

Evaluation of Undrained Failure in Foundation Soil Beneath Tailings Dam

by

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ABSTRACT

Since the dawn of civilization, the majority of basic human needs have been obtained from mines. The rapid growth of population and the increase in need of mining activities accordingly have led to a rise in the amount of waste products (i.e. mine tailings). There have been 46 tailings dam failures around the world over the last two decades and this figure has been rising continuously. Among various reasons for tailings dam failure, the undrained failure in underlying fine-grained soils is considered to be one of the major contributing factors. In this study, an attempt was made to estimate the stability of a tailings dam in Elkview Operations (EVO) located approximately three kilometers east of Sparwood in the southeastern part of British Columbia. For this, a series of numerical analyses were performed using finite difference software (FLAC3D) to predict effective stress path in the underlying clayey silt layer, considering staged construction under undrained condition. The behavior of clayey silt layer under each construction stage was estimated by adopting the Modified Cam-Clay constitutive model. The factor of safety was then computed based on the critical state line and effective stress path. The analysis results showed that additional deposition of tailings can lead to a failure in the tailings dam, which can be initiated from zones below the toe of tailings dam. The sensitivity analysis results showed that the slopes of critical state and normal consolidation lines are the most influential parameters to the stability of tailings dam.

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LIST OF SYMBOLS & ABBREVIATIONS

Abbreviations

CD	= Consolidated – Drained
CDA	= Canadian Dam Association
CSL	= Critical State Line
CU	= Consolidated – Undrained
DSS	= Direct Simple Shear
ESA	= Effective Stress Approach
ESP	= Effective Stress Path
FDM	= Finite Difference Method
FEM	= Finite Element Method
FOS	= Factor Of Safety
LEM	= Limit Equilibrium Method
MC	= Mohr-Coulomb
MCC	= Modified Cam-Clay
NC	= Normally Consolidated
NCL	= Normal Consolidation Line
OC	= Over Consolidated
SLL	= Safe Loading Line
SRM	= Strength Reduction Method
TSA	= Total Stress Approach
TSP	= Total Stress Path
URL	= Unloading - Reloading Line
USA	= Undrained Strength Analysis

UU = Unconsolidated – Undrained

VCL = Virgin Consolidation Line

Symbols

A_0, B_0, F = Index correlations

$\bar{B}_1$ = The ratio of incremental excess pore-water pressure to the incremental mean effective stress

C_c = compression index

C_s = swelling index

C_r = unloading-reloading index

$\bar{C}$ = hydraulic conductivity correlation factor

c' = effective cohesion

c_u = undrained shear strength

$c_{u(FV)}$ = undrained shear strength from field vane test

$c_{u(design)}$ = design undrained shear strength

D_{10} = size of grain in millimeter for which 10 percent of particles pass by weight

E = Elastic modulus

e = void ratio

e_0 = initial void ratio

G = shear modulus

I_L = liquidity index

I_p = Plasticity index

I_i = index property

K = lateral earth pressure coefficient

K_0 = lateral earth pressure coefficient at rest

$K_{0,nc}$	= value of K_0 for normally compressed soil
$K_{0,oc}$	= value of K_0 for overconsolidated soil
K'	= bulk modulus
K'_{max}	= maximum bulk modulus
k_{avg}	= average hydraulic conductivity
k_f	= fluid bulk modulus
M	= slope of critical state line in $q - p'$ plane
M_f	= biot modulus
m_v	= coefficient of volume compressibility
N	= specific volume at reference pressure of isotropic normal compression line in $v - \ln p'$ plane
n	= porosity
n_p	= isotropic overconsolidation ratio
OCR	= overconsolidation ratio
p	= mean stress
p'	= mean effective stress
P'_c	= mean preconsolidation pressure
P'_0	= initial mean effective stress
P'_i	= current mean effective stress
P'_f	= mean effective stress at failure condition
P'_{max}	= maximum mean effective stress
p_b, p_d	= preconsolidation pressure in $v - \ln p'$ plane
q	= deviatoric stress

q_f	= deviatoric stress at P'_f
q_{max}	= maximum deviatoric stress
r	= ratio of pressures on normal compression and critical state lines
s_u	= undrained shear strength from conventional testing
s_d	= drained shear strength
v	= specific volume
v_s	= specific volume at reference pressure of isotropic unloading - reloading line in v - $ln p'$ plane
v_f	= specific volume at P'_f
v_m	= specific volume at reference pressure of 1-dimensional normal compression line in $v - \log \sigma'$ plane
v_n	= specific volume at reference pressure of 1-dimensional unloading-reloading line in $v - \log \sigma'$ plane
w	= water content
w_l	= liquidity limit
Y	= yielding curve
σ'_p	= 1-dimensional preconsolidation pressure
σ'_h	= effective horizontal stress
σ'_v	= vertical effective stress
σ_1	= maximum principal stress
σ_2	= intermediate principle stress
σ_3	= minimum principle stress
σ'_1	= maximum principal effective stress

σ'_2	= intermediate principle effective stress
σ'_3	= minimum principle effective stress
σ'_{v0}	= initial vertical effective stress
σ'_{hmax}	= maximum horizontal effective stress
$\Delta\sigma_l$	= Change in vertical effective stress
Δu	= change in pore-water pressure
τ_f	= shear strength
τ_m	= mobilized shear stress
τ_{hc}	= horizontal shear stress
ϕ'	= effective internal friction angle
ϕ'_m	= mobilized effective internal friction angle
γ	= unit weight
μ	= Poisson ratio
ε_a	= axial strain
ε_p^p	= plastic volumetric strain
Λ	= $(\lambda - \kappa)/\lambda$
η_α	= slope of K - lines
$\bar{\lambda}$	= correction factor
λ	= slope of normal compression line in $v - \ln(p')$ plane
κ	= slope of unloading – reloading line in $v - \ln(p')$ plane
Γ	= specific volume at a reference pressure of critical state line in compression plane

1. INTRODUCTION

1.1 Problem Statement

With the development of modern technologies, the need for mines' products (i.e. oil, coal, gold, etc.) has also been increased drastically. This demands subsequently led to an increase in the volume of waste and tailings (byproducts remained after the mining process), which compelled us to build more subsurface tailings dams to store huge amounts of toxic materials. Tailings dam is one of the most critical geotechnical structures because the failure in a tailings dam can have a devastating impact on not only the environment but also human lives (Çinicioglu and Toğrol 1991; Lyu et al. 2019). Since the beginning of the 20th century, over 18,000 mines have been built. Among them, 46 tailings dams failed around the world between 1999 and 2018 (Armstrong et al. 2019) (Figure 1.1) and the number of failures is consistently rising. 43% of the world's most significant tailings dam failures occurred in North America (i.e. 37% in the United States and 6% in Canada) (Lyu et al. 2019). Tailings dams are generally constructed using either upstream, downstream, or centerline methods. According to Lyu et al. (2019), 58% of failed tailings dams were constructed using the upstream method. This indicates that the construction method of a tailings dam plays a vital role in controlling the stability of the structure. Apart from the construction method, the followings are considered to be the governing factors to the instability of tailings dams; i) seepage and internal erosion, ii) poor foundation soil, iii) overtopping, iv) static and seismic instability, and v) other reasons such as external erosion, heavy rain, snowmelt, and slope/structural instabilities (Davies 2002; Lyu et al. 2019; Rico et al. 2008; Villavicencio et al. 2013). The percentage of failure due to overtopping,

earthquake, seepage, and foundation failure is 20.6%, 17%, 21.6%, 17.3%, and 23.5%, respectively (Lyu et al. 2019). The failure rate of tailings dam is 1.2%, which is significantly higher compared to that of conventional water retention structures (i.e. 0.01%) (Arafat and Al 2017; Davies 2002; Lyu et al. 2019). This is mainly because i) tailings dams contain coarse and waste materials from mining operations, ii) elevation of deposited tailings increases as the elevation of tailings dam increases, and iii) there is a lack of proper design standard for tailings dams (Rico et al. 2008). Due to this reason, it is challenging and requires special attention to enhance the stability of tailings dams.

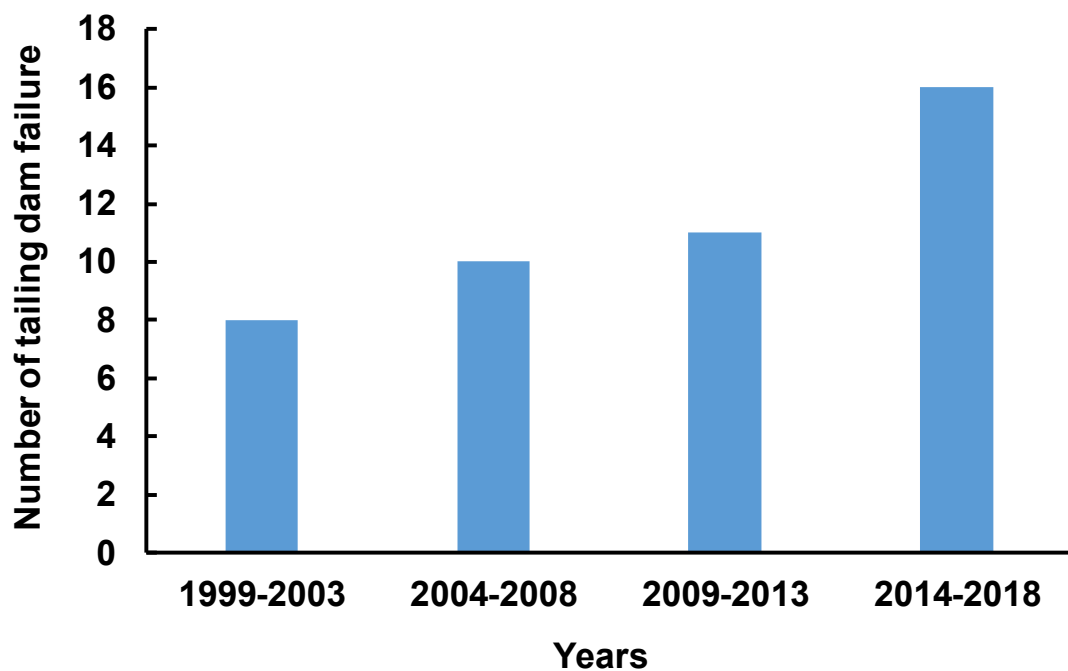


Figure 1.1. Number of tailings dam failures between 1999 and 2018 around the world (Armstrong et al. 2019)

Many tailings dams have been built in Canada over the past decades and most of them are underlain by soft clayey layers. In this case, predicting the behaviors of soft clayey layers is of great importance in the design of a tailings dam since the stability of the tailings dam is governed by the bearing capacity and deformability of the soft clayey layers (Davies 2002; Kossoff et al. 2014; Tavenas and Leroueil 1980). Soft clay has low permeability; therefore, loading on this type of soil can generate high excess pore-water pressure after deposition of tailings. Hence, it is reasonable to design a tailings dam extending the total stress approach based on the undrained shear strength of the soft clayey layers (Öser and Cinicioglu 2017; Taiebat and Carter 2002). In Canada, the eastern side of Ontario and Quebec, large parts of British Columbia and the southern side of James Bay possess clayey deposits at shallow depths.

Design and stability analysis of a tailings dams on soft clayey layers are usually performed using conventional limit equilibrium methods (LEM). However, these conventional methods have not been formulated to establish an appropriate scheme for finding stress variation under staged construction (Öser and Cinicioglu 2017; Tavenas and Leroueil 1980). In addition, previous studies on tailings dam failures showed that some failures were attributed to the undrained shear strength of soft layers that were determined inappropriately (Öser and Cinicioglu 2017; Oztoprak and Cinicioglu 2005; Tavenas and Leroueil 1980; Tavenas et al. 1978; Wheeler et al. 2003; Zhang et al. 1998).

1.2 Objectives of the Thesis

The main objective of this research is to evaluate the efficacy of including undrained failure in soft layers in estimating the stability of a tailings dam, taking account of the corresponding effective stress path. For this, a series of numerical analyses were performed for a tailings dam (Elkview Operations) using finite difference software (FLAC3D). In addition, a sensitivity analysis was also carried out to determine the most influential factor in the stability of the tailings dam.

1.3 Scope of the Thesis

Staged construction of a tailings dam causes a sequential undrained loading on soft foundation soil, which can consequently lead to a failure at a stress level less than the peak shear strength. To consider such a loading pattern, the effective stress path associated with staged construction should be taken into account in the stability analysis. In this study, standard geotechnical finite difference software, FLAC3D is used to simulate the behaviors of the clayey silt layer below a tailings dam (in Lagoon D of Elkview Operations (EVO)) under an undrained condition with elasto-plastic constitutive model (i.e. Modified Cam-Clay model). The factor of safety against undrained failure is obtained by establishing effective stress paths considering the development of excess pore-water pressure due to staged construction. Analyses were carried out by following three steps below:

- 1) Simulate the effective stress paths obtained from the triaxial tests to determine the input parameters for the Modified Cam-Clay model,

- 2) Evaluate the efficacy of including undrained failure in the clayey silt layer taking account of corresponding effective stress paths, and
- 3) Computing factor of safety against undrained failure using Undrained Strength Analysis (USA) approach

1.4 Thesis Hypotheses

To perform numerical modeling and establish effective stress paths during the construction of the tailings dam, the following assumptions were made in this research.

- 1) Each stage of construction comprises two consecutive steps; namely, deposition of tailings (under undrained loading) and consolidation,
- 2) Deposition of tailings occurs instantaneously,
- 3) Next stage of construction takes place after the consolidation in foundation soil completes, and
- 4) Foundation soil is homogeneous

1.5 Organization of the Thesis

This thesis consists of seven chapters including ‘Introduction’ (Chapter 1) and ‘Summary and Conclusions’ (Chapter 7).

Chapter 2 provides a literature review on properties of tailings, deposition methods of tailings, and types of tailings dams. Various causes of tailings dam failure are described with particular attention to the one associated with soft foundations. Different stability analysis approaches; limit equilibrium and numerical methods are also discussed.

In Chapter 3, the characteristic behavior of a soft foundation under staged construction is presented in terms of yielding and effective stress path.

Chapter 4 is devoted to the description of the Modified Cam-Clay constitutive model, including the details of virgin consolidation line, critical state line and yielding curve. In addition, the methodology of computing factor of safety against undrained failure taking account of effective stress paths is also presented.

Chapter 5 provides the details of EVO tailings dam facility, including geometry, location, and previous geotechnical investigations and stability analyses. The step-by-step procedure of identifying Modified Cam-Clay constitutive model input parameters based on the available laboratory test results obtained for the sampled collected from the site is given. The methodology used to simulate staged construction in FLAC3D is also explained.

In Chapter 6, the stability of the tailings dam (i.e. EVO) is discussed based on the numerical analysis results and effective stress paths. The estimated displacements in the foundation beneath the tailings dam and sensitivity analysis results are also briefly explained.

2. TAILINGS AND STABILITY OF TAILINGS DAM

2.1 Tailings

Tailings are byproducts left over after extracting the desired minerals from the uneconomic fraction of an ore. The definition of tailings is typically more pertinent to the byproducts from the mining process rather than other activities such as near-surface cut (Arafat and Al 2017). Tailings consist of crushed rocks and process effluents that are generated in a mine processing plant (Vick 1990).

It takes several steps to generate tailings and deposit them, as can be seen in Figure 2.1. The disposal method of tailings depends on topography and climatic, operational, economic, geotechnical, and geochemical considerations (Kujawa 2011). Generally, tailings are deposited in two ways: i) underground deposit or ii) on-surface deposit in the form of impoundment. Underground disposal of tailings is part of the mining planning, and its stabilization depends on the ore condition (Priscu et al. 1999)

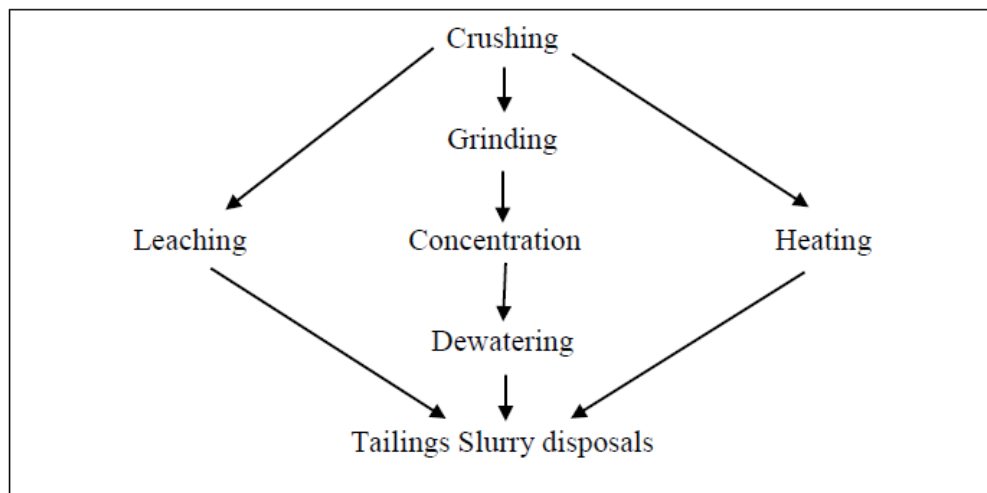
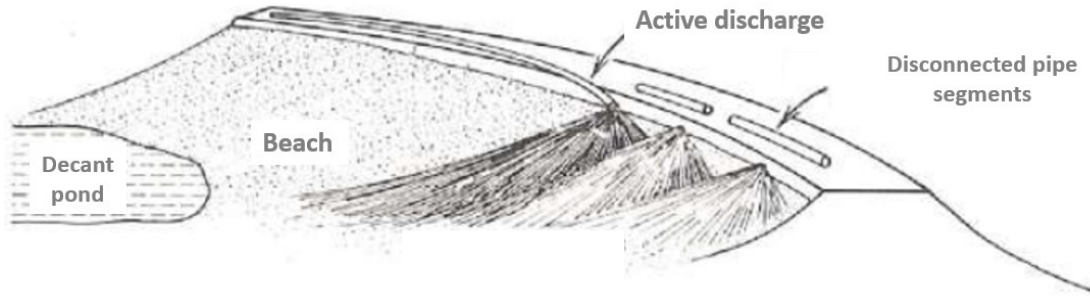


Figure 2.1. Tailings deposition process

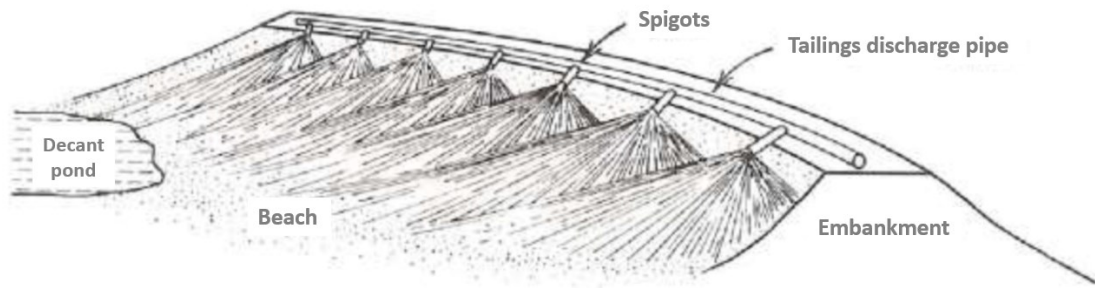
In the on-surface disposal method, tailings are mainly in a form of slurry, which are deposited into the impoundments by channels (Figure 2.2). Perimeter dams are then built to surround the impoundments. Single point discharge or multiple spigot discharge are typically used to deposit tailings into the impoundments (Agency 1994). Areas near the spigots contain coarser tailings compared to the areas in the middle of the impoundment. Recently, a new deposition method such as the filtering process was introduced. This method is, however, costly compared to the slurry disposal method since it includes the processes of lowering the water content of the tailings (i.e. dewatering) and compacting them to reach a 60% to 80% degree of saturation. The probability of failure due to liquefaction in filtered tailings is very low compared to other conventional slurry methods (Fitton and Roshdiah 2013).

Except for the tailings obtained from oil sands, tailings are usually angular, bulky, grained sand and silt size particles (Arafat and Al 2017). Estimating the mechanical properties of tailings is challenging because of the heterogeneity of chemical and particle composition (Vick 1990). The void ratio of the sandy tailings is influenced by the amount of fine particles. The typical void ratio of sandy tailings with a low and high volume of fine particles ranges from 0.6 to 0.9 and 0.7 to 1.3, respectively. The density of tailings varies in the range of 7 to 17 kN/m³, depending on the ore material and deposition techniques (Vick 1990). Hydraulic conductivity of tailings is another important parameter that can control the stability of the tailings dam. Estimating hydraulic conductivity is not straightforward because its magnitude depends on the grain size distribution, deposition method, stress history and chemical composition (Alsharedah 2015). Therefore, it has been

suggested to use the average value of hydraulic conductivity. The average hydraulic conductivity of tailings decreases with increasing silt and clay contents.



(a)



(b)

Figure 2.2. On-surface deposit of tailings: a) single point discharge; b) multiple spigot discharge

Mittal and Morgenstern (1975) proposed to use classic Hazen's equation Eq. (2.1) to estimate average hydraulic conductivity, k_{avg} of tailings

$$k_{avg} = \bar{C}(D_{10})^2 \quad (2.1)$$

where C is hydraulic conductivity correlation factor, and D_{10} is effective size (i.e. the diameter in the particle-size distribution curve corresponding to 10% finer).

Zardari (2011), however, stated that using any empirical equations for estimating k_{avg} can lead to overestimation because of uncertainties that exist in the tailings. For example, hydraulic conductivity of sandy tailings drops significantly at greater depth (Arafat and Al 2017; Gorakhki et al. 2016). Also, due to the vertical deposition of tailings, hydraulic conductivity in the horizontal direction differs from the vertical one. The ratio of horizontal to vertical hydraulic conductivity is typically between 2 and 10 for sandy tailings while it is more than 100 for slims (Davies 2002). The typical hydraulic conductivity values of tailings available in the literature are between 1×10^{-3} cm/s and 1×10^{-8} cm/s (Fitton and Roshdieh 2013; Lyu et al. 2019).

2.2 Tailings Dam

The purpose of building tailings dams is to retain toxic material from entering the environment. Several factors influence the design and construction of tailings dams such as the topography of the site, type of mining, foundation condition and disposal method (Berghe et al. 2011; Agency 1994; Lyu et al. 2019). In general, there are two types of tailings dams: i) retention impoundment and ii) raised embankment. Retention impoundment is the same as water retention dams that are built in full height before deposition of tailings. Figure 2.3 illustrates a retention tailings dam with a core in the center. Retention tailings dams are chosen in case where significant run-off is expected (Arafat and Al 2017).

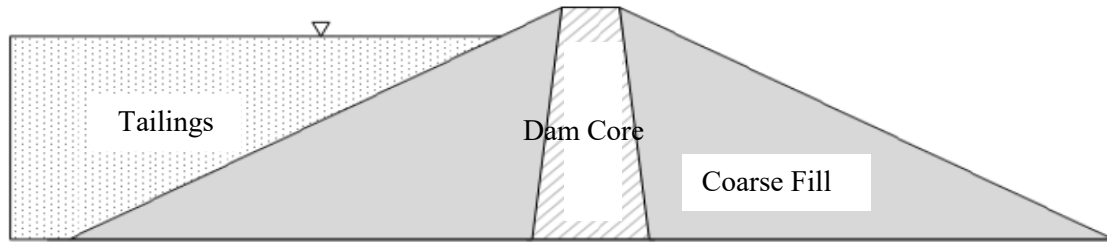
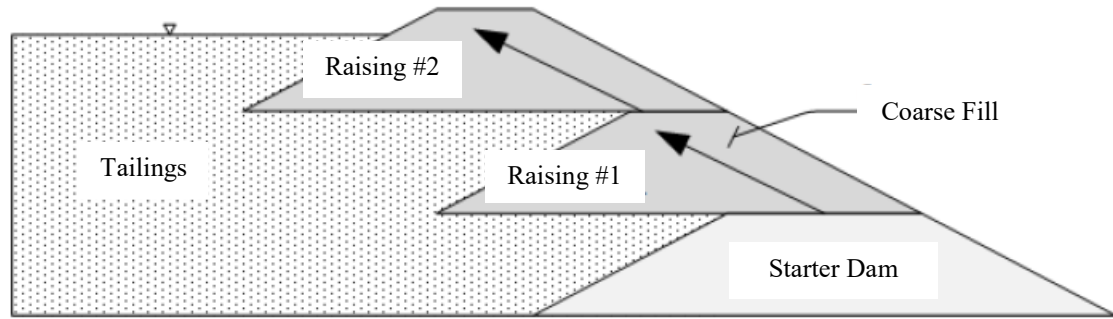
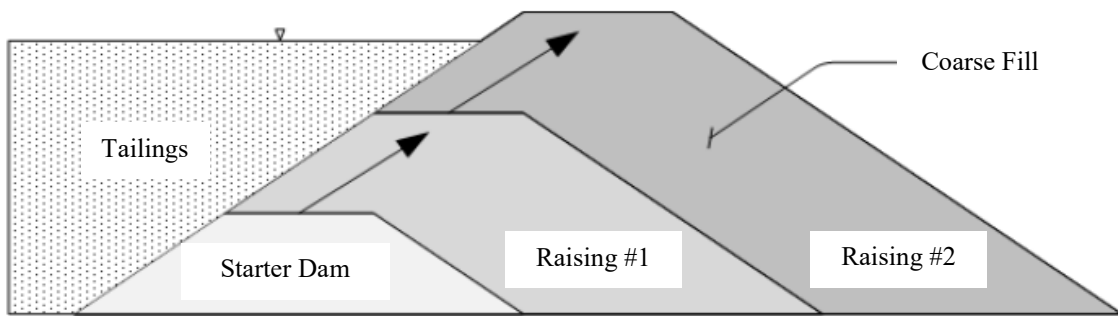


Figure 2.3. Retention tailings dam with a core in the center

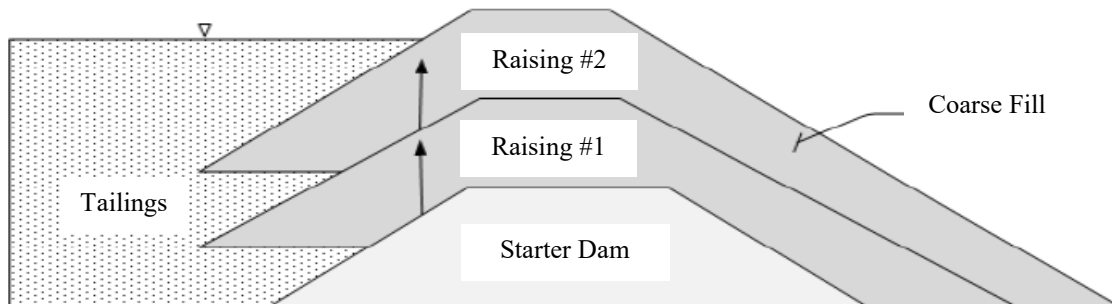
On the other hand, raised tailings dams are not built in full height. Instead, a starter dike is first constructed, followed by subsequent raisings. There are three methods of raising tailings dams; namely, upstream, downstream and centerline method (Deng et al. 2017; Lyu et al. 2019; Vick and Ladd 1992). If a tailings dam is raised in the upstream direction, it is called the upstream method (Figure 2.4(a)), while, in the downstream method, a tailings dam is raised in the downstream direction (Figure 2.4(b)). In the centerline method, a tailings dam is raised on both sides (upstream and downstream) simultaneously (Figure 2.4(c)).



(a)



(b)



(c)

Figure 2.4. Schematic cross-section of a tailings dam: (a) upstream; (b) downstream; (c) centerline method

In the upstream method, tailings are deposited in the impoundments in a shape of beach and are consolidated with time due to their weights. Each stage of raising is built on the

previously consolidated beach. This has been the most popular method in the mining industry because self-consolidation is more economical. However, its popularity is declining as more upstream tailings dam failures are reported recently (Lyu et al. 2019). The long-term stability of tailings dams built with the upstream method contains uncertainties in determining the level of water and pore-water pressure under each stage of raising. In this method, the magnitude of pore-water pressure is of great importance since high excess pore-water pressure may lead to the occurrence of static liquefaction within tailings (Jefferies and Been 2015; Lyu et al. 2019).

Generally, downstream tailings dams are more beneficial compared to upstream tailings dams in terms of stability and drainage. Any raising rates can be implemented in this method since the stability of the tailings dam does not depend on the strength of tailings. The impervious cores and drainage system can be implemented to provide better control over the fluctuation of water level. Building a downstream tailings dam, however, requires a lot of fill materials to stabilize embankments. This can cause the outward movement of the dam's toe (Arafat and Al 2017; Villavicencio et al. 2013). Due to this reason, the upstream method is still preferred in some cases among the designers (Lyu et al. 2019; Vick 1990). In the case of centerline tailings dams, it is possible to fully control both drainage and phreatic line (i.e. water level). Centerline tailings dams provide a balance between upstream and downstream sides and require less fills in comparison to the downstream method. They are more resistant to seismic loading because, even though the upstream part of the dam lies on tailings, the downstream's passive stresses contribute to the resistance to failure during a seismic loading (Arafat and Al 2017; Vick 1990). Centerline tailings dams can also hold less amount of tailings compared to other methods.

2.3 Tailings Dam Failure

As mentioned earlier, instability of tailings dams can be caused by various factors such as i) seepage and internal erosion, ii) poor foundation soil, iii) overtopping, iv) static and seismic instability, and v) external erosion, heavy rain, snowmelt, unknown causes, and slope and structural instabilities. Table 2-1 shows examples of tailings dam failures with dam height, dam type, failure cause, and fatalities (Cases in Canada are highlighted). Following subsections present details of the factors that contribute to tailings dam failures.

Table 2-1. Examples of tailings dam failures

Year	Name (location)	Dam height (m)	Dam type	Failure cause (fatalities)
1928	Barahona (Chile)	61	Upstream	Earthquake (54)
1937	Dos Estrellas (Mexico)	UN	Upstream	Seepage (70)
1962	Huogudu (China)	UN	Upstream	Foundation failure (171)
1966	Aberfan (UK)	UN	Water retention	Seepage (144)
1972	Buffalo Creek (USA)	14–18	Upstream	Seepage (125)
1975	Mike Horse (USA)	18	Upstream	Overtopping (UN)
1978	Syncrude (Canada)	UN	Centerline	Foundation failure (UN)
1979	Union Carbide (USA)	43	Upstream	Seepage (UN)
1985	Cerro Negro (Chile)	40	Upstream	Earthquake (UN)
1988	Lixi (China)	40	Upstream	Overtopping (20)
1991	Sullivan (Canada)	21	Upstream	Seepage (UN)
1995	Surigao (Philippines)	UN	Upstream	Foundation failure (12)
1998	Los Frailes (Spain)	27	Upstream	Foundation failure (UN)
2004	Pinchi Lake (Canada)	12	Water retention	UN (UN)
2009	Karamken plant (Russia)	20	UN	UN (1)
2010	Ajka (Hungary)	22	Downstream	Seepage (10)
2011	Kayakari (Japan)	UN	Unknown	Earthquake (UN)
2012	Padcal No. 3 (Philippines)	UN	Upstream	Overtopping (UN)
2014	Mount Polley (Canada)	40	Upstream	Foundation failure (UN)
2015	Fundo (Brazil)	90	Upstream	Seepage (19)
2017	Tonglvshan (China)	UN	Upstream	seepage (UN)
2019	Córrego de Feijão (Brazil)	86	Upstream	UN (UN)

UN: Unknown

2.3.1 Seepage

As tailings contain water, the stability of tailings dams can be significantly affected by the seepage field. In this case, determining the location of the phreatic line in the seepage field (also called ‘lifeline’) is of great importance in estimating the stability of tailings dams. This is because tailings below the phreatic line is susceptible to static liquefaction due to a slow rate of consolidation. Heavy rainfall and floods cause the phreatic line to move upward, which in turn leads to a drop in shear strength of the tailings dams and possible failure. Subsequent piping can also be expected due to the osmosis effect.

2.3.2 Overtopping

In case of a tailings dam built near mountains, the water level in the tailings dam can be significantly increased in short period of time due to heavy rainfalls or snow melting (Blight 1997). The water then may flow over the tailings dam and runoff on the surface of the tailings dam. This phenomenon is called overtopping, which substantially affects the stability of the tailings dam. Overtopping also causes hydraulic and gravity erosion, which intensifies the instability of the tailings dam (Azam and Li 2010). Limited studies have been undertaken regarding tailings dam failures induced by overtopping (Lyu et al. 2019).

2.3.3 Earthquake

Earthquake induces seismic loading onto a tailings dam, which weakens the strength of tailings and eventually may lead to liquefaction. Liquefaction within tailings is affected by various factors such as shape/size/gradation/arrangement of particles, compactness, depth of wetting line, and seismic intensity (Lyu et al. 2019). Seismic performance analysis of

tailings dams consists of seismic liquefaction analysis, seismic stability analysis and seismic deformation analysis. The undrained liquefaction strength is a vital parameter in evaluating the potential of liquefaction in tailings dams (Sadrekarimi 2014). This strength parameter can be estimated by either empirical equations, numerical modeling or in-situ experiments (Kujawa 2011; Villavicencio et al. 2013; Sadrekarimi 2014). Villavicencio et al. (2013) stated that the value of undrained shear strength estimated using numerical methods is influenced by the chosen input parameters.

2.3.4 Foundation Failure

According to previous studies, 17% of tailings dam failures were attributed to the existence of soft layers in their foundations (Gens and Alonso 2006; Rico et al. 2008). When a soft layer undergoes a multi-stage construction loading, the risk of undrained failure in the foundation increases. It is especially true in the case where each stage of loading is applied at a fast rate, which builds up the pore-water pressure in the soft layer due to low permeability. This phenomenon lowers the effective stress significantly and, at a certain point, undrained failure will take place as the mobilized strength of the soft layer becomes less than the undrained shear strength (Ladd 1991). However, the failure of Mont Polly tailings dam (Morgenstern et al. 2015) showed that a tailings dam can fail even if the preliminary design of a tailings dam is carried out using the permeability and undrained shear strength of a soft layer evaluated reliably through laboratory and in-situ tests. Arafat and Al (2017) performed back analysis on Mount Polly tailings dam failure case and concluded that neglecting the variation of stiffness in a soft layer might be a major reason for the failure. On the other hand, Morgenstern et al. (2015) stated that the increase in shear

strain rate between two consecutive stages of tailings deposition (from 2% to 8%) caused the tailings dam failure.

2.4 Approaches to Estimate Stability of Tailings Dams

In current practice, the stability of a tailings dam is analyzed extending the conventional slope stability method (i.e. limit equilibrium method). Tailings dams are considered as complex slopes. Factor of Safety (FOS) is defined as the ratio of available shear strength to mobilized shear stress Eq. (2.2) along the potential failure surface.

$$FOS = \frac{\sum \tau_f}{\sum \tau_m} \quad (2.2)$$

where τ_f is shear strength and τ_m is mobilized shear stress.

The acceptable FOS in slope stability analysis varies depending on the design codes and field conditions. Canadian Dam Association (CDA) suggests FOS = 1.3 during the construction period, 1.5 after the completion of construction, and 1.2 for rapid drawdown condition. The shear strength (or shear strength parameters) of a soil should be determined considering the drainage condition of pore-water pressure, which is governed by loading rate and soil type.

2.4.1 Drained and Undrained Shear Strength

The stability problem for a tailings dam built on a soft clayey layer by staged construction method can be conducted considering three different drainage conditions: i) undrained (or

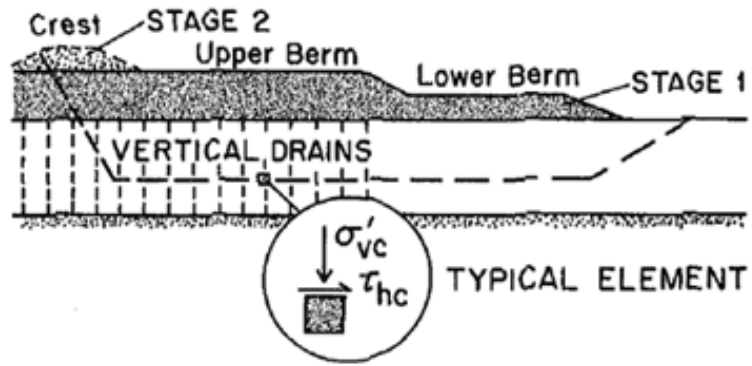
short-term), ii) drained (or long-term), and iii) partially drained (or intermediate) conditions.

The undrained condition (i.e. short-term) is achieved when construction and failure occur rapidly without causing any changes in water content in the soft clayey layer. In reality, however, water content slightly changes at the beginning of construction, which can be neglected in the stability analysis. The stability analysis performed assuming undrained condition is denoted as total stress approach (TSA). The TSA uses undrained shear strength that can be determined from Unconsolidated-Undrained (UU) triaxial tests or vane shear tests with zero internal friction angle. FOS becomes minimum under undrained condition, and then gradually increases with the dissipation of excess pore-water pressure (i.e. consolidation). However, the TSA does not consider the gain in strength in association with the dissipation of excess pore-water pressure during consolidation. On the other hand, the drained (i.e. long-term) condition is achieved when the construction and shearing rate are slow enough such that it can be assumed that no excess pore-water pressure is generated in a soft clayey layer. The stability analysis conducted assuming a drained loading condition is denoted as effective stress approach (ESA). The ESA uses the drained shear strength parameters, c' (effective cohesion) and ϕ' (effective internal friction angle). In ESA, it is assumed that the effective normal stress remains constant during failure. The assumptions in the ESA and the TSA can result in different FOS for the same stability problem.

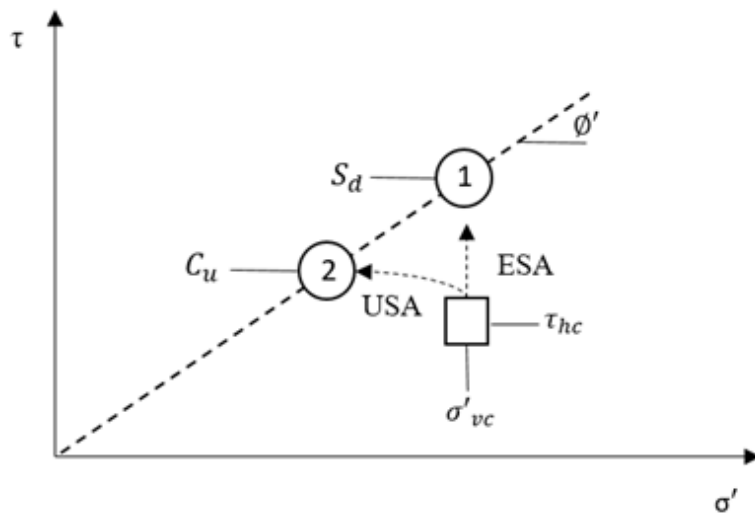
The partially drained condition represents the case where a soft clayey layer is first partially or fully consolidated under a loading. There has been a controversy about the drainage condition of pore-water pressure during shearing in soft soil simply because the pore-water pressure cannot be measured during shearing in the field. There are two hypotheses stated

in the literature regarding the pore-water pressure during shearing: i) Failure takes place very slowly and excess pore-water pressure is close to zero or ii) Excess pore-water pressure generated during shearing is significantly higher than hydrostatic condition. These two drainage conditions can be achieved using CD (consolidated-drained) and CU (consolidated-undrained) triaxial tests, respectively.

Casagrande and Fadum (1940) proposed an approach to estimate the shear strength of a soil under staged construction, which is denoted as undrained strength analysis (USA). This approach accommodates both ESA and TSA simultaneously. In other words, this approach considers the gain in undrained shear strength due to consolidation in computing FOS. Figure 2.5 compares ESA and USA for a staged construction scenario. Table 2-2 summarizes different stability problems classified based on drainage condition and the definition of FOS.



