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RESEARCH ARTICLE

Residual Strength Parameter Method for Slope stability on a Toll Road with Expansive Clay

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Abstract

The decreasing stability phenomenon needs to be considered during the design of cut slopes on problematic soil. Excavation slope of toll road construction tends to fail when it lies above clay-shale strata. Certain common correlations and ordinary analytical methods are not recommended for safety calculation. This study is intended to find out the characteristic of clay-shale and proper slope inclination design on Semarang Batang Toll Road. The behaviour of a clay-shale area on the cut slope of Batang-Semarang toll road segment STA 438+000–STA 439+000 was identified. The degradable and expansive properties caused slope failure of the initial design with an inclination of 1 H: 1 V. Laboratory tests found that the soil had a clay faction > 40% and can be categorized as high plasticity (LL > 50%). An empirical approach determined that the residual shear strength decreased to phi < 6 degrees. To describe the swelling after the excavation stage, the flow deformation was determined by a finite element simulation. During the swelling phase, the pore water pressure was maintained at a certain value, and a gentler slope fulfilled the minimum safety factor with an inclination of 1 V: 3 H. Furthermore, the shear strength of the clay-shale was reduced to that for a fully softened material, and all the slope factors for safety moved to a critical state. According to the simulation, the minimum suggested slope inclination is 5 H: 1 V. This approach is important for the maintenance of pore water pressure and the prevention of an additional reduction in the shear strength so as to avoid slope failure on clay-shale regions in the yielding stage.

Keywords: residual, expansive, clay, slope, stability, toll-road

1. Introduction

Technical and non-technical challenges may be encountered during the construction of new roads. One of the challenges is the cut slope stability. According to Knappett & Craig (2012), the cut slope safety factor decreases due to a decrease in the pore water pressure. Conversely, if the slope is a fill or embankment slope, the safety factor increases for long-term conditions when compared to the safety factor at the end of construction (EoS). The decreasing safety factor phenomenon needs to be considered during the design of cut slopes.

On Semarang Batang toll road segment STA 438+000 – STA 439+000, the slope comprises 15-20 meters of cut slope with an initial inclination design of 1.00H:1.00H. However, slope failure occurred during construction due to its geotechnical condition. The slope consisted of weathered claystone, clay-shale and very expansive clay. The STA 438+400 segment after the slope failure is displayed in Figure 1 and Figure 2.

Figure 1 shows soft clay debris after the slope failure. When compared to the original condition, the hard clay stone transformed into clay with a low shear strength. The slope failure did not only occur in the high cut profile. Slope failure also occurred in the STA 439+000 segment with a cut slope as high as 2.50 – 3.00 meters. However, because the height was not as high as that in STA 438+400, the debris material did not close the main road.

Another case of construction on Clay-shale is found on Cipularang Toll Road KM 97+500. On this case, more than 10 meter height of embankment lies on clay-shale strata. Based on field investigation, the failure mechanism is triggered by strength degradation of clay-shale due to stripping works (Irsyam, Susila, & Himawan, 2006) .

The behavior of cut/excavated slopes on problematic soil must be distinguished from that on ordinary soil. Certain common correlations and ordinary analytical methods are not recommended for the calculation of the geotechnical design on problematic soil whether technically and economical aspect (Onochie & Rezaei, 2016). Most of problematic soil which might lead to failure are expansive clays, dispersive soils, and collapsible soils (Rezaei, Ajalloeian, & Ghafoori, 2012). The soil behaves differently than ordinary soil when it is exposed to and interacts with the surrounding environment. Certain problematic soils show good behavior in their original condition. When the soil interacts with water, it shows behavior that is opposite to that of the original condition.

According to Boggs (2009), shale is a terminology for classifying sedimentary rocks that have lamination. Clay-shale is an indurated rock with a laminated size less than 10mm. According to the Neuendorf et al. (2011), shale is defined as fine-grained sedimentary rock that becomes settled and lithified due to the consolidation process. The typical character of shale is to have the appearance of a sheet that is consistent with its bedding.

The material quality of shale may decrease over time if it is taken from its original source. When it has undergone a stress release, the mechanical behavior also changes. Shale may change from a rock-like material to soil-like material and completely degrade to silt or clay. However, high-durability shale still maintains the mechanical behavior of stone (Richardson , 1985).

On Semarang – Solo Toll Road, similar case of clay-shale slope failure is found on KM 438, an excavation slope with more than 35 meters of slope height. When the clay-shale is exposed, the slope shows good behavior and stable condition. The failure is started when the wetting and drying cycle is occurred. This process causes the clay-shale weathering, so that the shear strength is also decreased. According to laboratory test, the residual strength of clay-shale on this location decreased to 8.3 kPa for saturated cohesion and 4.6 degree for saturated internal friction of angle (Alatas, Kamaruddin, Nazir, & Irsyam, 2015). According ro Pardoyo, et al. (2020), Semarang Bawen clay-shale strength is decreased to 59% for UCS value and 36 % for modulus elasticity parameter (E_{50}) after 90 minutes drying process.

