*The paper was published in the proceedings of the 8th Australia New Zealand Conference on Geomechanics and was edited by Nihal Vitharana and Randal Colman. The conference was held in Hobart, Tasmania, Australia, 15 – 17 February 1999.*

# Assessment of Liquefaction Flow Stability of a Thickened Tailings Stack

**K.D. Seddon**
B.E., M.Sc., MIEAust.
Senior Geotechnical Engineer, M.P.A. Williams & Associates Pty Ltd, Australia
**S.D. Murphy**
B.E., M.Eng.Sc., MIEAust.
Senior Geotechnical Engineer, M.P.A. Williams & Associates Pty Ltd, Australia
**M.P.A. Williams**
B.Sc., FIEAust.
Principal, M.P.A. Williams & Associates Pty Ltd, Australia

**Summary** A down-valley thickened discharge scheme of tailings disposal has been adopted for the Century mine in Queensland. As a result of this system, considerable quantities of tailings will be deposited above the crest level of the toe embankment. This paper covers the evaluation of liquefaction flow stability of the deposit. The analysis showed that liquefaction flow sliding is possible in the early years of mine life. The velocity of potential slides was calculated, and the retaining toe embankment is to be built to a height to contain any slide deposits. At later times some liquefaction is possible in parts of the deposits, but flow liquefaction instability does not occur. Further in-situ and laboratory testing will be carried out on the production material to confirm these estimates.

## 1. INTRODUCTION

The thickened discharge method of mine tailings disposal was developed in Canada (Robinsky, 1978). The method was first introduced into Australia in the 1980s and has had increasing acceptance (Williams and Seddon, 1999).

The essentials of the system are that prior to discharge the tailings slurry is thickened to a density where slurry is non-segregating, resulting in a constant beach slope angle (*not* a concave profile as is obtained from a segregating slurry). If discharged from a central point (Central Thickened Discharge, or CTD) the tailings fan out to form a low conical mound. Depending on the landform, little or no retaining embankments are necessary, and very efficient disposal schemes can be achieved.

In other cases, topography may dictate discharge from a hillside, or down a valley, in which case some segment of the full conical shape results.

A common feature of all of these schemes is that it is possible to configure a tailings storage so that a large proportion of the deposited material is unconfined, or is at an elevation above the crest of any confining embankment.

The risk of liquefaction flow sliding (particularly following seismic events) then becomes an important design consideration. Similar design considerations of course also apply to tailings storages constructed by the upstream raising method.

## 2. PROJECT OUTLINE

The Century zinc mine is located in north-west Queensland. The current study was based on a mine life of 20 years, and a tailings production rate of just under 4.2 Mtpa.

Extensive options studies were carried out to select the tailings disposal system. The adopted scheme is single point, down-valley discharge of thickened tailings. The layout is shown in Figure 1.

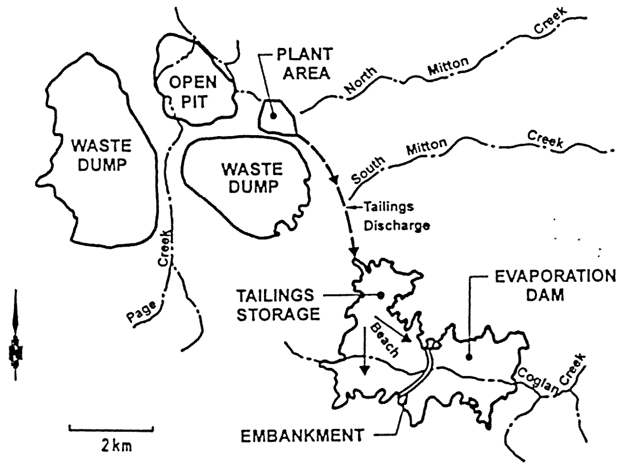


Figure 1. Site layout.

