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Remedial design of failed embankment founded on clay shale in east kalimantan, Indonesia

Lailatin, & Nabila Laras
PT PP Tbk, Indonesia

Hendra Jitno
Geotechnical Engineering Lecturer, Indonesia

Fitri Pratiwi
PT Adhi Karya Tbk, Indonesia

ABSTRACT: During construction of a 15-meter high embankment in Balikpapan - East Kalimantan, a section of the embankment at STA 2+905 failed following a prolonged rainfall. The failed embankment was subsequently repaired by re-excavating and re-compacting it to its original geometry. One year later, however, extensive cracks re-appeared at this section, leading to the failure of the adjacent box culvert. The second remedial work was carried out by installing mini piles to reinforce the foundation at the failed section. Yet, the cracks and slips still occurred at this section.
Geotechnical investigation was undertaken to ascertain the characteristics of the soil at this area. Based on the results of new geotechnical investigation, numerical analyses have been performed to find possible causes of the slips. We found that failures were due to significant loss of bearing capacity of the embankment foundation, related to the wetting of clay shale beneath the embankment during prolonged rainfall. The existence of box culvert possibly contributed to the source of water which caused strength reduction of the clay shale layer. In addition, the clay shale layer is thought to have been degraded due to prolonged exposure during the excavation for the culvert construction. This paper presents the remedial design of the embankment section adopting the residual strength of the clay-shale to obtain a robust design solution.

Keywords: Slope Stability, Embankment, Numerical Analysis, Clay shale

1 INTRODUCTION.

The 15-meters high road embankment considered in this paper is located in Balikpapan, East Kalimantan. The embankment foundation comprises sandy clay with moderate to hard consistency, overlying very stiff clay shale underlain by a relatively young coal layer. There are several box culverts installed in this section to manage surface runoff from the upper catchment area of the road. During culvert installation, ground cracks were observed at the culvert excavation area. The ground cracks re-appeared in this section a few months after this section was completed, leading to the failure of this section. to rectify the problem, the failed section was re-excavated and recompacted to its original geometry. However, one year later, extensive cracks were again observed at thies section with greater affected areas, leading to the embankment failure and broken culvert. After the second failure, rehabilitation work was conducted at the landslides area by installing mini piles to strengthen the embankment foundation. Yet, the cracks and landslides were again observed in this section. Soil investigation was carried out to get more understanding on the characteristics of the foundation soil beneath the failed embankment section.

Based on the result of recent geotechnical investigation and other available information, numerical analysis has been performed using RS2 to find the possible causes of the slips. Back analyses were conducted to determine material strength at the failed slope. The soaked and residual strength of the clay shale were obtained by back-analysis at the failed section. Soil reinforcement by means of bored piles were proposed to prevent the slope failure from reoccurring. This paper presents the results of failure investigation, back analysis, proposed remedial work to prevent the slope failure from reoccurring and recommendation of residual clay shale strength parameter as the analyses result from other studies.

2 MATERIALS AND METHOD

2.1 *Subsurface Geotechnical Condition*

Balikpapan is located at Balikpapan Formation consisting of interlayered quartz sandstone, siltstone and shale with limestone and coal layer insert. Limestone contains fossils of the upper middle miocene age. Deposition environment in shallow littoral regions with a thickness of 800 meters. The soil investigation carried out at site confirms the existence of a stiff shale layer underlain by a thin hard black coal layer. Typical subsurface geotechnical condition at the location of the failed embankment section consist of medium to stiff CLAY of high plasticity, underlain by dark grey stiff to hard silty CLAY SHALE above black hard coal layer and underlain by hard dark-grey silty CLAY SHALE. As shown in Figure 1, the SPT N values for the grey very stiff clay SHALE above the coal layer was ranging from 19 to 27. This was the layer that likely experienced strength reduction which lead to embankment failure.

