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Analysis of Embankment Slope Failure And Effectiveness Of Reinforcement With Full Displacement Column on Soft Soil (Case Study : Construction Of Padang Sicincin Toll Road Section STA 7+400 – 7+550)

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ABSTRACT

The package that is currently under construction on the Trans Sumatra toll road network section Padang - Pekanbaru is the Padang - Lubuk Alung - Sicincin section which is targeted for completion in July 2024. One of the obstacles in the work process is that there is an embankment landslide at STA 7 + 450 which is indicated due to a decrease in the subgrade. This resulted in the construction process not being able to continue so there was a delay in the progress of construction implementation. Therefore, a study is needed that focuses on analyzing the causes of landslides and follow-up recommendations to overcome problems in the STA 7 + 450 embankment work on the Padang - Pekanbaru Toll Road Construction project section Padang - Lubuk Alung - Sicincin. The method used in this research is finite element modeling analysis using the help of the PLAXIS 2D program by looking at the Safety Factor value of the embankment slope under initial conditions and back analysis. The landslide modeling results with the finite element method are also compared with the landslide lines that occur in the field. From the back analysis, new parameters are obtained when a landslide occurs as input in the reinforcement analysis to be used. For follow-up improvement, full displacement column reinforcement is proposed where the effect of FDC column spacing, thickness, and load transfer platform material combination on slope stability, settlement, and changes in soil mechanical parameters will be analyzed. It was found that the embankment structure before the collapse had a critical SF value of SF = 1.010. The cause of the landslide was indicated to be due to the additional load from the project vehicle traffic. Parameter changes were obtained in the form of c' = 8.17 kN/m2 and Ø' =19.68^{o}, cc = 0.8175, and cs =0.1204 in the organic soft soil which was indicated to be the cause of the landslide. From the modeling results to find the effectiveness of reinforcement with FDC, the optimum parameters were obtained concerning a distance of 3 times the column diameter, and a Load Transfer Platform thickness of 1.8 meters with geogrid reinforcement.

Keyword : Embankment, Back Analysis, Full Displacement Column, Soft Soil

INTRODUCTION

One of the main sections in the Trans Sumatra Toll Road Network Development Project is the section connecting Pekanbaru and Padang. This toll road is part of the supporting corridor of the Trans Sumatra Toll Road which will improve connectivity between Riau Province and West Sumatra Province. The presence of this toll road is expected to become a logistics route and tourism route that can support economic development in the two provinces. One of the work packages that is in the process of implementing construction on this network is the Padang - Lubuk Alung - Sicincin Toll Road Project. This section is located in West Sumatra Province according to Figure 1. The section is 36.6-kilometer stretches from Padang City to Sicincin, Padang Pariaman Regency. The land acquisition position for this section has reached 85 percent so construction can be accelerated with a completion target of July 2024.

Figure 1. Project Location of Trans Sumatra Toll Road Padang - Pekanbaru Section (PT. Hutama Karya, 2023)

One of the problems in the construction process in this package is the landslide of the preloading embankment at STA 7+450 on the right side, which is indicated due to a decrease in the embankment subgrade. This landslide resulted in a construction process that could not continue, resulting in a delay in construction progress. This landslide caused a delay in the construction progress because handling had to be carried out on the embankment that experienced the landslide. In addition, the embankment is one of the access roads from small to larger stationing that has potentially been late due to the landslide that occurred. Based on the documentation in Figure 2 and Figure 3, there are cracks on the top surface of the embankment and there is heave/bulging at the end of the right embankment toe that indicated embankment was collapsed.

