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CAPPING OF AN EXTREMELY SOFT NEUTRALISED URANIUM TAILINGS — A CASE HISTORY IN ENVIRONMENTAL GEOMECHANICS

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ABSTRACT

The environmental regulation for Ranger Uranium Mine, in the Northern Territory, Australia, requires the stored tailings (irrespective of where they are finally stored, whether in a tailings dam or in mined-out pits) be capped safely to minimise contamination, erosion and radon gas emission, and also to enable the site to be rehabilitated through revegetation. The hydraulically deposited neutralised tailings in the existing tailings dam have a 'gel-like' structure which hinders the consolidation process. Consequently, the *in situ* tailings have a high water content, low permeability and extremely low shear strength. The present paper describes the case history of a successful capping trial constructed on these extremely soft tailings. Environmental geotechnical issues are highlighted and investigated. Results, especially the characteristics of the tailings observed during the construction and subsequent monitoring, are also discussed.

KEYWORDS

Tailings, Capping, Geotextile, Monitoring, Strength, Bearing capacity, Environment, Geomechanics.

INTRODUCTION

Ranger Uranium Mine is located in the Northern Territory in Australia. Under the present Northern Territory legislation, The Uranium Mine Act. ER29 (1979), there are two Environmental Requirements that the mine must comply with, and plan for, regarding the final deposition of tailings at the mine. The first (ER29(a)) requires that all tailings shall be deposited in or transferred to the mine pits not later than five years after the cessation of mining. The second requirement states that

> *'If after 10 years from the date of issue of the Authority but before the cessation of mining on the Ranger Project Area, the Supervising Scientist reports that he is satisfied that, by dealing with the tailings in the manner outlined in the report, the environment will be no less well protected than by depositing or transferring the tailings to the mine pits …….'*

The main alternative method that had been considered for tailings rehabilitation is *in situ* capping of the tailings in the tailings dam (Unger and Milnes, 1992). Examination of this alternative method was commissioned by Ranger and it provided significant research information. Until such time as an application under Environmental Requirement (ER29(b)) is made, the legal situation is that all tailings must be returned to the pit. Given that a decision must be made by the Supervising Scientist regarding any application to change the rehabilitation method, considerable government resources have been directed to this issue as well as research by Ranger (Woods, 1996).

The milling operation for extracting uranium from the ore uses a sulphuric acid leaching process. Consequently, an acidic liquid with pH 2 to 2.5 is produced. To avoid radionucleides and heavy metals in solution and the acid itself from contaminating the groundwater, lime was used as a neutralising agent to increase the pH of the liquid to a value around 7. Thereafter, the tailings were hydraulically transported to the tailings dam and deposited sub-aerially. At present, all tailings are deposited in Pit No 1.

Due to the chemical reaction between lime and the acidic liquid, the tailings tend to develop a 'gel-like' structure which hinders the consolidation process. Consequently, *in situ* tailings have a high water content, low permeability (resulting from gelling) and extremely low undrained shear strength. If the tailings are to be stored for all time in the tailings dam, a decommissioning design needs to be developed. One possibility, suggested by the company, is to construct an initial 3 metre cap over the tailings surface. To achieve this goal, the bearing capacity of tailings needs to be significantly improved.

In plan view, Figure 1, the tailings dam is roughly square with an area of about 1.1 km^2. The depth of tailings deposits varies from 10 to 20m. From the site investigation and laboratory tests, the undrained strength of the tailings was shown to be very low (5 to 10 kPa). Consequently, extensive improvement in bearing capacity was necessary. In addition to the feasibility study of techniques for improving the bearing capacity of the tailings, the following issues were considered during the design stage:

- accessibility of heavy duty earth moving equipment to the tailings dam;
- optimal design parameters for improvement of tailings bearing capacity; and
- optimal capping structure for effective suppression of radon gas emission.

Two techniques were proposed, one using high tensile strength geotextile (HTSG) and the other using a wick drain technique. Two pilot trials were designed based on the two techniques. The trial with HTSG reinforcement was chosen, and implemented, as the first step in this project.

The capping trial project was started in May 1995, and it was executed in the following four phases:

- Phase 1 - Preliminary investigation, which included site investigation and sampling, and identification of one or two feasible techniques for soft tailings improvement.
- Phase 2 - Design of pilot capping trials, which involved the analysis and evaluation of all existing geotechnical *in situ* and laboratory testing results and the development of designs for the capping trials.
- Phase 3 - Construction of a capping trial, which also involved the development, design, construction and installation of the monitoring system, obtaining field data, and also ensuring that the trial was safely constructed.
- Phase 4 - Specification and recommendations for the final capping design.

