Post-Liquefaction Stability of Thickened Tailings Beaches

K.D. Seddon *Australian Tailings Consultants, Australia*

ABSTRACT

In considering liquefaction stability, a distinction must be made between failures of cycloned sand tailings embankments (essentially silty sands), and the failure of "all in" tailings beaches. Historical data provide broad support for the theory that liquefied soils do not reduce to a state of zero shear strength, but rather retain a non-zero, "post-liquefaction" shear strength.

For a tailings beach, stability and/or cyclic mobility during a shaking event is not considered to be a major issue because of the limited consequences, provided flow sliding does not subsequently develop. The analysis of post-liquefaction stability is therefore critical.

The stability of tailings beaches can be analysed using relatively simple equations obtained from infinite slope theory. The key questions are:

- *What are the stress / density conditions in the slope prior to failure?*

- *What values of post-liquefaction shear strength are appropriate?*

Shear strength data obtained from both vane shear tests and triaxial tests on tailings are presented, and appear to give reasonable agreement with limited previous data available from the literature.

1 INTRODUCTION

A fundamental aspect of thickened tailings disposal is that a tailings beach forms at a small but finite positive slope angle.

The influence of this slope angle is to some extent scale dependant. For a small storage (< 1 km in length), a slope of say 2% results in a height differential between the crest of the retaining embankment and the head of the beach of less than 20 m, and the quantity of "unrestrained" tailings is modest. However, for a large storage (e.g. length = 10 km) the height differential will be of the order of 200 m. In this instance the quantity of "unrestrained" tailings becomes substantial.

The stability of these slopes must therefore be addressed in the context of liquefaction flow following seismic events.

2 HISTORICAL BACKGROUND

Most of the historical data relating to seismic failures of tailings dams originates from Chile. In the last 100 years, there have been at least 5 separate earthquakes which have resulted in liquefaction failure of tailings dams in that country (Troncoso, 2000).

However, it is necessary to make a clear distinction between the case of stability of "conventional" tailings dams, and thickened tailings beaches. Most of the failed storages were formed by the "upstream method", and it is likely that the embankments were constructed using a sand fraction obtained by cycloning, and not "all in" tailings. It is probable that none of the embankments were compacted. In geotechnical terms, this material is loose, fine grained, silty sand, which is probably the material most susceptible to liquefaction. In addition, the method of construction of these dams results in much steeper downstream slopes – typically between 1.5:1 (66%) to 4:1 (25%), far steeper than the beach slope angles which apply for thickened tailings disposal.

However, in some cases, once the retaining structures breached, it is clear that a proportion of the retained tailings (or slimes) also liquefied and flowed. The typical mechanism of failure can be summarised (Dobry and Alvarez, 1967; Troncoso et al., 1993), as follows:

- Failure and collapse of the embankment slope.

- Escape of retained water through the breach, resulting in further erosion.

- Limited liquefaction and flow of the retained slimes.

This last point is important. It indicates that at steep slopes (i.e. behind a sand embankment which has just failed) tailings slimes may liquefy and flow. However, it is also clear that all of the tailings do not reduce to a condition of zero strength and flow out of the storage like water. There appears to be a limiting slope angle below which flow failures do not propagate. This implies that there is also a limiting (minimum) shear strength.

3 ASPECTS OF SEISMIC STABILITY

Two separate cases may be considered with respect to stability during seismic events:

- Deformations due to accumulated strains or failure during the actual shaking event.

- Liquefaction and flow failure after the seismic event.

In the first case, the stability analysis must include the lateral loads due to the horizontal acceleration from seismic shaking. In the second case, the actual shaking is assumed to have ceased, but significantly reduced strength values may apply because of liquefaction. This is the case for "post liquefaction" stability analysis.

For flow liquefaction to occur in a slope, three basic conditions must be satisfied:

- The slope must be saturated (at least at the level of the failure surface).

- There must be a sufficiently severe initiating event to generate the necessary additional shear strain to induce liquefaction (typically a seismic event, but other causes are possible).

- The reduction in strength must reduce the stability of the slope to a failure condition (i.e. FS <1).

Other possible initiating mechanisms which have the potential to induce liquefaction include blasting or vibrations due to construction activities, changed stress conditions (placement of fill, or undercutting the toe), and an increase in piezometric pressure.

For some cases of thickened disposal schemes, a design stage water balance has predicted that as a result of drainage and evaporative drying the tailings in the beach will not be saturated. In at least two cases, subsequent in situ testing has confirmed this. In these circumstances any possibility of liquefaction can be discounted. However this is not necessarily always the case, and to provide a more general analysis it is assumed in this paper that the tailings slope is saturated, or at least that some layers within the slope are saturated.

In much the same way, all sites are subject to a degree of seismic activity, although the probability of a given magnitude of shaking may vary markedly depending on the site. This paper is based on the assumption that an initiating event will or has occurred.

Given these two qualifying assumptions, the question of seismic stability and flow liquefaction of a thickened tailings beach is covered in the following sections of this paper as follows:

- Section 4: Deformation and failure induced in the slope during seismic shaking.

- Section 5: The resistance of tailings to liquefaction.