(a)



(b)

Figure 2.5. Comparison of ESA and USA: (a) field situation for partially or fully consolidated clay foundation; (b) strengths predicted from ESA and USA (after Ladd 1991)

Table 2-2. Stability problems classified according to drainage condition and definition of factor of safety (modified after Ladd 1991)

Drainage condition	Consolidation and drainage condition	Proposed classification	Factor of safety
Undrained or short-term	No consolidation under applied stress and undrained during shearing	UU test	s_u/τ_m or c_u/τ_m
Drained or Long-term	Full consolidation under applied stress and drained during shearing	CD test	$s_d/\tau_m = \phi'/\phi'_m$
Partially drained or Intermediate	Partial or full consolidation under applied stress and undrained during shearing	CU test	c_u/τ_m

where τ_m = mobilized shear stress required for equilibrium, s_u = undrained shear strength from conventional testing ($\phi_u = 0$), c_u = undrained shear strength (point 2 in Figure 2.5), and s_d = drained shear strength (point 1 in Figure 2.5)

As can be seen in Table 2-2, undrained shear strength is an important parameter in analyzing the stability of a slope. For this, the ratio of undrained shear strength (c_u) to effective overburden pressure (σ'_{v0}) (i.e. c_u/σ'_{v0}) has been used since last 1940s by correlating it to some index properties, I_i in a general form, shown in Eq. (2.3). The parameters in Eq. (2.3) are proposed by several researchers as summarized in Table 2-3 (Bowles 1992).

$$\frac{c_u}{\sigma'_{v0}} = A_0 + B_0(I_i)^F \quad (2.3)$$

Table 2-3. Summary of parameters in Eq. (2.3) (Bowles 1992)

Reference	A_0	B_0	I_i	F	Remarks
Skempton and Henkel (1953)	0.11	0.0037	I_p	1	$I_p > 0.5$
Bjerrum and Simons (1960)	0	0.45	I_p	0.5	$I_L^{a)} > 0.5$
	0	0.18	I_L	0.5	
Karlsson and Viberg (1967)	0	0.5	w_L	1	$w_L^{b)} > 0.20$
Ladd and Foott (1974)	0	0.23	-	-	OCR = 1
	0	0.21	-	-	OCR = 2
Mesri (1975)	0	0.22	-	-	
Larsson (1980)	0	0.23	-	-	$I_p > 0.6$
Wang et al. (2008)	0	0.204	-	-	$20^\circ \leq \phi' \leq 25^\circ$
	0	0.239	-	-	$25^\circ \leq \phi' \leq 30^\circ$
	0	0.269	-	-	$30^\circ \leq \phi' \leq 35^\circ$

a) liquidity index, b) liquid limit

It is worth mentioning that all suggested single values for c_u/σ'_{v0} ratio were obtained from the analysis using the cases of embankment failures for normally consolidated (or lightly overconsolidated) clays. The undrained shear strength for each case was obtained along the failure surface (Mesri and Huvaj 2007; Wang et al. 2008). Bjerrum (1972) investigated the field vane test results on post-glacial clays and showed that there is a non-linear relationship between c_u/σ'_{v0} ratio and I_p . Bjerrum (1972) then back-calculated undrained shear strength of soft clayey layers with the cases of embankment failures and suggested

that field vane test results should be corrected as shown in Eq. (2.4). The correction factor, $\bar{\lambda}$ is influenced by anisotropy of the natural soil and shearing rate.

$$c_{u(\text{design})} = \bar{\lambda} c_{u(FV)} \quad (2.4)$$

where $c_{u(\text{design})}$ is a design undrained shear strength, $c_{u(FV)}$ is an undrained shear strength from field vane test and $\bar{\lambda}$ is a correction factor (See Figure 2.6).

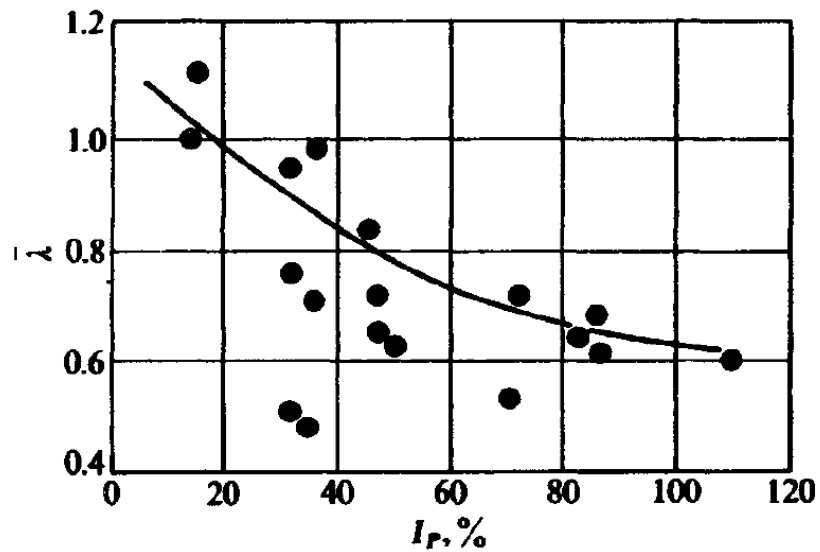


Figure 2.6. Bjerrum's correction factor for the vane shear test (Bjerrum 1972)

Mesri and Huvaj (2007) emphasized that because the undrained shear strength of a soil is a stress path dependent parameter, it is not rational to assign undrained shear strength values achieved from the triaxial compression test to the whole sliding surface. As can be seen in Figure 2.7, the state of elements along the sliding surface can be divided into three different stress paths; namely, triaxial compression, simple shear and triaxial extension. The stress path corresponding to each of these elements leads to different undrained shear strength that cannot be represented by a single triaxial compression test result. Mesri and

Huvaj (2007) suggested that the mean value of undrained shear strength values obtained for those three stress paths can be used in stability analysis. According to Ohta and Nishihara (1985), the magnitude of undrained shear strength is dependent on the stress history and effective stress at failure (i.e. basic concept of SHANSEP model).

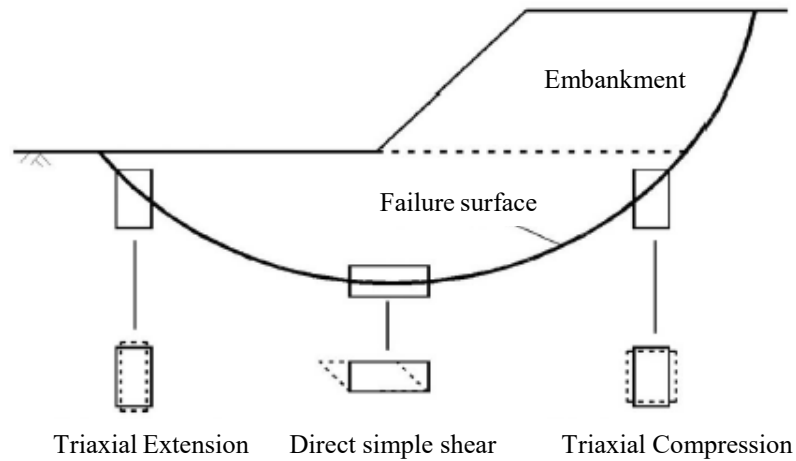


Figure 2.7. Stress paths of elements along the slip surface (Mesri and Huvaj 2007)

2.4.2 Limit Equilibrium Method

The approaches mentioned above (i.e. TSA, ESA and USA) have been traditionally implemented in Limit Equilibrium Method (LEM; Fellenius 1936) for the stability analysis. The concept behind the LEM is finding a circular slip surface with the minimum FOS against sliding. Based on the Fellenius model, several other approaches have been proposed to compute FOS against sliding based on different assumptions. Those assumptions are mainly about the shape of slip surface (i.e. circular or non-circular), calculation of FOS (i.e. moment equilibrium or force equilibrium) and interslice forces (Duncan 1996). The shear strength along the potential slip surface is assumed to be

governed by linear Mohr-Coulomb failure criteria in LEM. The influence of deformation towards the stability of slopes is not considered in the analysis (Dong et al. 2020)

2.4.3 Numerical Method

The numerical method can provide a solution to cases of which traditional stability analysis approaches (such as LEM) fail to take account of non-linear soil behavior, complex geometry, in situ stresses, etc. The numerical method is mainly divided into three groups; namely, continuum, discontinuum and hybrid (Eberhardt 2003). The continuum method is suitable for modeling soil slope and massive intact rocks, while the discontinuum method is an approximate solution for rock slope. The hybrid (or coupled method) is suitable for problems that need to be solved with more than one methodology.

Recently, the continuum modeling approach has gained more popularity in the field of geotechnical engineering. This approach includes Finite Element Method (FEM) and Finite Difference Method (FDM), in which the whole domain of soil is discretized into a finite number of elements. The difference between FEM and FDM is the approach used to approximate solutions for the problem. In FEM, the approximation is performed for the connectivity of elements and continuity of strains and stresses between two elements (Eberhardt 2003). In FDM, the global differential equation is approximated using discrete approximations. Generally, FEM approximates solution from the global point of view, while FDM uses local approximations (i.e. Solve a differential equation for discretized domain) for modeling (Gray 1984). However, the response of the soil to external and internal loadings can be approximated almost in the same manner for both methods.

FEM and FDM are increasingly used in slope stability analysis (Potts et al. 2001). The main advantage FEM and FDM over LEM is that the linear and non-linear stress-strain behaviors of soil can be simulated (Arafat and Al 2017). FEM and FDM allow users to estimate not only FOS but also deformation in soil. FOS can be computed by reducing the shear strength parameters (i.e. c' , ϕ' , c_u) by a factor (i.e. FOS) in each zone of the soil domain until a slope reaches a failure condition (Cheng et al. 2007). This method is called Strength Reduction Method (SRM). Cheng and Lau (2014) stated that FOS obtained using SRM against a failure seems to be more realistic since no assumptions are required in determining the shape or location of the slip surface. Several attempts were made to compare FOSs obtained from LEM and numerical method (with SRM) (Gens and Alonso 2006; Ladd 1991; Lyu et al. 2019). The results showed that there was a good agreement between the FOSs obtained from both methods; however, the location and shape of the critical surface did not match (Cheng et al. 2007).

3. BEHAVIOR OF FOUNDATION BENEATH TAILINGS DAMS

The behavior of the foundation governs the stability of a tailings dam since yielding or failure in the foundation can lead to the global failure of the tailings dam. Hence, to avoid such a failure, it is of utmost importance to predict the behavior of a foundation under a critical loading condition considering for different practical scenarios: i) during construction, ii) at the completion of the structure, iii) long-term condition and iv) rapid drawdown condition (Zardari 2011). Among these, the loading condition during the staged construction of the tailings dam is of more concern since the generation and dissipation of excess pore-water pressure during deposition of tailings take place simultaneously and repeatedly. The change in pore-water pressure during the staged construction can be predicted through numerical analysis. Based on this information, an effective stress path (ESP) for a certain location within a foundation can be established, which in turn can be used to check whether the foundation will reach yielding condition or not (Tavenas and Leroueil 1980).

To establish a reliable and rational ESP from numerical analysis, an appropriate constitutive model must be first chosen that can represent the behavior of the foundation under staged loading. Çinicioglu and Toğrol (1991) suggested that the behavior of soil under staged loading can be simulated by using the Modified Cam-Clay constitutive model (hereafter referred to as MCC model) originally proposed by Roscoe and Burland (1968). In MCC model, an increase in undrained shear strength with respect to effective overburden and preconsolidation pressures can be considered (e.g. SHANSEP model). Öser and Cinicioglu (2017) proposed a method to estimate the stability of an embankment

built with the staged construction technique based on the ESP in the foundation obtained using the MCC model. The proposed approach was verified by using the well-documented field data of the Cubzac-les-Ponts and Mura est embankments.

Typically, a soft foundation beneath an embankment does not undergo a uniform loading during construction. In this case, it is recommended to discretize the foundation into several zones and then construct ESP for each zone separately. The reliability of the predicted foundation behavior can be further improved by using the field measurements data (e.g. pore-water pressures and displacements during construction from piezometers and inclinometers, respectively) (Oztoprak and Cinicioglu 2005). Especially, the measured strain ratio (i.e. horizontal strain/vertical strain) in each zone provides the information of soil heterogeneity (i.e. Strain ratio other than 1 indicates anisotropic condition). However, the implementation of such methods is costly and almost impossible since usually i) the area of interest is extremely large, and ii) the construction period of tailings dams is more than 100 years. The heterogeneity in association with strain ratio is not considered in the present study.

3.1 Yielding of Soft Clay Foundation

Yielding of clay foundation beneath tailings dam has an important influence on the general behavior of the tailings dam. Continuous yielding in the foundation generates plastic or irrecoverable deformations, which subsequently increases pore-water pressure (Smadi 2002; Wood 1990). Yield locus in shear stress versus normal stress space, defined originally by Mitchell (1970), separates elastic behavior from plastic behavior. In other words, the stress states inside the yield locus indicate elastic (i.e. recoverable) deformation,

while stress states on or outside the yield locus indicate plastic (i.e. irrecoverable) deformation. The most common way of determining the yielding point of a clay is to conduct one-dimensional consolidation test. Figure 3.1 shows an example of one-dimensional consolidation (i.e. oedometer test) compression curve and the corresponding ESP. The yielding point is defined as the preconsolidation pressure (σ'_p) (Casagrande 1936). As can be seen, the slope of ESP changes when the stress exceeds the yielding point (i.e. σ'_p), where the clayey layer starts showing plastic behavior. The horizontal effective stress in the soil under one-dimensional consolidation condition (σ'_h) can be estimated using the effective vertical overburden pressure (σ'_v) and lateral earth pressure coefficient, K (i.e. $\sigma'_h = K\sigma'_v$). Unlike the oedometer test, in the triaxial test, samples can be consolidated under different preconsolidation pressures, which allows measuring an infinite number of yielding points. Figure 3.2 shows examples of yielding curves for different preconsolidation pressures in $q - p'$ space, where p' Eq. (3.1) and q Eq. (3.2) denote mean effective stress and deviatoric stress, respectively. An increase in σ'_p expands the size of the yielding curve, but not the shapes of curves.

$$p' = \frac{\sigma'_v + 2\sigma'_h}{3} \quad (3.1)$$

$$q = \sigma'_v - \sigma'_h \quad (3.2)$$

where σ'_v = vertical effective stress and σ'_h = horizontal effective stress.

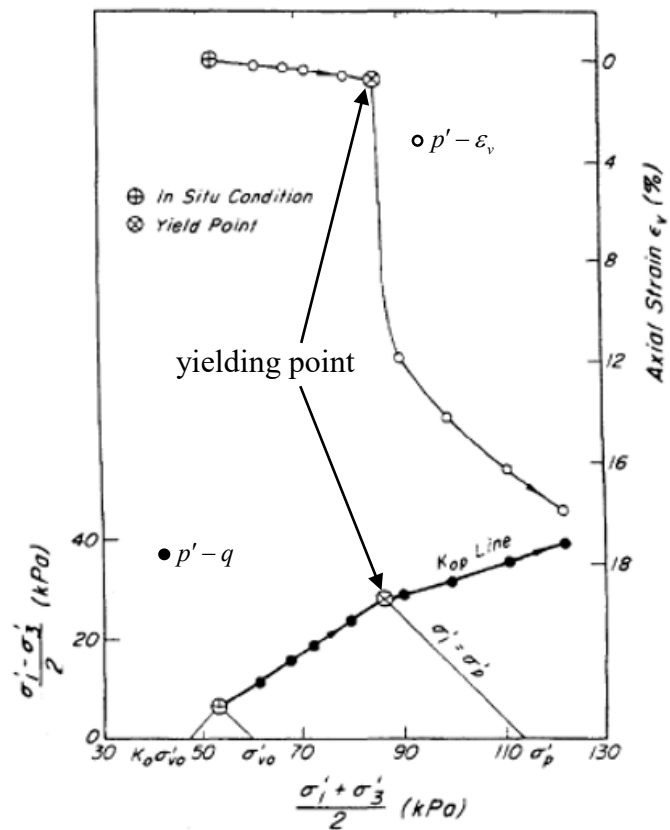


Figure 3.1. Effective stress path in the oedometer test (Terzaghi et al. 1996)

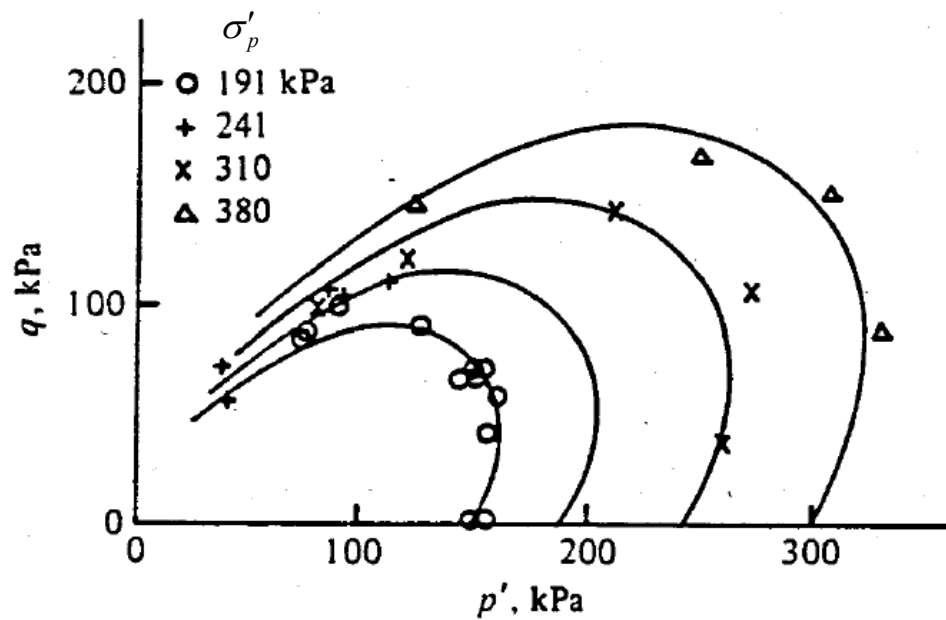


Figure 3.2. Yielding curves for different preconsolidation pressures (Wood 1990)

In most in-situ natural clay deposits, preconsolidation pressure is usually higher than vertical effective stress (i.e. overconsolidated). This overconsolidated condition can be due to various factors such as geologic process, snow loads, erosion, water table fluctuations, chemical changes, or secondary compression, etc. (Smadi 2002). An example of void ratio versus $\log \sigma'_v$ plot from an oedometer test is shown in Figure 3.3, where σ'_{v0} is in-situ vertical effective stress and σ'_v is final vertical effective stress (i.e. $\sigma'_{v0} + \Delta \sigma'$).

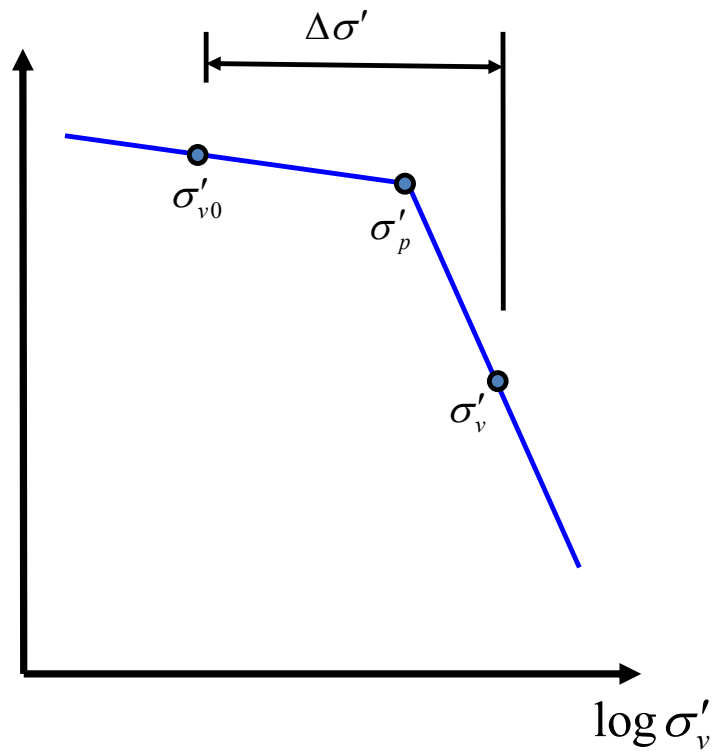


Figure 3.3. Typical compression curve from an oedometer test

Applying a load on an overconsolidated clay causes a relatively high dissipation of pore-water pressure during the initial phases of loading (Mesri and Rokhsar 1974). If the final effective stress is less than the preconsolidation pressure, small lateral deformation can be expected. On the contrary, yielding or large lateral deformation is expected in the clay once

et al. (1978) who investigated the excess pore-water pressure dissipation in a clay deposit beneath Saint-Alban test fills. The excess pore-water pressures under the center lines of three different fills were much lower compared to computed values. This behavior was irrelevant to the rate of embankment construction. Tavenas and Leroueil (1980) studied the excess pore-water pressures measured in clayey deposits beneath various embankments during the early stages of construction. The average pore-water pressure parameter, $\bar{B}_1$ in Eq. (3.3) was close to 0.4, which was significantly low compared to the value under undrained loading condition (Figure 3.5).

$$\bar{B}_1 = \frac{\Delta u}{\Delta \sigma_1} \quad (3.3)$$

where Δu is a change in pore-water pressure (i.e. excess pore-water pressure) and $\Delta \sigma_1$ is a change in vertical total stress.

The rapid dissipation of excess pore-water pressure causes ESP, $O'P'$ in Figure 3.4. Point P' corresponds to the preconsolidation pressure; therefore, this ESP stays within the yield envelope and the clayey deposit remains in a state of overconsolidated condition. If the first lifts of an embankment result in $\sigma'_v > \sigma'_p$ the clayey deposit behaves as a normally consolidated soil and ESP moves along the yield envelope from P' to F' with higher excess pore-water pressure development. With further increase in stress, ESP moves to the point F' where a localized failure is initiated (Mohr-Coulomb failure envelop for normally consolidated soil). Finally, strain-softening starts taking place as ESP moves to the point R' located on the critical state line (CSL).

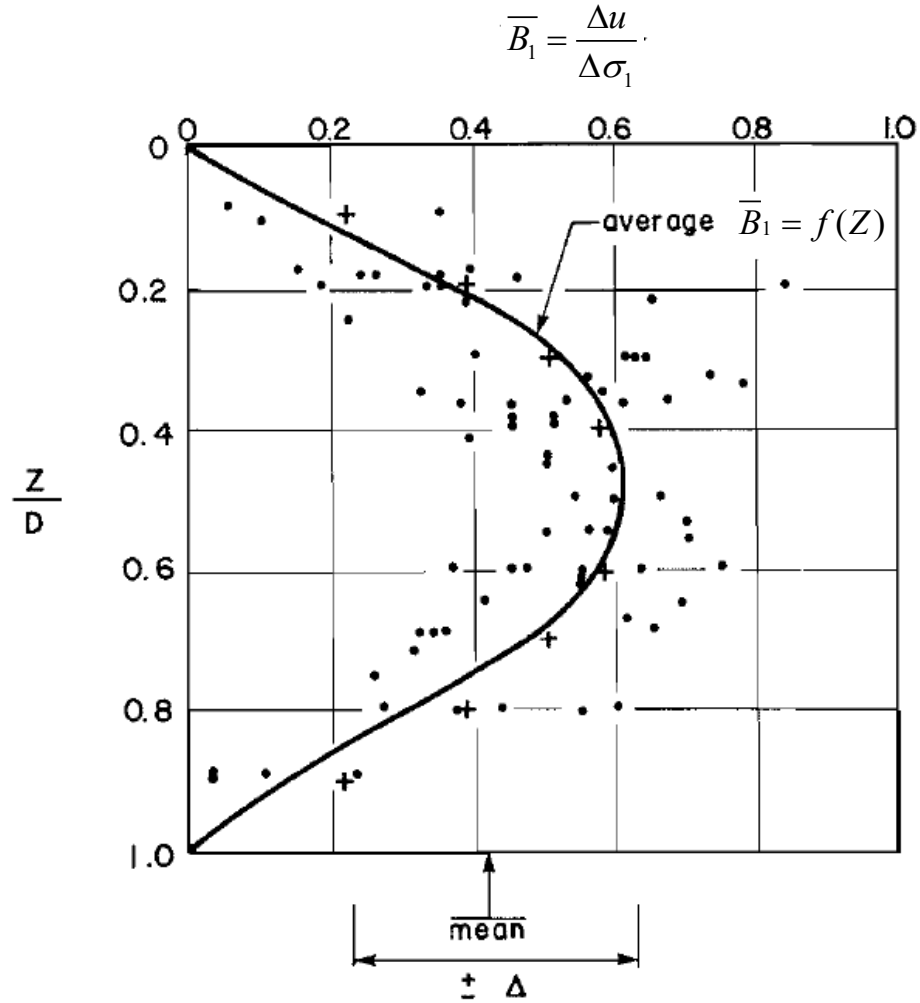


Figure 3.5. Compilation of observed average pore-water pressure parameters in clayey foundations at the beginning of embankment construction (Tavenas and Leroueil 1980)

In reality, ESP in a clayey deposit beneath an embankment can be different from the one in Figure 3.4; however, Folkes and Crooks (1985) and Leroueil and Tavenas (1986) pointed out that the majority of ESPs that they studied showed similar patterns. Leroueil and Tavenas (1986) introduced five possible ESPs under an embankment loading as shown in Figure 3.6.

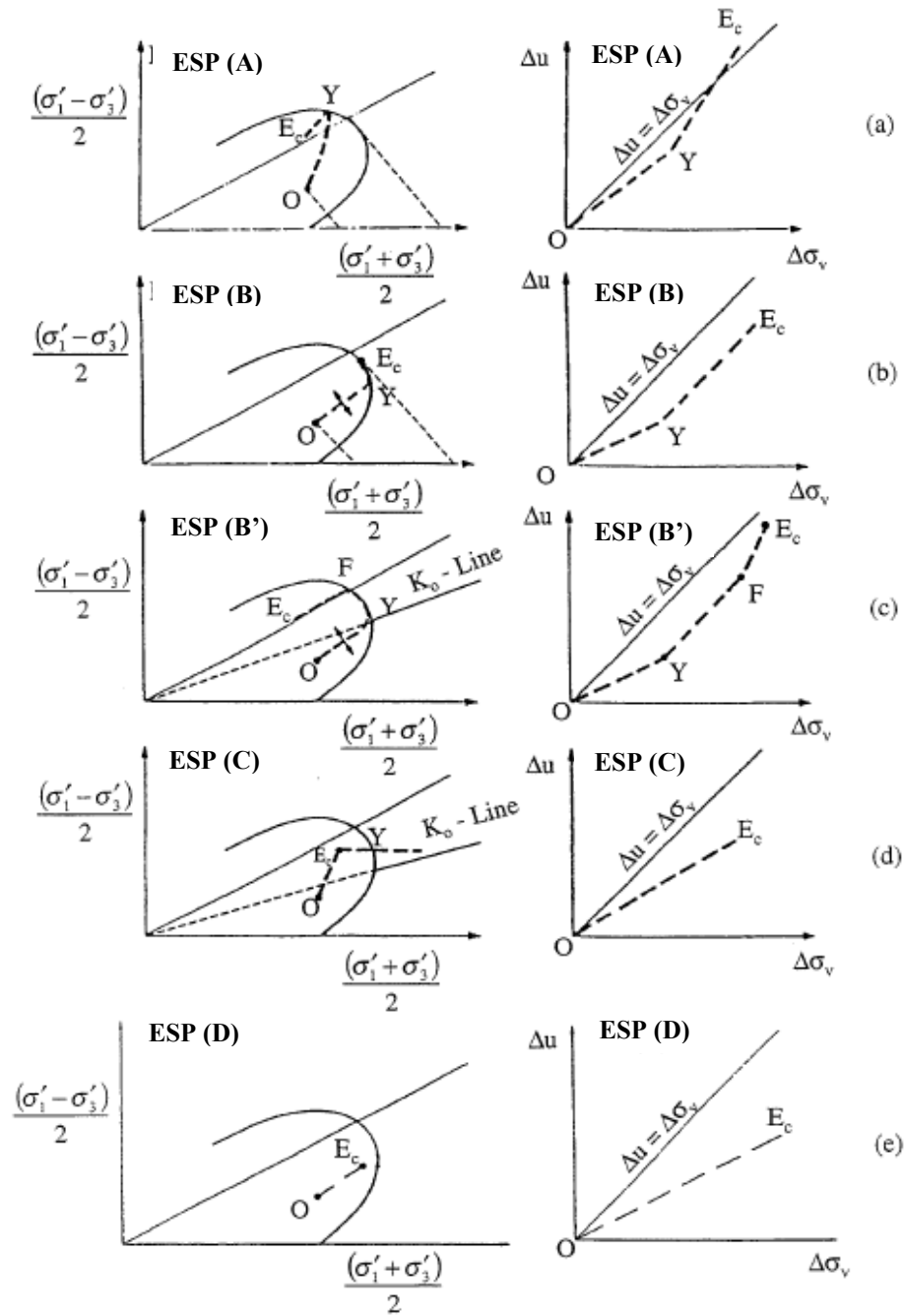


Figure 3.6. Effective stress paths and pore-water pressure generations under embankment loading (Leroueil and Tavenas 1986)

ESP (A) represents the case where failure and yielding occur at the same time. ESP moves from the initial stress condition O to point Y that is the yielding point located on the CSL.

This behavior is anticipated when the following conditions are satisfied in the field: i) small

difference between σ'_p and σ'_{v0} , ii) high loading rate, iii) low coefficient of consolidation value, and iv) thick clay deposit. The associated $\bar{B}_l$ value is slightly less than 1 before ESP reaches point Y and then gradually increases up to 1 at failure (or yielding). After reaching the point Y , $\bar{B}_l$ becomes greater than 1 (i.e. strain-softening).

ESP (B) applies to a heavily overconsolidated clayey deposit. Due to the great difference between σ'_p and σ'_{v0} , the consolidation and drainage take place at the early stages of loading, which results in $\bar{B}_l$ less than 1. Once ESP reaches a local failure condition on CSL (i.e. point Y), $\bar{B}_l$ becomes 1 (i.e. the slope of path $YE_c = 45^\circ$).

ESP (B') is a special case that is originated from ESP (B). ESP is the same as for ESP (B) until it reaches a local failure condition on CSL. With further stress increase, high excess pore-water pressure is generated (i.e. $\bar{B}_l > 1$) due to the considerable shear strain.

ESP (C) represents the case of rapid loading on a clayey deposit with a low coefficient of consolidation. Due to the rapid loading, the construction of an embankment completes before the end of consolidation. The dissipation of excess pore-water pressure starts at the end of construction, which increases effective stress under constant deviatoric stress. Since ESP does not reach the yielding curve, $\bar{B}_l$ is less than 1.

ESP (D) is a special case of ESP (C) in which final effective stress is less than preconsolidation pressure. Since ESP is entirely within the yielding curve, $\bar{B}_l$ is less than 1 and yielding does not occur.

Figure 3.7 shows typical TSP and ESP in a slightly overconsolidated clayey deposit beneath an embankment under three-stage construction. The TSP, OA indicates the first

construction phase. During this period, ESP becomes $O'P'A'$ due to the reason as explained previously using Figure 3.6(c). The initial limit state surface is also generated under the same stress condition.

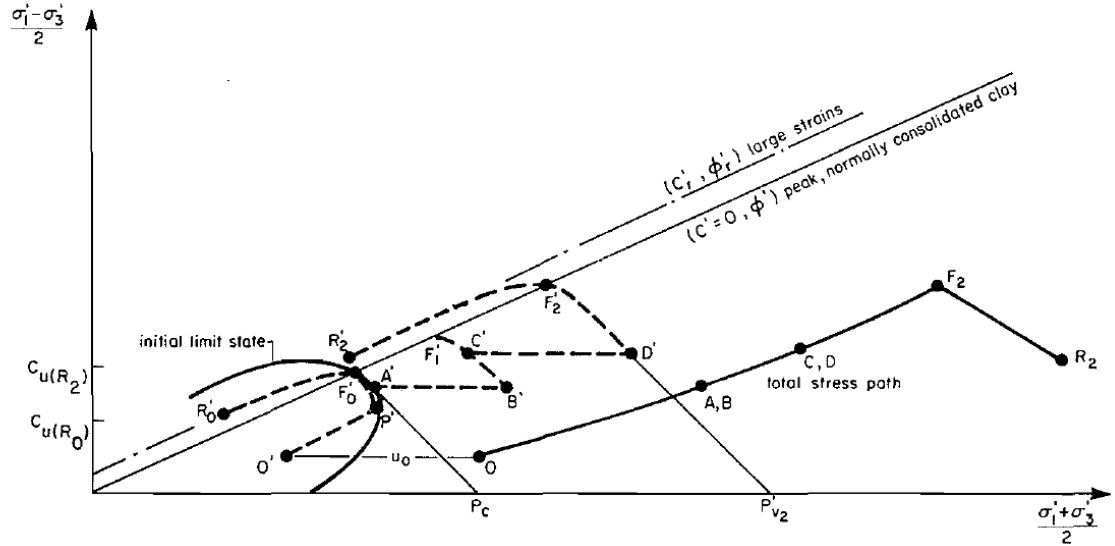


Figure 3.7. Total and effective stress paths under an embankment build up in three stages (Tavenas and Leroueil 1978)

The excess pore-water pressure starts dissipating with time under the same mean total stress (i.e. A and B at the same point). This causes point A' to move to point B' without a noticeable change in deviatoric stress. In a case where the consolidation phase brings a slight increase in deviatoric stress (i.e. shearing), the ESP, $A'B'$ may not be horizontal (Tavenas and Leroueil 1980). The ESP then follows $B'C'$ and $C'D'$ due to the second phase of construction and consolidation, respectively. If the embankment is built to a height that can initiate failure in the clayey deposit, ESP moves from point D' to point F'_2 with pore-water pressure parameter, $\bar{B}_l = 1$. The pore-water pressure parameter then exceeds 1 as ESP moves toward the point R'_2 along the new limit state surface, $D'F'_2R'_2$.

4. MODIFIED CAM-CLAY CONSTITUTIVE MODEL

The post-failure behavior of a clayey deposit is an important factor in the design of a tailings dam. Hence, part of tailings dam design includes choosing an appropriate constitutive model that can reliably predict the post-failure behavior. The post-failure response is associated with one of the following variables: i) shear dilatancy, ii) shear hardening/softening, iii) volumetric hardening/softening or iv) tensile softening (Itasca 2005)

When a normally consolidated or lightly overconsolidated clay is subjected to isotropic loading, volumetric hardening behavior takes place. This results in a permanent volume decrease and expansion of the yielding curve. Under the same condition, volumetric softening behavior occurs in a heavily overconsolidated clay, which leads to contraction of the yielding curve. Among various elasto-plastic constitutive models (i.e. Modified Cam-Clay model, Double-Yield model, Norsand model, S-Clay1, etc.), Modified Cam-Clay constitutive model (MCC model) is considered to be the simplest model that is capable of capturing volumetric both hardening and softening in a clay (Itasca 2005).

4.1 Basic of MCC model

Roscoe et al. (1963) developed an elasto-plastic strain hardening constitutive model for clays based on the Critical State Soil Mechanics, which is called Cam-Clay model. The yield surface of the original Cam-Clay model has a logarithmic spiral shape with a pointy tip (Figure 4.1(a)). Hence, if an associated flow (i.e. Plastic strain increment is normal to yield surface.) is adopted in modeling, an unrealistic plastic shear strain can be estimated. To overcome this drawback of the original Cam-Clay model, Roscoe and Burland (1968)

developed the MCC model by introducing the elliptical yield surface (Figure 4.1(b)). Hence, at the largest mean effective stress, the incremental deviatoric plastic strain becomes zero since the plastic strain increment vector is horizontal (i.e. purely hydrostatic stress condition). The MCC model is the most widely used constitutive model in the field of geotechnical engineering (Munda et al. 2014). Two aspects of soil behavior can be modeled using the MCC model: i) compression/dilation and ii) shearing and critical state (i.e. unlimited shearing at constant volume).

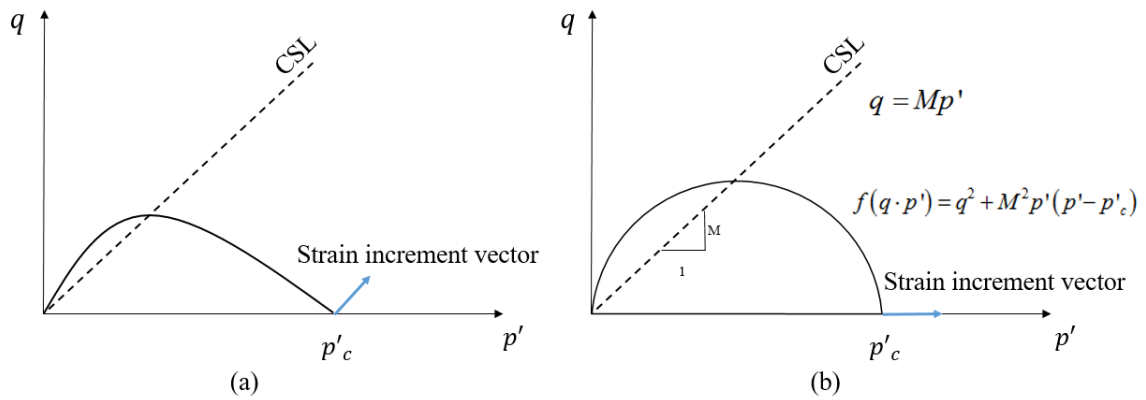


Figure 4.1. Yielding curves in p' - q plane: a) Cam-Clay model; b) Modified Cam-Clay model

The state of soil in the MCC model is described by four parameters: i) mean effective stress (Eq.(4.1)), ii) deviatoric stress (Eq.(4.2)), iii) specific volume (Eq.(4.3)) and iv) preconsolidation pressure. The preconsolidation pressure is defined as the maximum vertical pressure that has been applied to the soil throughout the whole time. The magnitude of preconsolidation pressure can be determined by conducting the oedometer test. The size of yielding curves in the MCC model is expressed by the mean preconsolidation pressures that can be obtained from triaxial tests using different states of stress.

$$p' = \frac{\sigma'_1 + \sigma'_2 + \sigma'_3}{3} \quad (4.1)$$

$$q = \sigma'_1 - \sigma'_3 \quad (4.2)$$

$$v = 1 + e \quad (4.3)$$

where v is specific volume, e is a void ratio, and σ'_1 , σ'_2 and σ'_3 are maximum, intermediate and minimum principal stresses, respectively.

The main advantages of the MCC model can be summarized as follows (Wood 1990):

- 1) The model is expressed in terms of mean effective stress with the capability of coupling the variation of pore-water pressure (i.e. dissipation or generation),
- 2) The yielding and plastic deformation are simulated,
- 3) Both shearing and compression are coupled in a single model,
- 4) The input parameters can be obtained from standard geotechnical tests, and
- 5) Stress and strain paths can be captured

4.2 Virgin Consolidation Line and Unloading-Reloading Line

In the MCC model, it is assumed that the behavior of soft soil under isotropic drained loading condition is represented as a straight line in $v - p'$ space. This straight line is denoted as Virgin Consolidation Line (VCL) or Normal Consolidation Line (NCL). Upon unloading, the soil swells along another straight line that is denoted as Swelling Line or Unloading-Reloading Lines (URL) (Figure 4.2). States of soil that lie on NCL (or VCL) and URL indicate normally consolidated and overconsolidated conditions, respectively.

The mean effective stresses, p'_b and p'_d are preconsolidation pressures, corresponding to different URLs. N and v_s are specific volumes of VCL and URL, respectively at unit pressure (i.e. $p' = 1$ kPa).

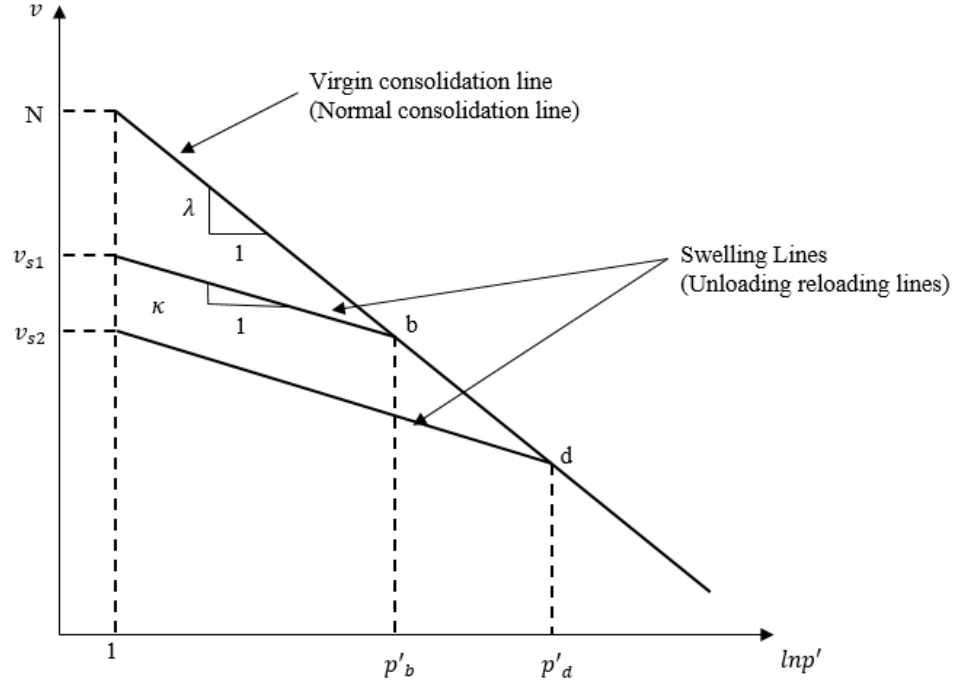


Figure 4.2. Typical virgin consolidation (normal consolidation) and unloading-reloading (swelling) lines from oedometer test on a clay soil (Inc 2003)

The VCL and URL in Figure 4.2 are expressed as Eq. (4.4) and Eq. (4.5), respectively.

$$v = N - \lambda \ln(p') \quad (4.4)$$

$$v = v_s - \kappa \ln(p') \quad (4.5)$$

where λ is slope of VCL, and κ is slope of URL.

