A review of geostatic stress measurements in mine tailings

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With the adoption of modern standards, advanced slope stability methods requiring knowledge of the soil stress state are required. An important aspect of stress state is the geostatic stress ratio (K_0). However, measurement of K_0 in mine tailings is not common. To address this, a review was conducted on four investigations where K_0 values in tailings were assessed. The review covered gold, iron ore and oil sands tailings. It was found that for all three sites, the tailings were not in an isotropic stress state. Excluding an alternative view for the oil sands tailings, K_0 values were estimated to be between 0.45 and 0.6. For the gold and iron ore tailings, reasonable correlations were found between the K_0 values obtained from the numerical analysis and those obtained using the relationship proposed by Jaky (1944) . However, for the oil sands tailings, where direct field measurements of K_0 were obtained, poor correlation was noted with Jaky (1944).

INTRODUCTION

With the increasing adoption of modern standards, such as the Global Industry Standard on Tailings Management (GISTM) (Oberle, 2020), there has been a move towards more advanced slope stability assessment methods of tailings dams. Conventional methods simply relied on soil strength characteristics, but these modern methods require knowledge of the soil stress state to be appropriately implemented. One particular aspect of the soil stress state which is of importance is the geostatic stress ratio (K_0).

It has been shown numerous times that the stress history of a soil influences its mechanical response during undrained shear. This is of particular importance for design and safety evaluations of tailings dams. As an illustration, Narainsamy & Jacobsz (2022) demonstrated that for sandy soils, the yield undrained shear strength can vary by a factor of two, depending on whether the sample underwent isotropic or anisotropic consolidation. Doanh *et al.* (1997) further illustrated that even for anisotropic stress states, the degree of anisotropy (represented by the magnitude of the K_0 value) has a significant influence on the mechanical behaviour of loose, sandy soils during undrained shear.

The value of K_0 reflects the in-situ stress state in the soil. K_0 values near unity imply that no or very little shear strength has been mobilised in the soil, while values deviating substantially from unity imply that a significant amount of shear is mobilised in the soil, potentially placing the stress state close to failure or instability. The consideration of triggers leading to instability should therefore take cognisance of the in-situ stress state in the soil and hence the value of K_0.

Knowledge of the in situ K_0 value is therefore critical for robust designs. However, even with this knowledge, measurement of K_0 in mine tailings remains limited. In fact, in their theme lecture, Jamiolkowski *et al.* (1991) noted that the assessment of the horizontal effective stress remains a sort of black hole in soil engineering. In the authors' view, this statement remains relevant to this day, especially when it comes to mine tailings.

To address this, a review was conducted on four investigations where an attempt was made to evaluate the in-situ geostatic stress ratio in mine tailings. Three of the investigations only considered a numerical approach, supported by field and laboratory test data, while the fourth investigation considered a direct field measurement of K_0. The four investigations covered three tailings materials: gold tailings, iron ore tailings and oil sands tailings.

GEOSTATIC STRESS RATIO

The geostatic stress ratio (K_0), also referred to as the coefficient of lateral earth at rest pressure, is defined as the ratio between the horizontal effective stress (σ'_h) and vertical effective stress (σ'_v) as shown in Equation 1.

$$K_0 = \sigma'_h / \sigma'_v \qquad [1]$$

The subscript '0' is important as it describes the conditions under which this stress ratio is applicable. For true K_0 conditions, there must be zero lateral strain. This is a relevant parameter in many geotechnical problems found in nature; as it is usually assumed that soil deposits have become consolidated under one-dimensional (zero lateral strain) conditions (Wroth, 1984).

It is important to distinguish between the types of earth pressures. In addition to the at-rest lateral pressure coefficient (K_0), bounds exist for the lateral earth pressure coefficient, with the lower bound being the active earth pressure coefficient (K_a) and the upper bound being the passive earth pressure coefficient (K_p). The key difference between these types of earth pressures lies in the movement imposed on the soil body to mobilise these pressures. Active pressures typically involve removal of confinement and expansion of the soil. Passive pressures typically involve increasing compression of the soil, while at-rest pressures involve no change in lateral geometry of the soil. Significant work has been done in the areas of active and passive earth pressures (e.g. Rankine, 1856). The scope of this study is limited to at-rest pressures, which is the situation most relevant to the in-situ stress state in tailings dams.

