Static liquefaction as a possible explanation for the Merriespruit tailings dam failure: Reply

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Static liquefaction as a possible explanation for the Merriespruit tailings dam failure

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Abstract: In 1994 the Merriespruit gold tailings dam in South Africa failed, resulting in 17 deaths. The post-failure investigation provided no explanation as to why the catastrophic flow failure, which contradicted all previous experiences of failures of gold tailings dams in South Africa, occurred. The documented history of the dam describes insufficient freeboard provision and often poor pool control, which is argued to have resulted in some areas of the dam having high in situ void ratios. Some of the undrained triaxial tests carried out on specimens obtained from zones adjacent to the failure scar exhibited nondilative behaviour. Laboratory triaxial tests that were conducted on reconstituted specimens and are reported in a companion paper defined a series of steady state lines that were dependent on the particle-size distribution of the tailings. Void ratios obtained from undisturbed samples taken during the post-failure investigation are compared with these steady state lines and it is shown that an appreciable percentage of the specimens were likely to have been contractant. The inference drawn is that a large volume of tailings was in a metastable state in situ and overtopping and erosion of the impoundment wall exposed this material, resulting in static liquefaction of the tailings and a consequent flow failure.

Key words: static liquefaction, gold tailings, Merriespruit, failure.

Résumé : En 1994, la digue de stériles de la mine d’or de Merriespruit en Afrique du Sud s’est rompue causant 17 pertes de vie. L’investigation après rupture n’a fourni aucune explication quant à savoir pourquoi cet écoulement catastrophique s’est produit, en contradiction avec les expériences antérieures de rupture de digues de stériles de mine d’or en Afrique du Sud. On prétend que l’histoire documentée de la digue décrivant la hauteur de revanche insuffisante et le contrôle souvent défectueux du bassin a résulté en la mise en place de stériles ayant des indices de vide *in situ* élevés dans certaines zones de la digue. Certains des essais triaxiaux non drainés réalisés sur des spécimens prélevés dans les zones adjacentes à la cicatrice de la rupture ont montré un comportement non dilatant. Des essais triaxiaux qui ont été faits en laboratoire sur des spécimens reconstitués et dont les résultats sont donnés dans un article ci-joint ont défini une série de lignes d’état permanent qui dépendaient de la granulométrie des stériles. Les rapports de vide obtenus à partir d’échantillons non remaniés prélevés durant l’investigation après la rupture sont comparés aux lignes d’état permanent, et on montre qu’un pourcentage appréciable des spécimens auraient vraisemblablement pu être contractants. On en déduit qu’un grand volume des stériles *in situ* étaient dans un état métastable et que le déversement et l’érosion de la face de la retenue a exposé ce matériau, ce qui a résulté en une liquéfaction statique des stériles et en conséquence une rupture par écoulement.

Mots clés : liquéfaction statique, stériles de mines d’or, Merriespruit, rupture.

[Traduit par la Rédaction]

Introduction

Failure of the 31 m high gold tailings dam adjacent to the village of Merriespruit, a suburb in the small town of Virginia, South Africa, in February 1994 resulted in the deaths of 17 people and widespread damage to the village and surrounding environment.

The failure occurred a few hours after a brief thunderstorm and about 50 mm of rainfall. Approximately 600 000 m^3 of liquid tailings flowed from the tailings dam through the village of Merriespruit and finally stopped about 3 km downgradient of the tailings dam when it reached a small tributary of the Sand River which flows through the northern boundary of Virginia. An aerial view of the failure is shown in Fig. 1. As the first row of houses in Merriespruit were only some 300 m from the breach location, the wave of tailings was about 2.5 m high when it reached these houses. Many houses were swept off their foundations and others had walls demolished. Figure 2 shows two of the many houses that were destroyed. The maximum height of the tailings flow was clearly visible just below the eaves level. The failure occurred at 21:00 and, because many people were in their homes at this time, the number of fatalities was probably higher than would be the case if the failure occurred during daylight. A combined inquest–inquiry was presided over by a judge appointed by the Minister of Justice, and eventually both the mine and the contractor that operated the tailings impoundment were held responsible for the disaster.

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Fig. 1. Aerial view of the Merriespruit tailings dam failure showing the path of the destructive mudflow that occurred.

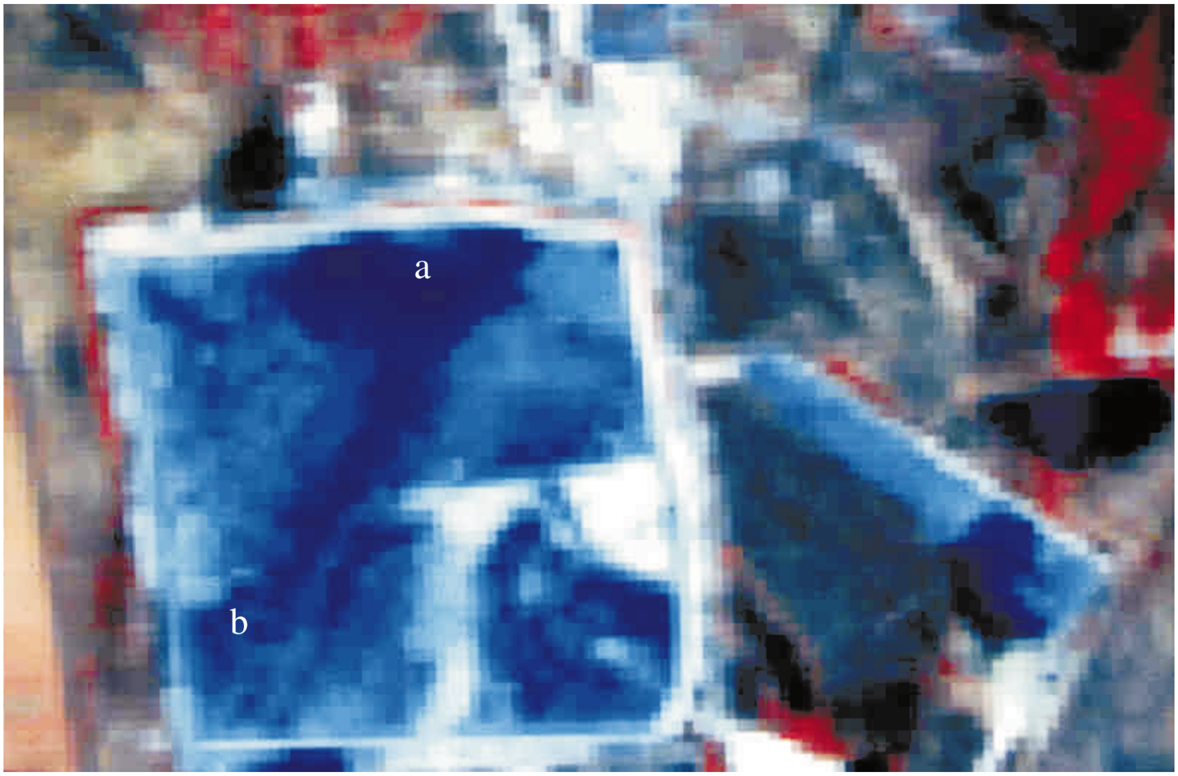
Fig. 2. Partially destroyed house that was about 350 m from the breach location.

