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Design of tailings embankments over soft and very soft subgrades

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Abstract

Over the last 5 years, Ausenco's tailings team has worked on several tailings projects involving designing and constructing tailings facilities over the soft and very soft subgrades. The type and the size of the facilities varied. Still, most involved the construction of 50–70 m high embankments/structural zones overlying soft and very soft soil deposits. The construction of such big facilities over soft subgrades warrants special care when it comes to the modelling of embankment-induced pore pressures and the non-linear effect of effective stresses on the angle of shear resistance.

The current practice of the design of tailings embankment slopes on top of soft subgrades is based on the 'undrained strength analysis', which combines the 'B-bar' methodology for the assessment of embankment-induced pore pressures with SHANSEP (stress history and normalized soil engineering properties) methodology for the undrained shear strength and total analysis slope stability. Embankment construction over a soft foundation creates a temporary, transient condition of excess pore pressure within the soft subgrade. Due to low permeability, the excess pore pressure within the clay cannot readily dissipate. This produces a non-steady state situation with elevated pore pressures and lower safety factors, and it could lead to slope failure.

In this paper, an alternative, effective-stress-based methodology of slope stability analysis that combines the finite element method (FEM) and limit equilibrium method (LEM) is proposed. The FEM portion of the methodology is used for modelling the embankment excess pore pressures, while the LEM is used for the effective-stress-based stability calculations. The FEM considers the dilatancy law of the soft subgrade material, while the LEM accounts for the pressure dependency of the angle of shear resistance. In this way, the methodology covers the full spectrum of effective stresses, from the low values (where the angle of shear resistance typically attains higher values) to very high values corresponding to the ultimate state of the embankment.

Keywords: *slope stability, staged construction, soft ground behaviour, effective stress analysis of undrained processes*

1 Introduction

The construction and the design of tailings facilities over the soft and very soft subgrades is often faced with stability challenge almost regardless of their type. With reduced construction times, most challenges are imposed by the construction of high rising rate embankments which often involve raising of 50–70 m high containing embankments over a span of 8–10 years.

When discussing stockpiles, waste rock, and tailings facilities, their mode of failure is controlled by the behaviour of the foundation as much by the properties of the tipped material (Bishop 1973). As modern tailings dams are usually built either of compacted sand, rockfill, or well compacted, semi-dry filtered tailings, the subgrade conditions are usually the important controlling factor for the stability of slopes.

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Discussion of soft subgrades encompasses not just soft and very soft clays with 'undrained shear strengths' (the quotes emphasise the fact that the undrained shear strength is behaviour rather than intrinsic property of the material) of less than 20 kPa, but any subgrade material the in situ strength of which cannot support the fill (e.g. loose, extremely weathered residual materials in tropical and sub-tropical regions). This broader definition includes a variety of materials that may present challenges in construction of tailings dams, such as soft saprolites, and historic tailings. Proper assessment and treatment of these subgrades are essential to ensure the stability and longevity of the structures built upon them. Depending on the type of tailings storage facilities, the exterior slopes of these embankments on competent subgrades typically range from 2H:1 V for rockfill embankments for conventional facilities to 4H:1 V and flatter for structural zones of dry-stack tailings. Therefore, construction of soft subgrades may require considerably flatter slopes and, consequentially, either larger volumes of fill materials or larger footprints of the facilities (Vick 1990). A combination of high production rates and low-grade ores producing large volumes of soft and liquefiable tailings and a relatively rapid rates of rising (often 5–10 m/year) on soft, water saturated subgrades exacerbate this problem, by inducing high excess pore pressures which may easily compromise the stability of containing dams on soft subgrades.

The current design codes (Canadian Dam Association 2017; International Council on Mining and Metals, United Nations Environment Programme, and Principles for Responsible Investment 2020; Australian National Committee on Large Dams 2019) prescribe a minimum factor of safety of 1.3 during the operations, 1.5 after closure, and 1.1 during the accidental seismic events. This value of safety factor may give a false feel of safety because, depending on the severity of failure consequences, the projects may require much higher standards of strength and load estimation. Estimation. This can be readily demonstrated by a simple fact that just 15% of shear strength overestimation and similar underestimation of shear loading may easily produce an unstable slope (Equation 1).

$$F_s = \frac{0.85 \cdot \tau_{res}}{1.15 \cdot \tau_{load}} = 0.74 \cdot \frac{\tau_{res}}{\tau_{load}} = 0.74 \cdot 1.3 = 0.96 < 1.0 \quad (1)$$

where:

τ_{res} = shear resistance

τ_{load} = shear loading.