- Section 6: Post-liquefaction shear strength and stability.

4 STABILITY DURING SHAKING

In the case of a tailings beach, the total cumulative strain (cyclic mobility) during the actual seismic event is not of prime importance. There are no structures on the beach that can be damaged. The consequences of movement during the earthquake are insignificant, provided liquefaction flow does not proceed after the event. It is therefore suggested that the following analysis should be viewed only as a screening approach, in that it identifies potential combinations of acceleration and pore-pressure response where temporary failure conditions may occur (and would therefore indicate further analysis).

The method was developed by Hadj-Hamou and Kavazanjian (1985). Their analysis is based on effective stress parameters, and considers stability primarily during earthquake shaking. It is for granular soils in general, but can be applied to slopes of tailings.

They derived an equation for the factor of safety of an infinite slope. For the case where the water table is at the surface this simplifies to the following:

$$FS = \left(1 - \frac{\gamma_w}{\gamma}\right) \cdot \frac{1-R}{K + \sin \beta} \cdot \cos \beta \tan \phi' \tag{1}$$

where:

FS = Factor of Safety.

γ_w = unit weight of water.

γ = total unit weight of the material.

K = acceleration (as a factor of g).

β = slope angle (degrees).

ϕ' = friction angle.

R = pore pressure ratio = $\Delta u / \sigma_o$.

(Δu = excess pore pressure, σ_o = total vertical stress).

For the critical case where $FS = 1.0$, combinations of R, K and slope angle which result in failure can be plotted. These results are shown in Figure 1, using a value of $\phi' = 37°$ for tailings, which is a typical value obtained from triaxial testing (refer to Section 6.2).

Figure 1 indicates that at the gentle slopes being considered here, values of acceleration (K) and the pore pressure generated (R) have much more influence than the actual slope itself.

The graph shows that if there is no excess pore pressure generated (i.e. $R = 0$), seismic acceleration of the order of 0.35 g is necessary to bring the slope to an instantaneous failure condition. On the other hand, high induced pore pressures ($R = 0.9$) require only low accelerations to reach a failure condition.

The issue of what is a realistic value of R to use in design is open to question. There are virtually no in situ records of pore pressure response in tailings to seismic shaking. The triaxial test results presented later in this paper (Section 6) suggest that values of $R = 0.3 - 0.6$ would not be unreasonable, for the "real" case of anisotropically consolidated tailings.

This then indicates that horizontal accelerations of the order of $0.1 - 0.25$ g would typically be sufficient to bring a slope to an instantaneous failure condition during shaking.
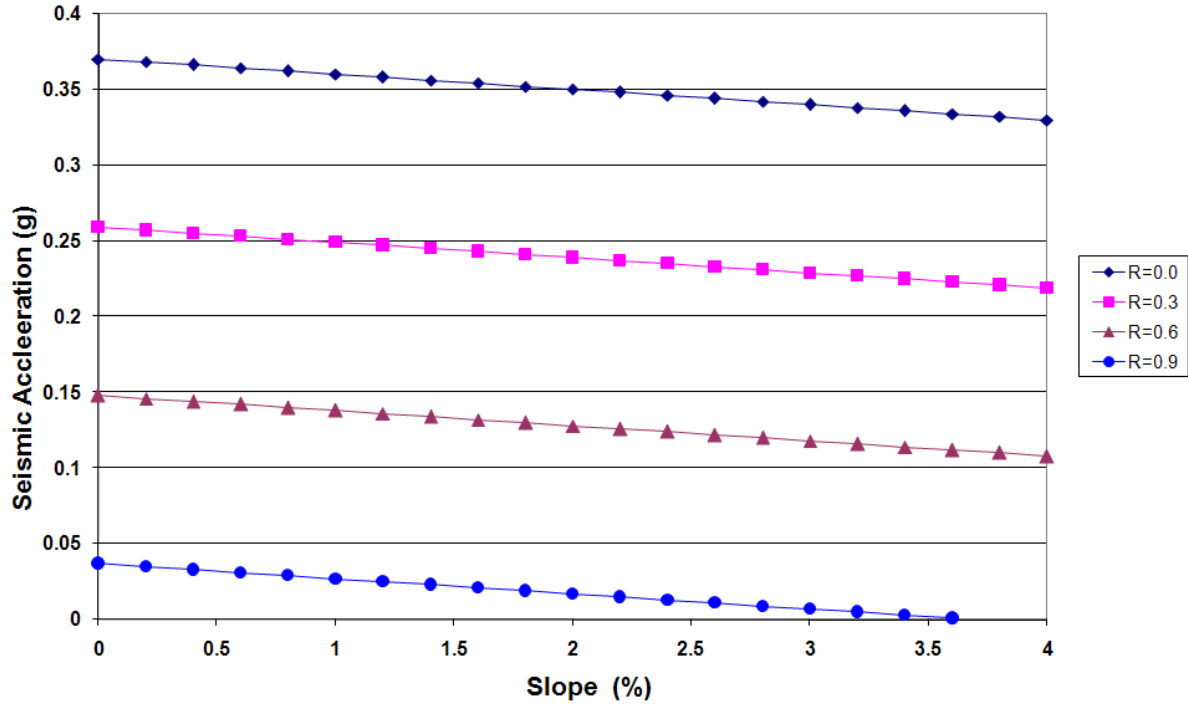
Figure 1   Critical slope angle versus acceleration and pore pressure response

5   INITIATION OF LIQUEFACTION

The possibility of liquefaction being initiated during seismic shaking can be analysed on the basis of cyclic triaxial tests results. Alternatively, it is possible to simply assume that the results of this analysis will confirm the initiation of liquefaction, and proceed to the analysis of flow liquefaction as discussed in the next section.