Laboratory measurements

There have been many attempts to measure K_0 in the laboratory. Early tests involved the use of large devices such as the lateral earth pressure meter used by Tschebotarioff & Welch (1948). Although these tests proved useful, testing was difficult and time consuming.

Since then, many researchers have successfully used modified oedometer tests to investigate the geostatic stress ratio in soils. This seems intuitive as 'at rest' conditions (i.e. zero lateral strain) are already imposed during the standard oedometer test. The challenge is to measure the radial stress that is being generated during consolidation. This has been successfully done by many researchers on a wide variety of soils. For example, Becker *et al.* (1987) proposed a method whereby interpreting conventional oedometer test data using work per unit volume as a criterion could be used to determine the in-situ effective stress (both vertical and horizontal). More recently, Teerachaikulpanich *et al.* (2007) proposed a modified oedometer apparatus, covered in a pressurised chamber and fitted with pore pressure transducers. Both approaches seemed to provide reasonable results of K_0.

An alternative to the oedometer test is the triaxial test. The biggest difference between the two tests is the rigidity of the radial boundary. In the oedometer test, there is a rigid boundary, typically a steel ring. In the triaxial test, however, there is a flexible boundary, typically a latex membrane. Unlike the oedometer test where significant modification is required, with the addition of radial callipers and local strain instrumentation, most modern triaxial test equipment is capable of performing K_0 consolidation. In the triaxial cell, the soil sample is consolidated to a target effective stress while maintaining a condition of zero radial strain. This can either be performed by leading with the loading ram and adjusting the cell pressures or leading with the cell pressure and adjusting the loading ram, depending on the response from the radial calliper (Lade, 2016). In the authors' experience, the latter approach seems to be the most popular amongst current researchers.

Although it is useful to be able to measure K_0 in the laboratory, a significant limitation lies in the accessible soil states that can be achieved in the laboratory, specifically related to those states that exist in the field. For example, Reid (2021) showed that in some cases, specifically related to sub-aqueous deposition, tailings can exist in a significantly looser state in the field than can be created in the lab. This is an important point as it has been shown that for sandy soils, density has a significant influence on K_0 (e.g. Obrician, 1969).

Other concerns include stress history which will be discussed later. It is for this reason that field measurements of K_0 are of particular importance for mine tailings.

Field measurements

As with the laboratory measurements of K_0, there have been many attempts to measure K_0 in the field. The obvious advantage of field measurements over laboratory measurements is that both the fabric and stress history of the soil are captured. These are aspects of soil state that have been particularly difficult to capture in the laboratory, especially for mine tailings (e.g. Chang *et al.*, 2011).

Field measurements are generally categorised into intrusive methods and non-intrusive methods. Intrusive methods represent those where probes are inserted into the ground, and although care is taken during testing, unavoidable disturbance is usually caused. Non-intrusive methods are those where there is limited disturbance. Examples of intrusive methods include the installation of piezometers for hydraulic fracturing, total pressure cells, dilatometers and the K_0 stepped blade. Currently, the only non-intrusive method is the self-boring pressuremeter test. An overview of each relevant method is described below but a more detailed description of these methods, as well as their associated advantages and disadvantages is provided by Hayat (1992).

Hydraulic fracturing

This method is based on the assumption that when an increase in fluid pressure is applied at a point in a soil, fissures in the soil develop in a direction perpendicular to the minor principal stress. Once the fissures have developed, the fluid pressure begins to drop and the fluid pressure at which the fissures close is equal to the in-situ pressure acting perpendicular to the fissures (which is assumed to be numerically equivalent to the at-rest lateral earth pressure). This concept was first proposed by Bjerrum & Anderson (1972) and has been successfully implemented in clays by many researchers (e.g. Lefebvre *et al*. 1991). When performing hydraulic fracture tests, it is important to consider the geometry of the piezometer used to measure the pore pressures, as well as to allow sufficient time between installation of the piezometer and start of the test such that installation disturbances are minimised.