Earthworks-based solutions such as staged construction and stabilisation using toe berms/shear keys often present the most economical and realistic mitigation measure to produce a slope with a satisfactory Factor of Safety (FoS). The main purpose of the slope design within the context of tailings storage facility design is then to develop a safe schedule of maximum lift heights and intermediate fill construction delay periods which satisfies the FoS set out by the codes. This schedule determines the safe rate of embankment raising and, eventually, the size of the facility. This paper addresses the design approaches that can help in optimising these measures.

2 Conceptual description of the problem

Rockfill dams constructed on soft clay foundations, such as those found in Canada's clay belt (Quigley 1980), are prone to stability issues due to the generation of excess porewater pressures and low shear strength of the underlying clay. A critical failure mechanism is the formation of a wedge-like sliding body within the dam and foundation, which can be divided into distinct zones based on their geotechnical behaviour and role in potential instability.

The wedge-like sliding body typically consists of 3 primary zones: the active zone, the passive zone, and the sliding plane. The active zone forms beneath the dam's upstream shoulder, where the applied load from the rockfill induces high shear stresses and excess pore pressures in the soft clay. This zone experiences downward and outward movement, driven by the dam's weight and the low permeability of the clay, which hinders drainage and reduces effective stress. Although in this zone the shear resistance comes in part from the strong rockfill, in the underlying clays, with hydraulic conductivities of around 10^{-9} m/s, the excess pore pressures can persist, increasing the risk of failure.

The passive zone, located near the downstream toe, resists the sliding motion. Here, the clay experiences compression and lateral displacement, often leading to heaving as the soil is pushed upward. The sliding plane – a critical interface between these zones – is a curved or planar surface within the soft clay where shear strength is mobilised to its limit. This plane typically follows the weakest layer, often at the clay's interface with the dam or bedrock.

Effective monitoring with piezometers and inclinometers, combined with staged construction and drainage systems like prefabricated vertical drains, can mitigate risks by controlling pore pressures and enhancing stability.

The construction of embankments on top of a soft foundation produces the outward lateral thrust which results in an increase of shear stress at the base of the potential sliding body. As explained, at the same time, when the rate of loading is relatively high, this loading also causes the build-up of excess pore pressures in the soil foundation, limiting the mobilisation of shear resistance. Both factors detrimentally affect the embankment's stability.

One of the key questions in this analysis is to establish when the soil will react in a fully undrained manner. In practice, that typically corresponds to the situation time where the time for drainage is less than the time that needed for 99% dissipation of excess pore pressures (Equation 2).

$$t \leq 0.01 \cdot \frac{H_{dr}^2}{c_v} \tag{2}$$

where:

H_{dr} = maximum drainage distance

c_v = coefficient of consolidation.

As the soft clays usually have coefficients of consolidation ranging from 5–6 m^2/yr, this practically means that the 20 m thick layer of clay will be subject to undrained loading whenever the construction and/or deposition time left for drainage is less than 0.5 year.

3 Characterisation of soft soil behaviour

As explained, the characterisation of soft soil behaviour for the undrained stability analyses goes well beyond shear strength characterisation. As the shear strength of granular, non-cohesive materials depends only on the effective stresses, establishing excess pore pressures is as important as establishing the load induced stresses.

In embankment design, accurate characterisation of the soil is critical for ensuring both short-term and long-term stability. Soft clays are characterised by low shear strength, high compressibility, and low permeability, typically with hydraulic conductivities of 10^{-9} to 10^{-10} m/s. All these properties play important roles in the stability and lead to significant challenges, including the generation of excess porewater pressures under embankment loading, which can reduce effective stress and trigger failures, whereas permeability and compressibility are crucial for analysis of the dissipation of construction induced excess pore pressures. Two key aspects of soft clay behaviour – dilation and the non-linear shear failure envelope – play pivotal roles in understanding and predicting the actual build-up of excess pore pressures and embankment performance.