(a) (b)

Fig. 1. Slope failure along the: (a) STA 438+000; and (b) 439+000 segments at end of the construction stage

(a) (b)

Fig 2. Slope failure along the STA 438+000 segment during: (a) early road operations; and (b) a recent failure in 2021

Clay-shale instability may be caused by the texture and structure of the sediment and the mineral structure. The texture is related to the clay fraction in the clay-shale. If the amount of clay fraction increases, the clay-shale tends to be a problematic material. The sediment structure of clay-shale, such as the joints, degree of fissility (sheet appearance), and bedding, may affect its stability when it is on a sloped area. The mineral structure can be recognized by microscopic examination only.

Stark & Duncan (1991) studied the mechanical behavior and failure mechanisms of the clay-shale at the San Luis Dam in the United States. Clay-shale is classified as fissured stiff clay. The high shear strength of clay-shale on the original source indicated that it is an over-consolidated material. When the condition is dry, clay-shale has a high shear strength for the peak strength parameter. Otherwise, when clay-shale encounters dry and wet conditions repeatedly, it reaches a fully softened condition.

When failure begins, the clay-shale has undergone degradation of its residual condition. When the construction of cut slopes removes the vertical stress that overburdens clay-shale, the balance of the horizontal stress changes. According to (Bell, 1978), as for a spring, when the stress in a loaded spring is released, the potential energy is also released. The clay-shale rebounds as result of this stress release. Clay-shale durability is affected by the stress release and depends on the bonding strength of each shale. The bonding strength depends on the mineral composition.

Unstable clay-shale behavior is mostly caused by the mineral content. According to Wilson & Wilson (2014), expansive behavior on clay-shale that belongs to the smectite shale group is caused by the interaction between Na and the water content. According to Putera, et al. (2017), the unloading of the stress will increase surrounding water content which is caused by dispersing interparticle bond. If the bonding strength is low, then clay-shale heaving occurs as result of increasing the rebound value. The heaving process may be compounded by interactions with water through the gaps between the mineral sheets of montmorillonite. This phenomenon is known as the swelling of clay-shale. The swelling causes a drastic decrease in the shear strength. In extreme conditions, a decrease in the shear strength occurs continuously until it reaches the residual strength.

2. Research Method

2.1 Engineering Parameter Approach

Upon referring to recent geotechnical research, it is clear that common correlations and assumptions during the design process are not applicable for clay-shale. The research of clay-shale mechanical behavior has shown important information. At first, Skempton (1964) suggested the use of the residual strength parameter for long-term stability analysis on a slope comprised of clay. The residual strength was obtained by a back calculation method.

Gartung (1986) and Irsyam et.al. (2007) divided the clay into four weathered zones. Zone 1 comprises fresh and unweathered rocks. Zone 4 is a clay zone that has met the final stage of weathering. Gartung (1986) stated that the average value of the residual friction angle (ϕ_r) is about 8.6 degrees. However, when using these values, the engineering design will be very conservative and inefficient. The recommended residual strength for engineering design by Gartung (1986) is c' = 20 kPa at ϕ'= 20 (see on Fig.3).

Stark and Hussain (2013) proposed a correlation to determine the residual shear strength based on the clay fraction and liquid limit. The correlation was obtained from a number of engineering tests by using reversal ring shear. The correlation equation is shown in Figure 4.

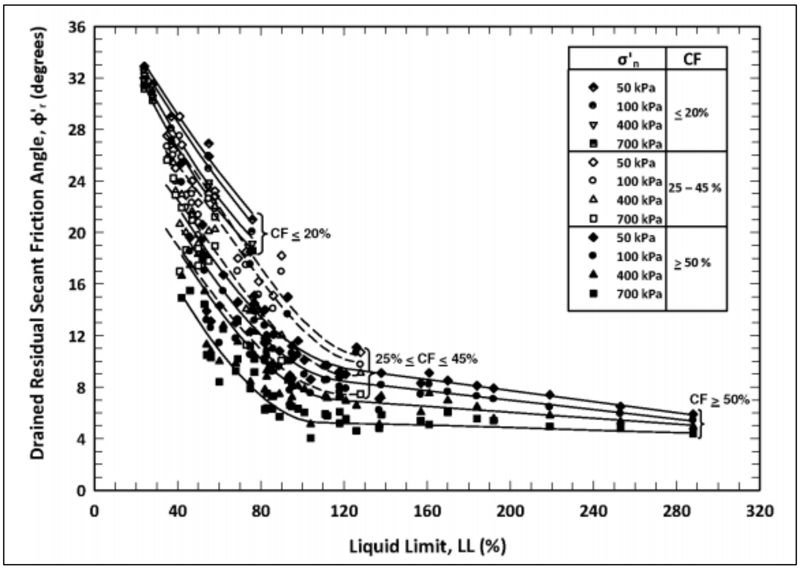

Fig 4. Empirical correlation for drained residual secant friction angle based on the liquid limit (LL), clay size fraction (CF), and effective normal stress by Stark and Hussain (2013)

2.2 Slope Stability Analysis Method

Slope stability analysis with long-term parameters is conducted by using field investigation and laboratory test data. The field investigation data consists of existing conditions, borehole and N-SPT. The stratigraphy is generated based on N-SPT and CPT data for finite element method (FEM) analysis.