Figure 2. Cracks on the top surface of embankment STA 7+450 R

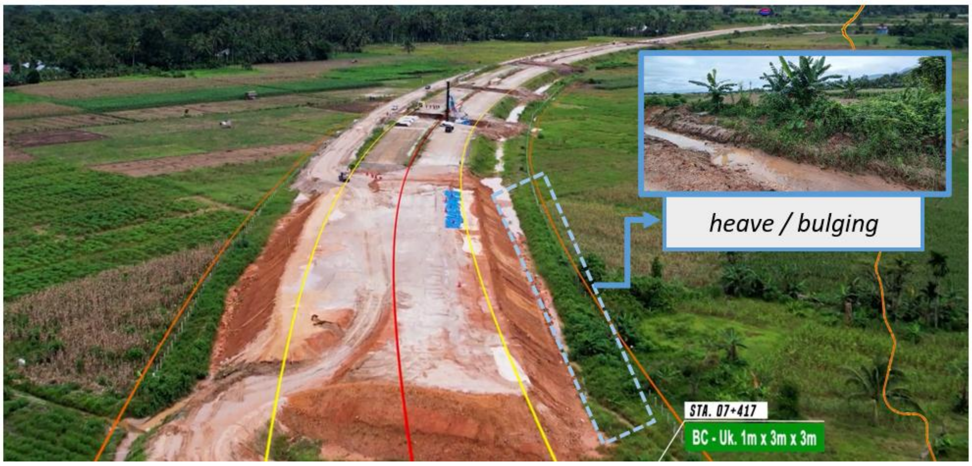

Figure 3. Heave at toe of embankment STA 7+450 R

The results of the subgrade investigation using deep drill testing (bore log) showed the N-SPT value of the soil is 4 at a depth of 0 - 3 m which is organic soil, then for a depth of 3 - 5 m the N-SPT value = 6 with the interpretation of silty sand soil. Meanwhile, for a depth of 5 - 7 m, the N-SPT value = 9 with the silty clay soil interpretation, then for a depth of 7 - 9 m, the N-SPT value = 35 with the silty clay soil type interpretation. For depths above 9 m, the N-SPT value > 40 is obtained with the interpretation of clayey sand soil type. Data from geotechnical instrumentation installed in the field, the piezometer, and the inclinometer also indicate the collapsed subgrade. From the piezometer data obtained in the period December 22, 2022 - February 28, 2023, the value of pore water pressure variation was higher than the preloading load given. Meanwhile, the inclinometer data obtained until March 02, 2023, showed a lateral movement of 771.75 mm. The instrumentation monitoring data can be seen in the figure 4.

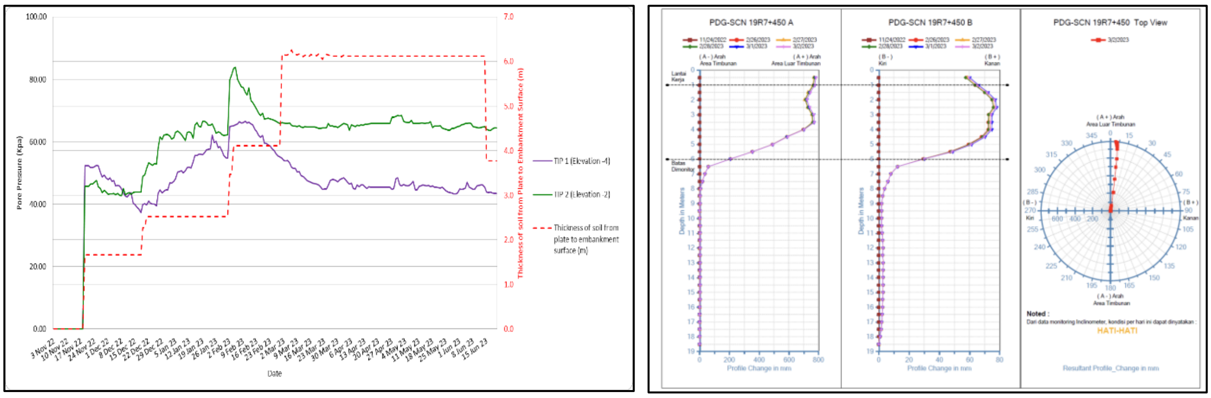

Figure 4. Data monitoring from Piezometer (left), inclinometer (right)