Dilation – the tendency of soil to a positive change of volume during shear – significantly influences soft clay behaviour under embankment loading. In overconsolidated clays, dilation occurs as soil particles rearrange under shear stress, generating negative pore pressures that temporarily increase effective stress and shear strength. However, soft clays tend to contract under the action of shear stress, increasing positive pore pressures and reducing effective stress, which heightens the risk of bearing capacity failure or slope instability. The stress ratio, q/p', governs the generation of pore pressures during undrained shear. At low ratios, soft clays contract, increasing positive pore pressures, which reduce effective stress and shear strength, heightening the risk of failure in embankments. As the ratio approaches the critical state, normally consolidated clays generate significant positive pore pressures. In slightly overconsolidated clays, higher ratios may induce limited dilation, producing negative pore pressures that temporarily enhance stability. The internal friction angle (20–30˚) influences pore pressure response by resisting particle rearrangement.

Higher friction mitigates contraction, reducing positive pore pressure generation. However, low permeability limits drainage, sustaining high pore pressures.

Dilative behaviour can be quantified using triaxial tests, where the dilation angle is derived from volumetric strain changes. In design, accounting for limited dilation in soft clays necessitates conservative stability analyses and mitigation measures like staged construction or prefabricated vertical drains (PVDs) to accelerate consolidation.

The prediction of the embankment-induced pore pressures is an old geotechnical problem (Skempton 1954). Lerouil et al. (1988) provided great insight into the behaviour of soft clays under the embankments. Based on their observations of pore pressures under the embankment B at Saint Alban, Lerouil et al. (1990) managed to identify 2 phases of behaviour during the embankment construction: the first, during which the increases in pore pressure are relatively low, and the second, during which the increase of pore pressure approximately equals the increase in total stress. In practical design, the stages should be quantified by Equation 3:

$$\bar{B} = \frac{\Delta u}{\Delta(\gamma \cdot H)} = \begin{cases} 0.6 - 2.4 \cdot \left(\frac{z}{D}\right)^2, for\ H < H_{crit} \\ 1.05 \pm 0.15, for\ H \geq H_{crit} \end{cases} \tag{3}$$

where:

H_{crit} = critical height of the embankment which causes yielding in the underlying material.

H = height of the embankment

Δu = pore pressure

γ = unit weight of the material.

Another very important point is the variation of the excess pore pressure under the embankment: the excess pore pressure under the centre of the embankment differs from the one under the slopes.

In typical engineering practice, the shear resistance of granular materials is characterised using linear Mohr–Coulomb law (Equation 4):

$$\tau = c' + \sigma'_n \cdot \tan\phi' \tag{4}$$

where:

τ = shear stress

σ'_n = normal effective stress.

Where the Mohr-Columb hypotheses applies to most engineering problems in certain situations, the linear Mohr-Columb law based on fixed cohesion intercept and constant angle of shear resistance (term consistently used in this paper; the angle of shear resistance encapsulates the effects of both friction and dilatancy (Maksimovic 1989) becomes a serious limitation as it does not correspond to reality over a wide range of stresses (de Mello 1977). As normal stress increases, the envelope curves, reflecting the predominant influence of stress-dependent frictional resistance in the overall behaviour. This non-linearity complicates stability analyses as traditional linear models overestimate strength at low stresses, leading to unconservative designs. Advanced constitutive models, such as the modified Cam-Clay model, capture this non-linear behaviour by incorporating stress history and critical state parameters.

The shear strength of soft clay is not a linear function of normal effective stress, as assumed in linear Mohr–Coulomb model. Dropping the Coulomb hypothesis of linear relation between shear strength and the effective stress, we keep the Mohr's hypothesis which stipulates that shear failure takes place along a surface over which the normal effective stress, σ'_n, and corresponding shear stress, τ, have a distinct functional relationship (Equation 5):

$$\tau = S(\sigma'_n) \tag{5}$$

The functional relationships, indicated by letter S in Equation 5, have different formats and use different parameters, but the most important for the discussion presented here is that they prescribe the non-linear relation between the intrinsic shear strength of the granular material and the effective stress.

In the case of soft clays and silts, there is substantial evidence of the non-linearity of shear strength envelope over a wide range of effective stresses (Perry 1994). At low effective stresses, fine-grained materials exhibit higher friction angles and practically zero (or at least very small) cohesion. The implications of this non-linearity at small effective stresses and corresponding errors are well documented (Day 1996; Atkinson & Farrar 1984). The errors using the linear Mohr–Coulomb envelope can be considerable at small stresses relevant to the stability of shallow landslides but without major consequences. However, at large stresses corresponding to foundation soils which needs to provide support to ~70–80 m high tailings embankments (and normal stresses of approximately ~1.5 MPa) they can be catastrophic.

