# Some considerations in the stability analysis of upstream tailings dams

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ABSTRACT: Upstream constructed tailings dams represent a significant challenge to the geotechnical engineer in terms of analysis of their stability, in large part because the shear strength of the loose sands and fine grained or "slimes" components of such structures is open to considerable uncertainty. In particular, it is critical that the behavior of the tailings under shear (contractant or dilatant, failure under drained or undrained conditions) be understood, and that this understanding be incorporated into stability analyses. The continued development of critical state soil mechanics has served to focus attention on this issue, but this focus has generally been directed towards problems involving seismic liquefaction. However, contractant versus dilatant behavior in shear is equally important in terms of static stability, particularly for upstream tailings dams which so often are constructed of primarily contractant, potentially liquefiable materials.

This paper presents a review of drained versus undrained methods of static stability analysis of upstream tailings dams, and how these relate to dilatant versus contractant behavior. The authors support the view that for upstream dams constructed of contractant, potentially liquefiable tailings, both undrained strength analysis and steady state strength analysis should be considered, and that effective stress analysis for such structures can be fundamentally incorrect and unsafe.

## 1 BACKGROUND

The upstream method has been employed for many tailings dams. Two idealized typical sections of upstream tailings dams are shown in Figure 1. The past and continued attraction of the method is obvious, as it represents the most economical of all methods of tailings dam construction. Mill tailings are hydraulically separated - either by cycloning or spigotting on beaches or in cells - with the coarser sand sized fraction used to build a retention shell or dam and the fines collecting in a pond. In some cases when beaches are short, sands are placed subaqueously and as a result are very loose. As the dam raises the more stable subaerial beaches [which are often strengthened by desiccation] step out over loose subaqueous sands or fines (slimes.) As dams are usually raised against some sort of regional slope, or in a valley, water pools against the starter dam. This combination of circumstances usually occurs where the dam is highest. Thus as the dam is raised loose sands and weak slimes in a wide range of relative proportion and degree of intermixing are trapped in the downstream section, often with disastrous results.

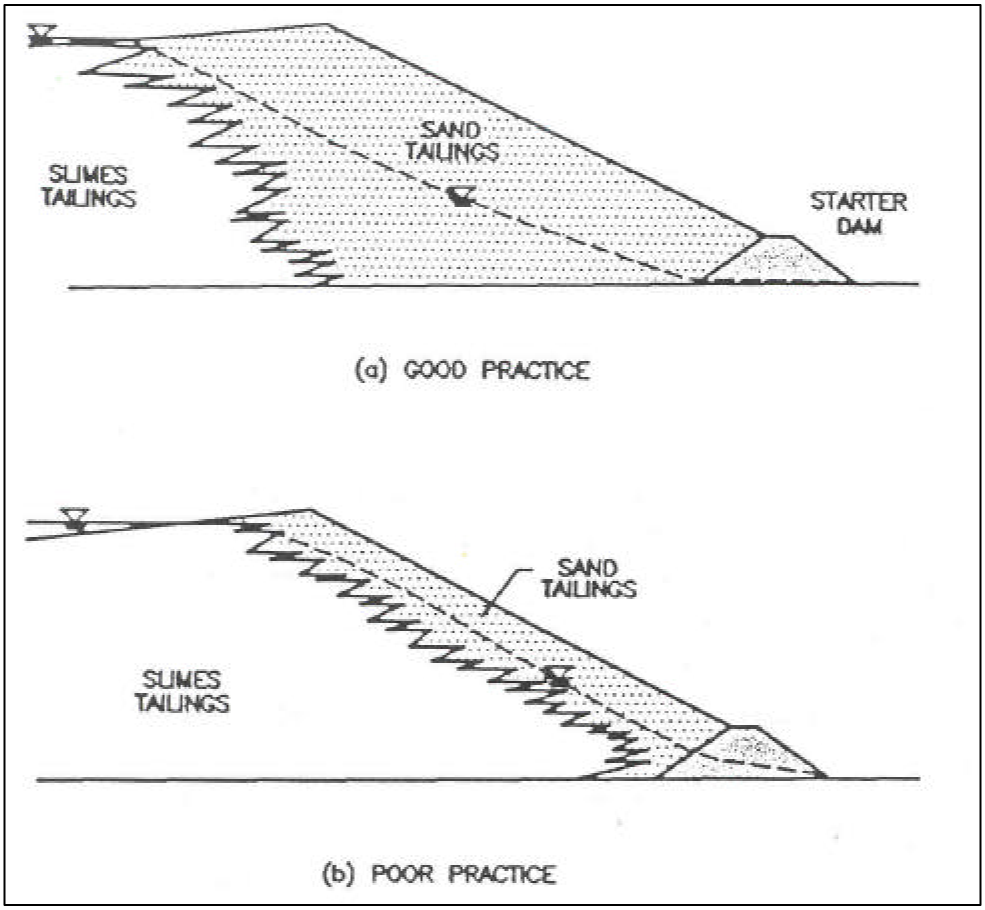



Figure 1. Idealized sections of upstream tailings dams.

Upstream tailings dams are unforgiving structures, and any one or combinations of improper design, construction and operation have resulted in a number of well-known, catastrophic failures, that have in some instances caused loss of life, such as the Stava failure in Italy (Berti et al., 1988, Chandler and Tosatti, 1995). The United States Committee on Large Dams (USCOLD, 1994) published a review of tailings dam failure records available to them in 1994. This review found that upstream-constructed tailings dams have recorded the largest share of documented failures.

The authors believe that many of these failures are due to the entrapment of loose sands and/or weak slimes in the downstream section. In many, and including recent designs reviewed by the authors, this factor is overlooked. This, as shall be described, is due to fundamental errors in the understanding of the operative strength of loose sands and weak slimes.

The susceptibility of upstream tailings dams to liquefaction and flow failures under seismic loading conditions is well known, and a number of case histories of such failures from the 1965 Chilean earthquake (Dobry and Alvarez, 1967), the 1978 Isu-Ohshima earthquake (Marcuson et al., 1979), and the 1985 Chilean earthquake (Castro and Troncoso, 1988) are well-documented. It is well understood that loose sands or weak slimes are contractant when sheared, and are therefore susceptible to this seismic triggering. What often appears to be not so well understood, and is the focus of this paper, is that undrained shear under static loading conditions can have the same consequences. As discussed in detail by McRoberts and Sladen (1992) there is little practical difference in the magnitude of the shear strength induced by seismic or undrained static loading.

The stability of tailings impoundments increases considerably with time once their operational life ends. This is particularly true for upstream tailings dams, and is due to the following factors:

1. Surface water is usually absent, particularly for impoundments regraded to shed runoff, allowing levels of saturation within the outer shell (and possibly the slimes) to gradually reduce.

