strength profiles for Cases 1 and 3.

The surface settlement versus time for Cases 1 to 4 is shown in Figure 5, which illustrates the importance of drainage boundary condition both on the final amount and on the rate of settlement (the use of a logarithmic scale of time tends to mask the contrast between the curves).

Figure 6 shows the strength profiles in Cases 1 and 3 (Cases 2 and 4 are not included as they give identical profiles to Cases 1 and 3, respectively). The effect of the final suction above the water table for Case 3 (and 4) is to give shear strength even at the surface of over 10 kPa, while this strength is only achieved in Case 1 (and 2) at a depth of about 10 m below the surface.

The progress of consolidation for Case 4 is illustrated by the series of plots shown in Figure 7. In these plots, the initial and final conditions are shown by solid square symbols (■) and hollow triangle symbols (△), respectively, with conditions at the intermediate times indicated being

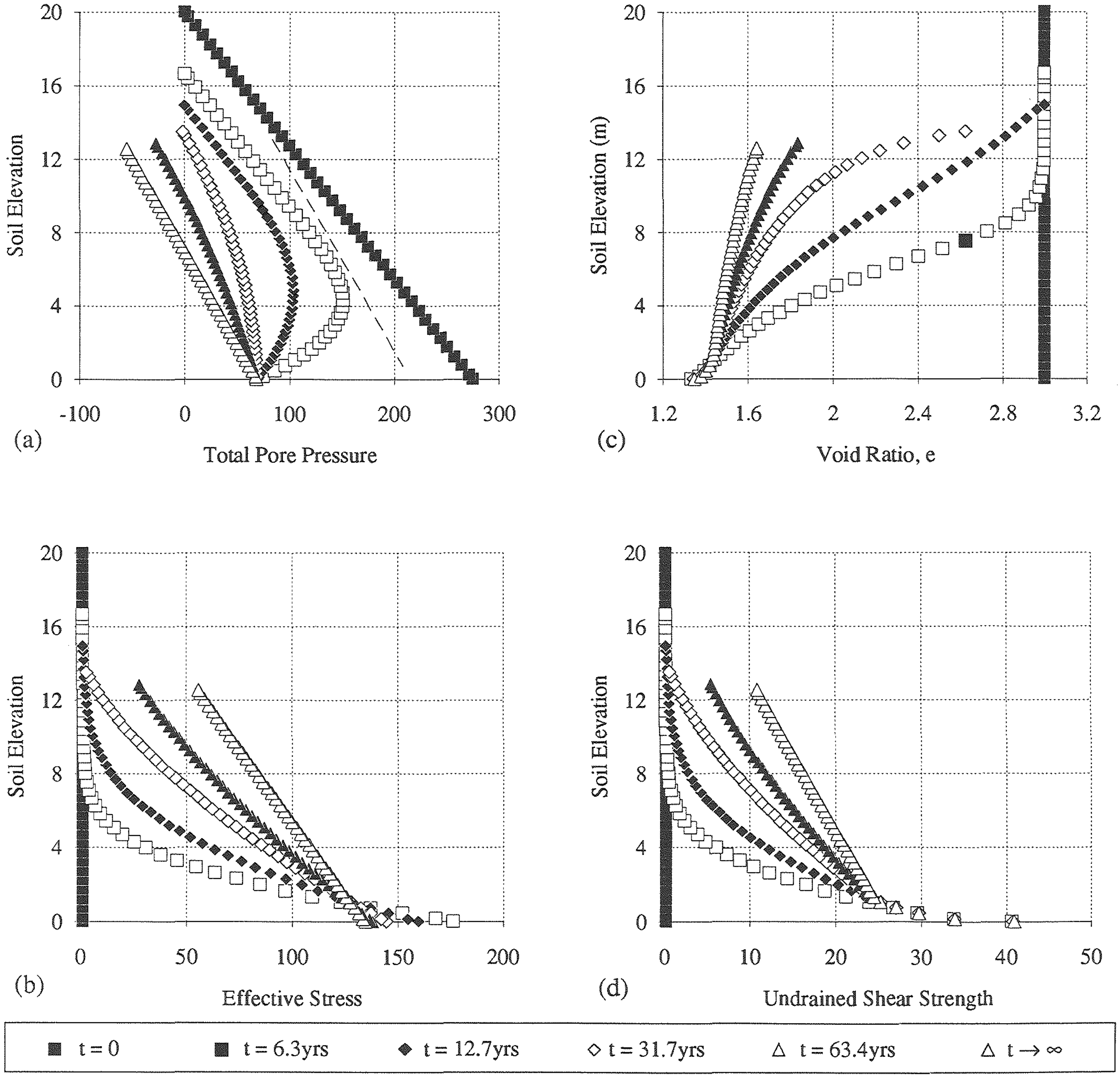


Figure 7. Conditions at different times for Case 4.

shown by the other symbols. Figure 7(a) shows the total pore pressures at these times. Initial conditions show total pore pressure equal to total overburden pressure, but the base pore pressure would fall to the value set in the base drainage layer (70 kPa) immediately after the start of consolidation.

In this Figure, a dashed line at a tangent to the curve for 6.3 years is drawn at the hydrostatic gradient ($du/dz = \gamma_w$). By comparing the gradients of the pore pressure isochrones with the hydrostatic gradient shown by the dashed line, the direction of water flow may be determined, so that the point of tangency