

STATIC LIQUEFACTION IN THE CONTEXT OF STEADY STATE/CRITICAL STATE AND ITS APPLICATION IN THE STABILITY OF TAILINGS DAMS

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Abstract. Mining operations produce large quantities of tailings that must be conducted and stored safely in a technical framework that reconciles economic restrictions and environmental sustainability. Unfortunately, the history of tailings deposits is marked by episodes of catastrophic failures that have caused many victims and have cost enormous material losses. This is confirmed by the recent catastrophic collapses of tailings dams that occurred in Canada (Mount Polley), Australia (Cadia) and Brazil (Samarco and Brumadinho), where the tailings that flowed downstream severely affected the environment and, in the Brazilian case, caused many casualties. Due to this alarming empirical evidence left by the mining industry worldwide, there is international concern about the stability of tailings dams and a demand to build these deposits safely from every point of view. In this scenario, the correct evaluation of the actual shear strength of the deposited tailings is crucial. The critical state soil mechanic (CSSM) and/or the steady state of deformation provide the conceptual framework for evaluating the ultimate strength mobilized by particulate materials. In the context of a performance-based design, these states are discussed, and based on them, procedures and a flow index, I_f , are presented to analyze the physical stability of tailings.

Keywords: Tailings, Liquefaction, Steady State.

1 Introduction

Mine tailings are generated from mineral ore processing and correspond to the waste material that remains after the valuable ore has been extracted from the processed rock. The expansion of the world economy and the increase in the world population has produced significant growth in the utilization of mineral resources. Therefore, mineral extraction has increased notably, as shown in Fig. 1 (Baker et al., 2020). Since the beginning of this century, there has been a sustained and strong increase in the extraction of metals and minerals, which ultimately has resulted in a substantial rise in mine residues (tailings), that must be managed and stored safely. Additionally, it is necessary to mention that in most mines, the ratio between the valuable mineral and waste material is

decreasing, increasing the generation of tailings. Indeed, the annual global generation of tailings is currently of the order of eight billion tons (<https://worldminetailingsfailures.org/>). The current world demand for minerals automatically implies the need to build tailings deposits with a large storage capacity, which means designing, building, and operating tailings dams of essential dimensions in length and height.

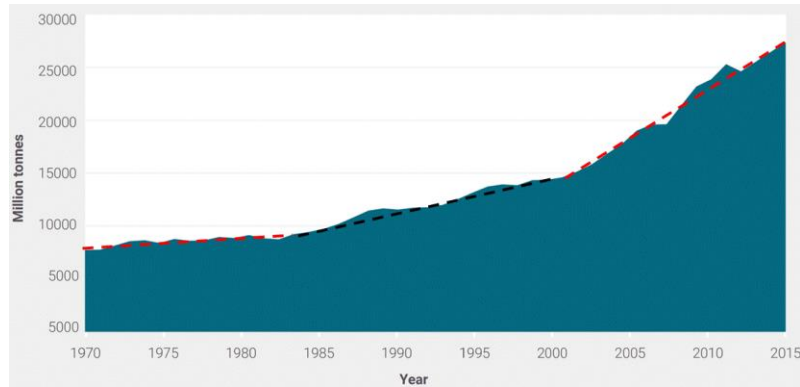


Fig. 1. The global extraction of metal ores from 1970 to 2017 (Baker et al., 2020)

For example, in Chile, there are several mining projects requiring tailings sand dams of more than 100 m in height such as, as Ovejería (120 m in height), Las Tórtolas (170 m), Quillayes (198 m), and El Mauro (237 m). Therefore, it is possible to point out that many tailings dams are among the largest earth structures. The design, construction, and operation of a tailings storage facility (TSF) must guarantee its physical stability at all events. However, the empirical fact is that the number of disasters caused by tailings dam failures is surprisingly high and increases. Fig. 2 shows the number of serious and very serious failures occurring per decade, as reported from 1958 to 2017.

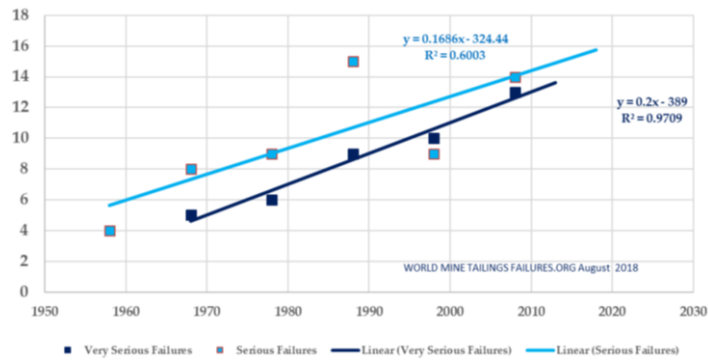


Fig. 2. Serious and very serious TSF failures from 1958 to 2017(<https://worldminetailingsfailures.org/>).

The trend of severe tailings dam failure events is confirmed by the recent catastrophic tailings dam failures in Canada (Mount Polley in 2014), Australia (Cadia in 2018) and

Brazil (Samarco in 2015 and Brumadinho in 2019). These failures have caused the flow of tailings to affect the environment severely. Unfortunately, the failures also resulted in many casualties in the cases of the Brazilian failures. A systematic increase in tailings dam failures is unacceptable to both society and the mining industry. The continued mismanagement of tailings dams has generated strong reactions from different organizations and demands to significantly improve the design, analysis, construction, operation, and closure of tailings deposits.

Several of the tailings dam failures have resulted from the occurrence of static liquefaction of the stored tailings. Therefore, identification of the initial states of the tailings and the undrained shear strength of the saturated tailings is crucial. Consequently, this paper focuses on this geotechnical aspect of tailings, which can be conceptualized and evaluated in the framework provided by the steady state of deformation (Poulos 1971) or the critical state soil mechanics (Schofield and Wroth, 1968). Considering that there are still some discrepancies between the equivalence or differences between the steady state and the critical state, it is deemed necessary to start with a brief review of these soil states.

The amount of tailings generated by the different minerals is presented in Fig. 3 (Baker et al., 2020), where it is observed that more than 75% of the tailings are produced from the processes of recovering copper (46%), gold (21%) and iron (9%). A summary of tailings types and geotechnical classification is presented in Fig. 4 (ICOLD 2019). It can be observed that majority of the aforementioned tailings are described as sandy silts of non to low plasticity.