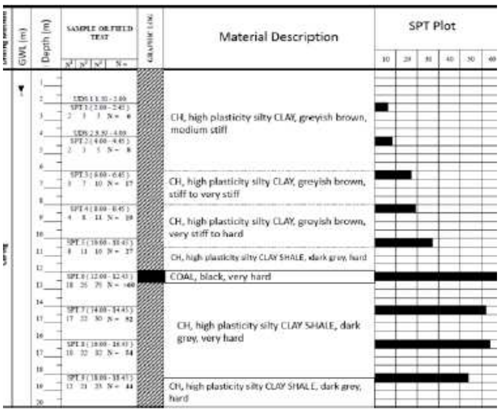

Figure 1. Typical subsurface geotechnical condition beneath the failed embankment section

2.2 *Chronology of The Failure*

Embankment failure generally occurs due to a combination of multiple causes. We believe the failure of this section was also due to a combination of many factors. To understand the root cause(s) of this failure, the chronology of the failure and the corresponding rainfall intensity at the area are provided in table 1.

Table 1. Timeline of Highway Embankment Failure in Balikpapan

Date	*Description*	*Rainfall (mm)*
24 Feb 2017	1st ground cracks observed during culvert excavation	50mm/previous 7 days
20 Aug 2017	Cracks observed on the completed embankment between 2 culverts, about 5-m from the edge	230mm/previous 13 days, average 18mm/day
26 Aug 2017	Excavation of the failed section and monitoring	
17 Feb 2018	Embankment re-constructed to original level	-
17 Mar 2018	New cracks observed at the embankment toe between two culverts	90mm day before
21 Mar 2018	Counter weight/berm was constructed to prevent slope failure	
18 May 2018	Cracks re-appear	95mm/previous 5 days
21 May 2018	Cracks widened and one of the culverts broken	
25 Aug 2018	Aimolement rehabilitation work using mini piles	
25 Sept 2018	Cracks re-appears and the culvert broke again	30mm of rainfall in one month

Figure 2. 1st ground cracks observed during culvert excavation (24 Feb 2017). Yellow dashed lines indicate the design road alignment.

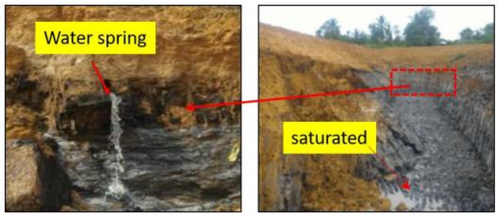

Figure 3. Hard dark grey silty clay shale at the culvert foundation exposed during the excavation. Note the seepage coming from the upstream side of the culvert.

2.3 *Stability of Embankment Founded on Clay Shale*

Previous researcher show that shale is very strong when it is subjected to high confining stress and protected from exposure to weather (e.g. Stark and Duncan, 1991) ; Irsyam et al, 2016). However, when the overburden stresses are remove by excavation and the shale layer is exposed to weathering, its strength deteriorates with time. The strength reduction accelerate if it is in contact with water due to softening.

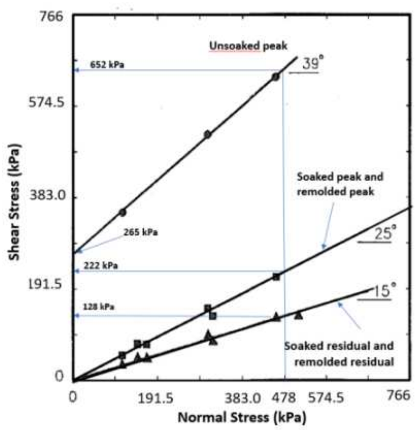

Figure 4. Effect of soaking on the strength parameters of very stiff clay from San Luis Dam downstream foundation (after Stark and Duncan, 1991)

Stark and Duncan (1991) tested stiff-fissured clay obtained from downstream of San Luis Dam, California, which failed during drawdown in 1981. The results of the direct shear tests on dry undisturbed samples indicate that the clay was very strong with c=265kPa and friction angle of 39°, as can be seen in Figure 4. However, when the samples were soaked, the strength reduced to c=0kPa and friction angle of 25° for peak strength and 15° for residual strength. This is equivalent to over 2/3 strength loss from 652kPa (peak) to 222kPa (soaked) and 128kPa (residual) for normal stress of 478kPa (blue line in figure 4).

2.4 *Geotechnical Model and Analysis*

The back analysis were carried out using a finite element computer program RS2 (Rocscience, 2019). RS2 is general two-dimensional finite element program capable of undertaking static and dynamic stress deformation analysis for general geotechnical problems including dam, tunnel, open pit, etc.