with the isochrone divides regions of upward from regions of downward flow. For the case indicated (at t = 6.3 years), drainage is occurring upwards in the zone above 6 metres, and downwards in the zone below that height. This point of inflexion gradually moves upwards, until at a time of about 31.6 years (◇), drainage is totally downwards. (Note however that at this stage, the amount of water flow would be very small). From this point onwards, suctions begin to be generated at the top surface, provided water has not been allowed to pond on the surface. If ponding has been allowed, this water must be drained back into the soil before suctions can initiate. This explains the difference between cases 3 and 4 in Figure 5 – the ponded water in Case 3 continues to drain back into the soil until about 150 years, and the re-starting of settlement at that time is the result of the initiation of suction. The final profile shows a hydrostatic pore pressure, with zero at 7 metres above the base.

It is instructive to note that in this case, the analysis was extended to a total time of over 1000 years to ensure conditions for "infinite time" had been achieved – i.e. to ensure that complete equilibrium had been attained. Even after 316 years (▲), significant excess pore pressures still remained. The parameters used in this analysis are those for pure kaolin, and this might therefore not be representative of any real situation. However, it is also true that many types of tailings contain a significant clay fraction, so that very long times to reach final equilibrium would be expected in some situations.

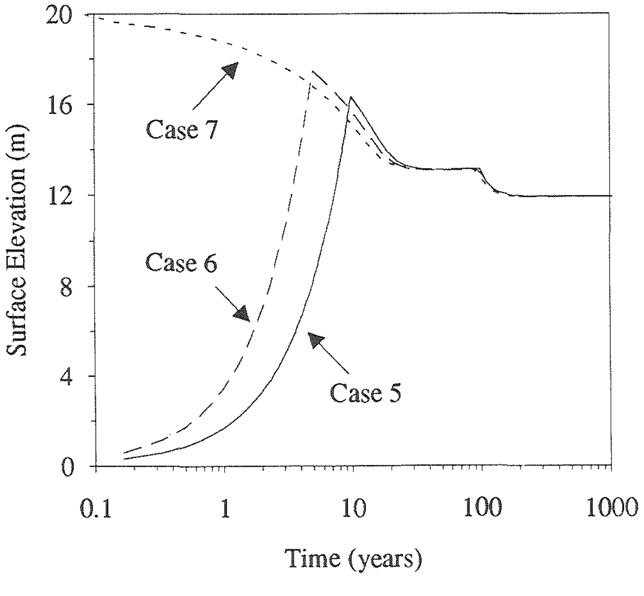


Figure 8. Changes in surface elevation for Cases 5–7.