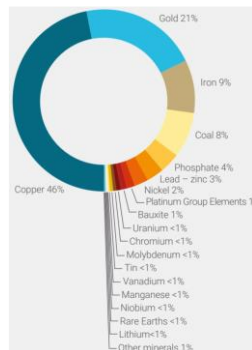


Fig. 3. Amount of tailings. (Baker et al., 2020)

| Tailings Type | Symbol | Description (compare) | Example of mineral/ore |
|-----------------------|--------|--|---|
| Coarse tailings | CT | Silty SAND, non-plastic | Salt, mineral sands, coarse coal rejects, iron ore sands |
| Hard Rock tailings | HRT | Sandy SILT, non to low plasticity | Copper, massive sulphide, nickel, gold |
| Altered Rock tailings | ART | Sandy SILT, trace of clay, low plasticity, bentonitic clay content | Porphyry copper with hydrothermal alteration, oxidized rock, bauxite, leaching processes |
| Fine tailings | FT | SILT, with trace to some clay, low to moderate plasticity | Iron ore fines, bauxite (red mud), fine coal rejects, leaching processes, metamorphosed/weathered polymetallic ores |
| Ultra Fine tailings | UFT | Silty CLAY, high plasticity, very low density and hydraulic conductivity | Oil sands (fluid fine tailings), phosphate fines; some kimberlite and coal fines |

Fig. 4. Tailings type and geotechnical classification (ICOLD 2019).

Therefore, the most significant quantities of tailings are from copper, gold, and iron mines, which in general, vary from fine sands to sandy silt. Thus, the observed behavior of sandy soils applies to these tailings.

Experimental results on the Japanese Toyoura sand from laboratory tests performed at the University of Tokyo, Japan, are presented in this paper (Ishihara et al., 1991, Verdugo, 1992; Ishihara, 1993; Verdugo & Ishihara, 1996). The tested Toyoura sand has a specific gravity, $G_s = 2.65$, $D_{50} = 0.17$ mm, and maximum and minimum void ratios of $e_{max} = 0.977$ and $e_{min} = 0.597$, respectively.

2 The ultimate state of soils

2.1 Steady state and critical state

The concept of critical void ratio was first postulated by Casagrande in 1935 at a Boston Civil Engineering Society conference and published a year later in the same society's journal (Casagrande, 1936). Using direct shear tests, Casagrande observed that during shearing, dense sands expand, increasing their void ratio, while very loose sands contract and reduce their volume and consequently their void ratio. Based on this observation and his intuition, Casagrande developed the concept of the critical void ratio in which dense and loose sands, when sheared in a drained condition, change the void ratio until a unique constant void ratio is finally reached, that Casagrande called the critical void ratio. In this state, the soil continues to deform under constant strength and constant volume, hence the soil behaves as a frictional fluid. Initially, Casagrande thought that the critical void ratio was a unique constant value for a given soil.

In the same direction, Roscoe and co-workers (Roscoe et al., 1958) studied the ultimate conditions reached by soil samples sheared to large deformation under drained and undrained loading conditions. They confirmed that in the $p' - e - q$ space, both for drained and undrained tests, there is a unique line where the loading paths converge, which they called the critical voids ratio line (CVRL). In the case of a drained test, the critical void ratio state is reached when any further increment of shear distortion does not change the voids ratio. In undrained tests, the effective mean stress, p' , is modified to bring the sample into an ultimate state such that the shear stress and the effective stress become constant. Roscoe obtained the CVRL experimentally for clayey soils, sandy soils, glass beads, and steel balls. For Weald Clay, this line is shown in Fig. 5.

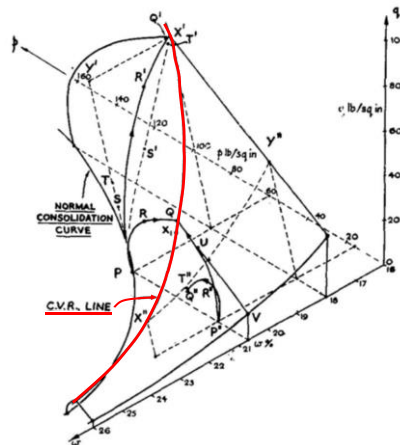


Fig. 5. Critical voids ratio line for Weald Clay (Roscoe et al., 1958).

Later, Schofield and Wroth (1968) published the book that can be considered the official birth of the concept known as Critical State Soil Mechanics (CSSM). The authors

in this book indicate that soil and other granular materials, if continuously distorted until they flow as a frictional fluid, will come into a well-defined critical state determined by the following two equations:

$$q = Mp' \quad (1)$$

$$e = \Gamma - \lambda \cdot \ln(p') \quad (2)$$

where $q = (\sigma'_1 - \sigma'_3)$ corresponds to the deviator stress, $p' = (\sigma'_1 + 2\sigma'_3)/3$ represents the effective mean stress, e is the void ratio, M is the shear stress ratio q/p at critical state, λ is the slope of the CVRL in the e vs. $\ln(p')$ plot, and Γ is the void ratio of the CVRL for $p'=1$ stress unit.

The first equation of the critical states determines the magnitude of the deviator stress q needed to keep the soil flowing continuously. The second equation states that the void ratio of flowing particles will decrease as the logarithm of the effective pressure increases. Schofield and Wroth (1968) emphasized that the total change from any initial state to an ultimate critical state where the soil flowed as a frictional fluid can be predicted with these two equations. It is interesting to note that, strictly speaking, there are no requirements regarding soil behavior. The definition of the critical state is pragmatic since it simply depends on empirical data that can fit the two equations described above.

Later in 1971, following both the concept of critical void ratio initially postulated by Casagrande and the experimental results reported by Castro (1969), which showed how a sufficiently loose sand collapses and deforms continuously under an undrained stress-controlled test, Poulos (1971) proposed the concept of the steady state of deformation. He defined it as the state in which a particulate material of any composition or particle shape deforms continuously under a constant state of effective stress at constant velocity and at a constant void ratio. In his monograph, Poulos explicitly states that the term "steady state" is used for two reasons: 1) it conveys correctly (by analogy to steady state flow of liquids) the concept of flow, and 2) the term "critical state" has been applied by other researchers to a completely different condition of soils. The steady state concept was then used by Castro (1975) and Castro and Poulos (1977) to explain the difference between "true" liquefaction and cyclic mobility. Later, Poulos (1981) stated that the steady state of deformation is achieved only after all particle orientation has reached a statistically steady-state condition and after all particle breakage, if any, is complete, so that the shear stress needed to continue deformation and the velocity of deformation remain constant. In addition, it is indicated that during the steady state of deformation, the original structure of the specimen is completely destroyed and reworked into a new "flow structure."

According to these definitions, the steady state of deformation has the following conditions that must be satisfied by the soil under continuous deformation:

- Constant effective stress
- Constant shear stress
- Constant velocity
- Constant void ratio
- Particle orientation has reached a statistically steady-state condition

- All particle breakage, if any, is complete
- Original structure is reworked into a new flow structure

These requirements are of such practical implication that the critical state can be clearly differentiated from the steady state of deformation depending on the soil type. Considering that the critical state is a state that does not pay attention to the rate of deformation or the orientation of particles, its actual application to clayey materials is somewhat limited, as shown below.