2. Excess pore pressures induced by the raising of the impoundment will gradually dissipate, resulting in an increase in strength in the tailings slimes.

3. Capping of impoundments for closure reduces infiltration and allows for further reduction in saturation levels, particularly in dry climates.

4. Aging effects related to cementation and oxidation processes of tailings in the unsaturated zone, which may increase the liquefaction resistance of tailings by as much as 250% over 30 years (Troncoso, 1988, 1990).

5. Aging effects related to particle rearrangement resulting in macro-interlocking of particles and micro-interlocking of surface roughness (Joshi et al., 1995), a particularly significant mechanism given the angularity of tailings particles.

## 2 SAFE DESIGN AND OPERATION OF UPSTREAM TAILINGS DAMS

There is nothing fundamentally wrong with upstream tailings dams provided that key principles are adhered to in the design, construction, and operation of such dams. Upstream dams were originally designed in an empirical manner by mine operators, without the insights derived from geotechnical principles (Vick, 1992, Casagrande and MacIver, 1970). The vast majority of upstream tailings dams have performed satisfactorily. Based on experience, both successful and unsuccessful, this empirical design approach identified the importance of several key fundamentals for upstream tailings dams (Lenhart, 1950, Vick, 1992):

1. spigotting of a wide, sand (drained) tailings beach from the embankment crest;
2. avoiding situations whereby the dam slope is underlain by fine tailings (slimes) deposited within the water pond;
3. prevention of seepage emerging on the dam face; and
4. having a well-drained foundation.

The dam section shown in Figure 1(a) satisfies the criteria above, while the section shown in Figure 1(b) clearly does not.

These experienced-based principles remain valid, but do not directly address the criticality of operating and monitoring practices in maintenance of the safety of upstream dams. Nor do they directly address the issue of characterization of shear strengths in terms of drained or undrained behavior under shear. The authors therefore expanded the checklist above to the following eight fundamental rules for design, construction and operation of upstream tailings dams:

1. A sufficiently wide beach, relative to the ultimate height of the dam, must be maintained at all times, to achieve segregation of the coarser tailings sizes and to form a relatively strong, wide, drained (unsaturated), and/or dilatant (non-contractant during shear) outer shell. The dam slope must not be underlain by tailings slimes, unless the designer has satisfied Rule 4 below. The shell must be of sufficient width to retain the "bursting pressures" [see Casagrande and MacIvor] of the upstream contractant beach sands or slimes if they liquefy.

2. The rate of raising of the dam must be sufficiently slow such that there is a sufficient degree of dissipation of excess pore pressures in the outer shell and in the slimes, and such that excess pore pressure buildup does not occur in foundation materials.

3. There must be sufficient underdrainage (drainage blanket, finger drains) and/or a pervious foundation to maintain the sand shell in a relatively drained condition, and to prevent seepage from issuing from the face of the tailings dam.

4. Design analyses must include both undrained strength analysis (USA) and effective stress analysis (ESA), with design controlled by the analysis type giving the lowest factor of safety. A wide range of factors including material type, degree of consolidation and stress path must be assessed in assigning the appropriate USA.

5. A high degree of regular performance monitoring, reviews, and ongoing involvement by the designer is essential to check that design intent is being satisfied, to confirm design assumptions, and to identify any design changes that may be required.

6. Conventional upstream dams cannot be considered for areas of moderate to high seismicity. Improved upstream construction, involving a combination of compaction of the outer shell and good internal drainage, can be used in such areas.

7. The design must be consistent in terms of design requirements (e.g. minimum beach width) versus operational requirements (e.g. pond size required for clarification, storm storage and freeboard). The geotechnical design of upstream tailings dams cannot be carried out in ignorance of operating constraints.

8. Seepage conditions within the dam must be well-defined, requiring a good understanding of pore pressure profiles and hydraulic gradients. The distinction between pore pressure measured at a given point, and saturation level, must be well understood and correctly applied in stability analyses, especially in instances where there is strong downward drainage.

The following sections of this paper are focused primarily on illustration and support of Rule 4 above. Rule 4 merits particular attention for the following reasons:

1. It becomes critical when Rule No. 1 is violated, as is frequently the case for upstream tailings dams;
2. A great many upstream tailings have been, and continue to be, designed and/or evaluated based on limit equilibrium slope stability analyses assuming only ESA parameters;
3. ESA analysis for upstream dams in many instances is based on fundamentally incorrect assumptions regarding pore pressures prior to and during shear failure, and can result in large overestimates of the factor of safety; and
4. USA analysis should be applied for staged construction, and upstream tailings represent a classic case of staged-construction.

Tailings deposited in upstream impoundments, and particularly tailings slimes, are generally loose to very loose, contractant during shear, and strain-softening (brittle). Exceptions occur where the outer shell of an upstream dam is compacted (e.g. Martin and Tissington, 1996), or where desiccation in arid climates leads to overconsolidated and unsaturated conditions within the tailings beach (Blight, 1988). Both of these exceptions are cases where the tailings are dilatant during shear (and/or unsaturated), and therefore the undrained strength is higher than the drained strength (negative pore pressures generated during undrained shear). For contractant tailings, the opposite is true, and the question then becomes whether shear will occur under drained (no shear-induced increase in pore pressure) or undrained (shearing induces increased pore pressure) conditions.

## 3 APPROPRIATE STRENGTH MODEL: USA VERSUS ESA

### 3.1 *Definitions of strength characterization*

ESA, following Ladd (1991) and Carrier (1991) is defined as limit equilibrium analysis that assumes effective stresses during shear are unchanged from those that existed immediately prior to the onset of shear. That is to say, measured insitu pore pressures describe the conditions at failure. In other words, ESA explicitly assumes that shear occurs slowly enough, and/or the material being sheared is sufficiently free-draining, that there are no shear-induced pore pressures. An ESA method of analysis is either correct if one is sure that no positive pore pressures are generated on shearing, or is conservative if it is known that shearing is dilatant. On the other hand if one did an ESA analysis using a method to predict the pore pressure response during shearing [i.e., at the moment of failure] then a correct answer would be obtained. What can be fundamentally wrong about ESA analysis is using existing as measured pore pressures in a dam to represent the conditions at failure. This simple fact is unfortunately not understood by many geotechnical engineers.

USA is an analytically economical way of accounting for the pore pressures generated by undrained shearing. USA is defined in this paper as limit equilibrium analysis that assumes shear occurs under undrained conditions. This type of analysis therefore accounts for positive shear-induced pore pressures during shear of contractant materials, and negative pore pressures in the case of dilatant materials. A convenient way to describe the USA strength is the undrained strength ratio ($c_u$/p'). The effective stress p' denotes the operative effective stress [or consolidation stress] that exists in a soil element at the moment that failure begins.