The effective vertical stress, voids ratio and shear strength profiles corresponding to these pore pressure isochrones are shown in Figures 7(b), (c) and (d), respectively. All these follow a consistent trend. It is clear, for example, that the improvement at the surface due to the low base water table is not felt until suctions are initiated, which is more than 30 years after deposition. Note also that in this case, the final effective stress at the base is slightly lower than the peak value reached shortly after the start of consolidation. This slight overconsolidation is the reason for the change in the final gradient of shear strength close to the base in Figure 7(d) (and Figure 6).

4.2 Effect of filling rate

The effect of varying the rate of filling the disposal area is examined in Cases 5, 6 and 7. In these three cases, the drainage conditions are identical, but whereas filling is instantaneous for Case 7, filling takes place at a uniform rate over periods of 10 years (0.167 m/month) and 5 years (0.333 m/month) for Cases 5 and 6, respectively.

The changes in the elevation of the tailings surface for these three cases are shown in Figure 8. The maximum heights of filling for Cases 5 and 6 are about 16.2 and 17.5 m respectively, which occurs at the end of deposition (after 5 and 10 years, respectively). When completely filled, the surface elevation in each case is close to that for the case of instantaneous filling. Settlement due to surface suction starts only after 100 years, since in all three cases, water was allowed to pond on the surface rather than being decanted.

This type of calculation would be required in planning a tailings disposal strategy, and shows the importance of being able to model the whole problem if a prediction of the rate of filling of a tailings disposal area is required.

The conditions at different stages of filling and subsequent consolidation for Case 6 are shown in Figure 9. Comparing these plots with those for Case 4 in Figure 7 allows the effect of the extra 7 m of drawdown of the base water table to be appreciated. Thus, for example, the final shear strength at the surface is about double that for Case 4, and the final void ratio profile is much more uniform. However, the long times required to even approach equilibrium should again be noted.

4.3 Changes in soil properties

Inspection of Figures 7(c) or 9(c) shows that the void ratio changes significantly during the process of consolidation, starting from a uniform value of 3.0 in each case, with a final value at the base of about 1.3. In Figure 10, the relationships between permeability and voids ratio (k-e) and voids ratio and effective stress ($e\text{–}\sigma'$) used in the prediction exercise are shown. On this Figure, the vertical dashed lines show the range of voids ratio applicable in this case. Also shown on this Figure is the value of coefficient of consolidation (c_v) obtained by combining permeability and stiffness in the usual way. As the voids ratio reduces from 3.0 to 1.3, it can be seen that the value of c_v varies by about an order of magnitude, as indicated by the arrows on the right-hand axis Thus, it is clear that it is not appropriate to carry out calculations on the assumption of a constant value of c_v.

5 CONCLUSION

The paper has summarised the approach to predicting the consolidation behaviour of mine tailings using a combination of centrifuge testing and numerical modelling. The approach consists of carrying out centrifuge tests on the tailings under simple drainage conditions, and using the results of these tests to provide parameters for a finite difference computer program, which can then be used to examine the effects of changing geometry or drainage conditions, predicting the response to different rates of filling, and determining the final density and strength profiles. In order to demonstrate the types of results which can be obtained, an example of applying the program to the case of a 20 m deep tailings deposit is described.

By carrying out centrifuge tests on the same material, but with different boundary drainage conditions, some verification of the approach can be obtained. However, to date, detailed verification has been possible with only one well-instrumented deposit.

The cases considered in the paper show that significant strength gain can be obtained right to the soil surface if the tailings disposal area has drainage through the base, with