4.3 Critical State Line

A soil element under a loading, experiences compression and shearing simultaneously, which subsequently results in volumetric and shear strains, respectively. These strains are divided into two components: elastic and plastic strains. In case where the plastic shear strain reaches its maximum value, the volumetric strain plastic strain becomes zero. This condition is called the critical state that is defined by a straight line (i.e. Critical State Line, CSL) with a slope M in $q - p'$ plane. The location of CSL relative to VCL in the $v - \ln(p')$ plane is shown in Figure 4.3, where Γ is the specific volume of the CSL at unit pressure ($p' = 1 \text{ kPa}$).

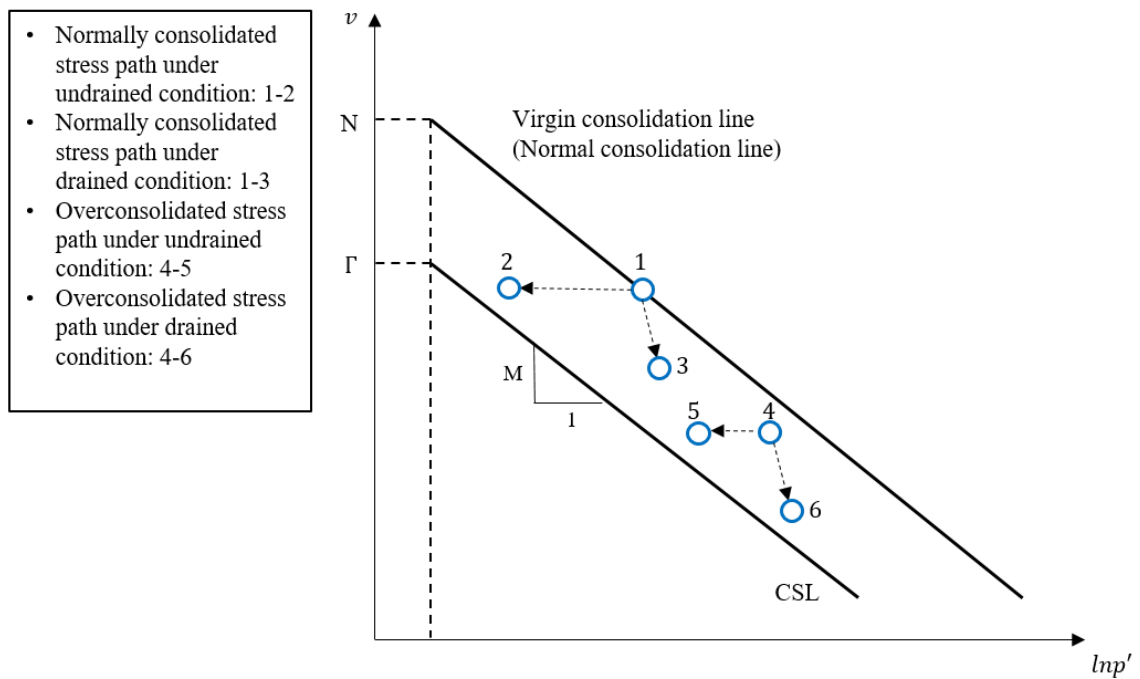


Figure 4.3. Typical critical state and virgin consolidation lines of clays in $v - \ln(p')$ plane

Since the CSL is parallel to the VCL, CSL can be expressed as Eq. (4.6).

$$v = \Gamma - \lambda \ln(p') \quad (4.6)$$

The relationship between κ and N in the MCC model is shown as Eq. (4.7).

$$\Gamma = N - (\lambda - \kappa) \ln(2) \quad (4.7)$$

Point 1 (on VCL) and point 4 (between VCL and CSL) in Figure 4.3 represent a normally and an overconsolidated soil, respectively. Under undrained loading condition, ESP moves toward CSL horizontally (i.e. $1 \rightarrow 2$ and $4 \rightarrow 5$) with no change in volume. Under the drained condition, ESP follows $1 \rightarrow 3$ and $4 \rightarrow 6$ for normally and overconsolidated soils, respectively with a decrease in volume. The determination of ESP under drained loading condition requires the measurement of deformation during shearing (Wood 1990). Soils fail on CSL under both drained and undrained loading conditions with critical specific volume.

4.4 MCC Yield Functions

The yielding curve in MCC model is in a shape of an ellipse (Figure 4.4; Eq. (4.8)).

$$f(q, p') = q^2 + M^2 p' (p' - p'_c) \quad (4.8)$$

where p'_c is mean preconsolidation pressure.

As shown in Figure 4.5, the size of yielding curve is determined by the initial preconsolidation pressure. The location of CSL in $q - p'$ plane is also an important factor in determining the state of soil. CSL crosses the yielding curve at the maximum deviatoric

stress (i.e. $Mp'_c/2$). If ESP of the soil starts from a point within the yielding curve and ends on the yielding curve above the CSL, the soil experiences negative plastic volumetric strain (ε_p^p) (i.e. dilation). On the other hand, if ESP ends below the CSL, the plastic volumetric strain becomes positive (i.e. compression or contraction). Further loading leads to the increase in preconsolidation pressure (i.e. $p'_{c0} \rightarrow p'_{c1} \rightarrow p'_{c2}$), which subsequently causes the expansion of the yielding curve.

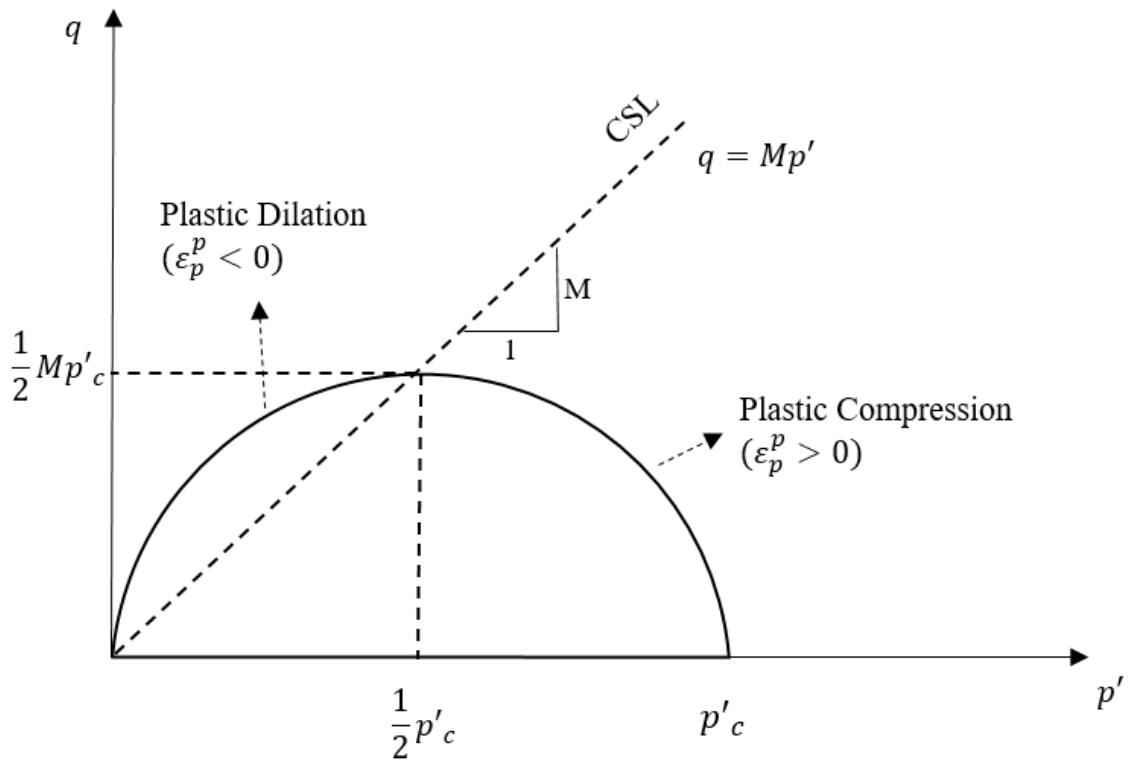


Figure 4.4. A yielding curve in the MCC model (Itasca 2005)

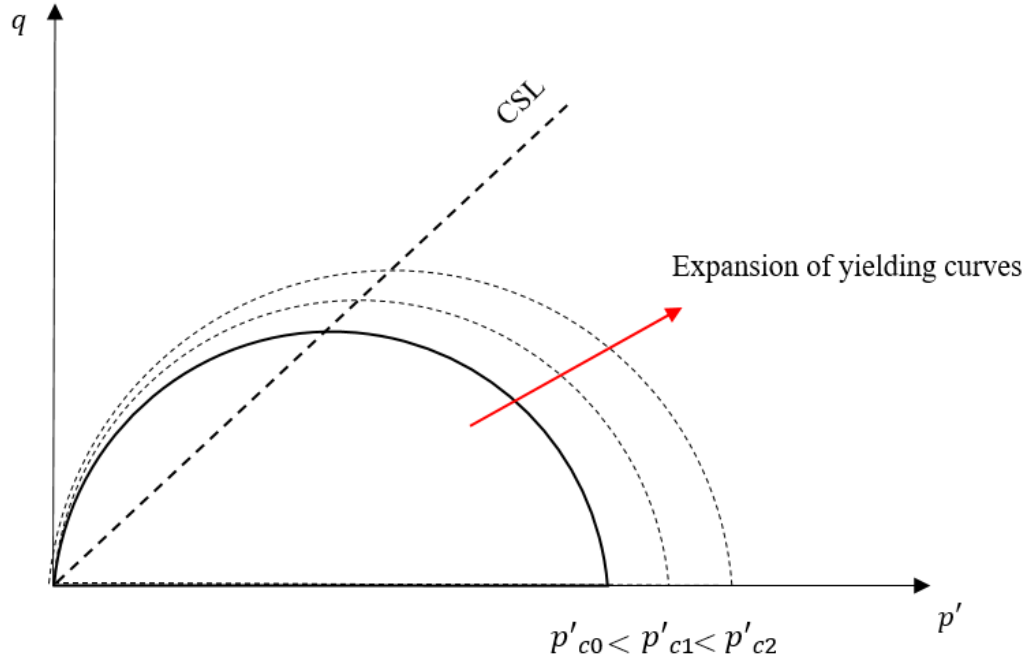


Figure 4.5. Expansion of yielding curves

4.5 Undrained Shear Strength in the MCC Model

Figure 4.6(a) and (b) show TSP and ESP in $q - p'$ plane and ESP in $v - p'$ plane from a conventional undrained triaxial compression test for an overconsolidated soil. The soil was first consolidated to the mean effective stress of p'_0 (point O in Figure 4.6(b)), followed by unloading to p'_i (point I in Figure 4.6(b)). The soil was then sheared under the undrained condition at I and fails on CSL at F (i.e. $p' = p'_f$). In an undrained test, the total volume of soil remains unchanged throughout the shearing stage (i.e. $v_0 = v_f$); therefore, the specific volume at failure is expressed as Eq. (4.9).

$$v_f = v_0 = \Gamma - \lambda \ln p'_f \quad (4.9)$$

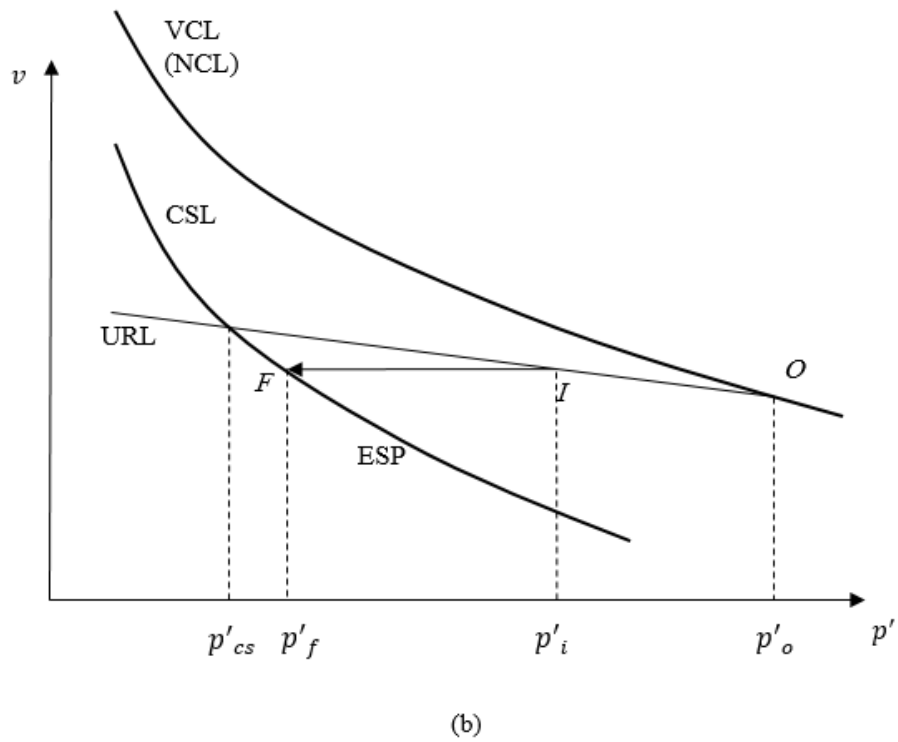
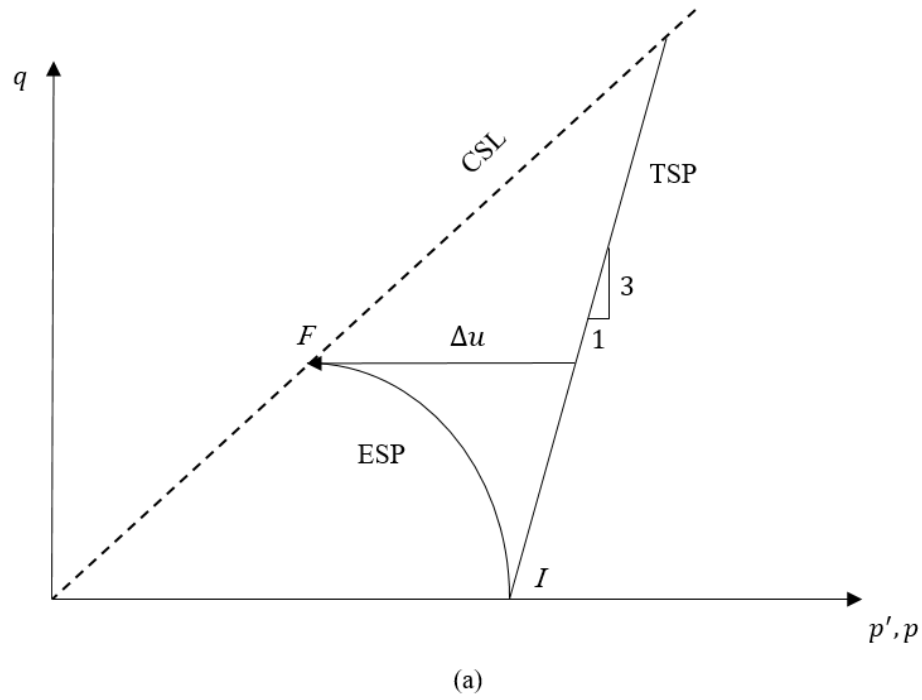


Figure 4.6. Typical behavior of an overconsolidated soil in conventional undrained triaxial compression test: a) effective and total stress paths in $q - p'(p)$ plane; b) effective stress path in $v - p'$ plane (Wood 1990)

Rearranging Eq. (4.9) in terms of p'_f yields,

$$p'_f = \exp\left(\frac{\Gamma - v_0}{\lambda}\right) \quad (4.10)$$

Since $q_f = Mp'_f$, Eq. (4.10) can be rewritten as Eq. (4.11).

$$q_f = M \exp\left(\frac{\Gamma - v_0}{\lambda}\right) \quad (4.11)$$

Therefore, the undrained shear strength of the soil is

$$c_u = \frac{q_f}{2} = \frac{M}{2} \exp\left(\frac{\Gamma - v_0}{\lambda}\right) \quad (4.12)$$

The undrained shear strength, c_u at F can also be computed using Eq. (4.13).

$$c_u = \frac{M}{2} \exp\left[\frac{(\Gamma - N)}{\lambda} + \ln p'_0 - \left(\frac{\kappa}{\lambda}\right) \ln n_p\right] \quad (4.13)$$

To link overconsolidation histories to undrained shear strength calculations, two parameters, r and n_p can be adopted as shown in Eq. (4.14) and Eq. (4.15), respectively.

$$r = \frac{p'_0}{p'_{cs}} = \exp\left(\frac{N - \Gamma}{\lambda - \kappa}\right) \quad (4.14)$$

$$n_p = \frac{p'_0}{p'_i} \quad (4.15)$$

Substitution of Eq. (4.14) and Eq. (4.15) into Eq. (4.13) leads to Eq. (4.16).

$$\frac{c_u}{p'_i} = \frac{M}{2} \left(\frac{n_p}{r} \right)^\Lambda \quad (4.16)$$

Extending Eq. (4.16), the c_u/p'_i ratio for normally and overconsolidated soils can be computed using Eq. (4.17) and Eq. (4.18), respectively.

$$\left(\frac{c_u}{p'_i} \right)_{nc} = \frac{M}{2} r^{-\Lambda} \quad (4.17)$$

$$\frac{c_u/p'_i}{(c_u/p'_i)_{nc}} = n_p^\Lambda \quad (4.18)$$

where $\Lambda = (\lambda - \kappa)/\lambda$.

4.6 Computation FOS against Undrained Failure

As explained before, tailings dams (i.e. embankments) on soft clayey deposits are commonly built using staged construction technique. To avoid undrained failure within the foundation soil, the applied loads should be adjusted through preliminary design. These loads cause changes to the in-situ stress states in the soil and eventually may cause local or even global failure, depending on the magnitude of the load. The stability of the soft clayey foundation beneath a tailings dam against undrained failure can be estimated by computing factor of safety at zones within the foundation by developing ESPs (Cinicioglu and Togrol 1991).

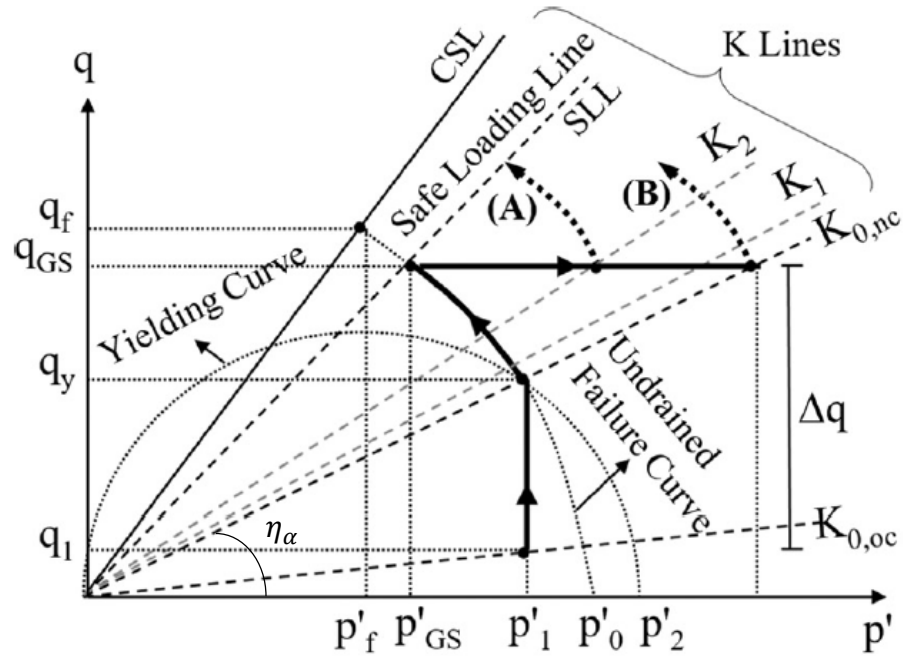
The stress paths are usually projected onto two-dimensional $q-p'$ and $v-p'$ planes rather than three-dimensional space, as shown in Figure 4.7(a) and (b), respectively. For vertical

effective stress, σ'_v and horizontal effective stress, σ'_h , p' and q are given as Eq. (4.19) and Eq. (4.20), respectively.

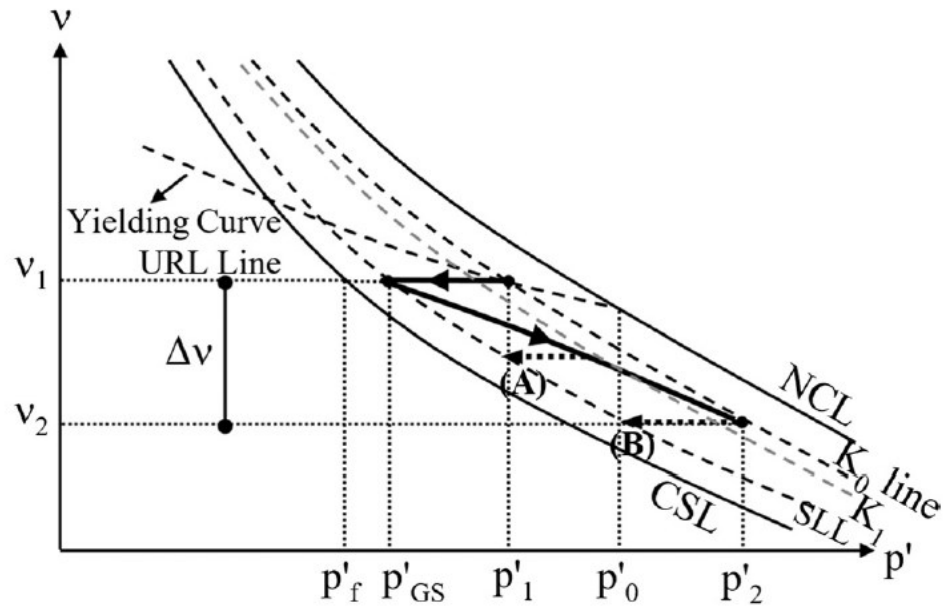
$$p' = \left(\frac{\sigma'_v + 2\sigma'_h}{3} \right) = \sigma'_v \left(\frac{1 + 2K}{3} \right) \quad (4.19)$$

$$q = \sigma'_v - \sigma'_h = \left[\frac{3(1 - K)}{1 + 2K} \right] \times p' \quad (4.20)$$

The initial specific volume can be estimated from the oedometer test. If a soil is initially in a normally consolidated condition, the state of soil lies on $K_{\theta,nc}$ - line in $q - p'$ plane, while the state of an overconsolidated soil lies on $K_{\theta,oc}$ - line (Subscripts nc and oc indicate normally and overconsolidated conditions, respectively).



(a)



(b)

Figure 4.7. Effective stress paths in a soft layer under staged embankment construction: a) $q - p'$ plane; b) $v - p'$ plane (Öser and Cinicioglu 2017)

In Figure 4.7, the soil element is initially overconsolidated within the yielding curve on the $K_{0,oc} -$ line. The coordinate of this state is given as (p'_l, q_l) in Figure 4.7(a) and (p'_l, v_l) in Figure 4.7(b). The size of the initial yielding curve is denoted by initial preconsolidation pressure p'_0 . As it was assumed that loading is applied instantaneously, the undrained loading condition ($\bar{B}_l = 1$) is prevailed in the soft foundation. However, in reality, loading is applied gradually and drainage takes place during the early stage of construction. Dissipation of pore-water pressure ($\bar{B}_l < 1$) increases the mean effective stress, which causes the deviation of corresponding ESP from vertical direction (See Figure 3.4 and Figure 3.7). However, the determination of $\bar{B}_l$ during construction is challenging in most cases since it requires continuous pore-water pressure measurements. In this regard, Öser and Cinicioglu (2017) suggested assuming an undrained loading condition ($\bar{B}_l = 1$) in the initial stage of loading. Hence, ESP moves vertically until it reaches yielding curve at point (p'_l, q_y) with p' constant (i.e. no volume change). The state of soft foundation changes to normally condition on the yielding curve. After passing the yielding curve, the ESP moves toward the CSL to reach the point, (p'_{GS}, q_{GS}) in $q - p'$ plane. If the initial loading was large enough, the foundation fails on CSL (i.e. p'_f, q_f). The relationship between q_f and c_u is shown in Eq. (4.12). The excess pore-water pressure starts dissipating from the point, (p'_{GS}, q_{GS}) , which leads to the increase in p' under constant deviatoric stress, q (i.e. horizontal movement in $q - p'$ plane). This process increases the strength of the foundation with the expansion of yielding curve. As consolidation proceeds, the soft foundation experiences different K_0 conditions since the ratio of horizontal strain to vertical strain continuously changes until it reaches $K_{0,nc} -$ line at the end of consolidation (Zhang et al. 1998; Öser and Cinicioglu 2017). Due to this reason, CSL and NCL are considered as the upper and the

lower bounds, respectively. The slope of $K_{0,nc}$ – line, η_α at the end of each consolidation is calculated using Eq. (4.21).

$$\eta_\alpha(K_{0,nc}) = \frac{3(1 - K_{0,nc})}{1 + 2K_{0,nc}} \quad (4.21)$$

As ESP moves horizontally during consolidation, the next round of loading can be applied at a targeted timeline (i.e. ESP (A)) or at the end of consolidation phase (i.e. ESP (B)) under undrained condition. The safe loading line (SLL) in Figure 4.7(a) provides a threshold before reaching CSL to ensure a safe design. FOS against undrained failure can be computed as the ratio of the slope of CSL (i.e. M) to the slope of K – line that corresponds to the final stress state at a particular time Eq. (4.22)

$$FOS = \frac{M}{\eta_\alpha} \quad (4.22)$$

Öser and Cinicioglu (2017) suggested that the stability of a soft clayey foundation beneath a tailings dam can be better estimated by dividing the foundation into several arbitrary zones. FOS is then calculated based on ESP for each zone. Figure 4.8 shows an example of such discretization of a foundation into 30 different zones and the computed FOS using Eq. (4.22).

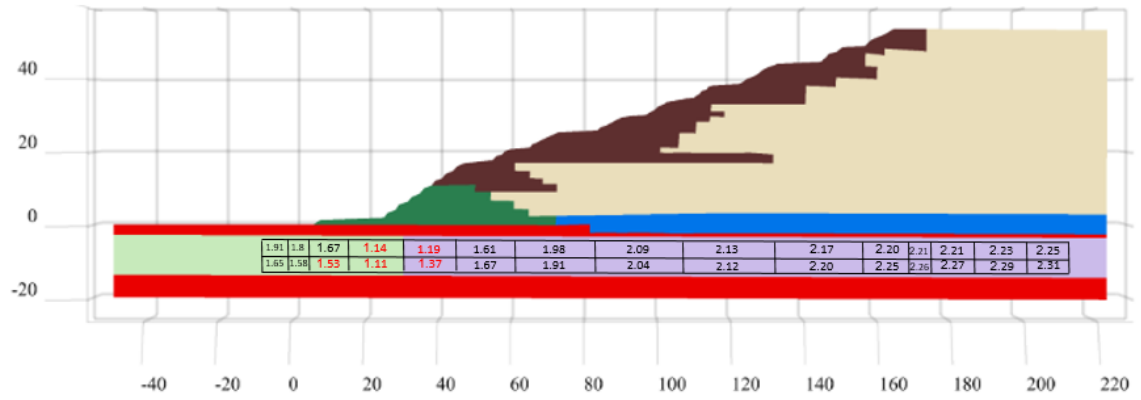


Figure 4.8. Example of discretization of a soft clayey foundation beneath a tailings dam into different arbitrary zones

5. METHODOLOGY

5.1 Description of Mining Site

This section provides a summary of the mining project (Golder Associates 1987; Wood Ltd. 2018) used in this research.

5.1.1 Lagoon D

Elkview Operations (EVO) is located approximately three kilometers east of Sparwood in southeastern British Columbia. The mine has coal reserves amounting to approximately 220.6 million tonnes, one of the largest coal reserves in Canada. EVO has been in nearly continuous operation since 1969. In mining, clean coal is obtained by washing the mined material to isolate the saleable coal from other materials. The waste from the washing process is subdivided into coarse materials and fine materials. In EVO, the fine refuse was traditionally placed into tailings storage areas designated as Lagoons A, B, C, and D as shown in Figure 5.1. The processing plant is shown on the right side of the figure and waste materials from the processing plant were drained by gravity to the lagoons. The lagoons are located along the east side of the Elk River, north of the confluence with Michel Creek. The elevation of Elk River in the vicinity of the lagoons is approximately 1,110 m above sea level.

All the lagoons are “paddock-type” facilities, i.e. containment is provided by the construction of embankments on all sides. Lagoons A and D tie into the original ground on their eastern sides, where the natural hillside provides containment. Lagoons A and B have a small storage capacity relative to Lagoons C and D, and there is limited technical

information available about their development. Lagoons A and B were apparently used for a relatively short period of time. Thereafter, Lagoons C and D became the primary tailings storage areas, with Lagoon C becoming inactive in 1987. Since that time, Lagoon D has become the focused development area while Lagoon C has been used primarily for the temporary storage of excess wash plant water decanted from Lagoon D. The focus of this research is directed towards the stability of the tailings dam in Lagoon D.

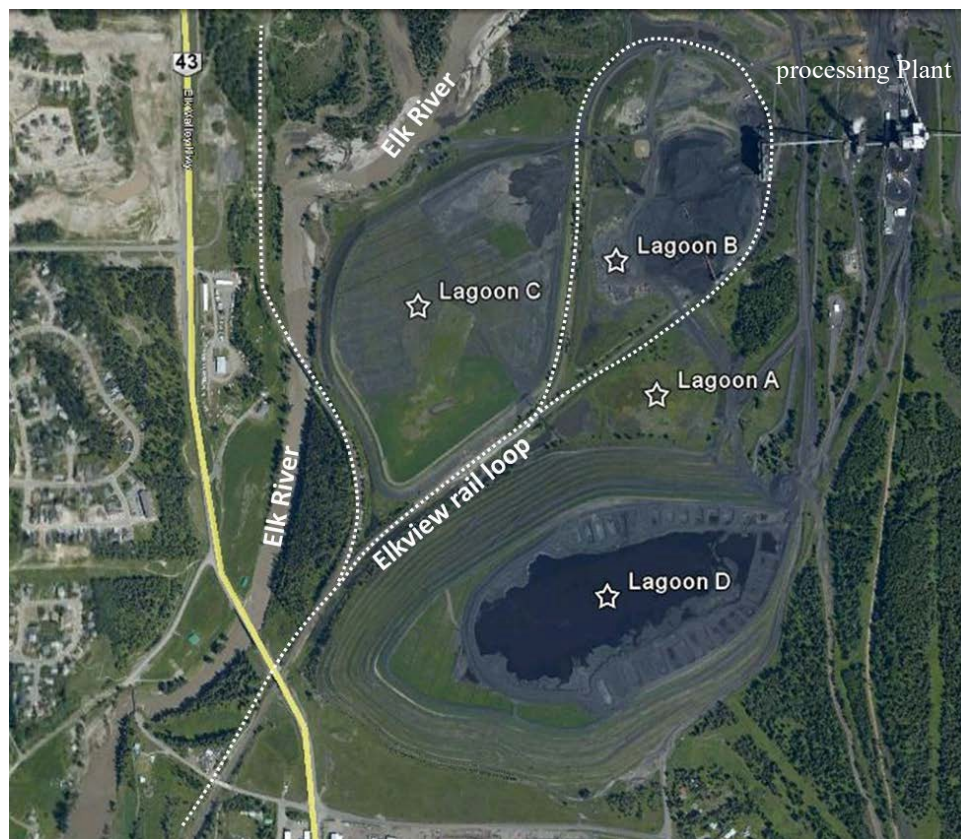


Figure 5.1. Layout of EVO Lagoons

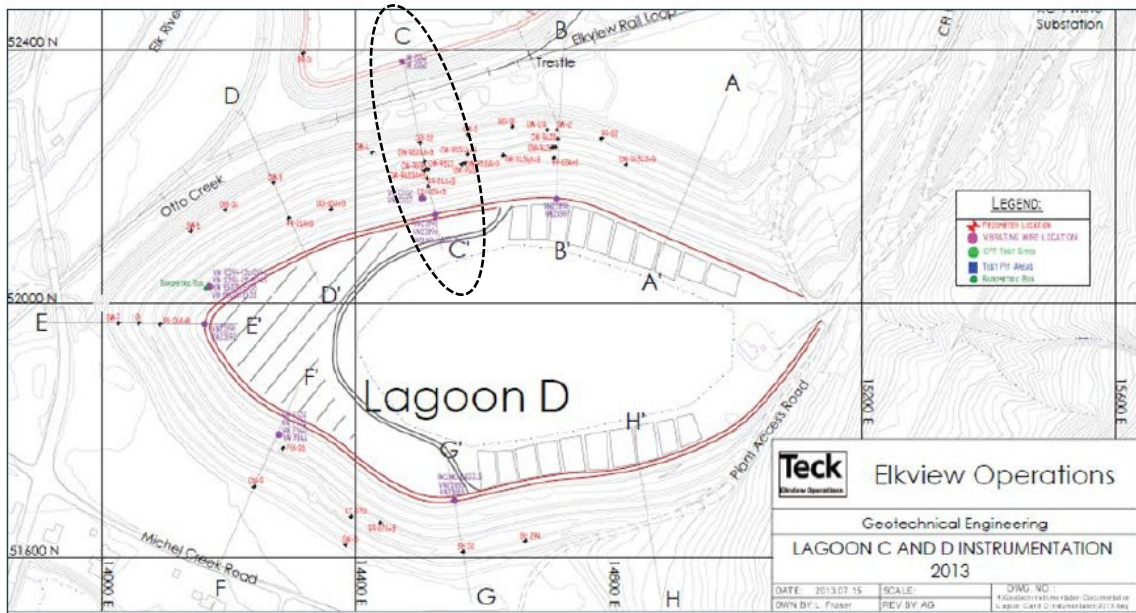
Lagoon D was developed in 1972 and reached its current elevation (i.e. 1166 m) in 2018. The raise construction and operational history are summarized in Table 5-1. The average overall downstream slopes of the perimeter dams containing Lagoon D range from 2.7H:1V to 3.4H:1V. The site plan and eight cross-sections (A to H) along the

embankments of Lagoon D are shown in Figure 5.2(a). The internal geometry of the cross-section C-C', the original ground elevation, and piezometer readings are shown in Figure 5.2(b). The piezometer readings indicated that the phreatic surface is close to the original ground surface.

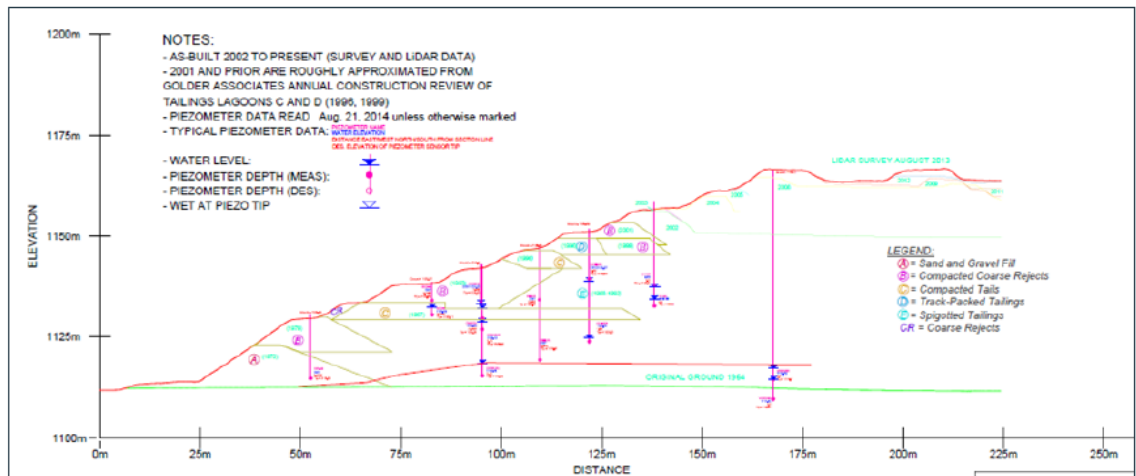
Table 5-1. Raise construction history of EVO

Year	Approx. crest elevation (m)	Construction material
1972	1123	Sand and Gravels
1979	1130	Coarse rejects
1987	1133	Compacted tailings
1993	1142-1146	Coarse rejects and coarse tailings
1996	1151-1155	Coarse rejects
1999	1155	Coarse rejects
2001	1155	Coarse rejects
2002	1158	Coarse rejects
2004	1160	Coarse rejects
2005	1162	Coarse rejects
2008	1164	Coarse rejects
2018*	1166	Coarse rejects

* From 2008 to 2018 the level of tailings dam increased gradually; therefore, it has been assumed the last stage of deposition of tailings occurred in 2018.



(a)



(b)

Figure 5.2. Tailings dam in Lagoon D: a) site plan; b) C-C' cross-section

5.1.2 Geotechnical Site Investigation

In 1984, triaxial tests were carried out on the samples obtained from boreholes BH84-4 and BH84-3 (Figure 5.3). At the location of BH84-4, 18 m of coal tailings were observed to overlie 0.6 m of sand and gravel and 1.5 m of medium to coarse-grained sand.

Subsequently, 1.7 m of stiff to very stiff clayey silt was encountered prior to termination of boring at the depth of 22 m (Elevation = 1,106 m).

In 2017, a series of Standard Penetration Test (SPT) (BH17-11), Nilcon Shear Vane Test (BH17-11 and BH17-18), Cone Penetration Test (CPT) (BH84-4) were carried out in Lagoon D. In late 2019, triaxial and direct simple shear tests were performed on the samples collected from SY19-02 at the depths of 9 m and 14 m.



Figure 5.3. Locations of boreholes in Lagoon D

The clayey silt layer was quite variable in grain size and thickness, ranging from clayey silt with no gravel and trace of sand up to gravelly sand with some silt and clay. The clayey silt was generally found to be very stiff to hard. A total of nine SPTs were conducted in the clayey silt layer and N values were between 10 to 67 (average = 28). Moisture content

varied in the range of 8.1% to 52.7%. The plasticity index of the tested samples was in the range of 5.8% to 11.5% with two non-plastic samples. Two Nilcon vane shear tests were also conducted in the clayey silty layer at the depths of 8 and 10 m below the ground surface. The first vane shear test (i.e. 8 m) showed a peak shear strength of 119 kPa, post-peak shear strength of 72.6 kPa and a remolded shear strength of 64.6 kPa. The results of the second vane shear test (i.e. 10 m) showed a peak strength of at least 123 kPa.

5.1.3 Previous Stability Analysis

Several stability analyses have been conducted to check the stability of the EVO tailings dam. According to EVO geotechnical report, one of the latest analyses was performed in 2017 using the software, SLOPE/W (GeoStudio 2007) (Figure 5.4). Stability analyses were carried out using the Morgenstern-Price method (Limit Equilibrium Method) for finding potential failure surfaces through embankment slopes and underlying foundations. Since FOS for the cross-section, C-C' was minimum (i.e. 1.54), this section was chosen as the main cross-section for the rest of the analysis.

