A CASE STUDY ON UTILISATION OF UNDRAINED STRENGTH ANALYSIS TO DESIGN AN UPSTREAM EMBANKMENT ON SOFT TAILINGS

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ABSTRACT

A case study is presented illustrating utilisation of Undrained Strength Analysis (USA) principles to design an upstream earth dam constructed on soft tailings to retain bauxite residue. A series of geotechnical investigations comprising cone penetrometer tests using piezocone with dissipation tests (CPTu) was carried out to ascertain the subsurface profile of the embankment footprint and to assess undrained strength ratios and permeability of soft soil layers. Targeted vane shear tests were carried out at selected CPTu locations as confirmatory tests. Additionally, laboratory tests including consolidated undrained (CU) triaxial tests and oedometer tests were performed on selected soil samples to further assess soil permeability and compressibility. Based on the outcomes of the investigation, a geotechnical model was developed for the proposed embankment which was then analysed for stability using both limit equilibrium and finite element methods. Staged construction of the embankment was modelled incorporating SHANSEP principles to account for strength gain in soft soil layers during staged construction due to consolidation. During the construction, subsequent geotechnical investigations and monitoring were carried out to assess the actual strength gain in soft soil layers and to refine stability models accordingly. At the conclusion of the project, a good agreement was observed between the predicted and observed behaviour of the embankment.

1 INTRODUCTION

The proposed development comprised construction of a new upstream embankment within a previously capped Residue Storage Area (RSA) to enable storage of residue mud resulting from aluminium ore processing. The new embankment was proposed to be constructed in an east-west orientation connecting two existing upstream embankments which form adjacent RSAs. The total length and the height of the proposed embankment were 720m and 6.5m at the highest point respectively. Once the construction was complete, slurry comprising 45% to 48% solids was proposed to be hydraulically discharged into the newly formed RSA.

Based on available information, the existing RSA was capped in 1980s using an approximately 3m thick residue sand layer. The underlying residue mud layer extends to the bottom of the pond which is approximately 15m below the surface of the capping sand layer. The existing embankment which forms the existing RSA was constructed using mechanically placed and compacted residue sand. The height of the existing embankment varies between 4m and 6m with upstream and downstream batter angles of 1:1.75 and 1:2 respectively. The base of the existing RSA and the downstream batter of the existing embankment has a clay liner installed to prevent leaking or seepage through the embankment.

The new embankment was to be built on the capping sand layer upstream of the existing embankment. The capping sand layer and the underlying residue mud would form the foundation for the new embankment. Due to the low thickness of the capping sand layer in comparison to the likely width of the new embankment, the foundation conditions for the new embankment were considered to be governed by the Undrained Shear Strength (S_u) of underlying residue mud. This would mean that the geometry and the rate of construction of the new embankment would predominantly be governed by S_u profile of the residue mud layer below the capping sand layer. This necessitated the assessment of existing S_u profile of the residue mud and the compressibility and permeability to assess the rate of consolidation and resulting strength gain due to consolidation during construction. These aspects including the geotechnical investigations carried out, interpretation and verification of existing S_u profile, assessment of strength gain during construction and the stability analyses are illustrated following sections of this paper.

It should be note that for the purpose of the design, the proposed new embankment was divided into four segments termed east, south-east, south-west and western. Some segments were further divided into sub-segments based on the geotechnical conditions revealed by the investigations. However, the content of this paper is limited to the presentation of stability analyses carried out for the south-east segment (SES) of the new embankment which was identified as the most critical section in terms of existing geotechnical conditions.

2 ASSESSMENT OF GEOTECHNICAL CONDITIONS

2.1 EXISITNG GROUND CONDITIONS

Based on the geology maps of the project area, the subsurface conditions of the existing RSA are considered to comprise alluvium of Guilford formation which extends to 20-30m below ground level. The ground water table is approximately 10m below the ground water level. Based on historical records provided by the client, the clay from Guilford formation was used to form the clay liner underlying the existing tailings storage areas. Based on satellite images of the project site, the existing RSA was observed to have been capped in 1980s. The capping layer comprises a residue sand layer placed on residue mud which is approximately 3.0m in thickness.

2.2 GEOTECHNICAL INVESTIGATIONS

A series of geotechnical investigations was carried out within the proposed embankment footprint to assess the geotechnical conditions using predominantly cone penetration tests with pore pressure measurements (CPTu). In addition, vane shear tests (VST) were undertaken at selected locations to verify the S_u interpreted using CPTu. Moreover, laboratory testing including Consolidated Undrained (CU) triaxial tests and oedometer tests were carried out to assess the shear strength parameters and the consolidation characteristics of residue mud respectively.

2.3 ESTIMATION OF S_U OF RESIDUE MUD USING IN-SITU TESTS

The S_u profile of residue mud was estimated using the expression proposed by Robertson et al (1989) shown in Equation 1.

$$S_u = \frac{q_t - \sigma_v}{N_{kt}} \qquad (1)$$

Where; q_t = CPTu Cone resistance corrected for pore water pressure; σ_v =Vertical overburden stress.

