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Comparison of Undrained A and Undrained C constitutive models in finite element method based on CPTu results from embankment on very soft marine clay

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ABSTRACT: Very soft marine clay is prevalent in coastal regions of Indonesia, particularly near the shorelines adjacent to seawater. These areas are often designated for sea-dike construction. In this study, a sea dike with embankment height varying from 11 - 14 m will be built on top of very soft marine clay with thickness ranging from 15 to 30 m, and compressible layer ranging from 35 m to 50 m. Due to the extremely low shear strength of the existing subsoil, the embankment has to be constructed in several stages to ensure stability and sufficient time for excess pore pressure dissipation. Safety factors in each construction stage is evaluated using undrained shear strength obtained from CPTu. The safety factor is maintained above 1.2 for all construction stages, and the safety factor is targeted to be 1.5 in long term condition. In this study, the safety factors are evaluated using two different drainage types, i.e., Undrained A and Undrained C. Undrained A is used to calculate the safety factor based on a variation of effective shear strength, while Undrained C uses the undrained shear strength from CPTU. Undrained C analysis was carried out using the same deformed mesh and condition obtained in Undrained A analysis, and the results are compared to analyzing the compatibility and effectivity between two models.

KEYWORDS: Undrained shear strength, Finite element method, Safety factor, Staged construction, Consolidation,

1 INTRODUCTION

From geotechnical perspective, the primary challenges associated with very soft clay are its low bearing capacity due to extremely low shear strength, and significant long-term settlement resulting from its high compressibility and low stiffness (Bjerrum, 1973). To solve these problems, the embankment can be constructed in multiple stages to prevent bearing capacity failure. Alternatively, basal reinforcement by using bamboo mattress and high-strength geotextile can be used. as Another option is to use prefabricated vertical drain (PVD) to accelerate consolidation settlement during construction, thereby reducing the amount of settlement in long-term condition.

To evaluate the stability of construction during all stages, a Finite Element Method (FEM) analysis incorporating appropriate soil constitutive models is employed. Advanced Undrained A constitutive models such as the Soft Soil (SS) & Soft Soil Creep (SSC) (Stolle, Bonnier, & Vermeer, 1997, Vermeer, Stolle & Bonnier, 1998, Vermeer & Neher, 1999, Brinkgreve, 2004), Hardening Soil (HS) (Schanz, Vermeer & Bonnier, 1999) and Cam-Clay Model (Chen and Mizuno, 1990) are example of constitutive models used to simulate the behavior of clay. Although these models can predict the behavior of very-soft marine clay accurately, these models often come with several limitation. One of its main limitations is the need to use effective stress parameters which are not directly representative of undrained behavior for clays. This limitation can lead to overestimation of undrained shear strength (s_u) which leads to overprediction of factors of safety (FoS) during construction. However, this can be circumvented by checking the undrained shear strength from the initial phase output and during each stage construction (based on the design degree of consolidation), which leads to more accurate and detailed stability analysis that incorporates the excess pore water pressure generated during the embankment process that can also be checked with proper instrumentation.

The Mohr-Coulomb Undrained C model has been widely used for assessing short-term stability under undrained loading conditions, particularly in critical applications such as embankments on soft clay. The Mohr-Coulomb Undrained C model offers improved representation of soft clay behavior and remains a practical and commonly accepted approach for preliminary and short-term stability evaluations. However, this model has several inherent limitations, particularly in its inability to calculate for effective stress paths, excess pore pressure and stress history effects (pre-consolidation pressure, stress decrease due to embankment submerging below sea water level, etc.) which are significant when dealing with very soft and highly compressible marine clay. In this paper, a method of recalibrating both constitutive models to get the undrained shear strength of the existing soil as close as possible to the CPTu during construction. The undrained shear strength obtained from Soft Soil Creep Undrained A analysis is calibrated to until it matches the CPTu result and using the deformed mesh obtained from Undrained A analysis and using incremental increase of undrained shear strength from Piezocone Penetration Test (CPTu) to compare and determine the reliability of Undrained C analysis during stage construction. Parameter adjustment is performed once at the beginning of the analysis and subsequently cross-checked during each construction stage. The adjusted parameters are compared against the CPTu results obtained at each embankment stage.