Summarising the results of experimental findings, Baker (2004) set out the general functional relations of non-linear Mohr envelopes $S(\sigma'_n)$. The two most important principles are that $S(\sigma'_n)$ is positively semi-definite and convex (i.e. it has negative curvature or the angle of shear resistance that decreases with increasing effective stress).

Based on the above considerations, embankment design on soft clay requires integration of dilation and non-linear shear behaviour into stability analyses. Finite element modelling (FEM), calibrated with field and laboratory data, can simulate pore pressure generation and dissipation. Techniques like staged construction, PVDs, and lightweight fill materials mitigate risks by controlling pore pressures and enhancing strength gain. By thoroughly characterising soft clay properties, engineers can ensure safe and cost-effective embankments in challenging geotechnical environments.

4 Current state of practice

Current design of staged construction of embankments on soft subgrades relies on the undrained strength analysis (USA) method, described by Ladd & Foott (1974). It couples a B-bar methodology for modelling of pore pressure increase with SHANSEP (stress history normalised engineering parameters) methodology for shear strength modelling.

The undrained shear strength, s_u, of the subgrade is directly proportional to the effective stresses prevailing before the imposition of external loading. The construction of embankments over soft foundations produces higher shear stresses in the ground and a temporary, transient excess porewater pressures within the subgrade.

In B-bar method the change in pore pressure (the excess pore pressure) is directly proportional to a change in vertical stress, i.e. Equation 6:

$$\Delta u = \bar{B} \cdot \Delta\sigma_v \tag{6}$$

where $\bar{B}$ is a material's overall pore pressure coefficient. Although it's usually assumed as 1, it is effectively an additional parameter that depends not just on loading, but on other factors such as mineralogy and stress history.

The undrained strength component is usually modelled using the USA method (Ladd & de Groot 2003). The USA method is a ‘ϕ=0’ total stress analysis characterised by the following steps:

1. Ladd & Foott (1974) introduced the concept of SHANSEP which provides the relation between effective stress and undrained shear strength. The undrained strength is expressed as a function of the current vertical effective stress. It is expressed by Equation 7:

$$\frac{s_u}{\sigma'_{vc}} = S \cdot OCR^m \tag{7}$$

   where the value of a normally consolidated undrained strength ratio, S, depends on the failure mode, and m typically ranges from 0.75–0.80. s_u represents the undrained shear strength, σ'_{vc} is the vertical effective stress of consolidation and OCR is the overconsolidation ratio (OCR=1 in case of normally consolidated clays).

2. Ladd & Groot (2003), note that the definition of ‘undrained strength’ depends on the type of the analysis. For bearing capacity calculations, the s_u corresponds to the peak value on Mohr circle. For stability analyses with a method of slices, the s_u corresponds to the maximum shear stress available on the failure plane. As demonstrated by Ladd & Groot (2003), these differences may not be trivial and can easily reach 15%.

3. The design value of ‘undrained strength’ must be adjusted for strain compatibility, the intermediate stress effect (‘b-effect’), and the 3D effects (which are often ignored in practice). These adjustments may easily reach $\pm$15%.

4. Further adjustments are required for the selection of anisotropic undrained strength parameters for non-circular stability analysis. These adjustments require quite a substantial testing program which, as a recommended minimum, comprises anisotropically consolidated triaxial compression tests, triaxial extension and direct simple shear tests (CK_0U TC, TE, and DSS, respectively). This is rarely, if ever, done in practice.

5. Ladd & de Groot (2003), recommended averaging as a way to account for the adjustments. They demonstrate that the highly plastic undisturbed Canadian quick clays produce the normally consolidated strength ratio, S ranging from 0.185–0.225.

Despite above shortcomings, SHANSEP and B-bar represent a significant advancement in analysing and addressing this critical engineering challenge. A fundamental, tacit assumption of this method is that embankment construction does not alter the in situ effective stresses or the existing shear strength of the foundation soil. This assumption may not hold, for instance, in cases susceptible to static liquefaction. Furthermore, as highlighted, the ostensibly straightforward USA formulation necessitates an elaborate characterisation and adjustment process that is seldom fully implemented in practice. In addition, if the anisotropy of undrained shear strength is not explicitly modelled (which is rarely done in practice), the total stress methodology does not account for the directional dependence of shear strength, while the effective based shear strength is inherently directional. Consequently, its restricted application can engender a misleading perception of safety.