According to Robertson and Campanella (1989), N_{kt} typically varies from 10 to 18 for normally consolidated (NC) clays and increases with increasing soil plasticity and decreases with soil sensitivity. During the design phase, it was considered that the existing residue mud below the sand cap is either NC or under consolidated (UC). On this basis, for initial interpretation, a N_{kt} of 10 was adopted. The resulting initial S_u profiles based on CPTu s conducted in SWS footprint are presented in Figure 1. It should be noted that only the CPTu s indicated lowest S_u are presented in the plot for clarity.

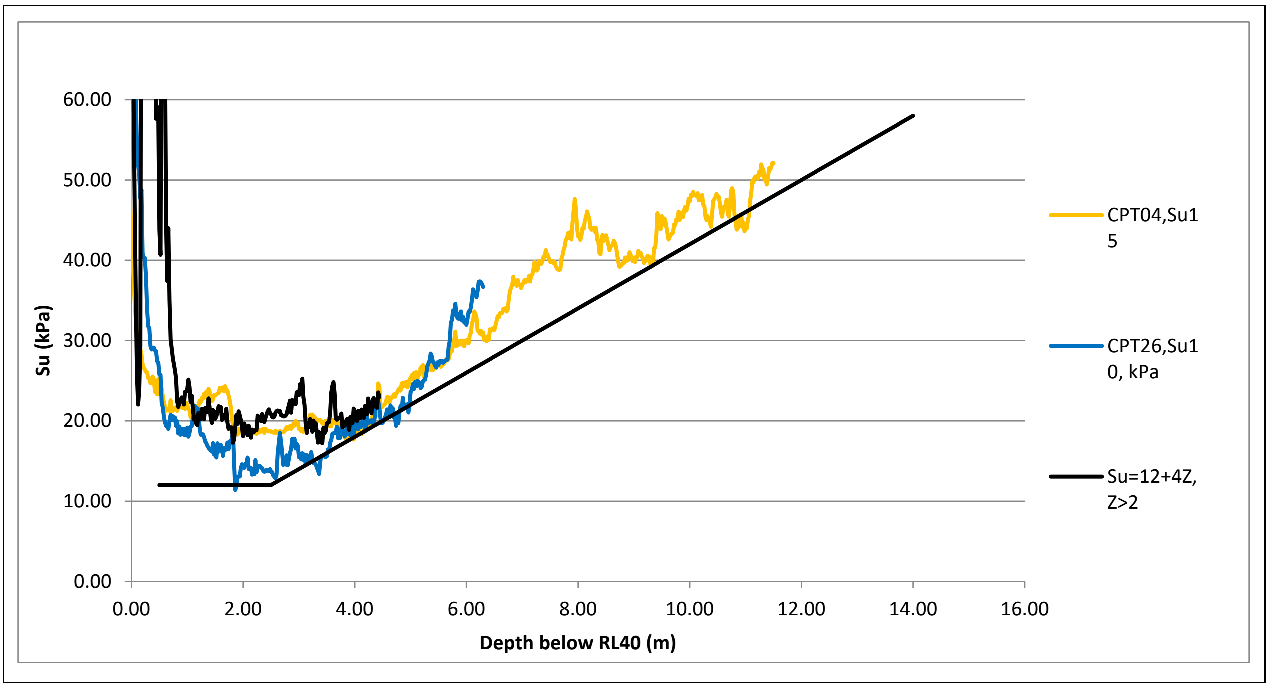


Figure 1: Initial S_u Profile

In order to verify the value of N_{kt} used in above S_u interpretation, a comparison of S_u interpreted using CPTu at selected locations was carried out with S_u estimated using VSTs undertaken within 1.0m. A comparison between CPTu and VST outcomes at a test location on SES including the design S_u profile is shown in Figure 2.