Limited data on cyclic strength parameters of tailings are available from the literature. Ishihara et al. (1980), include results for the cyclic resistance ratio (CRR) for both tailings sand, and slimes or "all in" tailings. For the purposes of thickened tailings beach slope stability, the results for "all in" tailings are relevant. The cyclic resistance ratio (CRR) is defined as:

$$CRR \quad = \quad \frac{\Delta\sigma}{2\sigma_{vo}{}'} \tag{2}$$

where:

$\Delta\sigma$ = cyclic deviator stress, and $\sigma_{vo}{}'$ = isotropic consolidation pressure.

Their results indicate CRR values of 0.15 for tailings at void ratios in the range 0.8-1.0, for 20 cycles of shaking. For typical 'hard rock' tailings ie tailings derived from crushing and grinding of primary ore, this void ratio is likely to be equivalent to confining pressure of 10-100 kPa (i.e. depths of 1-10 m).

Additional results are provided by Poulos et al., 1985b, and also Garga and McKay, 1984. These suggest values in the same range.

It should be noted that Ishihara et al. (1980) use a value of CRR measured at 5% cumulative strain, rather than at ultimate failure of the sample. This convention is widely used in the interpretation of cyclic load tests. It is derived from tests on clean or slightly silty sands, where liquefaction at strains less than 5% is common. The applicability of this criterion to the case of general liquefaction has been questioned (e.g. Poulos et al., 1985a). It is certainly the case that in all of the static (monotonically loaded) triaxial tests on "all-in" tailings which have been carried out by Australian Tailings Consultants (ATC) (refer Section 6), the strain to failure was 10-30%, i.e. well in excess of the 5% value. The analysis results based on these data

may therefore be conservative (i.e. the results underestimate the degree of seismic shaking required to induce liquefaction).

Using the results of cyclic triaxial tests, the factor of safety against liquefaction (FS_L) occurring under any given magnitude and duration of earthquake shaking can be defined as:

$$FS_L = \frac{CRR_{F,N}}{\tau_c / \sigma_{vo}'} \tag{3}$$

where:

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

τ_c $=$ the critical shear stress due to earthquake shaking.

This may be expanded (Seddon et al., 1999) to give

$$FS_L = \frac{C_F \times C_N \times CRR_{L,20}}{0.65 \times \dfrac{a_{max}}{g} \times \dfrac{\sigma_o}{\sigma_o'} \times rd} \tag{4}$$

where:

C_F $=$ correction factor for triaxial results to field values (= 0.6).

C_N $=$ correction factor for number of shaking cycles.

$CRR_{L,20}$ $=$ cyclic resistance ratio for 20 cycles, from the laboratory tests (taken as 0.15 after the results of Ishihara et al., 1980 as discussed previously).

a_{max} $=$ peak ground acceleration.

$\dfrac{\sigma_o}{\sigma_o'}$ $=$ ratio of total stress to effective stress.

rd $=$ depth based stress reduction factor (taken as 1.0 in this case).

The value of $\dfrac{\sigma_o}{\sigma_o'}$ is dependent on the bulk density of the deposited tailings layer, but for many common types of tailings a value of 2.0 has been found to be representative for the case of a saturated beach with water table at the surface.

For the critical case where $FS_L = 1.0$ (i.e. there is just sufficient shaking to bring the material to liquefaction), and adopting $C_N = 1.0$ (i.e. assuming a large earthquake with 15-20 significant cycles of shaking), substitution of the adopted values into the above equation gives:

$$\frac{a_{max}}{g} = 0.07 \tag{5}$$

This indicates that 20 cycles of shaking from a seismic event with peak ground acceleration $a_{max} = 0.07$ g can be expected to induce a liquefaction condition (or, strictly, that this level of shaking should generate 5% cumulative strain, which is assumed to be a liquefaction condition).

Consideration of this result provides some insight into the conditions where this type of analysis might be useful. In high seismic regions (e.g. Chile, Iran, etc.) it could be argued that this level of shaking would be almost inevitable within the design return period of the facility. In other words, it would not be unreasonable

to assume the incidence of liquefaction. In this event the better testing strategy would be to focus on determining the post liquefaction shear strength S_{us}, rather than the CRR.

6 POST LIQUEFACTION STABILITY

6.1 General

The term "liquefaction" can be misleading. It tends to imply that under certain conditions, a soil can literally "turn to water" and consequently "flow like water".