### 3.2 *Behavior of non-dilatant or contractant material during shear*

Traditional soil mechanics has and continues to develop strength models for two basic classes of materials, clays and sands. Many typical mill tailings streams produce relatively unique artificial materials that tend to be bounded by these more traditional models. However as the guidance offered by this rich literature database often seems to be ignored by tailings dam designers it is useful to briefly review some fundamental aspects of it.

The term USA-D is introduced, (D designates ductile) to denote in a simple way the response of normally consolidated clays during shearing. During shear excess pore pressures are generated but with straining the available strength does not fall off or reduce. Key references are for example the work of Ladd (1991) and Wroth (1984). This literature makes several key points:

1. The USA-D strength of normally consolidated clays as a first estimate is in the range of $c_u$/p' = 0.20 to 0.25.

2. The USA-D strength is dependent on stress path. For example see Mesri (1989) who indicates that for triaxial extension (TE) the $c_u$/p' can be from 50% to 70% of the triaxial compression (TC) strength and the strength in direct simple shear (DSS) is intermediate between these limits.

3. The USA-D is dependent on the method used to measure undrained strength. Different results are obtained by different methods (Wroth, 1984).

These issues are well known in geotechnical practice, the lesson being that the determination of USA-D strength is not straightforward. Specification of pore pressure response at failure in an ESA type approach is even less straightforward, as demonstrated by Carrier (1991).

The term USA-SS is used here to designate the SS or steady state of loose sands [also called the residual strength.] Following Casagrande (1975), Poulos (1988) and as discussed more recently by McRoberts and Sladen (1992) loose sand can exhibit an undrained strength response at high strain, and a response that is highly brittle. The strength of clean loose sands at high strain or at steady state can also be expressed in a normalized manner, see for example Wride et. al. (1998). These authors follow others in normalizing the back-calculated strength of several case records in which the residual or steady state strength can be obtained. The normalized strength from these case records is as low as 0.01 and ranges up to about 0.20.

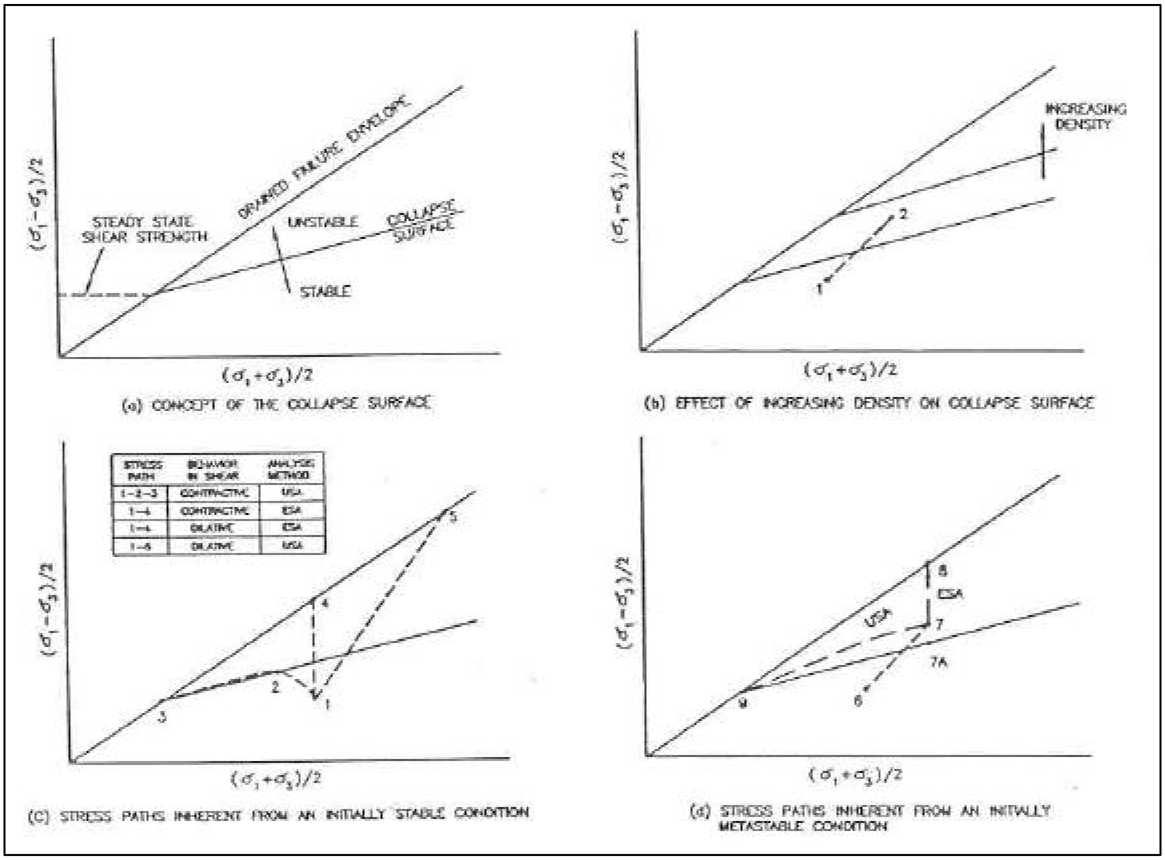
A recent series of USA-SS laboratory test on loose clean sands has been reported by Yoshimine et.al. (1998) for TC, TE, and DSS modes. This work following on earlier studies by others indicates that at the relatively low strains measurable in the laboratory that - and entirely similar to USA-D behavior for clays - the mode of shear has a strong influence on the magnitude of the normalized $c_u$/p'.

### 3.3 *Collapse Surface Approach*

The collapse surface approach presented by Sladen et al. (1985), as an extension of critical state theory, provides a useful framework to illustrate the authors' contention that USA analysis is mandatory for upstream tailings dams where contractant materials are involved. The concept is particularly useful in this application because it ties together undrained behavior in shear, the need for triggering of undrained behavior, and the brittle, flowsliding nature so often associated with tailings dam failures.

The concept of a collapse surface is illustrated in Figure 2a, which shows that for a given void ratio (density), there is a unique condition of stress state at which collapse of the sand structure takes place and undrained failure (liquefaction) is initiated. This unique state of stress is termed the collapse surface, and it exists below the drained failure envelope for contractant tailings.