The hydraulic fracturing method is not used on tailings dams as the mechanism of hydraulic fracturing (although limited in extent in the test method) is generally one of the failure modes that are considered when performing risk assessments. This test is therefore typically only conducted on sensitive clays, and specifically clays where the geostatic stress ratio is less than unity (i.e. the minor principal stress is in the horizontal direction).

Flat dilatometer testing

The flat dilatometer is a push-in device similar to a cone penetrometer. The dilatometer itself consists of a stainless steel blade with a thin, flat, circular, expandable steel membrane on one side (Marchetti, 1980). The dilatometer is pushed into the ground typically using a penetrometer rig and at selected depths; the membrane is inflated by means of pressurised gas. Readings are taken during the inflation of the membrane and there are several correlations available to estimate soil properties from these readings. When using the dilatometer, it is important to correct the pressure readings for membrane stiffness and seating pressures. These corrections are fairly standard and are included in most standard test methods. Most modern flat dilatometers are designed to fit with standard cone penetration rods so no additional rods or rigs are required. However, it seems that despite a recent increase in cone penetration testing, flat dilatometer testing remains limited in South Africa.

Self-boring pressuremeter
As discussed earlier, the self-boring pressuremeter is deemed to be the least intrusive of the current field methods available to estimate K_0 in the field. However, this statement is dependent on the skill of the operating contractor; it is common to over-bore which causes significant disturbance. The most common method of testing using the self-boring pressuremeter it to drill a borehole to just above the desired test level. The self-boring pressuremeter is then lowered into the hole and pushed ahead to the target depth while pumping jetting fluid (Shuttle *et al.*, 2021). This approach was used in the Muskeg River mine investigation which will be discussed later. Once at the target depth, the pressuremeter is inflated and, as with the dilatometer, the pressure readings are recorded. There is some uncertainty associated with interpretation of the pressure readings from the self-boring pressuremeter, specifically related to lift-off pressures. Additional details regarding these uncertainties and methods to address them can be found in Shuttle *et al.* (2021).

Performance of the field test methods
All the above-mentioned field methods tend to perform well in sensitive clays with low to moderate overconsolidation ratios (OCRs). A good example of this is the study performed by Hamouche *et al.*, (1995) where three difference field measurement approaches were used to investigated the K_0 value at three different sensitive clay sites in Eastern Canada. In the study, the three devices that were used were: the Cambridge-type self-boring pressuremeter, the flat dilatometer and hydraulic fracturing. For each site, the three methods were implemented and the results compared. Excellent correlation was observed at the Berthierville and Louiseville sites where OCRs varied from 1.1 to 1.4 and 2.4 to 4.7, respectively. Some scatter was noted for the Mascouche site, specifically below 5 m depth. This scatter was attributed to the higher OCRs at the site with varied from 4.9 to 5.5. Overall, it was concluded that all three field methods work in sensitive clays with low to moderate OCRs.

Methods to estimate K_0

There are several methods available to estimate the in situ K_0 value of a soil. Perhaps the most well-known method to estimate the geostatic stress ratio is that proposed by Jaky, (1944), shown in Equation 2, where only knowledge of the critical state friction angle (φ') is required.

$$K_0=1-\sin\varphi' \quad [2]$$

Although Equation 2 is used frequently in the literature, the work where this relationship was proposed was published in Hungarian and there are numerous researchers who have translated the original text and found some discrepancies with common interpretations. For example, Handy (1983) noted that Equation 2 was actually only published by Jaky (1948) and that the equation proposed in the Hungarian text in Jaky (1944) contained additional terms and was not strictly developed to define the lateral stress ratio under a broad, level, loaded area. Rather, the equation as derived in 1944 defined a stress ratio at the centre of a pile of sand whose surfaces are inclined at the angle of repose, φ'. This statement has been supported by other researchers (e.g. Mesri & Hayat, 1993; Michalowski, 2005). Tschebetarioff (1951), at times, went as far as to say that the satisfactory numerical correlation between the measured values of lateral pressures of soils at rest and the measured values of their physical properties still awaits further theoretical and experimental research. However, despite these limitations, the method still seems to provide reasonable results for many soils, notably as reported by Kulhawy & Mayne (1982).