To conduct simulations with FEM analysis, it is necessary to know the ideal drainage behavior of the soil material at a certain loading condition. During FEM analysis, there are two drainage behaviors: 1) undrained; and 2) drained. The undrained condition is used if the loading on the soil increases the pore water pressure and does not have time to dissipate. The drained condition is used if loading increases the pore water pressure, but dissipation occurs instantly.

The first stage of a FEM calculation is generating the initial static stress. After the initial static stress calculation, the factor of safety (FoS) calculation can be conducted by finding the soil plastic point. At the critical state, the soil becomes plastic and forms a plastic point with an extreme incremental strain. The plastic points form a band of failure planes so that the safety factor can be obtained.

A slope almost 20 meters height with a critical cut on the Semarang Batang toll road is located on the STA 438+400 segment. Two CPT tests and geotechnical drilling with an N-SPT test were conducted. All CPT tests were concluded at a depth of 8 – 10 meters. The drilling was conducted until a depth of 20 meters was reached. The slope stratigraphy, as shown in Figure 5, had four layers: 1) a weathered layer, 2) a stiff clay layer, 3) a hard clay layer, and 4) a soft rock layer. The material properties of the layers are listed on Table 1.

In Table 1, a shear strength difference between the N-SPT and CPT can be seen. A comparison of the field test shows that the CPT data is greater than the N-SPT data, possibly because of the different times of the tests. The CPT test was conducted several months earlier than the N-SPT test. Thus, the simulation used the N-SPT data, as it was close to the actual conditions. However, the different results between the CPT and N-SPT tests indicated that the weathering process of the clay-shale along the STA 438+400 segment occurred rapidly.

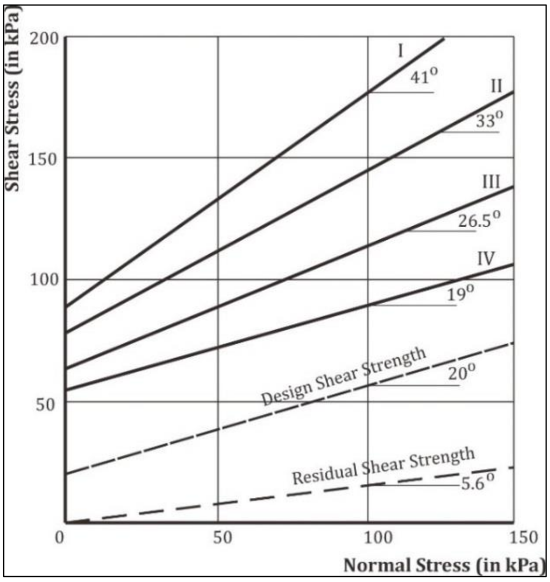

Fig 3. Clay-shale shear strength proposed by Gartung (1980)

Stark & Hussain (2013) proposed an empirical method to obtain the residual parameters based on the amount of the clay fraction and liquid limit. To validate the clay fraction and liquid limit results, a swell test and durability test must be conducted. Yulistiawati & Zaman (2018) conducted a laboratory test for several samples from the STA 438+400 segment. The laboratory test results can be seen in Table 2 and Table 3.

PT CND Geoteknika (2018) conducted a further investigation on the STA 438+000 segment due to a request by PT Waskita Karya. Two samples were taken from BH 01 for laboratory analysis. The laboratory test results can be seen in Table 4. The low shear strength indicates that softening of the clay occurred. As shown in Table 4, the liquid limit value was smaller than that from Yulistiawati and Zaman (2018). Therefore, the liquid limit value and clay fraction from Table 4 were used as input parameters for the Stark and Hussein (2013) residual strength correlation. The residual parameters can be seen in Figure 6 and Table 5.

Table 1. Shear strength parameters based on field tests

Layer	N-SPT	CPT (kg/cm2)	Unit weight saturated (kN/m3)	Unit weight Dry (kN/m3)
Weathered layer	10	45	17	18
Stiff Clay (softening)	15	70	17	18
Very Stiff Clay	22	100	17.5	19
Soft rock	50	250	19	20

Table 2. Atterberg Limit Test Result for the STA 438+400 segment (Yulistiawati and Zaman, 2018)

Sample	Colour	Atterberg Limit				
		Not Oven Dried (%)	Oven Dried (%)	Classification	PL (%)	PI (%)
BSTR-2	Black	100	92	Anorganik	33.83	58.17
BSTR-3	Greyish black	88	82.5	Anorganik	34.31	48.19
BSTR-4A	Grey	61.5	66	Anorganik	37.41	28.59
BSTR-4B	Black	83.6	83.6	Anorganik	36.21	47.39
BSTR-4C	Black	88	83.6	Anorganik	31.78	51.82
BSTR-5	Reddish clay	89	78.6	Anorganik	39.34	39.26
BSTR-6	Black	89	86	Anorganik	16.54	69.46
BSTR-7	Greyish brown	81	90	Anorganik	37.17	52.83
BSTR-8	Greyish brown	77.3	88	Anorganik	39.47	48.53
BSTR-9	Grey	46	45	Anorganik	29.92	15.08
BSTR-10	Light grey and white	48.5	58	Anorganik	23.98	34.02