The geotechnical model, the embankment geometry before 1st failure and the finite element mesh at STA 2+905 are shown in Figure 5. Soil layers and the location of water table were developed based on the results of borehole and data before and after slope failure. The soil parameters for the compacted fill, clay shale, sandy clay, and coal layer were obtained from the results of laboratory data and available correlations applicable for the soil at the site. The soil parameters used in the analysis ate presents in table 2.

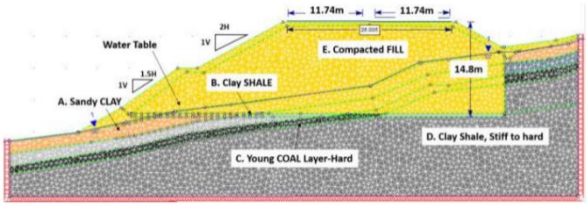

Figure 5. Embankment geotechnical model and FE mesh for case 1 (before 1st failure)

Piles were modelled as elastic beam element so the moment, shear, and axial forces on the piles can be computed. Note that this elastic assumption may give the computed moments that are higher than the pile ultimate moment capacity, without showing indication of pile failure. The structural parameters used in the analysis are presented in Table 3.

Elastic-plastic stress-strain model with Mohr-Coulomb failure criterion under plane-strain condition were adopted in the analysis. Plane strain assumption is considered realistic since the embankment length is much longer than the embankment width.

To obtain a suitable solution, appropriate prediction of shear strength parameters of the soaked and remolded clay shale is very important. The result of laboratory test for undisturbed clay shale are available, but no laboratory test on the soaked and remolded clay shale were available to guide back analysis at the time of writing. Fortunately, the failure mechanism and the time when the failure occurred was well reordered. Hence the strength parameters can be estimated from the back analysis of embankment failure.

Table 2. Soil parameters used in the analysis

Layer Number	*Soil Type*	*γ (kN/m³)*	*c' (kPa)*	*ϕ (°)*	*E (kPa)*	*u*
E	Fill Material	19	10	32	30000	0.3
A	Sandy Clay	19	5	35	20000	0.3
B	Clay Shale (upper layer)	17	50(p) 0(s) 0(r)	25(p) 22(s) 15(r)	30000 3000 1000	0.3
C	Young Coal Layer	13.5	100	35	100000	0.3
D	Clay Shale (lower layer)	18.5	50	25	40000	0.3

Table 3. Cases considered in the analysis

Case	*Description*	*Strength parameter used*
Case 1	Condition before failure	Laboratory and empirical correlations from SPT N value
Case 2	Condition during 1st failure-no reinforcement	Back-analysis, soaked strength; c=0kPa ; ϕ=22°
Case 3a	Reinforced with prestressed concrete pile	Clay shale soaked strength; c=0kPa; ϕ=22° PC pile 200x200; E_c=25GPA; M_{crack} =15.5kNm; M_{ult}=26.5kN)
Case 3b	Condition during 2nd failure+PC piles	Clay shale residual strength; c=0kPa; ϕ=15° PC pile 200x200; E_c=25GPA; M_{crack} =15.5kNm; M_{ult}=26.5kN)
Case 4	Proposed bored piles+berm reinforcement	Clay shale residual strength; c=0kPa; ϕ=15° 3 rows bored piles D=1m; E_c=25GPA; M_{crack} =600kNm; M_{ult}=850kN)

3 RESULT OF ANALYSIS

3.1 *Numerical Analysis Result*

The result of safety against failure of the original embankment at STA 2+905 was analyzed using Strength Reduction Method (SRM) using RS2 software (Rocscience, 2019). The analysis were conducted using the peak strength of all materials comprising the dam, with clay shale layer strength of c' = 50kPa and ϕ = 25°. The calculated factor of safety of the embankment section for the original condition was around 1.45, with potential failure surface as shown in Figure 6.

The clay shale strength was then reduced to soaked strength parameters to simulate the actual slope failure where cracks were observed at the top of embankment. The results are presented in Figure 7. As shown in the figure, the slope failure zones developed from the top of the embankment and extends down to the toe of embankment, which agree quite well with the field observations. Results of back analysis indicate that the shear strength parameters of the soaked clay shale layer at failure (FOS~1) were c'=0 kPa and ϕ = 22°.