Figure 2a illustrates that stress states lying on or above the collapse surface are highly unstable, as liquefaction can be triggered by even minor disturbance. Stress states below the collapse surface line are stable.




| STRESS PATH | BEHAVIOR IN SHEAR | ANALYSIS METHOD |
|---|---|---|
| 1-2-3 | CONTRACTIVE | USA |
| 1-4 | CONTRACTIVE | ESA |
| 1-4 | DILATIVE | ESA |
| 1-5 | DILATIVE | USA |

Figure 2. Collapse surface framework for comparison of USA and ESA approaches.

The collapse surface occurs in three-dimensional void ratio-shear stress-mean normal stress space. Figure 2b shows the effect of increasing density (decreasing void ratio) on the collapse surface, as projected onto two-dimensional shear stress - mean normal stress space. Also shown is a drained-loading stress path. Although point 2 lies above the collapse surface that corresponds to point 1, the fact that loading has taken placed under drained conditions, in concert with consolidation to a lower void ratio, has resulted in a shift upwards in the collapse surface line. Therefore, as long as undrained behavior is not triggered during the loading, a stable stress state can exist above the original collapse surface line, but only because the position of the collapse surface line has been shifted upwards during slow, drained loading.

Figure 2c presents stress paths for contractant and dilatant materials inherent to both USA and ESA limit equilibrium analysis. For a contractant material, USA assumes failure at point 2, with the mean normal effective stress at failure reduced from that at point 1 because of positive pore pressures generated during shear. USA also indicates a post-peak reduction in shear strength to point 3, which corresponds to the residual, or steady state strength. Stress path 1-4 represents that which corresponds to a conventional ESA, which by definition assumes slow shearing with complete dissipation of any shear-induced pore pressures during failure. Therefore, ESA assumes the mean normal effective stress at failure (point 4) to be the same as that at the initiation of shearing (point 1). It also assumes that failure occurs slowly. Figure 4c shows the shear stress at failure (and therefore the factor of safety) for ESA to be about twice that for USA, a typical result (Ladd, 1991) when comparing the two methods of analysis.

Assuming the material to be dilatant, then for undrained shear (assumed by USA), negative pore pressures developed by shearing would result in mean normal effective stress at failure (point 5) higher than that at the initiation of shear (point 1). Under fully drained shear assumed by ESA, failure would occur at point 4. Therefore, USA predicts a higher shear stress at failure (and therefore a higher factor of safety) than ESA for dilatant materials. In this case, ESA represents the more critical (and proper) method of analysis.

The differences between the stress paths assumed in the two types of analyses for contractant versus dilatant materials represent an important point. Advocates of ESA for upstream tailings dams, irrespective of whether saturated zones are contractant or dilatant, ignore completely the physical behavior of the materials under shear, surely one of the most important principles in soil mechanics. Furthermore, there is an inherent contradiction in accounting for contractant response when analyzing the seismic safety of upstream tailings dams, but failing to do so for static loading conditions. Finally, since ESA is the type of analysis applied for compacted embankments, it seems counterintuitive to blindly apply this type of analysis to uncompacted, loose tailings.

Figure 2d shows a case whereby loading from a point below (point 6) to above (point 7) the collapse surface occurs without failure, because undrained shear was not triggered during the loading. A stress state can therefore exist in the unstable zone, which in this case would be better referred to as the metastable zone, where spontaneous collapse (static liquefaction) can occur with even slight disturbance. USA for this circumstance would yield a factor of safety of less than 1 (failure at a shear stress corresponding to point 7A), and interpreted literally would suggest that such a stress state could not exist. Therefore, USA carried out in isolation as a standard limit equilibrium analysis ignores the need for some disturbance to trigger undrained behavior. Note again that ESA for such a metastable state ignores completely the potential for collapse of the soil structure, and gives a completely misleading impression as to the safety of the dam.

### 3.4 *Evidence Of USA Response In Upstream Tailings Deposits*

In a previous section we have reviewed the soil mechanics background and shown that for the typical range of soft normally consolidated clay soils to loose sands that a USA type strength response is often encountered. What then is the evidence from tailings deposits?

Probably the first reference on the normalized strength of copper tailings slimes is from Castro and Troncoso (1988) as summarized in Table 1.

Table 1. Data from Castro and Troncoso (1988)

| Dam | Undrained Peak Strength Ratio USA-D | Undrained Steady State Ratio USA-SS | Comments |
|---|---|---|---|
| Cerro Negro CN4 | 0.27 | 0.07 | Slimes, slightly plastic clayey silt PI of 5-20% |
| Veta deAgua VA1 | 0.21 | 0.11 | Slimes, clayey silt. |
| El Cobre EC4 | 0.29 | 0.08 | Slimes, no details. |

Note: Tests at steady state done with rapid undrained vane tests.

This testing indicates both the USA-D type response as well as the substantially lower steady state or USA-SS mode.

A summary of other case records is given in Table 2.

Table 2. Summary of USA Response in Tailings

| Case | Ratio $c_u/p'$ | Reference |
|---|---|---|
| Aluminum red mud (residual or SS) | 0.025 | Poulos et. al. (1985) |
| Lead – Zinc non-plastic slimes | 0.2 to 0.22 | Vick (1990) |
| Copper tailings slimes , PI = 6% | 0.26 to 0.33 | Bromwell (1984) |
| Copper tailings slimes, PI = 10+/-3% | 0.275 | Ladd (1991) |
| Copper tailings slimes $N_{1,60}$ of from 1 to 5 | >0.20 | Vidic et. al. (1995) |
| South African slimes | 0.25 CPT<br>0.17 to 0.4 Vane | Blight (1997) |

Writing on possible mechanisms for the runout of tailings derived mudflows, Blight (1997) presents conflicting information on tailings strength from Merriespruit which clearly demonstrates the USA vs ESA issue. The author states that:

> "Tailings consist of sand and silt sized particles of milled rock. The tailings referred to in this paper contain hardly any clay-size particles and, when sheared, behave as frictional, cohesionless materials with angles of shearing resistance in the range of 29-35$^{\circ}$."

Data from laboratory undrained dynamic shear tests report dilatant behavior, and Blight (1997) concludes that the low shear strengths that occurred in the field cannot be explained by postulating a form of undrained shear mobility of the tailings. However the author also presents the results of in situ vane and CPT tests (see Table 3 of Blight, 1997). The cone tip resistances are interpreted by Blight (1997) to give a shear strength gradient of about 2 kPa/m in the interior of the impoundment i.e., slimes or fine tailings. For an effective vertical stress gradient of about 8.0 kPa/m the USA-D ratio is therefore about 0.25. It is reasonable to think that these CPT probes are essentially undrained. The author also presents the results of vane testing. The vane peak tests, which are likely drained give a high strength/vertical effective stress ratio of 0.75 or more indicating dilatant response for drained shear. Remoulded vanes probably undrained give $c_u/p'$ of 0.17 in the upper 15 m and about 0.40 in the lower 15-30 m of the deposit. It is not known if the laboratory tests were on undisturbed or remoulded samples. Experience indicates that obtaining either undisturbed samples of loose tailings or re-creating the in situ fabric by laboratory techniques is difficult. The field evidence in the form of vane and CPT probes clearly indicates undrained response, as does the flowslide nature of the failure itself. More exotic explanations of the failure hardly seem necessary.