Following the failure, a number of investigations were launched. Three different teams of geotechnical engineers undertook these investigations, one representing the state, one the owner, and one the operator of the dam. A comprehensive evaluation of the findings of these three teams is given by Wagener et al. (1998), and only points relevant to the discussion of static liquefaction as an explanation for the failure are repeated here.

Operational history of the dam

Construction of the tailings dam commenced in 1978, with a low "starter wall" at the low end of the dam perimeter. The dam was operated in a manner similar to that of the vast majority of gold tailings dams in South Africa, using the upstream daywall paddock construction method (ICOLD 1982). A blanket drain was provided below sections of the starter wall with a view to depressing the phreatic surface. Instead of a single ringmain around the dam, as is current practice in South Africa, three delivery pipelines were used during the last few years of operation of the dam, with delivery stations at about 500 m intervals. Water was extracted from the top of the dam using permanent, embedded penstocks. No water storage dam was provided and the penstocks thus decanted into a sump, from where water gravitated back to the metallurgical plant on demand. Because of the inadequate storage capacity of the sump, this arrangement often led to storage of water on top of the tailings dam, which as described later was a major contributor to the failure.

Fig. 3. Landsat photograph taken approximately 3 weeks before the failure, showing tailings pond adjacent to the north wall of the tailings dam. The top of the photograph is north. (a) Where pool developed on north wall (black indicates surface H_2O); (b) where deposition occured.



At the location where the breach occurred, the dam crossed a shallow, natural drainage valley. Although there was originally a delivery pipe at this low point, it was subsequently relocated to assist the operators in controlling the pool position. The delivery line relocation resulted in the finer fraction of the tailings accumulating in this low section of the dam wall. According to Wagener et al. (1998), the tailings dam operator reported great difficulty in construction during the early years of operation, primarily due to the low slurry density of the tailings received from the reduction works. These construction problems later manifested as severe sloughing of the toe of the outer slope and in 1991 a small rock buttress was placed against the slope in the area where the failure occurred. Subsequent to this, a number of other palliative remedial measures were used at various points around the dam, including a hydraulically placed tailings buttress.

It is evident from the above description that there were many indicators pointing to the unsatisfactory state of the tailings dam prior to the actual failure. However, it is not uncommon for minor problems of toe sloughing to occur on tailings dams without inevitably leading to a catastrophic failure. Indeed, Fourie and McPhail (1993) described how a platinum tailings dam continued to be operated successfully for many years after problems similar to those noted above occurred, principally due to implementation of appropriate remedial measures (active dewatering) and regular monitoring using piezometers, precision surveying, and monitoring flow rates of the dewatering wells.

Problems of excessive seepage and toe sloughing continued even after placement of the tailings buttress. In March 1993 the decision was taken to suspend tailings deposition on the northern compartment of the dam (i.e., the compartment that breached in 1994). For reasons that were attributed to poor communication, unauthorized deposition of tailings still took place on this compartment from time to time after March 1993. In addition, spillage from the adjacent compartments into the supposedly decommissioned compartment also occurred sporadically. These activities combined to push the pool on the northern compartment progressively towards the northern wall. Photographs taken after the failure indicated that the vertical freeboard at this location was a mere 300 mm prior to the failure. Storage of water on the top of tailings dams in South Africa is allowed, subject to the provision at that time that the basin on the top of the dam have sufficient capacity to retain, with a minimum freeboard of 500 mm, the 1 in 100 year recurrence interval storm of 24 h duration. This was clearly not the case at the time of failure of the Merriespruit dam. During initial evidence given at the inquest–inquiry at least one witness denied that there had been any water on top of the impoundment in February 1994 (Strydom and Williams 1999). However, as reported by these authors, images subsequently obtained from a Landsat satellite that were taken only 3 weeks before the failure provided irrefutable evidence of there having been free water on top of the impoundment and that it was abutting the outer (northern) rim shortly before the breach occurred. Faced with this evidence, individuals who had previously denied the existence of a tailings pond confessed that there had indeed been water on top of the impoundment and it was then learned that water was being pumped to the top of the impoundment even on the night of the disaster. One of the Landsat images in question is shown in Fig. 3. Deposition can be seen to have been taking place in the southwest corner of the tailings dam, with the resulting path of water towards the north wall clearly evident.

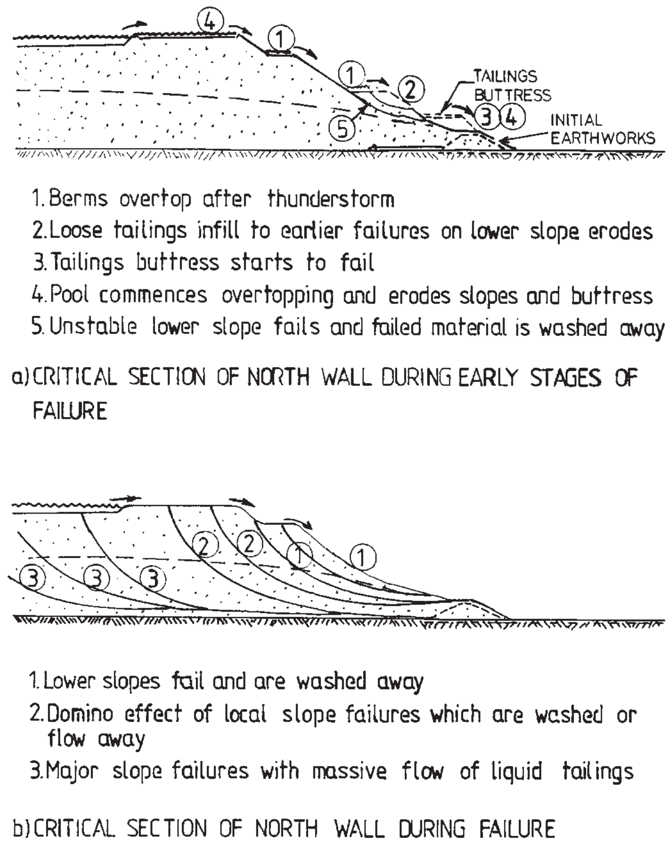
The foregoing discussion has briefly summarized aspects of the history of the Merriespruit tailings dam which are pertinent to the later discussion on likely mechanisms of failure. Of particular relevance are the sloughing attributed to initial problems with low delivery slurry density, the relocation of a delivery line that resulted in subaqueous deposition of fine tailings at the breach location, unauthorized tailings deposition that resulted in unacceptably low freeboard existing on the top of the dam, and the high phreatic surface that made revegetation of the slope in this area impossible.

Mode of failure

It is accepted that the primary cause of the failure was overtopping, which resulted in large-scale removal of tailings from the slope face (Wagener et al. 1998). The post-failure investigations found that the dam overtopped at the breach location and that water flowed over the crest of the dam for between 1 and 2 h. Removal of tailings from the outer slope would certainly have exposed tailings inside the dam that had previously been confined. Conventionally, however, gold tailings produced from Witwatersrand quartzites are considered to be strongly dilatant (Blight 1998), and thus even if confinement were removed, the tailings should not have moved for any significant distance, since as explained by Zhang and Garga (1997), for a strain-hardening soil the resistance increases continuously during undrained loading. Moreover, it was considered that the outer zone of the tailings dam was well consolidated due to the cyclic deposition procedure wherein thin (150–200 mm) layers of tailings are placed sequentially around the dam perimeter and allowed to drain, desiccate, and consolidate (Blight and Steffen 1979). It therefore came as a great shock to the geotechnical community in South Africa that as much as 600 000 m^3 of gold tailings would in fact liquefy and flow as far as 3 km (Wagener et al. 1998).