In their research, Kulhawy & Mayne (1982) conducted a rigorous review of available laboratory test data on a wide range of soils to investigate the K_0-OCR relationship. They reviewed over 170 soil types (including clays, silts and sands) and found that Equation 2 seemed to provide reasonable results. No relationship was found between K_0 and any other index parameter such as liquid limit, fines content, etc. What was also found was that K_0 is significantly influenced by stress history. Following on the work by Schmidt (1966), who noted that K_0 should be related to the effective friction angle, Kulhawy & Mayne (1982) proposed an equation to estimate K_0 based on the effective friction angle and the OCR of the soil. Although this was a useful finding, and seemed to fit the experimental data well, several researchers have shown that there are limitations to this approach (e.g. Lefebvre *et al.*, 1991; Hamouche *et al.*, 1995).

In fact Jefferies *et al.* (1987) showed some experimental data on Beaufort sea clays where no influence of OCR on K_0 was noted.

Clearly, there is still some uncertainty regarding the estimation of K_0 using index parameters, especially for overconsolidated soils. This was highlighted by Tschebetarioff (1951) who noted (in terms of natural clays) that further progress cannot be expected to be made from a continuation of classical approaches which attempt to relate lateral earth pressures only to the ultimate shear strength of the soil at failure. It was noted that too many other factors influence the results and that direct field measurements supplemented with improved laboratory testing apparatus are essential. It is likely that this sentiment is still appropriate today and that it is applicable to sandy soils such as mine tailings.

GEOSTATIC STRESS RATIOS MEASURED IN TAILINGS

Although it is known that mine tailings are likely not in an isotropic stress state in the field, there are surprisingly few published studies on K_0 values in tailings in the literature. For this study, four such investigations are reviewed. Of the four, three are based on numerical analyses to estimate K_0 while only one is based on direct field measurements.

Cadia North Tailings Storage Facility

Cadia Valley Operations is a gold/copper mining and processing complex, 25 km south of the town of Orange in New South Wales, Australia. On 9 March 2018 a slump occurred at the North Tailings Storage Facility (NTSF). Fortunately, the slump did not lead to loss of containment and no injuries were reported. However, this was seen as a serious occurrence and an independent investigation was conducted (Jefferies *et al.*, 2019). As part of the investigation, numerous soil samples were collected from site and a rigorous laboratory testing, field testing and numerical modelling campaign was carried out.

The NTSF is a modified centreline facility and was approximately 60 m high at the time of the failure. The particle size distribution of the material is shown in Figure 1. With a D_{50} of approximately 60 μm, the Cadia gold tailings are the finest of the three tailings assessed and classifies as a sandy silt of low plasticity (MS) according to the British Soil Classification System (BSCS) (BSI, 1999).

As part of the numerical investigation, the tailings material was modelled using the NorSand soil constitutive model. NorSand is a powerful numerical model based on critical state soil mechanics principles and has been shown to be effective in capturing a wide variety of soil behaviour, in particular, collapse of loose sandy soils which was of interest in this study (e.g. Jefferies, 1993; Jefferies & Shuttle, 2005). The constitutive model was calibrated using the results from the laboratory tests.

Two and three-dimensional numerical models were analysed using the finite difference software Fast Lagrangian Analysis of Continua (FLAC) (Itasca, 2015). To incorporate the stress history during the life of the facility, the NTSF was constructed in layers in the numerical model. Using satellite images and construction records, the model was constructed in several lifts and consolidation was allowed to take place. This process was validated by comparing the numerical results with nearby monitoring data such as surveys, drain readings and piezometer readings and generally good correlation was observed.