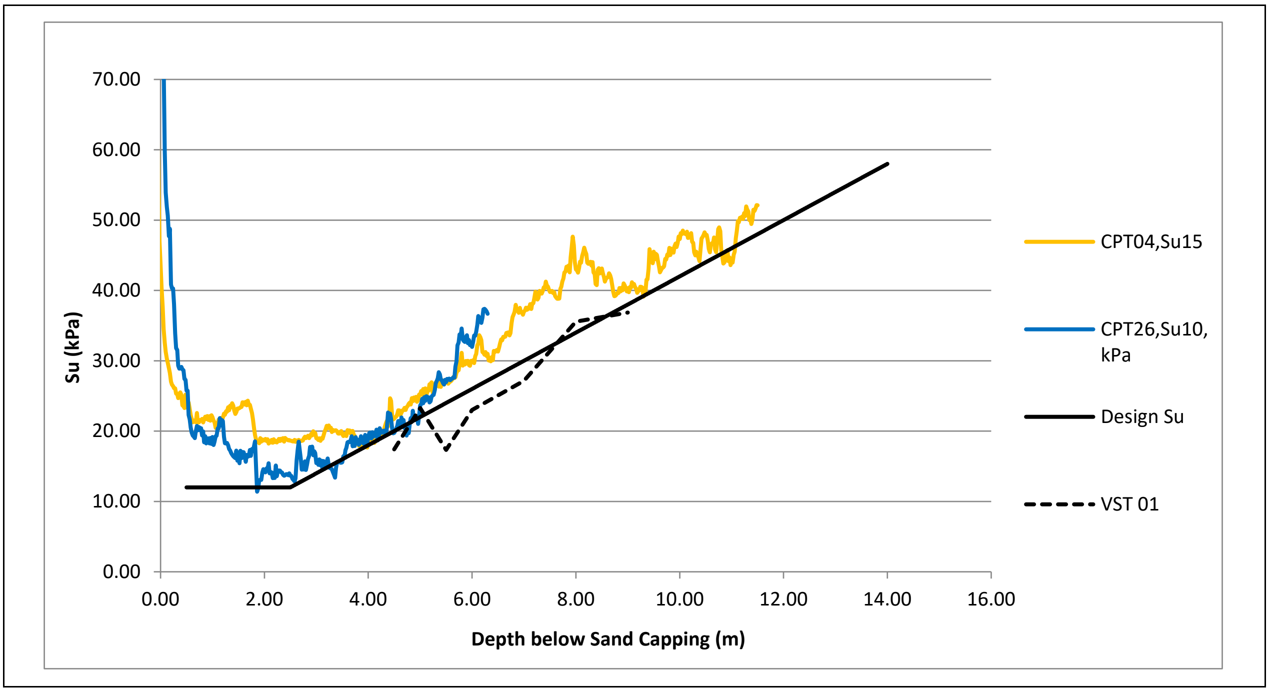


Figure 2: Comparison of S_u interpreted using CPTu and SVT

As can be seen from Figure 2, the design S_u profile estimated using $N_{kt} = 10$ is observed to indicate a reasonably good agreement with S_u estimated using VST (Refer to VS01). On this basis, $N_{kt} = 10$ was considered suitable for interpretation of S_u from CPTu results.

2.4 LABORATORY TESTS

In addition to the insitu tests mentioned above, multiple Consolidated Undrained (CU) tri-axial tests and oedometer tests were conducted on undisturbed fine tailings samples. The main objective of laboratory testing was to estimate input parameters required for advanced soil models including deformation parameters, consolidation parameters and the permeability parameters of residue mud. Interpretation of CU triaxial tests was carried out by plotting the outcomes in p' vs. q space to obtain the gradient M of the critical state line which is presented in Figure 3.

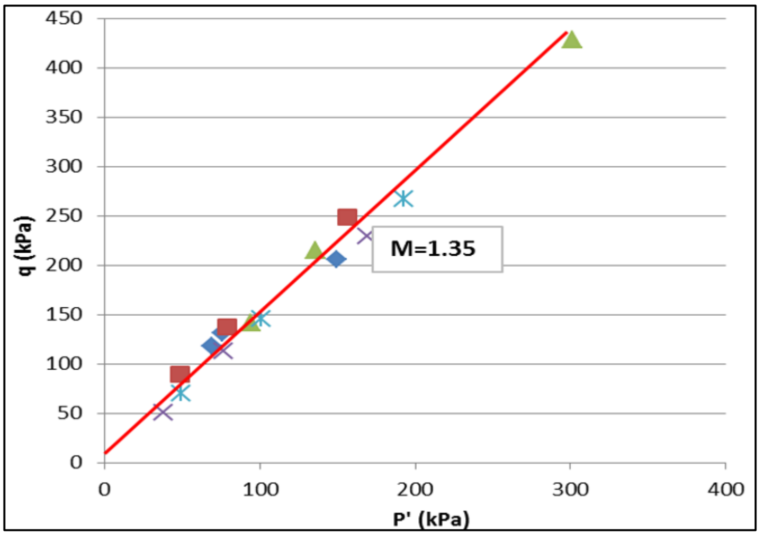


Figure 3: p' Vs. q plot using CD Triaxial Test Results

Based on the plot presented above, the *M* was estimated to be 1.35 which was then used to estimate effective shear strength parameters using Equation 2.

$$M = \frac{6\sin\phi'}{3-\sin\phi'} \tag{2}$$

Based on Equation 3, the effective friction angle (ϕ') was calculated as 33 degrees. The effective cohesion (c') was determined from Figure 3 to be 2.5kPa.

In addition to the CU triaxial tests, several oedometer tests were carried out in order to estimate the initial void ratio (e_0), virgin compression index (C_c), the coefficient of consolidation (C_v) and the permeability(k) of residue mud. On the basis of the test outcomes, e_0 was estimated as 1.0 with Cc and C_v were estimated as 0.3 and 1.0m^2/year respectively. The median permeability of the residue mud was assessed to be 0.8 x 10^{-8} m/s.

Culminating the discussions presented above, the design shear strength parameters required for the initial stability analyses of SES was determined as presented in Table 1. Other model parameters utilised in the analyses are presented in following sections.