Based on Manudianto, et al., 2023, and Shoffiana, et al., 2022, embankment as a geotechnical structure in soft soil is important to be reinforced as one of the efforts to manage infrastructure and facilities assets. At other locations in the same project, reinforcement of subgrade soil has also been applied using cement column combined with rigid inclusion, namely Full Displacement Column at STA 4+600 - 4+800. By considering the speed of construction implementation, the type of reinforcement with a Full Displacement Column is also an alternative that can be applied at STA 7+450 but there has been no technical study related to this reinforcement if applied at this location. For this reason, it is necessary to study the analysis of the cause of the landslide and recommend reinforcement using the Rigid Inclusion with Full Displacement Column type which has been applied in other locations to catch up with the delay in construction progress by applying variations in column installation distance, thickness, and reinforcement of load transfer platform stiffeners.

LITERATUR REVIEW

Soft Soil

In a general engineering definition, soil is defined as a material that consists of aggregates (grains) of solid minerals that are not segmented (chemically bound) to each other and of decomposed organic matter (solid particles) accompanied by liquids and gases that fill the empty spaces between the solid particles. Soil consists of grains of material resulting from the weathering of massive rock masses, where the grain size can be as large as chunks, numbers, sand, silt, clay, and grain contact is not segmented including organic matter (Terzaghi, 1984).

A soil layer designated as a soft soil layer is a layer of clay or silt that has a standard penetration N-SPT of less than 6. There is a correlation between N-SPT and other soil consistency types as described in Table 1 below.

Table 1. Correlation Table of N-SPT value with Other Parameters

	Cohesionless Soil/Sol Pulvérulent				
N (*blows*)	0 – 3	4 – 10	11 – 30	31 – 50	> 50
γ(KN/m^3)	-	12 – 16	14 – 18	16 – 20	18 – 23
Ø (°)	-	25 – 32	28 – 36	30 – 40	> 35
State	*Very Loose*	*Loose*	*Medium*	*Dense*	*Very Dense*
Dr (%)	0 - 15	15 - 35	35 - 65	65 - 85	85 - 100
	Cohesive Soil/Sol Cohérent				
N (*blows*)	< 4	4 – 6	6 – 15	16 – 25	> 25
γ (KN/m^3)	14 – 18	16 – 18	16 – 18	16 – 20	> 20
q_U (kPa)	< 25	20 – 50	30 – 60	40 – 200	> 100
Consistency	*Very Soft*	*Soft*	*Medium*	*Stiff*	*Hard*

Source: Bowles, 1984

Organic Soil

Organic soils are soils that are categorized in this way based on their organic content. Organic soils are defined as having an organic content of 25% to 75%. Furthermore, these organic soils are further categorized into OL and OH groups based on their level of plasticity.

a. Soil with low organic content

According to ASTM D-4427 on the classification of peat soils based on laboratory testing, soils with low organic content can be classified as mineral soils, namely clay, silt or sand, except that they have an organic content between 5% and 25%.

b. Organic soil

Organic soils (O) are soils that have organic content ranging from 25% - 75%. These organic soils are categorized into low-plasticity organic soils (OL) and high-plasticity organic soils (OH).Peat soil

According to Landva, et al (1982), Kearns, et al (1982), and ASTM (1985) in Yulianto, et al (2016), peat soils are those with an organic content of > 75%. MacFarlane and Radforth (1965) divide peat soils into two broad groups: fibrous peat with a fiber content of 20% or more and amorphous granular peat with a fiber content of less than 20%; ASTM D 4427-92 (1992) classify peat soils based on five factors: fiber content, ash content, acidity, absorption rate, and plant composition as shown in Table 1 below.