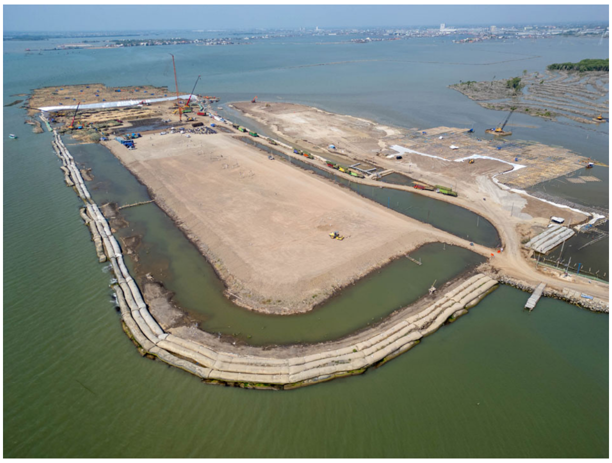

Figure 1. Current condition of embankment construction

2 CASE STUDY

2.1 *Soil condition*

This embankment construction is part of Semarang-Demak Toll Road Integrated with Sea Dike Project, located in Semarang, Indonesia. The embankment has a total height of 13.6 meters, with a crest width of 55 meters and a base width (toe-to-toe) of 150 meters. The current development of the project can be seen in Figure 1. In general, the location of construction is built on top of three layers of marine clay with different consistency. Several field tests were carried out including Deep Boring and Piezocone Penetration Test (CPTu). The first layer consists of very soft to soft marine clay, with an N_{SPT} ranging from 1 to 4. This layer varies in thickness from 15 to 30 meters. According to CPTu data, this layer is fairly homogeneous across the construction site, with corrected cone resistance (q_t) values increasing from 0 MPa at the existing seabed to an average of 1.1 MPa at 30 meters depth. The q_t value of the existing soil can be seen in Figure 2.

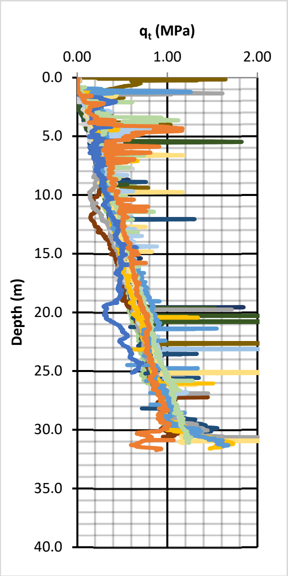

Figure 2. CPTu results during pre-construction phase.

The second layer below this was medium stiff to stiff marine clay with N_{SPT} ranging from 4-10. This layer varies from 10 to 20 m in thickness. Directly below this layer is stiff to very hard clay that has N_{SPT} that ranges from 11 to 20. This hard clay is found directly below the medium stiff to stiff marine clay until the end of boring. According to USCS classification system, all three layers of soil consist of high plasticity fat clay (CH) and elastic silt (MH) that are distributed in various depth. Properties of the existing soil before embankment construction can be seen in Figure 3 and Figure 4.

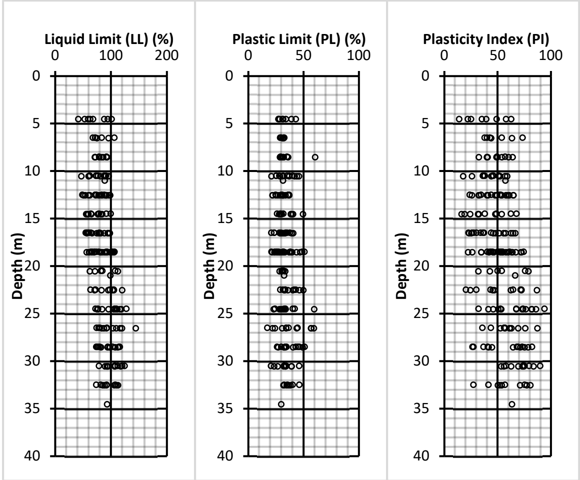

Figure 3. Atterberg limit of existing soil.