Table 3. Grain size distribution and clay-shale durability for the STA 438+400 segment (Yulistiawati and Zaman, 2018)

Sample	Slake Durability (%)		Free Swelling (%)	Hydrometer Sieve Analysis					
	1st Cycle	2nd Cycle		Sand (%)	Silt (%)	Clay (%)	Passing # 40 (%)	Passing #200 (%)	Soil Class.
BSTR-2	3.34	0	355	0	48	52	100	100	Clay
BSTR-3	0.00	0	155	22.22	38.78	39	98.21	77.78	Clay
BSTR-4A	0.00	0	100	32.56	32.44	35	94.77	67.44	Clay
BSTR-4B	1.96	0	130	10.39	41.61	48	100	89.61	Clay
BSTR-4C	7.98	2.89	115	20	39	41	99.78	80	Clay
BSTR-5	3.81	0	110	21.61	38.4	40	100	78.4	Clay
BSTR-6	0.00	0	270	3.66	36.34	60	100	96.34	Clay
BSTR-7	0.85	0	220	0	42	58	100	100	Clay
BSTR-8	0.00	0	240	9.21	43.79	47	100	90.79	Clay
BSTR-9	52.16	41.03	87.5	15.38	41.62	43	100	84.62	Clay
BSTR-10	12.00	10.97	90	30.57	34.43	35	95.73	69.43	Clay

Table 4. Index properties provided by a contractor for the STA 438+400 segment (CND Geoteknika, 2018)

Sample	Atterberg Limit			Sieve Analysis				Triaxial UU	
	LL (%)	PL (%)	PI (%)	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	c_u kPa	ϕ_u deg.
3.50 – 4.00	118	35	83	52	46	2	0	85.7	6.2
9.50 – 10.00	93	36	57	52	47	1	0	21.2	11.8

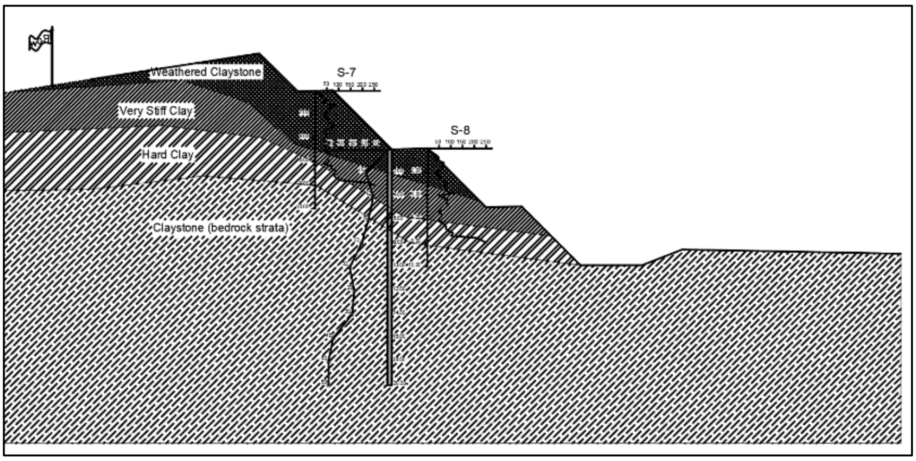

Fig 5. Slope stratigraphy interpretation on the STA 438+400 segment based on CND Geoteknika (2018)

2.3 Slope Stability Simulation

Many researchers in geotechnical engineering have been conducted slope stability simulation of clay-shale. Pratama (2021) calculate the clay-shale slope stability by using limit equilibirum method (LEM) and probabilistic approach. The residual strength parameter is assumed on certain correlated value of c_r and ϕ'_r. The simulation result show that slope safety is affected by ground water table. The limitaion of LEM can not simulate the contribution of swelling in failure mechanism by using only simple Mohr-Coulomb model.

The simulation of the slope stability was conducted by using FEM analysis (SIGMA/W). The most crucial aspect for the design of a cut slope is the determination of the safe slope inclination for the long-term stability based on SNI 8460:2017. Residual parameters were used for the clay-shale shear strength parameter. The stability calculation was simulated for 5 stages of excavation. Each excavation stage was simulated using 10-time steps for a duration 10 days. The model geometry was simplified and idealized as two main layers composed of a clay-shale layer and claystone layer.