Table 2. Peat Soil Classification (ASTM 4427-1985)

Parameter	Category	Description
Fiber Content	1. Fibrous 2. Hemic 3. Sapric	Fiber Content > 67 % Fiber Content 33 – 67 % Fiber Content < 33 %
Ash Content	1. Low Ash 2. Medium Ash 3. High Ash	Ash Content (AC) < 5 % Ash Content (AC) 5 – 15 % Ash Content (AC) > 15 %
Acidity	1. High Acidity 2. Moderate Acidity 3. Slightly Acidity 4. Basic	pH > 4.5 pH 4.5 – 5.5 pH 5.5 – 7 pH > 7
Absorbency	1. Extremely 2. Highly 3. Moderately 4. Slightly	Water capacity > 1500 % Water capacity 800 – 1500 % Water capacity 300 - 800 % Water capacity < 300 %
Botanical Composition	1. Single Botany 2. Multiple Botany	At least 75% of its fiber content is from one type of forming plant. At most 25% of its fiber content from one type of forming plant.

Source: ASTM 4427-1985

Road Embankment on Organic Soil

Based on research by Manudianto et al, 2023 embankments on organic soils can be reinforced using preloading, wood piles, and micro piles and then analyzed which reinforcement is most effective to apply. Efforts to find the most effective reinforcement value are one of the efforts to improve infrastructure asset management. One of the reinforcements that can also be applied to organic soils is using a rigid inclusion type full displacement column. Based on Menard, 2021 this reinforcement is effective in very soft soils, can be installed to extremely deep depths, has high load-carrying capability, only minimal spoils generated during installation, and does not provide a pathway for groundwater contamination migration. This reinforcement technique is shown in the figure 5 below.

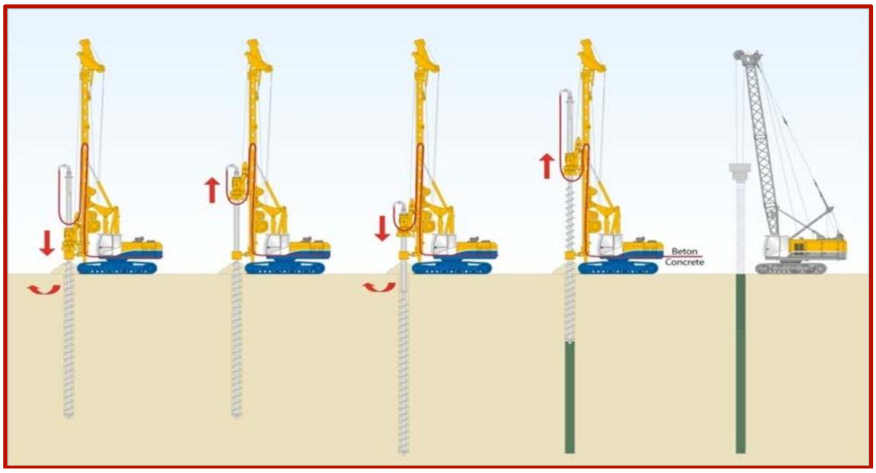

Figure 5. Soil improvement with Full Displacement Column

RESEARCH METHOD

This research on the analysis of the causes of embankment slope collapse and changes in soil mechanical properties combined with alternative reinforcement of landslides with a Full Displacement Column takes the object of location on a toll road construction project in West Sumatra that is focused on STA 7 + 450. The data used in the research is divided into secondary data and primary data. Primary data is the result of field surveys and soil sampling at the location being observed. Secondary data obtained from the project team includes field topographic data, soil investigation results in the form of Borlogs, soil data from laboratory tests, detailed engineering drawings, and data from monitoring geotechnical instrumentation.

Soil data from laboratory testing and field interpretation results from secondary data were then checked for accuracy before being used in the analysis and modeling stages. Contour data, DED, and investigation data are used as interface data for soil layer stratification in modeling in a finite element program that represents soil layers according to field conditions. The parameter data from laboratory tests and secondary data interpretation in the form of specific gravity, water content, cohesion (c), internal friction angle (Ø), and elastic modulus (E) were inputted into the Plaxis 2D program as parameters used in the analysis using the finite element approach. Meanwhile, the monitoring data from geotechnical instrumentation was used as comparison data to determine the condition of the collapse plane on the embankment subgrade.