The authors had occasion to investigate the recent failure of a tailings dam in South America designed in the early 1990's by an internationally known company experienced in tailings dam design. This dam was designed solely with an ESA framework. After the failure which was due to undrained loading and resulting shear failure a major site investigation was including SPT and CPT testing was undertaken by the designer. During this investigation slimes were encountered and described by the designer as follows:

> "Fine tailings (slimes) that induce porewater pressure response during cone penetration are interpreted by this classification as clayey silts to clay. This interpretation may be correct in interpreting predominant particle sizes, but suggests a degree of plasticity which is not present in non-plastic rock flour"

The designers went on to analyze the structure using an ESA approach and determined that it was safe, notwithstanding the recent failure, to raise the structure as the investigation had determined that the design parameters were consistent with the original design. Analysis of the CPT data using the same procedures as Vidic et. al. (1995) by the authors indicated in situ $c_u/p'$ values of about 0.20. The use of this magnitude of strength readily explained the failure which occurred during a construction lift of the dam. In the authors' opinion this case record offers a classic example of the ESA versus USA issue and the absolute fallacy of assuming that "non plastic rock flour" can only have a so-called drained strength. In this particular case the designers recognized that the cone testing in slimes induced a pore pressure response but chose to rationalize the fact away by the "rock flour" model. The dam was condemned.

## 3.5 *Discussion*

Ladd (1991) discusses the differences between ESA and USA for staged construction, and argues convincingly that USA is the correct approach, because when failure of an upstream tailings dam does occur, it will be under rapid, undrained conditions if the tailings are contractant under shear, as is so often the case. Carrier (1991), in a paper that should be required reading for any engineer involved with upstream tailings dams, extends Ladd's argument specifically to the case of upstream tailings dams, for which he too convincingly advocates use of USA analysis. Carrier (1991) also pointed out the fallacy of justifying the assumption of drained shear in tailings on a high $c_v$ value (i.e. assuming a relatively free-draining material would necessarily undergo shear under fully drained conditions). Comparing an upstream tailings dam to a stress-controlled triaxial test, Carrier pointed out that, once the peak deviator stress is reached, the strain rate to the steady state (residual) strength for an undrained, contractant sand can be about 170% per second (Been and Jefferies, 1985). At such a strain rate, even a highly free-draining material would undergo shear under undrained conditions.

For upstream dams composed of loose, contractant tailings, failure will therefore typically occur very rapidly and under undrained conditions, and this is borne out by the fact that upstream tailings dam failures so often take the form of massive flowslides. Unfortunately, such field behavior is often not duplicated in laboratory testing, where, out of testing convenience and equipment limitations, testing is carried out under strain-controlled rather than stress-controlled conditions.

Casagrande (1975) pointed out that rapid (i.e. undrained) collapse deformation is an important behavior related to flow liquefaction in the field that cannot be observed in strain-controlled tests in the laboratory. Zhang and Garga (1997), describing the results of triaxial tests on loose sands carried out under stress-controlled conditions with a rapid data collecting system, suggested that laboratory equipment limitations and procedures may inhibit collapse deformation behavior, supporting Casagrande's view. This work suggests some caution in the adoption of USA-SS strength characterization from laboratory testing, and especially of the strain-controlled variety.

The advocacy of USA for upstream tailings dams presented in this paper is therefore a well known approach, best supported by Carrier's (1991) back analysis of the well known Tyrone failure case history. The Tyrone failure provided a classic example of the misconceptions as to tailings strength. This seems to have its origin in the old soil mechanics precepts that sands were frictional and clays cohesive. The misconception is that if one has non-plastic rock flour such material is obviously frictional and an ESA analysis applies. The flawed corollary is that if slimes are not cohesive, then a USA is not required - irrespective of whether or not the slimes are contractant in shear, and potentially liquefiable. Given these misconceptions, it is not surprising that tailings dam failures continue to occur. These lessons, shorn of theoretical aspects, were well understood many years ago by the likes of Casagrande and MacIver (1970), Smith (1972), and Lenhart (1950), and yet still have not entirely permeated the practice of tailings dam design and analysis.

# 4 REQUIREMENT FOR TRIGGERING OF UNDRAINED SHEAR

ESA analysis typically overestimates the factor of safety by a factor of two relative to USA analysis. This in turn would suggest that a great many upstream tailings dams have USA factors of safety of less than one (since they are rarely designed using ESA to a factor of safety of 2 or more), and should have failed (based on limit equilibrium analysis), but have not. This in turn suggests that in many cases an undrained trigger has been absent.

The discussion above for Figure 2d addresses this apparent contradiction. Vick (1992), in a discussion of Ladd's Terzaghi lecture (Ladd, 1991) also addressed it, pointing out that for upstream dams constructed as shown on Figure 1a, which satisfy Rule No. 1 above, ESA had served well. Vick (1992) also discussed how the breakout of seepage on the dam slope could lead to rapid, undrained progressive failure even of relatively coarse (free-draining) but loose (contractant) sands. In this case, the seepage breakout triggered undrained shearing in a contractant material, even though drained conditions existed immediately prior to failure and the material was free-draining, a result consistent with the studies by Eckersley (1990). This reinforces the notion that some trigger is required to initiate undrained shear failure in upstream tailings dams.

For upstream tailings dams, unfortunately, potential triggers of undrained failure are numerous, and include those listed in Table 3.

Table 3. Triggering mechanisms for undrained failures of upstream tailings dams.

| Mechanism | Trigger |
|---|---|
| Oversteepening at toe due to: | Erosion (intense storm runoff, pipeline break causing washout)<br>localized, initially drained sloughing<br>construction activities (excavation) |
| Overloading due to: | rapid rate of impoundment raising<br>steepening at crest<br>construction activities at crest |
| Changes in pore pressures due to: | seepage breakout on face of dam<br>deterioration in performance of underdrainage measures<br>inhibited volumetric creep<br>concentrated tailings discharge from one location for extended period<br>leakage/rupture of low level outlet<br>accelerated rate of construction<br>foundation and/or embankment movement<br>intense rainstorms<br>increased pond levels |
| Triggering collapse surface by reduction in mean effective stress (see Figure 3) | Consider an element of soil below the collapse surface with a low shear stress and high mean effective stress due to low or absent phreatic surface. Saturating the slope reduces mean effective stress, but leaves shear stress constant. The reducing mean stress results in contact with the collapse surface and liquefaction is triggered. |
| Overtopping due to: | severe storm runoff<br>failure of diversion dams/ditches<br>blockage and failure of spillways/decants<br>seismic deformation and loss of freeboard |
| Acceleration/vibrations due to: | Earthquakes<br>construction traffic<br>Blasting |

It is noteworthy that earthquakes represent perhaps the only of the above triggers for which analytical methods account for, and are indeed based on, the distinction between contractant versus dilatant material in shear.