The tailings will be discharged at a slurry density of around 40% solids, and the estimated beach slope is around 0.5%. This is relatively flat, and steeper beach slopes and a more efficient containment system could be obtained at higher slurry densities. However, overall project economics limit the slurry density and beach slope to those stated above.

A toe retaining embankment is required to limit the down-valley extent of the tailings beach. This embankment will be constructed in stages, as is typically the case for tailings storages.

## 3. STABILITY OF THICKENED TAILINGS STORAGES

Two previous studies of the stability of thickened tailings storages have been published. Both were for storages with much steeper beach slopes than apply at Century.

Poulos, Robinsky and Keller (1985) report the results of an evaluation of the liquefaction resistance of an aluminium "red-mud" tailings stack. The material was extremely fine grained, with a $D_{50}$ of only 2μm, and a USC classification of ML (LL=48, PI=13). The design slope for the deposit was 2.9°, or 5%, which is steep by Australian standards. Steady state undrained strength $s_{us}$ was measured by vane shear tests. The dimensionless strength $s_{us}/p_o'$ was found to be around 0.05 at low confining pressures, reducing to about 0.02 at depth. The factor of safety against liquefaction flow sliding was calculated to be 0.26.

Cyclic load triaxial tests were carried out on normally consolidated samples of tailings, and the cyclic resistance determined. These results showed that for the site design earthquake (equivalent to 10 cycles at peak ground acceleration of 0.1 g) the cyclic strength was about 3 times the earthquake induced shear stresses. Therefore, liquefaction was not expected to occur.

The Argyle Diamond mine tailings (McMahon et al., 1996) are at the other end of the mine waste spectrum. The material is essentially a gravelly silty sand, with a $D_{50}$ about 1000 μm. Because of the coarse grain size, some segregation had occurred. However, the central part of the beach had a slope of about 10%, and was saturated. It has been calculated that the rate of rise of this stack was of the order of 10 m/year; consequently it is not surprising that the beach area was saturated and had little or no penetration resistance.

At Argyle the evaluation of resistance to liquefaction was based on in-situ testing using a dynamic cone penetrometer; the results were converted to equivalent SPT values and hence to liquefaction resistance using established correlations. This is a procedure which is only applicable to an operating mine, and a relatively coarse grained tailings product. A significant risk of liquefaction flow sliding was found to exist, and the design of the facility was extensively modified.

## 4. SEISMICITY OF THE SITE

A seismic risk analysis of the site was carried out by the Royal Melbourne Institute of Technology Seismology Research Centre (RMIT, 1995).

The analysis was carried out using probabilistic methods, and results included plots of acceleration and magnitude as a function of recurrence interval. The details are summarised in Table 1. In accordance with current earthquake engineering practice, two separate design levels were adopted: Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE).

Table 1. Summary of seismic design criteria.

| Event | A.R.I. (Years) | Magnitude ($M_L$) | $a_{max}$* | No. Cycles |
|---|---|---|---|---|
| OBE | 500 | 4.5 | 0.06g | 2 - 4 |
| MDE | 10,000 | 6.25 | 0.21g | 5 - 8 |

* $a_{max}$ = Peak ground acceleration

## 5. EVALUATION OF LIQUEFACTION

### 5.1 Tailings Density Profiles

Unlike natural soil deposits, the density profile in a tailings storage is variable as a function of position, time and rate of deposition. For this project, density profiles at varying points down the beach were calculated using finite strain consolidation techniques (Murphy and Williams, 1990).

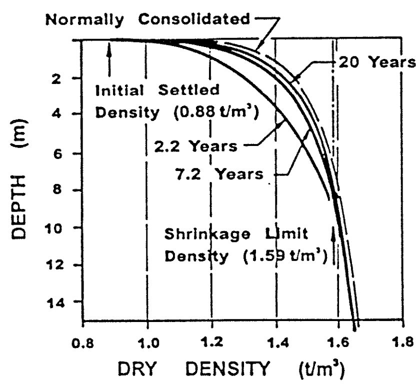


Figure 2. Typical density profiles.