2.2 *Soil improvement and stage construction*

The embankment to be constructed will have a total height of 13.6 meters, built in 6 stages, most of the stages consist of constructing a 2-meter-high embankment. To accelerate consolidation settlement that will occur due to embankment load, prefabricated vertical drains (PVDs) were selected as the soil improvement method. The existing ground level varies from -0.5 meters to -2.0 meters Mean Sea Level (MSL). Due to the seabed's elevation below seawater, the very soft marine clay is constantly saturated due to sea level. This also hinders the construction of (PVD), due to lack of dry surface for the PVD rig to install PVD properly. To address this condition, first stage of embankment is carried out before PVD installation. Using bamboo mattresses and bamboo cluster as basal reinforcement, 3.1 m high embankment is constructed to provide a stable platform for PVD rig to operate and install the PVD. Once the PVDs were installed, second stage of embankment construction was carried out, adding 2 meters in height and high strength geotextile with T_{ult} = 1200 kN/m followed by a consolidation period of approximately 330 days. After this period, a third stage added another 2 meters of embankment, with a consolidation time of 75 days. The fourth and fifth stage repeats with the same configuration of 2 meters embankment increments and a consolidation period of 75 days. In the final stage, 2.5 meters of embankment was added, followed by a consolidation period of 150 days.

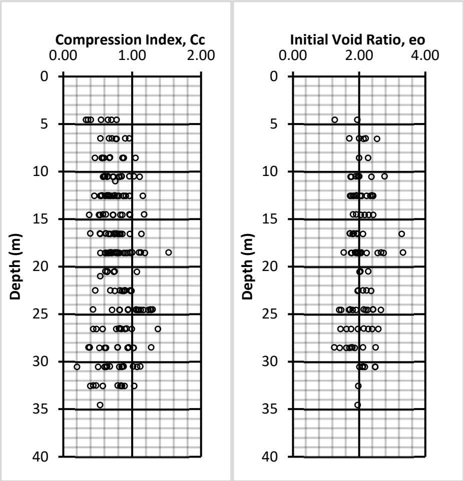

Figure 4. Consolidation parameter of existing soil.

2.3 *Instrumentation*

Several instruments were installed during construction. Settlement plates were installed before the first stage of embankment was constructed. While other instrumentations such as Vibrating Wire (VW) piezometer and inclinometer were installed after first stage of embankment due to lack of stable dry surface for their installation. Six rows of settlement plates, with eight settlement plates per row and four VW piezometers (installed every 7 m, starting from 1.5 m below the very soft soil to a depth of 36 m) are placed along the centerline of the embankment, as seen in Figure 5.

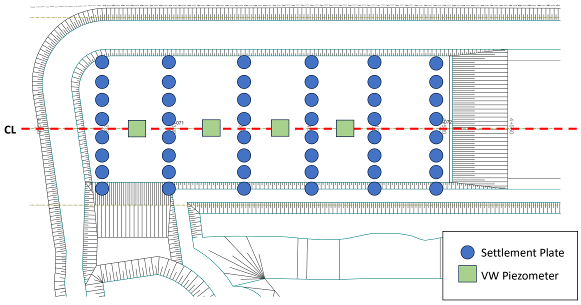

Figure 5. Layout and locations of settlement plate and piezometer

2.4 *Field test results during construction*

During construction, CPTu tests were conducted after the consolidation periods of the second, fourth, and fifth stages. Three CPTu tests were carried out on each of these stages. The tests were conducted on the same coordinate to ensure the change in the cone resistance was not caused by natural variation of the clay. All of the data were used to ensure embankment stability before proceeding to the next construction stage. The layout and coordinate of each CPTu tests is shown in Figure 6. Comparison between the corrected cone resistance (q_t) of each location for all three stages for location 1, 2 and 3 can be seen in Figure 7 and Figure 8. From these CPTu results, the q_t value of the very soft marine clay increases after each stage of embankment construction.