Based on the interpretation of the cone penetration test data, an initial K_0 value of 0.7 was specified for the tailings at deposition. However, during consolidation and subsequent raising, this value had reduced to approximately 0.5 in the region where the slump occurred. Unfortunately there were no additional direct field measurements of K_0 conducted to verify this value. Using the critical state friction angle of 36° determined from triaxial testing, Equation 2 results in a K_0 value of 0.40.

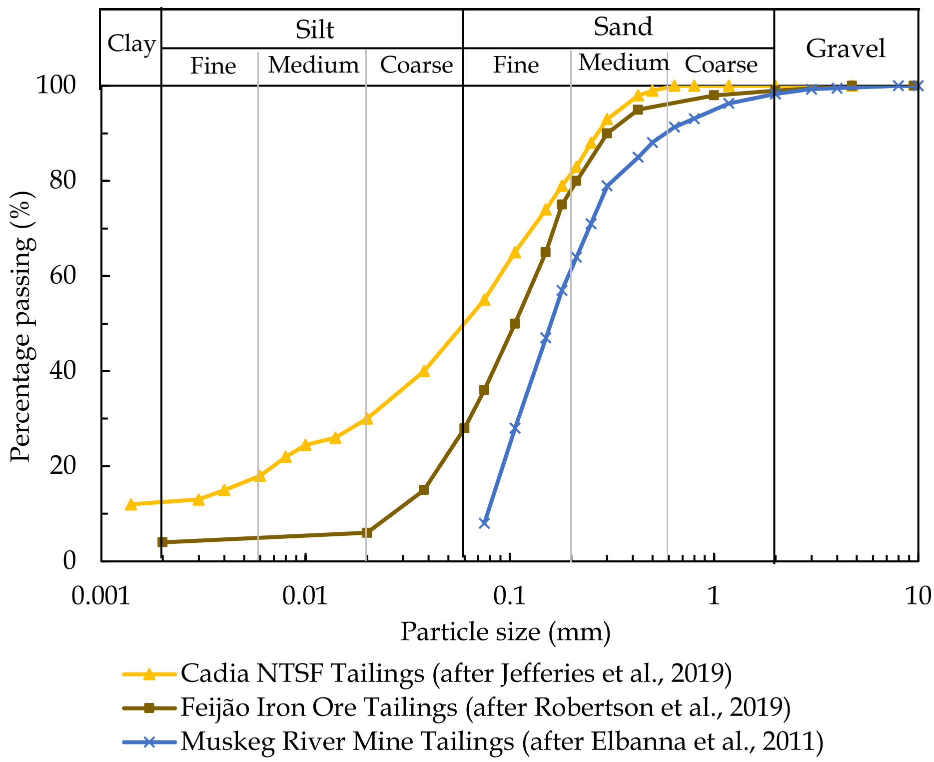

Figure 1. Particle size distributions of the material considered in the investigations.

Feijão – Independent Investigation

The Córrego de Feijão Iron Ore Mining and Processing Complex is located 9 km North-East of the town of Brumadinho in Minas Gerais, Brazil. On 25 January 2019, the Feijão tailings dam B1 failed. The ensuing mudflow resulted in catastrophic environmental damage and at least 250 deaths. This was a unique failure in the sense that the actual moment of failure was captured on video (Chadwick, 2019) which allowed for increased scrutiny of the failure. An independent expert panel was set up to investigate the cause of the failure (Robertson *et al.*, 2019). As was done with the NTSF failure, numerous samples were obtained from site and a rigorous laboratory testing, field testing and numerical modelling campaign was carried out.

The Feijão tailings dam was dormant at the time of the failure, with deposition last occurring in 2016. During operations, the dam was constructed using the upstream construction methodology and the tailings was hydraulically deposited. The dam was approximately 80 m high and had a crest length of 700 m. An average particle size distribution of the Feijão iron ore tailings is shown in Figure 1. With a D_{50} of approximately 105 µm, the Feijão iron ore tailings have as intermediate grading of the three tailings and classifies as a sandy silt of intermediate plasticity (ML) according to the BSCS (BSI, 1999).