In the case of flat ground, or gentle slopes, this is not necessarily the case. The key factor to appreciate is in the liquefied state soils do not reduce to zero shear strength (i.e. they do not really behave like water). What happens in liquefaction is a reduction in shear strength, not a total loss of shear strength. Flow liquefaction will occur only if the reduction in strength reduces the factor of safety of the slope to less than 1. The measurement of post liquefaction shear strength is therefore of major importance to the assessment of flow liquefaction in tailings beach slopes. Liquefaction flow slides have generally been observed to occur over time periods of the order of seconds to minutes. Therefore in the following sections of this paper the relevant strength parameter will be taken to be the undrained strength.

6.2 Post Liquefaction Strength Assessment
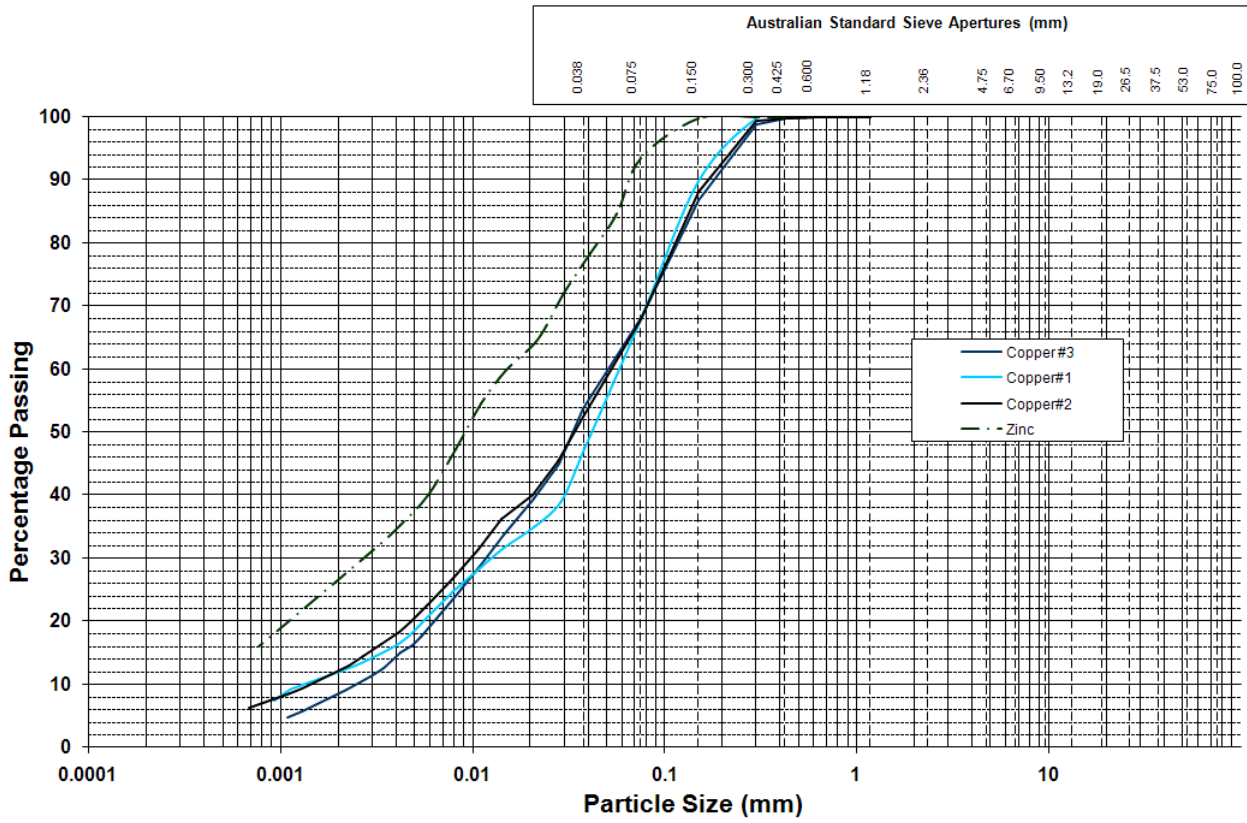
6.2.1 Triaxial test results

The post liquefaction (or steady state) shear strength of a soil is considered to be a function of the initial density/void ratio. Provided the amount of strain is enough to bring the sample to a post peak "steady state" condition, the actual stress path is not considered to be a factor (Poulos et al., 1985a). This means that information on steady-state shear strength can be obtained from static triaxial tests as well as from cyclic tests.

ATC has carried out triaxial strength testing in a number of different studies. Selected results from this testing are presented here, comprising samples of copper tailings, and a finer grained zinc tailings. The grading curves (particle size distribution) for these materials are presented in Figure 2. It is apparent that the copper tailings are very similar, with about 70% fines (material passing the 75μm sieve size). The zinc tailings are substantially finer (up to 95% fines), but the overall shape of the curve is similar. All of the samples are of low plasticity.

The method of testing was similar in each case. Samples were prepared either by moist tamping or consolidation from slurry, at a low confining pressure. The samples were set up in the triaxial cell, saturated, and then consolidated along an an-isotropic consolidation path (equivalent to K_o consolidation). This is the stress path the tailings would follow in the actual beach as they are deposited, and then progressively covered by additional tailings.