5 Alternative methodology

An alternative, effective-stress-based methodology of slope stability analysis described in this paper combines FEM and limit equilibrium method (LEM). The finite element component of the method estimates the build-up of the excess pore pressures in the subgrade, which are then used as an input into the LEM-based slope stability calculations.

5.1 Modelling of excess pore pressures

The FEM part of the method uses the hardening soil (HS) model (Schanz et al. 1999), which combines elements of the Duncan & Chang (1970) and critical state mechanics models to incorporate key features of soft soil response to monotonous loading:

- development of normalised stress ratio, τ/σ' with shear strain, γ (Wood et al. 1994)
- plastic straining with shear hardening, through dilatancy law, $D = \frac{d\epsilon_{vol}^{pl}}{d\gamma^{pl}}$
- effective-stress-dependent elastic stiffness (Janbu 1967)
- pre-consolidation or, equivalently, volumetric plastic yielding
- Mohr–Coulomb failure criterion.

Phenomenological explanation of the excess pore pressure build-up starts from the analysis of volumetric strains. Under undrained loading conditions, the total volumetric deformation, which is comprised of the plastic and elastic volumetric strains is 0, $\Delta\varepsilon_{vol} = 0$. As the elastic volumetric strain is directly related to the increase of the effective compressive stress, i.e. Equation 8:

$$\Delta\varepsilon_{vol}^{el} = \frac{\Delta p - \Delta u}{K} \tag{8}$$

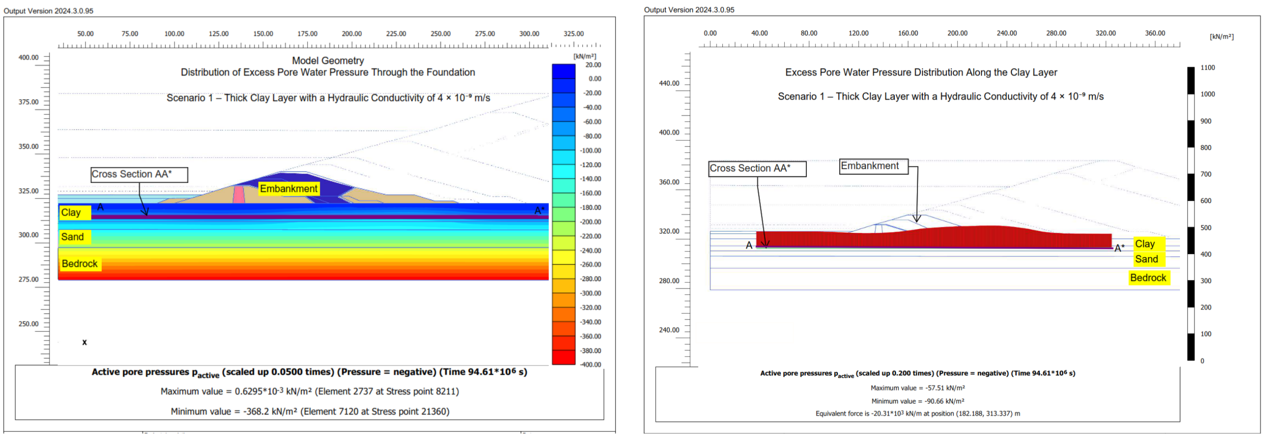
where K is the bulk modulus this means that the excess pore pressure will be equal to Equation 9:

$$\Delta u = \Delta p + K \cdot D \cdot \Delta\gamma^{pl} \tag{9}$$

This indicates that the excess pore pressure has 2 components: one that is purely due to increase of all around pressure, Δp, and the other which is related to the dilatancy and yielding of soil. This is in broad compliance with findings documented in literature (Lerouil et al. 1990).

One added benefit of FEM methodology is that it allows modelling of partial consolidation. It doesn't require the assumption of fully undrained conditions which in certain situations may be oversimplistic and unrealistic. Depending on the value of parameters, the HS model can capture the full gamut of excess pore pressures under undrained monotonic compressive loading, from a relatively limited amount to full liquefaction.