The calculated density profile at a typical point on the deposit is shown in Figure 2 for a range of mine life times between 2.2 and 20 years. The normally consolidated curve is also shown. It is apparent that at 2.2 Years the deposit is substantially underconsolidated. However, from Year 7 onwards the profile progressively approaches normal consolidation.

The effects of evaporative drying have to be added to this profile. The shrinkage limit density for the tailings was measured at 1.59 t/m³. Water balance calculations taking into account the initial slurry density, the rate of evaporative drying, and the rate of upwards percolation of consolidation water, indicated that the shrinkage limit density would be achieved from about Year 2 onwards.

The exception to this was the decant pond area at the toe of the beach, which was considered to be permanently wet (see Figure 4).

Consolidation testing of the tailings slurry has indicated that the shrinkage limit density is equivalent to an overburden pressure of 70 kPa. Therefore, in beach areas subject to evaporative drying, the tailings will be over-consolidated due to evaporative drying to a depth of between 5 m and 12 m depending on water table, and normally consolidated below this.

## 5.2 Cyclic Strength

For the purposes of the initial design assessment, published cyclic strength data on other tailings materials were utilised. In particular, results by Ishihara et al. (1980) contain the results of detailed cyclic triaxial testing on tailings material from sources in Chile and Japan. Test results are presented for five separate low plasticity "slimes" or fine fractions. These materials have grading and plasticity characteristics similar to Century tailings, as summarised in Table 2. It is reasonable to assume that cyclic strength results will also be similar.

Table 2. Comparison of material characteristics.

| Test | Tested Samples (Ishihara et al.) | Century Tailings |
|---|---|---|
| Particle Size<br>-75μm %<br>-10μm % | <br>85-100<br>25-40 | <br>86<br>41 |
| Atterberg Limits<br>LL<br>PI | <br>24-34<br>non plastic - 13 | <br>23<br>7 |
| Soil Particle Density (G) | 2.64-2.88 | 2.79 |

For natural sand deposits, and sand tailings, it is common to express in-situ density in terms of relative density (density index AS 1289.5.5.1). However, this test is limited to materials containing no more than 12% fines. The test results lack all relevance to fine grained tailings materials. Ishihara et al. (1980) consequently presented their data in terms of void ratio. The results are shown in Figure 3. It should be noted that these points relate to a cumulative double amplitude strain of 5% at 20 cycles. The use of 5% strain to define liquefaction is a convention carried over from testing of sands, where it is a relatively good indicator of the onset of liquefaction. For fine grained materials it is considered to be conservative. The data presented by Ishihara et al. also include values of 10% cumulative strain, and "initial liquefaction" ($u = \sigma_{30}'$). For plastic slimes these are typically higher than the 5% strain value. The data in Figure 3 show only the 5% strain results. The best fit curves drawn by Ishihara et al. for high and low plasticity tailings are also shown. One additional point from the results of Poulos et al. (1985) has been added to the data, and shows reasonable agreement.

For the purpose of the subject investigation, the average curves for low plasticity material were adopted. The laboratory results were then corrected as follows:

$$CRR_{F,N} = CRR_{L,20} \times C_F \times C_N \qquad (1)$$

where

$CRR_{F,N}$ = cyclic resistance ratio for field conditions, and N cycles of shaking

$CRR_{L,20}$ = cyclic resistance ratio from laboratory triaxial tests for 20 cycles