Currently the project is approaching the end of Phase 3. The present paper gives an overview of the project history and results to date.

GEOTECHNICAL PROPERTIES OF TAILINGS

The geotechnical properties of tailings have been studied through both site investigation and laboratory testing programs. The site investigation included vane shear, cone penetration and permeability tests, as well as core sampling. Tailings deposited at the testing site were about 12m deep. The undrained strengths from the vane shear tests varied from 3 to 10 kPa, at the time when the *in situ* testing was conducted. The higher values were attributed to thin coarse graded layers between softer finer-graded sediments. There was no noticeable increase in shear strength within the top 8m of the deposit. The distributions of core penetration tip resistance with depth were similar to those of the strength values for vane shear tests. No significant increase in tip resistance was evident with depth, which indicated that the consolidation of tailings was progressing very slowly.

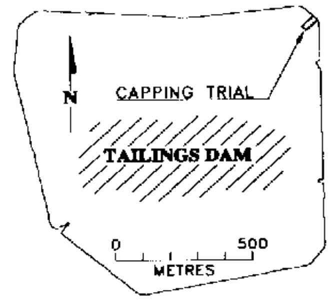

Figure 1. Ranger tailings dam and the location of the trial site.

The *in situ* permeability of tailings was tested at various depths using two sets of nested piezometers, each comprising 7 standpipes. The coefficient of permeability derived from the *in situ* data varied between 3×10^{-6} and 1×10^{-8} m/s, with higher values measured for the thin coarse layers.

The naturally deposited tailings had high water contents and low densities. The water contents of the tailings varied between 35% to 102%, with values averaging around 55%. The dry densities varied from 0.73 to 1.32 t/m^3, whilst the average was about 1.04 t/m^3. The particle densities varied from 2.76 to 2.84 t/m^3.

Unconsolidated undrained shear strengths, C_u, from the undisturbed samples tested in triaxial apparatus was consistent with that obtained through *in situ* testing. The C_u values were around 3 to 4 kPa for the fine samples and 7 to 12 kPa for the coarse samples. The undisturbed tailings had a friction angle of 30° with no cohesion in the consolidated undrained triaxial tests. Consolidation and compression behaviour of the undisturbed tailings were tested in oedometers. The average coefficient of consolidation, C_v, was about 3×10^{-7} m^2/s. The compressibility index, C_c, was about 0.2.

CAPPING TRIAL DESIGN

Design Criteria

Apart from the requirement for overall stability against a sliding or bearing failure, the size, thickness of the capping trial and the properties of the reinforcement material had to be considered in the design of the capping trial.

Considering the costs associated with improvement and utilisation of the strength gained from consolidation some time after the surcharges were placed, it was proposed that initially a 2m thick cap would be constructed on the tailings surface at the trial site.

Heavy duty earth moving equipment, commonly used in the mining industry, is normally not suitable for carrying out construction on extremely soft ground, such as that which is known to exist in the Ranger tailings dam. The smallest available bulldozer was a Komatsu D65P which had an operating weight of nearly 30 tons and a ground contact pressure over 25 kPa. In addition to the self weight of the bulldozer, the dynamic loading due to mobilisation of the bulldozer might also have a significant impact on the stability and cause potential liquefaction in tailings.

The rectangular area of the trial site was 25m wide and 65m long, extending towards the west, from the eastern wall of the tailings dam, as shown in Figure 1. The longitudinal direction of the trial was perpendicular to the dam wall. Site selection took into account the above-mentioned factors, the configuration of the proposed monitoring system and dimension of the geotextile. The average depth of tailings under that half of the trial, furthest from the dam wall, was about 12m. The designed capping trial thickness using available waste rock was 2m. The bulk unit weight of the loosely loaded waste rock was tested at a value of 18.5 kN/m^3.

Stability Analysis

Undrained sliding stability analysis using the Bishop approach gave a factor of safety of about 0.68 for a 2 metre high cap, indicating that significant reinforcement was required to support the designed waste rock cap. In view of the undrained strengths from both *in situ* and laboratory, an average value of 5 kPa was adopted in the stability analysis. The depth of the slip surface was estimated to be about 6 metres below the tailings surface. For the capping system, the reinforcement design aimed to achieve a degree of stability, with a factor of safety of 1.3, which is a value commonly adopted for embankments on soft clays (Fowler, 1982; Koerner, 1994).