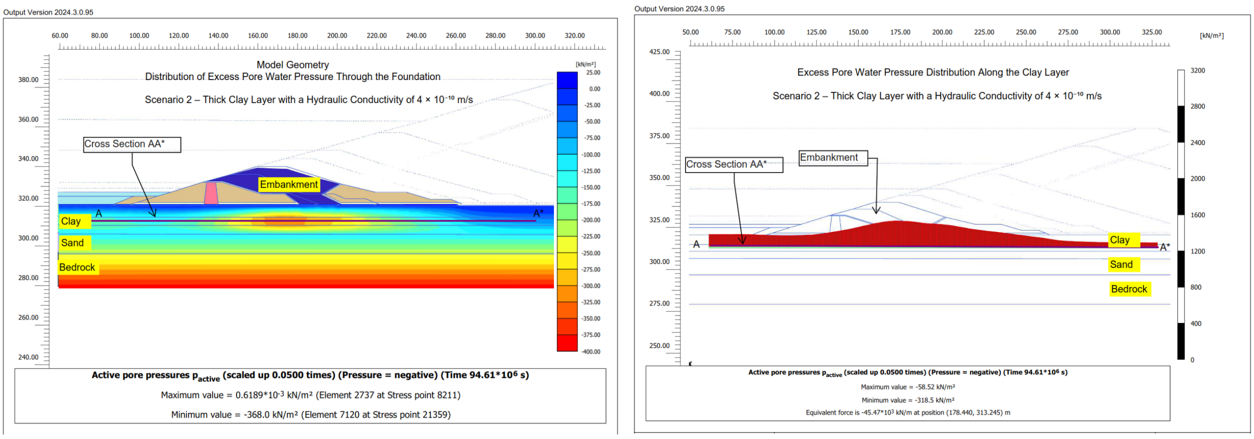
Figure 1a illustrates the induced porewater pressure beneath the embankment, while Figure 1b presents the porewater pressure distribution along the clay layer at cross-section AA*. These results correspond to scenario 1, which considers a thick clay layer with a hydraulic conductivity of 4 × 10^{-9} m/s.



(a) (b)

Figure 1 (a) Model geometry: scenario 1; (b) Excess porewater pressure distribution along clay layer: scenario 1

To evaluate the influence of permeability on excess pore pressure generation and dissipation, a second scenario was analysed using a lower hydraulic conductivity of 4 × 10^{-10} m/s. The corresponding results are presented in Figures 2a and 2b.



(a) (b)

Figure 2 (a) Model geometry: scenario 2; (b) Excess porewater pressure distribution along clay layer: scenario 2

As the results show, FEM enables assessment of both spatial and temporal distribution of excess pore pressures. The excess pore pressure distribution is captured in all its entirety, and the model manages to capture key aspects, including the effect of stress ratio, and partial consolidation. The single advantage of the HS model is that all the parameters are derivable from triaxial and oedometer tests, either directly or by curve fitting. The key shortcoming of the model is that it assumes a constant angle of shear resistance. This aspect is resolved by employing the limit equilibrium method.

5.2 Modelling of shear strength

Conventional limit equilibrium methods are geotechnical procedures of analysing slope stability by examining the equilibrium of a soil mass prone to gravity-driven downslope movement. These 2D analyses assume plane strain conditions and that shear strengths along a potential failure surface follow linear (Mohr–Coulomb) or non-linear relationships with normal stress. An FoS, representing the ratio of available shear resistance to that required for equilibrium, is calculated for an assumed slip surface, with a value greater than 1.0 indicating stability and less than 1.0 suggesting instability; this factor is assumed constant along the entire surface. The most common approach, the method of slices, divides the soil mass into vertical sections, but different versions exist, potentially yielding varying safety factors due to differing assumptions made to achieve static determinacy and the fact that some methods don't satisfy all equilibrium conditions.

As stated by Baker & Garber (1978), the calculation of the FoS requires 2 functions: the equation of the potential slip surface, and the normal stress distribution along this surface. The main power of LEM lies in its focus on the 3 most important aspects of slope stability – search for the critical slip surface and the distribution of normal and pore pressures which determine the effective normal stresses controlling the shear strength along the potential slip surface. Once the pore pressure increase in the subgrade has been estimated, these pore pressures are used as an input into the standard, limit equilibrium stability analysis. The procedure continues by seeking a critical slip failure surface with a minimum FoS using the effective-stress-based soil parameters, $\phi(\sigma')$, to characterise the strength of the subsoil. The resulting FoS, therefore, captures both important aspects of soft subgrade's behaviour: the propensity towards build-up of positive pore pressures, and non-linearity of shear strength.