Table 1: Shear Strength Parameters

Embankment Segment	Type of Material	Unit Weight, kN/m^3	Depth, m	Design Shear Strength Parameters		
				c', kPa	ϕ', degrees	Su, kPa
South East	Capping sand	18.0	EGL-3.0	0.0	33.0	--
	Residue mud layer 1	16.0	3.0-5.0	--	--	12.0
	Residue mud layer 2	16.0	5.0-Base	--	--	12+4z

3 GEOTECHNICAL DESIGN OF THE EMBANKMENT

3.1 PRINCIPLES OF UNDRAINED STRENGTH ANALYSIS

The stability analyses of the new embankment were carried out using Undrained Strength Analysis (USA) principles. USA method considers the insitu effective stresses as consolidation stresses which are then used to estimate the corresponding insitu undrained shear strength (Ladd C 1991). Usually this is denoted as c_u to differentiate from the S_u associated with the Total Stress Analyses (TSA), which is used hereafter accordingly.

In comparison to USA, the conventional Effective Stress Analysis (ESA) treats the existing effective normal stress as the effective normal stress at failure representing consolidated drained (CD) strength of the soft soil layer which is based on the assumption that failure occurs sufficiently slowly so that any excess pore pressure generated during shearing would be dissipated completely. Conversely, USA assumes rapid failure resulting in excess pore pressure generation and consequent reduction in effective normal stress along the failure plane, which is more representative of the likely failure scenario which would occur in soft residue mud. Due to the reduction in effective normal stress during shearing, the Factor of Safety (FS) calculated for a particular stability scenario using USA method would provide a lower FS compared to that of using the ESA method.

On the other hand, the difference between the USA and conventional TSA method is that TSA assumes S_u is equal to the shear stress applied to a failure plane consistent with $\phi = 0$ assumption whereas USA adopts c_u applied to the presumed actual location of the failure plane (Ladd C 1991). It is widely accepted that undrained conditions certainly prevail during failure events of tailings dams (Jeyapalan et al 1983), hence the utilisation of USA method, of the three methods discussed above, to analyses the stability of the proposed new upstream embankment.

3.2 STRENGTH GAIN DUE TO CONSOLIDATION

Assessment of strength gain in residue mud was a crucial part of the design process as this would dictate the rate of construction that could be adopted as well as the embankment cross section geometry. Stress History and Normalised Soil Engineering Properties (SHANSEP) technique (Ladd et al 1977) was utilised to estimate the strength gain in residue mud during construction of the embankment.

According to the SHANSEP model, the c_u used for USA can be approximated by the following relationship;

$$c_u = S(OCR)^m \sigma'_{vc} \qquad (3)$$

Where; OCR = Over consolidation Ratio; α = The normally consolidated value of c_u / σ'_{vc}; m = The strength increase exponent; σ'_{vc} = Effective vertical consolidation stress.

According to Ladd and Foot (1974), estimation of *S* and *m* parameters ideally entails establishing initial stress history of the subject soil layer followed by undertaking CK_0U tri-axial tests to consolidate the soil specimen to stresses beyond insitu consolidation stresses into the normally consolidated zone and rebound at varying OCR to measure the over consolidated behaviour. However, in lieu of a sophisticated laboratory testing campaign, during this stability analysis, *S*=0.25 was adopted for modelling based on the initial S_u profiles estimated using CPTu results presented in preceding sections. A typical *m* value of 0.8 was adopted according to Ladd C (1991) assuming low to moderate sensitive clay with plasticity index in the range of 20%-80%.

In order to implement SHANSEP technique in the numerical model, SHANSEP MC model available in Plaxis 2D as a user defined model was utilised. In addition to the *S* and *m* parameters discussed above, implementation of SHANSEP model would require additional parameters for residue mud as presented in Table 2.

Here, the ratio of G / S_u was estimated using the following relationship by Seah and Lai (2003).

$$\left(\frac{G}{S_u}\right) oc = \frac{13(OCR)^{0.867}}{0.265(OCR)^{0.735}} \qquad (4)$$

G/S_u ratio calculated using CPTu data for cohesive foundation soils generally varied from 50 – 60, and accordingly a G/S_u value of 55 was used for modelling. The minimum shear strength, $S_{u,min}$ was specified based on the initial S_u profiles presented in Table 1. The minimum over consolidation ratio, OCR_{min}, was intentionally set at 0.2, although a value of 1.0 was more likely, to cater for any likelihood of under-consolidation of the residue mud due to generation of excess pore water pressure during embankment construction. It should be noted that G/S_u was estimated with OCR of 1 which remained constant through the modelling and the slope stability is not sensitive to G/S_u.