2.2 Behavior of clayey soils in relation to their ultimate state

A comprehensive set of tests are reported for the Japanese Kawasaki clay ($G_s = 2.69$, $LL = 55.3$, $PI = 29.4$, $FC = 83.9\%$), which has been tested at different rates of deformations (Nakase and Kamei, 1986). Fig. 6 presents the results of a series of undrained triaxial tests started from a K_0 -consolidation and loaded in compression and extension. Three different rates of deformations were applied during loading: $7 \cdot 10^{-1}$, $7 \cdot 10^{-2}$ and $7 \cdot 10^{-3}$ %/min.

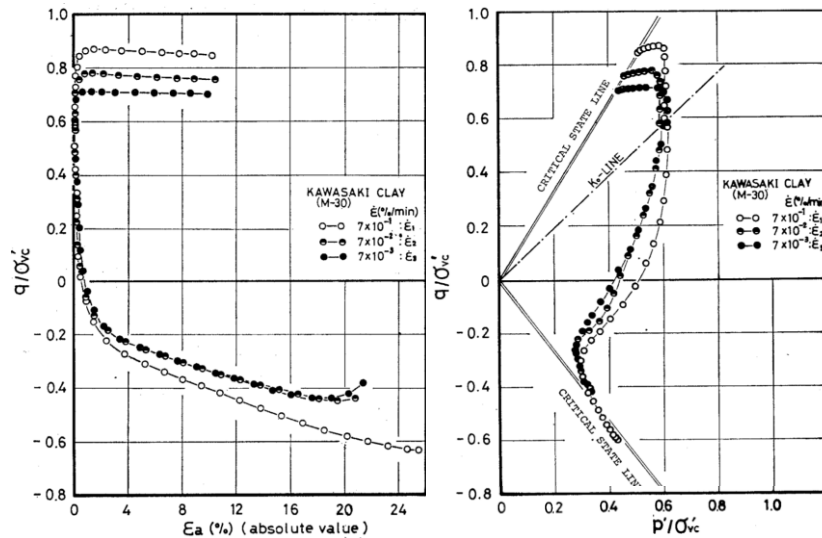


Fig. 6. Effect of strain rate on the behavior of clay (Nakase and Kamei, 1986).

These results clearly show that the rate of deformation has an important effect on both the stress-strain curves and the undrained shear strength; the higher the rate of deformation the higher the undrained strength. For clayey soils, different researchers have reported the dependence of the undrained strength on the rate of strain (Bjerrum 1969; Sheahan et al., 1996; Díaz-Rodríguez et al., 2009; Chow et al., 2012, among many others). However, the angle of friction mobilized at the ultimate state or the critical state is not influenced by the rate of deformation. This means that the critical state line in the q - p' plane is unaffected by the rate of deformation, but the critical state line in the e - $\log p'$ plane is definitely influenced by the rate of deformation.

On the other hand, with respect to the particle orientation, this phenomenon has been reported in clayey material with an important amount of platy particles of clays. The re-orientation of the platy particles in the direction of the applied shear stress has made possible to explain the reduction of the resistance to a "residual strength" that has been measured at significantly large deformations under a considerable low rate of deformation (Bishop et al., 1971; Lupini et al., 1981; Skempton, 1985). As example, in Fig. 7 the stress-displacement curve obtained from a ring shear test is presented, where after about 30 days of shearing, a residual strength associated with a friction angle, $\phi_r = 8.6^\circ$, is mobilized (Skempton, 1985).

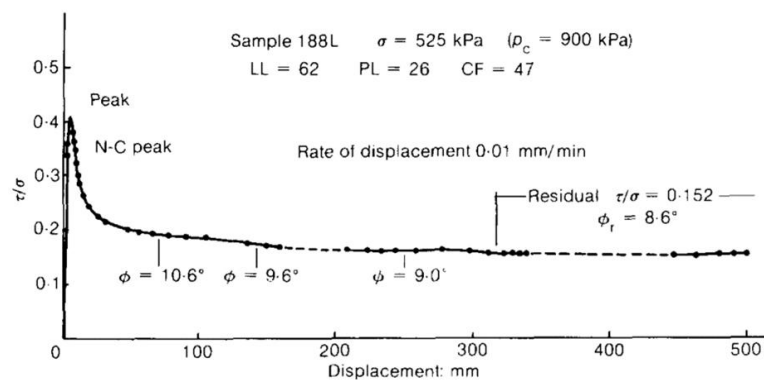


Fig. 7. Residual strength on clayey soil (Skempton, 1985).

In this matter, Schofield and Wroth (1968) explicitly show that the residual strength developed by clayey soils with platy particles differs from the strength established at the critical state, as indicated in Fig. 8, where for consistency the critical state should be at point D, but the actual ultimate state is at point E.

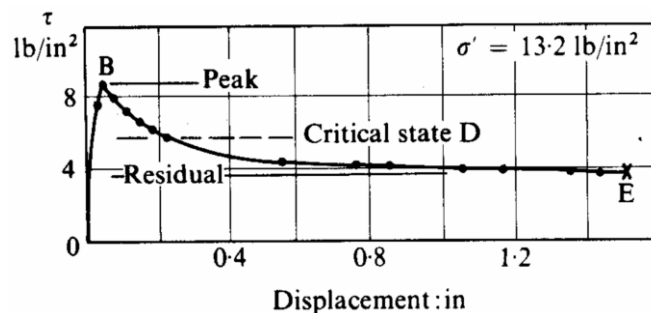


Fig. 8. Critical state (D) and residual strength (E) (Schofield and Wroth 1968).

Therefore, in the particular case of clayey materials and those that behave as clay-like soils, the requirements imposed by the steady state are definitely not satisfied, unless different steady states are accepted depending on the strain rate and the level of deformation. In this context, it is important to point out that the critical state, by not imposing

requirements for the ultimate state, somehow allows a free choice of what will be used as critical state, adjusting the variables to the particular problem under analysis.

2.3 Behavior of sandy soils in relation to their ultimate state

In the case of sandy soils, the rate of deformation is significantly less important in the response at large deformations, as can be inferred from the experimental results reported by Been and Jefferies (1991) as reproduced in Fig. 9. Contractive samples under stress-controlled tests collapse, developing a high rate of deformation during the collapse, which is several orders of magnitude higher than the common rate of deformation imposed in strain-controlled tests. Nevertheless, it is apparent in Fig. 9 that the same ultimate state is achieved in contractive samples under load-controlled and strain-controlled tests.