Figure 4 shows the sequence of retrogressive failures postulated by Wagener et al. (1998), based on eyewitness accounts of the Merriespruit disaster. The outer slope failed due to erosion by water overflowing from a small breach at the top of the slope. The slope became steepened locally and a sequence of slumping failures occurred because the shear strength was inadequate to maintain these steepened slopes. This slumped material was washed away by water escaping from the tailings pond. At some point it seems that this retrogressive failure exposed tailings that were in a metastable state in situ and instead of merely slumping, flowed as a liquid. This then appears to be the essential difference between the Merriespruit failure and other gold tailings dam failures in South Africa, where retrogressive failure of an outer slope has eventually stopped, either due to water from an exposed tailings pond being exhausted or the shear strength of exposed tailings eventually becoming sufficient to accommodate some localized oversteepening. This latter concept is illustrated in Fig. 5, which is a view of the Merriespruit

Fig. 4. Sequence of retrogressive failures postulated by Wagener et al. (1998).



post-failure scar. The steep sides of the exposed tailings are clearly evident.

Unanswered questions

A few months prior to the failure, the tailings dam operator had carried out a stability analysis for the section of the dam that ultimately failed. Using an angle of internal friction ϕ' of 35° and an assumed cohesion c' of 2 kPa below and 11 kPa above the phreatic surface, a stability analysis using circular failure surfaces and the Bishop method of analysis was carried out. Of eight piezometers installed in the vicinity of where the breach occurred, five were still being monitored at the time of the failure and data from these piezometers were used in the analysis. A minimum factor of safety of 1.34 was calculated. The rationale for using a higher apparent cohesion above the phreatic surface than below it was presumably to account empirically for the contribution of matric suction to shear strength.

Subsequent to the failure, the same section of the dam was analysed using effective stress strength parameters of ϕ' = 33° and c' = 0, which are lower bound values for tests done on the Merriespruit tailings. Updated phreatic surface results were used in this stability analysis, which gave a minimum factor of safety of 1.24. Although a factor of safety of 1.24 may be considered to be somewhat low, the fact that contributions of matric suction to the tailings shear strength were ignored in this case and that a lower bound value for the angle of internal friction was used, the feeling was that the dam was acceptably safe from a slope-stability perspective. However, failure was not due to slope instability. As men-

Fig. 5. View of post-failure scar from inside the failed area, showing the steep sides of the exposed tailings.

Fig. 6. Aerial view of two of the three failures that occurred at the Saaiplaas tailings dam.

tioned earlier, the trigger for the flow slide was overtopping. The real question that remains is why, given an overtopping occurrence, did a large volume of tailings flow out of the dam? In 1992, three slope failures occurred at the nearby Saaiplaas tailings dam. As shown in Fig. 6, none of these failures resulted in catastrophic flow slides, although in one instance the tailings did flow until halted by the wall of the return-water reservoir (Blight 1998). In the other two failures, the post-failure profile was typical of a slope instability, with steep back scarp and bulged toe. The tailings at the Saaiplaas dam were essentially the same as those at Merriespruit, as was the deposition methodology. Why then were the two failures at one dam so localized and immobile, whereas the single breach at Merriespruit resulted in such catastrophic consequences?

Static liquefaction of Merriespruit tailings

Evidence from post-failure investigation

Triaxial tests on undisturbed specimens

As part of the post-failure investigation, a number of consolidated undrained triaxial compression tests were carried out on undisturbed specimens trimmed from block samples taken from the sides of the failure scar inside the tailings dam. Although most of these tests resulted in dilatant behav-

Fig. 7. Stress paths for consolidated–undrained triaxial tests on undisturbed specimens obtained from the Merriespruit tailings dam as part of the post-failure investigation (after Wagener et al. 1998). Axes are shear stress t (= $(\sigma_1' - \sigma_3')/2$) and mean effective stress s' (= $(\sigma_1' + \sigma_3')/2$).

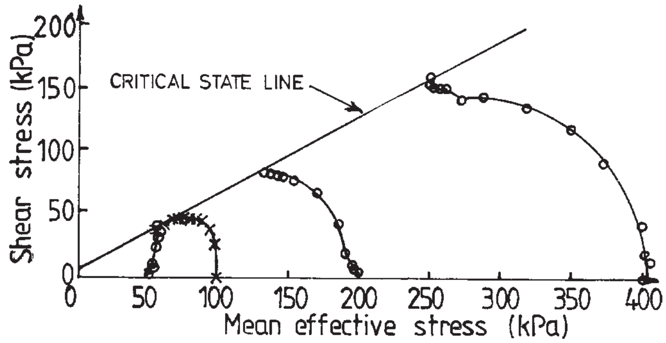
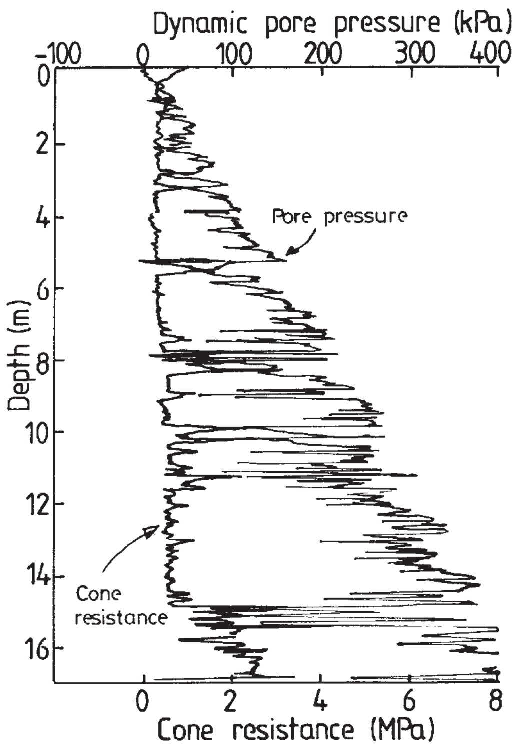
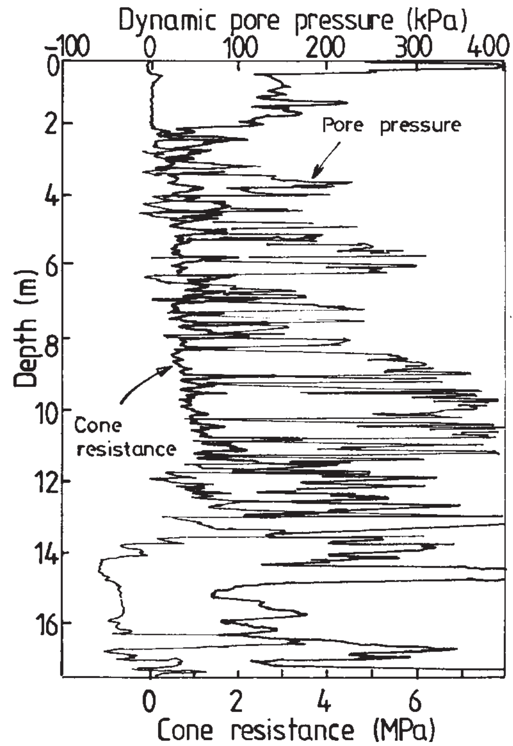


Fig. 8. Piezocone test result for test conducted adjacent to the decant facility (after Wagener et al. 1998).



iour that is so typical of gold tailings in South Africa (Blight 1998), a few tests showed a slightly contractant response to loading. These results are shown in Fig. 7 in terms of a stress path plot using the parameters $s' = (\sigma_1' + \sigma_3')/2$ and $t = (\sigma_1' - \sigma_3')/2$, where σ_1' and σ_3' are the major and minor principal effective stresses, respectively (after Wagener et al. 1997). Although the stress paths do not show completely contractant behaviour, in that they do not move downwards and to the left after reaching a limiting shear stress (with commensurate post-peak softening in terms of shear stress against axial strain), there is no change to dilative behaviour either, despite axial strains of 20% being reached in these tests. Considering that these specimens were obtained from the section of the tailings dam that did not flow (but were in-

Fig. 9. Piezocone test result for test located west of the breach on the middle berm (after Wagener et al. 1998).