Table 2: SHANSEP Parameters

SHANSEP Parameter		Remarks
$\frac{G}{S_u}$	55.0	Based on Equation 4 by Seah and Lai (2003)
S	0.25	c_u / σ'_{vc} based on CPTu
m	0.8	Typical value for NC clay
$S_{u,min}$	12 kPa	Based on CPTu results
OCR_{min}	0.2	Assumed low value to account for possible UC conditions

3.3 STABILITY ANALYSES

Stability analyses were carried out using predominantly Finite Element (FE) methods with verification checks carried out using Limit Equilibrium (LE) methods. Plaxis 2D and Rocscience Slide 7.0 were used for FE and LE analyses respectively. During the stability analysis, the staged construction of the new embankment was modelled using staged construction facility available in Plaxis 2D. Accordingly, two dimensional (2D) FE models were created for SES based on the geotechnical model presented in Table 1. During FE modelling, residue sand was modelled using Mohr-Coulomb (MC) model and the residue mud layers were modelled using the Modified Cam Clay (MCC) model (Roscoe and Burland 1968). The objective of using MCC model was to account for the plastic volumetric strains in soft soil layers. The MCC model parameters were derived using CU tri-axial and oedometer tests results. Both MC and MCC model parameters utilised are summarised in Table 3 and Table 4 respectively.

Table 3:Mohr Coulomb Parameters for Residue Sand

Soil Parameter		Remarks
Effective friction angle (ϕ')	34.0 degrees	Obtained from oedometer tests
Effective cohesion (c')	0.1kPa	0.1kPa was used for enhanced numerical accuracy
Young's modulus (E)	200*S_u	--
Permeability	1x10^{-4} m/s	Typical permeability for sand

Table 4: Modified Cam-Clay Parameters for Residue Mud

Soil Parameter		Remarks
Initial void ratio (e_0)	1.0	Obtained from oedometer tests
Compression index ($C_c/1+e_0$)	0.13	Median C_c obtained from oedometer tests
Modified compression ratio (λ)	0.056	$\lambda = C_c/2.303$
Modified unload-reload compression ratio (K)	0.0056	$K = \lambda/10$
Gradient of critical state line (M)	1.35	Median M obtained from CU tri-axial tests

The embankment was incrementally raised to the final level in 1.0m increments with each raise taking an initially assumed 30 days to complete. Underlying residue mud was allowed to consolidate during construction. Two stability scenarios were assessed to account for the stability after construction (short term), and after the impoundment in the long term. The minimum required stability for tailings dams in terms of FS based on ANCOLD (2012) are presented in Table 5.

Table 5: Minimum Stability Requirements (ANCOLD 2012)

Loading Condition	Min FS	Shear Strength to be Used for Evaluation
Long Term	1.5	Effective shear strength
Short Term (no potential loss of containment)	1.3	Consolidated undrained shear strength

It should be noted that an additional stability analysis for post-seismic event was also carried out in accordance with ANCOLD 2012 guidelines. However, this is not presented herein as the intent is to present utilisation of USA method for stability analysis during staged construction.

The numerical discretisation of SES cross section is presented in Figure 4.