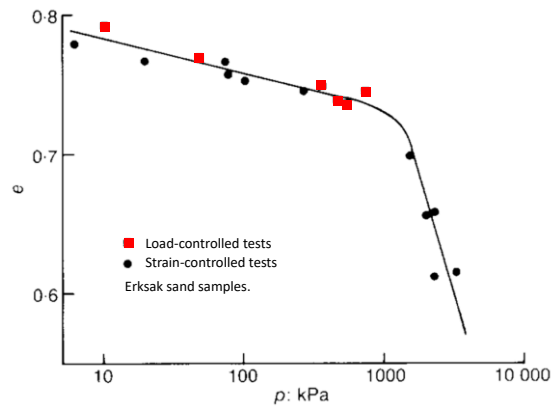


Fig. 9. Critical state line Erksak sand (Been and Jefferies 1991).

The phenomenon of re-orientation of particles, which can significantly reduce the shear strength at extremely large deformation, needs an important presence of platy particles that do not exist in sandy soils.

When the ultimate state is reached in sandy soils, the particle breakage, if any, is definitely complete. Otherwise, the pore pressure would increase under undrained loads, or equivalently, the volumetric strains would show contraction under drained loads, which would not be an ultimate state. Regarding Poulos's concept, it is difficult to verify that the original structure is reworked into a new flow structure in the ultimate state. However, experimental results reported by Verdugo and Ishihara (1996) may provide some insight that suggests at least the development of a unique fabric during the ultimate state. Fig. 10 shows the results of stress-strain curves and effective stress-paths of a series of undrained triaxial tests carried out at effective confining pressures of 0.1, 1, 2, and 3 MPa on samples with the same void ratio of $e = 0.735$ after isotropic consolidation. Once the ultimate condition was reached, each sample was unloaded to zero shear stress. Besides the fact that these results confirm that, under undrained load, the void ratio controls the ultimate state, it is essential to realize that during unloading, the

effective stress paths of the four tested samples converge into one, independently of the confining pressure of each sample. The development of a unique effective stress path during unloading suggests that the induced fabric of the four samples was the same when the unloading started from the ultimate state.

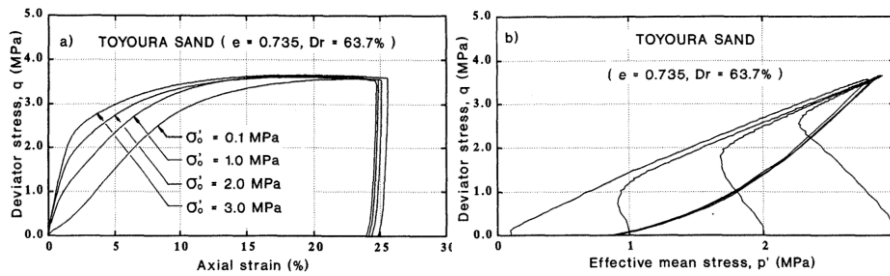


Fig. 10. Unloading from the steady state (Verdugo and Ishihara 1996).

Other potential evidence about the creation of a particular structure during the steady state is presented in Fig. 11 (Verdugo 1992), where the results of two samples prepared at the same density by different procedures such as wet tamping and water sedimentation, show different stress-strain curves but identical ultimate shear strength. Furthermore, the effective stress paths during unloading are practically the same.

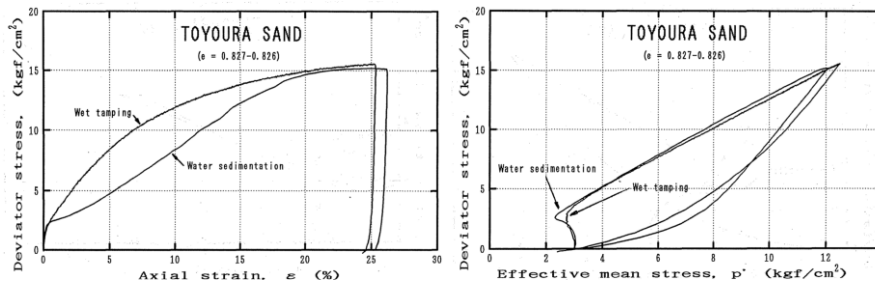


Fig. 11. Undrained tests on samples prepared by different methods (Verdugo 1992).

These experimental results suggest that the different original fabrics were modified during loading so that upon reaching the ultimate state, a new and similar fabric was developed in these samples, which is manifested through the same shear strength and the same effective stress-path during unloading.

2.4 Critical and steady states are different

Following what was presented above, there are similarities, but also differences, between the critical state and the steady state, which can be described as follows:

- Both states are associated with the ultimate condition reached at large deformations by a soil subjected to shear stress.
- The critical state has no special requirements to establish how it has been reached. Simply, in the case of drained loading, it is reached when any further increment of shear

distortion does not change the void ratio. In the case of undrained loading, it is reached when the ultimate state is such that both the shear stress and the effective stress become constant under continuous shear distortion.

- The steady state establishes conditions that are definitely not specified by the critical state. These conditions are constant velocity, particle orientation, particle breakage ended, and modification of the original structure into a new flow structure.

Considering that the mechanical response of clayey materials can be seriously affected by both the rate and the amount of deformation, they should develop different ultimate states according to the characteristics of the applied loads. Accordingly, the critical state is more applicable because this state can be established for the particular problem under analysis. On the other hand, the steady state is appropriate in sandy soils because its requirements are satisfied by granular soils. Obviously, these soils also satisfy the less rigorous framework imposed by the critical state.

In summary, the steady state of deformation corresponds to the ultimate state that any particulate material reaches when it is sheared at large deformations. All the aspects observed in the behavior of soils are considered. On the other hand, in a more practical application, the critical state of soils corresponds to the ultimate state reached at large strains, when there is no change of void ratio in drained condition, and the effective stresses become constant in undrained load condition.

In the case of several tailings, its behavior can be considered sand-like materials. Accordingly, the ultimate state of these tailings can be referred to as steady state or critical state. However, because the requirements of the steady state are more rigorous, this concept is used in this article as a synonym for ultimate state.

3 Characteristics of the contractive states

3.1 Condition of minimum shear strength in sandy soils

As shown in Fig. 12, sandy soils in a contractive state may develop under undrained loading a stress-strain curve with a clear peak followed by the development of a minimum strength at a medium level of strain. Thereafter, an increase in the strength toward the end of the test is observed.

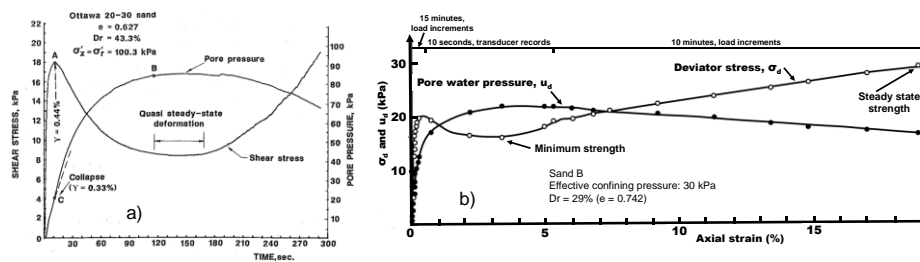


Fig. 12. Stress-strain curves with development of a minimum strength (a) Alarcon-Guzman et al., 1988, b) Castro 1969).