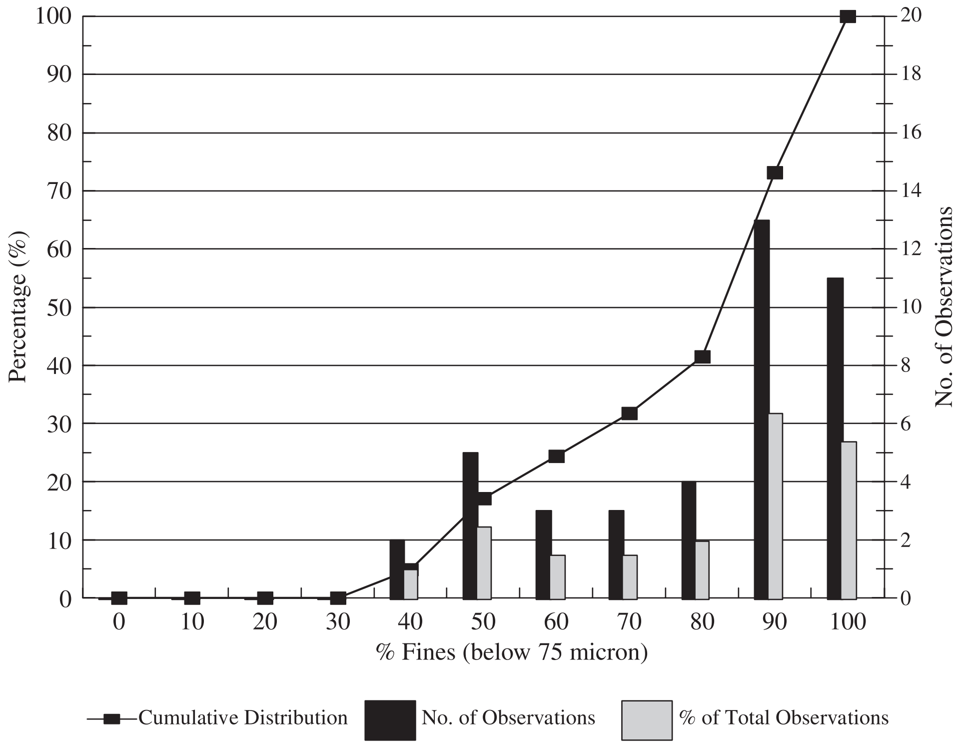


deed at the transition between flow and no-flow), it is not surprising that the specimens did not show pronounced strain softening. Although it is only conjecture, it is possible that the tailings that did flow were in a looser state than the specimens shown in Fig. 7 and thus exhibited truly contractive behaviour upon undrained shearing.

Piezocone testing

Five piezocone tests that were carried out at various locations around the dam as part of the post-failure investigation were reported by Wagener et al. (1998). The piezocone typically used in South Africa does not include a friction sleeve, and results are thus only obtained for end resistance and pore-water pressure (with the sensing ceramic located behind the shoulder of the cone). Two of these tests were conducted to the east of the failure scar on the middle and upper berms, and two similar tests were conducted to the west of the scar. The fifth test was carried out close to the penstock. Figure 8 shows a profile of end resistance q_c and pore pressure u_c for this latter test. As expected, the tailings at this location are very poorly consolidated, with dynamic pore pressures in excess of 300 kPa and cone resistance values of only 500 kPa at depths of as much as 14 m below the tailings surface. Figure 9, which is a profile of end resistance and pore pressure for the test conducted to the west of the breach and on the middle berm, shows end resistance values between depths of about 2 and 10 m which are significantly lower than those in any of the other three piezocone profiles measured on the middle and upper berms. At a depth of 8.5 m, for example, the cone resistance was only approximately 500 kPa. Using the following equation for undrained shear strength s_u,

Fig. 10. Cumulative distribution of the Merriespruit tailings samples showing percent finer than 75 μm.



$$s_u = \frac{q_c - \sigma_{v0}}{N_{kt}} \quad [1]$$

where q_c is the cone end resistance, σ_{v0} is the in situ vertical total stress, and N_{kt} is an empirical cone factor analogous to the bearing capacity factor N_γ, the undrained shear strength s_u is about 23 kPa for an N_{kt} value of 15, which is a typical average value for normally consolidated clay deposits (Senneset et al. 1989). This is an extremely low shear strength for a depth of around 8 m but is consistent with the in situ vane shear tests discussed in the next section. It indicates that the tailings over this depth (from 2 to 10 m) were at a low density and the increase in pore pressure during cone penetration indicates tailings that were not dilative.

Equation [1] was obtained for normally consolidated clay deposits. Its suitability to a mine tailings, which will usually consist of fine sand and silts, is unproven. Tailings are likely to drain much more rapidly than a normally consolidated clay and it may thus be difficult to measure a truly undrained shear strength. If anything, the piezocone is likely to overestimate the undrained shear strength (since some consolidation and thus increase in effective stress might occur in the vicinity of the advancing probe). Konrad and Law (1987) suggest that the value of N_{kt} decreases slightly with increasing plasticity index. For mine tailings, which usually have nonplastic fines, an N_{kt} value of 15 appears to be a good approximation from the data of these authors. Other values for mine tailings reported in the literature are summarized in Table 1.

Table 1. Reported values of the empirical cone factor N_{kt} for mine tailings.

Tailings	N_{kt}	Reference
Copper	15.5±4.5	Mlynarek et al. 1994
Unknown	9–12	East et al. 1988
Uranium	15–19	Larson and Mitchell 1986
Unknown	10.4	East and Ulrich 1989

In situ shear vane tests

At three locations where piezocone tests had indicated low values of penetration resistance, holes were hand augered and vane shear tests carried out. Strydom and Williams (1999) report that in some cases it was possible to forego excavation of the hole because the tailings were so soft that it was possible to insert the vane manually. The results of the vane tests were highly variable, with peak strengths from 16 to 62 kPa and large strain strengths from 3 to 13 kPa. The large strain strengths were often only about one-fifth of the peak values.