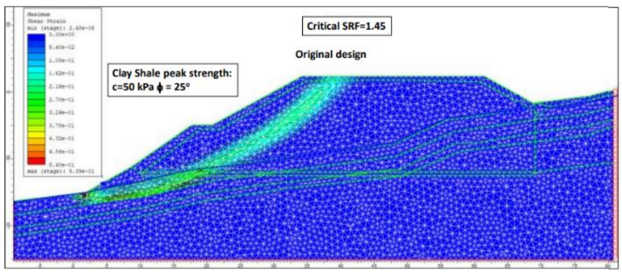

Figure 6. Computed potential failure surface for Case 1. (Clay shale peak strength of c=50kPa; ϕ = 25°; FOS=1.45)

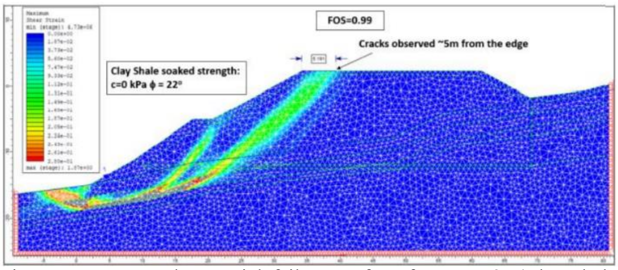

Figure 7. Computed potential failure surface for Case 2. (Clay shale soaked strength of c=0kPa; ϕ = 22°; FOS=0.99)

The next back analysis were carried out to find the clay shale strength after the slope was rehabilitated and reinforced with 200x200 prestressed concrete (PC) piles @0.75m c/c, at depth of 6-m and 8-m. As shown in Table 1, one month after the installation of PC piles and the embankment was constructed to its original level, ground cracks were observed again at the embankment surface, at about half width of the embankment.

The installation of PC piles took several days. This prolonged exposure of the clay to weather (also rain) and disturbance of the clay layer at the surface due to construction equipment during pile installation have likely caused further degradation of the clay shale layer strength. Hence while the reinforcement was intended to increase the embankment stability, the disturbance and exposure to weather and rain have caused the clay shale to reach its remolded/residual strength.

When the soaked strength of c' = 0kPa and ϕ = 22° was assumed dor the clay shale strength, the computed slope factor of safety was 1.29, which was theoretically stable (Figure 8). However, the computed maximum bending moment in the piles was 47.7kNm, which was much higher than the ultimate moment capacity of the piles of 26.7 kNm, suggesting that the piles were already broken when the fill was re-constructed to its original design level. Hence the piles were not able to withstand the embankment weight and started to deform.

When the shear strength was further reduced to its remolded strength with c'=0kPa and ϕ = 15°, the computed factor of safety was dropped to 0.99 (Figure 9). As shown in the figure, the zones of slope failure developed all the way from the top of the embankment to the area up to 10 meters away from the embankment toe. The grater affected slip area appears to be caused by the pile reinforcement in the pile reinforcement was able to mobilize larger area to provide more resisting forces, the PC piles were not strong enough to withstand the forces exerted by the embankment when the clay shale deforms due significant loss of its original strength and stiffness.

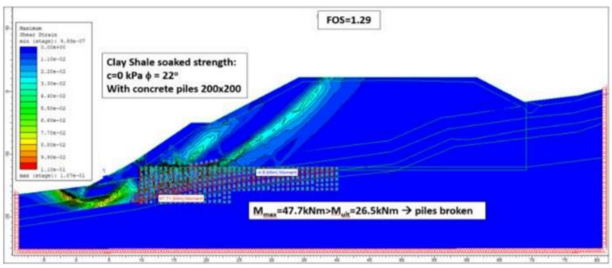

Figure 8. Computed potential failure zones for Case 3a, PC piles 200x200 reinforcement. (Clay shale soaked strength of c=0kPa; ϕ = 22°; FOS=1.29)

The back-calculated residual strength parameters were similar to those found from literature, such as those reported by Skempton (1977), Stark and Duncan (1991) which showed friction angle of 13° – 25° for stiff clays. Irsyam et al. (2006) also reported similar back calculated residual strength value of c=5kPa and ϕ = 13° for the weathered clay shale underneath the failed section of Cipularang Toll Road Km 97+500.