Figure 3 shows the results of stability analysis for the excess pore pressures presented in Figure 1. Note the line of hydrostatic pressure which corresponds to the pore pressure right after the construction of the embankment. The procedure employed a search for an optimised slip surface, which significantly departs from the circular shape. The failure envelope corresponds to a standard de Mello's power curve (Perry 1994), although this methodology can be employed for others form of envelopes (Duncan & Wong 1983; Maksimovic 1989. The methodology presented here is based on the effective stresses fully, and it directly accounts for the directionality of shear resistance in the ground. As the effective normal stresses depend on the orientation of the slip surface, the modelling shear resistance follow these changes. This is achieved naturally and it does not require introduction of the additional assumptions or functions.

The long-term FoS, of course, corresponds to the situation with fully dissipated excess pore pressures. As is well known (Bjerrum & Bishop 1960), these conditions are never critical for the embankment but they still need to be checked (Figure 4).

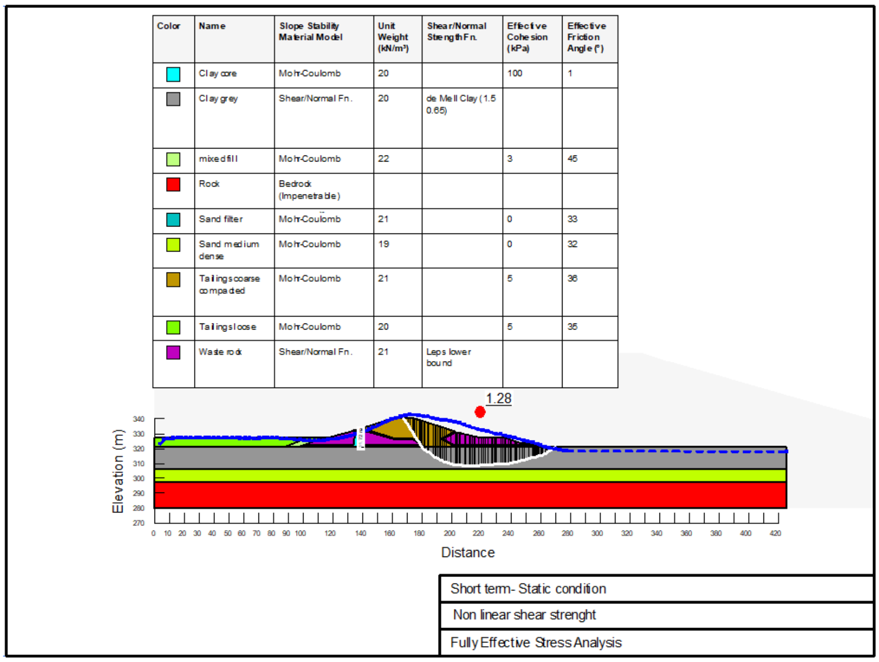


Figure 3 Factor of Safety corresponding to the excess pore pressures. Short-term conditions, fully effective stress analysis