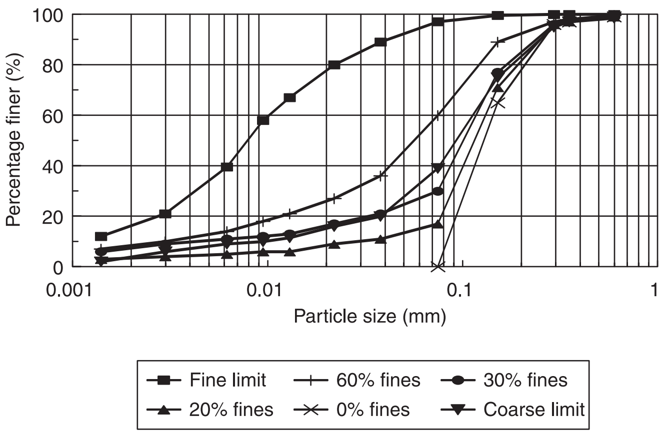
The post-failure investigators concluded that there was indeed poorly consolidated material in the slope of the tailings dam and this material could be expected to flow in the event of disturbance or a removal of support (Wagener et al. 1998).

Laboratory tests to determine steady state line

In a companion paper, Fourie and Papageorgiou (2001) describe in detail the laboratory testing programme that was carried out to define the steady state line (SSL) for Merriespruit tailings. Only salient results are presented here.

At the outset, it was recognised that it would be inadequate to determine an SSL for only one particle-size distribution of the tailings. Figure 10 shows a cumulative distribution of the Merriespruit tailings, gradings in terms of

Fig. 11. Particle-size distribution curves for the four tailings samples tested.



the percent fines (<75 μm), obtained from particle-size distribution tests carried out on samples obtained at various locations around the failure scar during the post-failure investigation. Some samples have as much as 95% finer than 75 μm. In our tests, however, we decided to use a particle-size distribution with 60% finer than 75 μm as our finest tailings sample (M4). Three other particle-size distributions were tested, having 1%, 20%, and 30% less than 75 μm (designated M1, M2, and M3, respectively). The four tailings tested are shown in Fig. 11, and the SSLs that were obtained are shown in Fig. 12. The SSLs in Fig. 12 include an error allowance on void ratio of 0.05 to account for measurement errors during specimen preparation, as discussed by Fourie and Papageorgiou (2001). The results indicate that as the percentage of fines in the specimen increases, a higher relative density is necessary to ensure noncontractive behaviour during undrained shearing (Fourie and Papageorgiou 2001), although the 1 and 20% specimen SSLs are virtually indistinguishable.

Likelihood of in situ tailings being at a void ratio that plots above the SSL

To determine whether the Merriespruit flow failure occurred because potentially strain-softening tailings became unconfined, it is necessary to ascertain the likelihood of the tailings in situ plotting above the relevant SSL. In answering this question, two factors need to be considered. First, the relevant SSL must be defined (in terms of the particle-size distribution of the tailings at a particular location), and second it must be determined whether the in situ void ratio at this location plots above this particular SSL or not. Unfortunately this information could not be obtained for the tailings that failed and flowed and we thus have to restrict our attention to the tailings that did not flow.

Figure 13 shows a cumulative distribution of in situ void ratios determined during the post-failure investigation. These void ratios were calculated from both Shelby tube samples recovered from depth at various locations around the dam and block samples taken from positions adjacent to the failure scar. Figure 13 shows that about 20% of the samples tested had void ratios of about 1.2 or higher. Bearing in mind that any disturbance of the samples during recovery would most likely result in a decrease in void ratio of the sample, this means that at 20% of the locations tested, the in situ void ratios plotted above any of the SSLs determined in this study. Although we did not determine the SSL for samples with more than 60% finer than 75 μm and, as shown in Fig. 10, a large percentage of the samples tested were in fact finer than this, the data in Fig. 12 indicate that as the percent fines increases, the SSL moves downward. Samples with more than 60% less than 75 μm would thus probably have an SSL below that of M4 in Fig. 12.

The results discussed above can be used to estimate the likelihood that tailings adjacent to the failed mass could also have been in a "metastable" state and thus may have flowed if circumstances were conducive to this occurring. If tailings at a reasonable number of locations can be shown to have been in such a metastable state, then a convincing argument can be made for static liquefaction being the cause of the mass flow failure at Merriespruit. Figure 10 shows that about 75% of the 41 samples tested contained 60% or more fines. Using the SSL for M4 tailings shown in Fig. 12 and a mean effective stress of 50 kPa (equivalent to about 3 m depth for a deep phreatic surface), we are interested in in situ void ratios of greater than about 0.84 (i.e., 0.79 + 0.05 as an error allowance), since this would place the in situ tailings above the upper envelope SSL for 60% fines tailings. From the results presented in Fig. 13 this was calculated to be about 61% of the measured void ratios.

With 75% of the samples tested having more than 60% finer than 75 μm and 61% of the measured void ratios lying above the upper envelope of the SSL for tailings with 60% fines, it is clear that a reasonable proportion of the tailings that did not flow may well have been in a potentially liquefiable state. This possibility was checked by looking for locations where both of the above conditions did in fact occur simultaneously. A total of 21 samples satisfied both conditions, which represented 84% of the samples where both the particle-size distribution and in situ void ratio were determined.

Been and Jefferies (1985) introduced the concept of the state parameter Ψ for describing the liquefaction susceptibility of a tailings or sand deposit. The parameter Ψ represents the difference between the in situ void ratio and the void ratio on the relevant SSL at the appropriate mean effective stress. A positive value of Ψ indicates specimens looser than the SSL and negative values of Ψ are obtained for specimens dense of the SSL. A cumulative distribution of Ψ values for the 21 samples that simultaneously satisfied both conditions is shown in Fig. 14. More than 85% of the samples have Ψ values of 0.1 or greater, i.e., a difference between the in situ void ratio and the equivalent value on the SSL of 0.1 or more. This represents a compelling argument in favour of the proposition that at the time of the failure a sufficiently sizeable proportion of the Merriespruit tailings was in a metastable state in situ and potentially liquefiable.

An obvious question that arises in view of the above discussion is why did the tailings that were identified during the post-failure investigation as being in a metastable state not flow, particularly considering the steep slopes surrounding the failure scar? A possible reason is that drainage of the tailings occurred during the failure event. Once the pond of retained water on top of the dam had flowed out of the breach, water within the tailings mass would have begun

Fig. 12. The steady state lines obtained for Merriespruit tailings from undrained triaxial compression tests that showed contractive behaviour. The shaded areas represent an error allowance on void ratio of 0.05 either side of the steady state line.