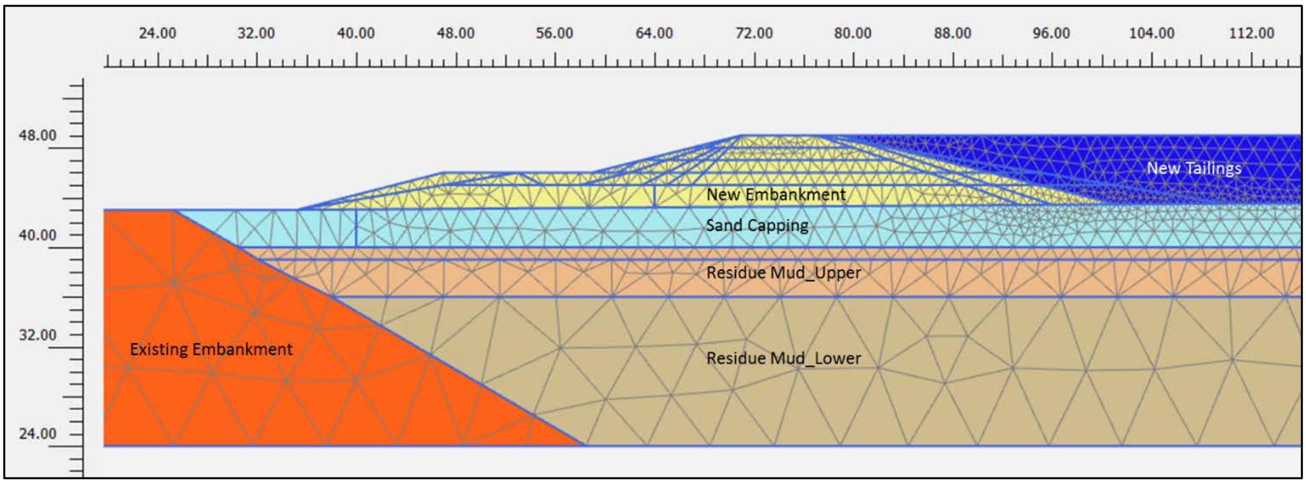
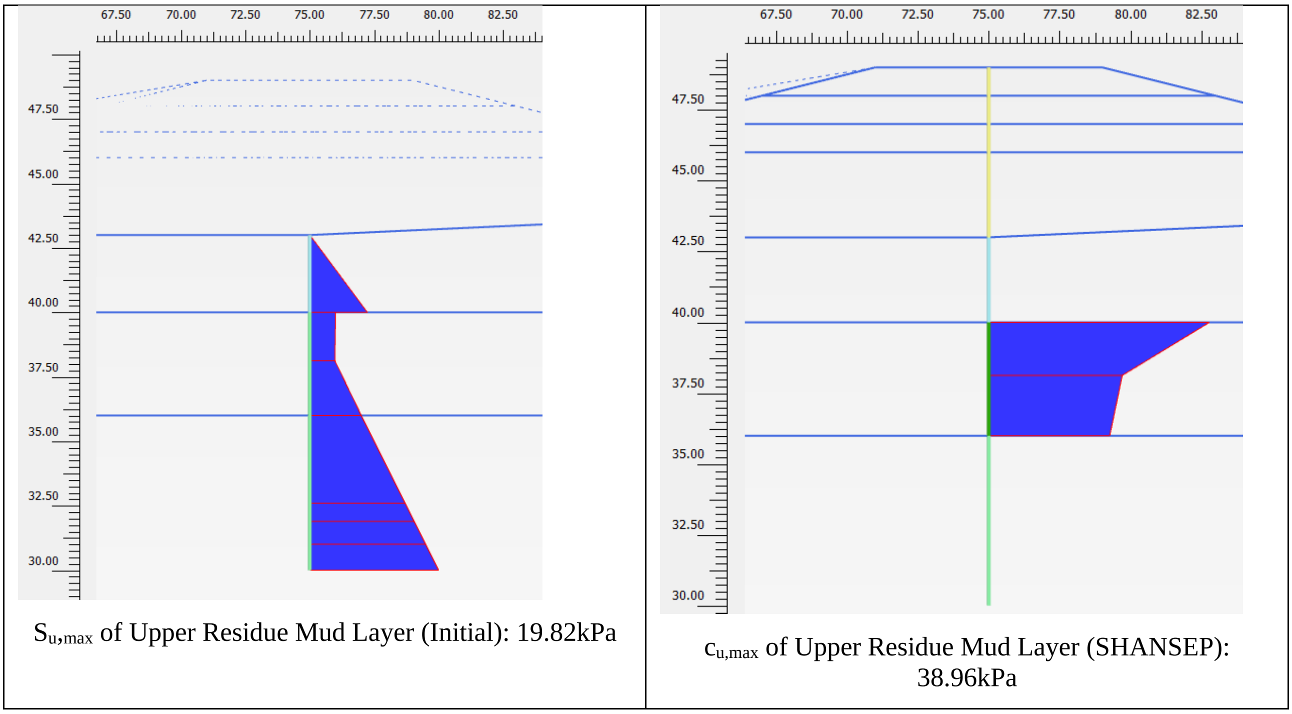


Figure 4: Numerical Discretisation of the Embankment

After reaching the design finish level, the upper residue mud layer shown in Figure 4 was switched to SHANSEP model to obtain c_u (Note that the CD shear strength during construction is denoted as c_u) followed by safety analysis to account for the effect of strength gain due to consolidation. A comparison of initial S_u profile (Refer Table 1) of the upper residue mud layer with c_u (i.e. CD shear strength) after construction (i.e. short term) calculated by Plaxis 2D using SHANSEP method is presented in Figure 5. Note that the upper residue mud layer is located between RL 40.0m – 36.0m as shown on the vertical scales.



$S_{u,max}$ of Upper Residue Mud Layer (Initial): 19.82kPa

$c_{u,max}$ of Upper Residue Mud Layer (SHANSEP): 38.96kPa

Figure 5: Initial Vs. SHANSEP Predicted c_u profiles

As can be observed from Figure 5, the CD shear strength of the upper residue mud layer has increased as a result of consolidation during the construction. In particular, the highest increase in c_u can be observed close to the sand cap interface due to the proximity to the sand layer and consequent rapid dissipation of excess pore pressure. Increase in c_u lowers with depth due to slow excess pore pressure dissipation as a result of increased length in drainage path.

The resulting FSs including the failure mechanisms for upstream and downstream batters are presented in Figure 5 and Figure 6 respectively. As can be observed from the figures, the upstream embankment was critical for the short term loading scenario with FS of 1.47 after construction and the downstream batter becomes critical with an FS of 1.60 in the long term after the impoundment. Moreover, based on the figures, the upstream slope indicates a circular failure mechanism whereas the downstream slope indicates a non-circular wedge type failure mechanism.