For the initial stress calculation, the groundwater level was set at a depth of 2 meters. The SIGMA/W calculation using the coupled-stress/pore water pressure change method requires specific values for the volumetric water content (VWC) and hydraulic conductivity function. The VWC is the critical parameter of unsaturated soil mechanics due to its role to describe the yield stress–suction and shear strength–suction relationships (Sheng, 2011). According to Chao, et al. (2014) the VWC has relationship with water content change during wet/dry cycle. The VWC is obtained by using the clay sample function. The hydraulic conductivity was set based on the Van Genuchten equation with a saturated conductivity of 10-9 m/s for the claystone and 10-8 m/s for the clay-shale.

The clay-shale softening was simulated by using the elastic-plastic Mohr-Coulomb model as it can describe elastic and plastic deformations clearly (Li, Vanapalli, & Li, 2016). For the initial stage calculation, the peak shear strength was used to describe the short-term stability of a proposed slope inclination design. The simulation was continued using the parameters in Table 5. During the excavation stage, the clay-shale (weathered layer) used the shear strength parameter suggested by Gartung (1980).

After the 5th excavation, the swelling process was simulated for 10 years. The pore water pressure was assumed and maintained at -20 kPa along the surface boundary. The slope stability calculation was conducted by using finite element stress Slope/W at the end of the swelling simulation. The final stage of the simulation conducted the stability calculation with the residual parameter by using the strength reduction method. The simulation stages are listed on Table 6.

The slope stability simulation was conducted for 5 proposed slope inclinations. The slopes were 1V:1H, 1V:2H, 1V:3H, 1V:4H, and 1V:5H in the simulation. All the proposed slope inclination safety factors were checked for all conditions (e.g., the EoS, swelling, and residual condition). Therefore, the simulation can suggest the most suitable inclination design for a slope cut from weathered clay-shale.

Table 5. Shear strengths for the FEM stability simulation

Strata	Friction angle ϕ' (degree)	Cohesion c' (kPa)	Elastic modulus (kPa)	Poisson ratio υ	Unit weight saturated (kN/m3)	Unit weight Dry (kN/m3)
Weathered layer (peak strength)	20	60	9000	0.2	17	18
Weathered layer (residual)	6.00	6.50	4000	0.2	17	18
Weathered layer (fully softened)	6.00	17.00	4000	0.2	17	18
Weathered layer (design strength)	20	20	8000	0.2	17	18
Claystone	25.00	100	15000	0.3	17.5	19

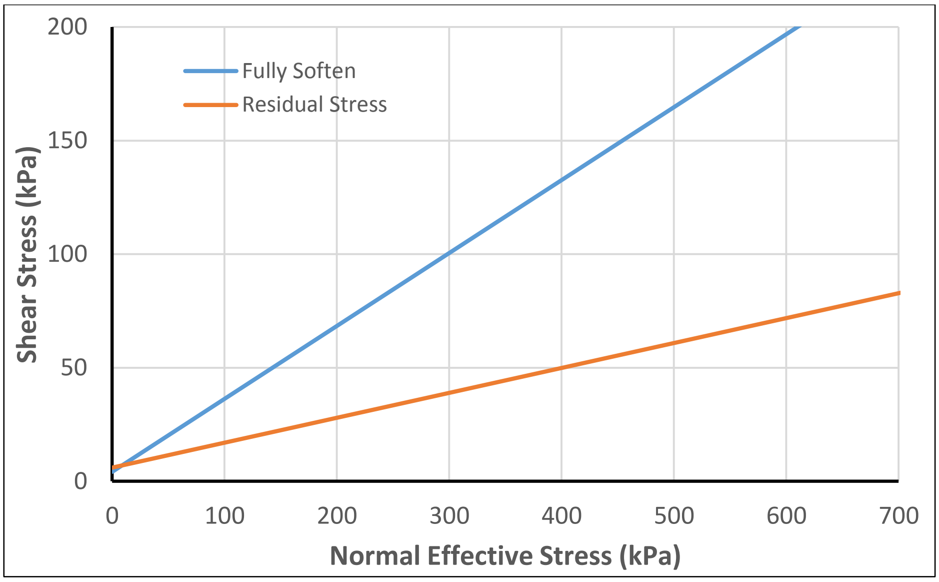

Fig 6. Residual strength correlation by LL dan CF based on Stark and Hussein (2013)

3. Result and Discussion

The short-term calculation results satisfied the safety factors for the slope stability. The peak strength parameter during excavation analysis provided a safety factor value at the EoS stage that was too optimistic. For example, the safety factor at the 1V:1H inclination was 1.62. From the 1V:1H simulation result, it can be predicted that the safety factor would be more satisfied for a gentler slope. The slope stability simulation results are listed in Table 7 and Figure 7. According to Renania & Martin (2020), stability simulation using peak strength can produce over optimistic safety factor for strain-softening soil.