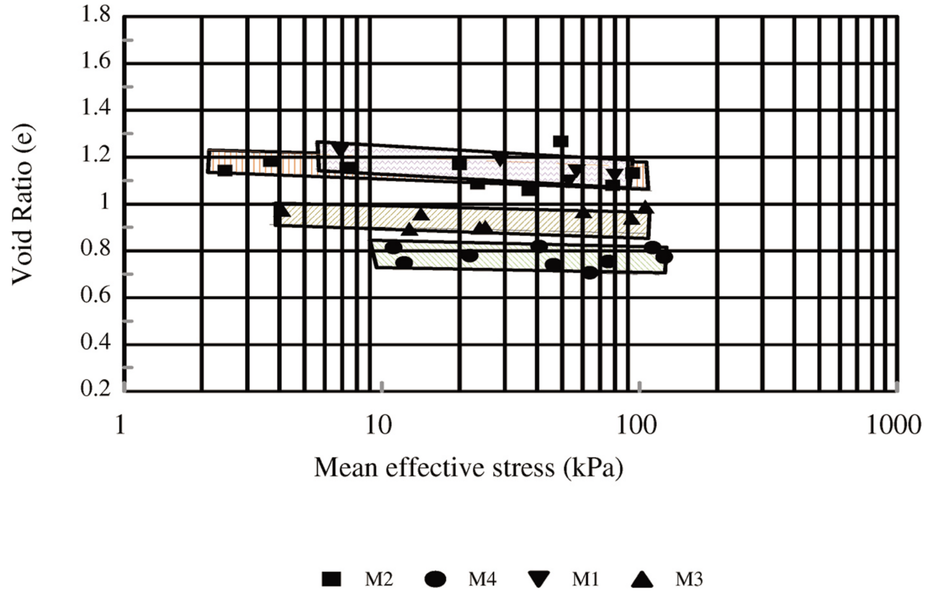
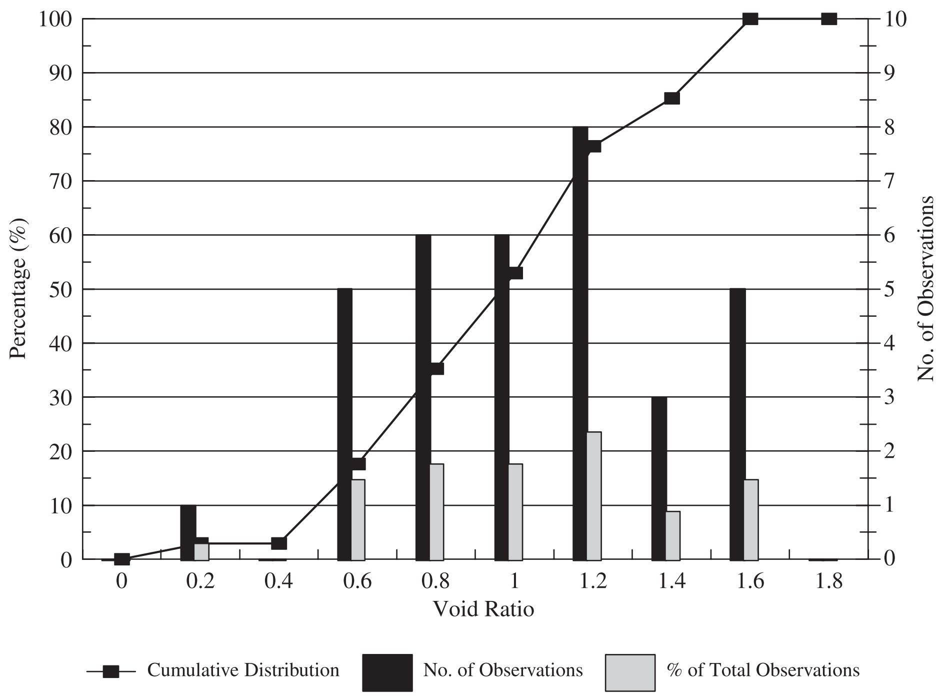


Fig. 13. Cumulative distribution of in situ void ratios obtained during post-failure investigation.

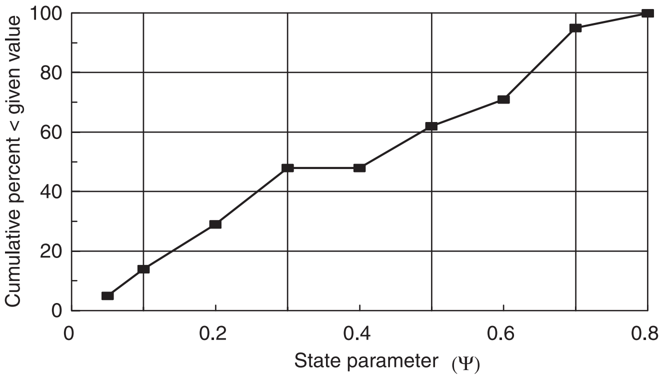


seeping from the sides of the newly exposed tailings surfaces (see the steep sides of the failure scar in Fig. 5). Although this may not have caused consolidation of the tailings, because it remained saturated, it would have the effect of reducing the pore-water pressure within the tailings, perhaps increasing the mean effective stress sufficiently to move the tailings away from a state of incipient failure. Although Strydom and Williams (1999) report relatively low in situ permeabilities of between 1 and 5 m/year, the hydraulic gradient induced by the retrogressive sequence of failures postulated by Wagener et al. (1998) in Fig. 4 would be high and could have the effect of causing some flow of water from the exposed failure surface. This particular argument is purely hypothetical and difficult to prove without measurements of pore-pressure decreases within a tailings body during a flow failure event.

Was the flow failure foreseeable?

In the previous section it has been argued that a significant proportion of the tailings that were not part of the flow failure was in fact in a metastable state at the time of the disaster and it is logical to suppose that the tailings that did

Fig. 14. Cumulative distribution of state parameter Ψ for undisturbed tailings samples obtained during post-failure investigation.



flow were equally or even more unstable. Before discussing the possibility of having foreseen the disaster, it is worth recapping the likely sequence of events, interpreted in the light of the post-failure inquiry together with the results of the laboratory tests summarized in this paper.

Sequence of failure events

In trying to postulate a coherent picture of the sequence of events that occurred on 22 February 1994, it is useful to recount some of the eyewitness accounts reported by Wagener (1997) and Strydom and Williams (1999). Apparently a strong stream of water (originating from the tailings dam) entered the top end (i.e., that nearest the tailings dam) of the village at about 19:00 on the evening of the failure. Water was seen flowing over the impoundment crest in three separate streams, which later became four streams. An employee of the tailings dam operator rushed to the dam and began removing sections from the penstock tower, with a view to increasing the rate of decant of supernatant water from the top of the dam. A second employee, who was near the location of the subsequent breach, saw blocks of tailings toppling from the tailings buttress (probably as a result of erosion undercutting this material). Soon after this, at about 20:00, two loud bangs from the direction of the impoundment were heard, followed by a great deal of water flowing through the village. Although the accounts are somewhat jumbled, it appears that around 20:45 a large crack in the impoundment became visible in the moonlight and soon thereafter the upper 10 m of the impoundment wall collapsed, accompanied by a further loud bang. The village was thereafter engulfed in a wave of tailings and water.

The reported loud bangs may be more important than seems to have been acknowledged in the post-failure investigations. Sasitharan et al. (1994) carried out drained triaxial tests on anisotropically consolidated loose specimens of Ottawa sand, in which the deviator stress was maintained constant while the mean effective stress was reduced (in effect simulating a rise in phreatic surface within a slope). When the specimens failed they found that the velocity of collapse was so rapid that it was impossible to collect detailed data. More relevantly, they report that the specimen reached such a large momentum during collapse that when the loading head (of the triaxial apparatus) hit the restricting nuts, the entire laboratory felt the vibration. Although this was probably largely attributable to the loading system used (i.e., dead weights for application of vertical load), it confirms the instantaneous nature of static liquefaction failures. The sudden collapse of a large volume of tailings (the volume of the Merriespruit flow failure was estimated as 600 000 m^3 (Wagener et al. 1997)) in this manner would be expected to cause a very loud noise, certainly consistent with a loud bang being heard some 300 m away.