This condition of minimum shear resistance has been called quasi-steady state (QSS) because it is a transient state in which the steady state requirements are mostly satisfied (Alarcon et al., 1988; Ishihara 1993). In the experimental results presented in Fig. 13, the minimum strength can be as low as 16% of the strength mobilized at the critical state. Therefore, from a practical point of view, it is important to pay attention to this transient state due to the significant difference between the minimum strength and the strength mobilized at large deformation where the steady state is achieved.

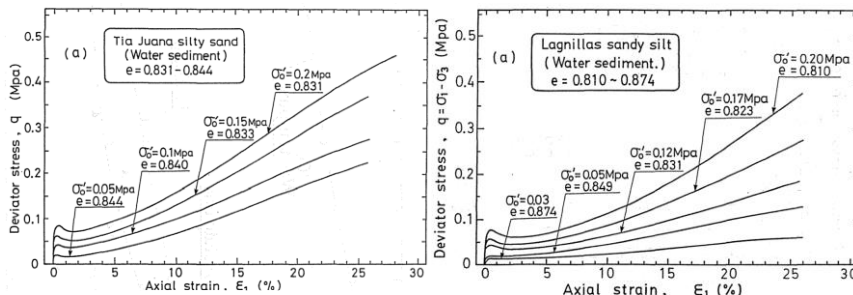


Fig. 13. Minimum strength and steady state strength (Ishihara 1993)

The state of minimum resistance is associated with a rather significant level of deformation, which could be severe for some structures. However, when analyzing a flow failure, exceptionally large deformations are necessarily involved, which are definitely not manifested during the condition of minimum strength. Therefore, this condition does not represent a true flow failure. Nevertheless, it is important to understand how these limited deformations and excess pore pressures may interact negatively in the soil mass.

Unlike what is observed in the steady state, the development of the minimum strength is affected by the initial fabric of the soil and is relatively affected by the level of confining pressure. This state of minimum strength is obtained in load-controlled tests (Castro, 1969) as well as in strain-controlled tests (Alarcon et al., 1988, Ishihara, 1993) and in triaxial tests with enlarged and lubricated ends that allow a more uniform deformation of the samples (Verdugo, 1992). It is not clear what is the cause of this transient state. Still, it is important to be aware of its real existence. Depending on the project, it can have an important effect on the global stability of a soil mass as illustrated below.

It is important to point out that there is a special condition in which the static shear can be greater than the minimum strength (i.e., the quasi-steady state strength), but smaller than the steady state strength, as is illustrated in Fig. 14. A load that triggers the undrained soil response can generate soil deformations and excess pore pressure that reduces the mean effective stress from the initial state at point A to the point B. Therefore, in those initial states where the soil can develop a quasi-steady state condition and the associated minimum resistance is less than the acting static shear, a significant increase of both deformations and pore pressure can be generated. The interaction of deformation and dissipation of pore pressure with the rest of the soil mass can trigger a

failure mechanism, which depends on the initial state of static shear, void ratio, and mean effective stress of the soil mass.

The phenomenon of minimum strength, or QSS, is not captured by the state parameter, since the latter is based on the ultimate condition or steady state. In fact, the state parameter in these cases is negative, meaning the soil response is dilatant and stable.

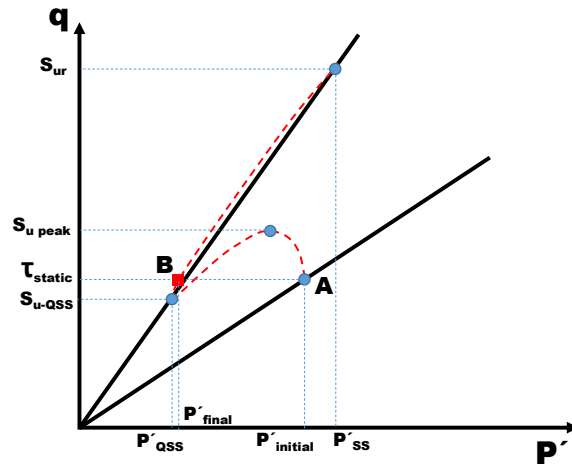


Fig. 14. Static shear stress greater than QSS strength (S_{u-QSS}) and smaller than SS strength (S_{ur})

3.2 States with loss of strength

For Toyoura sand, the steady state line established as the ultimate state reached at large strains, in the order of 25%, is presented in Fig. 15a. In comparison, the quasi steady state is presented in Fig. 15b (Ishihara 1993). In both cases, the initial states of each test are indicated by white squares, making it clear that the QSSL can only be obtained from a contractive initial condition.

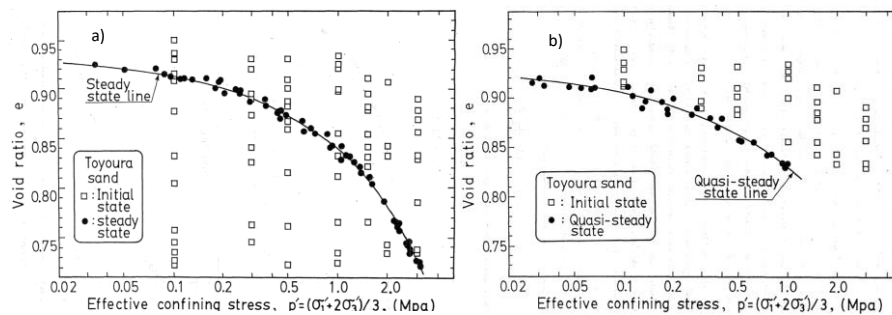


Fig. 15. a) Steady state line, b) Quasi-steady state line. Toyoura sand (Ishihara 1993).

Additionally, in Fig. 16 are presented the SSL, the QSSL and the so-called IDL (Initial Dividing Line). The IDL is the line above which the initial states that develop loss of strength are located.

It is important to note that the IDL is below the SSL in a significant range of effective mean pressure. Therefore, it is not entirely true that only those initial states located above the SSL are contractive and with evidence of instability.

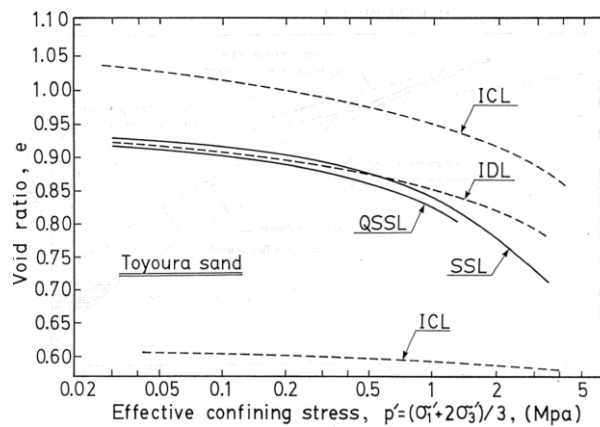


Fig. 16. Characteristic lines in the e - p' plane (Ishihara 1993).