Ladd (1991) observed that failure for contractant materials will occur under undrained conditions even if drained conditions prevailed immediately prior to the failure, an assertion supported by work by Eckersley (1990), in his modelling of flowslides. The most elegant demonstration of this, and the role of Sladen's collapse surface and drained triggering mechanisms can be found in a series of tests reported by Sasitharan et. al. (1993). Figure 3 reports on one of these tests. In this Figure the state boundary or collapse surface for a clean Ottawa sand is given at a void ratio of 0.809. In the lab a new sample of sand at a void ratio of 0.804 and at a stress state represented by a deviator stress of 100 kPa and mean consolidation stress of 300 kPa was prepared. This sample exists well below the state boundary surface and any minor excursions of shear stress or reduced or increased mean effective stress would result in a stable response to loading. However by appropriate manipulation of the loading conditions and back pressures, the mean effective stress on the sample was reduced but the deviator stress was kept constant. As the mean stress was reduced in a series of drained increments, the collapse surface was reached. At this time the void ratio had increased to 0.809, the collapse surface was reached and an undrained collapse of the sample was triggered. As shown on Figure 3 the deviator stress that could be supported by the sample reduced to about 20 kPa. The USA-SS strength for this sample is therefore about 20/300 or 0.067.

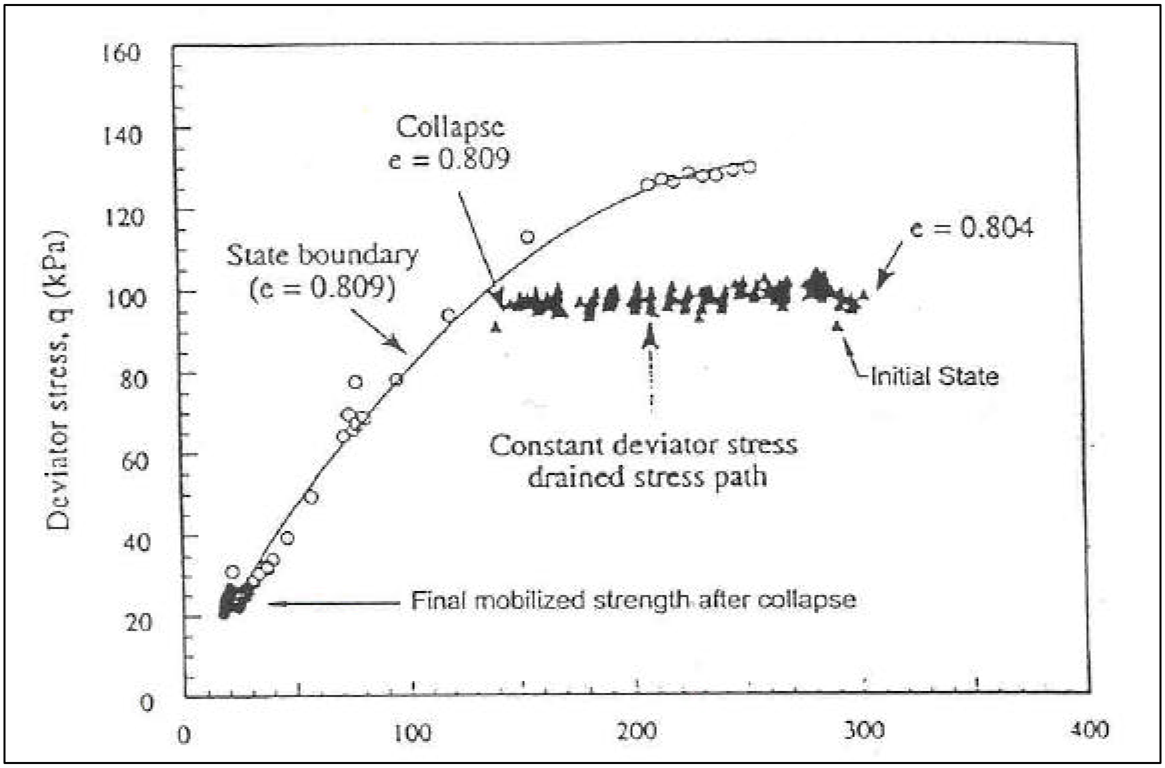



Figure 3. State boundary defined by void ratio 0/809 and constant deviator (q=100kPa) drained stress path. After Sasitharan et al (1993).

Consider then an element of soil in the relatively loose shell of a tailings dam. Assume a low phreatic surface and a resulting stress state below the collapse surface. This stress state has reasonable high shear stress and high mean effective stress due to low or absent phreatic surface. Assume further that the slope saturates due to heavy rainfall, lateral migration of the phreatic surface due to dam raising or some combination of these or other events. Saturating the slope reduces mean effective stress, but leaves shear stress constant. If the reducing mean stress results in contact with the collapse surface liquefaction is triggered. Right up to the initiation of collapse the soil elements have been drained. This is an example of drained loading triggering an undrained collapse.

## 5 RECOMMENDED APPROACH

The authors therefore strongly concur with Carrier's (1991) assertion that upstream dams should be assessed based on both USA and ESA analysis. However, in recognition of the metastable nature of upstream tailings dams, the USA analysis results must be interpreted in the context of the potential triggers of undrained shear listed in Table 1, rather than a limit equilibrium factor of safety in isolation. Unless considerable effort is expended in the design process and due consideration is given to all failure modes it is considered appropriate to assume that if a soil can liquefy it will.

Given that many of these triggers represent operational rather than geotechnical factors, it follows that it is essential that operational aspects (e.g. rate of raising) be integrated into any stability assessment and design of an upstream tailings dam. Furthermore, the margin of stability against shear failure of upstream tailings dams is better expressed jointly in terms of both the probability of undrained behavior being triggered, and a limit-equilibrium factor of safety.