A sequence of retrogressive failures (see Fig. 4) as postulated by Wagener et al. (1998) seems to be consistent with the above eyewitness accounts. The berms may have been the first to overtop, causing the tailings buttress to begin to fail. These events on their own would have caused relatively minimal damage and been easily remediated. What happened thereafter is probably crucial. The pond of water on top of the tailings dam (which was proved to have been very close to the northern wall, as postulated in Fig. 4, by satellite photographs taken only 3 weeks before the disaster) began overtopping and eroding the slopes and the tailings buttress. The oversteepened and thus unstable lower slope failed and was washed away. A sequence of relatively shallow failures then occurred due to local oversteepening, with the debris from each of these “subfailures” being washed away by water escaping from the breached pond. At some point, tailings within the dam that were in a metastable state were exposed by this sequence of slumping failures. This material collapsed, generating large positive excess pore pressures and causing a mud flow to develop. The eyewitnesses report that the third loud bang discussed above was soon followed by a wave of tailings and water that engulfed the town. Once again, this is consistent with instantaneous static liquefaction.

In hindsight it appears obvious that the impoundment was in a potentially unstable state. However, as mentioned previously, it is not unheard of for gold tailings dams to exhibit excessive seepage on the outer slopes, although the problem is usually addressed immediately, generally by reducing the volume of stored water, decreasing the tailings tonnage deposited on the impoundment, active dewatering, or constructing a toe buttress. Although an attempt was made to address the problems at Merriespruit, the buttress that was constructed appears to have been relatively insubstantial. The fact that no gold tailings dam in South Africa had previously failed with such devastating consequences probably contributed to the complacent response to the excessive seepage and toe sloughing that occurred.

Conventional stability analyses carried out only a few months before the failure indicated a satisfactory factor of safety (1.34) against slope instability. Based on a traditional geotechnical engineering approach to slope stability, perhaps the perception of the dam as being intrinsically safe can to some degree be excused. It is suggested that many practising geotechnical engineers, if today faced with the information available at the beginning of February 1994, might also come to the conclusion that the tailings impoundment, while not in a satisfactory state, was not intrinsically unsafe. It is only when the concepts of static liquefaction and the collapse surface concept of Sladen et al. (1985) are invoked that a prediction of incipient instability might have been predicted. Such a prediction would have been difficult, if not impossible, without either piezocone profiles or preferably in situ void ratios and a knowledge of the SSL for an appropriate particle-size distribution of the tailings concerned.

The liquefaction and flow of the tailings at Merriespruit are only part of the equation. A trigger mechanism had to develop before the tailings would liquefy. Such a trigger was provided by storage of an excessive volume of water on the top of the impoundment, which caused a breach to develop when a relatively small rainfall event occurred. It is unarguable that the failure would not have occurred if the existing regulations regarding freeboard had been adhered to, and Wagener et al. (1998), in their summing up of the causes of the failure, point to the poor communication that took place within the organisations concerned with the tailings dam operation, which they suggest may have been aggravated by the lack of relevant technical knowledge of those charged with responsibilities beyond their expertise.

In summary, it is suggested that the trigger mechanism that initiated the failure could have easily been avoided, but that the catastrophic consequences that resulted from the erosion of the impoundment wall were, at the time, not foreseeable. Needless to say, that excuse no longer applies to existing impoundments.

Why did a flow failure occur at Merriespruit even though other incidences of gold tailings dam slope instability have never shown this behaviour?

The results presented so far in this paper indicate that much of the in situ tailings were at a void ratio higher than the equivalent value on a relevant SSL and were thus potentially strain softening during undrained loading. While recognising that undrained triaxial compression tests suffer certain important limitations (e.g., the applied total stress path is very limited and specimens are usually sheared from an initially isotropic stress state), they provide the most widely used method for defining SSLs and a vast amount of experience has been obtained using such tests to interpret actual failures. Such tests certainly appear to adequately define a void ratio – mean effective stress relationship below which the material under consideration is highly unlikely to exhibit strain-softening behaviour when loaded undrained. Although the deposition procedure used on gold tailings dams in South Africa utilises a relatively low slurry density (typically 1400–1500 kg/m^3), the approach whereby significant sun-drying of the tailings is allowed prior to subsequent deposition is key to understanding why flow failures have not previously been experienced on gold tailings dams. As illustrated by Blight (1988), the effect of sun-drying is to increase the mean effective stress (by inducing an increased matrix suction), thereby consolidating the tailings more than would be possible under the tailings self-weight alone. When these tailings are subjected to undrained shear, they dilate because the sun-drying-induced consolidation has resulted in an in situ void ratio that is lower than the steady state value. Such dilative behaviour is characteristic of gold tailings, as reported by Blight.

Although the Merriespruit tailings dam was operated in the same way as described above (a cycled deposition programme allowing sun-drying of the tailings), in the area of the breach there were significant deviations from this procedure that resulted in underconsolidated tailings. As discussed in the review of the operational history of the dam, poor pool control resulted in the tailings pond pushing up against the northern wall of the impoundment in the weeks prior to the failure. Tailings deposition in this area thus often occurred subaqueously, first resulting in a very low settled density and second minimising the potential benefits of sun-drying. Even the piezocone test carried out adjacent to the failure scar, in unfailed tailings (Fig. 9), showed extremely low shear strengths (and thus low densities), confirming this scenario.

Significance of Merriespruit tailings dam failure

Aside from the obviously tragic immediate consequences of the tailings dam failure, it had some major positive effects. It caused a major reconsideration of operating procedures on tailings dams in South Africa and contributed in some measure to the development and publication of a Code of Practice for Mine Residue Deposits (South African Bureau of Standards 1998). Although such improvements are laudable, it is important not to lose sight of the fact that the Merriespruit failure was a single, isolated incident and that many dozens of gold tailings dams in South Africa have been successfully and safely operated for close to 100 years.

That said, the stark lessons learned from Merriespruit should not be forgotten. Poor pool control, subaqueous tailings deposition, and inadequate freeboard are all well-known sources of operational difficulties on tailings dams. The Merriespruit failure simply emphasized this point, with tragic consequences.

Concluding remarks

The arguments presented in this paper, which are based on eyewitness accounts of the disaster, together with the results of laboratory tests on both reconstituted and undisturbed tailings specimens and a range of in situ tests carried out soon after the failure (on unfailed areas), paint a clear picture of static liquefaction as the cause of the flow slide that involved 600 000 m^3 of tailings. Static liquefaction was triggered by removal of confinement to a large volume of tailings that were in a metastable state in situ. This trigger mechanism resulted from overtopping and erosion of one wall of the impoundment, principally due to insufficient freeboard provision on the impoundment and poor pool control. Although initially denied by many witnesses, the location of the tailings pool immediately adjacent to the breach location in the weeks immediately preceding the failure was proven by evidence obtained from satellite photographs taken only 3 weeks before the failure. As an aside, one can only wonder whether the true cause of the Merriespruit failure would ever have been established had this critical evidence not been unearthed.

The overtopping water would have caused erosion of parts of the outer confining slope of the impoundment, causing small, localized slip failures due to local oversteepening. With water continuing to flow over the breached crest, these disturbed tailings were washed away, causing further oversteepening of newly exposed tailings. A sequence of progressive slip failures, possibly combined with headward erosion of a gully by the stream of escaping water, eventually exposed tailings material that was in an intrinsically unstable state because its void ratio was higher than the critical value required to prevent the tailings behaving in a contractive (and thus strain softening) manner when loaded undrained.