3.3 Peak undrained strength

The undrained response of sandy soils in a contractive state, with a drop of strength, shows a maximum value of shear stress, commonly called undrained peak strength or yield strength. The sketch of Fig. 17 shows how a medium amplitude cyclic load could trigger the ultimate resistance, without the need of mobilizing the peak strength (Poulos, 1988). Experimental evidence confirming that a cyclic load can bypass the peak strength is presented in Fig. 18.

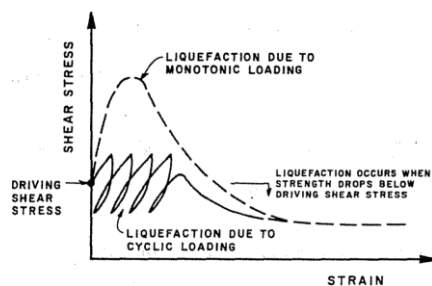


Fig. 17. Cyclic and monotonic loadings. (Poulos 1988).

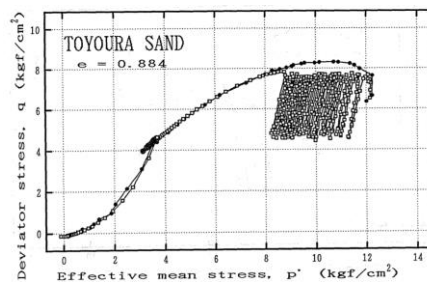


Fig. 18. Cyclic and monotonic loadings Tests on Toyoura sand (Verdugo 2009).

From the above, it can be concluded that the peak strength is not a reliable parameter in cyclic stability analyses. However, it is useful to establish the collapse surface as it is explained below.

3.4 Flow failure or static liquefaction

Even when a soil mass is in an initial contractive state such that it may suffer a loss of its strength, an additional condition is still necessary for a flow failure (or static liquefaction) to occur. This initial condition is that the driving shear stresses (static shear stress) must be greater than the residual undrained strength. Therefore, the static shear stress also plays an essential role in the occurrence of a flow failure, as shown in Fig. 19.

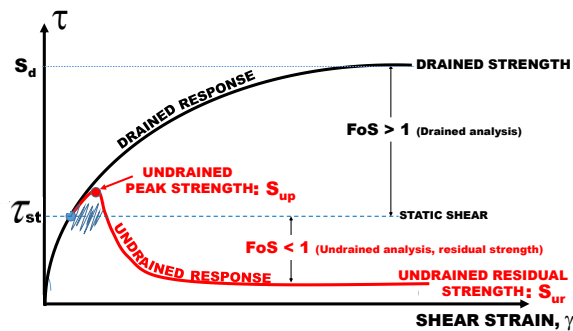


Fig. 19. Concept of static liquefaction.

The application of an initial load under a drained condition is considered until a static shear stress τ_{st} is reached. If the loading continues under a drained condition, theoretically, the soil element will move along the black curve until it mobilizes the drained resistance S_d . However, if the undrained response is triggered, for instance due to a fast disturbance, the strength of the soil would drop from S_d to S_{ur} . In Fig. 19, the static shear stress is greater than the mobilized residual undrained strength, S_{ur} , which means that a flow failure would occur. It is of great importance to mention again that the peak undrained strength can be bypassed by the soil response.

3.5 The collapse boundary

The collapse surface concept establishes in the q - p' plane the existence of a boundary that effective stress paths cannot cross without the collapse of the soil, reaching the steady state condition. Initially, the collapse surface was defined in the q - p' plane based on the monotonic effective stress path of a contractive sample with loss of strength. The collapse surface was identified as the straight line between the peak resistance and the ultimate state as is shown in Fig. 20a (Sladen et al., 1985). Figs. 20b and 18 presents experimental results that confirm the existence of the collapse boundary (Verdugo 2009).

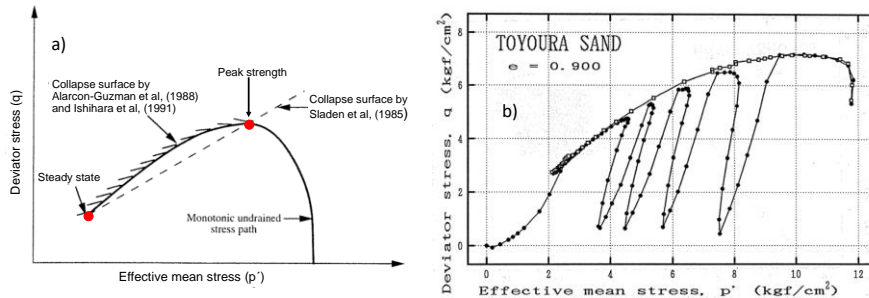


Fig. 20. a) Collapse surface (from Sladen et al., 1985), b) Test results on Toyoura sand.

It is observed in Fig. 20b that the monotonic effective stress path acts as a boundary of the cyclic load. These tests were performed under a strain-controlled regime, which allows the measurements during the post-peak soil response.

For Toyoura sand, the monotonic effective stress paths obtained for the loosest states that were achieved are presented in Fig. 21. It is seen that a straight line can be drawn through the peak strength points. The line can be considered as a lower boundary above which the soil would collapse. Also shown in Fig. 21 are the monotonic effective stress paths of those initial states that present just a marginal loss of strength. These stress paths also allow to establish a line that can be seen as an upper boundary of the states that exhibiting a strength loss. The two boundaries shown in Fig. 21 represent the zone in the e - p' plane associated with the collapse of this soil. Between these lines, there are many other lines with slopes that are functions of the initial void ratio.

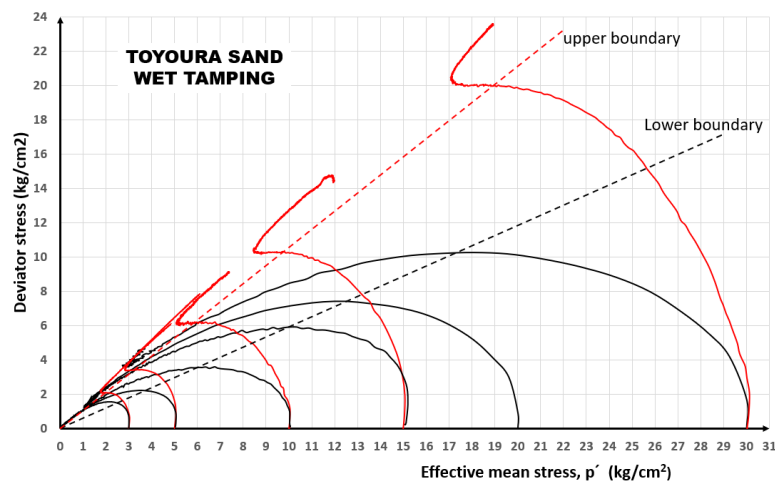


Fig. 21. Lower and upper collapse lines for Toyoura sand.