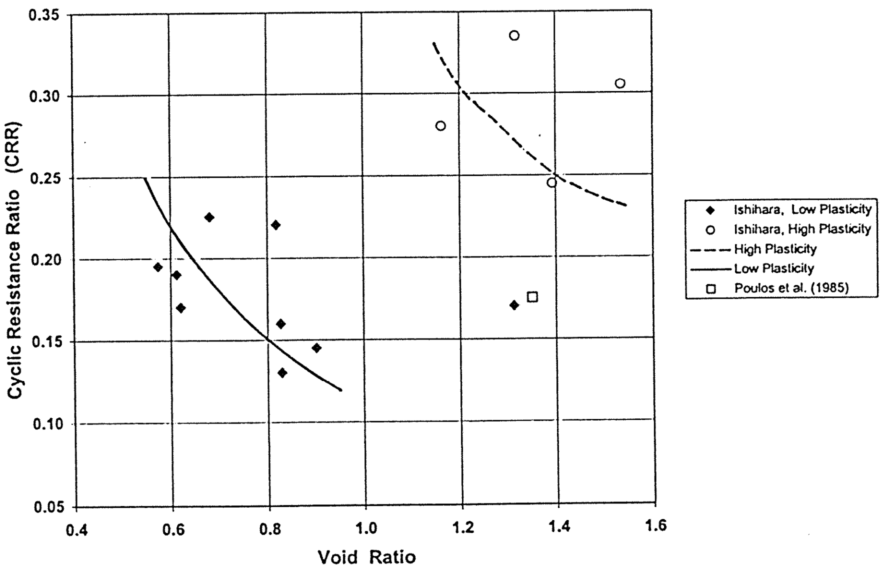


Figure 3. Results of cyclic triaxial tests.
(data from Ishihara et al., 1980; and Poulos et al., 1985)

$C_F$ = correction factor for triaxial to field results (Seed and Idriss, 1971)

$C_N$ = correction factor due to reduced number of cycles for lower magnitude earthquakes (Seed, Idriss and Arango, 1983).

For this project the following values were used for correction factors:

$C_F$ = 0.6
$C_N$ = 1.9 (MDE)
$C_N$ = 3.0 (OBE)

### 5.3 Magnitude of Induced Stresses

Induced stresses due to earthquake shaking were calculated from the equation

$$\frac{\tau_c}{\sigma_0'} = 0.65 \times \frac{a_{max}}{g} \times \frac{\sigma_0}{\sigma_0'} \times r_d \qquad (2)$$

(Seed and Idriss, 1971)

where

$a_{max}$ = peak ground acceleration
$g$ = acceleration due to gravity
$\sigma_0$ = total overburden stress
$\sigma_0'$ = effective overburden stress
$r_d$ = a stress reduction factor dependent on depth (and typically in the range 0.9 - 1.0 for the critical depths at the Century site).

### 5.4 Extent of Liquefaction

The factor of safety against cyclic liquefaction was defined as

$$FS_{CL} = \frac{CRR_{F,N}}{\tau_c/\sigma_0'} \qquad (3)$$

where

$CRR_{F,N}$ was obtained from (1) and $\tau_c/\sigma_0'$ from (2).

In the calculation, it was assumed that the phreatic surface in the tailings deposit was horizontal, at the level of the decant pond at the toe of the stack. However, the tailings above the phreatic surface were also assumed to be saturated and potentially liquefiable. The calculation was carried out for different times and locations throughout the tailings stack (each having a different density profile as discussed above), and for different magnitudes of earthquake equivalent to the OBE and the MDE.

The results of the calculation are summarised as follows:-

OBE: A few metres of liquefaction is possible near the toe of the deposit up to Year 2. No liquefaction is predicted in other areas and at later times.

MDE: Extensive areas of liquefaction are predicted in the initial years of mine life. The extent of liquefaction reduces with time but some liquefactional areas are present throughout the mine life.

Typical results for the MDE at Year 20 are shown in Figure 4.

## 6. POST LIQUEFACTION STABILITY ANALYSIS

### 6.1 Strength Assessment

Post liquefaction strengths were based on the concept of undrained, steady state strength (Poulos, Castro and France, 1985).