As the behaviour and characteristics of the Ranger tailings are significantly different from those of soft clays, it is not yet clear whether the principles and theories established for soft clays are applicable to soft tailings. Three analytical methods, namely the Ordinary (Fellenius), the Bishop and the Janbu methods, were employed to model the stability of the proposed capping system. In order to compare the results, it was assumed that the same slip surface was followed in the reinforced case for all three methods. The details have been reported by Sheng et al. (1997). Based on the analysis, it seems that the Janbu approach seems more appropriate for the stability analysis of tailings with geotextile reinforcement. The Bishop approach gives the most conservative results.

To achieve the desired factor of safety of 1.3 (Janbu approach), the geotextile has to provide a tensile strength of some 300 kN/m in the direction of the major principal stress (ie, a direction perpendicular to the longitudinal direction of the capping). In addition to the surcharge load, the weight of the construction equipment, (ie. bulldozer) must be taken into account, as this equipment also applies an additional load to the reinforcement. Previous studies (Koerner, 1994) indicate that an additional tensile strength of 150 kN/m for the geotextile would be required just because of the use of the additional 30 kPa ground pressures applied by the bulldozer. This would require a significant increase in the strength of geotextile and therefore the costs.

In view of the fact that the capping was planned to be constructed in a staged manner, the strength of tailings would increase through consolidation. The stresses initially induced in the geotextile will gradually release (shared by the tailings). As a result, Welback woven geotextile with a tensile strength of 300 kN/m in the warp direction was selected for the reinforcement. The tensile strength in the weft direction of the selected geotextile was 150 kN/m.

Bearing Capacity

Current recommended methods for calculating the bearing capacity of soft clay in embankment design (Koerner, 1994, US FHA, 1992) assume that the global bearing capacity of the reinforced foundation is independent of the presence of a geotextile. If the global bearing failure occurs, the failure area is normally located beyond the limits of the reinforced zone. The recommended approach for estimating bearing capacity in the plane strain situation is

$$q_a = cN_c = F_s q_r \tag{1}$$

where q_r (= γH_{max}, in which γ is the unit weight of fill material and H_{max} is the maximum height of fill) is the allowable bearing capacity; c is the cohesion of the foundation material; N_c is the bearing capacity factor from 3.5 to 5.7 (Koerner, 1994); and F_s is the factor of safety for the global bearing capacity.

According to Equation 1, the maximum bearing capacities would vary from 17.5 to 28.5 kPa for a foundation with an undrained strength of 5 kPa. If a global factor of safety of 1.3 is applied, the tailings would only provide a safe bearing capacity of 13.5 to 21.9 kPa. The required bearing capacity for a 2m waste rock cap, assuming that the unit weight is 18.5 kN/m^3, would be 37 kPa. Therefore, the global bearing capacity can apparently only support about a 1m thick cap. Consequently, a stage loading construction strategy is required.

Deformation Analysis

Significant deformation may occur in a foundation prior to a complete failure. To study the effect of deformation on the stability of the capping system, non-linear finite element analysis was undertaken for both reinforced and unreinforced cases (Sheng et al. 1997). The largest lateral movements in both cases were predicted at a depth of 4m below the tailings surface, which is 2m shallower than that predicted using the limit equilibrium stability analyses. However, the lateral displacement of the tailings can be significantly reduced by using the geotextile, especially in the near surface areas.

CAPPING TRIAL CONSTRUCTION

Capping construction commenced in August, 1996. After surveying the site, the HTSG, with the joints stitched at the site, was placed on the trial site with weft direction coinciding with the axis of the trial. After installation of the monitoring devices, a sand layer was spread on to the HTSG. Due to other activities at the mine having a higher priority, the construction of the first layer of waste rock was postponed until October, 1996. The second lift of the waste rock was constructed in May 1997. The loading for this stage was limited to the western end of the capping trial. Therefore, the whole capping system includes one layer of sand, and two layers of waste rock, with thicknesses of 0.6m, 1.0m and 0.9m, respectively. *In situ* tests showed that the bulk density of the loosely dumped waste rock was about 1.85 t/m^3. Therefore, a loading history relationship can be developed as shown in Figure 2.