Experimental evidence suggests that the monotonic effective stress path is the boundary in the e - p' plane that controls the onset of soil collapse. However, from any practical point of view, the maximum shear strength locus established in the e - p' plane can be

considered as a good approximation to define the limit that triggers soil collapse, which is a function of the void ratio.

Due to the field conditions are load-controlled, two practical stress paths can lead the soil collapse (Fig. 22):

1) Rise of the water table or the seepage flow that decreases the effective mean stress while keeping the static shear constant (Sasitharan et al., 1993).

2) Initially drained load, which combination of q and p' passes the collapse line. In this case, there is a sudden change of the drained load into an undrained response of the soil.

The in-situ condition of load-controlled impedes the application of drained stress beyond the collapse line. The soil responds with a sudden increase of both strains and pore pressure that triggers the undrained response and the collapse of the soil.

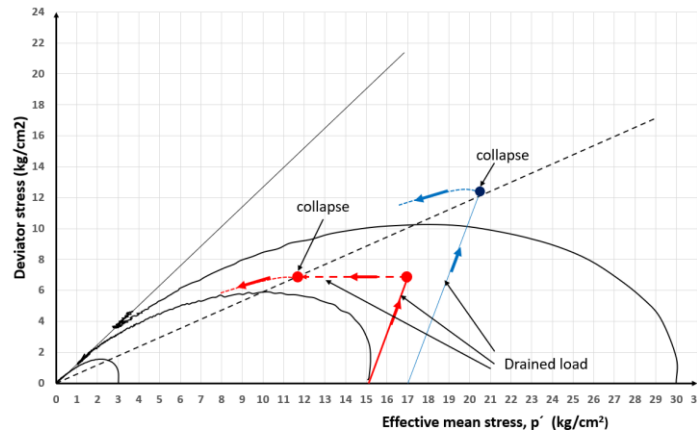


Fig. 22. Examples of initial drained stress paths that trigger the collapse of soil in a contractive state. (in red: constant shear with decreasing mean effective stress; in blue: high ratio q/p')

4 Characterization of the initial states of a soil

4.1 State parameter ψ

Using the steady state line as the reference state, Been and Jefferies (1985) introduced the so-called state parameter, ψ , which considers both the degree of packing and the pressure effect. The state parameter is defined in the $e-p'$ plane as the difference between the current void ratio of the soil and the void ratio at the steady state, at the same mean effective stress. This means that initial states that have the same distance to the steady state line in the $e-p'$ plane would have a similar response. Indeed, this observation is similar to the conclusion reported by Roscoe and Poorooshasb (1963): any two samples of a given soil, when subjected to geometrically similar stress path, have the same strains provided that the difference between the initial void ratio and the void ratio at the critical state at the same normal stress is the same for each sample. In this

statement, Roscoe and Poorooshasb have referred to the critical state instead of the steady state, but for the present purpose these states can be considered identical.

The positive values of the state parameter are associated with initial states that are located above the steady state line in the e - p' plane, which means that the initial state is contractive. The negative values of the state parameter correspond to states below the steady state line, which represent initial dilatant states. Although the state parameter is widely used, it has a severe limitation of incorporating the potential instability induced by the static shear, as explained in detail below.

4.2 State Index I_s

The state index I_s proposed in the e - p' plane by Ishihara (1993) is considered a more rigorous parameter to characterize the response of sandy materials and low-plasticity silts as tailings. I_s uses two reference lines: the quasi steady state line (QSSL) and the isotropic consolidation line ICL of the loosest state. Along the ICL, the state index takes the value $I_s = 0$, and along the QSSL the state index takes the value $I_s = 1$. Then, for a given effective mean pressure p' , the state index is defined as the linear interpolation between the current void ratio e , the void ratio e_o at the ICL, and the void ratio e_s at QSSL. Fig. 23 gives the definition of I_s including the zone of zero strength.

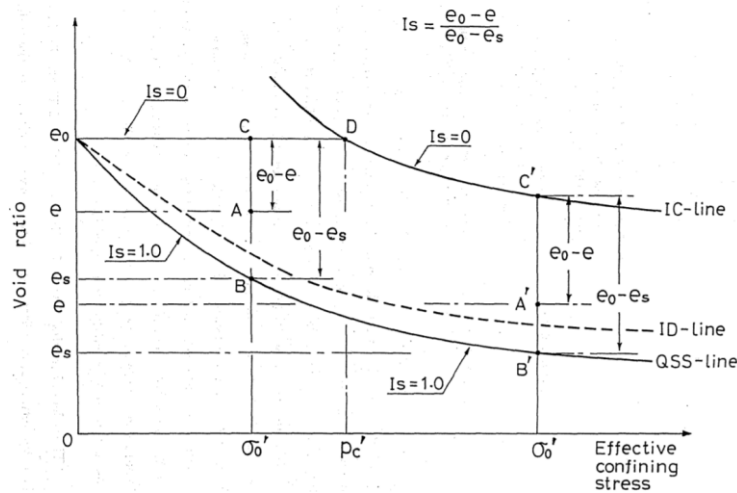


Fig. 23. Definition of State Index (Ishihara 1993).

It is important to highlight that using the QSSL as a reference line allows considering the minimum resistance condition that can generate significant increases in pore pressure and deformation. This condition depends on the soil fabric, which cannot be captured when the ultimate state is used as a reference line.

The state index I_s can be a good predictive capacity of soil behavior according to the initial state of density and confining pressure. However, the fact that it does not consider the shear stress means that it also suffers from the same deficiency as the state parameter.

5 The importance of static shear in the state of a soil

The state parameter ψ and the state index I_s are based on isotropic triaxial consolidated tests. For this initial isotropic stress condition, the distance, in terms of void ratio to the SSL or QSSL, is a reasonable approximation to the expected soil response. However, when initial static shear stress is included, or in other words, when the stress paths to reach the initial permanent condition are different, these parameters can give a misleading interpretation of the soil response. Note that if a given combination of e and p' is reached, but from totally different stress paths, the response to a new given stress path could be different depending on the previous stress paths.

To illustrate the effect of the initial static shear stress on the undrained responses, two different samples tested in the triaxial equipment are considered for simplicity, but the concept is valid for real situations in situ. Fig. 24 illustrates how the static shear stress can drastically modify the soil behavior from a stable undrained contractive-dilatative response to an unstable fully contractive response that can experience flow failure.