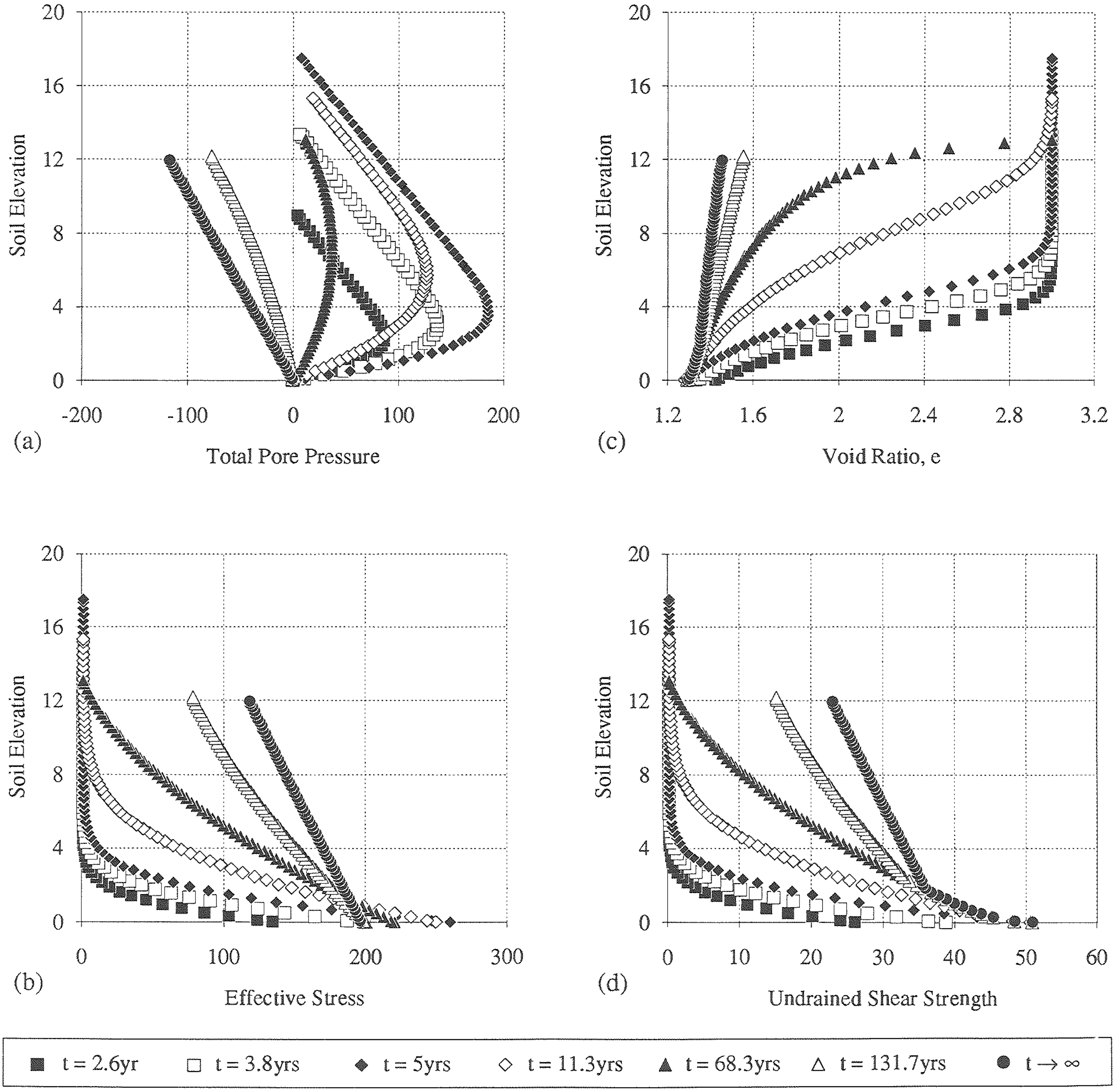


Figure 9. Conditions at different times for Case 6.

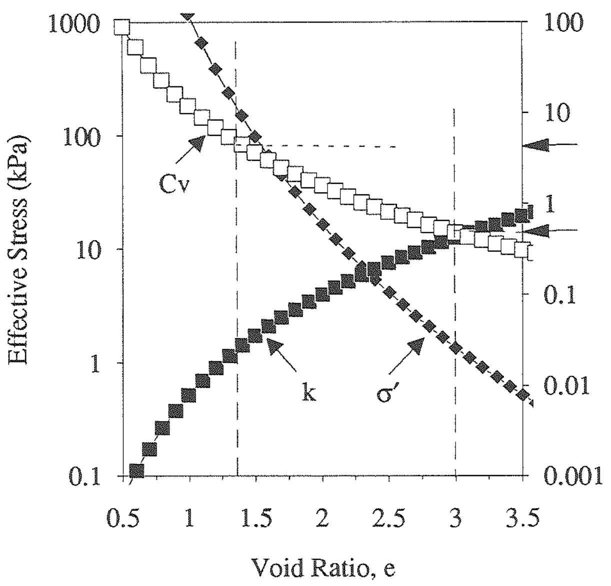


Figure 10. The σ'–e and k–e relationships used in the analysis (and deduced c_v–e relationship).