As was done for the NTSF investigation, the NorSand soil constitutive model was used for the Feijão investigation. The constitutive model was calibrated using the results from the laboratory tests. Two and three-dimensional numerical models were analysed using FLAC and the facility was constructed in layers in the numerical model. Using satellite images and construction records, the model was constructed in several lifts and consolidation was allowed to take place. This process was validated by comparing the numerical results with nearby monitoring data such as surveys, drain readings and piezometer readings and generally good correlation was observed.

Since there was video footage of the failure, there was reasonable certainty regarding the shape and position of the failure surface. This, combined with the extensive laboratory and field testing regimes meant that a three-dimensional numerical model of the failure could be developed. Following a similar approach to Olson (2001), it was assumed that with yield undrained strength ratios mobilised along the slip surface, a factor of safety of 1.0 against failure would be achieved.

Since a three-dimensional stress state existed in the model, by comparing the yield undrained strength ratio defined in terms of vertical effective stress, and the yield undrained strength ratio defined in terms of mean effective stress, it was possible to determine the average K_0 value along the slip surface in the model at the time of the failure. A value of 0.45 was determined using this approach. Again, unfortunately there were no additional direct field measurements of K_0 conducted to verify this value. Using the critical state friction angle of 34° determined from triaxial testing, Equation 2 results in a K_0 value of 0.44.

Feijão – Computational Investigation

In a rather surprising development, about a year after the failure, and only a few months after the release of the Robertson *et al.* (2019) report, a second independent investigation was commissioned into the failure. This investigation focused on a numerical approach to the failure. Although the same field testing results were used, the second investigation conducted a completely new laboratory testing plan (CIMNE, 2021).

In the study, the Clay and Sand Model, also a critical state model, was used as the soil constitutive model (Yu, 1998). This model was further extended to include a visco-elastic component so that creep could be modelled. This was important as creep was listed as one of the causes of failure in the Robertson *et al.* (2019) report. Interestingly, the CIMNE (2021) came to completely different, and conflicting, reasons as to the cause of failure. A detailed discussion on these differences is outside the scope of this study. However, it is remarkable that with all the monitoring data available, and video footage of the failure, that there is no consensus within the tailings community regarding the cause of failure, or at very least, the ability to rule out one of the conflicting causes.

What is of interest for this study is the K_0 value used in the numerical analysis. Unfortunately, there is not as much detail provided in the report as was found in the Jefferies *et al.* (2019) and Robertson *et al.* (2019) reports. However, it was made clear that a K_0 value of 0.5 was used for the tailings. Again, as was noted earlier, it is unfortunate that no direct field measurements of K_0 were attempted which could have been used to validate the assumption in the numerical model. Using the critical state friction angle of 34° determined from triaxial testing, Equation 2 results in a K_0 value of 0.44.

Muskeg River Mine

The Muskeg River mine is an oil sands mine located 70 km North of Fort McMurray in Alberta, Canada. The mine has an external tailings facility (ETF) where oil sands tailings are deposited. The ETF is an upstream constructed, ring-dyke facility. The tailings in the dyke's outer shell are deposited in wide shells and are compacted with D7 or D8 bulldozers, while the tailings inside of the dyke are hydraulically deposited (Shuttle *et al.*, 2021). Unlike the previous two sites described, no failure was reported at this site and this investigation was conducted as part of routine operations. In addition, this is the only site in this study where direct field measurements of K_0 were attempted.

The particle size distribution was obtained from Elbanna *et al.* (2011) and is shown in Figure 1. With a D_{50} of approximately 150 μm, this Muskeg River mine oil sands tailings are the coarsest of the three tailings and classifies as a poorly graded sand (SP) according to the BSCS (BSI, 1999).