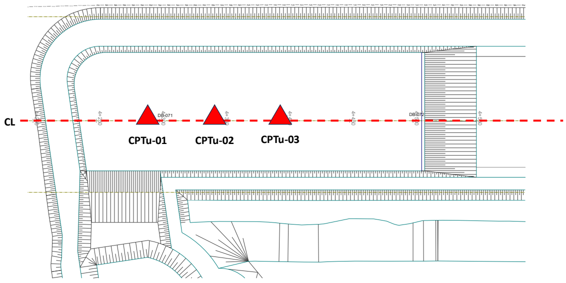

Figure 6. Layout and locations of CPTu tests

3 UNDRAINED A ANALYSIS

Undrained effective stress (Undrained A) analysis can be used in combination with effective strength parameters φ' and c' to model the material's undrained shear strength (Undrained A). In this case, the development of the pore pressure plays a crucial role in providing the right effective stress path that leads to failure at a realistic value of undrained shear strength (s_u). However, some soil models are not capable of providing the right effective stress path in undrained loading. As a result, they will produce the wrong undrained shear strength if the material strength has been specified on the basis of effective strength parameters. To confirm the result from Undrained A analysis, the undrained shear strength based on the calculation result has to be compared to the CPTu data from field tests.

To better simulate the behavior of very soft marine clay, and simulate creep during construction, Soft-Soil Creep Model is chosen to represent the very soft marine clay to medium stiff marine clay up to 43 m depth. Based on the soil properties before embankment construction begun as stated in Figure 3 and Figure 4, the compressible marine clay is further detailed into four layers as follows:

- Very soft marine clay from 0 to 1.5 m depth.
- Very soft to soft marine clay from 1.5 to 8 m depth
- Soft marine clay from 8 to 22 m depth
- Soft to medium stiff marine clay from 22 to 43 m in depth.

Embankment is modelled using Hardening-Soil Model, while the soft marine clays are modelled using Soft Soil Creep. The unit weight, creep compression index, effective shear strength and permeability parameters for the four layers of marine clays and the embankment can be seen in Table 1. While the compression index and swelling index for soft marine clay layers can be seen in Table 2 to 5.

Table 1. Parameters for marine clays and embankment

Parameter	Symbol	Value		Unit
		Embankment	Marine Clays	
Bulk unit weight	γ_{sat}	18.5	16.1	kN/m^2
Creep Compression Index	c_α	-	0.09	-
Overconsolidation Ratio	OCR	-	1	-

Cohesion intercept	*c'*	1	0.5	kN/m^2
Friction Angle	*o*	30	19	kN/m^2

Using the defined parameters for analysis, the undrained shear strength of the very soft marine clay is evaluated based on CPTu data obtained during the second, fourth, and fifth stages of embankment construction. The undrained shear strength after three consolidation stages can be evaluated using the soil test feature in PLAXIS FEM analysis. For comparison, triaxial and direct shear tests in the soil test feature are used to simulate and assess the undrained shear strength after consolidation, with the effective stress being the input of the soil tests at each stage. Comparison between undrained shear strength and CPTu data for second stage, fourth stage and fifth stage can be seen in Figure 9, Figure 10 and Figure 11 respectively. Along with data from CPTu tests, the amount of settlement for each stage in FEM analysis is also calibrated with settlement plate reading from the instrumentation.

Table 2. Stiffness parameters for first layer (very soft marine clay)

Parameter	Symbol	Value	Unit
Compression Index	C_c	1.36	-
Swelling Index	C_s	0.08	-
Void Ratio	e_0	3.00	-

Table 3. Stiffness parameters for second layer (very soft to soft marine clay)

Parameter	Symbol	Value	Unit
Compression Index	C_c	1.18	-
Swelling Index	C_s	0.08	-
Void Ratio	e_0	2.00	-

Table 4. Stiffness parameters for third layer (soft marine clay)

Parameter	Symbol	Value	Unit
Compression Index	C_c	1.09	-
Swelling Index	C_s	0.05	-
Void Ratio	e_0	2.10	-