As there is no well established design approach currently available for construction of capping on extremely soft tailings, the establishment of an *in situ* monitoring system is important. It can be used to ensure the safety of the capping construction, to verify the design parameters that may be used in the future and to design the construction of large scale capping over the whole of the tailings dam.

PERFORMANCE OF THE TAILINGS

The instruments installed in the tailings included vibrating wire and standpipe piezometers, surface and layered settlement stations, layered displacement stations and strain gauges. Figure 3 shows the configuration of the instruments installed.

Settlement of Tailings

A total of 10 surface settlement stations (SS1, SS3 to SS10) were installed. One settlement station, SS1, was installed in the eastern side of the capping trial to monitor the effect of equipment mobilisation. The rest of the stations, SS3 to SS10, were installed in the western side, which is the main part of the instrumentation site. The first group (SS4, SS7 and SS10, from the east to the west) was located along the centre line of the cap, while group 2 (SS3, SS6 and SS9) and group 3 (SS4, SS8 and SS11) were located at the northern and southern sides of the centre line, respectively. Figure 4 shows the settlement that has developed on the surface of tailings under the capping. As expected, the settlements along the centre line were greater than those at both sides.

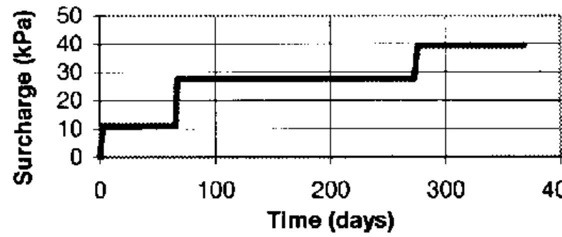

Figure 2 Loading history of the capping trial.

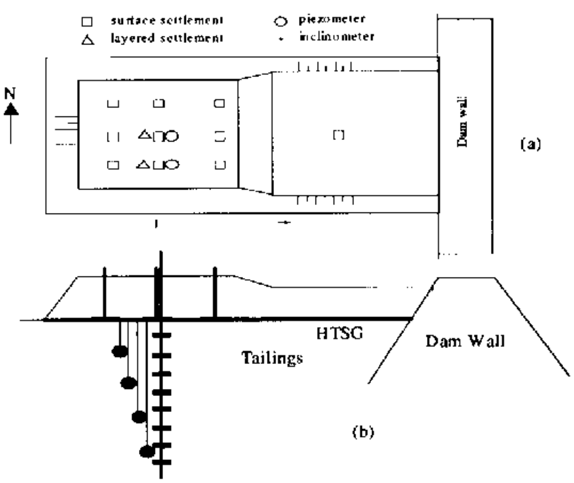

Figure 3. Schematic diagram of the capping trial and monitoring system installed.

The second stage of waste rock was only loaded at the western side of the capping trial. The centre for this stage loading was located around the station SS7. Therefore, the largest settlement induced by this stage loading would occur near the station SS7. From Figure 4, it can be seen that the settlement of SS7 was approaching the settlement of SS4 after the second stage loading.

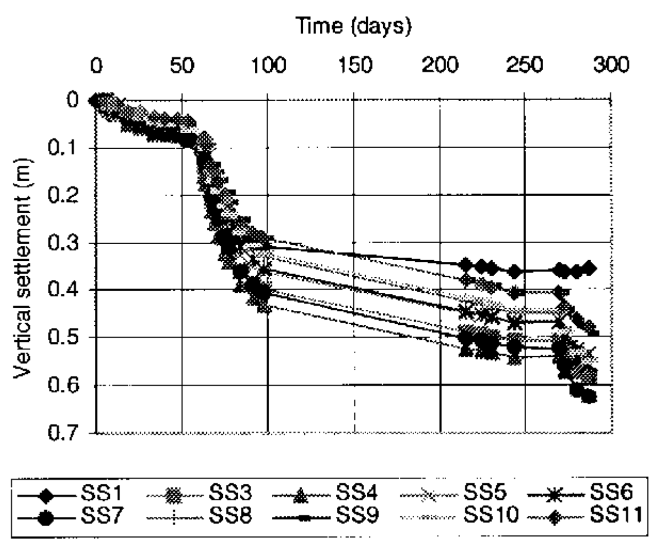

Figure 4. Surface settlements of tailings.

Two layered settlement stations, LS1 and LS2, were installed in the capping trial. Figure 5 shows settlements down the profiles of tailings from the station, LS1, installed at the centre of the major instrumentation area near SS7. The settlements were indicated by a set of magnetic rings installed along the standing settlement pipe. These were spaced at 1m intervals from the surface to a depth of 9m.