At the time of this study, only tailings from a pilot plant stage were available, and in-situ testing was not possible. Strength testing which was carried out included:-

- Vane shear tests (peak and residual)
- Isotropically consolidated, undrained triaxial tests, with pore pressure measurement
- Anisotropically consolidated, undrained triaxial tests with pore pressure measurement.

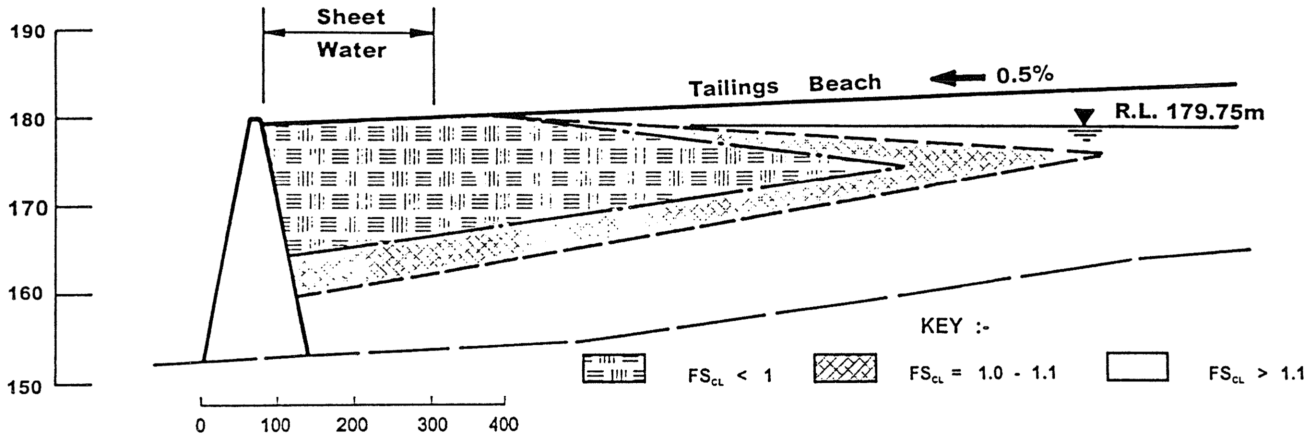


Figure 4. Extent of liquefaction (MDE @ T = 20 Years).

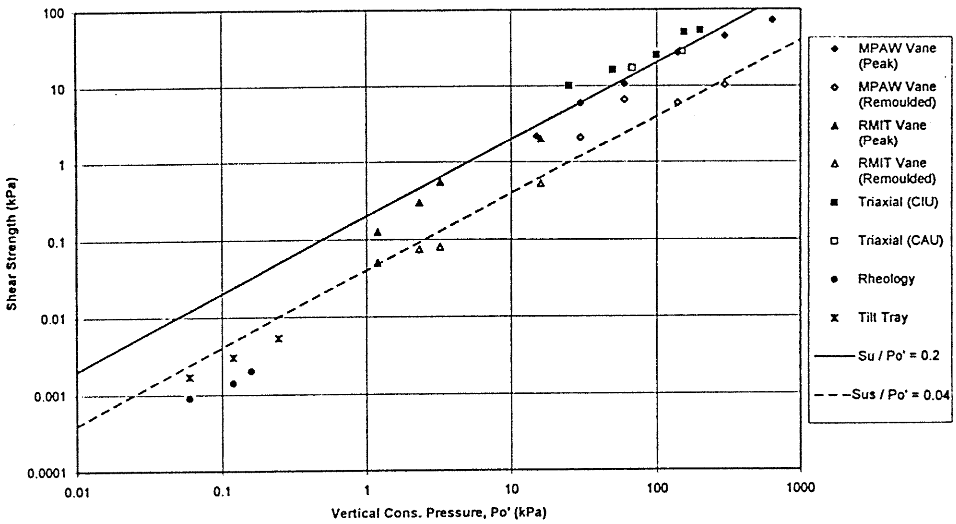


Figure 5. Shear strength test results.