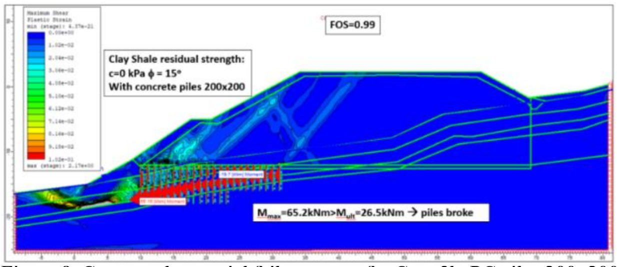

Figure 9. Computed potential failure zones for Case 3b, PC piles 200x200 reinforcement. (Clay shale soaked strength of c=0kPa; ϕ = 15°; FOS=0.99)

3.2 *Proposed Soil Improvement Design*

A series of bored piles was selected as the most suitable solution to improve the safety factor of the embankment system mainly due to its minimum disturbance to the clay shale layer during construction. In addition to the bored piles, a stabilizing berm was needed to prevent local failure of the toe. PC piles would consists of 3 rows of 16m-deep bored piles, installed @2m center to center perpendicular to the failure plane, 17 rows of 6-m deep 400x400 PC piles @1.5 center to center and stabilizing berm. The broken piles on the culvert foundation were not taken into account in the analysis.

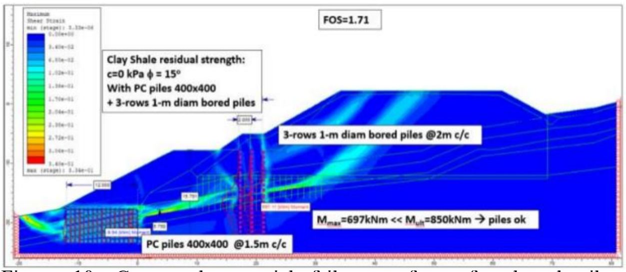

Figure 10. Computed potential failure surface after bored piles installation. (FOS=0.99)

The results of finite element analysis in Figure 10 shows that the bored piles configuration has improved the global safety factor to 1.71, even using clay shale residual strength parameter. The computed moments in the bored piles and the PC piles were less than the moment capacity of the piles. The figure also suggests that the potential failure surface developing along the previous failure surface.


Figure 10. Bored piles construction

Figure 11. Progress of soil reinforcing using bored pile, PC piles and berm

3.3 *Clay Shale's Strength Parameter*

From the analysis above we know there are various way to affect clay shale shear strength. Shear strength degradation can be identified by its behavior. Bambang Pradoyo (2020) conducting a study about the behavior of clay shale on slope stability from slake durability test and swelling test through 6 samples and resulting the number of shale rating. And this behavior is likely to be affected by plasticity index value (PI) and point load strength. The higher shale rating, the steeper angle of natural slope may be reached. Shale rating next could be further analyzed because in the study, Jon A. Franklin (1981) mention that as shale rating increases the shale fill becomes progressively more frictional.

Meanwhile, the hydration process to clay shale influence the mechanical properties. According to Liu, Xiangjun (2019), with increasing soaking time, hydration damage become stronger, and forming fracture initiation on clay shale. And this is an evolutionary process which has three stages, which are initial stage with a slow increment, middle stage with a rapid increment, and later stage with stabilization. In stress-strain curves in hydration damage evolution shows that the hydration can decrease shale compressive strength and brittleness and increase shale plasticity.

From the study of similar case, Botts (1986) observed nearly a 75% drop in shear strength of Pierre Shale samples, or 6-degree drop in the internal friction angles and a reduction of cohesion from 848kPa to 0kPa after one wetting and drying cycle. The residual strength of Pierre Shale then reobserved by Schaefer (2013) that the low residual friction angles (under 6°) tend to decrease an additional 0.8°-1.4° during the wet/dry cycles.