Table 5. Stiffness parameters for fourth layer (soft to medium marine clay)

Parameter	Symbol	Value	Unit
Compression Index	C_c	0.73	-
Swelling Index	C_s	0.05	-
Void Ratio	e_0	2.50	-

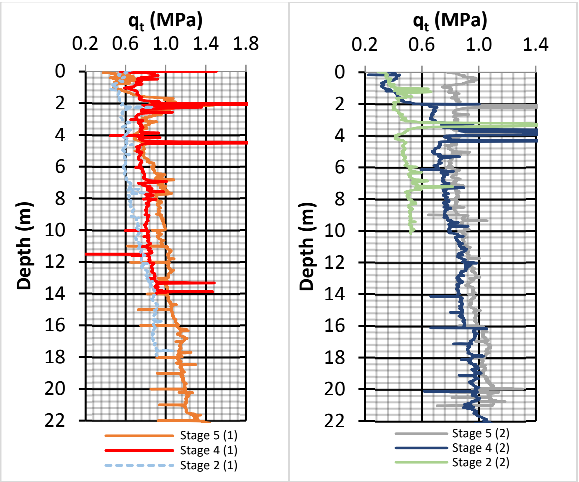

Figure 7. Corrected cone resistance for location 1 & 2 at the end of second, fourth and fifth consolidation stages

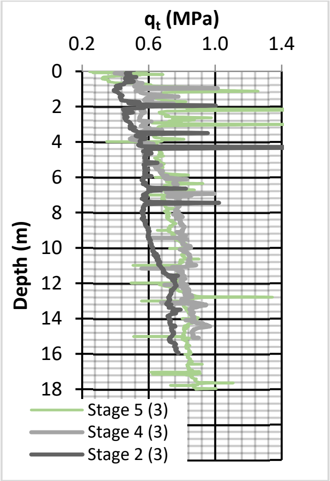

Figure 8. Corrected cone resistance for location 3 at the end of second, fourth and fifth consolidation stages

With all the parameters calibrated, factor of safety analysis using c'-ϕ' reduction method and consolidation analysis using effective parameters can be performed on all construction stages.

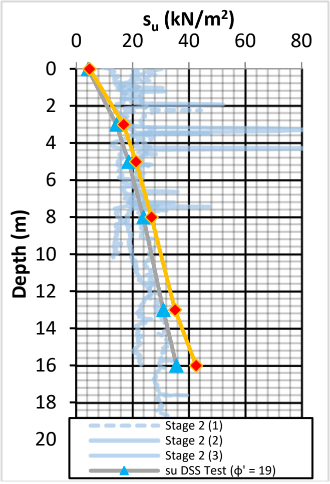

Figure 9. Undrained shear strength result from Undrained A model vs CPTu result after second stage consolidation.

The parameters in Table 1 to Table 5 are result from this back-analysis, calibrated to produce settlement values as close as possible to the actual settlement plate reading. A comparison between the settlement results from the FEM analysis and the observed settlement plate readings along the centerline of the embankment is shown in Figure 12. The factor of safety for each stage after calibration is summarized in Table 6. Additionally, comparison of the deformed mesh and the potential failure plane of the embankment after fifth stage consolidation can be seen in .

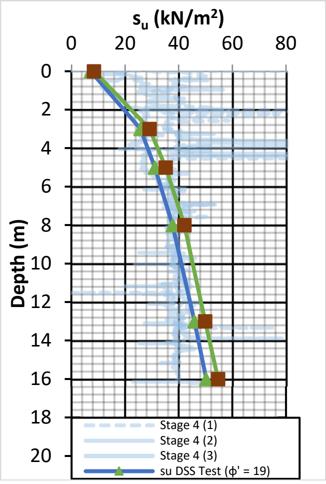

Figure 10. Undrained shear strength result from Undrained A model vs CPTu result after fourth stage consolidation.