As part of the investigation, 14 self-boring pressuremeter tests were conducted at two locations, for a total of 28 tests. Since it is known that there are some uncertainties with the interpretation of self-boring pressuremeter data, a numerical analysis was conducted to improve the interpretation of the test results. A spreadsheet-based iterative forward modelling approach was used where cylindrical cavity expansion theory was implemented, with the Non-Associated Mohr-Coulomb failure model selected as the soil material model.

A summary of the results from the self-boring pressuremeter tests are shown in Figure 2a. Tests where over-boring was noted have been highlighted and were not considered in the analysis due to increased disturbance during drilling. For the oil sands tailings, two K_0 lines are suggested. The first is a simple line where K_0=0.6 is suggested to match best with the material without a complicated geological history. A second line is then drawn which also represents this K_0 value of 0.6 but special consideration is given to what is termed 'locked-in' stresses. These stresses are believed to have been imposed during the compaction process and subsequent wetting-drying and freeze-thaw cycles. The initial offset to the K_0 line is linked to a horizontal effective stress of 60 kPa which is noted to match the track bearing pressures of the D8 bulldozers, used for compaction purposes.

An alternate view is also presented in Figure 2b where the results are shown to with a K_0 value of 1.0. There does seem to be some merit to this approach but the fact that many of the data points do not agree with this assumption is a concern which is perhaps why it was presented as an alternative approach and not the main approach. Using the critical state friction angle of 32° determined from triaxial testing, Equation 2 results in a K_0 value of 0.47.

Summary

Four investigations where the in-situ K_0 values in mine tailings were estimated were reviewed. A summary of the key parameters from the review are shown in table 1. In addition to the estimated K_0 values from each investigation, the K_0 values estimated using the equation proposed by Jaky (1944) have been provided, as well as details of the sites, the tailings material and the K_0 estimation method.

With the exception of the alternate view of the Muskeg River mine oil sands tailings where a K_0 value of 1.0 was proposed, the estimated K_0 values range from 0.45 to 0.6. The K_0 value estimates using the equation proposed by Jaky (1944) correlate well for the gold and iron ore tailings but not so well for the oil sands tailings. Regardless, it is clear that there is evidence to suggest that the in-situ stress state of mine tailings is not isotropic.