The authors consider that, for a great many upstream tailings dams, especially those where the dam slope is underlain by slimes (i.e. violates Rule No. 1), ESA analysis has given the right answer (i.e. a stable dam), but for the wrong reasons. That is an undrained mechanism never existed. The fact that such dams have remained stable cannot be considered an endorsement of universal application of ESA analysis. Fortunate happenstance is no substitute for good design and analytical practice. Only where Rule No. 1 is satisfied, and the dam configuration is as shown on Figure 1a, can reliance solely on ESA be justified. Continued reliance on such an approach for upstream dams that violate Rule No. 1 ignores the fundamentals underlying triggering and pore pressure response during undrained shear in contractant materials, and is likely to lead to future failures of upstream tailings dams.

### 5.1 *Estimation of Drained Shear Strength Parameters for Analysis*

For assessments of the stability of a number of upstream tailings dams at base metal mines in which the authors were recently involved, little or no data was available with which to estimate the drained and undrained strength parameters for the tailings dams assessed. There is abundant case history experience to draw upon in terms of drained strength parameters. Effective friction angle ($\phi$') values typically range between 25° and 35° for base metal tailings, with the lower portion of this range applying for silt/clay tailings slimes, and the upper portion applying for coarser tailings deposited on beaches. Table 2.8 of Vick (1990) gives a summary of typical values of $\phi$' for various types of tailings.

### 5.2 *Estimates of Undrained Shear Strength Parameters*

The brief review presented earlier for normally consolidated clays and loose sands indicate two forms of USA behavior: ductile and brittle or steady state / residual modes. We have also reviewed a series of references where many examples of undrained strength ratios ($c_u/p'$) reported in the literature, where a range of 0.2 to 0.3 appears to cover most of the cases for tailings at least for USA-D type response. This suggests that many tailings have an USA response very similar to normally consolidated clays.

However the test data presented by Castro and Troncoso (1988) introduces the considerable caution that a USA-SS response may also be present. Much of the data available and discussed above is based on insitu interpretations of CPT response.

What we have called USA-SS mode or strength response has been discussed in detail by Poulos (1988) who recommends an approach more conservative that the USA approach (which is based on peak undrained shear strength). For the USA-SS mode design analyses are carried out based on the steady state, or residual, undrained strength. This approach assumes that the worst case scenario (straining of the contractant tailings sufficient to cause undrained collapse of the soil structure) governs design, reducing shear strength from peak (USA analysis, point 2 on Figure 2c) to residual (Poulos method, point 3 on Figure 2c) levels. Carrier (1991) supports consideration of this approach. He emphasizes that it is necessary to consider the stress-strain characteristics of the tailings, with brittle, cohesionless tailings (strain to peak about 1% typically) more relevant and critical for a Poulos type analysis that clayey, ductile, cohesive slimes (strain to peak much larger than 1%).

The authors advocate that, for upstream dams, the Poulos approach should also be considered for both loose sands and well as slimes. This forces the stress-strain characteristics of the materials forming the dam to be considered in the analysis. The steady state approach is widely applied in analyzing the seismic stability of upstream tailings dams. Given that seismic loading is but one of many potential mechanisms of undrained loading (McRoberts and Sladen, 1992), it is inconsistent that the steady state approach not be at least considered under static loading conditions. Application of the method is not without its problems, particularly with respect to determination of the appropriate steady state strength, which has been the subject of much research and debate within the geotechnical profession for many years. Considering the difficulties of predicting the appropriate USA strength is however a step in the right direction; assuming the designer has at least abandoned a sole reliance on the ESA mode.

### 5.3 *Pore pressure conditions for stability analyses*

Use of both ESA and USA requires an estimation of the effective consolidation stresses at any given time, which in turn require a good understanding of the pore pressure conditions (hydrostatic versus downward drainage, normally consolidated versus excess pore pressures) within the dam. Pore pressure conditions within upstream tailings dams are very often complex, misunderstood, and improperly incorporated into stability analysis. Misinterpretation of piezometer data can easily occur if adequate piezometer coverage does not exist. Vick (1990) suggests that, for rates of impoundment rise of between 15 and 30 ft/year, excess pore pressures are usually assumed to dissipate as rapidly as the load is applied, and therefore a normally consolidated state (i.e. zero excess pore pressure) can be assumed. Mittal and Morgenstern (1976) also suggested this range as being sufficient to generate excess pore pressures in slimes.

The authors caution that these experience-based criteria on rate of rise can be safely applied only in cases with good underdrainage (permeable foundation relative to the tailings slimes), relatively smaller embankments (35 m in height or less) with relatively short drainage paths, and slimes free of significant clay content and plasticity. For example, the authors are aware of one large upstream dam in which very high excess pore pressures exist in the clayey slimes despite a rate of rise of only about 7 ft/year. Another example that emphasizes the need for caution is the Tyrone tailings dam, which failed under undrained conditions at a construction rate of 12 to 15 ft/year.

### 5.4 *Recommended approach*

To summarize, the authors provide the following general recommendations for any static stability analysis of an existing upstream tailings dam:

1. Determine whether or not the dam slope is comprised of materials that are contractant or dilatant under shear. Characterize pore pressure conditions within the dam, to properly determine effective stresses. If the materials are dilatant and/or fully drained (unsaturated), then only a ESA is required.

2. If the dam slope is fully or partially composed of contractant materials, then both ESA and USA should be carried out. The factor of safety from the USA will better represent the margin of safety of the dam. Considerable care must be exercised in the selection of the appropriate USA strength mode. Careful attention must be given to whether or not a USA-D or USA-SS mode might be triggered.

3. Review the operational and design factors necessary to assess the probability of undrained shear being triggered by the various mechanisms listed in Table 1. The probability of these mechanisms should be considered jointly with the USA factor of safety in evaluating the safety of the dam. Unless one is very sure that undrained triggers absolutely do not exist, it is considered prudent to assume the worst.

4. Evaluate the stress-strain behavior of the tailings comprising the dam. If the tailings are both contractant and brittle (low strain to peak strength, and significant post-peak reduction in strength), then a Poulos (steady state strength or USA-SS mode) analysis should be carried out.

The results of this analysis should be considered jointly with the assessed probabilities of undrained shear triggering mechanisms.

## 6 EXAMPLE RESULTS OF STABILITY ANALYSES

A dam configuration typical of several of those recently reviewed by the authors is shown on Figure 4. This dam was designed in the early 1990's and includes a compacted starter dam with drainage and filter zones as shown. The dam is being raised about 25 m above the starter dam crest, at a slope of 3H:1V, using spigotted tailings produced from a flotation circuit (base metals mine). The design specified that a minimum 30 m wide beach be maintained at all times. Fortunately, the operators were able to maintain somewhat wider beaches than this in all but one of the cases.