Table 6. Simulation stages for slope stability analysis STA 438

Stage	Analysis Type	Duration	Remarks
Proposed Existing Design	Undrained Behaviour		Short term simulation
Initial Stress	In situ	0 day	Initial condition
1st Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
2nd Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation
3rd Excavation stage	Coupled stress/ PWP change	10 days	2-meter excavation depth
4th Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
5th Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
6th Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
7th Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
8th Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
9th Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
10th Excavation stage	Coupled stress/ PWP change	10 days	2 meter excavation depth
Swelling phase	Coupled stress/ PWP change	3650 days	
Slope Stability FoS	Finite element stress	All time steps	
Residual strength	Stress redistribution	Last step	
Slope Stability Residual FoS	Finite element stress	Last step	

In Table 7, it can be seen that the optimum design for the slope inclination is 1.00V:1.00H. The safety factor for this 1.00V:1.00H design satisfies the minimum FoS > 1.50 criterion. The use of a gentler slope inclination is inefficient in terms of construction budget and time. However, the peak strength parameter is not recommended for use in the long-term stability of clay-shale slopes. The weathering and swelling phenomena must be considered when determining the optimum design for clay-shale slope inclinations. According to Qi & Vanapalli (2016), ignoring the softening parameter may lead to unstable slope design especially for expansive soil.

In Table 7, it can be seen that the optimum design for the slope inclination is 1.00V:1.00H. The safety factor for this 1.00V:1.00H design satisfies the minimum FoS > 1.50 criterion. The use of a gentler slope inclination is inefficient in terms of construction budget and time. However, the peak strength parameter is not recommended for use in the long-term stability of clay-shale slopes. The weathering and swelling phenomena must be considered when determining the optimum design for clay-shale slope inclinations.

Table 7. Slope stability simulation results by using the peak strength parameter

Slope Inclination	Safety Factor
1.00V : 1.00H	1.50
1.00V : 2.00H	1.83
1.00V : 3.00H	2.17
1.00V : 4.00H	2.64
1.00V : 5.00H	3.07

The simulation continued to calculate the clay-shale slope stability considering the weather and swelling conditions. For this stage, the peak strength parameter turned into the design strength, as suggested by Gartung (1970). The stability simulation resulted in a lower safety factor than the peak strength. The 1.00V:1.00H slope inclination had the minimum FoS of 1.11 at the EoS or after the initial swelling time. However, at the end of the swelling simulation, the FoS decreased to 1.02. For the 1V:5H inclination, the FoS decreased from 2.27 to 2.01. This phenomenon also occurred for the other slope inclination design. The decrease in the FoS as a function of time is shown on Figure 8.

As shown in Figure 8, the slope safety factor decreased gradually along with the swelling time. It seems that the minimum design for the slope inclination is 1V:3H, as the safety factor meets the minimum criterion (FoS > 1.50) at the end of the swelling time. However, the swelling simulations still used the design strength, not the fully softened and residual strength. By using the residual and fully softened parameters, the safety factor was calculated based on the pore water pressure and deviatoric strain. The slope stability simulation results with the residual strength are listed in Table 8.

The critical slip surface was obtained based on the deviatoric strain. As shown in Table 8, the stability factor increased as the slope inclination decreased if the fully softened parameter was used. However, the stability factor changed when the shear strength parameter was reduced to the residual strength. For a slope inclination from 1.00V:1.00H to 1:00V:5.00H, the stability had a similar tendency in the critical safety factor range. The simulation stability for a slope inclination from 1.00V:1.00H to 1:00V:5.00H did not reach the minimum FoS. All the proposed slope inclinations failed for that FoS if the residual parameter was used.