A large volume of these tailings underwent rapid strain softening, resulting in flow of tailings over a distance of some 3 km. Eyewitness accounts of the sequence of events of the failure concur with the above description, including reports of a number of large bangs, which are consistent with the sudden collapse of a large volume of tailings.

Laboratory tests on reconstituted specimens of the Merriespruit tailings defined a series of steady state lines, depending on the particle-size distribution of the tailings tested. Comparison with measured in situ void ratios of tailings from zones of the impoundment adjacent to the failure scar showed that a large amount of these tailings plotted above the relevant steady state line and were thus likely also in a metastable state. The tailings that did flow were concluded to have been in an even looser state and thus flowed when confinement was suddenly removed. Piezocone tests and in situ vane shear tests carried out adjacent to the failure scar soon after the failure confirmed that there were some zones of the tailings that were very soft and loose. It is not entirely clear why these areas of the tailings impoundment that were found to be in a metastable state did not also flow. It is probably a combination of a small amount of drainage being sufficient to move the in situ tailings away from a state of incipient failure and a balance being reached between potentially contractive zones and the more common dilatant tailings that usually occur on the outer reaches of gold tailings dams finally achieving a state of equilibrium. Once equilibrium was established, drainage of interstitial water from the steep, exposed sides of the tailings scar would have further increased the effective stresses in the tailings body, eventually bringing the remaining mass of tailings to a state of full equilibrium.

A strong argument has been presented in this paper in support of the view that static liquefaction during undrained loading of the Merriespruit tailings was the cause of the catastrophic flow slide that occurred in February 1994. The question that logically follows from this argument is why previous incidents of slope failure or overtopping of gold tailings dams in South Africa did not also result in catastrophic flow-slide failures. It is suggested that the reason for much of the Merriespruit tailings being in an intrinsically unstable state in situ prior to the failure is the same reason that the failure was triggered, i.e., poor management of the tailings impoundment and particularly poor control of the tailings pond. As is now accepted, at the time of the failure the tailings pond was abutting the wall where the breach occurred. Tailings deposition in this area would thus likely have often taken place subaqueously, resulting in a very high void ratio, particularly if these tailings were not subsequently exposed to sun-drying. Laboratory tests showed that specimens prepared using the moist-tamping technique achieved very high void ratios, even after consolidation, and exhibited strongly contractant (and thus strain softening) behaviour when loaded undrained. The void ratios at the start of these laboratory triaxial tests have been shown to be similar in many instances to in situ void ratios of tailings adjacent to the failure scar.

Although the literature on static liquefaction of hydraulically placed fill is vast, the Merriespruit failure provides one of the few case studies where extensive prefailure information has been complemented by comprehensive post-failure investigations that included in situ and laboratory testing. Recognition of the role of static liquefaction in this particular failure should lead to improved tailings impoundment design in the future.

References

Been, K., and Jefferies, M.G. 1985. A state parameter for sands. Géotechnique, **35**(2): 99–112.

Blight, G.E. 1988. Some less familiar aspects of hydraulic fill structures. *In* Hydraulic fill structures. *Edited by* D.J.A. Van Zyl and S.G. Vick. American Society of Civil Engineers, Geotechnical Special Publication 21, pp. 1000–1027.

Blight, G.E. 1998. Destructive mudflows as a consequence of tailings dyke failures. Proceedings of the Institution of Civil Engineers, **125**: 9–18.

Blight, G.E., and Steffen, O.K.H. 1979. Geotechnics of gold mining waste disposal. *In* Current geotechnical practice in mine waste disposal. *Edited by* Committee on Embankment Dams and Slopes of the Geotechnical Engineering Division. American Society of Civil Engineers, New York, pp. 1–52.

East, D.R., and Ulrich, B.F. 1989. The electric piezocone for profiling of mine tailings deposits. *In* Engineering geology and geotechnical engineering. *Edited by* R.J. Watters. A.A. Balkema, Rotterdam, The Netherlands, pp. 35–38.

East, D.R., Cincilla, W.A., Hughes, J.M.O., and Benoit, J. 1988. The use of the electric piezocone in mine tailings deposits. *In* Proceedings 1st International Symposium on Penetration Testing, Orlando, U.S.A., March 1988, Vol. 2, pp. 745–750.

Fourie, A.B., and McPhail, G.I. 1993. Stabilisation of a tailings dam by de-watering. *In* Proceedings of the 3rd International Conference on Case Histories in Geotechnical Engineering, St. Louis, Mo., June 1993, pp. 223–331.

Fourie, A.B., and Papageorgiou, G. 2001. Defining an appropriate steady state line for Merriespruit gold tailings. Canadian Geotechnical Journal, **38**: 697–706.

ICOLD. 1982. Manual on tailings dams and dumps. International Commission on Large Dams (ICOLD), Bulletin 45.

Konrad, J.M., and Law, K.T. 1987. Undrained shear strength from piezocone tests. Canadian Geotechnical Journal, **24**: 392–405.

Larson, N.B., and Mitchell, B. 1986. Cone penetrometer use on uranium mill tailings. *In* Proceedings of the American Society of Civil Engineers Conference on Use of In-situ Tests in Geotechnical Engineering. Blacksburg, U.S.A, pp. 700–713.

Mlynarek, Z., Tschuschke, W., and Lunne, T. 1994. Technique for examining parameters of post-flotation sediments accumulated in the pond. *In* Proceedings of the 3rd International Conference on Construction on Polluted and Marginal Land, London, U.K., pp. 17–23.

Sasitharan, S., Robertson, P.K., Sego, D.C., and Morgenstern, N.R. 1994. State-boundary surface for very loose sand and its practical implications. Canadian Geotechnical Journal, **31**: 321–334.

Senneset, K., Sandven, R., and Janbu, N. 1989. Evaluation of soil parameters from piezocone tests. Transportation Research Record 1235, National Research Council, Washington, D.C., pp. 24–37.

Sladen, J.A., D’Hollander, R.D., and Krahn, J. 1985. The liquefaction of sands, a collapse surface approach. Canadian Geotechnical Journal, **22**: 564–578.

South African Bureau of Standards. 1998. Code of practice for mine residue deposits: SABS 0286 (1998). South African Bureau of Standards (SABS), Pretoria, South Africa.

Strydom, J.H., and Williams, A.A.B. 1999. A review of important and interesting technical findings regarding the tailings dam failure at Merriespruit. Journal of the South African Institution of Civil Engineering, **41**(4): 1–9.

Wagener, F. 1997. The Merriespruit slimes dam failure: overview and lessons learned. Journal of the South African Institution of Civil Engineering, **39**(3): 11–15.

Wagener, F., Strydom, K., Craig, H., and Blight, G. 1997. The tailings dam flow failure at Merriespruit, South Africa, causes and consequences. *In* Proceedings of the Conference on Tailings and Mine Waste '97, Colorado State University, Fort Collins, January 1997, pp. 657–666.

Wagener, F., Craig, H.J., Blight, G.E., McPhail, G., Williams, A.A.B., and Strydom, J.H. 1998. The Merriespruit tailings dam failure — a review. *In* Proceedings of the Conference on Tailings and Mine Waste '98, Colorado State University, Fort Collins, January 1998, pp. 925–952.

Zhang, H., and Garga, V.K. 1997. Quasi-steady state: a real behaviour? Canadian Geotechnical Journal, **34**: 749–761.