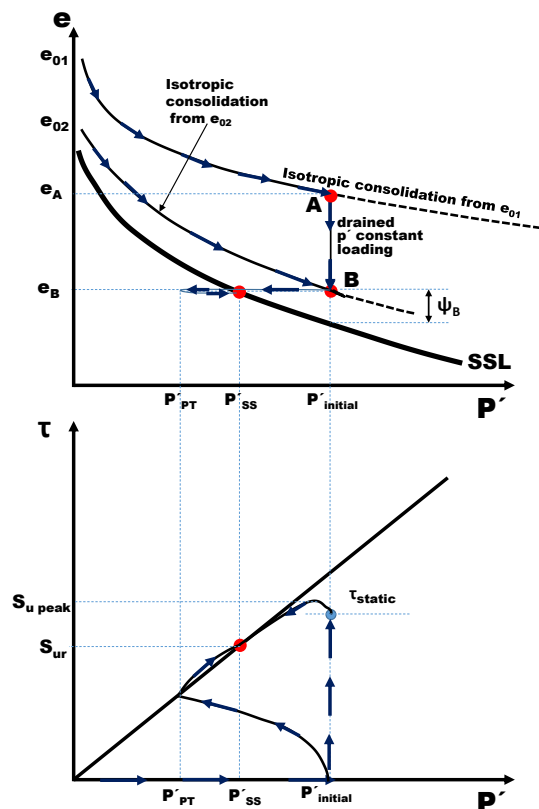


Fig. 24. Effect of initial static shear stress on the state of a soil

First, a loose sample prepared with a void ratio, e_{01} , is considered, which is isotropically consolidated up to a pressure $p'_{initial}$ that generates a void ratio, e_A . Then, this hypothetical sample is loaded under a drained p' constant condition up to a shear stress identified as τ_{static} . This drained load decreases the void ratio from e_A to the value e_B . A second hypothetical sample prepared with a lower void ratio, e_{02} , is considered. This void ratio is such that after the sample is isotropically consolidated to $p'_{initial}$, the void ratio achieved is e_B . These two samples have the peculiarity of having the same void ratio and the same effective mean stress, which implies that they have the same residual undrained strength, S_{ur} , and the same state parameter, ψ_B . However, one of the samples has high static shear stress that is greater than the undrained residual strength, which means it can develop a flow failure. Whereas the other sample has no static shear stress, and additionally, its undrained response does not present any instability.

The main conclusion of this example is that the state parameter does not capture the existence of any static shear. In other words, any given distance to the SSL in the $e-p'$ plane can be associated with a fairly significant number of levels of static shear stress. In some cases, the static shear stress can be less than the undrained residual strength, but in others, it may be the other way around, and therefore, the state of the soil can be stable or ready for a flow failure.

6 Flow State Index

It is recognized that a flow failure requires that the existing static shear stress be greater than the undrained residual strength. On the contrary, if the static shear does not exceed the undrained residual strength, a flow failure cannot be triggered, unless there is migration of water from one sector to another, which is a more complex phenomenon beyond the present analysis. Therefore, an index that directly considers this situation would be useful. Accordingly, the Flow State Index I_f , defined below, is proposed:

$$I_f = \frac{S_{u-min}}{\tau_{static}} \quad (3)$$

As it has been shown, there are cases where the difference between the minimum strength and the one mobilized at the ultimate state can be drastically different; it is estimated that in problems of stability of soil masses against flow failure, it is appropriate (conservative) to use the undrained minimum strength, S_{u-min} , which can be evaluated by means of in-situ testing in the case of existing tailings dams, for example, using vane shear tests with a high speed of rotation to ensure the undrained response of the tested soils or by CPT-u measurements (Roberson 2010), or just the sleeve resistance (Vergo et al., 2014).

The driving static shear stresses, τ_{static} , can be estimated through numerical analysis. In general, it can be indicated that the assessment of the static stress field in earth structures is not sensitive to the constitutive model used, which is not valid for the strain field. In tailings dams, the basic resistance and stiffness properties of the different materials involved in the stability can be reasonably estimated. Also, the static stress field

in the different parts of a tailings dam can be numerically evaluated with an acceptable accuracy using relatively simple stress-strain constitutive models. In addition, it is necessary to measure and/or calculate the seepages to estimate the pore pressure distribution, and accordingly, the effective stresses. Therefore, using a rather simple numerical analysis it is possible to estimate both the effective mean stresses and shear stresses existing in the different zones of a tailings dam.

Accordingly, the proposed flow index I_f is evaluated considering field data and numerical analysis. Obviously, values of I_f less than one mean that the permanent static shear is greater than the minimum undrained shear strength, and therefore, the state of the soil is such that a flow failure can be triggered. Conversely, values of I_f greater than one represent soil states that are not associated with a potential failure due to static liquefaction.

The question that naturally arises is what would be the extension of the zone where the values of I_f are less than one, in such a way that a flow failure of the soil mass can be triggered. The answer to this question involves more sophisticated numerical analyses. Still, the fundamental point is that the proposed parameter I_f makes possible to identify the existence of soil states that can collapse and be aware of their existence, if they exist.

7 Concluding Remarks

Mining operations worldwide produce vast amounts of tailings that need to be stored safely, reconciling economic constraints and environmental sustainability. However, the recent history of tailings deposits is marked by episodes of catastrophic failures that have caused many victims and have cost enormous material losses. One of the leading causes of these failures is the static liquefaction of the stored tailings.

The concepts of steady state and critical state, which allow the analysis of static liquefaction, have been reviewed and discussed, resulting in the steady state concept being more rigorous to study static liquefaction. It is possible to think that after so many years it is not acceptable to still be stuck with the potential differences between SS and CS. It is true that at the end it does not matter whether CS and SS are different or the same; in practice it might be irrelevant. However, behind this discussion, which could be seen simply as a matter of semantics, it is the real behavior of the soil, which in many aspects is different for clays and sands and that in practice can seriously affect its modeling and analysis. It was shown that the behavior of clays is strongly affected by the strain-rate, also clays can undergo reorientation of the platy particles that results in a significant reduction of the drained strength. These phenomena are not relevant in sandy soils, but the behavior of the sands can present a state of minimum resistance, followed by significant dilation and a strong recovery to the ultimate strength that could be useless in some practical problems. Therefore, what really matters is to specify which conditions are considered in soil characterization and modeling, and which are left out.

The occurrence of static liquefaction requires the soil to be in a contractive state with a loss of strength. This condition is currently addressed using parameters that provide information on the state of the soil, in terms of its void ratio and confining pressure,

and its location concerning the steady state line or quasi-steady state line. However, this approach has severe limitations because it does not consider the driving static shear stress, or the stress paths to reach the initial states.

A very simple but straightforward index has been proposed that gives a clear indication of the state of a soil with respect to its potential for flow failure.

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