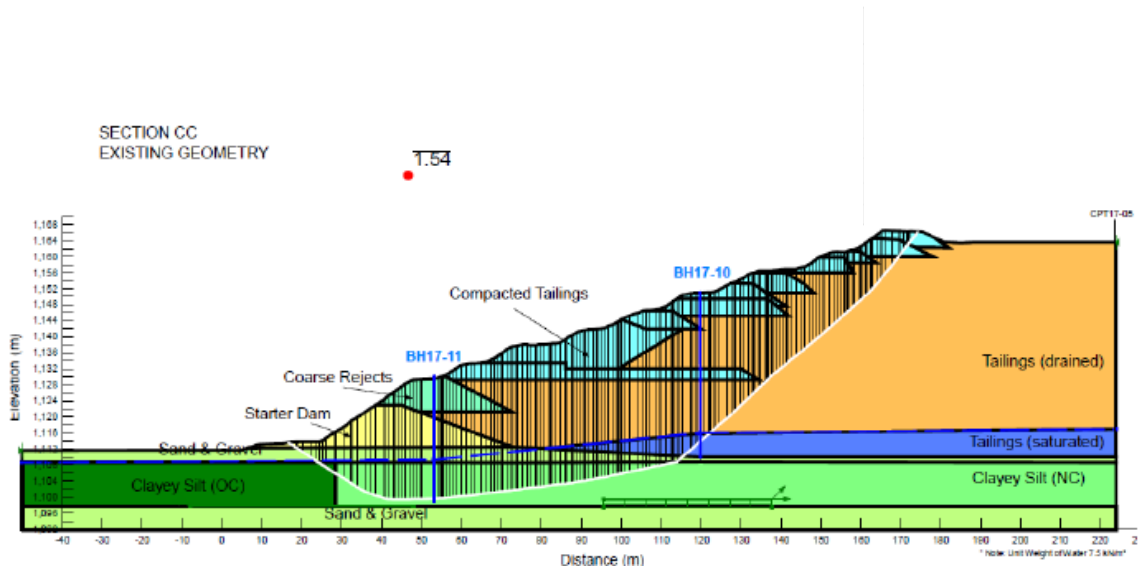


Figure 5.4. Stability analysis using SLOPE/W (GeoStudio 2007) for the section, C-C'

The clayey silt layer was divided into two zones; namely, normally consolidated and overconsolidated zones based on the preconsolidation pressure (200 to 240 kPa). In the stability analyses, the undrained shear strength ratio, $s_u/\sigma'_v = 0.3$ (using undrained shear strength from vane shear test, 123 kPa) was used to simulate the behavior of normally consolidated clayey silt zone by SHANSEP constitutive model. For the overconsolidated zone, the Mohr-Coulomb constitutive model was used ($c' = 30$ kPa and $\phi' = 0^\circ$). A series of sensitivity analysis was also performed for different undrained shear strength ratio. When $s_u/\sigma'_v = 0.22$ was assigned to the normally consolidated zone, FOS was computed as 1.12, which is far less than the acceptable FOS (i.e. 1.5). This indicates that the undrained shear strength ratio governs the stability analysis of the tailings dam. The undrained shear strength ratios obtained from different in-situ and laboratory tests are summarized in Table 5-2.

Table 5-2. Undrained shear strength ratios obtained from different in-situ and laboratory tests in Lagoon D

Test type	s_u/σ'_v
Nilcon vane	0.26 to 0.43
SPT	> 0.22 (can be greater than 1.00)
CPTu	> 0.27 (can be greater than 1.00)
Triaxial test (1984)	0.52

5.2 Laboratory Test Results

Two sets of triaxial and oedometer tests were performed on the samples obtained from the clayey silt layer (i.e. BH84-4 (El. = 1,108) and SY19-02 (El. = 1,103)). Locations and depths where these samples were collected are indicated in Figure 5.5.

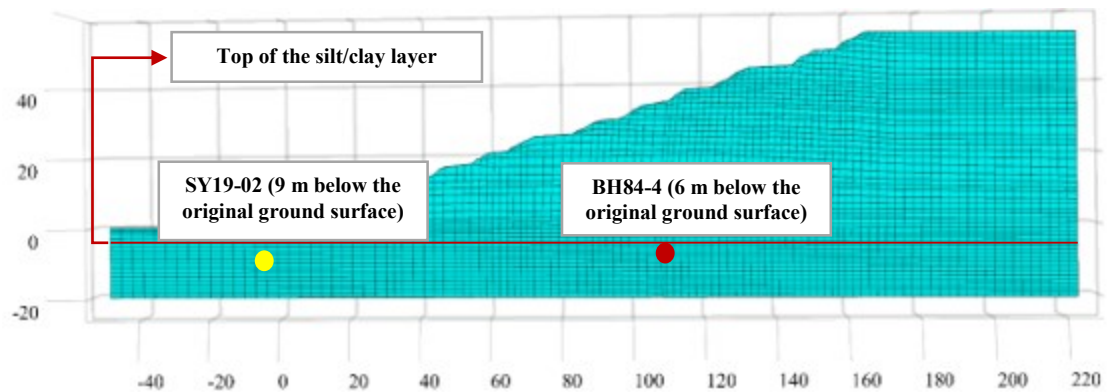


Figure 5.5. Locations and depths where the samples for consolidation and triaxial tests were collected along the cross-section, C-C'

5.2.1 Samples from BH84-4

5.2.1.1 Triaxial Test

Figure 5.6. shows the variation of deviatoric stress and pore-water pressure with respect to axial strain, ε_a . The pore-water pressure reached its peak at $\varepsilon_a = 2\%$. The increase in pore-water pressure during shearing indicates the contractive behavior of soil and vice versa. Unlike the pore-water pressure, a well-defined peak point was not observed from $q - \varepsilon_a$ relationship. Based on the results in Figure 5.6, ESP and TSP are plotted in Figure 5.7.

The slope of TSP is $\Delta q / \Delta p = 3$, while the slope of ESP is first positive (i.e. decrease in pore-water pressure), and then changes to negative (i.e. increase in pore-water pressure) and finally becomes positive at high deviatoric stresses. Table 5-3 summarizes the triaxial test results for the sample obtained from BH84-04.

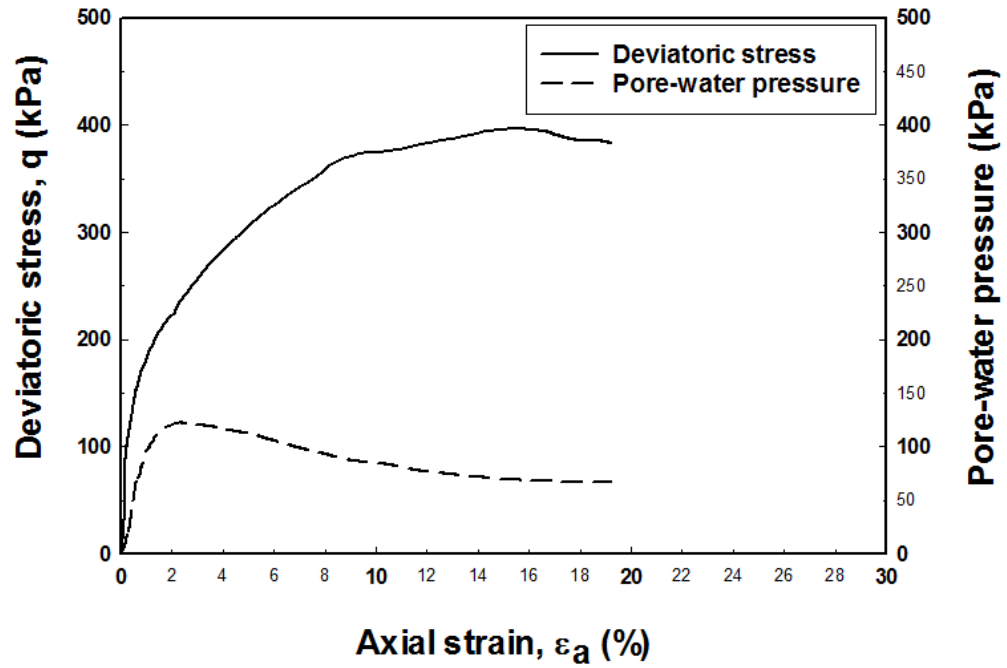


Figure 5.6. Variation of deviatoric stress and pore-water pressure with respect to axial strain (BH84-04)

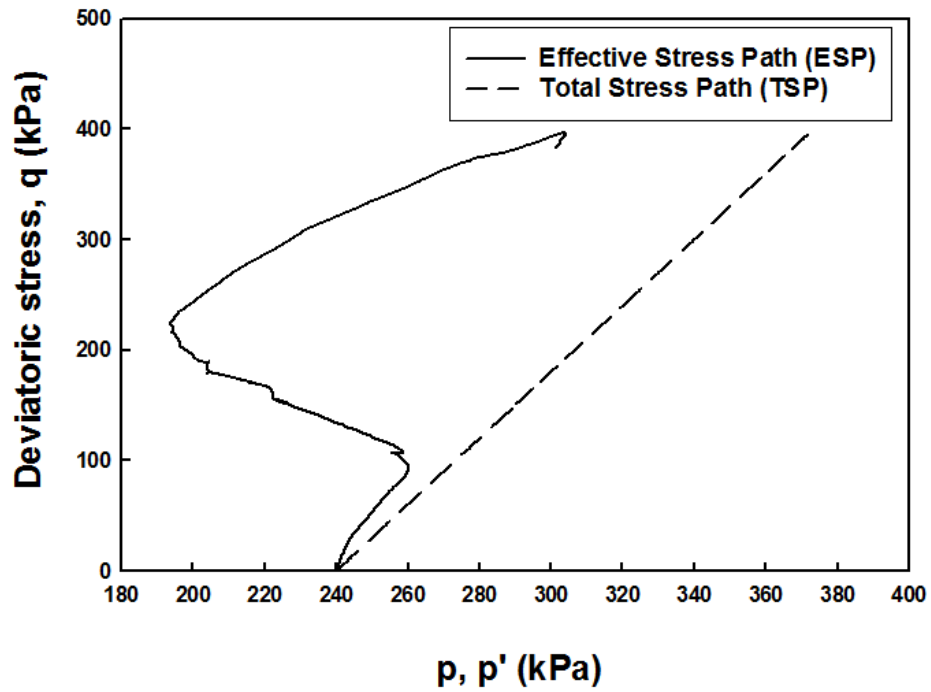


Figure 5.7. Effective and total stress paths from triaxial test (BH84-04)

Table 5-3. Summary of triaxial test (BH84-04)

Depth (m)	w (%)	γ (kN/m ³)	End of consolidation		σ'_3 (kPa)	$\sigma'_1 - \sigma'_3$ (kPa)	ε_a at failure (%)	u at failure (kPa)
			w (%)	e				
6	20.6	20.5	17.8	0.480	240	395	15.3	69

5.2.1.2 Consolidation Test

Figure 5.8 shows the $e - \log \sigma'_v$ plot from the oedometer test. The preconsolidation pressure, σ'_p was estimated to be 240 kPa with the Casagrande method (1936). To determine the in-situ virgin consolidation curve, the methodology proposed by (Schmertmann 1953) was used as shown in Figure 5.9.

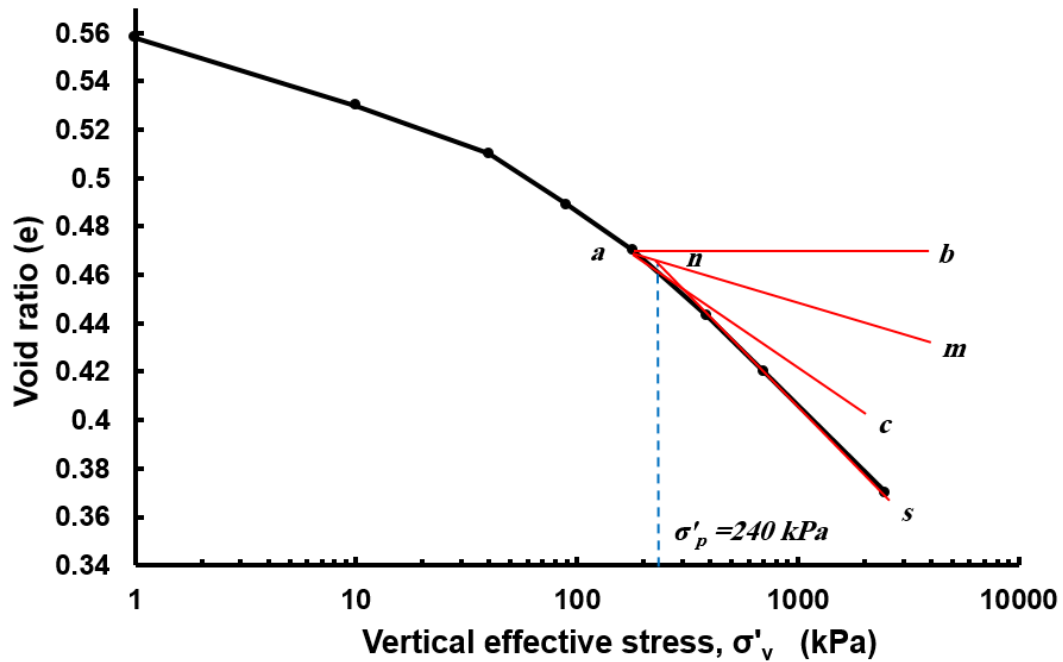


Figure 5.8. Oedometer test result and determination of preconsolidation pressure using Casagrande (1936) method (BH84-4)

The compression index, C_c of in-situ virgin consolidation line was determined to be 0.12. The slope of URL, C_r can be determined in the oedometer test. However, as the unloading stage did not perform in the test, it was assumed that the slope of the swelling part, C_s is equal to C_r . Wood (1990) suggested that the swelling index, C_s is typically αC_c , where α is in the range of 1/5 to 1/3. In this research, it was assumed that $C_s = (1/5) C_c$. The summary of the main parameters obtained from the oedometer test is given in Table 5-4.

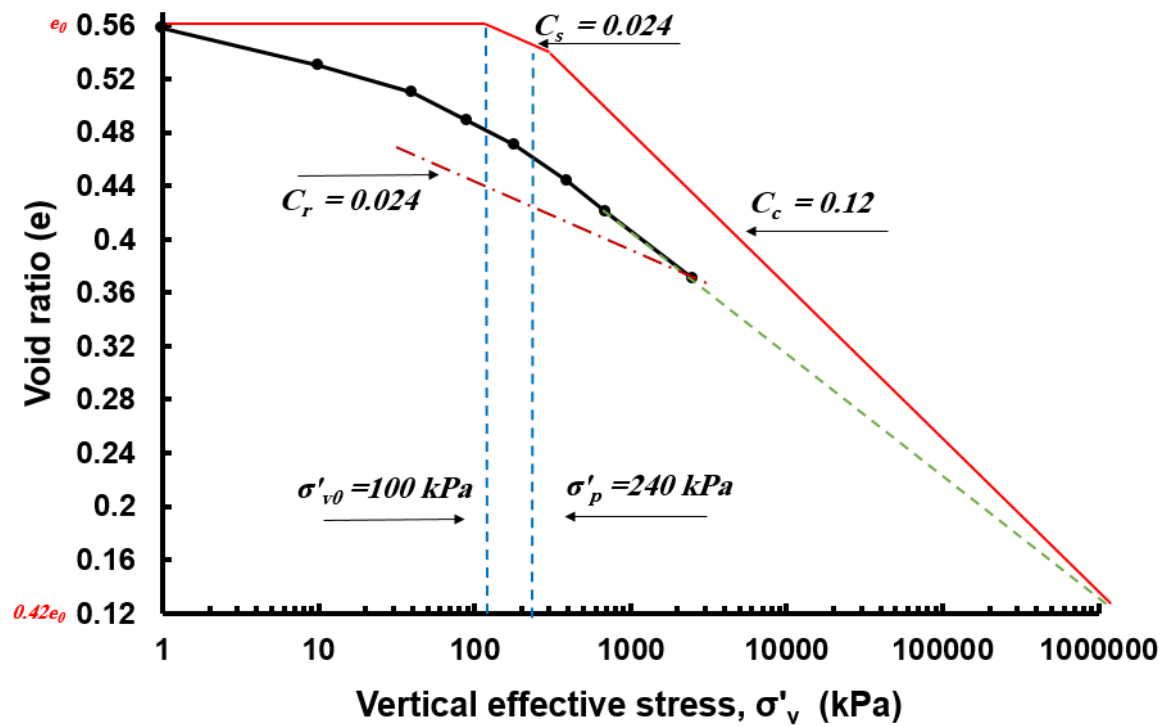


Figure 5.9. Estimation of in-situ virgin consolidation line using Schmertmann (1955) method (BH84-4)

Table 5-4. Summary of oedometer test results (BH84-04)

e_0	C_c	C_s	σ'_p (kPa)	σ'_{v0} (kPa)	OCR
0.558	0.12	0.024	240	100	2.4

5.2.2 Samples from SY19-02

5.2.2.1 Triaxial Test

Figure 5.10 shows the variation of deviatoric stress and pore-water pressure with respect to axial strain. The pore-water pressure remained unchanged beyond the axial strain of 5%. On the contrary, a peak point was observed at a small axial strain (2%) from $q - \varepsilon_a$ relationship. The ESP indicates that the sample has a contractive tendency during shearing throughout the triaxial test (Figure 5.11). Table 5-5 summarizes the triaxial test results for the sample obtained from BH84-04.

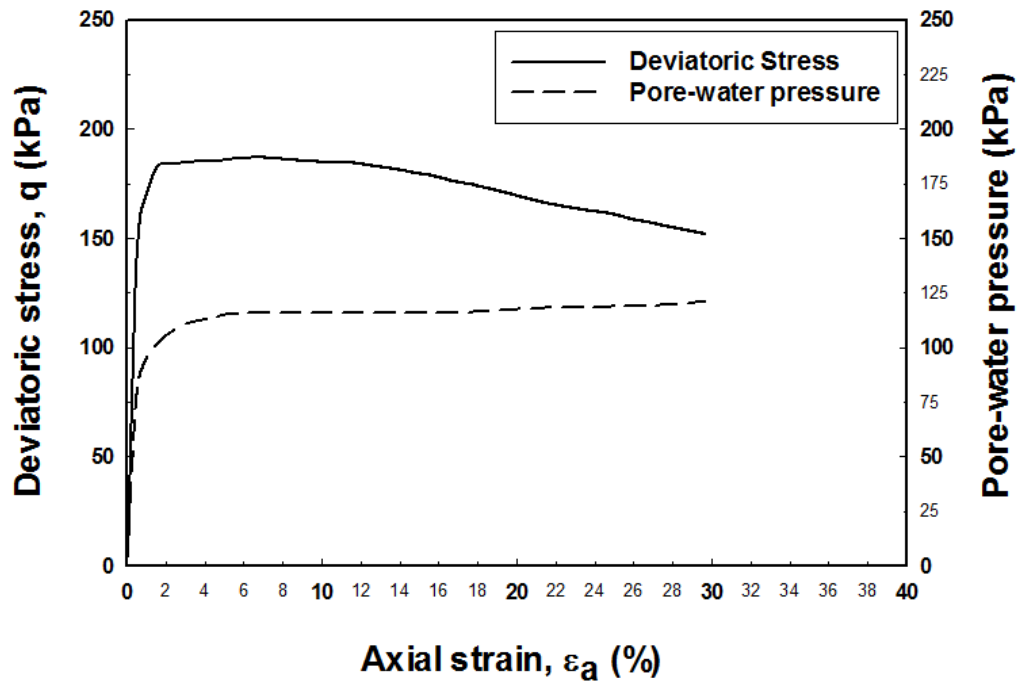


Figure 5.10. Variation of deviatoric stress and pore-water pressure with respect to axial strain (SY19-02)

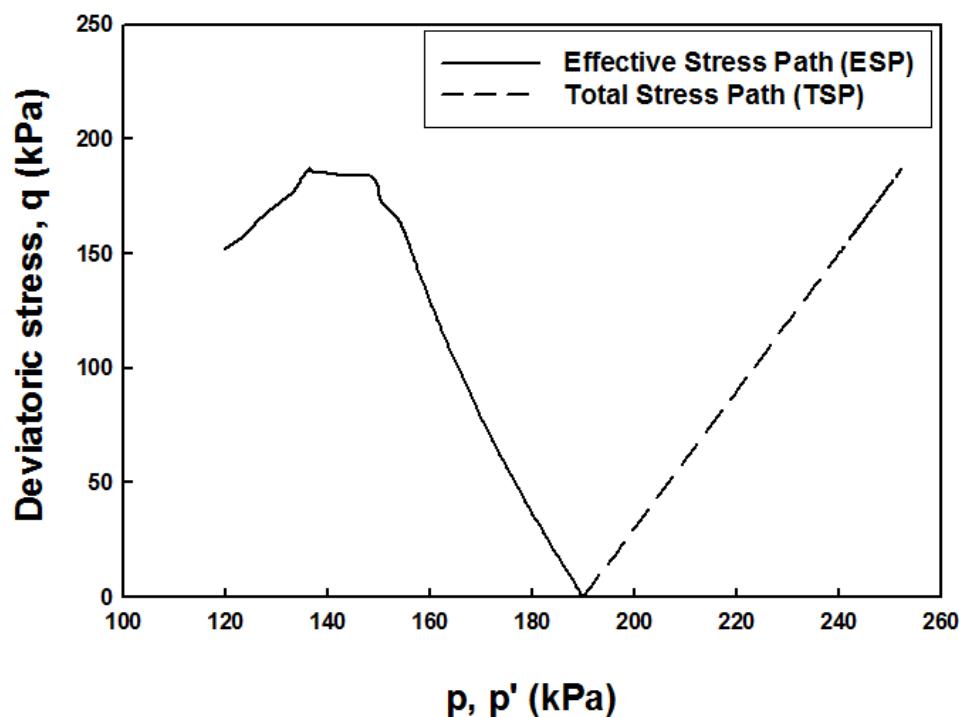


Figure 5.11. Effective and total stress paths from the triaxial test (SY19-02)

Table 5-5. Summary of the triaxial test (SY19-02)

Depth (m)	w (%)	γ (kN/m ³)	End of consolidation		σ'_3 (kPa)	$\sigma'_1 - \sigma'_3$ (kPa)	ε_a at failure (%)	u at failure (kPa)
			w (%)	e				
9	21.9	20.9	19.97	0.55	190	186	8.15	120

5.2.2.2 Consolidation Test

The oedometer test was also performed on the sample obtained from SY19-02, and the result is shown in Figure 5.12. The procedures to find preconsolidation pressure and slopes of in-situ virgin consolidation line (C_c) and URL (C_s) are explained in Section 5.2.1.2. The summary of the main parameters obtained from the oedometer test is given in Table 5-6.

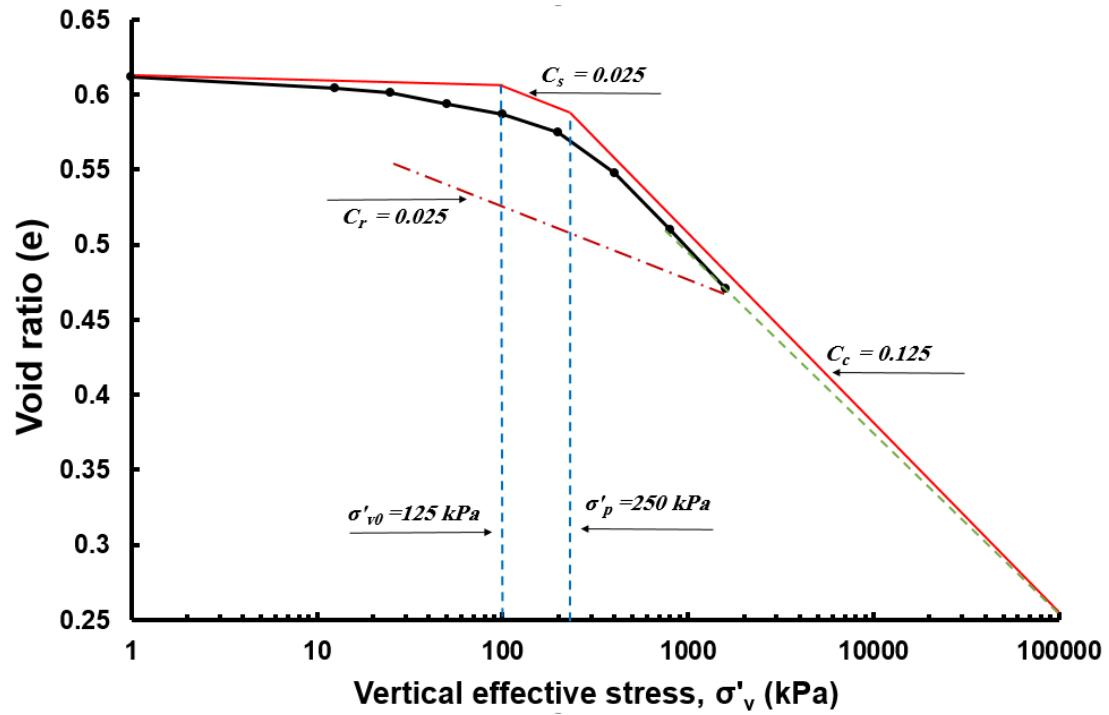


Figure 5.12. Estimation of in-situ virgin consolidation line using Schmertmann (1955) method (SY19-02)

Table 5-6. Summary of oedometer test results (SY19-02)

e_0	C_c	C_s	σ'_p (kPa)	σ'_{v0} (kPa)	OCR
0.611	0.125	0.025	250	125	2

5.3 Determination of MCC Input Parameters

In the following sections, the procedures to determine the input parameters required in the MCC model are described.

5.3.1 Frictional Constant M

The constant M is the slope of CSL in $q - p'$ plane. The shearing test should continue until no more volume change takes place at a large strain (i.e. critical state). The constant, M can also be written in terms of effective internal friction angle, ϕ' , shown in Eq. (5.1).

$$M = \frac{6 \sin \phi'}{3 - \sin \phi'} \quad (5.1)$$

5.3.2 Slopes of Normal Consolidation Line (λ) and Swelling Line (κ)

The parameters, λ and κ can be obtained based on consolidation test results with loading and unloading stages. A triaxial test under isotropic condition or conventional oedometer test can be conducted to obtain these two parameters. In case of the oedometer test, the normal compression phase and swelling phase can be expressed as Eq. (5.2) and Eq. (5.3), respectively.

$$v = v_m - C_c \log \sigma'_v \quad (5.2)$$

$$v = v_n - C_s \log \sigma'_v \quad (5.3)$$

where v_m and v_n are intercepts for $\sigma'_v = 1$ kPa.

By correlating [Eq. (4.4) and Eq. (5.2)] and [Eq. (4.5) and Eq. (5.3)], λ and κ can be computed using Eq. (5.4) and Eq. (5.5), respectively.

$$\lambda = \frac{C_c}{\ln(10)} \quad (5.4)$$

$$\kappa = \frac{C_s}{\ln(10)} \quad (5.5)$$

5.3.3 Location of Normal Consolidation Line in $v - \ln(p')$ Plane

The locations of normal consolidation line (NCL, Eq. (4.4)), unloading-reloading line (URL, Eq. (4.5)), and critical state line (CSL, Eq. (4.6)) are shown in Figure 5.13. The slopes of NCL (i.e. λ) and URL (i.e. κ) and initial void ratio (e_0) are determined experimentally. CSL and NCL are parallel to each other with the same slope, λ . N (Eq. (4.4)) and v_s (Eq. (4.5)) are the specific volumes that correspond to the reference pressure ($p'=1$ kPa) on NCL and URL, respectively. Finally, the specific volume that corresponds to the reference pressure on CSL is computed using Eq. (4.7) to find the location of CSL.

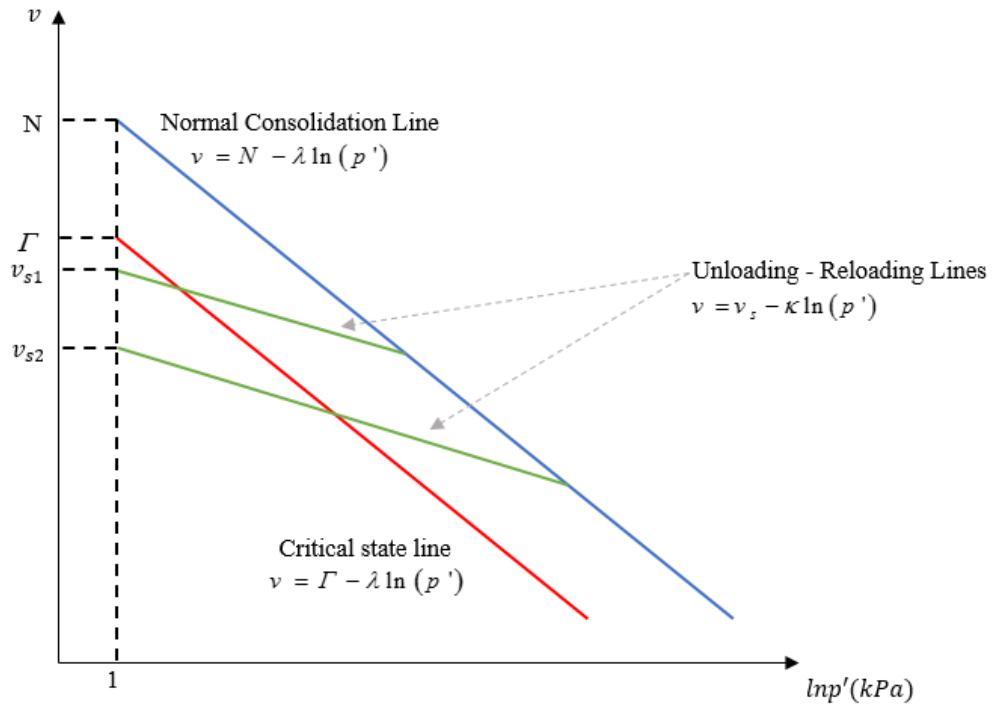


Figure 5.13. Location of normal consolidation, unloading – reloading and critical state lines in $v - \ln(p')$ plane

5.3.4 Preconsolidation Pressure, P'_{c0}

The size of the initial yielding curve in the MCC model is determined based on the preconsolidation pressure, P'_{c0} obtained from the isotropic triaxial test. In case of the oedometer test, P'_{c0} can be obtained from the previous maximum p' and q , using Eq. (5.6).

$$q_{\max}^2 = M^2 \left[p'_{\max} (p'_{c0} - p'_{\max}) \right] \quad (5.6)$$

Eq. (5.7) through Eq. (5.9) can be used to compute $p'_{\max}$ and $q_{\max}$ based on oedometer test results.

$$p'_{\max} = \frac{\sigma'_p + 2\sigma'_{h\max}}{3} \quad (5.7)$$

$$q_{\max} = \sigma'_p - 2\sigma'_{h\max} \quad (5.8)$$

$$\sigma'_{h\max} = K_0 \times \sigma'_p \quad (5.9)$$

where K_0 is the lateral earth coefficient at rest and $\sigma'_{h\max}$ is maximum horizontal effective stress.

5.3.5 Maximum Value of Elastic Parameters, K' and G

In the MCC model, stiffness of soil is defined using bulk modulus, K' . As the volume of soil changes during loading K' changes accordingly as shown in Eq. (5.10).

$$K' = \frac{vp'}{\kappa} \quad (5.10)$$

In FLAC3D, instead of the net value of K' , maximum bulk modulus (i.e. threshold) is assigned to each element, and at each step of running, the model evaluates whether the assigned value is adequate for the stability of the model or not. The maximum bulk modulus is given as Eq. (5.11)

$$K'_{\max} = K' + \frac{4}{3}G \quad (5.11)$$

where G is the shear modulus.

In this research, K' and G were estimated using Eq. (5.12) and Eq. (5.13), respectively, assuming elastic behavior. E was determined based on the initial linear portion of stress-strain curve from the triaxial test.

$$K' = \frac{E}{3(1-2\mu)} \quad (5.12)$$

$$G = \frac{E}{2(1+\mu)} \quad (5.13)$$

5.3.6 Summary of Input Parameters used in the MCC Model

The input parameters used in the MCC model for BH84-04 and SY19-02 samples are summarized in Table 5-7.

Table 5-7. Input parameters used in the MCC model obtained based on triaxial and oedometer test results for BH84-04 and SY19-02 samples

Test	Parameter	unit	Value	
			BH84-4	SY19-02
Oedometer	C_s	-	0.024	0.025
	C_c	-	0.12	0.125
	σ'_p	kPa	240	250
	p'_{c0}	kPa	210.37	215.22
	λ	-	0.052	0.052
	κ	-	0.01	0.01
	OCR	-	2.4	2
	$k_{0,nc}$	-	0.48	0.46
	N	-	2.458	2.511
Triaxial	ϕ'	°	32	32
	c	kPa	0	0
	M	-	1.28	1.28
	E	kPa	2.62×10^5	2.62×10^5
	K'	kPa	2.18×10^5	2.18×10^5
	G	kPa	1×10^5	1×10^5
	K'_{max}	kPa	3.5×10^5	3.5×10^5
	μ	-	0.3	0.3
	$Failure\ strain$	%	15.3	8.15

5.4 Estimating ESP using the MCC Model

Figure 5.14 shows the comparison of measured $q - \varepsilon_a$ relationship from the triaxial test with that estimated from the MCC model for the BH84-04 sample. The estimated relationship well captured the initial portion of the measured $q - \varepsilon_a$ behavior (i.e. $\varepsilon_a < 2\%$). The estimated shear strength was significantly low compared to the measured value.

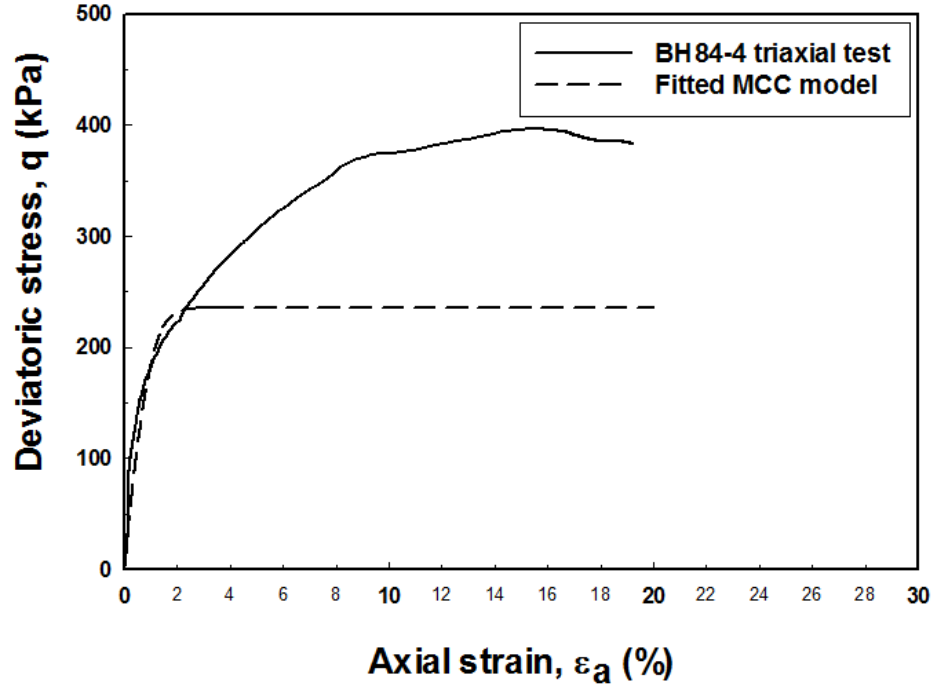


Figure 5.14. Comparison of $q - \epsilon_a$ relationships from the triaxial test and the MCC model (BH84-04)

Figure 5.15 shows the comparison of the ESPs from the triaxial test and the MCC model for BH84-04 sample. A dilative behavior was observed at the beginning of shearing. The soil then reaches the first phase-transformation point in which the state of soil changes from dilative to contractive behavior. In the end, the soil again exhibited dilative behavior as it approaches the CSL at a relatively constant stress ratio, q/p' (i.e. M). Several researchers showed that this is typical behavior of silty soils (BH84-04 sample consists of about 75% of silt and 25% of clay with a trace of fine sand) (Jefferies and Been 2015; Usmani et al. 2011; Yu et al. 2014). However, they did not provide detailed explanations of this behavior. As can be seen in Figure 5.15, the MCC model did not capture this characteristic behavior. The phase-transformations during shearing can be captured by using different constitutive models such as MIT-E3, MIT-S1 and Norsand. However, these constitutive models require

comprehensive in-situ and laboratory tests. Although the MCC model failed to capture the phase-transformations, the estimated undrained shear strength was conservative compared to the measured value (Figure 5.14 and Figure 5.15).

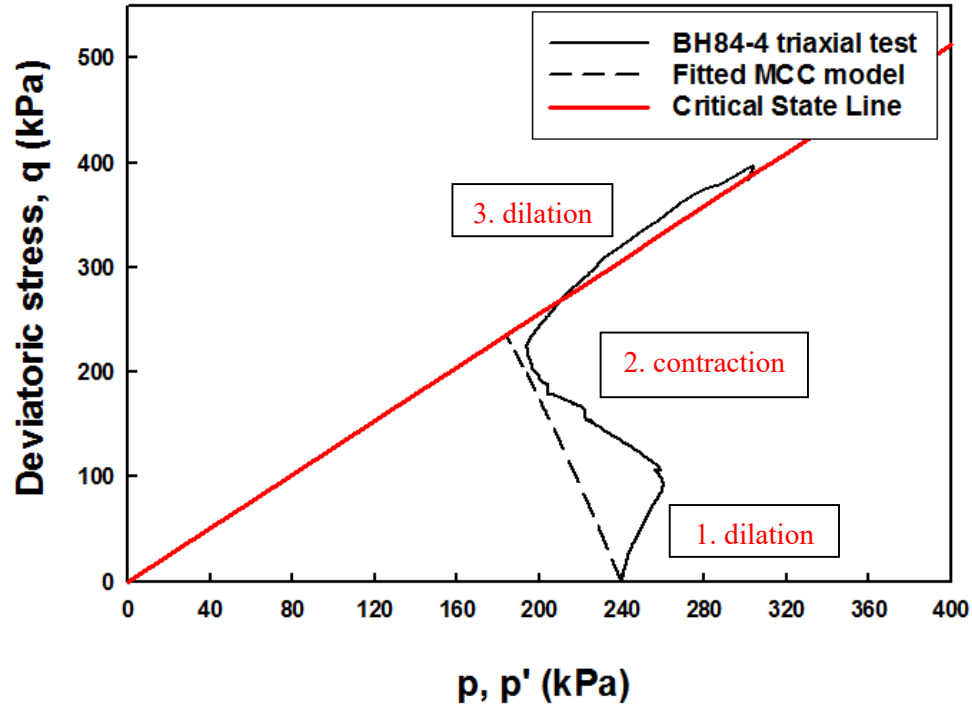


Figure 5.15. Comparison of ESPs from the triaxial test and the MCC model (BH84-04)

Figure 5.16 shows the comparison of the measured q versus ε_a relationship from the triaxial test and that estimated using the MCC model for the sample obtained from SY19-02. The comparison between the measured and estimated ESPs is shown in Figure 5.17. The soil exhibited a contractive behavior at the beginning of the shearing. As can be seen, the MCC model well captured q versus ε_a relationship and ESP.