The triaxial tests confirmed the generally contractive (i.e. potentially susceptible to liquefaction) nature of the material. However, the results also showed that large strains, well in excess of 10% would be required to generate the necessary pore pressure increase to initiate liquefaction.

The results of strength testing are plotted in Figure 5 These show a peak strength ratio of $s_u/p_o' = 0.2$ or greater, and a residual or steady state strength ratio $s_{us}/p_o' = 0.04$.

The method proposed by Stark and Mesri (1993) for evaluation of $s_{us}$ was used as a check. This gave a value of $s_{us}/p_o' = 0.055$, which was considered to be good agreement.

Consequently, post seismic stability analyses were carried out using the following strength ratio values for the relevant tailings zones:

| | | |
|---|---|---|
| where $FS_{CL} > 1.1$ | Peak: | $s_u/p_o' = 0.2$ |
| where $FS_{CL} \leq 1.1$ | Steady State: | $s_{us}/p_o' = 0.04$ |

## 6.2 Stability

Stability analyses were carried out using conventional limit equilibrium techniques. Analyses were carried out for soil strength profiles corresponding to times t = 2.2, 7.2 and 20 years. The results were as follows:-

OBE:
At time = 2.2 years : Limited failures indicated
At time = 7.2 years+ : No liquefaction and no failure

MDE:
At time = 2.2 years : Flow slide failure possible to depths of about 4 m.
At time = 7.2 years+: No flow sliding failure will develop (despite the presence of some liquefied zones in the deposit).

Progressive drainage and densification of the deposit is expected after mine closure, and liquefaction sliding under these conditions is not indicated.

## 7. FLOW SLIDE VELOCITY

### 7.1 Background

Some documented cases of liquefaction have resulted in relatively high velocity flow slides (Jeyapalan et al., 1983b). In these cases the energy of the sliding mass may be such that additional freeboard is required to provide containment. In the case of Century, with a slope angle of only 0.5% high flows velocities are intuitively unlikely. Nevertheless, an engineering estimate was required so that the degree of security of containment would be evaluated.

Slurried tailings, including liquefied tailings in a flow slide can be considered to act as a Bingham plastic fluid (Jeyapalan et al., 1983a). Two parameters are used to characterize Bingham fluids: yield stress ($\tau_y$) and plastic viscosity ($\eta_p$). These quantities are defined in Figure 6. They are dependent on the actual density or solids content of the tailings slurry.

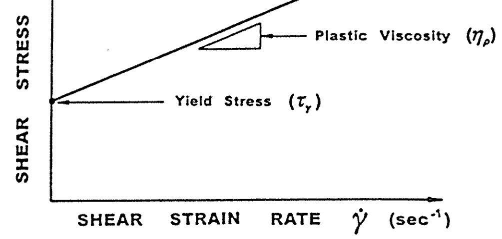


Figure 6. Definition of rheological parameters.

A design method is available for estimating runout distance of liquefied tailings following a breach or failure of an embankment (Jeyapalan et al., 1983a, 1983b), but no method applicable to beached tailings could be located. It was therefore necessary to develop a new theoretical solution.

## 7.2 Theory

For a liquefaction flow down a sloping beach, the driving force due to gravity at any depth is $\sigma_v \sin \beta$, and the resistance of flow is $\tau_y + \eta_p \gamma$,

where

$\sigma_v$ = total vertical stress due to weight of tailings

$\beta$ = beach slope angle

$\tau_y$ = yield stress

$\eta_p$ = plastic viscosity, and

$\dot{\gamma}$ = shear strain rate.

At some flow velocity these force components will be equal, and for a semi-infinite slope essentially steady flow conditions will result.