The next step is to perform a back analysis of the embankment slope failure, which is the process of determining the geotechnical characteristics that caused the slope failure. This method involves analyzing and evaluating the soil parameters and geometry of the slope that has already failed, and then analyzing the factors that influence the failure. At this stage, the analysis is carried out with a finite element method approach using the Plaxis 2D program by applying variations in subgrade soil parameters that can be in the form of physical and mechanical parameters. From this analysis, it is expected to obtain soil parameters and factors that cause embankment failure.

After the causes of the embankment slope failure are known, the next step is to analyze the reinforcement using a full displacement column. This analysis is carried out by considering the factors that cause landslides in the previous analysis. This stage is carried out by modeling the full displacement column on the subgrade of the road embankment with a finite element approach using the Plaxis 2D program.

DATA ANALYSIS

Secondary Data

1. **N-SPT and Laboratory Test Result**

   The main data used in the analysis of the embankment slope failure is the Boring log results (N-SPT) and data from the soil laboratory test. According to the project, the closest soil data is located at STA 7+364, namely point BH-R-02. The boring log data of point BH-R-02 and the results of the interpretation of soil type can be seen in Figure 5.

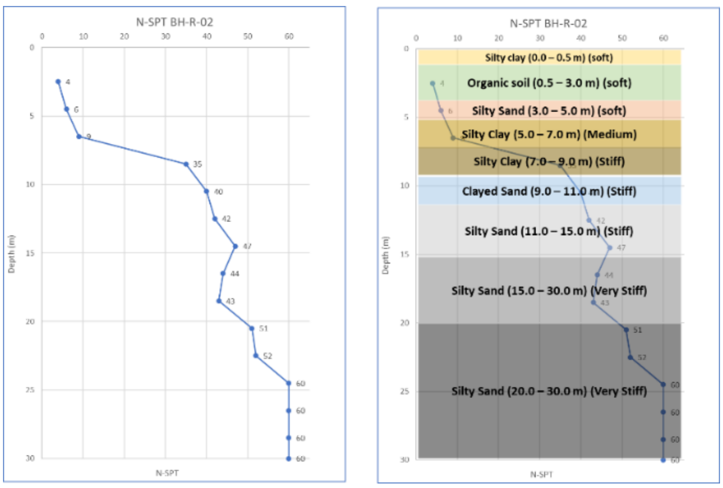

Figure 5. N-SPT value from Boring log

From the results of the boring logs performed by the project, several soil samples were taken for laboratory testing for depths of 1.5 meters, 3.5 meters, and 7.5 meters with undisturbed sample conditions, while depths of 17.5 meters and 21.5 meters with disturbed sample conditions.

Table 3. Summary of laboratory test result (secondary data)

Layer	Depth	N-SPT	Soil Description	γ (t/m3)	Wc (%)	Gs	Sr (%)	eo	n	Cu (kN/m2)	Ø
1	0 – 0.5	4	Silty Clay	-	-	-	-	-	-	-	-
2	0.5 – 3	4	Organic	1.070	861.024	1.637	100	13.698	0.932	9.23	25.263
3	3 – 5	6	Silty Sand	2.045	29.827	2.640	100	0.676	0.403	25.17	37.433
4	5 – 7	9	Silty Clay	-	-	-	-	-	-	-	-
5	7 – 9	35	Silty Clay	1.514	72.087	2.603	95.851	1.957	0.662	5.06	6.32
6	9 – 11	40	Clayey Sand	-	-	-	-	-	-	-	-
7	11 – 15	42	Silty sand	-	-	-	-	-	-	-	-
8	15 – 20	43	Silty sand	2.099	18.739	2.773	91.332	0.569	0.363	15.61	39.55
9	20 – 30	60	Silty sand	2.227	14.865	2.719	100	0.403	0.287	34.35	45.039