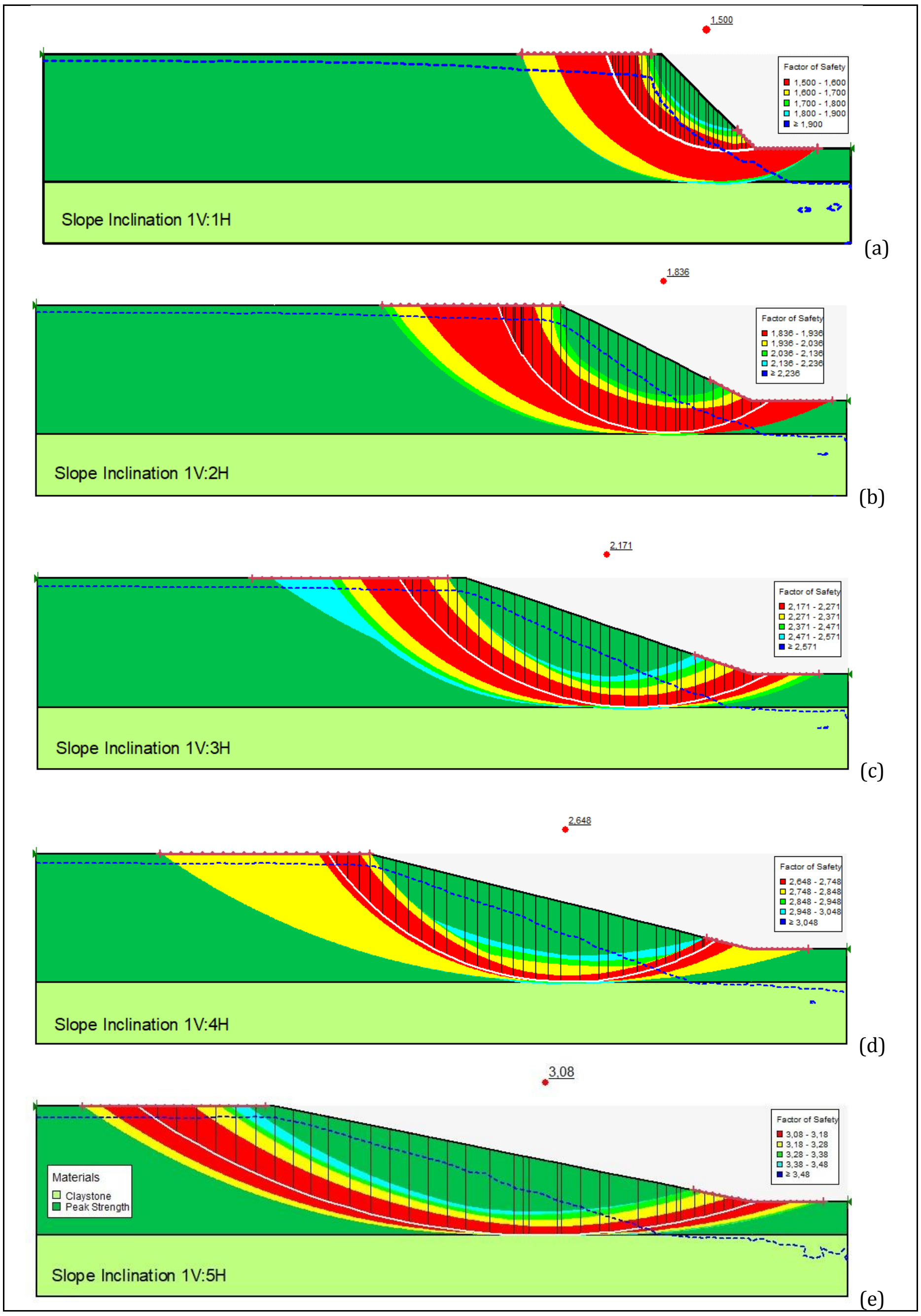

Fig 7. Result simulation by using the peak strength parameter: (a) Slope failure plane 1.00V:1.00H inclination; (b) Slope failure plane 1.00V:2.00H inclination; (c) Slope failure plane 1.00V:3.00H inclination; (d) Slope failure plane 1.00V:4.00H inclination; (e) Slope failure plane 1.00V:5.00H inclination;

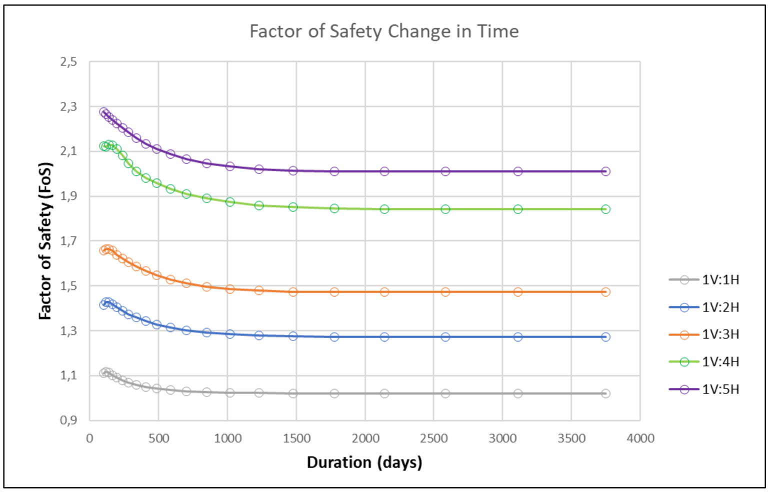

Fig 8. FoS change during the swelling phase

Table 8. Slope stability simulation results by using the peak strength parameter

Slope Inclination	Fully Softened Critical Safety Factor	Residual Critical Safety Factor Range
1.00V : 1.00H	0.69	0.46 – 0.56
1.00V : 2.00H	0.77	0.48 – 0.58
1.00V : 3.00H	0.83	0.46 – 0.56
1.00V : 4.00H	1.03	0.46 – 0.56
1.00V : 5.00H	1.16	0.47 – 0.57