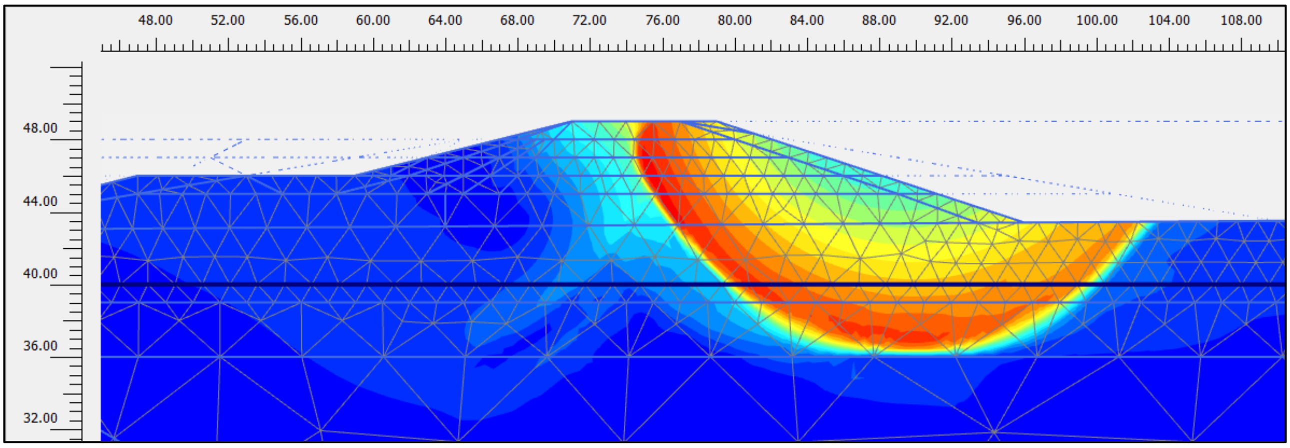


Figure 6: Failure Mechanism After Construction (Upstream Critical) – FOS 1.47

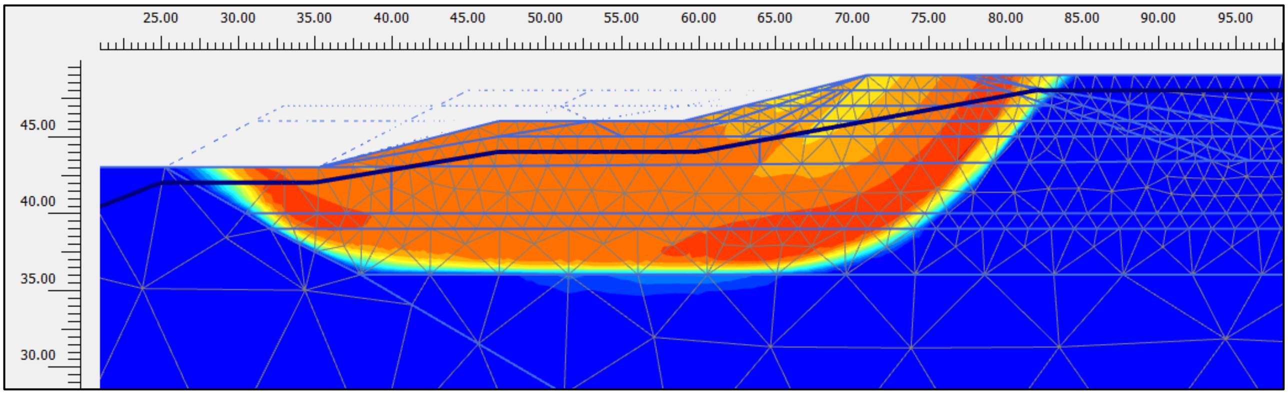
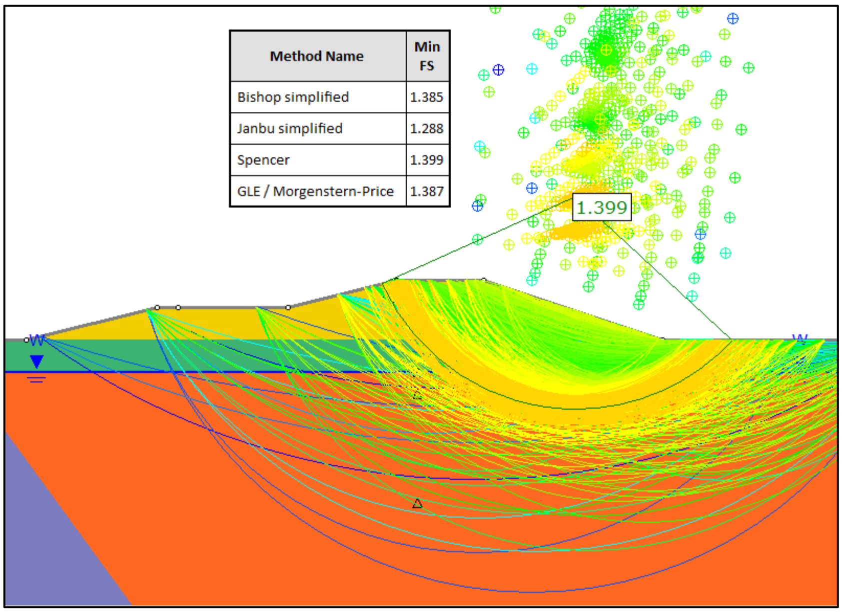


Figure 7: Failure Mechanism After Impoundment (Downstream Critical) – FOS 1.60

3.4 VERIFICATION OF FS USING LE METHODS

In order to verify the outcomes of the FE stability analyses, confirmatory LE models were developed using Rocscience Slide 7.0 software package for the cross sections analysed using FE methods. MC model and the vertical stress ratio method were utilised to model the capping sand and residue mud layers respectively. Increase of S_u due to consolidation was calculated using SHANSEP technique available in Slide 7.0 based on the parameters summarised in Table 2.