the water table at the base being maintained at or below the base of the tailings. However, it is also shown that the times for consolidation can be very long, and hence this strength gain may not occur until some time after the end of tailings disposal. In some of the centrifuge tests where evaporation was not prevented, surface desiccation results in much higher strengths being achieved (Randolph *et al.*, 1991).

As stated in the paper, the current version of the numerical model does not take any account of the effects of evaporation. In many parts of Australia, net evaporation rates are very high, and therefore should be taken in to account, especially in assessing the rate of strength gain at the surface. During the early stages of evaporation, the rate of evaporation is equal to the pan evaporation rate, and this is currently being incorporated into the model as an option for the boundary condition at the soil surface. More sophisticated models will be required for the later stages of evaporation where the rate falls below the pan evaporation rate.

ACKNOWLEDGMENTS

The work described in this paper was supported by a grant from the Australian Research Council. The second author is supported by an Overseas Postgraduate Research Studentship from the Commonwealth Government, and by a Studentship from the Geomechanics Group at the University of Western Australia.

REFERENCES

Aiban, S.A. and Znidarcic, D. (1989). Evaluation of the flow pump and constant head techniques for permeability measurements. *Géotechnique*, **39**, No.4: 655–666.

Al-Tabbaa, A. and Wood, D.M. (1987). Some measurement of the permeability of kaolin. *Géotechnique*, **37**, No.4: 499–503.

Fahey, M., Finnie, I., Hensley, P.J., Jewell, R.J., Randolph, M.F., Stewart, D.J., Stone, K.J.L., Toh, S.H. and Windsor, C.S. (1990). Geotechnical centrifuge modelling at The University of Western Australia. *Australian Geomechanics*, No. 19 (December), 33–49.

Fahey, M. and Toh, S.H. (1992). A methodology for predicting the consolidation behaviour of mine tailings. *Proceedings Western Australian Conference on Mining Geomechanics*, Kalgoorlie, 445–452.

Fahey, M, Toh, S.H. and Gower, A. (1993). Modelling consolidation of mine tailings. Paper in preparation for: *Conference on Geotechnical Management of Waste and Contamination*, Institution of Engineers, Australia, Sydney, March.

Gibson, R.E., England, G.L. and Hussey, M.J.L. (1967). The theory of one-dimensional consolidation of saturated clays – finite non-linear consolidation of thin homogeneous layers. *Géotechnique*, **17**, No.3: 261–273.

Randolph, M.F., Hensley, P.J., Bhattarai, B, and Toh, S.H. (1991). *Rehabilitation of mineral sands tailings: Report to Westralian Sands*, Department of Civil and Environmental Engineering, The University of Western Australia, May, Report Geo:91103.

Stewart D.P. and Randolph M.F. (1991). A new site investigation tool for the centrifuge. *Proceedings of the International Conference on Centrifuge Modelling: Centrifuge '91*, Boulder, Colorado, 531–538.

Toh, S.H. and Fahey, M. (1991). Numerical and centrifuge modelling of large-strain consolidation". *Proc. 7th International Conference for Computer Methods and Advances in Geomechanics*, Cairns, Vol. 1, 279–284.

Toh, S.H., Fahey, M. and Kitamura, R. (1991). The effect of water table lowering on the consolidation behaviour of soft clay. *Proceedings of the International Conference on Geotechnical Engineering for Coastal Development: Theory and Practice on Soft Ground (Geo-Coast* '91),Yokahama, Japan, Vol 1, 267–272.

Toh, S.H. and Randolph, M.F. (1992). Consolidation behaviour of mineral sands tailings. To appear in *South East Asian Journal of Geotechnical Engineering*.

Wood, D.M. (1990). *Soil behaviour and critical state soil mechanics*. Cambridge University Press.