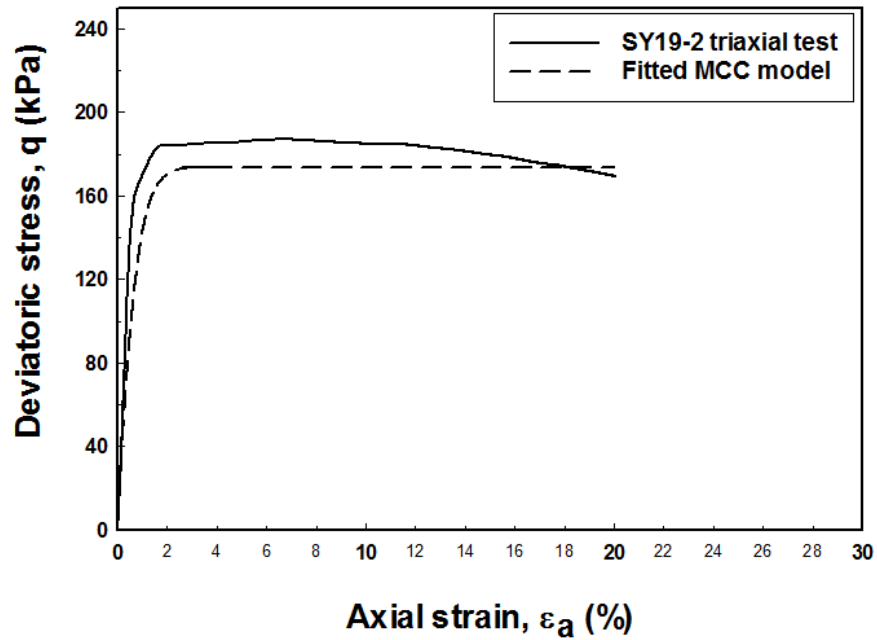


Figure 5.16. Comparison of $q - \epsilon_a$ relationships from the triaxial test and the MCC model (SY19-02)

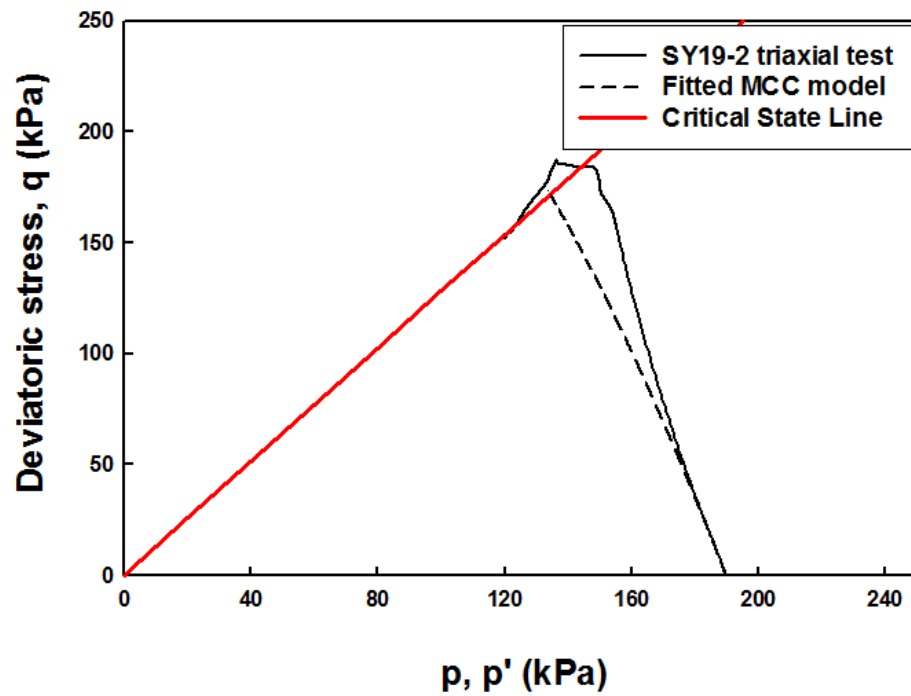
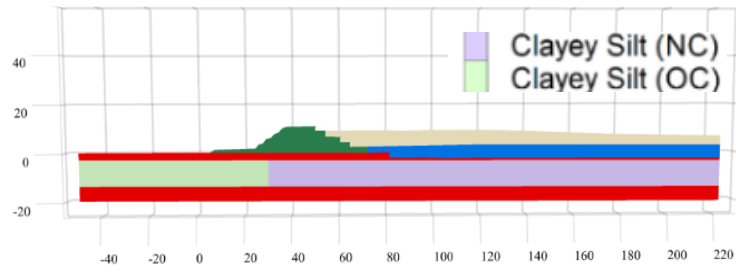


Figure 5.17. Comparison of ESPs from the triaxial test and the MCC model (SY19-02)

5.5 Numerical Modeling

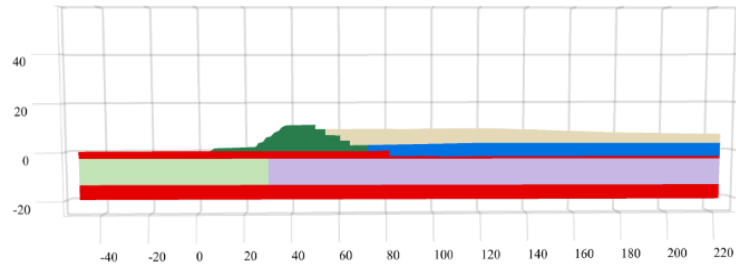
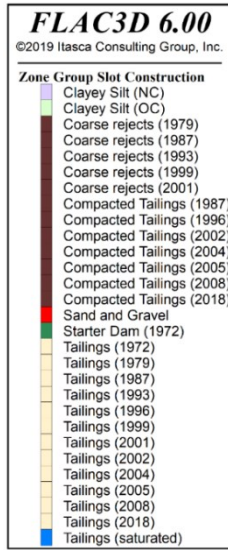
Since the stability of the tailings dam was carried out for Section C-C', a plane strain condition was chosen in the analysis. The dimensions of the domain in x , y , and z directions are -50 m to 225 m, 0 m to 1 m, and -25 m to 50 m, respectively. The foundation consists of a clayey silt layer and two layers of sand and gravel. Based on the geotechnical investigations, the clayey silt layer was divided into two regions; Normally Consolidated (NC) region (beneath a tailings dam) and Over Consolidated (OC) region (below the toe). The BH84-04 sample was obtained from the NC region while the SY19-02 sample was obtained from the OC region. It was assumed that the behaviors of both sandy layers are similar. The properties of all deposited tailings are assumed to be the same, except the unit weight of saturated tailings at the ground level. Other parts of the tailings dam include compacted coarse sandy rejects (i.e. waste materials obtained from mining process) and compacted tailings.

The tailings dam construction was modeled in 12 stages by following the raise construction history summarized in Table 5-1, starting from 1,112 m above sea level. The level of the tailings dam in the year 2020 is 1,166 m. It was assumed that raising a tailings dam and deposition of tailings took place instantaneously. As the measured coefficient of volume compressibility, m_v , is approximately $2 \times 10^{-3} \text{ cm}^2/\text{s}$ the drainage takes place for long period of time, which reflects that the clayey silt layer is relatively stiff. Figure 5.18 shows the profile views of section C-C' modeled in FLAC3D between 1972 and 2005. The profile view in the year 2008 (also current condition) is shown in Figure 5.19.



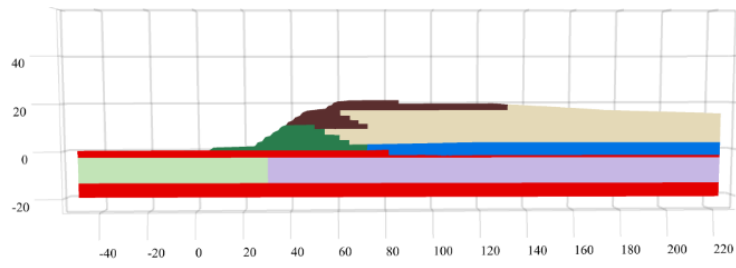
(1972)

(a)



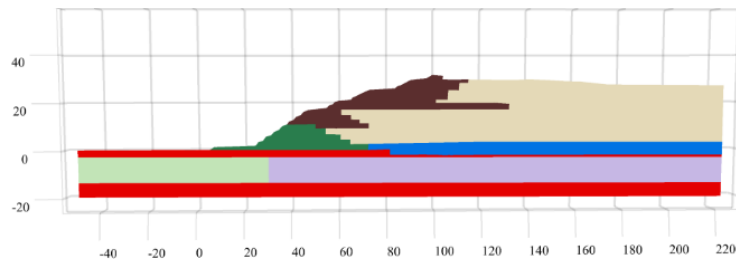
(1979)

(b)



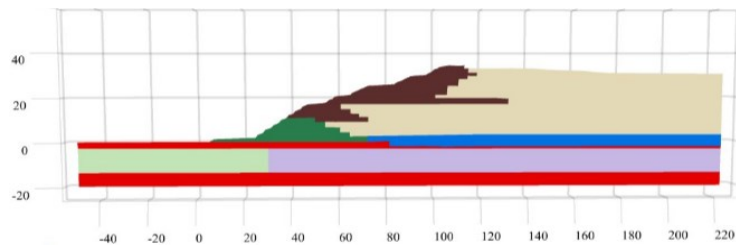
(1987)

(c)



(1993)

(d)



(1969)

(e)

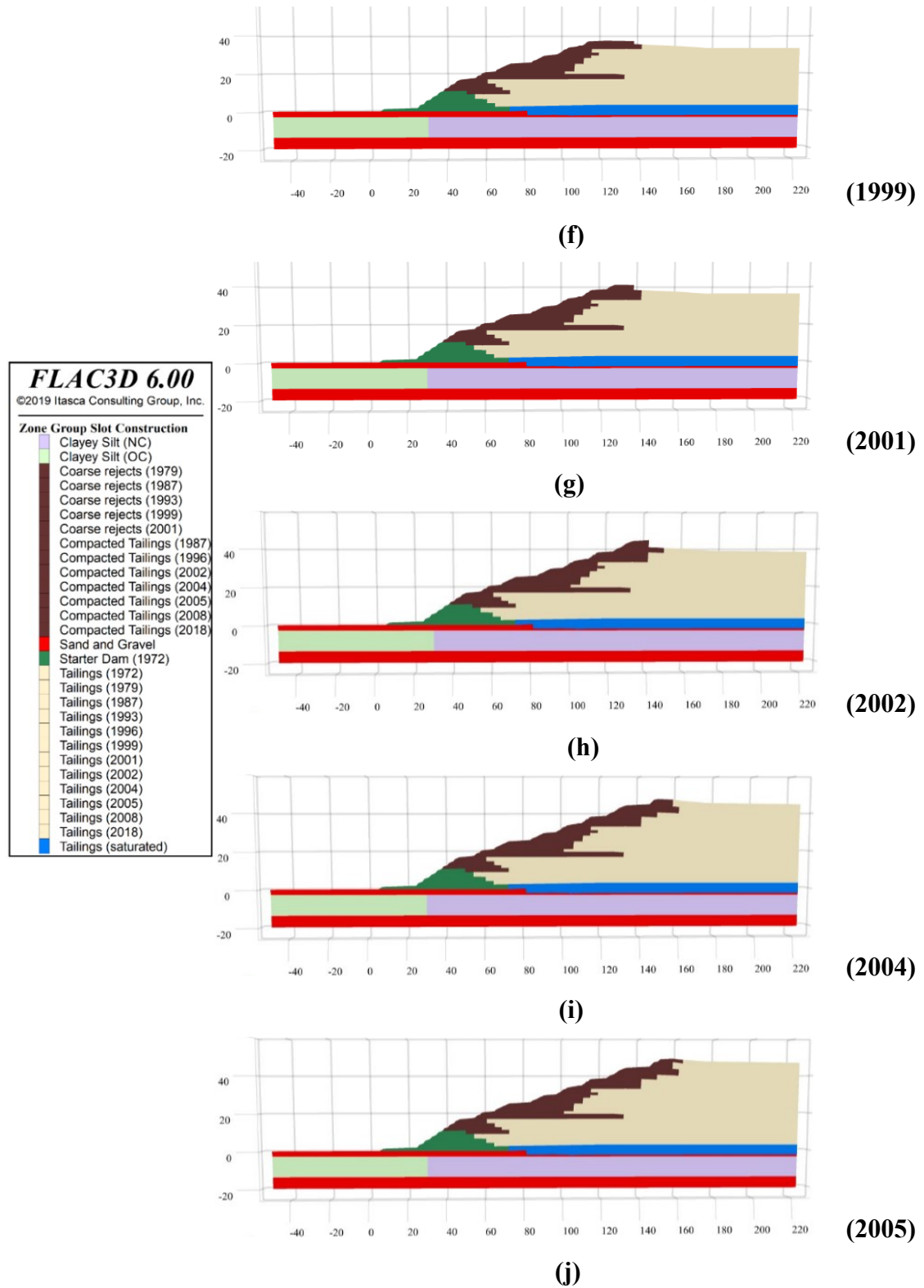


Figure 5.18. Profile view of section C-C' modeled in FLAC3D for different years: (a) 1972; (b) 1979; (c) 1987; (d) 1993; (e) 1996; (f) 1999; (g) 2001; (h) 2002; (i) 2004; (j) 2005

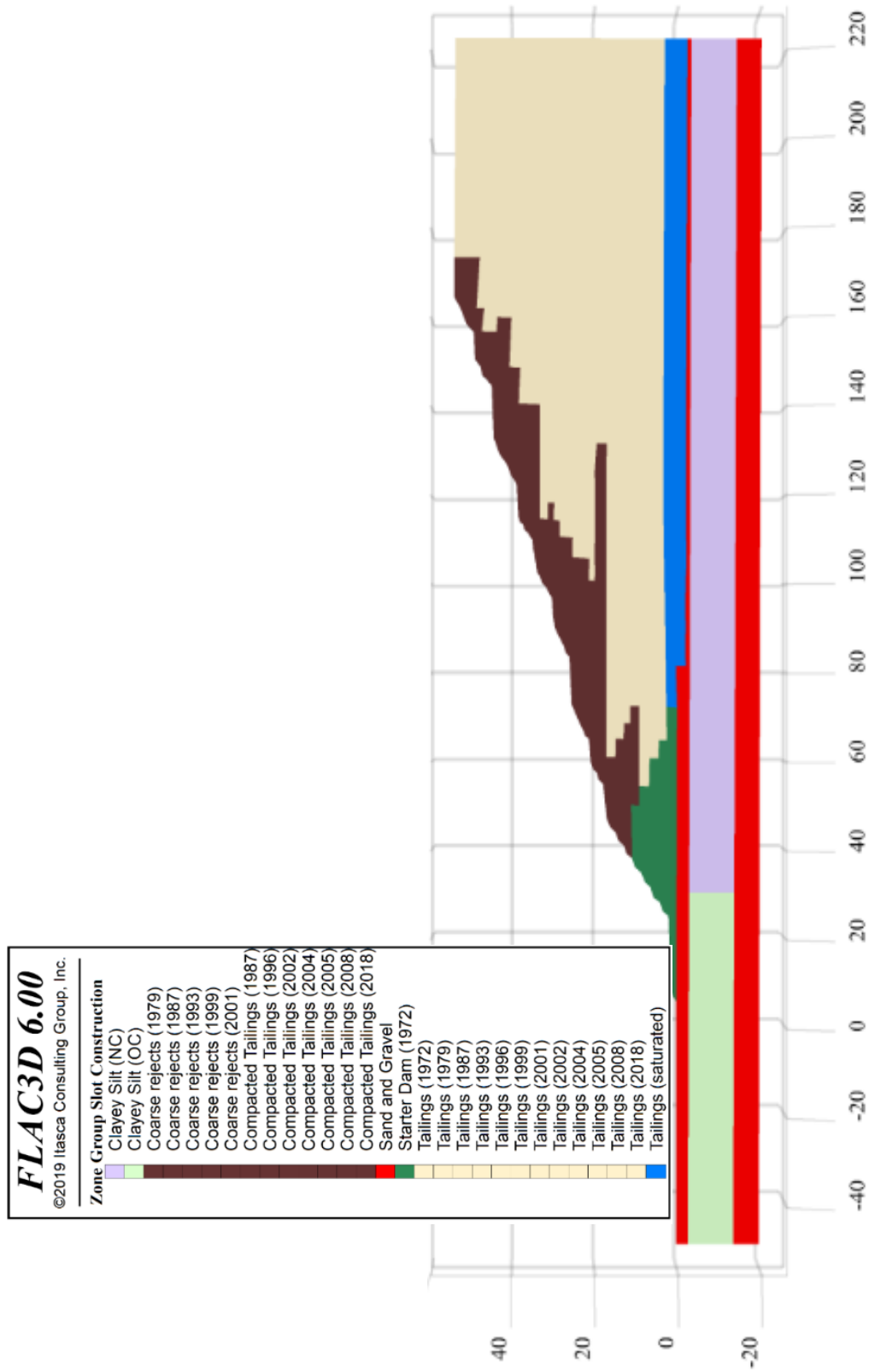


Figure 5.19. Profile view of section C-C' modelled in FALC3D in the year 2008

A total number of 3826 elements were used to create the mesh. Mesh size and thickness were determined based on a mesh-convergency study conducted with different element lengths and thicknesses (1 m). The mesh size within the clayey silt layer starts at $1 \text{ m} \times 1 \text{ m}$ elements, transitioning to a larger size with a ratio of 1.1 for the parts beneath the toe of the tailings dam. The mesh size within the tailings and dams starts at $1.2 \text{ m} \times 1.2 \text{ m}$ and increases with the ratio of 1.2 in some parts. A finite element mesh pattern of ‘quads’ was used to provide a smooth transition between areas of interest. Unlike other typical finite element software, in FLAC3D, the user cannot directly assign restrained displacement boundary conditions at the ends of the domain in horizontal and vertical directions. Instead, ‘velocity’ is assigned as a displacement boundary condition, which is defined as the ratio of displacements over the number of steps. For example, normal velocity-zero in x -direction applied to a surface on a boundary indicates that the displacements in the x -direction are restrained on this surface. In case of the EVO, velocities normal to eastern, western, northern, and southern sides were assumed to be restricted to zero, and a fixed boundary condition (i.e. x - y - z velocities are zero) for the base of the model is chosen, assuming tailings dam lies on a dense, rigid layer (see Figure 5.20). The coefficient of earth pressure at rest, $K_0 = 0.48$ was used in numerical modeling using the equation proposed by Jaky (1944; Eq. (5.14)).

$$K_0 = 1 - \sin \phi' \quad (5.14)$$

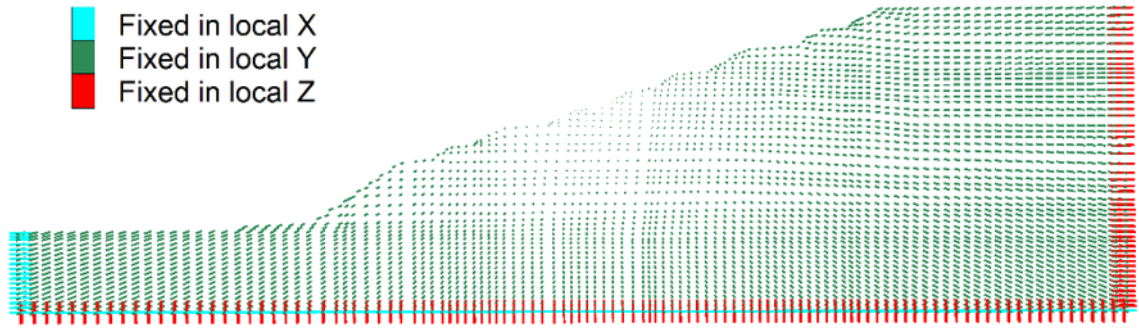


Figure 5.20. Displacement boundary condition assigned in FLAC3D for the section C-C'

The Mohr-Coulomb constitutive model was chosen for all the materials (i.e. compacted sand or gravel and compacted coarse rejects as dam, sand and gravel layer in the foundation) other than the clayey silt layer. Poisson's ratio, $\mu = 0.3$ was used for all material. Since the clayey silt soil was dominant in the foundation with contractive tendency during shearing, the angle of dilatancy was assumed to be zero. The elastic modulus for tailings and compacted soils were obtained based on the triaxial test results. The shear strength parameters for all materials were obtained from the triaxial and direct simple shear (DSS) test results. Summary of the input parameters used in the MCC model for the clayey silt layer is presented in Table 5-8. Table 5-9 shows the input parameters used in the Mohr-Coulomb model for tailings, dams, and sandy layer.

Table 5-8. Summary of the input parameters used in the MCC model for clayey silt layer

properties	Unit	Normally consolidated clayey silt	Overconsolidated clayey silt
γ	kN/m ³	19.6	19.6
K'_{max}	MPa	350	350
M	°	1.28	1.28
λ	MPa	0.05	0.05
κ	-	0.01	0.01
P'_c	kPa	240	240
P'_l	kPa	1	1
N	-	2.511	2.511
M_f	MPa	50	50

Table 5-9. Summary of the input parameters used in the Mohr-Coulomb model for tailings, dams and sandy layer

Properties	Unit	Compacted coarse rejects	Compacted tailings	Sand and gravel	Starter dam	Tailings (drained)	Tailings (saturated)
γ	kN/m ³	16.5	16.5	19.3	14	12	13.4
c'	kPa	0	0	0	0	0	0
ϕ'	°	35	35	36	32	32	32
E	MPa	50	50	100	100	50	50
μ	-	0.3	0.3	0.3	0.3	0.3	0.3
dilation angle	°	0	0	0	0	0	0

The piezometer readings indicated that there was no noticeable change in the level of the phreatic surface during the construction of dams and deposition of tailings. In addition, the coefficient of consolidation of the clayey silt layer was reported as $1.5 \times 10^{-3} \text{ cm}^2/\text{s}$. Hence, it can be postulated that the dissipation of excess pore-water pressures within the clayey silt layer occurred fairly quickly. Based on these observations, the phreatic surface was assumed to remain unchanged throughout the construction period.

One of the most important aspects of numerical modeling in FLAC3D is the selection of a modeling approach. The model, in this study, contains both fluid and solid materials. In FLAC3D, four modeling approaches are available for cases in which water and soil are in contact with each other, including: i) fixed pore pressure, ii) flow-only calculation to establish a pore pressure distribution, iii) no-flow (mechanical generation of pore pressure) and iv) coupled flow and mechanical calculations. Although the coupled analysis is the most appropriate approach it is time-consuming to reach convergence. Hence, it is suggested to use one of the other three approaches rather than coupled analysis with reasonable simplification in the modeling process (Itasca 2005). As the phreatic surface is in a steady-state condition throughout the construction, the no-flow boundary condition was used in the analysis, which is associated with the undrained condition. In FLAC3D, the undrained (i.e. short-term) response of soil is analyzed by incorporating pore-water pressure that is generated due to mechanical deformations (i.e. volumetric strains). That is, once a saturated soil undergoes instantaneous loading, pore-water pressure increases as a result of volumetric strains that occur in each zone. The change in pore-water pressure in association with volumetric strain can be defined with Biot modulus (M_f) as shown in Eq. (5.15).

$$M_f = \frac{k_f}{n} \quad (5.15)$$

where k_f is fluid bulk modulus and n is porosity.

For pure water at room temperature, k_f equals to 2×10^3 kPa (Itasca 2005). The fluid bulk modulus of the water in the field can be lower than that of pure water since it contains air bubbles. The magnitude of fluid bulk modulus has a substantial effect on the convergence time, where a high magnitude of the fluid bulk modulus results in a decrease in convergence time. From a numerical analysis point of view, it is not necessary to use k_f larger than 20 times K'_{max} in simulation (Itasca 2005). In this study, an optimum fluid bulk modulus of 50 MPa was used through sensitivity analysis.

The clayey silt layer was divided into 30 zones (Figure 5.21). FOS was then computed for each zone against undrained failure, rather than a global FOS, to enhance the reliability of stability analysis. Stability and deformation analyses were carried out for the center of each zone. Table 5-10 summarizes the coordinate of the center of each zone.

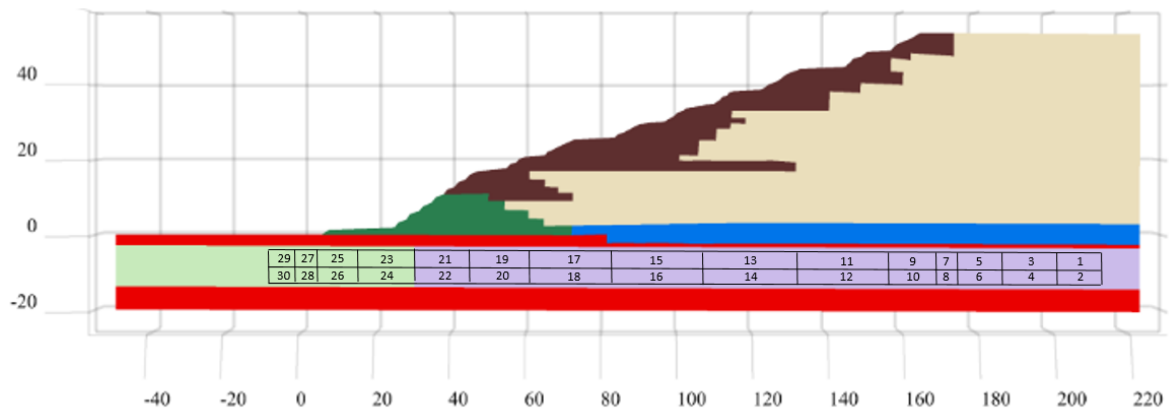


Figure 5.21. Discretization of the soft clayey layer beneath the tailings dam into 30 different arbitrary zones

Table 5-10. Coordinate of the center of each zone shown in Figure 5.21

zone	1	3	5	7	9	11	13	15	17	19	21	23	25	27	29
x	210	195	180	175	170	145	120	95	70	50	37	23	5	0	-5
-z	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45	6.45

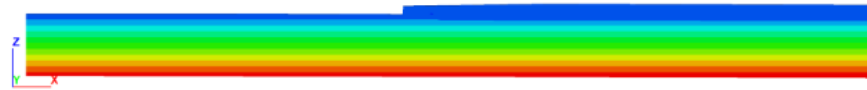
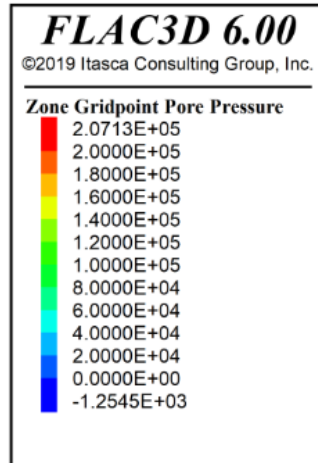
zone	2	4	6	8	10	12	14	16	18	20	22	24	26	28	30
x	210	195	180	175	170	145	120	95	70	50	37	23	5	0	-5
-z	12	12	12	12	12	12	12	12	12	12	12	12	12	12	12

6. RESULTS AND DISCUSSION

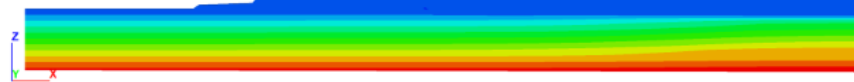
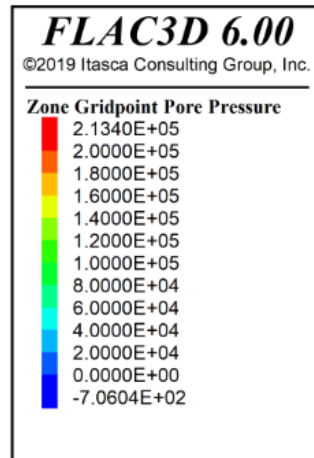
6.1 Preliminary Study

As a preliminary study, the variation of mean effective stress, deviatoric stress, and preconsolidation pressure during the construction were studied in FLAC3D. Distributions of pore-water pressure in the clayey silt layer for the initial condition and at the completion of construction are shown in Figure 6.1(a) and (b), respectively. As can be seen, no significant excess pore-water pressure was generated due to the loading from the tailings dams. This matches the field observation of which the phreatic surface remained unchanged throughout the construction. Figure 6.2(a) and (b) show the distribution of mean effective stress with depth in the clayey silt layer for the initial condition and at the completion of construction, respectively. The initial mean effective stress ranges from 35 kPa to 100 kPa at the top and bottom of the clayey silt layer, respectively (Figure 6.2(a)). Upon the completion of construction, mean effective stress increased by approximately 5 and 2 times in NC and OC regions, respectively.

Distributions of the vertical effective stress ($\sigma'_v = \sigma'_{zz}$) with depth for initial condition and at the completion of construction are shown in Figure 6.3(a) and (b), respectively (compression is negative). The maximum vertical stress increment is 600 kPa due to the construction of the tailings dam. The change in vertical effective stress in the OC region (i.e. before the toe of tailings dam in the clayey silt layer) is negligible. Figure 6.4(a) and (b) show the distributions of the horizontal effective stress ($\sigma'_h = \sigma'_{xx}$) with depth for the initial condition and at the completion of construction.

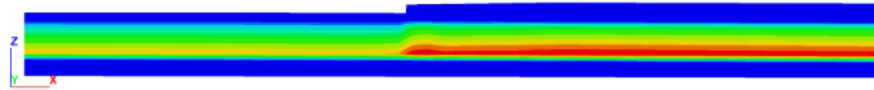
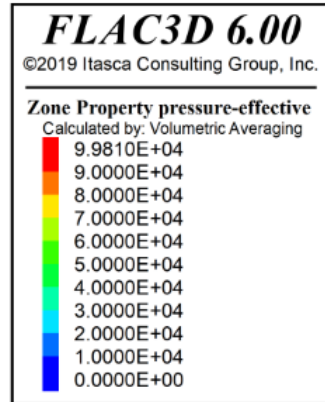


(a)

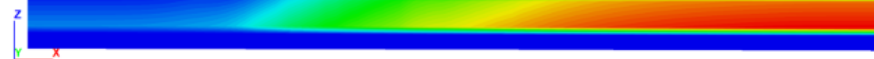
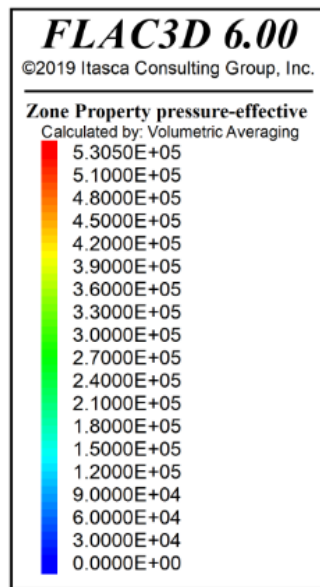


(b)

Figure 6.1. Distribution of pore-water pressure in the clayey silt layer: (a) initial condition; (b) at the completion of construction

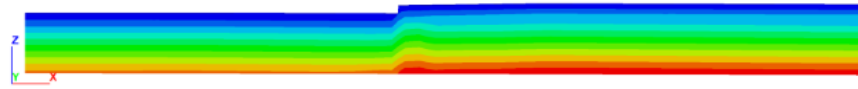
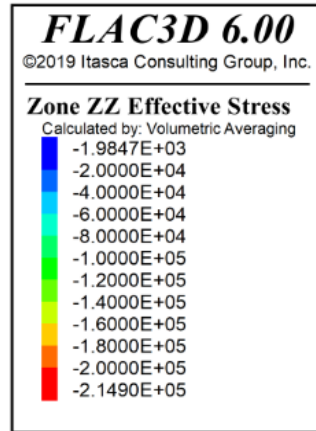


(a)

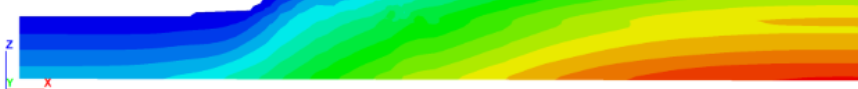
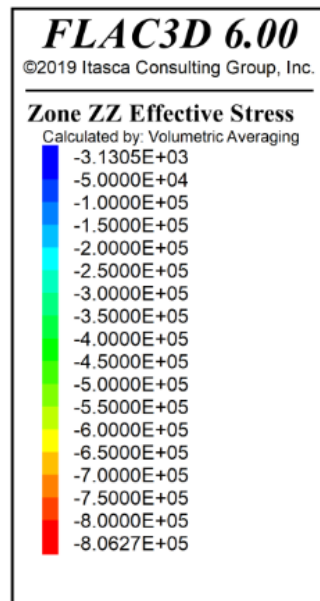


(b)

Figure 6.2. Distribution of mean effective stress in the clayey silt layer: (a) initial condition; (b) at the completion of construction

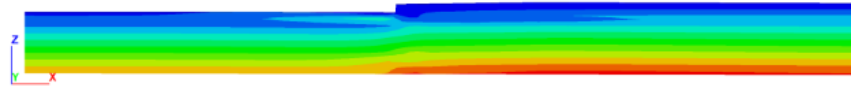
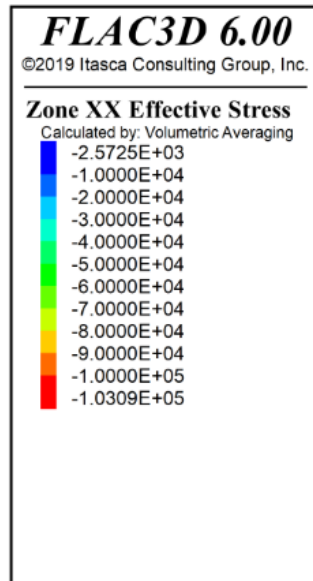


(a)

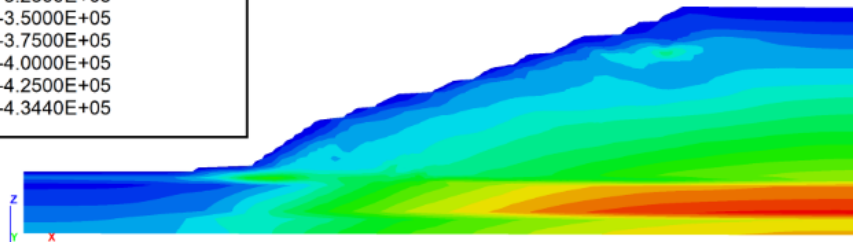
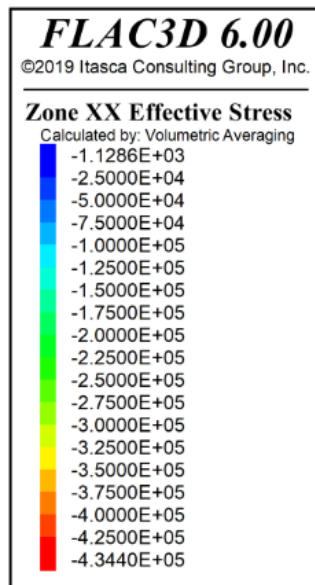


(b)

Figure 6.3. Distribution of vertical effective stress: (a) initial condition; (b) at the completion of construction



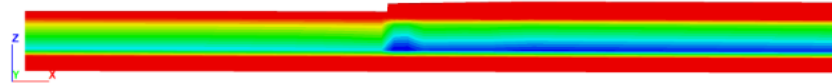
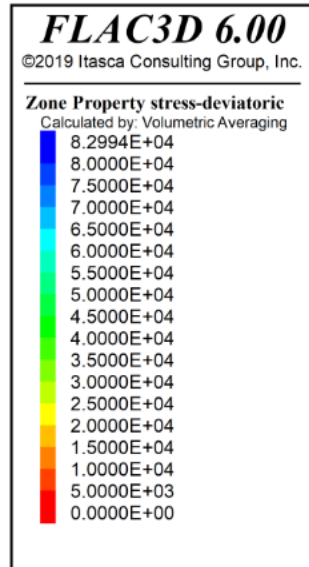
(a)



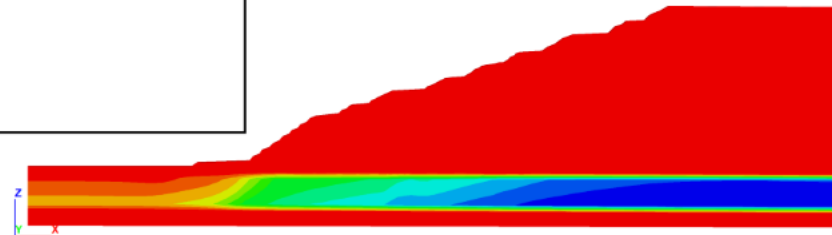
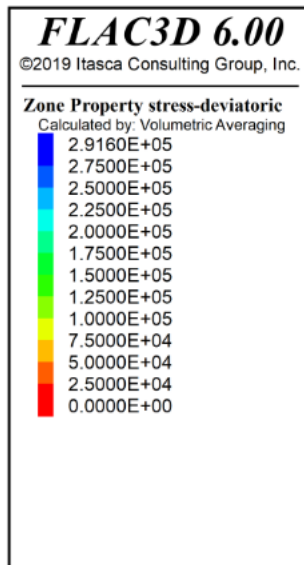
(b)

Figure 6.4. Distribution of horizontal effective stress: (a) initial condition; (b) at the completion of construction

Distributions of deviatoric stress in the clayey silt layer with depth for the initial condition and at the completion of construction are shown in Figure 6.5(a) and (b), respectively. The range of deviatoric stress for the initial condition was between 5 kPa and 85 kPa and the maximum increment at the completion of construction was approximately 300 kPa. The preconsolidation pressure of 240 kPa was assigned to the clayey silt layer (Figure 6.6(a)). At the completion of construction, the preconsolidation pressure increased up to 630 kPa in some parts. This increase results in an increase in undrained shear strength of the clayey silt layer.

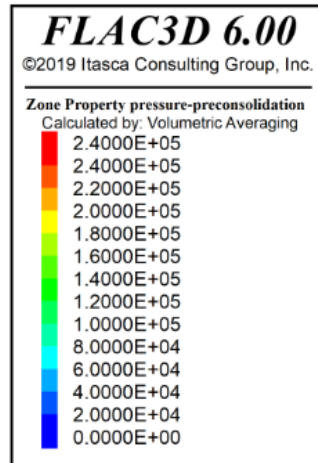


(a)

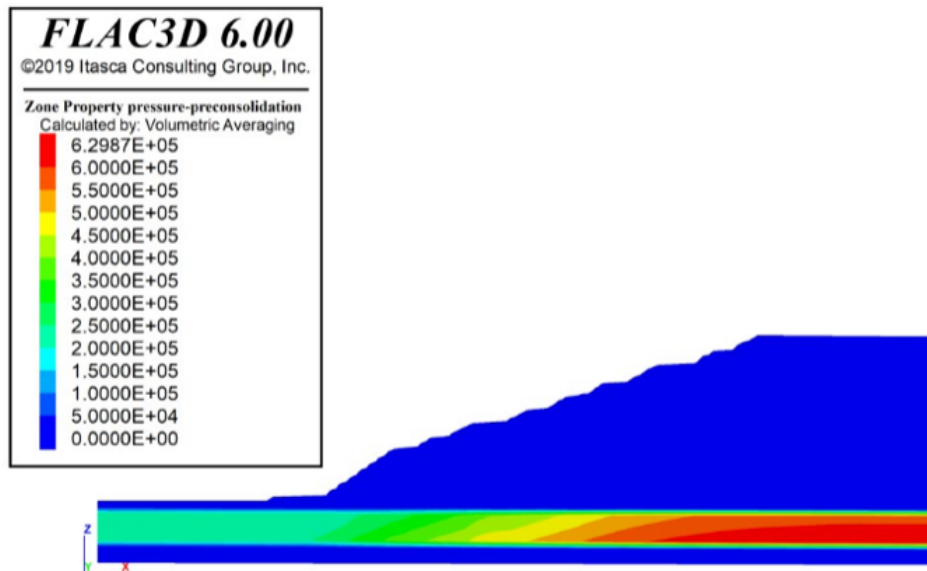


(b)

Figure 6.5. Distribution of deviatoric stress in the clayey silt layer: (a) initial condition; (b) at the completion of construction



(a)



(b)

Figure 6.6. Distribution of preconsolidation pressure in the clayey silt layer: (a) initial; (b) at the completion of construction

6.2 Stability Analysis considering Stress Path and FOS in Each Zone

ESP and TSP of Zone 1 under the staged construction are shown in Figure 6.7. The size of yielding curve for each construction stage was determined based on the corresponding preconsolidation pressure. Point O was used to initialize the mean effective stress in MCC model. The stress condition moves from A to B due to the construction of the starter dam and deposition of tailings in 1972. The mean effective stress then increases from B to C during consolidation without a noticeable change in deviatoric stress. The clayey silt layer was in a state of OC condition between 1972 and 1987 (i.e. A to F). Hence, during this period, ESP remains within the initial yield curve that corresponds to the preconsolidation pressure of 240 kPa. During the consolidation period after the construction in 1987, state of the clayey silt layer changes from OC to NC ($F \rightarrow G$) condition, which enlarges the yielding curve. The construction in 1993 then causes the movement of effective stress path along the new yielding curve (i.e. $G \rightarrow H$). These procedures are repeated until the completion of construction.

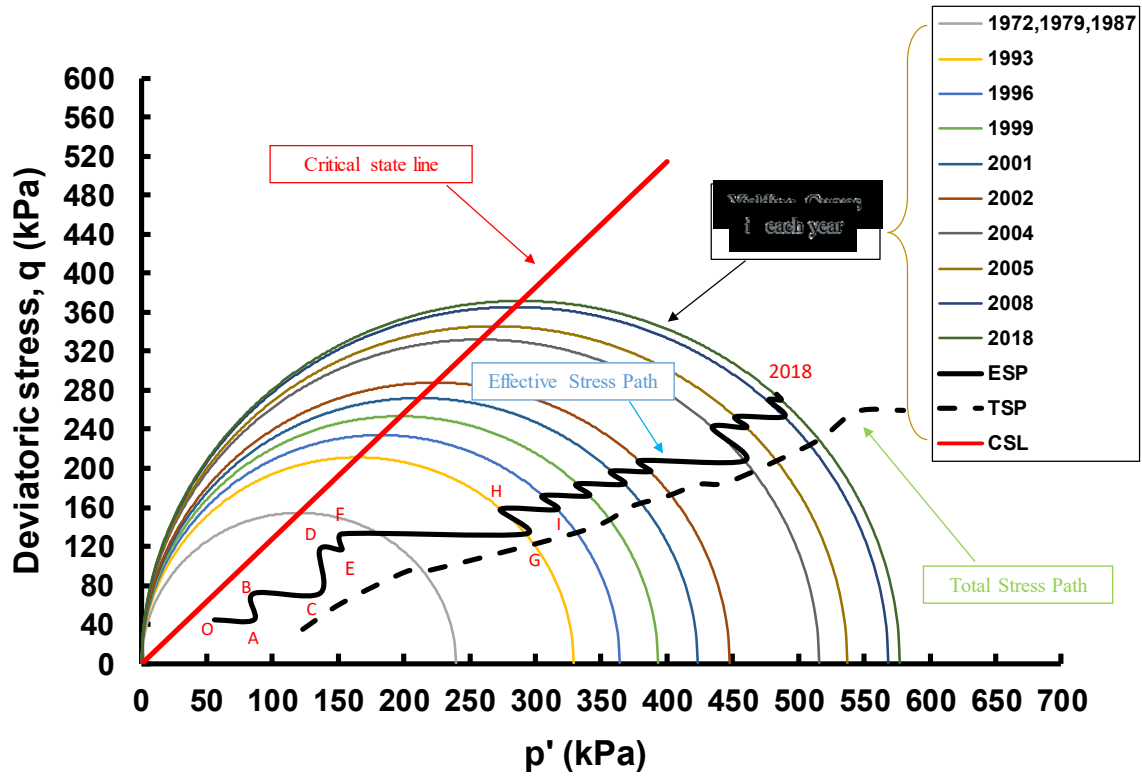


Figure 6.7. ESP and TSP of Zone 1

FOS of Zone 1 against undrained failure was computed using Eq. (4.22) which is the indicator of how far the ESP is away from CSL. The variation of FOS with time in Zone 1 is plotted in Figure 6.8. FOS decreased rapidly right after a staged construction under the undrained condition and then increased during the consolidation phase. The initial FOS was 2.46 and it dropped to a minimum of 1.48 in 1987. FOSs greater than 2.22 were maintained since 1993 when the state of the clayey silt layer changed to NC condition. The minimum FOSs for different years are summarized in Table 6-1.