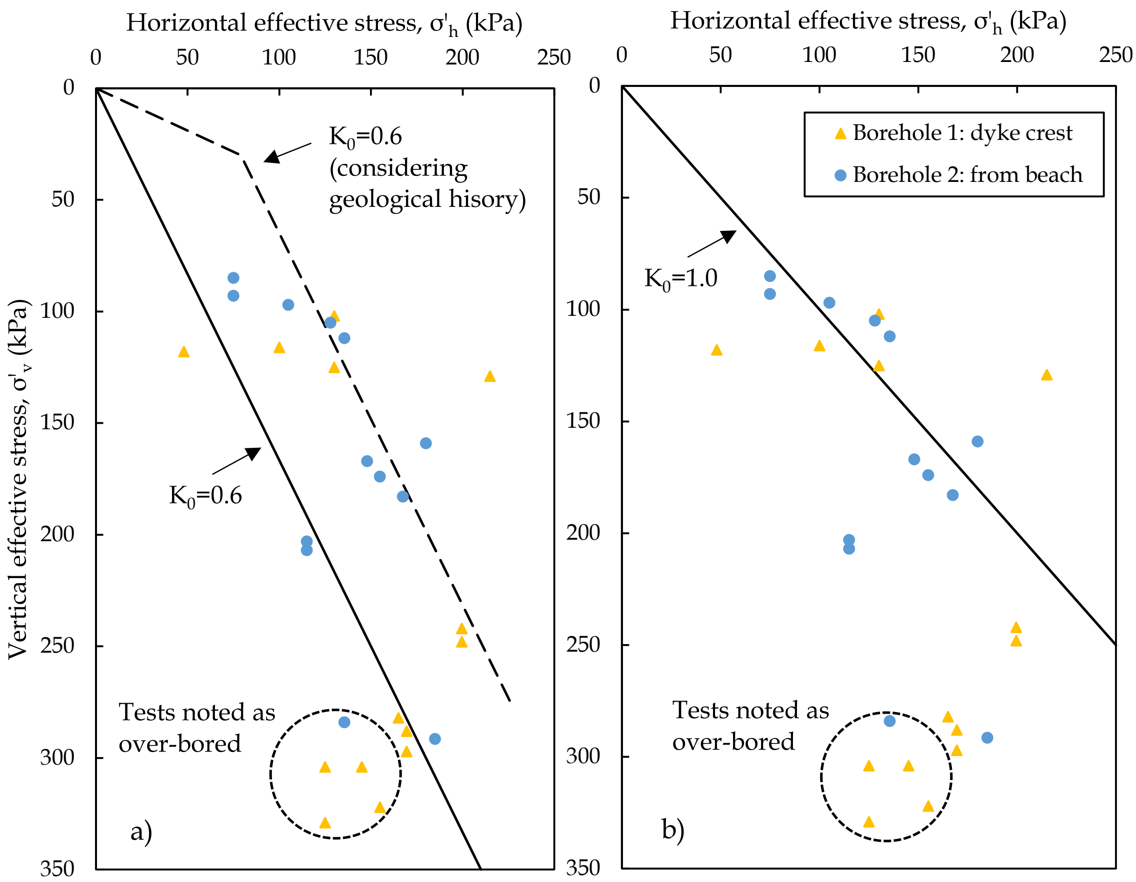

Figure 2. Geostatic stresses for the Muskeg oil sands tailings: a) considering a geostatic stress ratio of 0.6; b) considering a geostatic stress ratio of 1.0 (after Shuttle et al., 2021).

Table 1. Summary of key parameters of the four investigations reviewed

Site	Material	Estimation method	Estimated K_0	K_0 using Jaky (1944)	Reference
Cadia NTSF	Gold tailings	Numerical back-analysis of failure	0.5	0.40	Jefferies *et al.* (2019)
Feijão Dam B1	Iron ore tailings	Numerical back-analysis of failure	0.45	0.44	Robertson *et al.* (2019)
Feijão Dam B1	Iron ore tailings	Numerical back-analysis of failure	0.5	0.44	CIMNE (2021)
Muskeg River Mine TSF	Oil sands tailings	Self-boring pressuremeter tests	0.6 considering geological history or 1.0	0.47	Shuttle *et al.* (2021)

CONCLUSIONS

It is well known that the in-situ stress state influences the mechanical response of loose mine tailings during undrained shear. This in turn has a direct influence on the results of designs and safety evaluations. A key component of the in-situ stress state is the geostatic stress ratio (K_0) and there are many methods available to measure K_0 in-situ. However, measurement of K_0 seems to be rare in practice.

In this study, four investigations were reviewed where an attempt was made to evaluate the in-situ geostatic stress ratio in mine tailings. Three of the investigations only considered a numerical approach, supported by field and laboratory test data while the fourth investigation considered a direct field measurement of K_0. The four investigations covered three tailings materials: gold tailings, iron ore tailings and oil sands tailings. Based on the review conducted, the following was found:

- On all three sites, it is clear that the mine tailings considered were not in an isotropic stress state.
- With the exception of an alternative view on the oil sands tailings, where a geostatic stress ratio (K_0) of 1.0 was proposed, the K_0 values were estimated to be between 0.45 and 0.6 across the three materials.
- Comparisons of the estimated K_0 value from the investigations to the estimated K_0 value obtained using the equation proposed by Jaky (1944) showed fairly reasonable results for the gold and iron ore tailings. However, with no direct field measurements to validate the estimates, it is unclear how accurate these estimates are.
- For the oil sands tailings, where direct field measurements were made, poor correlation was found between the estimated K_0 value from the investigation and the K_0 value estimated using the equation proposed by Jaky (1944).
- Care should therefore be taken when using the equation proposed by Jaky (1944) for mine tailings. This is especially true for cases where no direct field measurements of K_0 have been made to evaluate the accuracy of Jaky's (1944) method for a particular site.

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