The samples were then tested in undrained shear, commencing from a wide range of initial stress conditions. The resulting stress-path plots for the tests on two of the copper tailings are shown in Figure 3, using the MIT stress convention p' = $(\sigma_1' + \sigma_3')/2$. (The results for the zinc tailings were similar, but are not shown for clarity).

It can be seen that during the undrained shear, all of the samples were initially strongly contractive. Some samples exhibited contractive-dilative behaviour (i.e. initially contractive, but changing to dilative behaviour once the failure condition was reached). Others remained contractive throughout. However, the most notable result from this testing is that few samples show any significant brittle behaviour (i.e. there is little difference between peak shear strength and steady state shear strength).

Figure 2   Particle size distribution curves for test samples
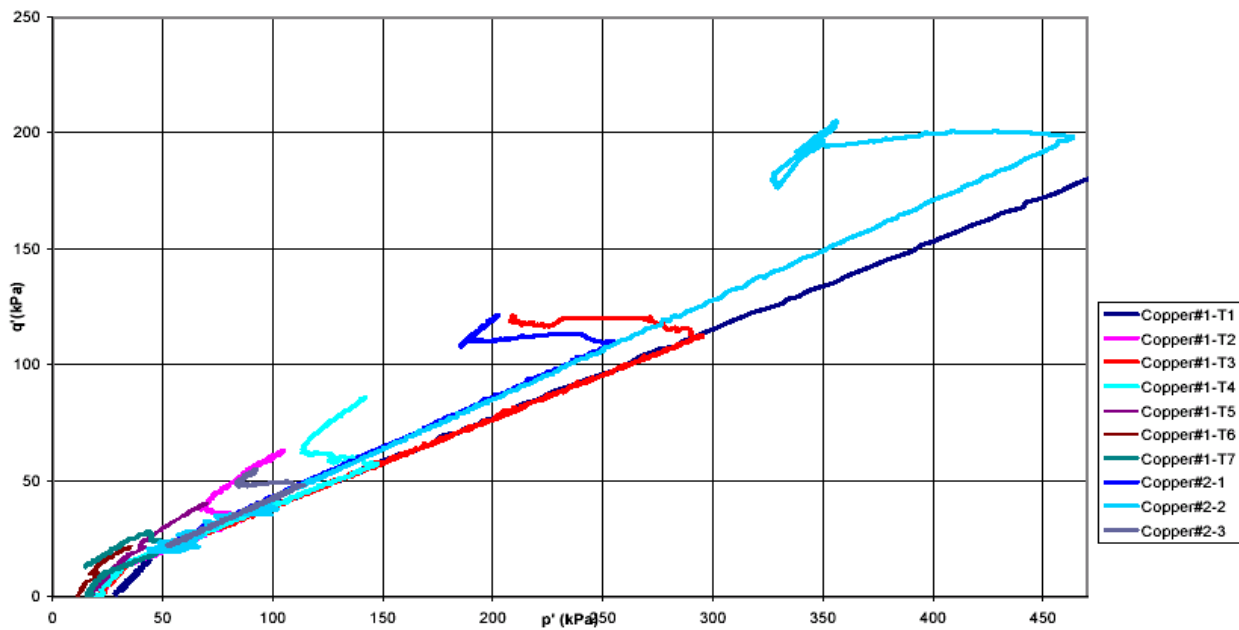
Figure 3   Stress path results from triaxial tests (copper tailings)
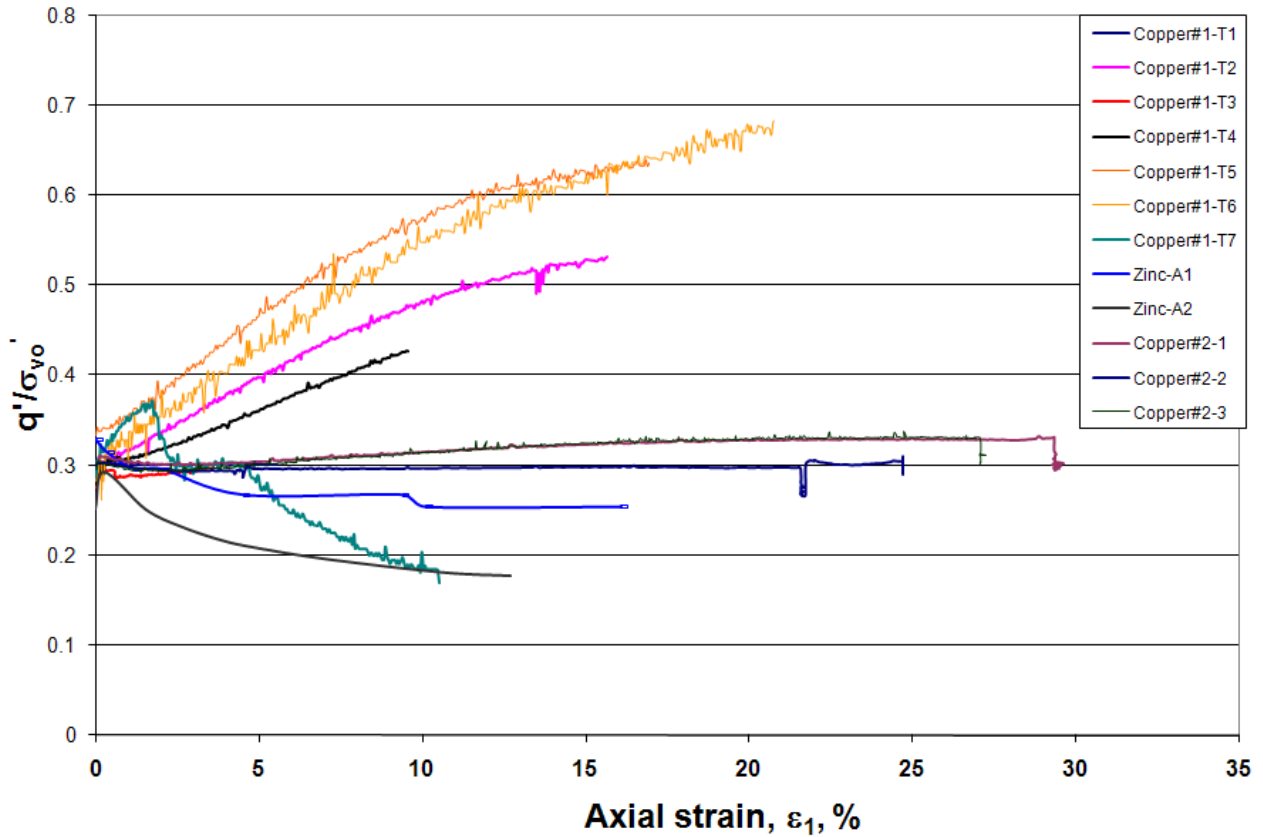
Figure 4   Normalised stress — strain results from triaxial tests