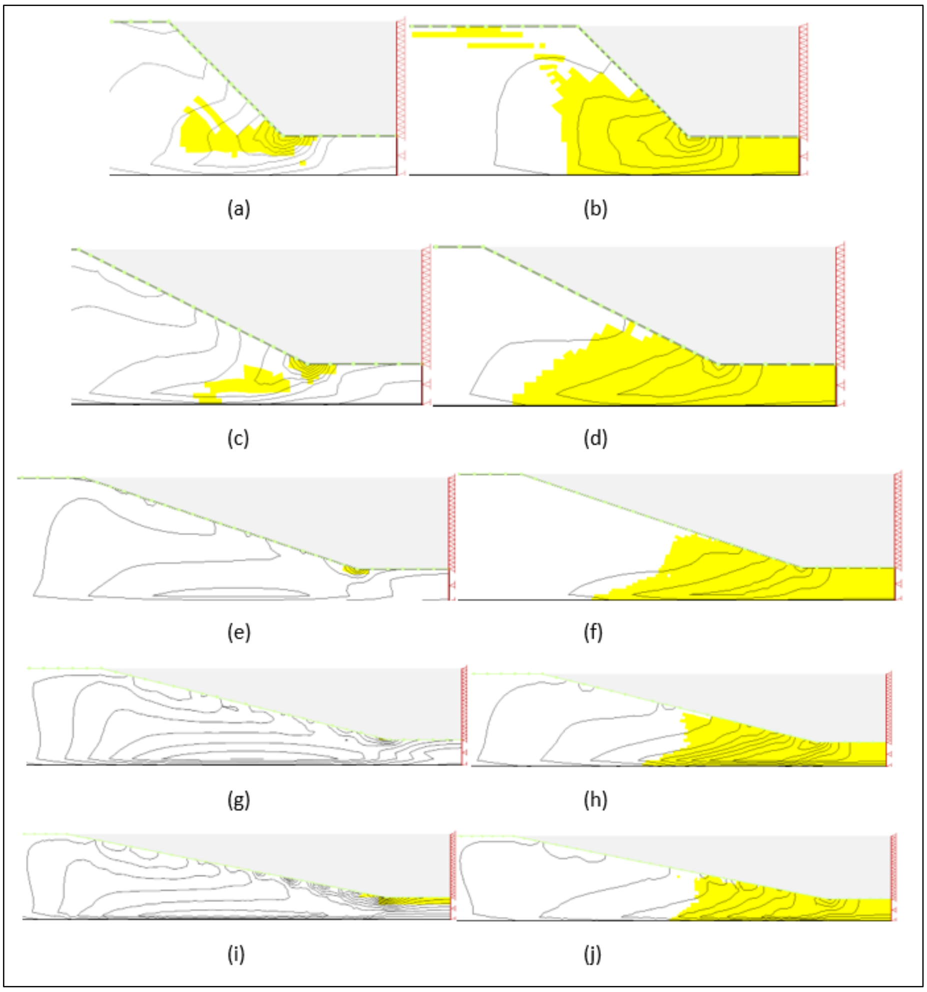

Fig 9. Yield zone propagation on slope Sta 438: (a) 1.00V:1.00H at EoS, (b) 1.00V:1.00H swelling, (c) 1.00V:2.00H EoS, (d) 1.00V:2.00H swell, (e) 1.00V:3.00H EoS, (f) 1.00V:3.00H swelling, (g) 1.00V:4.00H EoS, (h) 1.00V:4.00H swelling, (i) 1.00V:5.00H EoS, and (j) 1.00V:5.00H swelling.

The plastic yield point can be used to identify the propagation of a slope failure zone. At the EoS, the yield (failure) zone was located near on the excavation base. This means that no failure or yielding occurred at the construction stage. Hereinafter, the yield zone changed when the swelling phase occurred. The yield zone propagated deeper and spread wider into the slope (Figure 9). The yield or failure zone propagation decreased the stability factor during the construction stage and swelling phase.

Similar to the swelling stage, the yield zone also propagated when the fully softened and residual parameters analysis was done. The zone spread deeper and wider than the stage before. At the fully softened simulation stage, the yield zone spread to over half of the entire slope. Hereinafter, the yield zone spread throughout the entire clay-shale region at the residual simulation. The soil deterioration also contributes to long-term reduction of stability factor (Postilla, et al., 2021). All the proposed slope inclinations did not reach the minimum safety factor (FoS < 1.00) for this condition.

The simulation may not be used to predict the time to failure of a slope. However, the simulation is intended to investigate the swelling and shear strength reduction influence on the progress of slope failure. The failure occurred simultaneously and was caused by the pore water pressure equilibration that occurs after the construction ends. Based on the recent condition, the failure still occurred even when the slope was gentler. This may have been caused by a decrease in the clay-shale residual strength. The reduction zone occurred deeper so that the failure continued until the equilibrium state (FoS > 1.00) was reached.

From the stability simulation results, it may be concluded that it is important to control the pore water pressure change after excavation is done. The swelling phase occurs as result of pore water pressure equilibration and stress release due to excavation. If the pore water pressure along the slope cannot be maintained at a certain level, the safety factor will continue to decline. However, it is also important to prevent further softening of clay-shale. During the simulation, if the entire clay-shale strength changed to the residual parameter, the slope inclination must be very gentle to be stable.

4. Conclusion

Slope stability analysis for a degradable material requires a special approach to investigate the long-term stability. Clay-shale stability may have a satisfactory FoS at the end of construction. Due to the expansive behavior, the FoS decreases as result of pore water pressure equilibration and stress release due to excavation. The simulation may not be used to predict the time to failure, but it can be used to investigate failure behavior and the possibility of slope progressive failure. By understanding the clay-shale slope condition and behavior, the proper design and reinforcement can be determined. For this case, the minimum suggested slope inclination is 1.00V:5.00H.

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