At this flow velocity

$$\sigma_v \sin \beta = \tau_y + \eta_p \dot{\gamma} \qquad (4)$$

Hence

$$\dot{\gamma} = (\sigma_v \sin \beta - \tau_y)/\eta_p \qquad (5)$$

The rate of shear strain $\dot{\gamma}$ depends on the distribution of velocity throughout the flow profile. Boundary layer theory suggests that over the distances applicable in a flow slide a parabolic velocity distribution over the full depth of the flowing mass is appropriate. In this case, the maximum shear strain rate can be approximated by:

$$\dot{\gamma} = \frac{1.5 V_{max}}{h} \qquad (6)$$

where

$V_{max}$ = maximum equilibrium flow velocity (at the surface), and

$h$ = depth of flow.

Combining (5) and (6) gives the required expression for maximum equilibrium flow velocity:

$$V_{max} = (\sigma_v \sin \beta - \tau_y)h/1.5\,\eta_p \qquad (7)$$

## 7.3 Rheological Parameters

At time t = 2.2 years, the tailings' dry density in the part of the beach which is subject to liquefaction ranges between around 1.15 $t/m^3$ and 1.4 $t/m^3$, as shown in Figure 2. This density range is equivalent to a solids content in the range 66 to 74%.

Rheological testing was carried out at four nominal solids contents, 66, 69, 71 and 74%, to cover this range. Testing was undertaken by RMIT Rheology and Materials Processing Centre.

At each nominal solids contents, shear vane testing was carried out at a range of rotational speeds equivalent to a shear strain rate varying between 0.01 and 1.0 $sec^{-1}$. Tests were continued to a post-peak equilibrium value, and the required Bingham fluid parameters were obtained from the resulting plot of equilibrium shear stress against shear strain rate. The results of the testing are shown in Table 3, and the strength values are also included in Figure 5.

## 7.4 Equilibrium Flow Slide Velocity

The results of the calculation of equilibrium flow slide velocity using (6), for a beach slope of 0.5%, are summarised in Table 3. It will be seen from the table that because of the high solids content and strength of the sample tested at 75.6% solids, flow sliding does not occur. Results were therefore interpolated for a surface at 74% solids.

The maximum velocity likely to occur in a flow slide is just over 1 m/sec. The velocity head at this flow rate is only 0.05 m. It therefore seems that apart from an allowance for horizontal containment of the tailings involved in the flow slide, there is no need for a further allowance for a "run-up" velocity, or for additional allowance of impact forces in the design of retaining toe embankments.

Table 3. Shear-vane/rheology test results and equilibrium flow slide velocity.

| | | | | | | |
|---|---|---|---|---|---|---|
| Solids Content | (%) | 67.1 | 69.8 | 71.0 | 74.0 | 75.6 |
| Peak Shear Stress | (kPa) | 0.13 | 0.30 | 0.55 | - | 2.0 |
| Yield Stress, $\tau_y$ | (Pa) | 50 | 75 | 80 | - | 530 |
| Plastic Viscosity, $\eta_p$ | (Pa.$sec^{-1}$) | 70 | 160 | 155 | - | 470 |
| Dry Density | ($t/m^3$) | 1.18 | 1.26 | 1.30 | 1.41 | 1.46 |
| Depth, h | (m) | 0.75 | 1.5 | 2.2 | 4.0 | 5.0 |
| Vertical Pressure, $\sigma_v$ | (kPa) | 13 | 26 | 39 | 72 | 90 |
| Maximum Equilibrium Velocity, $V_{max}$ (m/s) | | 0.1 | 0.33 | 1.05 | 0.53 | No failure |

## 8. DESIGN MODIFICATIONS AND FURTHER STUDIES

The design criteria adopted for the site have been to provide horizontal containment of all potential flow slide material up to the MDE event, at all times throughout the mine life.

Major changes to the scheduling of the staged raising of the toe embankment were not required. The initial toe embankment has been sized to provide containment for the first 6 years' production (at the design beach slope). This has a horizontal capacity equivalent to the first 3 years of production. It is proposed that following commissioning, and within the three-year period of horizontal con-tainment, further in-situ and laboratory testing should be carried out on the actual tailings product, to confirm:

- achieved density profiles,
- actual cyclic resistance ratios, and
- steady state undrained strength parameters.