A normalised plot of the stress-strain curves of the undrained shear component of these tests is presented in Figure 4. This includes the results of the zinc tailings as well as the copper tailings. These results suggest a minimum or lower bound undrained shear strength ratio:

$$\frac{q}{\sigma_{vo}{}'}\left(=\frac{S_u}{\sigma_{vo}{}'}\right)=0.18-0.20 \tag{6}$$

The results indicate that samples showing strengths in this range were at a steady state condition (i.e. there would be no further change in shear strength with continuing shear strain). Some of the higher strength results had not reached a steady state at the end of the test, and were still increasing.

6.2.2   Vane shear results

It is also possible to use vane shear tests to measure peak and steady-state residual shear strength. Troncoso (2000) reports the results of laboratory tests on normally consolidated tailings from three Chilean copper tailings deposits:
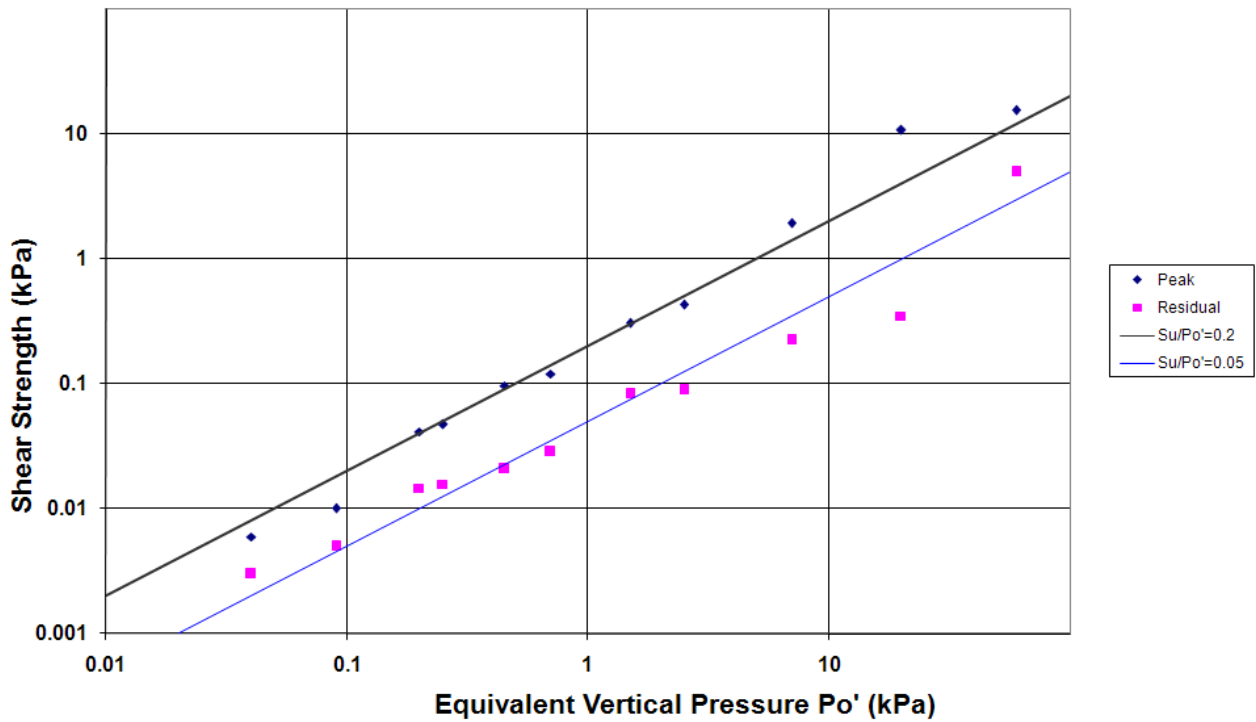
$$\text{Peak strength} \qquad \frac{S_u}{\sigma_{vo}{}'}=0.2-0.29 \tag{7}$$