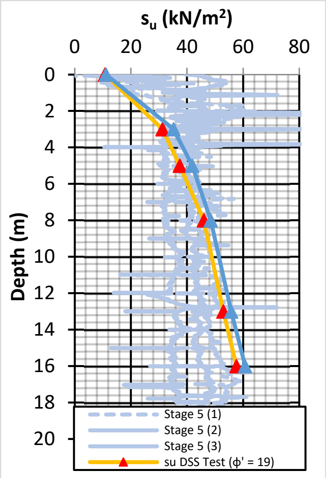

Figure 11. Undrained shear strength result from Undrained A model vs CPTu result after fifth stage consolidation.

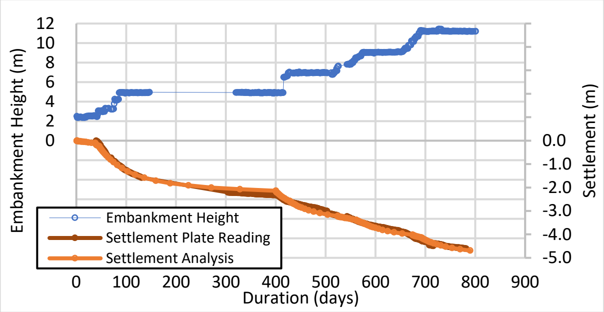

Figure 12. Settlement comparison between FEM Analysis and settlement plate reading

Table 6. FoS of embankment for each construction stage

Stage	Loading Type	FoS
Stage 2	*Undrained (Plastic)*	2.05
Stage 2	*Consolidation*	2.87
Stage 3	*Undrained (Plastic)*	2.32
Stage 3	*Consolidation*	2.83
Stage 4	*Undrained (Plastic)*	2.10
Stage 4	*Consolidation*	2.58
Stage 5	*Undrained (Plastic)*	2.02
Stage 5	*Consolidation*	2.37
Stage 6	*Undrained (Plastic)*	1.71
Stage 6	*Consolidation*	1.76

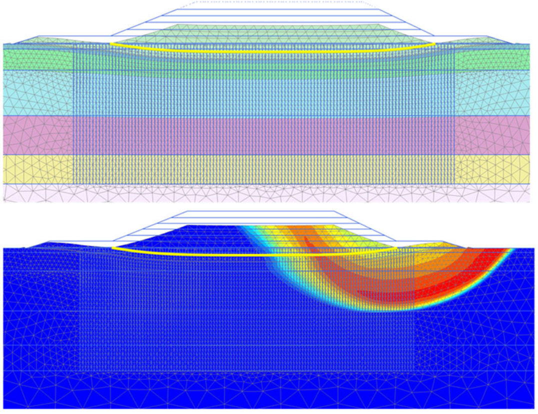

Figure 13. Deformed mesh and slope stability analysis with Undrained A model based on CPTu results

4 UNDRAINED C ANALYSIS

4.1 *Limitations*

As mentioned above, limitations of Undrained C model lie in its inability to account for effective stress paths, excess pore pressure and stress history effects (pre-consolidation pressure, stress decrease due to embankment submerging below sea water level, etc.). Using undrained shear strength derived from CPTu results from second, fourth and fifth stage after the consolidation period, the undrained shear strength profile of the existing soil beneath the embankment can be modeled similarly to that used in the Undrained A analysis. The failure plane obtained from the undrained C analysis does match the analysis from undrained A. However, the safety factor value does not match; the undrained C analysis yields a 1.38, as shown in Figure 14, while the undrained A analysis yields 2.02. This proves the limitation of Undrained C analysis, particularly its inability to account for the effects of embankment submergence and settlement in the safety factor evaluation.