The stability of the deposit would then be checked, and the raising schedule of the toe embankment would be modified if required.

## 9. CONCLUSIONS

The study has indicated that the Century thickened discharge tailings storage will only be subject to liquefaction flow instability in the early years of mine life, and under relatively infrequent earthquake loads. The increasing densities and strengths which result from evaporative drying effects preclude liquefaction flow slides at later stages.

The velocity of any flow slide is expected to be low, and no run-up or impact allowances are indicated.

The existing embankment staging provides for horizontal containment of potentially slumped material up to the end of Year 3 of mine life.

Additional testing will be carried out to confirm these predictions within that time period, and the subsequent stage raising modified if required.

## 10. ACKNOWLEDGEMENTS

The work described in this paper was carried out for Minenco-Bechtel on behalf of Century Zinc Pty Ltd (now Pasminco). However, the opinions expressed are those of the authors, and not an official statement of the views of Minenco-Bechtel or Pasminco.

## REFERENCES

Ishihara, K., Troncoso, J.H., Kawase, Y. and Takahashi, Y. (1980). Cyclic Strength Characteristics of Tailings Materials. *Soils and Foundations*, Vol. 20, No. 4.

Jeyapalan, J.K., Duncan, J.M., and Seed, H.B. (1983a). Analysis of Flow Failures of Mine Tailings Dams. *ASCE Journal of Geotechnical Engineering*, Vol. 109, No. 2, pp. 150-171.

Jeyapalan, J.K., Duncan, J.M., and Seed, H.B. (1983b). Investigation of Flow Failures of Tailings Dams. *ASCE Journal of Geotechnical Engineering*, Vol. 109, No. 2, pp. 172-189.

McMahon, B., Phillips, J.T., and Hutton W.A. (1996). The Evaluation of Thickened Tailings in a Seismic Area. *Tailings and Mine Waste 96, Fort Collins, Colorado, USA;* A.A. Balkema.

Murphy, S.D. and Williams, M.P.A. (1990). The Application of Finite Strain Consolidation Theory to the Design of Tailings Disposal Systems. *Proceedings of the ICOLD International Symposium on Safety and Rehabilitation of Tailings Dams,* Sydney.

Poulos, S.J., Castro, G. and France, J.W. (1985). Liquefaction Evaluation Procedure. *ASCE Journal of Geotechnical Engineering*, Vol. 111, No. 6.

Poulos, S.J., Robinsky, E.I. and Keller, T.O. (1985). Liquefaction Resistance of Thickened Tailings. *ASCE Journal of Geotechnical Engineering,* Vol. 111, No. 12.

RMIT Seismology Research Centre (1995). Review of Seismicity, Century Project.

Robinsky, E.I. (1978). Tailings Disposal by the Thickened Discharge Method for Improved Economy and Environmental Control. *Proceedings of the 2nd International Tailings Symposium,* Denver, Colorado.

Seed, H.B. and Idriss, I.M. (1971). Simplified Procedure for Evaluating Soil Liquefaction Potential. *ASCE Journal of Geotechnical Engineering,* Vol. 97, No. 9.

Seed, H.B., Idriss, I.M. and Arango, I. (1983). Evaluation of Liquefaction Potential Using Field Performance Data. *ASCE Journal of Geotechnical Engineering,* Vol. 109, No. 3.

Stark, T.D. and Mesri, G. (1993). Undrained Shear Strength of Liquefied Sands for Stability Analysis *ASCE Journal of Geotechnical Engineering,* Vol. 118, No. 11.

Williams, M.P.A. and Seddon, K.D. (1999). Thickened Discharge: A Review of Australian Experience. *Tailings and Mine Waste 99, Fort Collins, Colorado, USA*; A.A. Balkema,.