2. Correlation Parameter

In the modeling using the finite element program, several parameters must be input to the program. These parameters affect the output results of the program in the form of safety factor values or settlement that occurs. These parameters are soil physical parameters such as soil volume weight (γ) pore number (e), and porosity (n). Meanwhile, there are also mechanical parameters such as cohesion (Cu), internal angle friction (Ø), and modulus of elasticity (E). There are also consolidation parameters such as compression index (cc), development index (cs), and permeability index (k). All these parameters are required to be input into the program to get accurate results. However, not all parameters at each depth are obtained from existing secondary data, so correlations with empirical approaches are needed to obtain the parameter values. The summary of the secondary data properties and the correlation is shown in Table 1 below.

Table 4. Summary of soil properties from correlation

Layer	Depth	N-SPT	γ (t/m3)	eo	n	C (kN/m2)	Ø	Cc	Cs	K (m/day)	E (kN/m2)
1	0 – 0.5	4	1.154	1.957	0.662	5.06	6.32	2.14759	0.63623	8.64×10^{-06}	1380
2	0.5 – 3	4	1.070	13.698	0.932	9.23	25.263	9.9018	0.63623	8.64×10^{-03}	1380
3	3 – 5	6	2.045	0.676	0.403	25.17	37.433	0.121	0.02	8.64×10^{-03}	10350
4	5 – 7	9	1.514	1.957	0.662	5.06	6.32	0.75704	0.11528	8.64×10^{-05}	5865
5	7 – 9	35	1.514	1.957	0.662	5.06	6.32	0.75704	0.11528	8.64×10^{-05}	13800
6	9 – 11	40	2.099	0.569	0.363	15.61	39.55	0.05175	0.01922	8.64×10^{-04}	65280
7	11 – 15	42	2.099	0.569	0.363	15.61	39.55	0.05175	0.01922	8.64×10^{-03}	68544
8	15 – 20	43	2.099	0.569	0.363	15.61	39.55	0.05175	0.01922	8.64×10^{-03}	66827
9	20 – 30	60	2.227	0.403	0.287	34.35	43.039	0.0399	0.005167	8.64×10^{-03}	69000

3. **Detail Engineering Drawing**

The DED data obtained will be used to determine the geometry of the embankment construction built in the field. Based on the DED data, it is known that STA 7 + 450 is planned to have a 4-meter-high embankment with a 3-meter-high preloading embankment. The cross-section of the STA 7+450 embankment plan in the detailed engineering drawing can be seen in Figure 6.

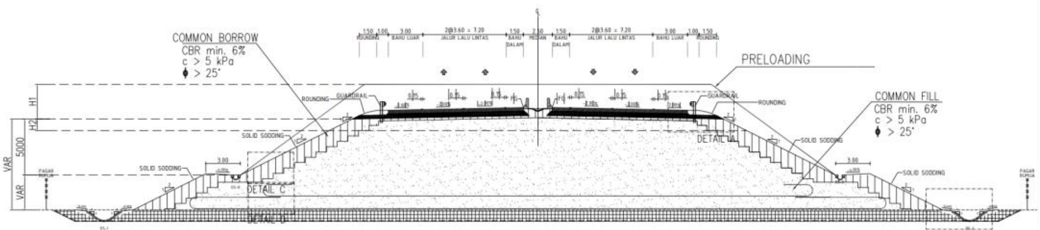

Figure 6. Cross section of STA 7+450 from Detail Engineering Drawing

4. **Geotechnical Instrumentation**

As monitoring instruments on the site, the project team used several geotechnical instrumentation tools such as Piezometer, Settlement Plate, and Inclinometer. The results of these instrumentation readings were used as modeling validation and reference lifts for the addition of embankment loads in the finite element program. The monitoring results based on these three geotechnical instruments are shown in Figure 4 for Piezometer and Inclinometer then Figure 7 for the settlement Plate.