The effect of weathered by wetting-drying process in Indonesian Clay Shale sample on shear strength reduction (residual friction angle and cohesion) was studied by Idrus M. Alatas et al (2016). The study was conducted for clay shale sample in Semarang (Central java) and Hambalang (West Java). The total percentage of its clay minerals (smectite, illite, kaolite, chlorite) content around 56%-62%. The samples were weathered by wetting and drying up to 80 days. Both cohesion and friction angle were decrease as the increased of the frequencies of wetting-drying cycles as shown in Figure 12 and 13.

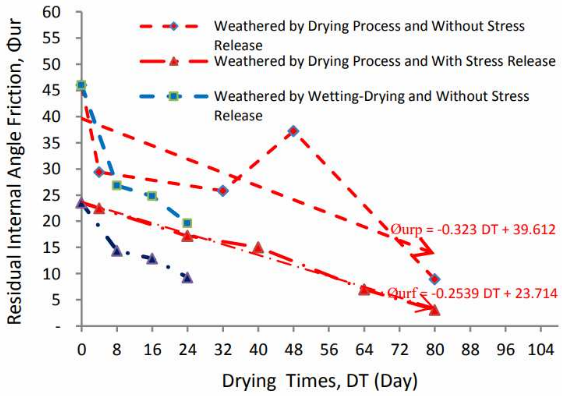

Figure 12. Residual internal friction angle reduction of unsaturated Semarang-Bawen clay shale due to natural drying and wetting cycles (soaked twice in every 8 days) (Idrus, M. Alatas).

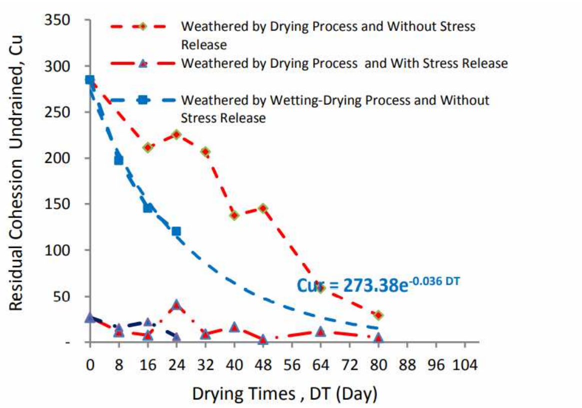

Figure 13. The residual cohesion reduction of unsaturated Semarang-Bawen clay shale due to natural drying and wetting drying cycles (soaked 2 times in every 8 days) (Idrus, M. Alatas).

4 SUMMARY AND CONCLUSIONS

This study aims to finding the most suitable embankment's reinforcement design to stand on the weathered clay shale soil. The study takes advantage of using FEM software so we could perform back calculation analysis more easily, which can change the strength parameter and look up to response of the failure plane location precisely. The main findings of this study include:

- The cracks development and slips of road embankment at STA 2+905 in Balikpapan was likely due to combinations of several factors including the existence of clay shale layer, extended period of culvert construction, and frequent rainfall. The clay shale lost significant strength and stiffness likely due to prolonged exposure to weather and rain, and disturbance during culvert construction and during embankment rehabilitation after the first slip. The results of back analysis indicate that the clay shale layer lost about 30% of its strength due to soaking, and more than 50% of its original strength due to soaking plus disturbance (remolding). The initial clay shale strength was c=50kPa and ϕ = 25° and reduced to c=50kPa and ϕ = 22°, for soaked strength, and further reduced to c=0kPa and ϕ = 15° for residual/remolded strength. This friction angle and cohesion of soaked and residual state value is close to a value which is recommended by Duncan and Stark (1981). The reduction of friction angle and cohesion value is highly affected by wetting-drying frequencies and duration. A further research is necessary for knowing the effect of longer wetting-drying duration (more than 80 days).

- A group of bored piles was selected as the most suitable solution to increase the global safety factor of the road embankment. The proposed design consists of 3 rows of 16-m deep 1000mm diameter bored piles @2m center to center, with stabilizing berm reinforced with 6m deep 400x400 PC piles. Due to further technical judgement of consultant, it modified into 1-row bored piles. Care must be exercised during the installation of the piles to prevent disturbance or wetting on the clay shale. The result of analysis using RS2 software (Rocscience, 2019) using remolded strength of the clay shale layer indicate that the new remedial work would be stable with computed factor of safety of 1.7.

5 ACKNOWLEDGEMENTS

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