$$\text{Steady state} \qquad \frac{S_{us}}{\sigma_{vo}{}'}=0.07-0.11 \tag{8}$$

ATC carried out vane shear tests on another Chilean tailings (Copper #3) as part of a recent study. These were carried out on a sample which was progressively dried back from an initial moisture content of 80% (about 53% solids) to around 30% (77% solids). The results of the shrinkage limit density test indicate that the sample remained substantially saturated over this range, and hence it is possible to convert moisture

content to an equivalent effective vertical confining stress for a normally consolidated sample. The vane shear results are presented in this manner in Figure 5.

Figure 5   Typical vane shear test results

It is apparent that the relationship:

$$\frac{S_u}{\sigma_{vo}'} = 0.2 \qquad \text{provides a reasonable fit for the peak strength results.}$$

$$\frac{S_{us}}{\sigma_{vo}'} = 0.05 \qquad \text{provides an approximate fit for the steady state results.}$$

There is hence reasonable agreement between these results and the data ranges reported by Troncoso (2000).

6.3   Stability Analysis

The results in the previous section indicate that post-liquefaction steady-state (residual) shear strength of tailings material is not constant, but is dependent on the initial density/void ratio, or alternatively, for normally consolidated tailings, on the vertical effective stress, σ_{vo}'.

It is therefore possible to adopt the form:

$$S_{us} = k \, \sigma_{vo}' \qquad\qquad (9)$$

where:

S_{us} =   undrained, steady state shear strength.

k =   a constant.

σ_{vo}' =   maximum vertical effective stress = $\gamma' z$.

(γ' = effective unit weight, z = depth).

The analysis presented below assumes that the tailings are normally consolidated, i.e. that there is no excess pore pressure in the deposit due to placement/consolidation effects.

A tailings beach can be considered to be a semi-infinite slope. For a normally consolidated deposit, the factor of safety can be expressed as:

$$FS \quad = \quad \frac{k}{\sin \beta}\left(1 - \frac{\gamma_w}{\gamma}\frac{(h-z)}{h}\right) \tag{10}$$

where:

β =   slope angle (degrees).

γ_w =   unit weight of water.

γ =   total unit weight.

h =   depth of potential failure surface.

z =   depth to the water table.

The case where the water table is at the surface (i.e. $z = 0$) provides a lower bound limit, for which:

$$FS \quad = \quad \frac{k}{\sin \beta}\left(1 - \frac{\gamma_w}{\gamma}\right) \tag{11}$$

For the case where the water table is deep enough to be at or below the sliding surface (i.e. $z = h$):

$$FS \quad = \quad \frac{k}{\sin \beta} \tag{12}$$

Typical results for this latter case are plotted in Figure 6.

Results have been presented for two different values of $k \ (= S_{us}/\sigma_{vo}')$ derived from vane shear tests.

The first curve is for $k = 0.05$, as an indication of the possible lower bound strength at low confining pressures (Figure 5). This curve gives a factor of safety around 1.25 at 4% slope, and a limiting stable slope of 5%. If on the other hand, we adopt the lower bound of the Tronosco (2000) data, i.e. $k = 0.07$, a higher factor of safety results.

For this strength relationship, the plot indicates that beach slopes to steeper than 5% will be stable. These low $k \ (= S_{us}/\sigma_{vo}')$ values have been derived from vane shear testing at relatively high moisture contents/low confining pressures, i.e. at shallow depths. These values are not supported by triaxial testing at higher stress ranges (typically between 30 kPa and 1800 kPa). This testing has returned typical lower bound values of $k = 0.20$ (Figure 4). This value represents the third (upper) curve plotted in Figure 6. This indicates that at the strength range indicated by the triaxial tests, slopes well in excess of 5% should be stable.