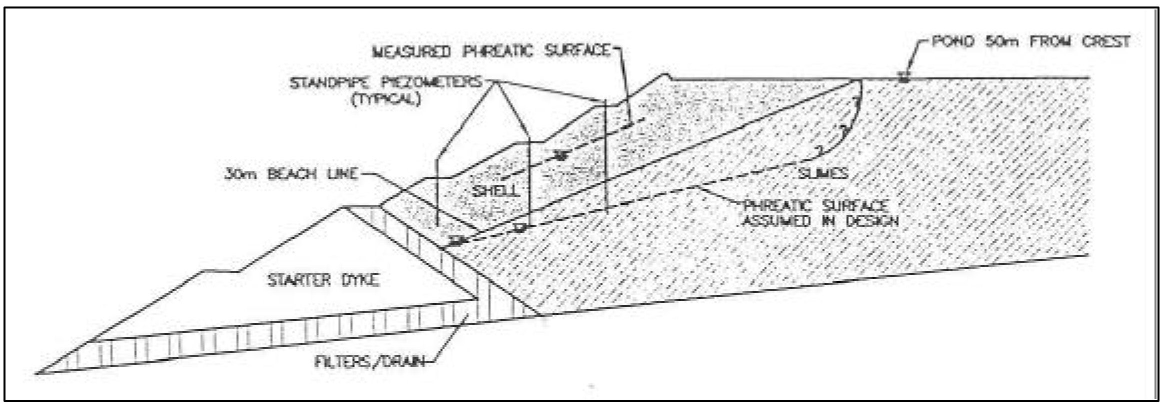



Figure 4. Typical design section of upstream dams reviewed.

The designer's stability analyses were of the ESA type, and assigned both the sand shell and the slimes an effective friction angle of 30°. Therefore, by assuming the same shear strength for both zones, the only effect of the beach width in terms of the design analyses was on the assumed location of the phreatic surface. The designer's analysis also assumed hydrostatic pore pressure conditions.

The stability analysis geometry and results obtained by the authors are illustrated graphically on Figures 5 and 6, for the design minimum beach width of 30 m. They clearly show that ESA provides an acceptable factor of safety to dam heights of up to 30 m (above the starter dam crest), while USA-D analysis indicates inadequate factors of safety. The USA analysis results shown on Figure 6 may well be slightly unconservative because they assume ESA can be applied for the outer sand shell. However, this material may also be contractant in shear, so undrained strengths could be more appropriate in the saturated portions of this zone. The analyses also do not account for the stress path dependence of USA-D strength discussed in Section 3.2. Moreover, depending on the ability of the tailings to absorb a degree of straining a lower USA-SS strength may be appropriate

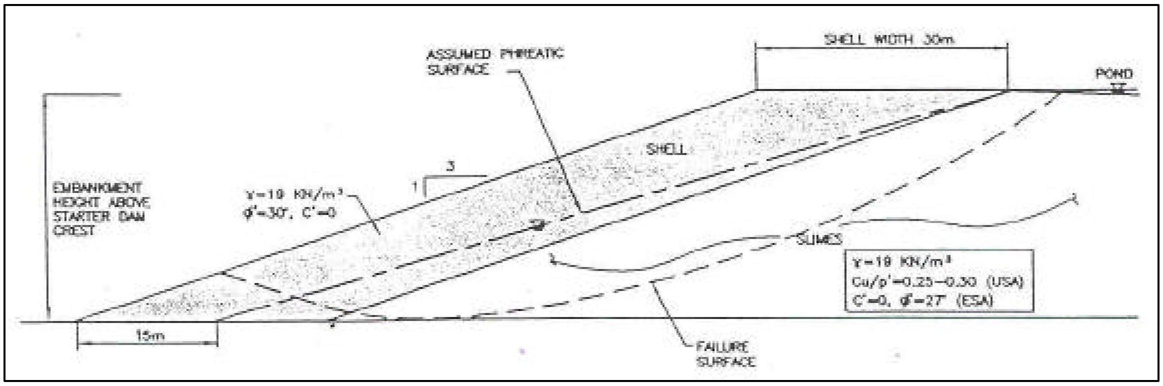



Figure 5. Stability analysis section.

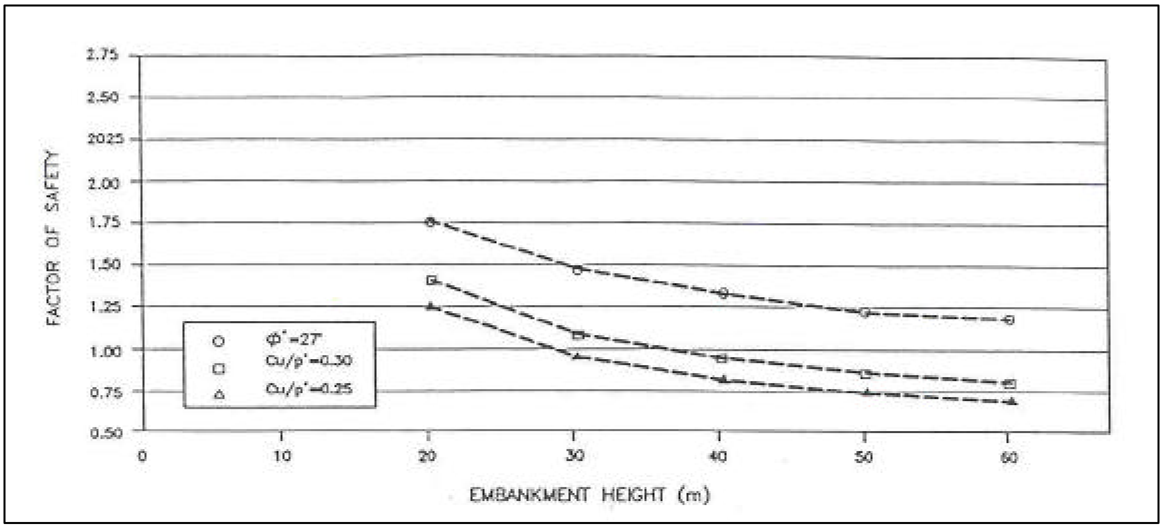



Figure 6. Results of preliminary stability analyses.

## 7 CONCLUSIONS

Upstream tailings dams are complex structures for which shear strength and pore pressure conditions are difficult to predict in advance. It is essential that stability analyses of these structures be predicated on a thorough understanding of the behavior of the tailings under shear, and an understanding of the likelihood of potential triggering mechanisms of undrained failure. Stability analysis of upstream tailings dams should be carried out using both drained strength analysis and undrained strength analysis. In some cases it may also be necessary to consider steady state strength analysis. This approach forces the designer to come to grips with the issue of contractant versus dilatant behavior in shear, and with the stress-strain characteristics of the material, two of the most fundamental precepts of soil mechanics that are ignored at the geotechnical engineer's peril.

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