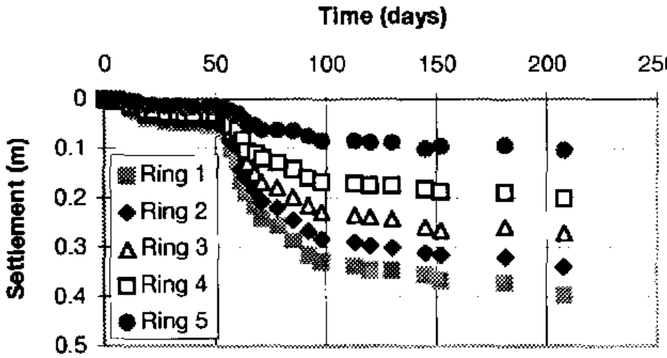

Figure 5. Layered settlement in tailings from LS1.

Pore Pressure Responses

Two types of pore pressure monitoring instruments were installed, namely electronic vibrating wire transducers and modified standpipe piezometers that were specially developed for soft tailings by CSIRO. The results from these two types of pore pressure sensors were very similar. The pore pressure responses presented in Figure 6 were obtained from the vibrating wire piezometers installed in the tailings profile at the centre of the instrumentation area. Pore water pressures responded quickly to external loads, particularly within the top 3m of the original tailings surface.

Three standpipe piezometers were installed to monitor possible tailings liquefaction induced by the bulldozer vibration. It was found, during construction, that tailings were totally liquefied in certain areas due to this vibration. The induced pore pressures at a depth of 0.5m were around 1.5m above the values estimated to result from the fill and static bulldozer loads. The bulldozer was noticeably 'waving' on the capping and 'tension' cracks were clearly visible on the surface of the adjoining depressions formed by earlier bulldozer tracks.

Displacement

Two layered lateral displacement stations, namely I1 and I2 were installed in the trial site, located along the southern border of the trial, one at the western end and the other about midway, as shown in Figure 3. The lateral displacements down the profile developed at I1 are shown in Figure 7. The lateral displacement can also be presented in a form of historic development for the points at different depths, as shown in Figure 8.

The maximum lateral displacement occurred at a depth of about 4 metres, which is consistent with the predictions from FEM analysis and shallower than those predicted, 6m deep, by the equilibrium analyses (Sheng et al. 1997). From the results obtained, it can be seen that the presence of the HTSG curtailed lateral movement occurring outwards and thereby prevented the tailings near the surface from getting squeezed sideways by the surcharge loads. Consequently, with the development of consolidation and settlement in tailings, the combined effects of the displacement caused the surface of tailings to move towards the capping trial.

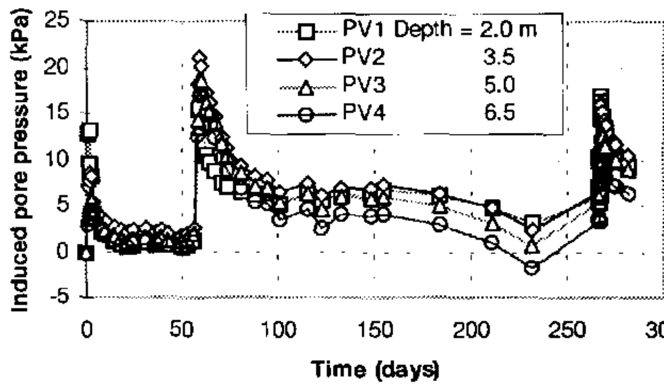

Figure 6. Pore pressure responses to loads.