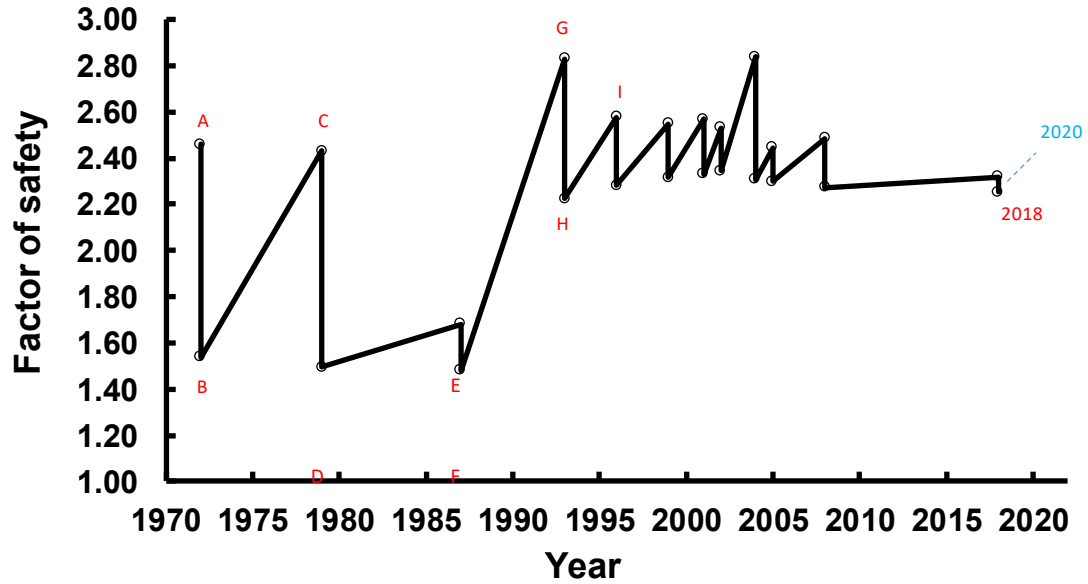


Figure 6.8. Variation of FOS under staged construction in Zone 1

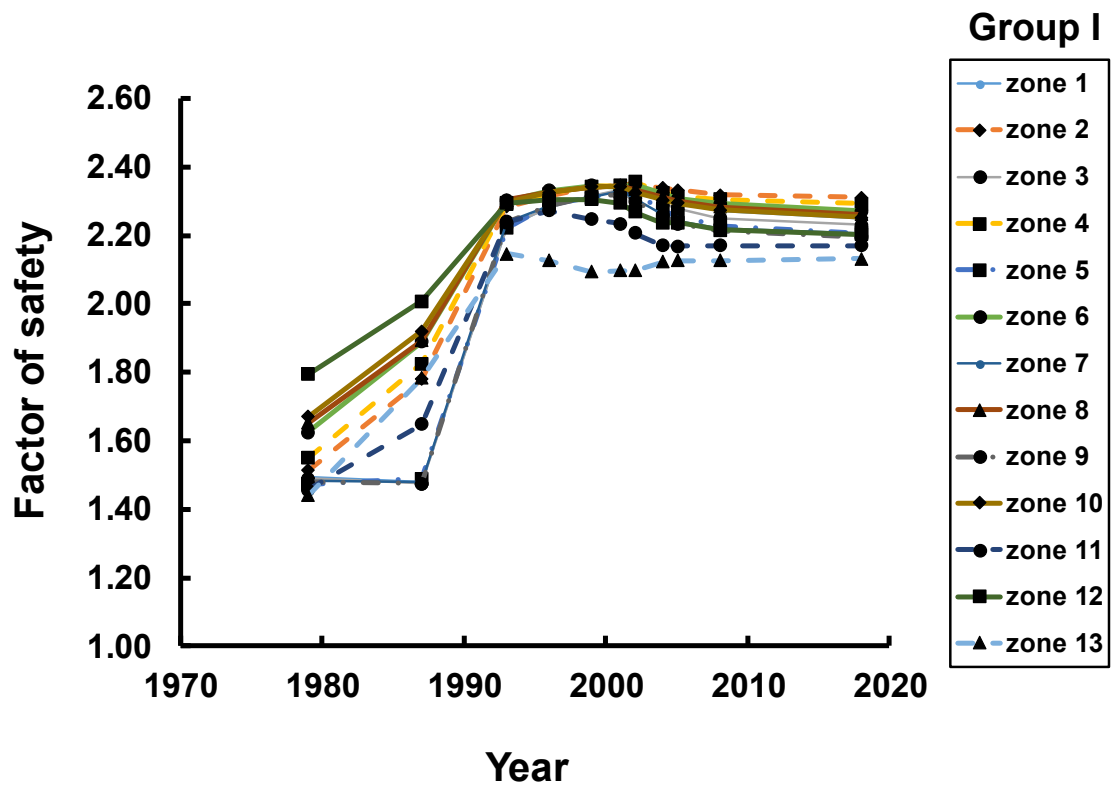
Table 6-1. Summary of minimum FOS in Zone 1 for different years

Year	1972	1979	1987	1993	1996	1999	2001	2002	2004	2005	2008	2018
FOS	1.54	1.5	1.48	2.22	2.28	2.31	2.33	2.34	2.31	2.30	2.27	2.25

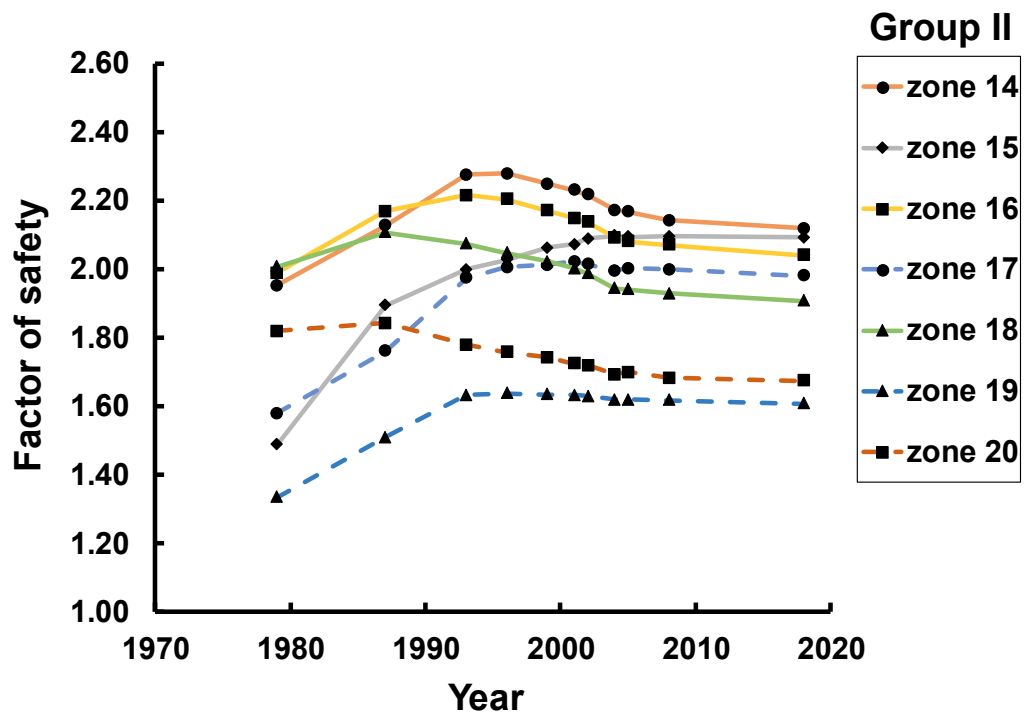
Effective stress paths for the remaining 29 zones (i.e. Zone 2 through Zone 30) are presented in the Appendix. Except for zones 23 through 30 that remained as OC condition, the state of other zones changed to NC before 1993. The EPSs in zones 1 to 9 are similar, which entered NC condition after 1993 except for zones 4, 6 and 8 (i.e. before 1993). The EPSs in zones 10 to 16 also showed similar patterns, which entered the NC condition before 1993. Since zones 17 to 25 are directly below the starter dam they are most susceptible to failure condition. For zones 26 through 30, ESPs were within the initial yielding curve and showed the behavior of OC soil during the entire period of construction. A noticeable gap was observed between 1987 and 1993 yielding curves. This is attributed to the reason that

a relatively large volume of tailings was deposited during this period compared to other deposition stages.

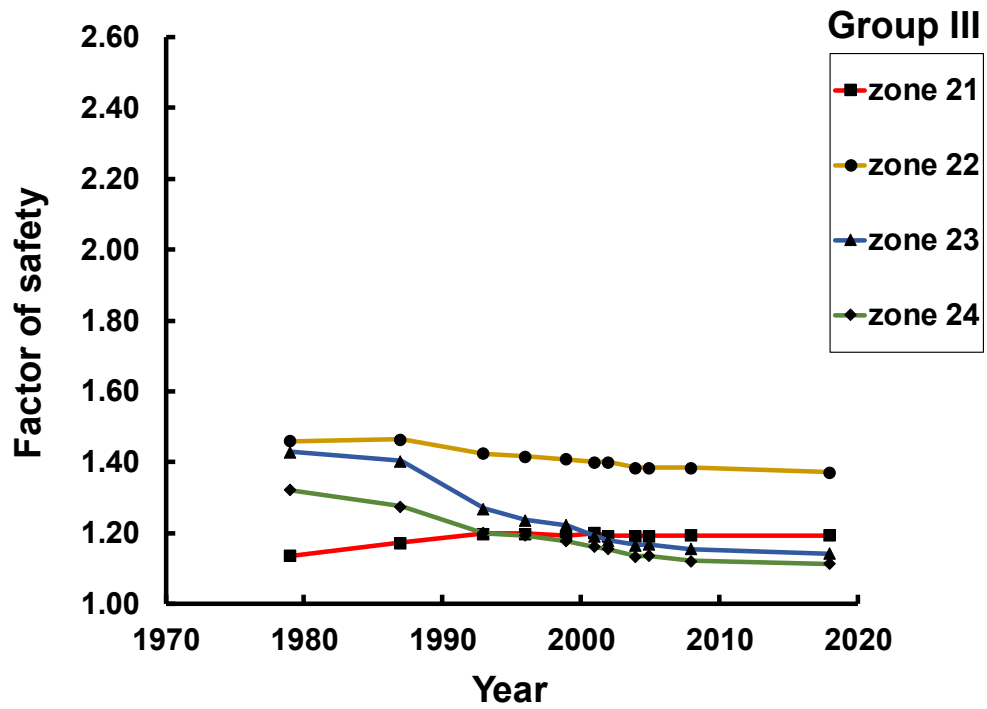
The variation of minimum FOS with time for different groups of zones is shown in Figure 6.9. For the zones in Group I and II, FOS increased until 1993 and then started decreasing thereafter. The ranges of FOS for the zones in Group I and Group II are 1.4 - 2.4 and 1.3 - 2.3, respectively. For the zones in Group III (except Zone 24), FOS gradually decreased until the completion of construction, while the change in FOS of the zones in Group IV was negligible. The ranges of FOS for the zones in Group III and Group IV are 1.1 - 1.5 and 1.6 – 2.0, respectively.



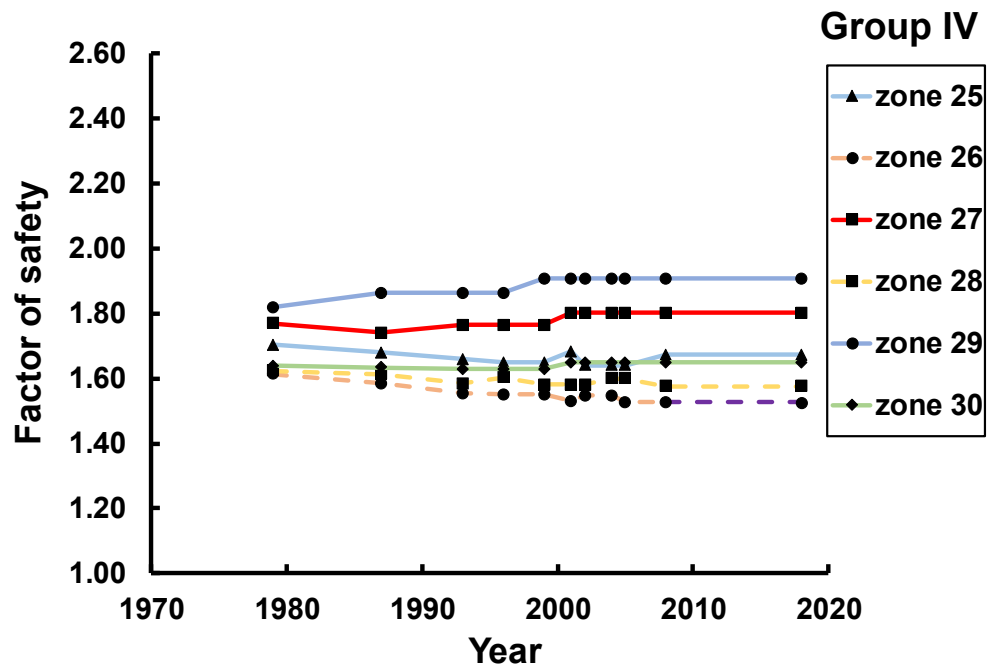
(a)



(b)



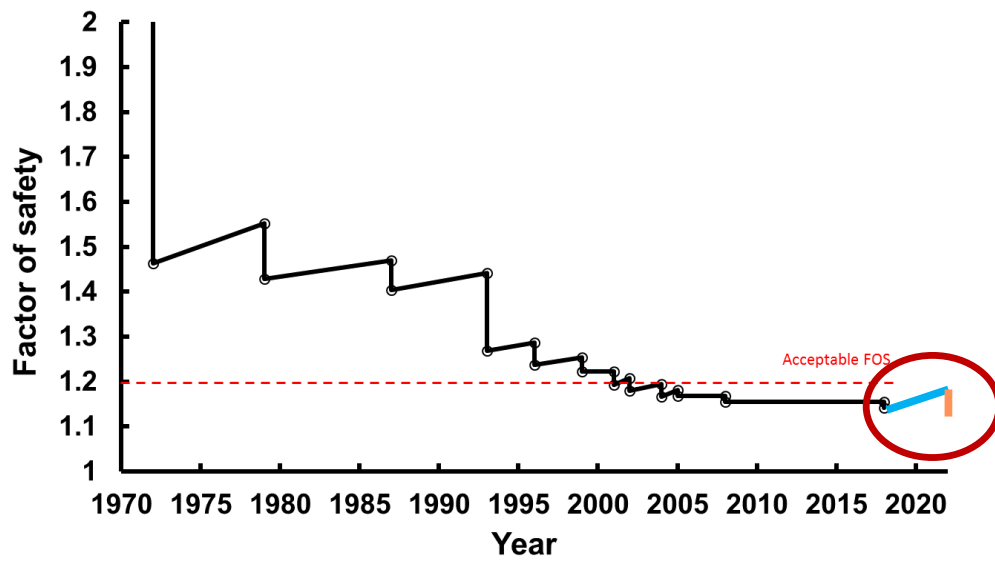
(c)



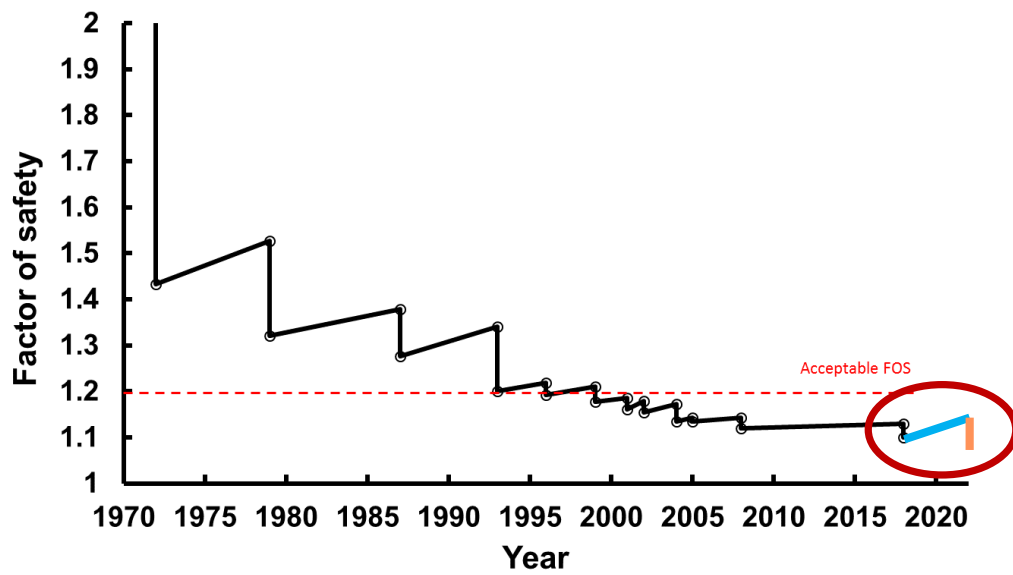
(d)

Figure 6.9. Variation of minimum FOS with time: (a) Group I (zones 1 – 13); (b) Group II (zones 14 – 20); (c) Group III (zones 21 – 24); (d) Group IV (zones 25 – 30)

Distribution of local FOS in different zones right after the deposition of tailings in 2018 is shown in Figure 6.11. As can be seen, local FOSs in the zones beneath the toe of the dam is lower compared to other zones. It turned out that the most vulnerable zones to undrained failure were Zone 23 and Zone 24 with FOS of 1.14 and 1.11, respectively. Figure 6.10 shows variation of FOS with time for zones 23 and 24, respectively. According to the numerical analysis, the local FOSs for Zone 23 and Zone 24 were still less than 1.2 (Acceptable FOS), even at the end of consolidation in 2020 (blue lines). Assuming deposition of 2 meters of tailings in 2021, FOSs will drop to below 1.1 for both zones (orange lines). The red solid line in Figure 6.11 shows the critical failure surface from LEM with undrained shear strength ratio of 0.3. This critical failure surface passes through the zones with minimum local FOSs in the clayey silt layer beneath the starter dam (i.e. zones 23 and 24). For the purpose of comparison, local FOSs for the zones along the critical failure surface from LEM were estimated. FOS ranged between 1.25 and 1.6, which are greater than those of zones 23 and 24. Hence, it can be postulated that there is a high chance of undrained failure that can be initiated from the zones below the toe of the tailings dam (i.e. zones whose FOSs are less than 1.2; Canadian Dam Association) if the tailings dam is further raised.



(a)



(b)

Figure 6.10. Variation of FOS with time: (a) Zone 23; (b) Zone 24

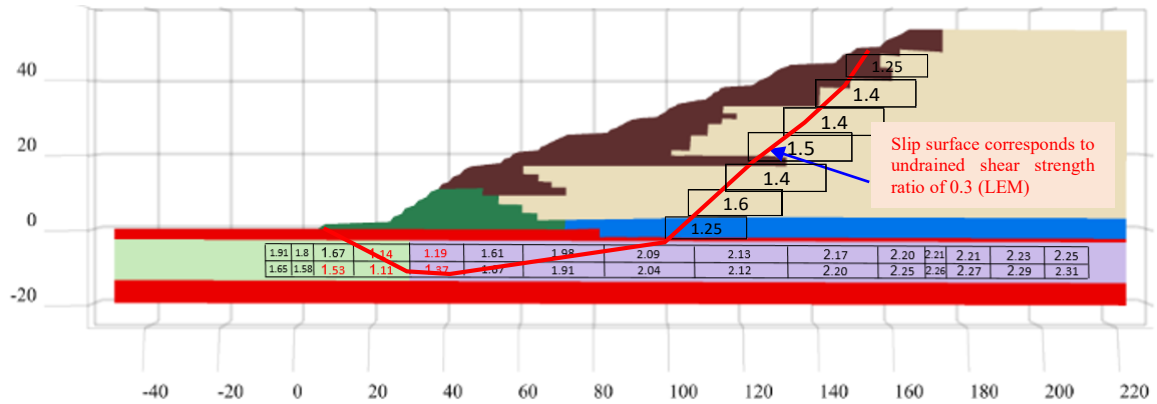
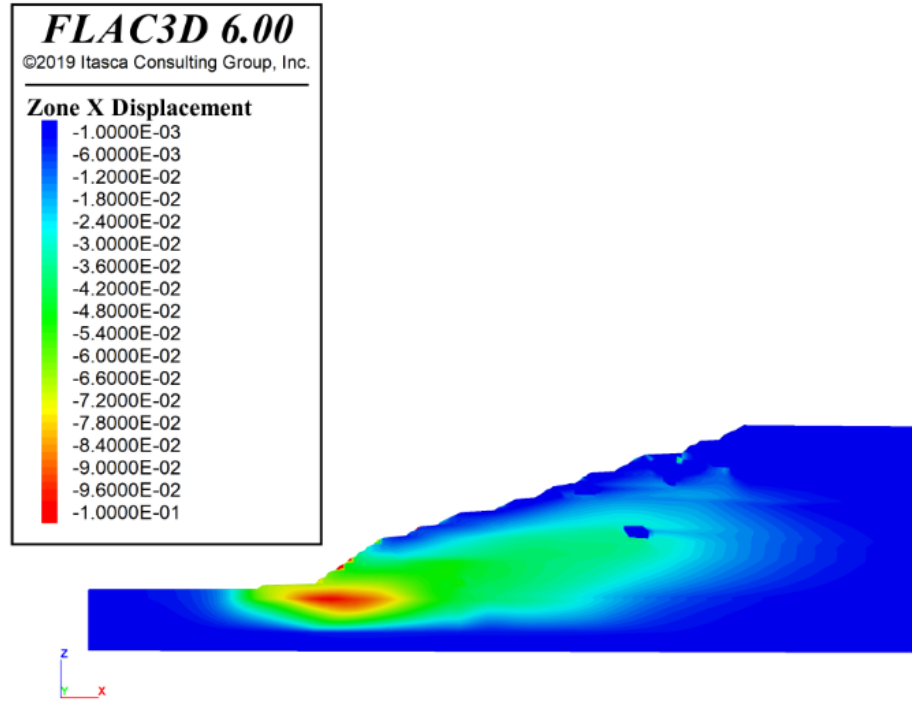


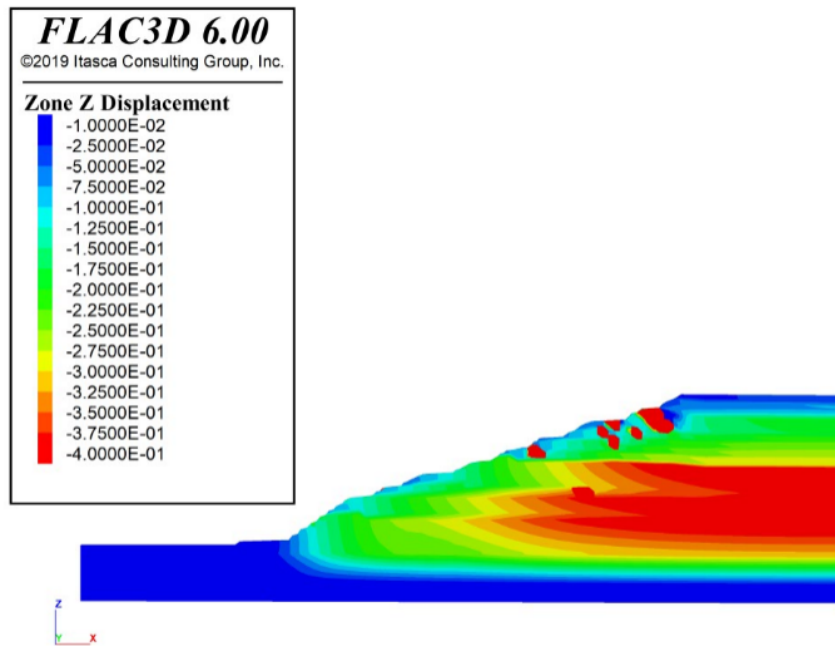
Figure 6.11. FOSs in different zones right after the construction in the year 2018

6.3 Estimation of Displacements

The horizontal and vertical displacements estimated from numerical analyses for the year 2018 are illustrated in Figure 6.12(a) and (b), respectively. The maximum horizontal displacement of approximately 0.09 m was estimated in the foundation beneath the starter dam. This is the location of the zone with minimum FOS. The maximum vertical displacement due to consolidation in the clayey silt layer was estimated to be less than 0.3 m. This vertical displacement was prevailed at the upper part of the clayey silt layer. Only 0.04 m of vertical settlement was estimated at the bottom of the foundation. The estimated displacements within the tailings are not reliable since M-C model used for tailings does not capture the contractive behavior under staged construction. It should be noted that the vertical displacements in Figure 6.12(b) show the compression only (i.e. negative vertical displacement). Heaving (positive vertical displacement) in the vicinity of toe of the tailings dam was negligible as shown in Figure 6.13.



(a)



(b)

Figure 6.12. Deformations in the foundation and within the tailings dam: (a) horizontal displacement; (b) vertical displacement

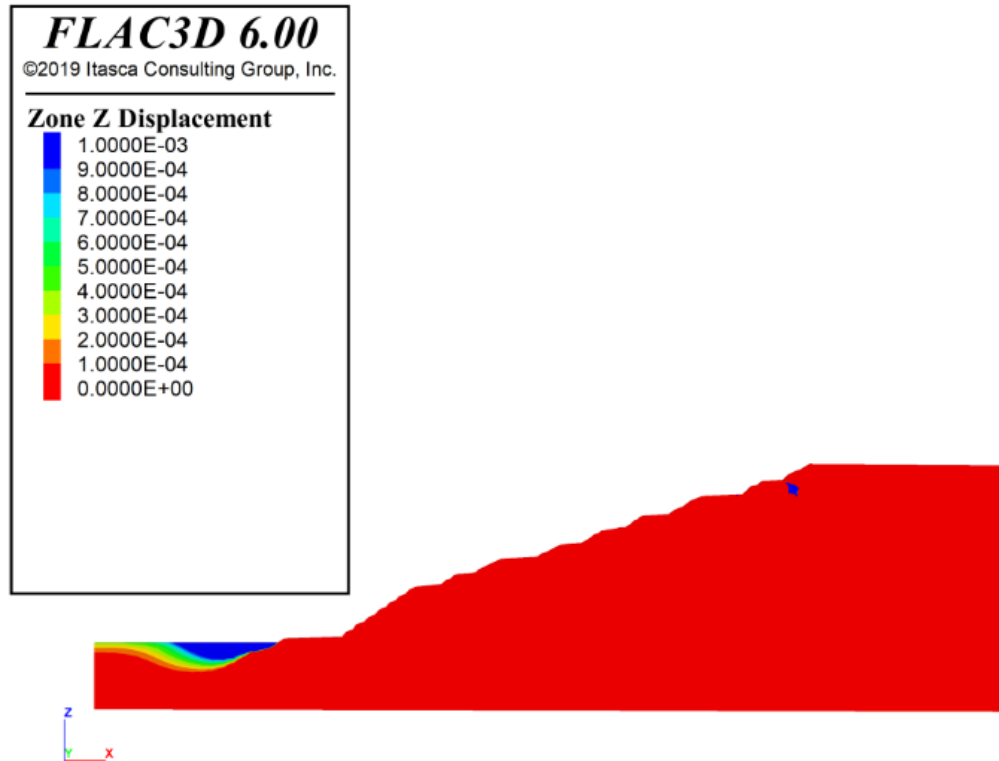


Figure 6.13. Heaving (i.e. positive vertical displacement) in the vicinity of the toe of tailings dam

Additional analyses were also carried out to investigate the variation of vertical displacement under staged construction along the two horizontal lines at the elevations of -6.45 m and -12 m (Figure 6.14).

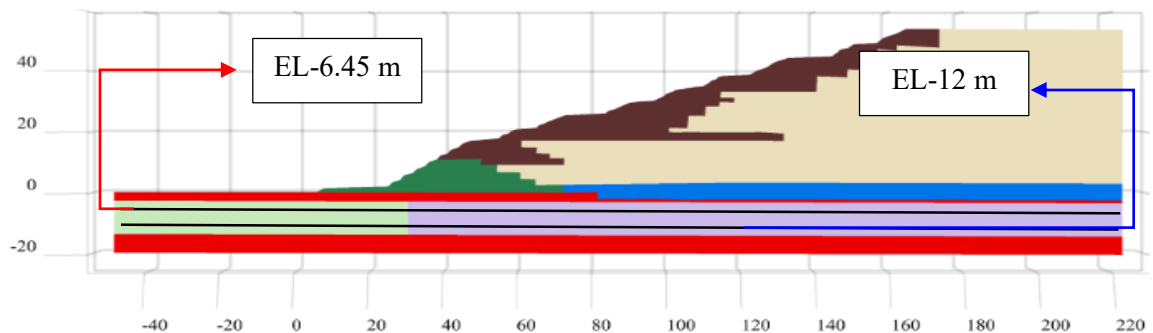
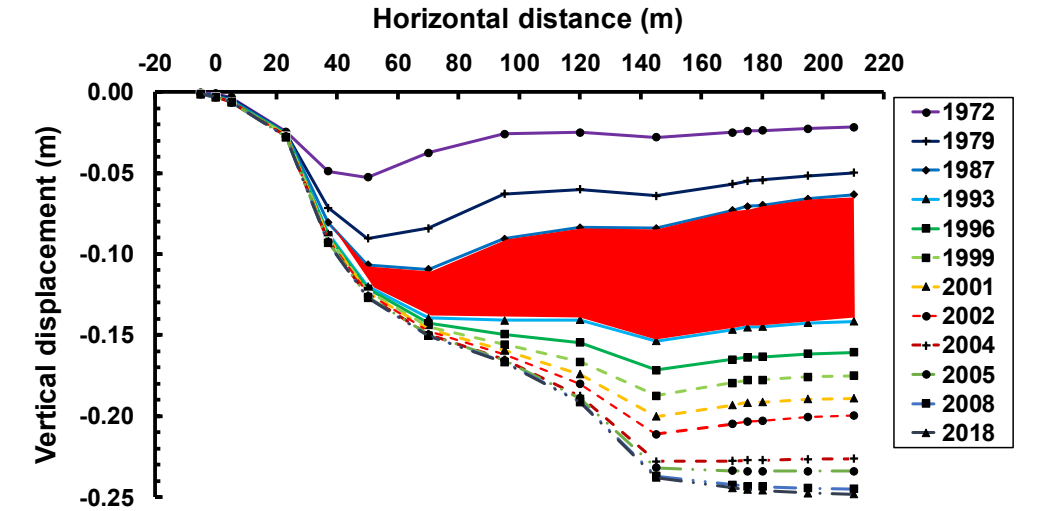
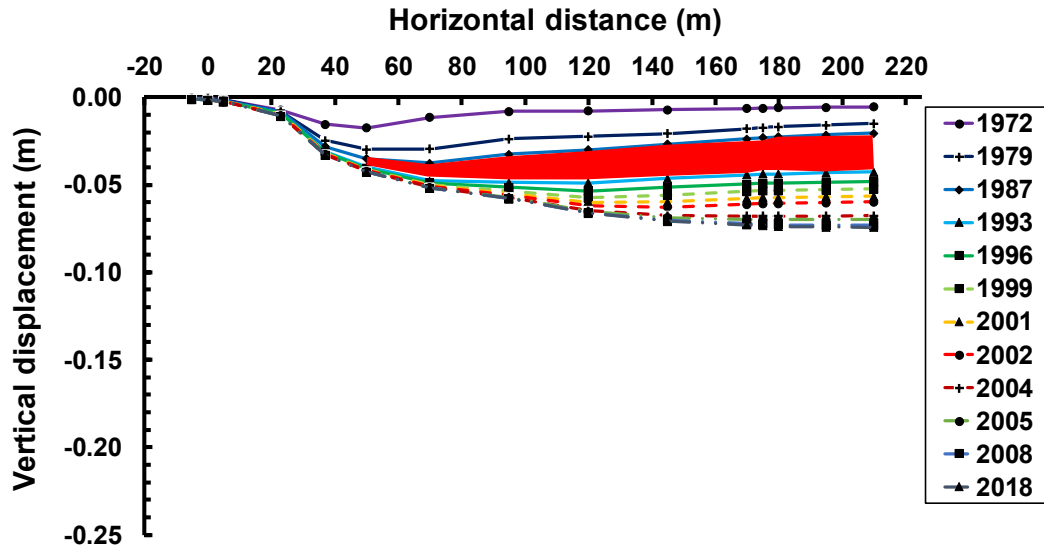


Figure 6.14. Two elevations where the variation of vertical displacement under staged construction were estimated

As explained earlier, most zones in the foundation entered NC state after 1993. This caused a sudden increase in vertical displacement between 1987 and 1993 as shown in Figure 6.15.



(a)



(b)

Figure 6.15. Variation of vertical displacement in the foundation with time: (a) EL-6.45; (b) EL-12m

6.4 Sensitivity Analysis

A sensitivity analysis was performed to investigate the influence of parameters on FOS right after the deposition of tailings in the year 2018. For this, 8 parameters were chosen based on engineering judgment, including 6 parameters for the clayey silty layer (initial specific volume, N ; maximum bulk modulus, K'_{max} , slope of normal consolidation line, λ ; pre-consolidation pressure, P'_c ; effective friction angle, ϕ' ; unit weight, γ_{cs}), elastic modulus (E) of the sand/gravel layer and unit weight of tailings ($\gamma_{tailings}$). The influence of each parameter was evaluated in percentage (out of 100) for the 30 zones, shown in Table 6-2. For example, the most influential parameter for Zone 1 was determined to be ϕ' of the clayey silt layer (i.e. 33%). λ of the clayey silt layer and E of sand and gravel layer ranked the second (i.e. 25%) and the third (i.e. 19%) most influential parameters, respectively. For zones 2 through 22, λ of the clayey silt layer was the most influential parameter (i.e. over 80%) with negligible effect by other parameters. In contrast, ϕ' of the clayey silt layer was the most influential parameter for the zones 23 through 30; instead, the influence of λ of the clayey silt layer was not significant. The sensitivity analysis results clearly showed that stability of the tailings dam is governed by λ and ϕ' for the NC (beneath a tailings dam) and OC (below the toe) clayey silt layer, respectively. In other words, the most influential parameter on the stability of the zones at the toe is the slope of critical state line (M) that is a function of ϕ' .

Table 6-2. Sensitivity analysis results

Zone	Parameters							
	N	K'_{max}	E	λ	$P'c$	ϕ'	γ_{CL}	$\gamma_{tailings}$
	Influence on FOS (out of 100)							
1	4	0	19	25	7	33	0	11
2	4	0	0	85	0	7	0	5
3	4	0	0	84	0	7	0	5
4	5	0	0	83	0	6	0	6
5	5	0	0	83	0	6	0	6
6	4	0	0	83	0	6	0	7
7	5	0	0	82	0	6	0	7
8	4	0	0	83	0	6	0	7
9	3	0	0	83	0	6	0	8
10	3	0	0	83	0	5	0	8
11	1	0	0	84	0	5	0	10
12	1	0	0	82	0	4	0	13
13	1	0	0	81	0	3	0	15
14	0	0	0	77	0	2	0	21
15	0	0	0	79	0	2	0	19
16	0	0	0	78	0	1	0	20
17	0	0	0	71	1	1	0	27
18	1	0	0	69	1	0	0	29
19	1	0	0	82	2	0	0	15
20	1	0	0	84	2	0	0	13
21	5	0	0	82	5	1	0	8
22	8	0	0	81	5	2	0	4
23	19	0	0	26	5	26	0	24
24	14	0	0	9	5	66	0	6
25	1	0	0	16	2	78	0	3
26	2	0	0	2	1	91	0	5
27	6	0	0	4	0	86	0	3
28	2	0	0	1	0	94	0	3
29	5	0	0	2	1	86	0	6
30	0	0	0	6	1	92	0	1

7. SUMMARY AND CONCLUSIONS

In this research, the stability analysis of EVO tailings dam against undrained failure is investigated, which was constructed on a clayey silt layer. For this, finite difference software, FLAC3D was used to estimate the behaviors of clayey silt layer under the staged loading. It was assumed that i) the clayey silt layer follows contractive behavior and ii) the excess pore-water pressure in the clayey silt layer due to the instantaneous each staged loading is negligible. The clayey silt layer was modeled with the Modified Cam-Clay (MCC) constitutive model, which was discretized into 30 different zones. The staged construction was simulated by raising the tailings dam in 12 consecutive stages assuming plane strain condition, following raise construction history. The factor of safety was estimated based on the slope of effective stress path and the slope of critical state line for each zone. Major observations from this research are as follow:

1. The effective stress path can be effectively used to judge at which point the state of the clayey silt layer changed from overconsolidated condition to normally consolidation condition. According to the numerical analysis, the clayey silt layer beneath the tailings dam has entered the normally consolidated condition since 1993. This information can be used to compute the variation of consolidation settlement with time.
2. It turned out that the most vulnerable zones to undrained failure are Zone 23 (FOS = 1.14) and Zone 24 (FOS = 1.11) (i.e. below the toe of tailings dam) that contributed to the deposition of tailings in 2018. The FOSs of these zones were less than 1.2 even at the end of consolidation. In other words, there is a high chance of undrained failure in these two zones if the tailings dam is further raised in the future. Hence, it is strongly

- recommended that the additional raise should be carefully carried out considering the corresponding effective stress path.
3. The potential slip surface from the limit equilibrium method passed through Zone 23 and Zone 24. Hence, it can be postulated that the failure in the tailings dam can be initiated from these two zones.
 4. One of the major parameters that govern undrained failure is an undrained shear strength ratio. Hence, it is recommended that the undrained shear strength ratio be determined through various laboratory and field tests to determine the range for sensitivity analysis. The existence of silt in the foundation led to a dilative behavior at large strains. In this research, the undrained shear strength ratio of 0.24 was used assuming contractive behavior in the clayey silt layer for conservative analysis.
 5. The sensitivity analysis results showed that stability of the tailings dam is governed by λ and ϕ' (or the slope of critical state line, M) for the NC (beneath a tailings dam) and the OC (below the toe) clayey silt layer, respectively.

7.1 Limitations of the Research

In this research, the dilative behavior of the clayey silt layer was not considered. Hence, a more comprehensive constitutive model such as Norsand model should be examined to capture phase transformation during the shearing stage.

As sediments deposited vertically layer by layer, the soil properties in vertical direction are different from those in horizontal direction (i.e. anisotropy). Considering anisotropy in the numerical analysis may lead to the rotation of the yielding curves. This anisotropy was not

considered in the present research; instead, it was assumed that the soil is under isotropic loading condition.

The following assumptions were made in the numerical analysis for the purpose of simplification: i) Raise of the tailings dam and subsequent tailings deposition took place instantaneously at the same time, ii) The tailings dam was raised after the consolidation under the previous stage of loading completed, iii) Shearing in the soil does not occur during consolidation. Such simplification can be overcome by using observational method to update the state of soil during loading and consolidation phases.

The proposed approach was applied to one case study only. To enhance the reliability and applicability of the proposed approach, more case studies need to be done.