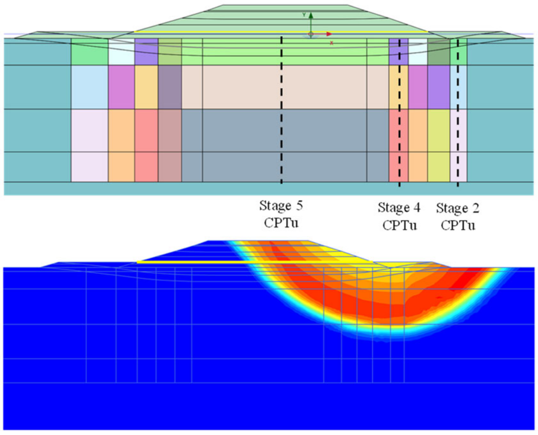

Figure 14. Undrained shear strength modelling using Undrained C, FoS = 1.38 (Stage 5)

4.2 *Adjustment of Undrained C analysis*

To adjust this factor of safety to be as close as the Undrained A analysis, embankment submerging below sea water level is modelled to be the same as the deformed mesh result from Undrained A analysis. This adjustment produced results that were surprisingly similar, with the FoS = 2.16 for the fifth stage construction, while FoS = 2.02 was obtained from undrained A analysis. The failure plane can be seen in Figure 15. To verify the accuracy of this adjustment compared to the Undrained A analysis, the sixth embankment stage, consisting of an additional 2.5 meters in height, was also compared using the same undrained shear strength parameters as those applied in the fifth stage. The outcome closely matched the Undrained A analysis, both in terms of failure mechanism and the FoS value, which was found to be 1.69 for undrained C vs. 1.71 for undrained A. The results of this analysis are illustrated Figure 16.

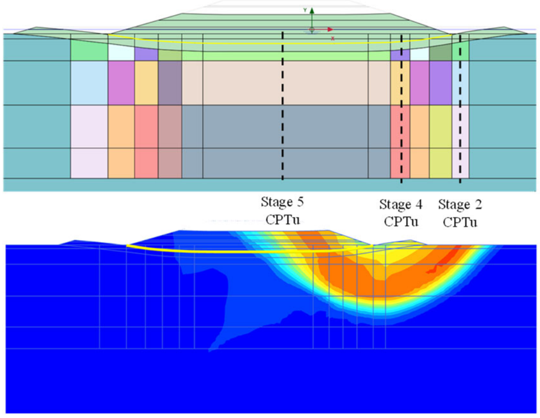

Figure 15. Adjusted model using deformed mesh and geometry obtained from Undrained A analysis, 5th stage FoS = 2.16

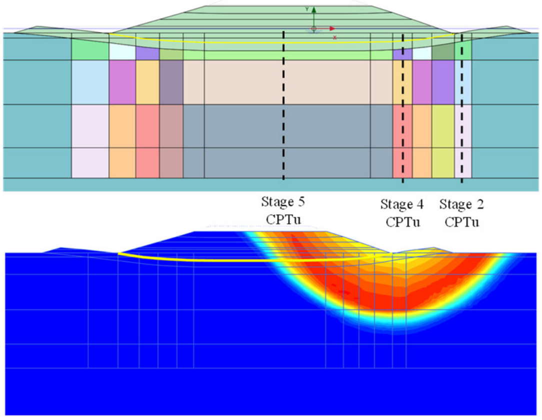

Figure 16. Sixth stage undrained stability analysis after fifth stage consolidation using Undrained C, FoS = 1.69

5 CONCLUSIONS`

The Undrained A analysis offers a more advanced and accurate method for predicting the slope stability and safety factor of embankments constructed on very soft marine clay. It is crucial, however, that the undrained shear strength derived from the Undrained A model be verified with all field data at each construction stage.

The Undrained C model has limitations in predicting settlement, excess pore pressure, and in accurately calculating the unit weight of the embankment when submerged due to settlement. As a result, the safety factors produced by the Undrained C analysis may be more conservative than those obtained from the Undrained A model. Nevertheless, the Undrained C analysis can yield comparable results to the Undrained A model if the following adjustments are made:

- All field soil investigation data that can provide undrained shear strength has to be obtained during each embankment construction stages on very soft marine clay. These data correspond to the undrained shear strength parameters that are used in undrained C analysis.
- The geometry of the Undrained C model must be modified to match the settlement predicted by the Undrained A analysis and all available monitoring data; for example, the extent of the embankment submerged below sea level and the distribution of settlement throughout the compressible soil layer

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