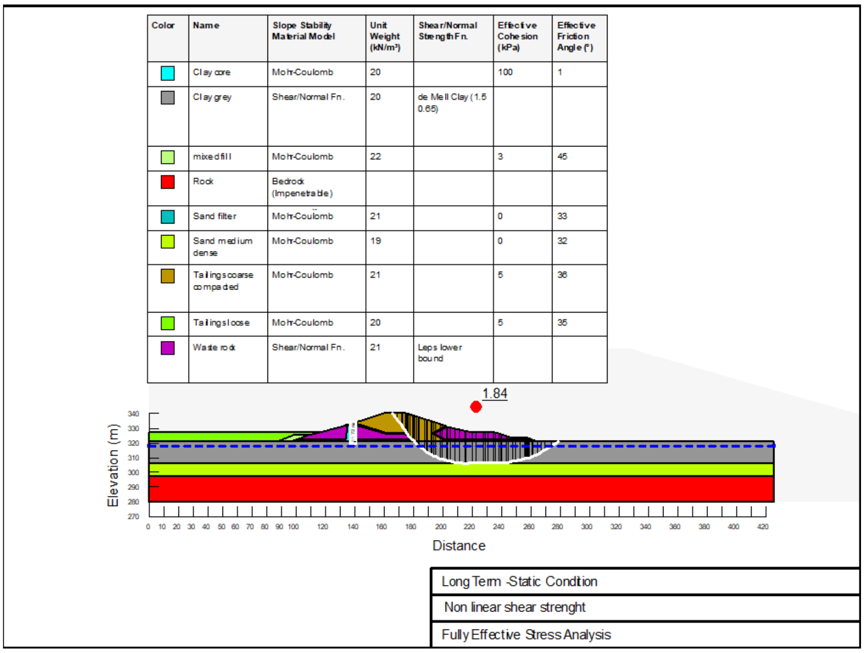
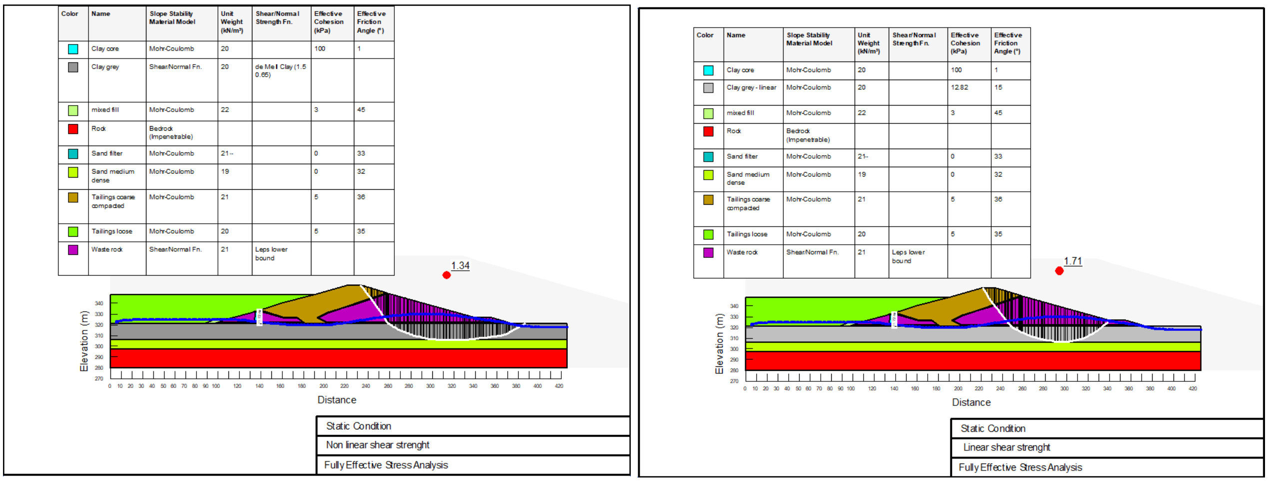


Figure 4 Factor of Safety corresponding to fully dissipated excess pore pressures. Long-term conditions, fully effective stress analysis

Finally, graphs on Figure 5a and 5b demonstrate the effect of the shear failure envelope non-linearity on the FoS. The linear failure envelope was derived from the same set of data as the non-linear failure envelope, but just from the limited set of shear stresses which corresponded to 50, 100 and 200 kPa. The minimum FoS were produced for the same set of excess pore pressures. Based on presented results, it is obvious that non-linearity of shear failure envelope affects not just the value of FoS, but also the shape of the critical slip surface. Obviously, using the linear Mohr–Coulomb envelope can lead to significant overestimation of the safety conditions and unconservative design.



(a) (b)

Figure 5 (a) Factor of Safety (FoS) corresponding to the non-linear failure envelope; (b) FoS for the linear Mohr–Coulomb failure

6 Conclusion and recommendations

The paper proposed and demonstrated the use of an alternative, effective-stress-based method that could be generally used particularly in the design situations which involve the interplay of undrained (or quick) loading and subsequent dissipation of pore pressures (such as staged construction design, for example). The method combines the FEM and LEM; the finite element part of the method is used for an estimation of the embankment-induced build-up of the excess pore pressures, while the limit equilibrium is subsequently used for slope stability calculations, search of the critical slip surface and the minimum FoS.

The finite element part of the method presented in this paper uses the HS model to estimate the build-up of excess pore pressures and consolidation analysis to estimate their subsequent dissipation, Depending on the rate of loading, this process may be analysed simultaneously; one of the important aspects of this analysis is that this analysis accounts for partial yielding that may occur during this process (at least to the first approximation).

The excess pore pressures obtained by the FEM are used as an input into the second step of the method which utilises LEM to obtain the critical slip surface and spatial distribution of normal and, more importantly, effective stresses. The LEM part of the method is based on non-linear shear failure envelope and it intrinsically accounts for the directionality of soil resistance to sliding.

It is our belief that the above methodology deserves to be considered and be used in conjunction with the existing B-bar–SHANSEP methodology in resolving one of the most important aspects of the design of the tailings storage facilities over soft subgrades which is the physical stability of the containing embankment.

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