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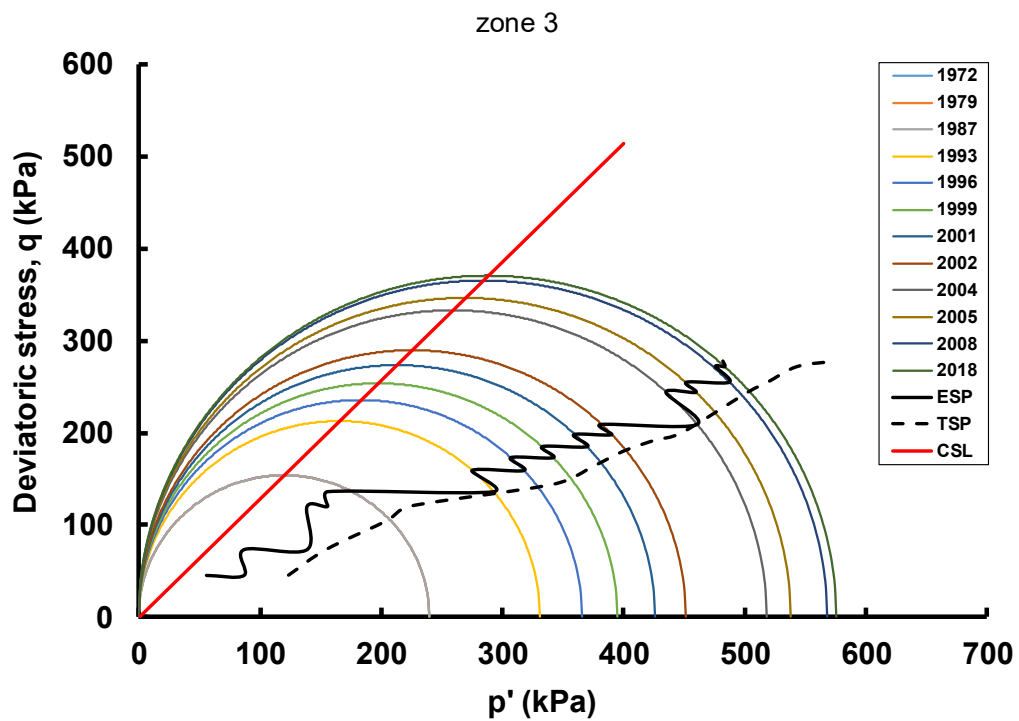
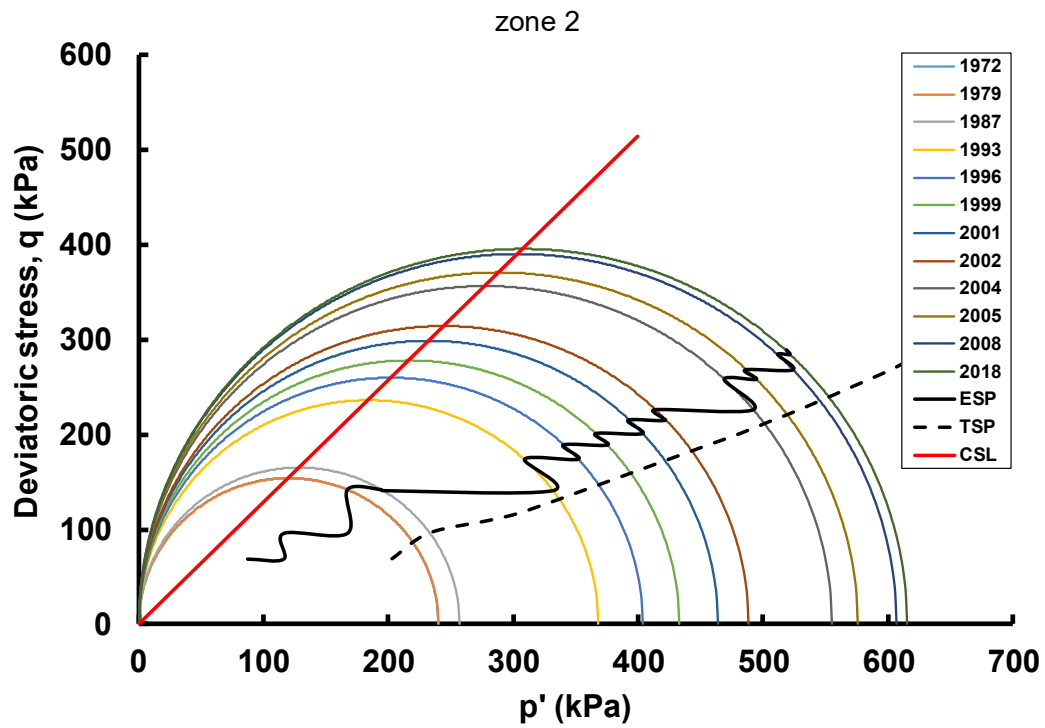
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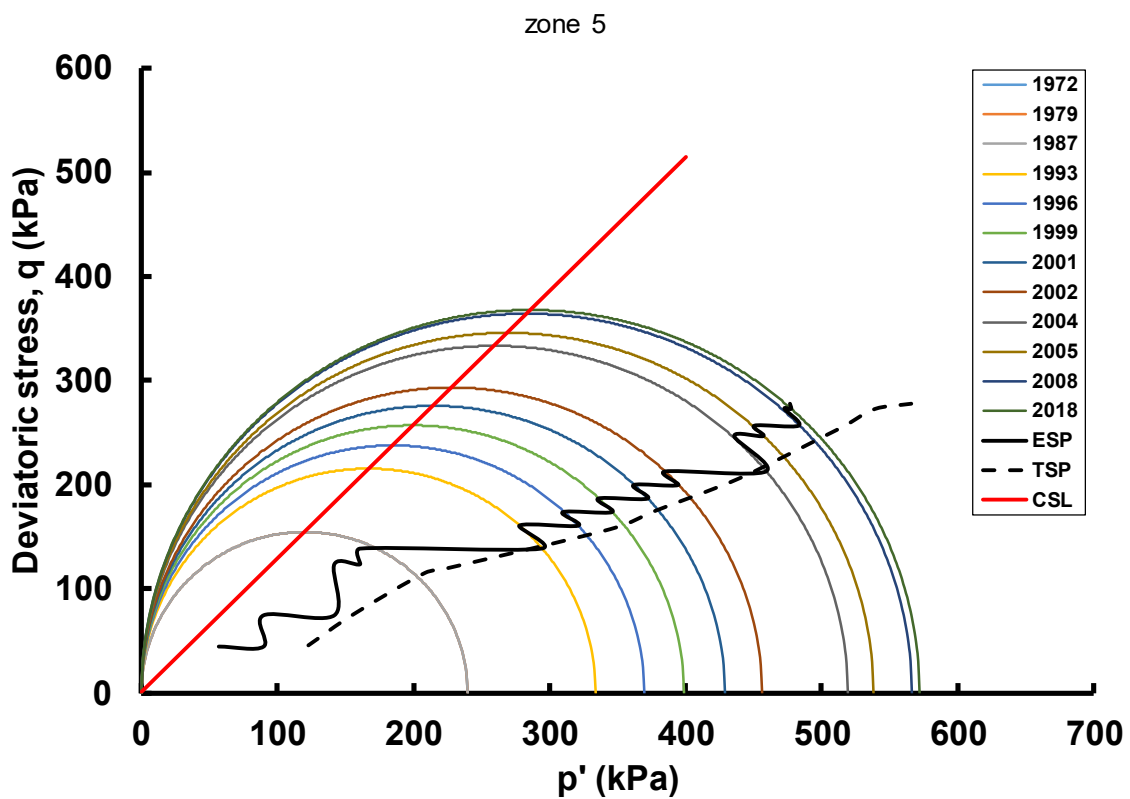
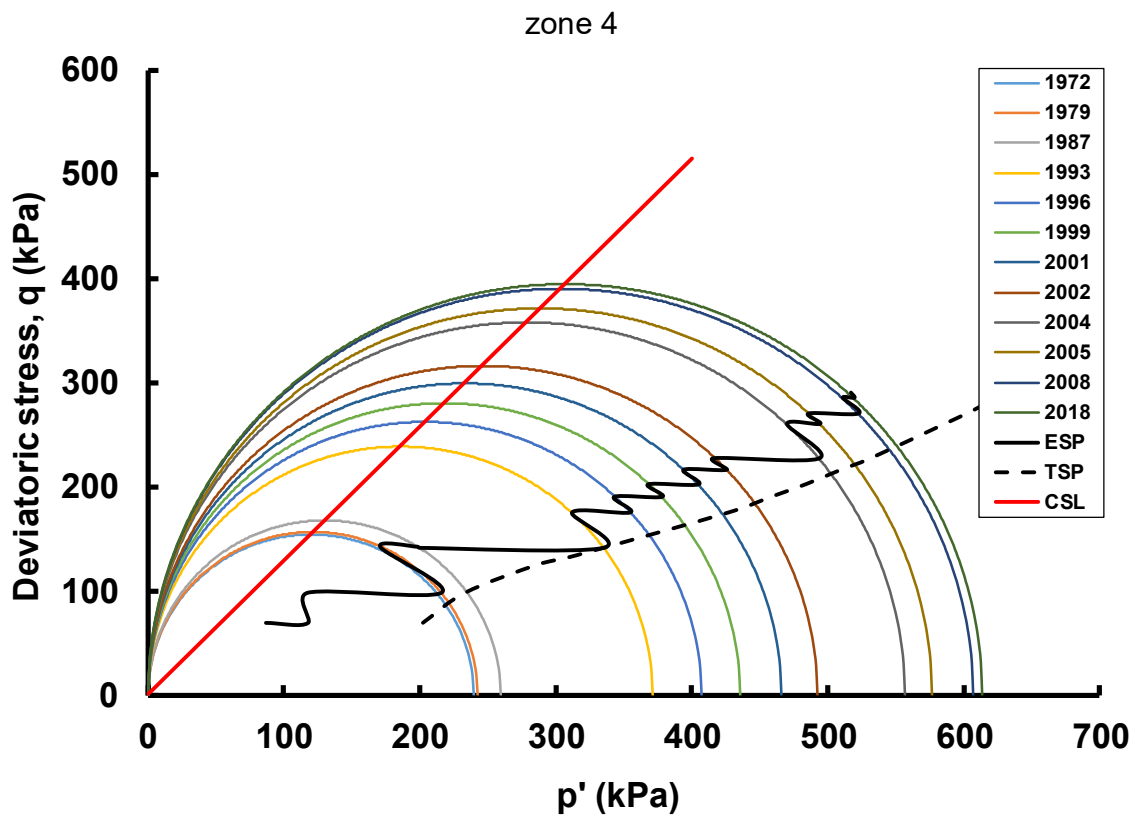
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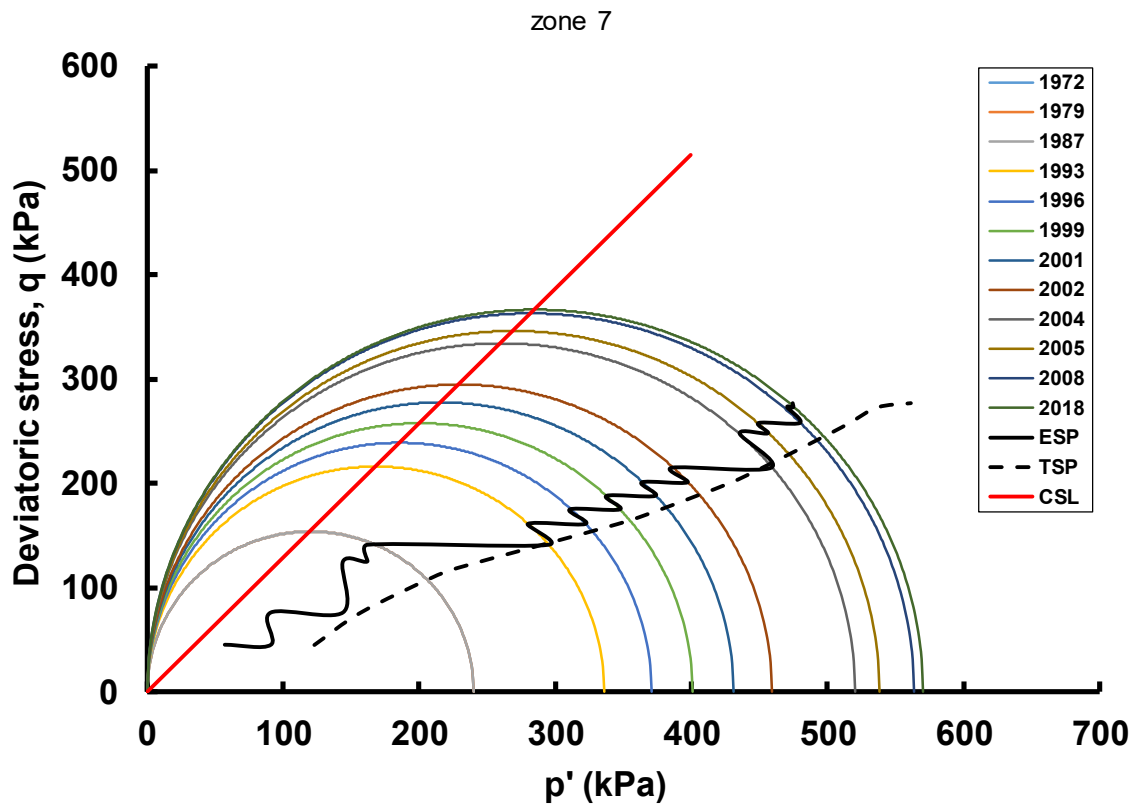
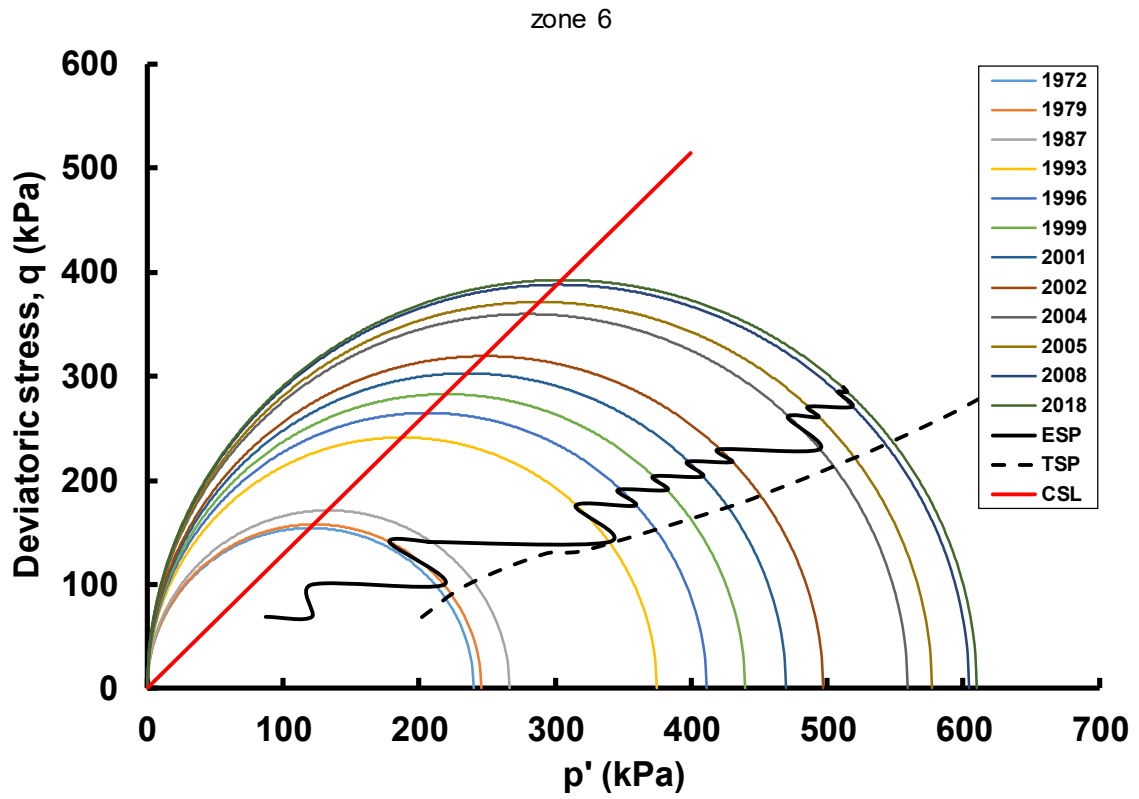
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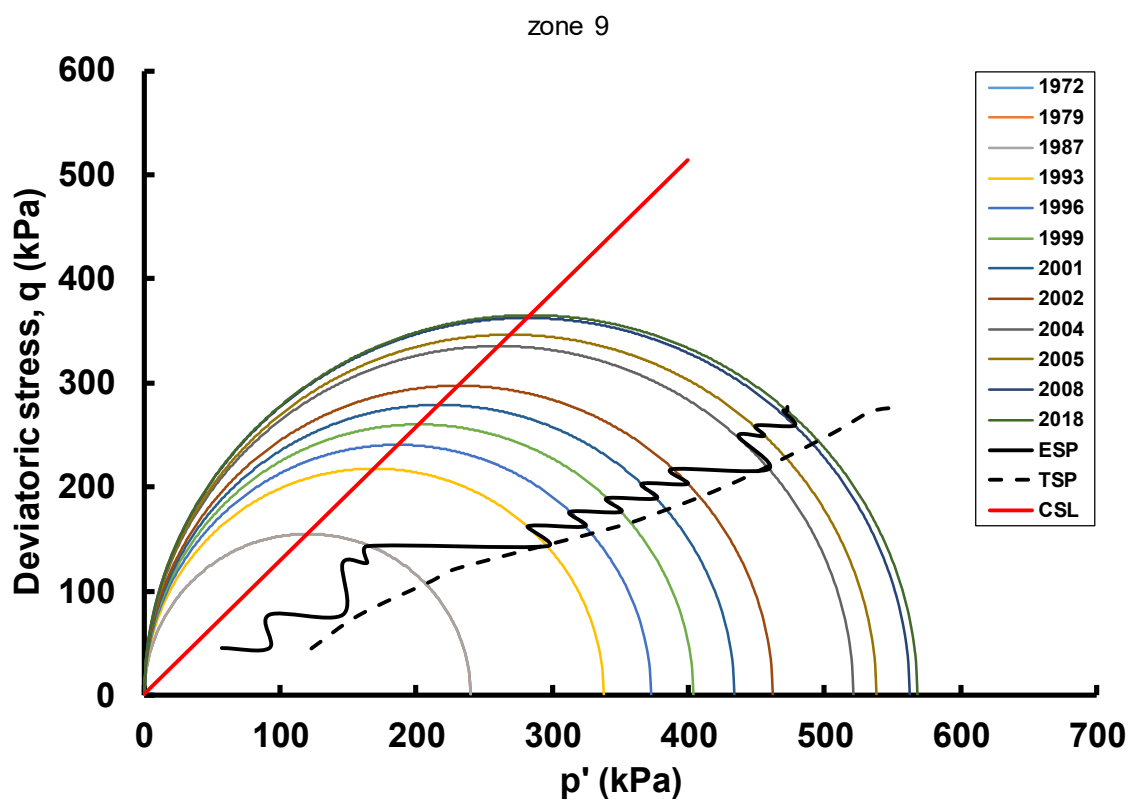
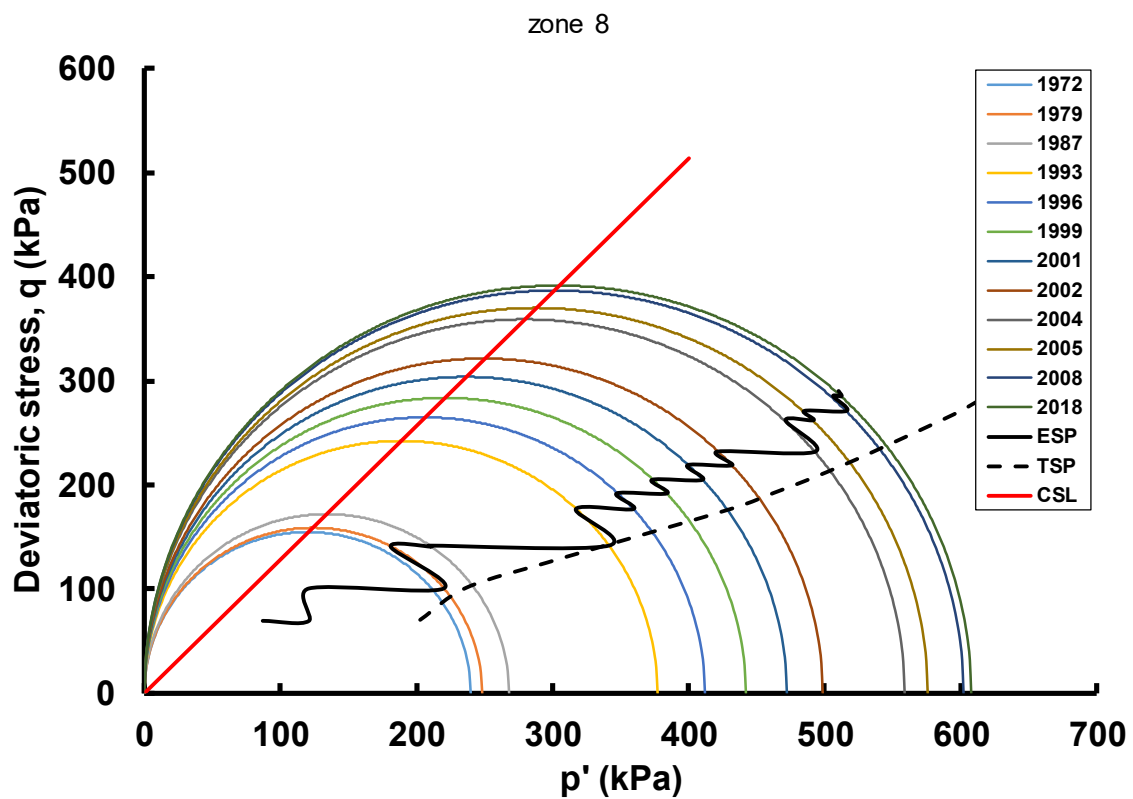
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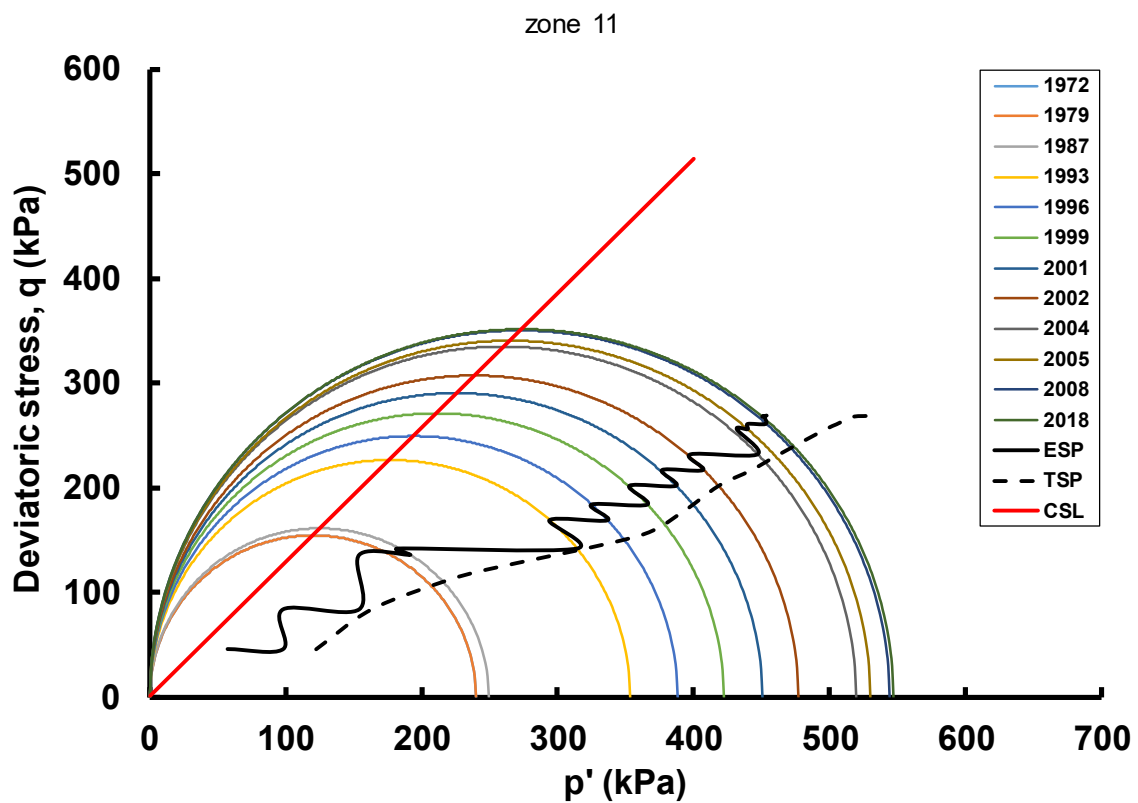
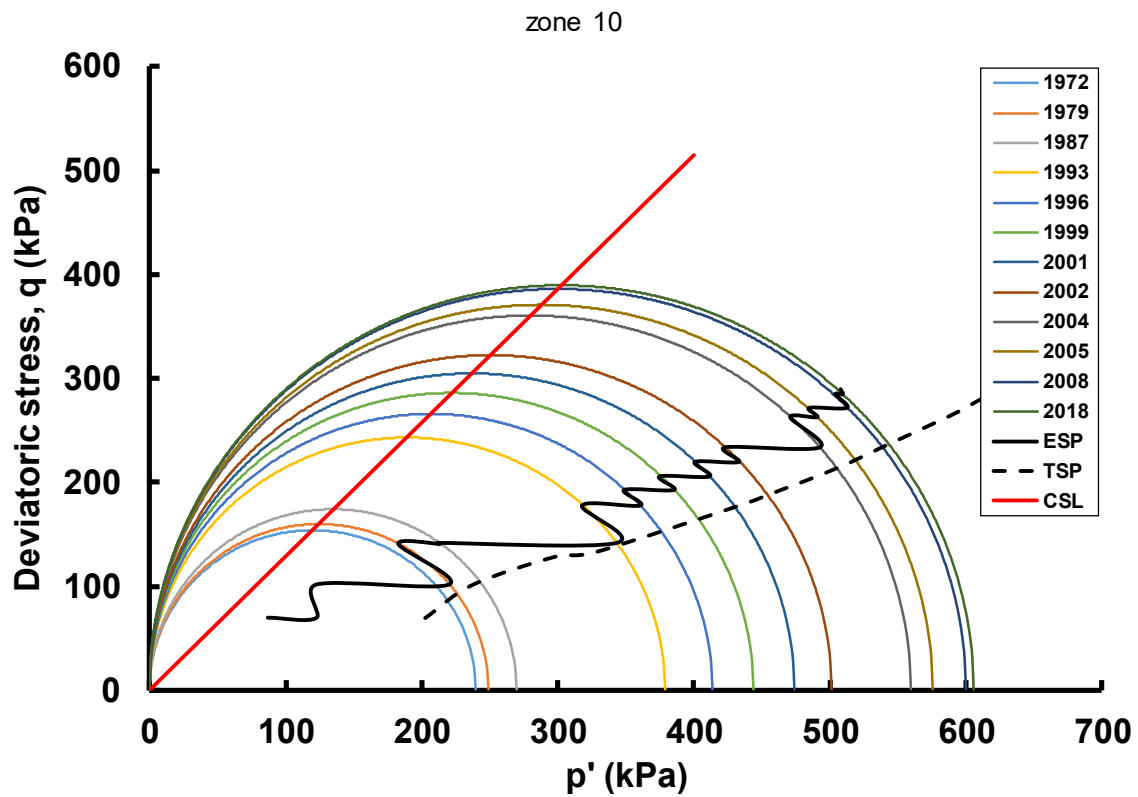
APPENDIX

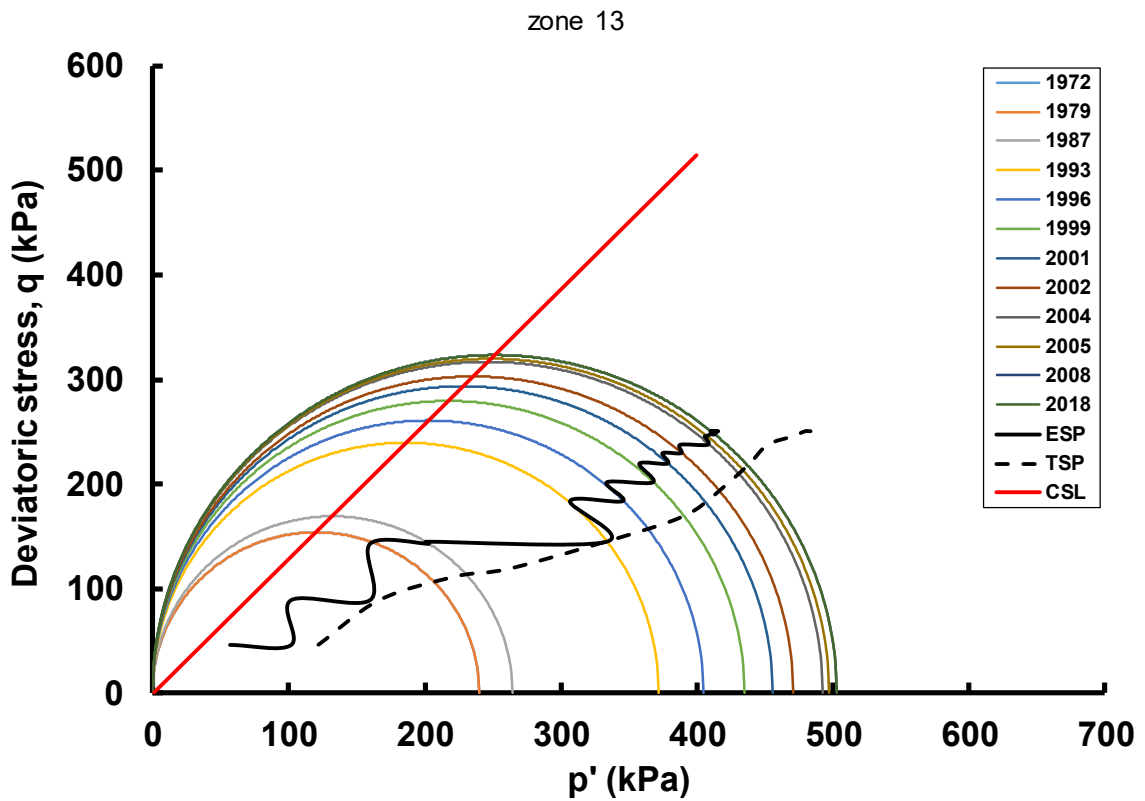
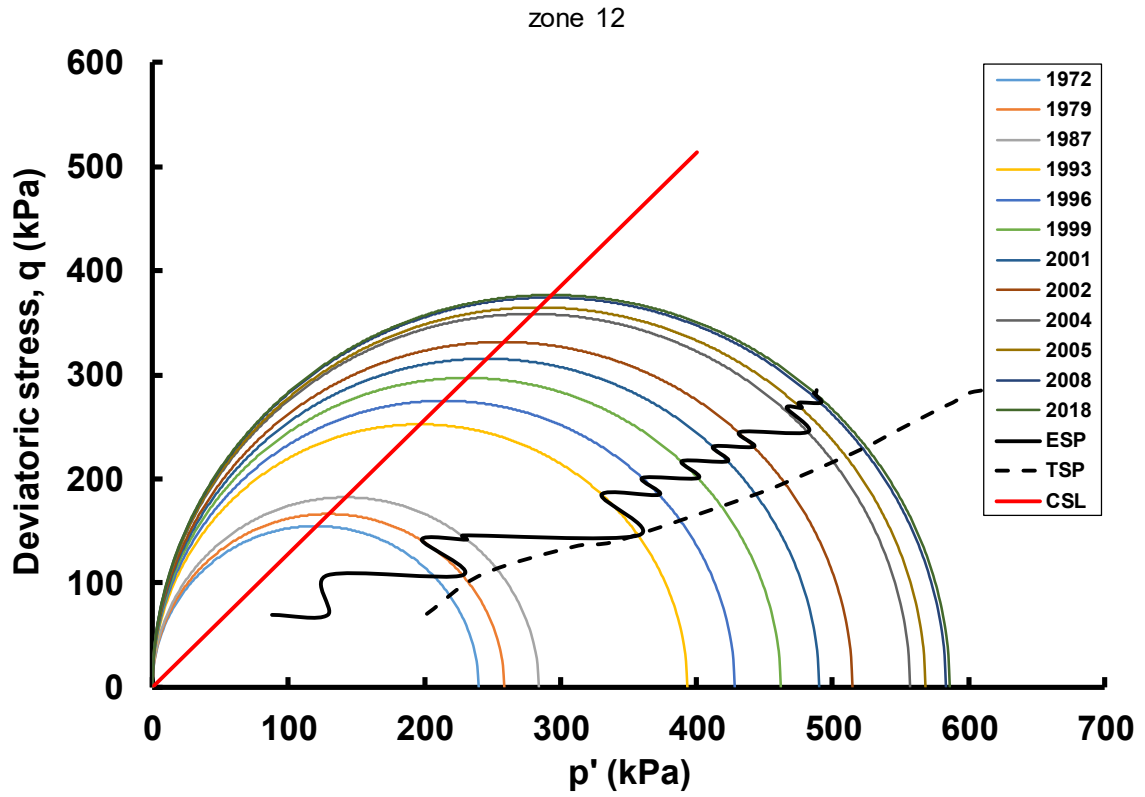


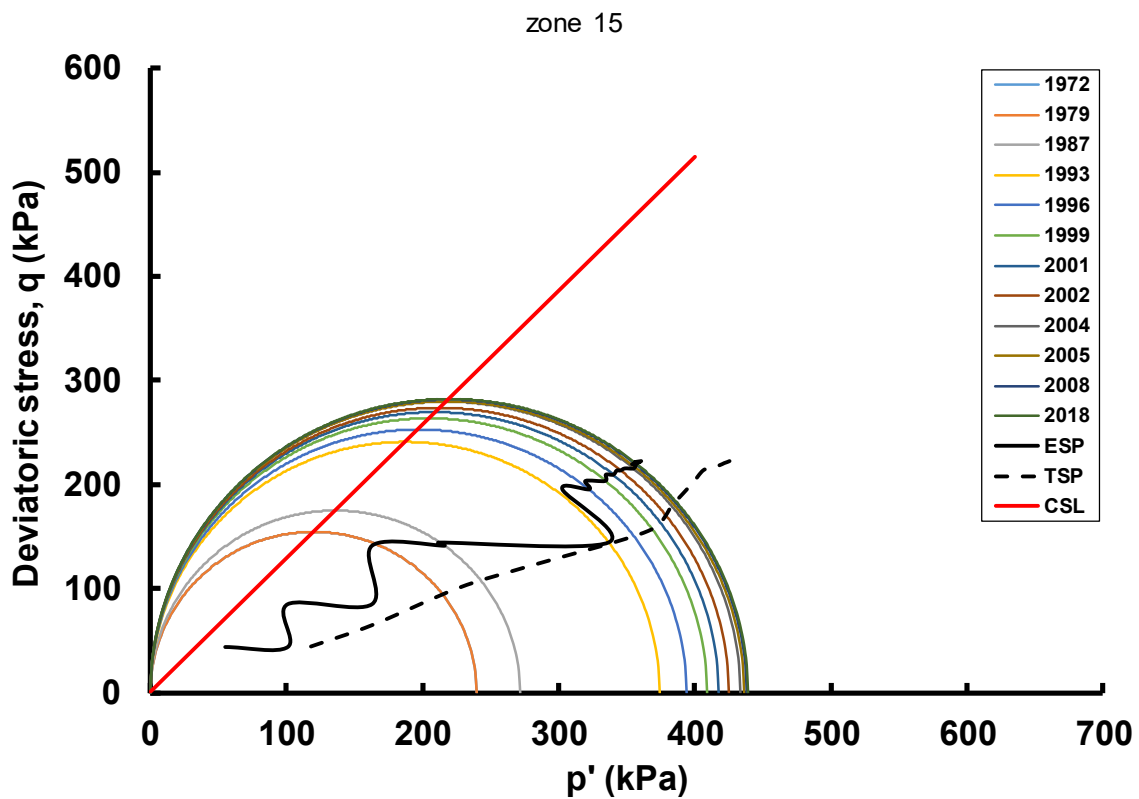
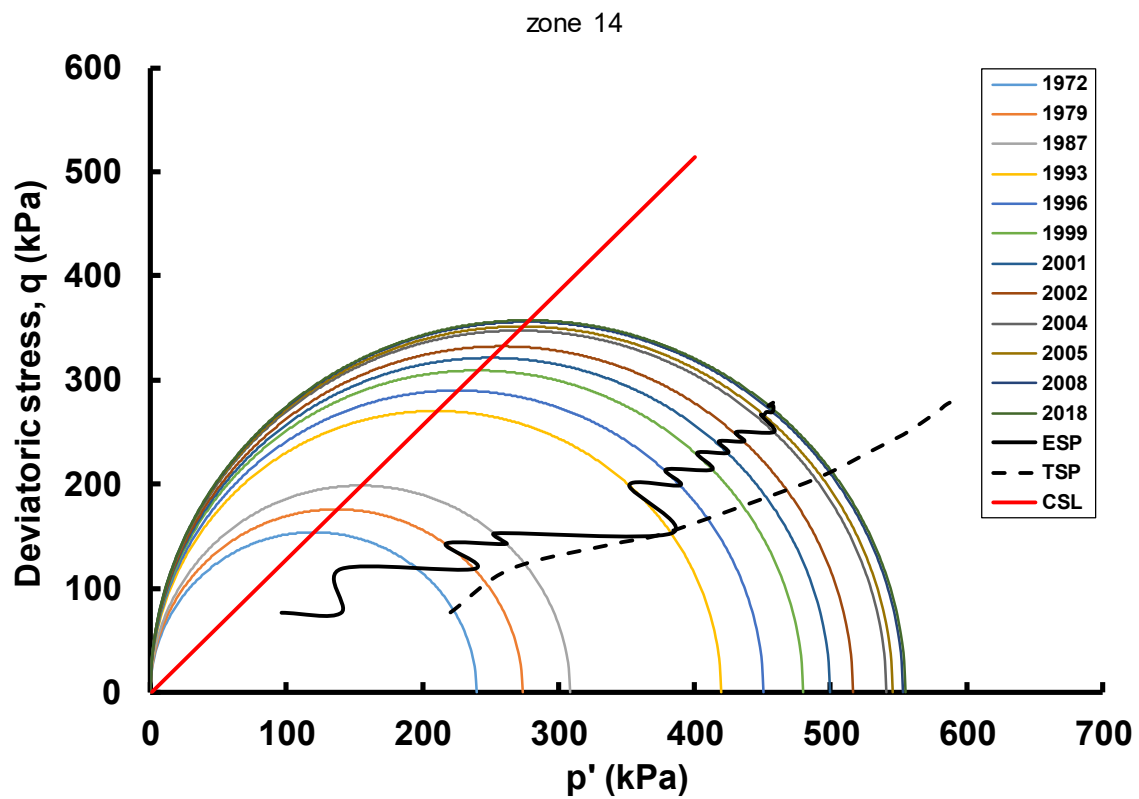