Further work is required to fully define the undrained steady state strength (S_{us}) across the full range of confining pressures which exist in a tailings beach slope. It may be found that the low residual shear strength ($k \sim 0.05 - 0.07$) defined by the vane shear testing does not persist at higher confining pressures. This may be expected on the grounds that for a normally consolidated deposit, higher initial confining pressures give a higher density (lower void ratio) in the material prior to commencement of the undrained shearing. A higher resulting value of S_{us} would not be unexpected.
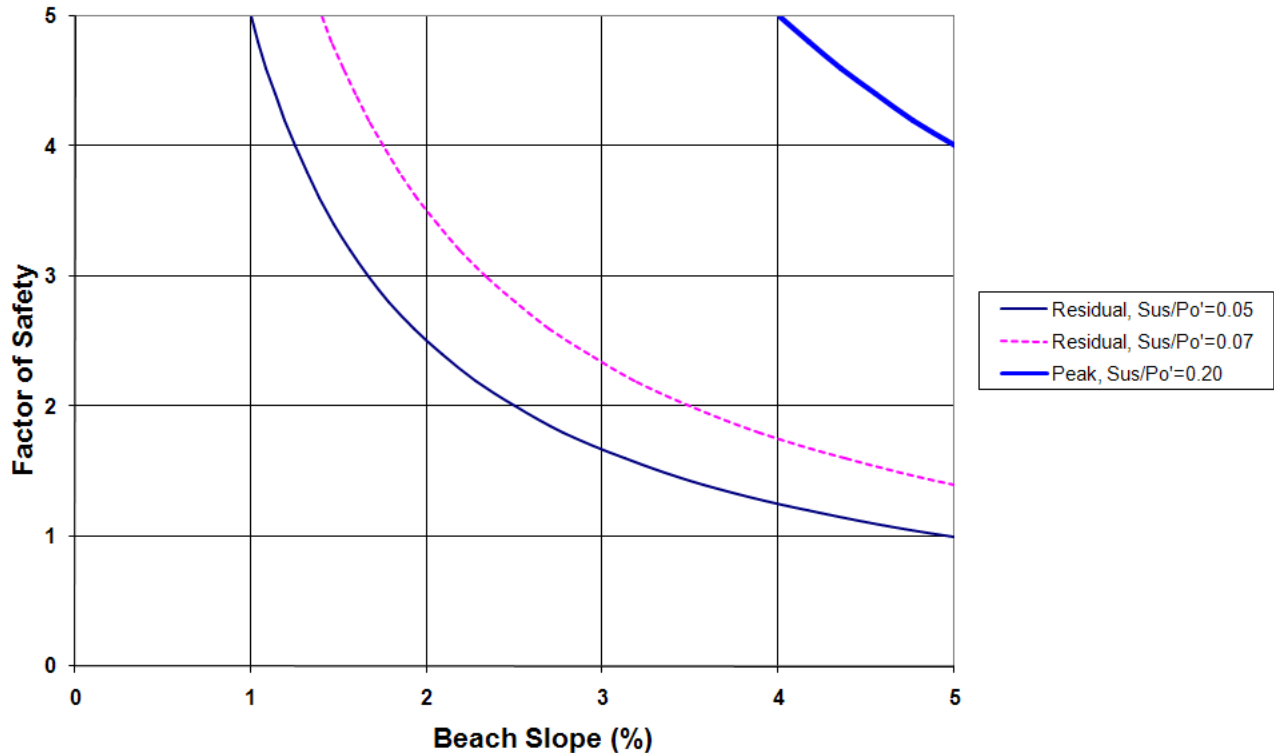
Figure 6   **Post liquefaction beach slope stability — deep water table**

7   CONCLUSIONS

A review of the literature of tailings dam failures has shown that "all in tailings" or slimes behind an embankment dam may liquefy and flow if the embankment fails and removes lateral support. However, the loss of strength is not total, and the remaining tailings surface exhibits a finite slope (i.e. is not flat).

Calculations of the stability of a tailings beach during seismic shaking indicate that saturated gentle slopes may reach an instantaneous failure condition during seismic shaking. However, this does not necessarily mean that liquefaction will result.

An analysis of the data on the cyclic strength of tailings indicates that large strains (>5%) may develop during seismic events. The conventional assumption for sands is that this will result in liquefaction. The available data for tailings suggest that strains well in excess of 5% are likely to be required to initiate liquefaction, and hence there is a degree of conservatism in the application of these results to tailings beaches.

Whether or not post-liquefaction flow sliding will occur depends on the slope, and the post-liquefaction shear strength. The latter will in turn depend on the pre-seismic pore pressure condition and depth. The analysis suggests that shallow surficial sliding is more likely than deep seated failure.

REFERENCES

Dobry, R. and Alvarez, L. (1967) Seismic Failures of Chilean Tailings Dams, ASCE J. SM &FE, V93/6.

Garga, V. and McKay, L. (1984) Cyclic Triaxial Strength of Mine Tailings, ASCE J. Geotechnical Engineering V110/8.

Hadj-Hamou, T. and Kavazanjian, E. (1985) Seismic Stability of Gentle Infinite Slopes, ASCE J. Geotechnical Engineering, V111/6.

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

Poulos, S.J., Castro, G. and France, J.W. (1985a) Liquefaction Evaluation Procedure, ASCE J. Geotechnical Engineering, V111/6.

Poulos, S.J., Robinsky, E.I. and Keller, T.O. (1985b) Liquefaction Resistance of Thickened Tailings, ASCE J. Geotechnical Engineering, V111/12.

Seddon, K.D., Murphy, S.D. and Williams, M.P.A. (1999) Assessment of Liquefaction Flow Stability of a Thickened Tailings Stack, Proc. 8th ANZ Conference on Geomechanics, Hobart.

Troncoso, J.H. (2000) Failure Mechanisms and Shear Failures of Tailings Dams under Earthquake Loads, Proceedings Geo Eng 2000, Melbourne.

Troncoso, J.H., Vergara, A. and Avendano, A. (1993) The Seismic Failure of Barahona Tailings Dam, Proceedings 3rd International Conference on Case Histories in Geotechnical Engineering, St. Louis, USA.