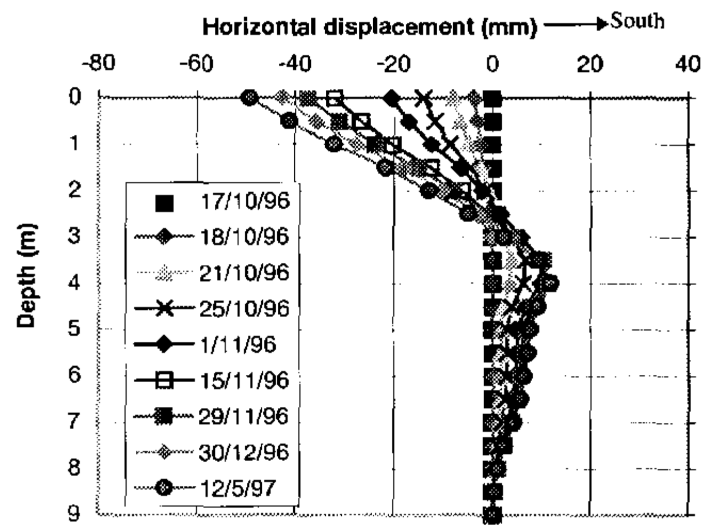

Figure 7 Lateral displacement at the toe of the capping trial

BACK ANALYSES OF TAILINGS PROPERTIES

Consolidation in Tailings

The predicted settlements due to consolidation under the first lift of waste rock were 0.52m at the centre, 0.32m at the middle of the edges and 0.18m at the corners of the capping trial. The predicted values agree well with those observed as shown in Figure 4. Back analysis, using layered settlement data in Figure 5, gives a coefficient of consolidation of 6.3×10^{-6} m^2/s which is greater than the average value, 3×10^{-7} m^2/s, obtained from laboratory data. The back-analysis using pore pressure records, shown in Figure 6, gave an average coefficient of consolidation of 4.2×10^{-7}, which was close to the laboratory value. The diversity of the coefficient of consolidation from various sources may be attributed to the strong layered property of the tailings.

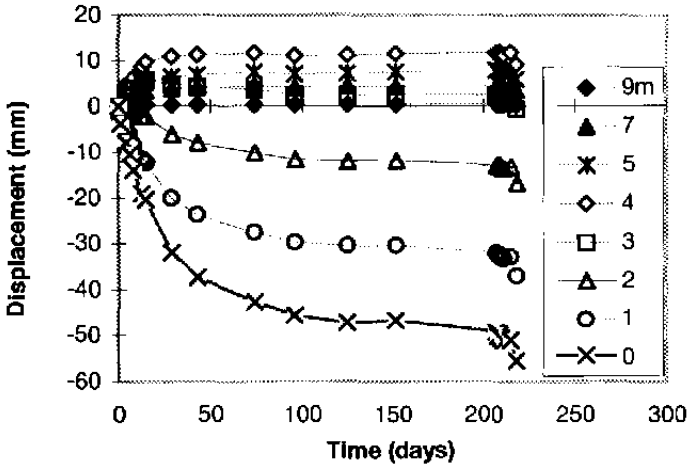

Figure 8 Development of lateral displacement down the tailings profile at I1 with time.

Evaluation of Stability

In situ monitoring has verified that the reinforced foundation provided sufficient bearing capacity to support the capping. The overall stability was assured during and after construction, except for local temporary failure due to liquefaction induced by machinery vibration and slight overloading at the initial stage. There was an amount of waste rock, that was used for the second lift of the trial, trucked into the eastern area of the capping trial, before it was spread to the western area. This temporary loading did not appear to cause notable settlement in tailings under the capping as shown by the surface settlement station SS1 in Figure 4. Therefore, the stability related to the capping is crucial at the initial stage of construction.

Based on the monitored data and experience gained during the construction, it seems that the design theories currently used for soft clays, when used on soft tailings, tend to give conservative results. Although an insufficient factor of safety was predicted, overall bearing failure did not occur. This indicates that the actual bearing capacity factor, N_c, for the Ranger tailings appears greater than that for soft clays. Various factors may contribute to this. One possible reason could be attributed to less deformation occurring in the tailings than that would otherwise occur in soft clays under the same loading conditions.

CONCLUSIONS

A stable cap system has been constructed over the extremely soft tailings in the tailings dam at the Ranger mine. Results obtained from continuous *in situ* monitoring of the capping trial system testify that the tailings strength is improved. The measured deformations agreed well with those predicted. The results obtained from this project indicate that the use of the geotextile may reduce the depth of the slip surface predicted by equilibrium methods.

It appears that the recommended factor of safety of 1.3 (US FHA, 1992, Koerner, 1994) tends to give conservative results in the stability analyses against rotational and bearing failure for the geotextile-reinforced Ranger tailings, if the Bishop approach is adopted in an analysis. The Janbu approach seems to be the most suitable for stability analysis of the reinforced tailings, with a horizontally laid geotextile, for the case presented.

Vibration induced liquefaction is a serious problem on extremely soft tailings, especially at the initial stage of the capping construction, when heavy duty earth moving equipment is being used. Effective design methods against construction induced liquefaction are yet to be developed.

ACKNOWLEDGMENTS

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