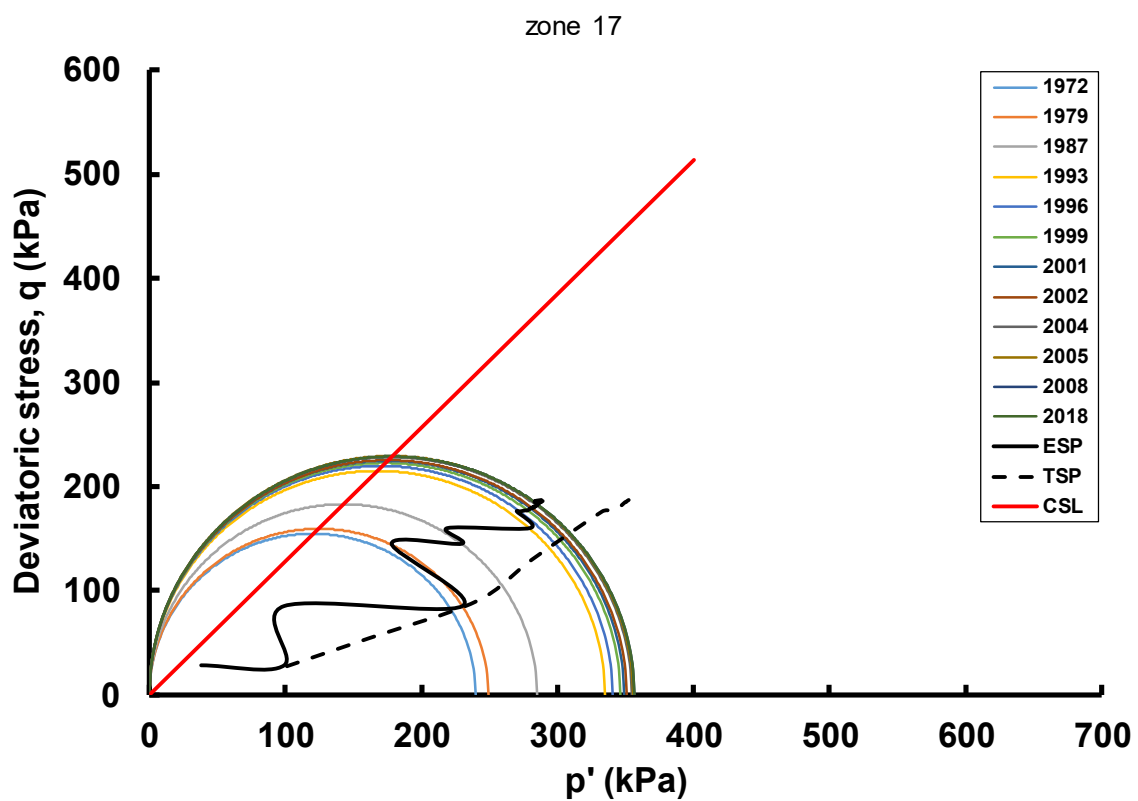
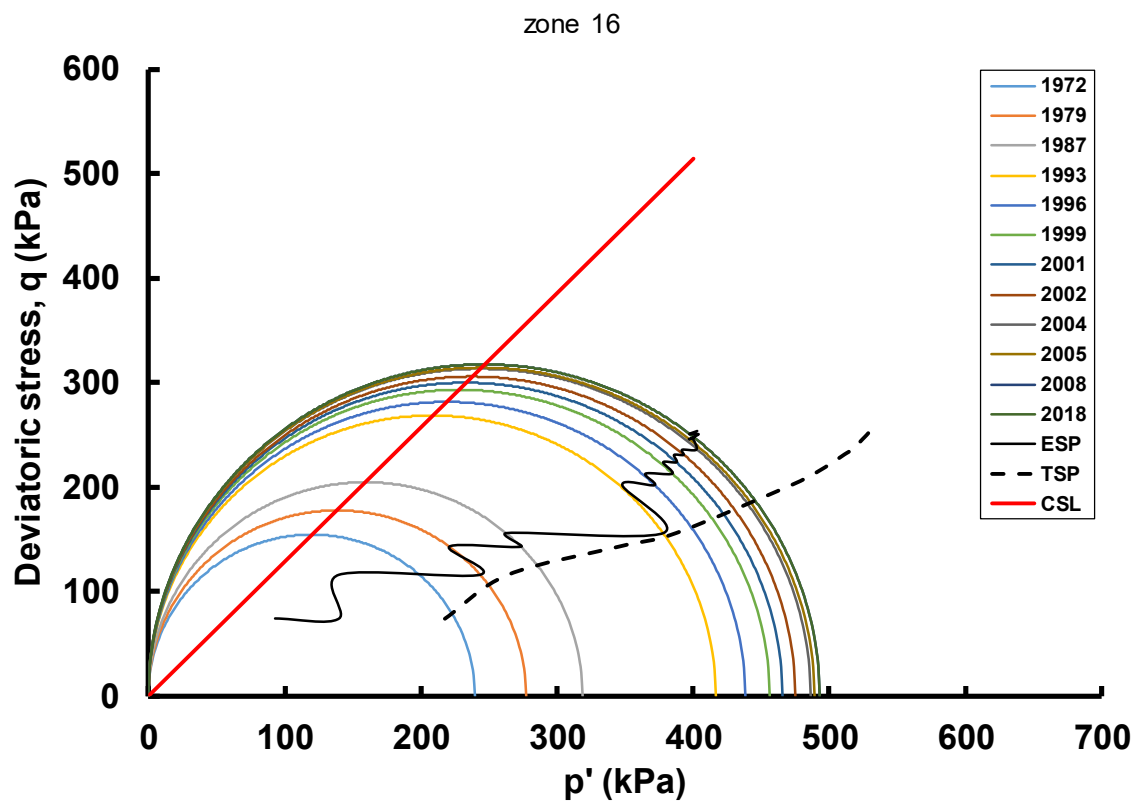


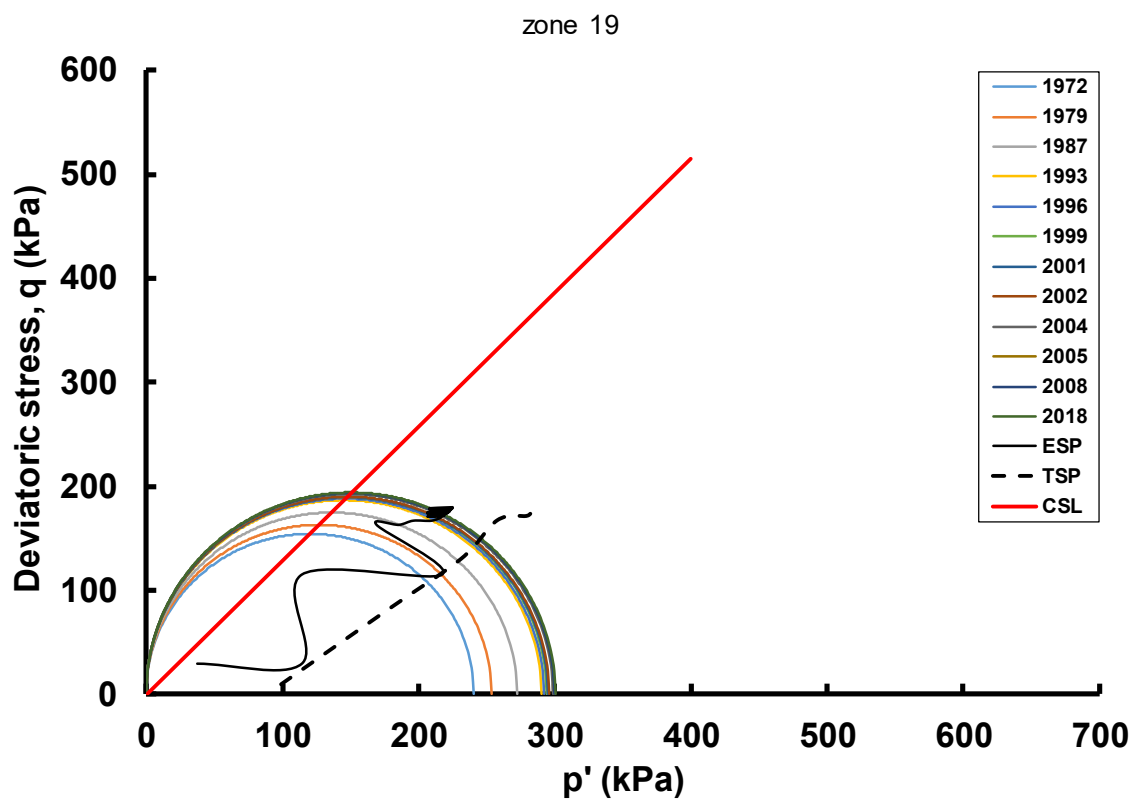
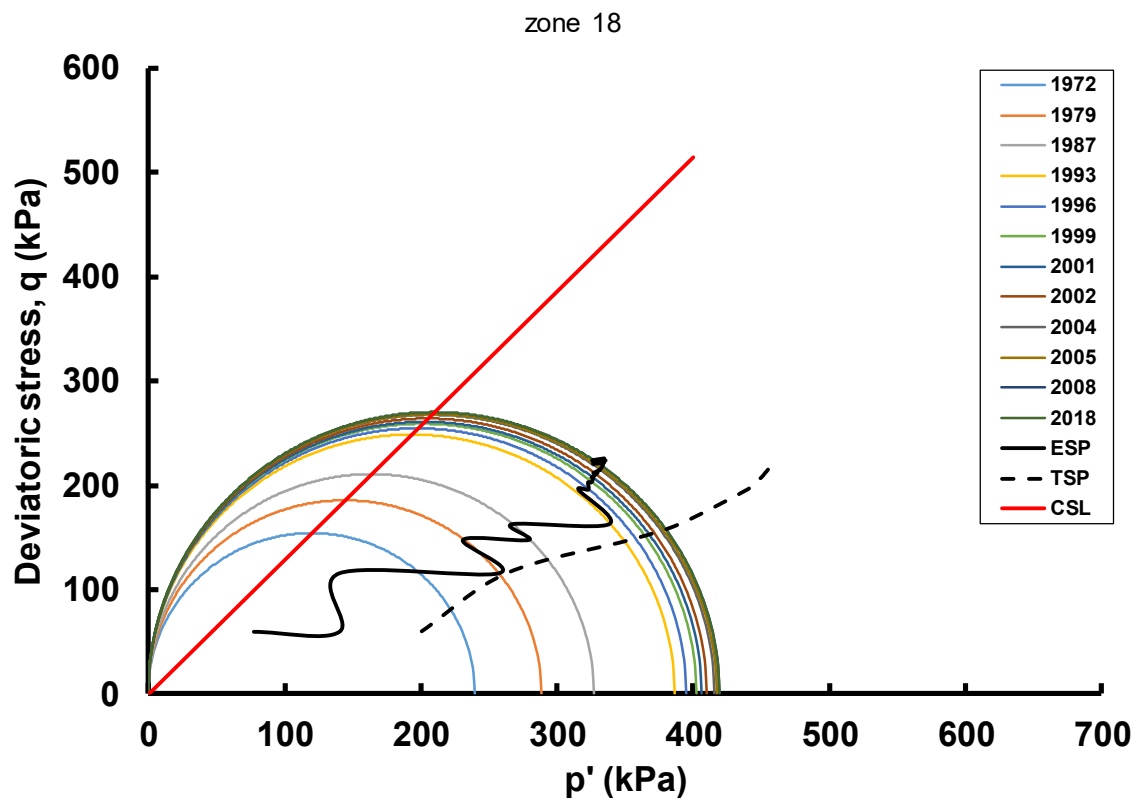


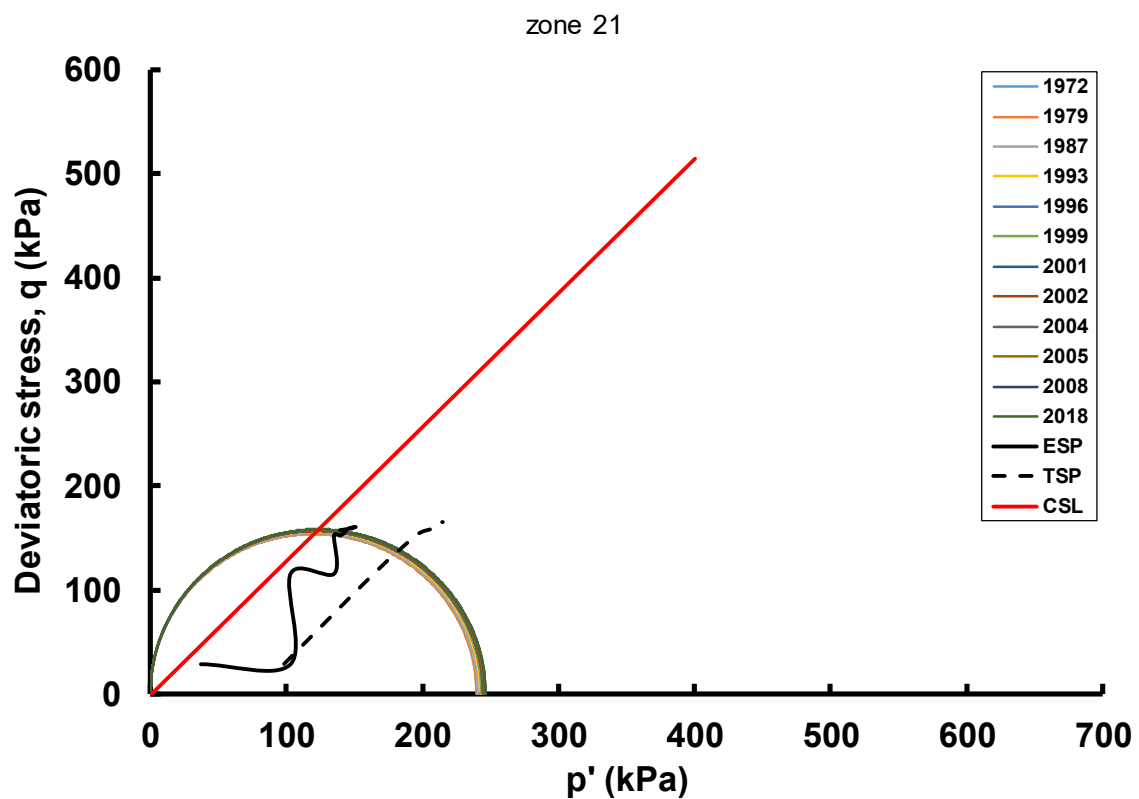
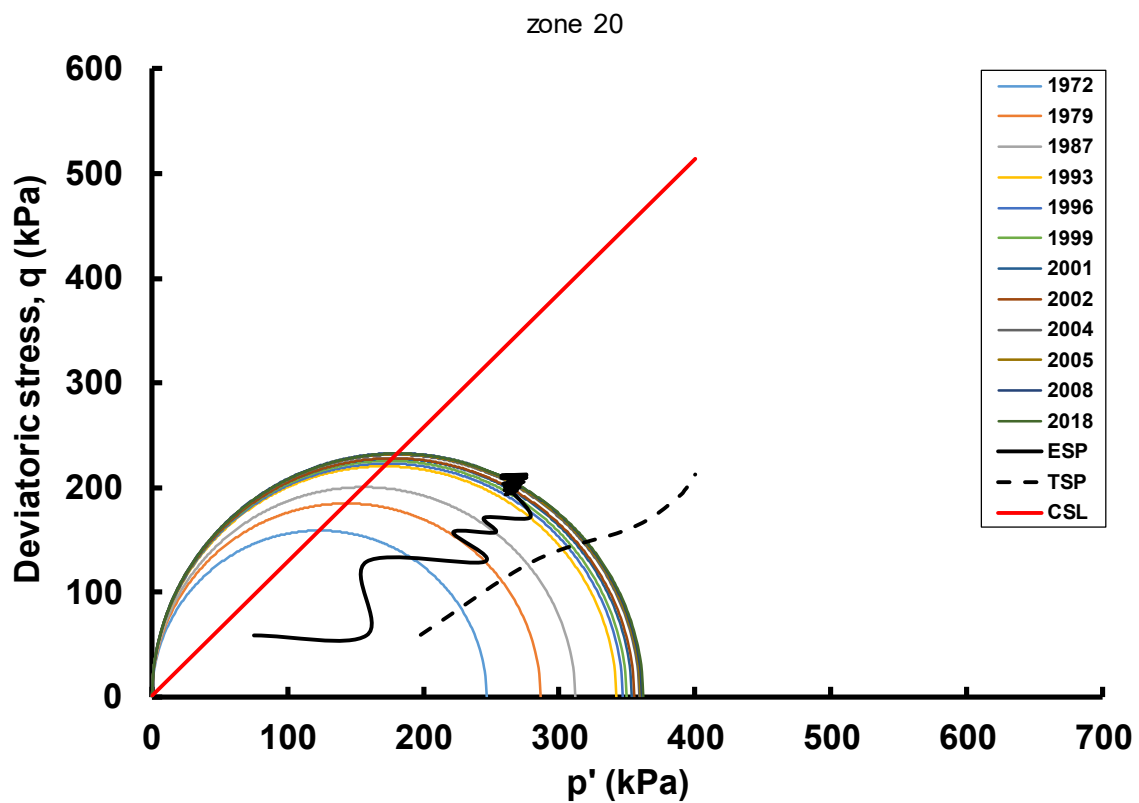


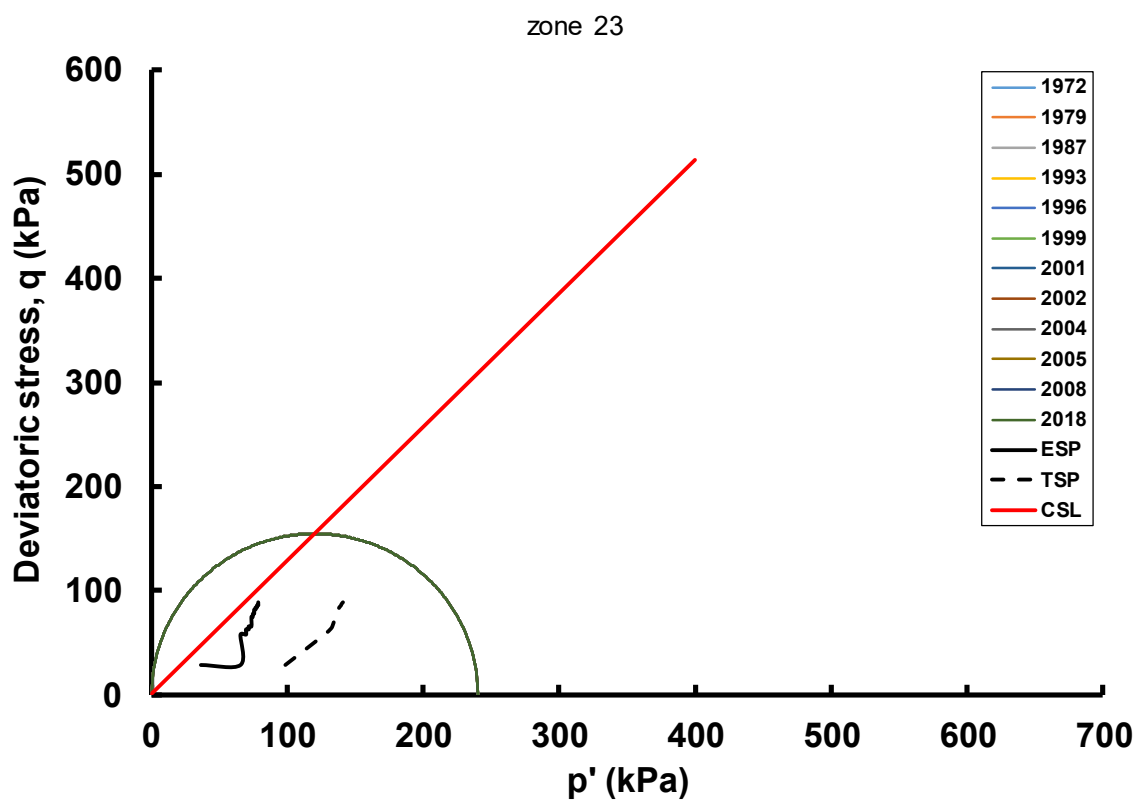
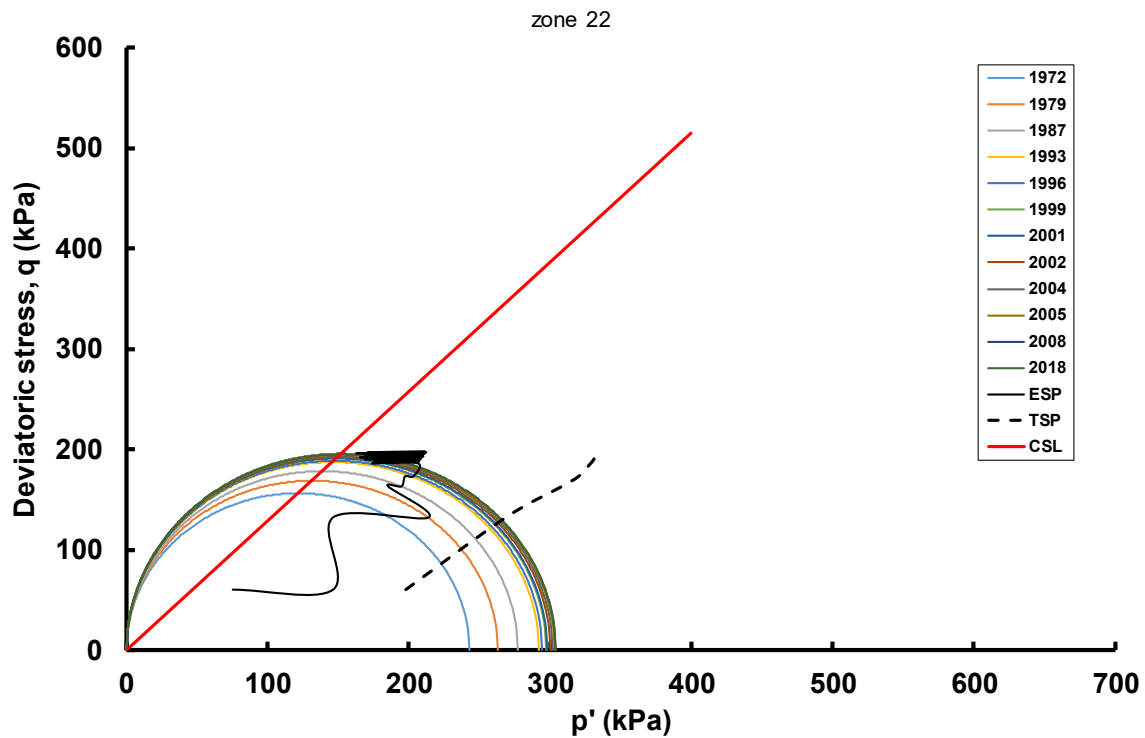


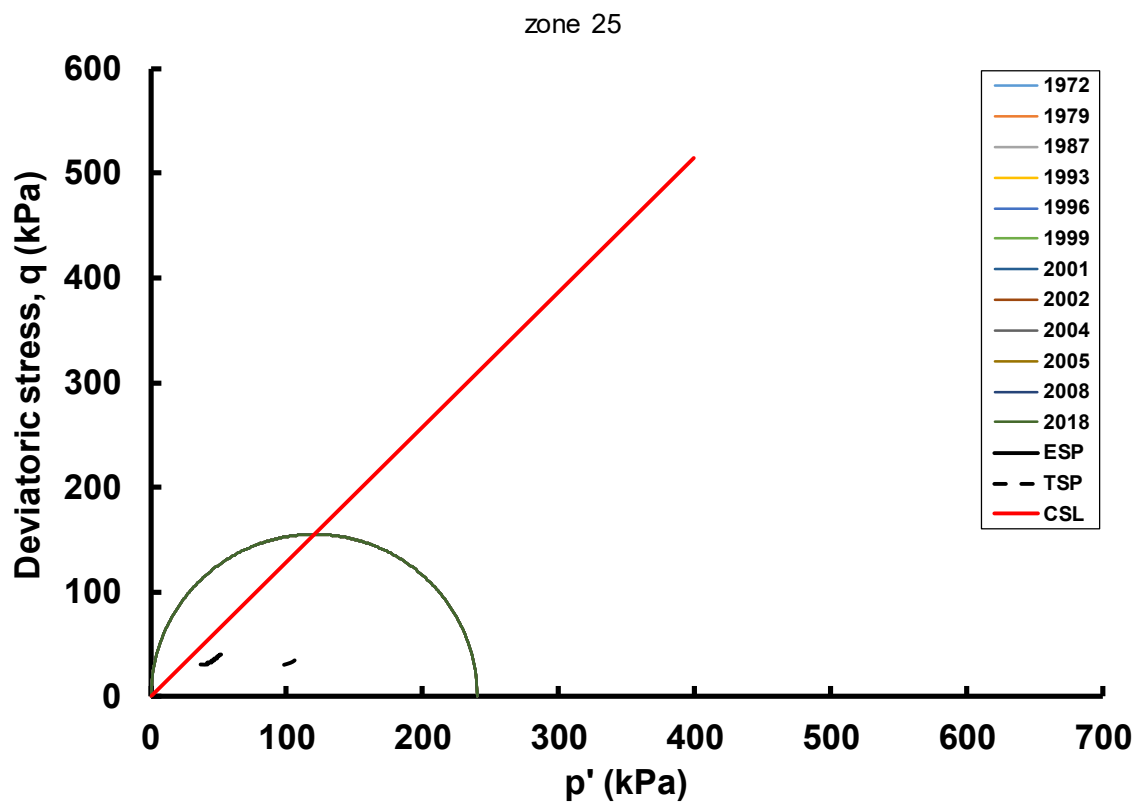
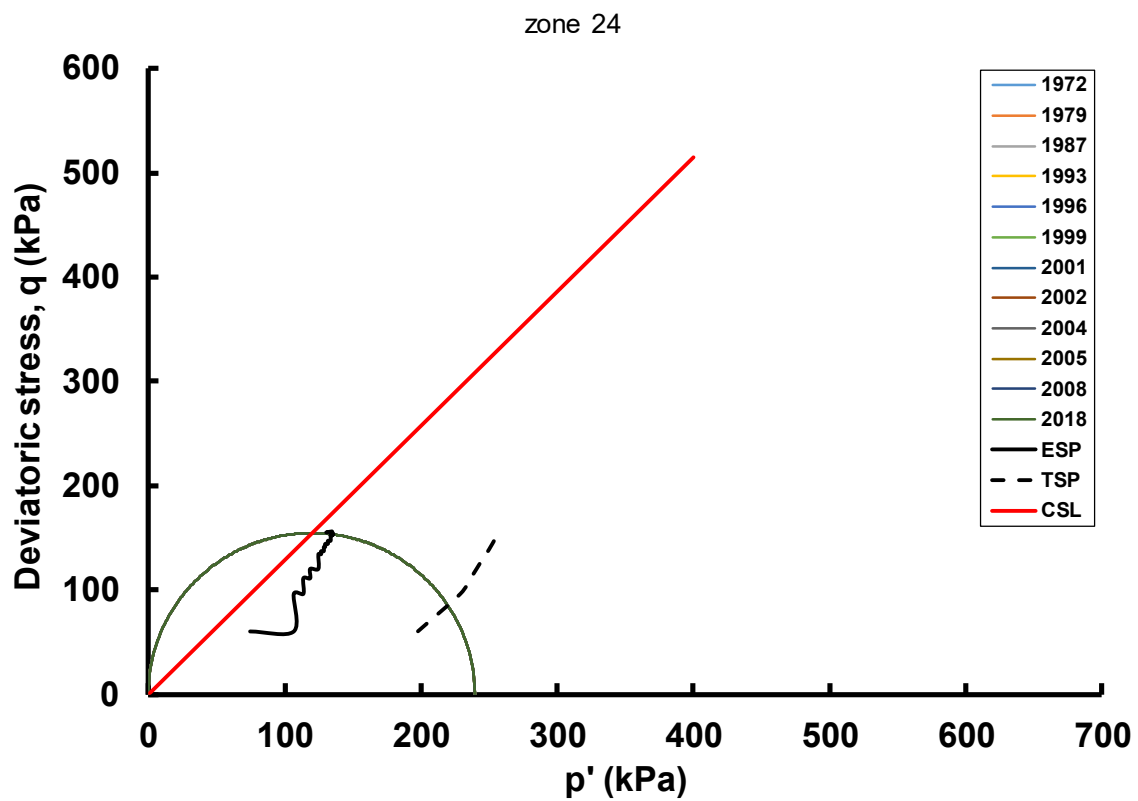


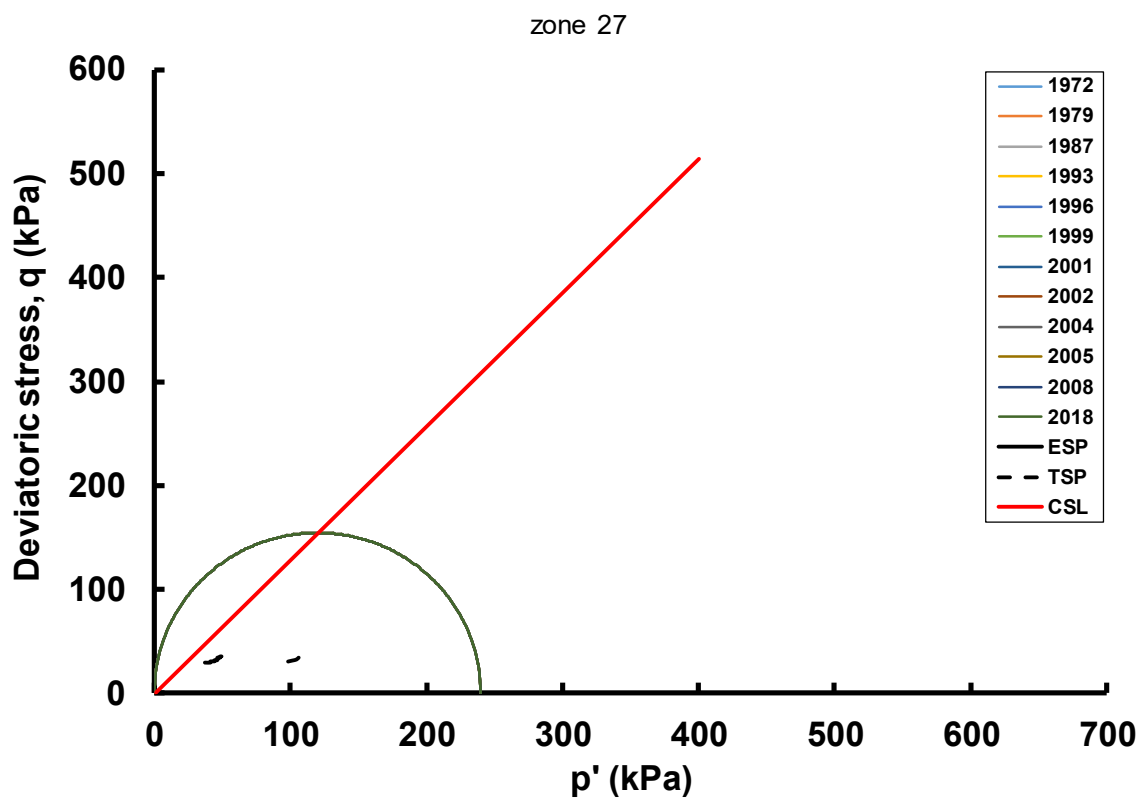
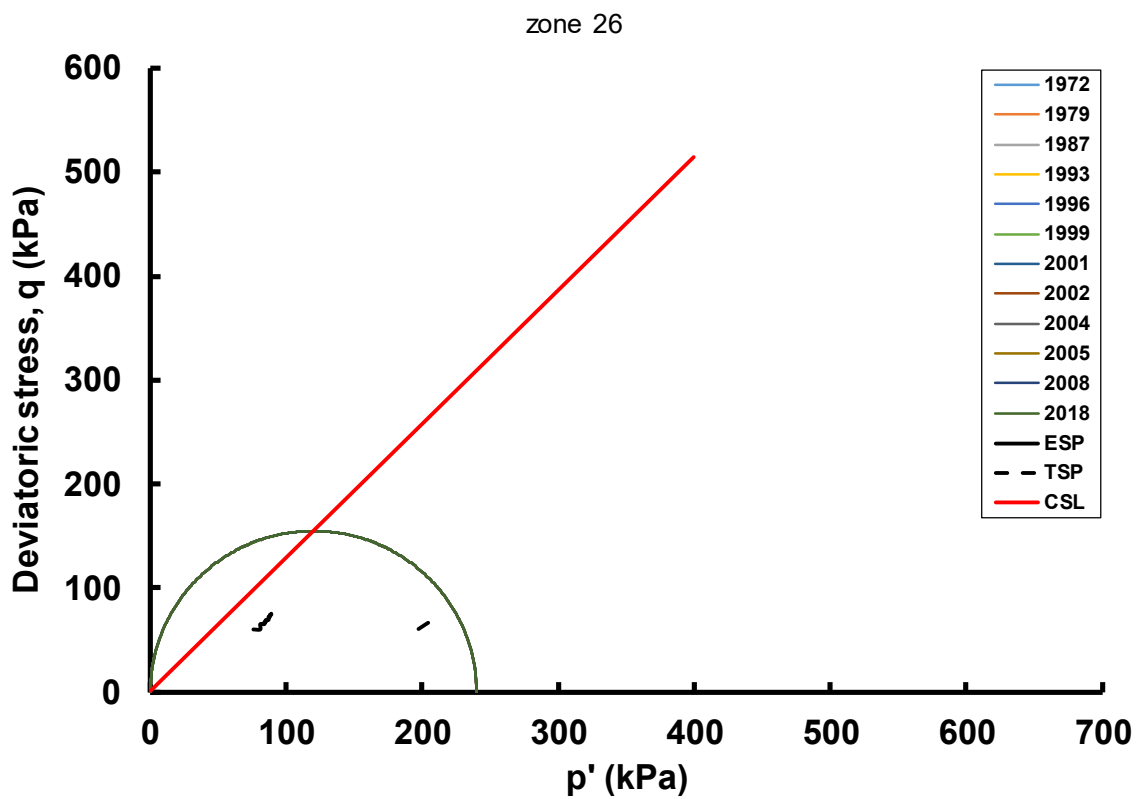


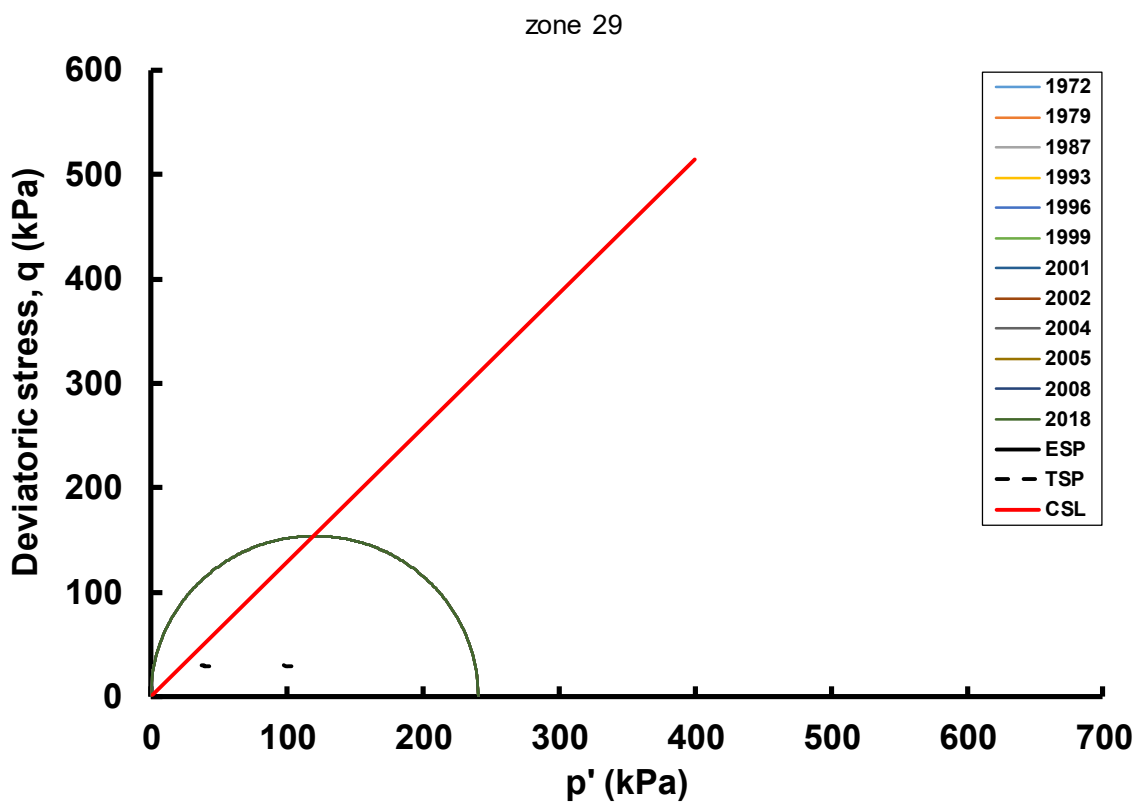
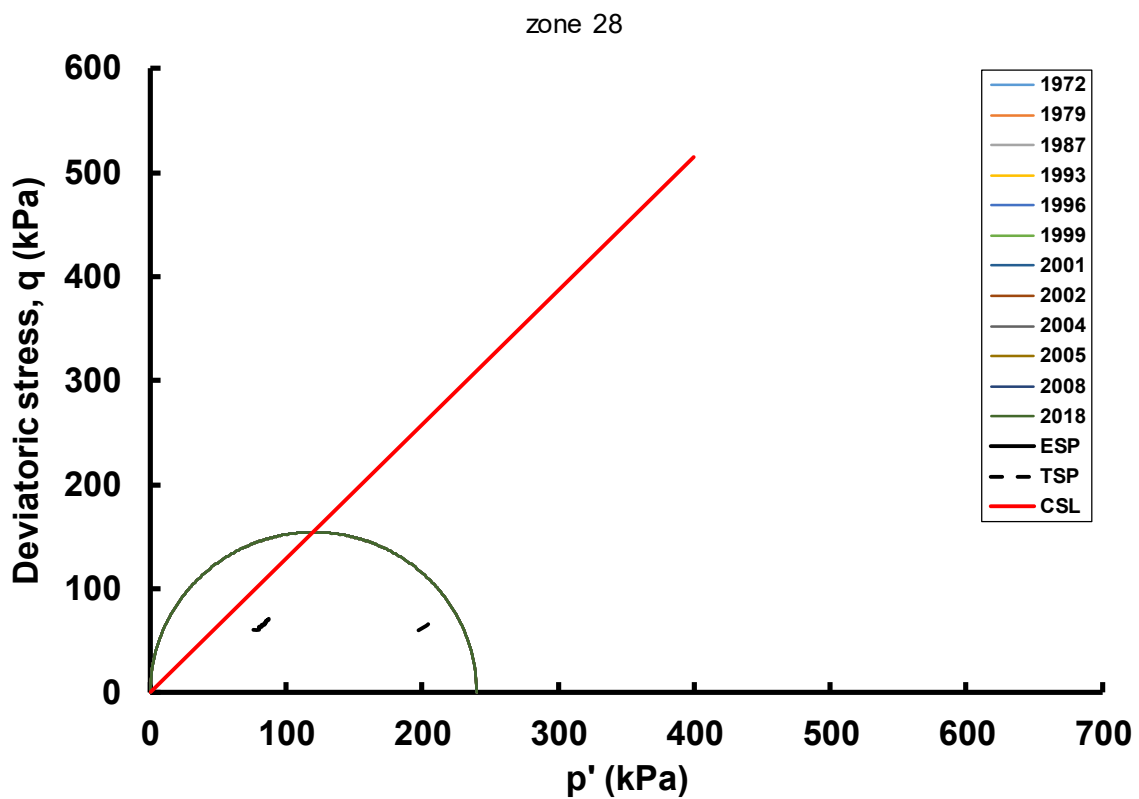


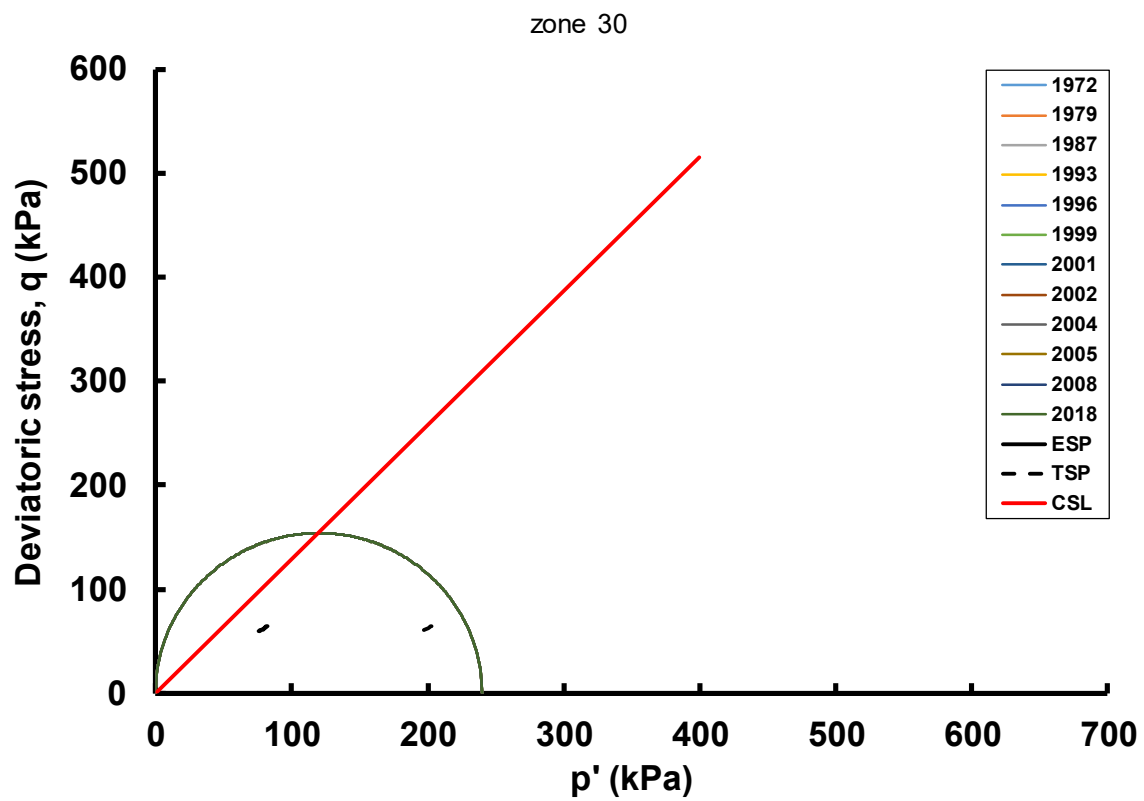












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2. **M. Poormoosavian**, A. Fakher "Deep excavation reliability analysis based on finite element method and random set theory", Geo OTTAWA Conference, Oct 2017, Ottawa, Canada.
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7. **M. Poormoosavian**, WT. Oh "Evaluation of undrained failure in foundation soil beneath Tailings dam: case study", IFCEE, May 2021, Dallas, Texas, USA, (Abstract submitted)