Analysis was carried out using Spencer and GLE/Morgentern-Price methods which satisfy both force and moment equilibrium conditions. Both circular and non-circular failure surfaces were investigated using cuckoo path search method. The outcomes of the LE analysis indicated a FS of 1.39 for the upstream slope for the embankment for short term loading conditions in comparison to the FS of 1.47 indicated by Plaxis 2D. The results of LE stability analysis are presented in Figure 8.



Method Name	Min FS
Bishop simplified	1.385
Janbu simplified	1.288
Spencer	1.399
GLE / Morgenstern-Price	1.387

Figure 8: LE Stability Analysis

Continuing the aforementioned procedure, different combinations of cross section geometry and rate of construction were trialled until the required minimum FSs were achieved. Inputs from the client in relation to the planned earthworks quantities/capacity and limitations in borrow materials for construction were also considered in developing the final designs. Culminating the above, the design upstream and downstream batter angles were proposed as 1:3 and 1:4 respectively with a 12.0m wide bench in the downstream side. The design rate of construction was proposed as 30 days per 1.0m rise of the embankment.

3.5 VERIFICATION OF STRENGTH GAIN DURING CONSTRUCTION

As presented above, the design process relied on the strength gain in residue mud during construction to achieve the minimum FS requirements for the embankment. This necessitated the requirement to verify the strength gain predicted using SHANSEP in order to ensure that the embankment is adequately stable. Accordingly, a supplementary investigation comprising VST was carried out after 2 months from commencement of construction. At the time of the supplementary investigation, the embankment in the area had been raised by approximately 2.0m from the foundation level. Based on the outcomes of the VSTs, S_u profile was interpreted using the process set forth in preceding sections which was compared with the initial S_u profile as presented in Figure 9. A comparison of design S_u profile and predicted c_u profiles using SHANSEP method are also presented.

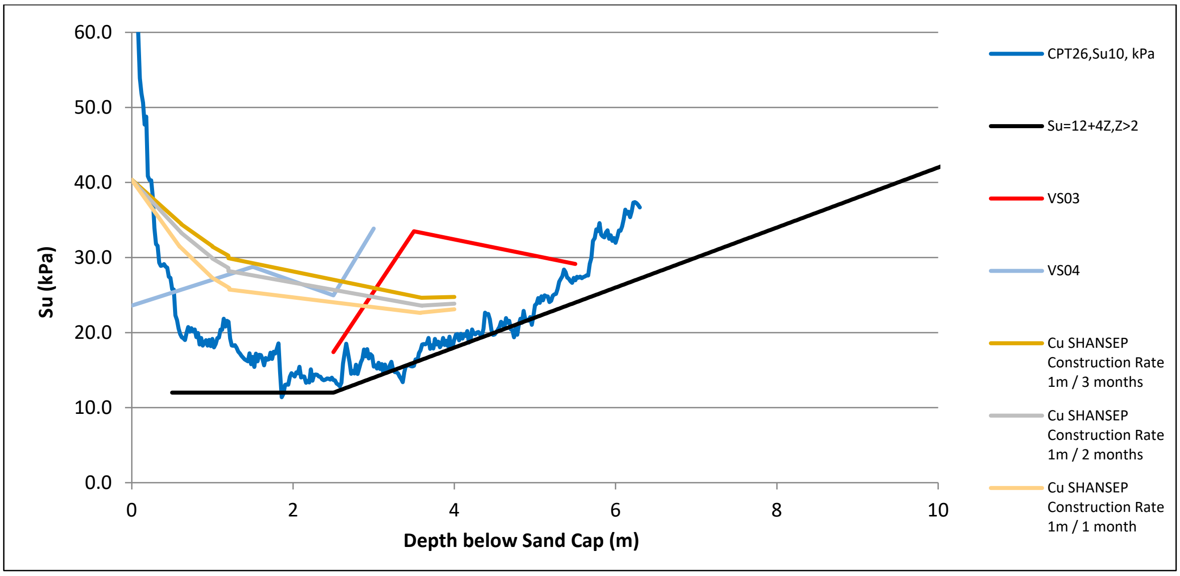


Figure 9: Predicted Vs. Measured Su during construction

As can be seen from Figure 9, a reasonable agreement was observed between the SHANSEP predicted shear strength and the VST results thereby confirming the reliability of the embankment stability assessment.

4 CONCLUSION

To conclude with, it is noted that USA principles incorporating SHANSEP technique provide an effective approach to design embankments on soft soils. The enhanced facilities provided in FE software packages to implement SHANSEP method enables effective assessment of strength gain in soft foundation soils on which embankments with complex geometries are constructed. During the design phase, this approach would enable the designer to trial different combinations of construction rates and embankment geometries. Supplementary CPTu and/or VST undertaken during construction would enable the designer to optimise/alter the rate of construction and/or the geometry of the embankment depending on the actual strength gain achieved.

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