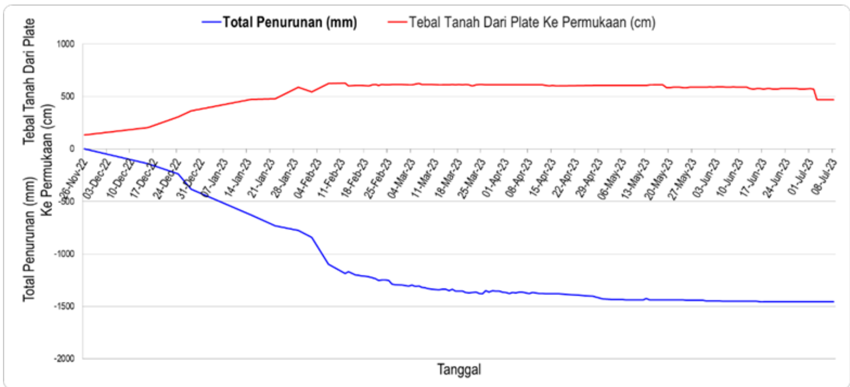

Figure 7. Result of monitoring settlement from settlement plate

Primary Data

Besides secondary data, primary soil sample testing was also carried out at the location that was affected by the landslide, at STA 7+450 on the right side of the embankment toe. The soil samples taken were soil samples in the layer that indicated the cause of the collapse of the embankment body, which is the organic soil layer at a depth of 1.5 - 2.0 meters. Based on the test results of soil investigation from secondary data at a depth of 1.5 - 2.0 meters is organic soil, then testing with a standard approach for organic soil or peat is using the Peat Testing Manual standard in 1979. The summary of the results of testing the properties of soil samples is shown in Table 2 below.

Table 5. Summary of Primary Soil Laboratory Testing

No	Parameters	Unit	Primary Data Result		Secondary Data Result
1	Water Content (wc)	%	367.222		861.024
2	Unit Weight (γt)	gr/cm3	1.050		1.070
3	Spesific Gravity (Gs)	-	1.5908		1.637
4	Rubbed Fiber Content	%	18.72		-
5	Unrubbed Fiber Content	%	43.59		-
	▪ Field Method		42.11		
	▪ Laboratory Method				
6	Fiber size distribution	%			-
	▪ Coarse fiber		70.39		
	▪ Medium fiber		23.43		
	▪ Fine fiber		3.74		
7	Atterberg Limit		**Air Dry**	**Oven**	340.64
	▪ Liquid Limit (LL)	%	52.903	96.008	210.62
	▪ Plastic Limit (PL)	%	47.904	73.670	-
	▪ Shrinkage Limit (SL)	%	40.940	50.120	130.02
	▪ Plasticity Index (PI)	%	4.998	22.337	
8	Liquid Limit Ratio (LLR)	-	0.551		-
8	Ash Content (AC)	%	51.52		63.96
9	Organic Content (OC)	%	48.48		36.04
10	Direct Shear Test (DS)				
	▪ Cohesion (C)	kg/cm2	0.1144		0.0923
	▪ Internal Angle Friction (∅)	degrees	49.92		25.262
11	Consolidation Test				
	Compression Index (Cc)	-	0.8175		-
	Swelling Index (Cs	-	0.1204		-
	Void Ratio (eo)	-	3.713		-

RESEARCH ANALYSIS

Back Analysis

Back analysis is a method to determine the parameters that influence the landslide that occurs, the parameters can be mechanical parameters or physical parameters that can increase or decrease the stability value of a geotechnical structure. In the case of the landslide at STA 7+400 - 7+550, it will be analyzed using finite element modeling. In this analysis process, a new parameter value will be found that causes the landslide. These parameters will be used as input parameters in the next stage, which is in modeling the repair treatment of the landslide that occurred. Soil stratification and geometry in the modeling program are adjusted to the field condition where the maximum preloading embankment is at a height of 6,291 meters according to the settlement plate data shown in Figure 8 below.

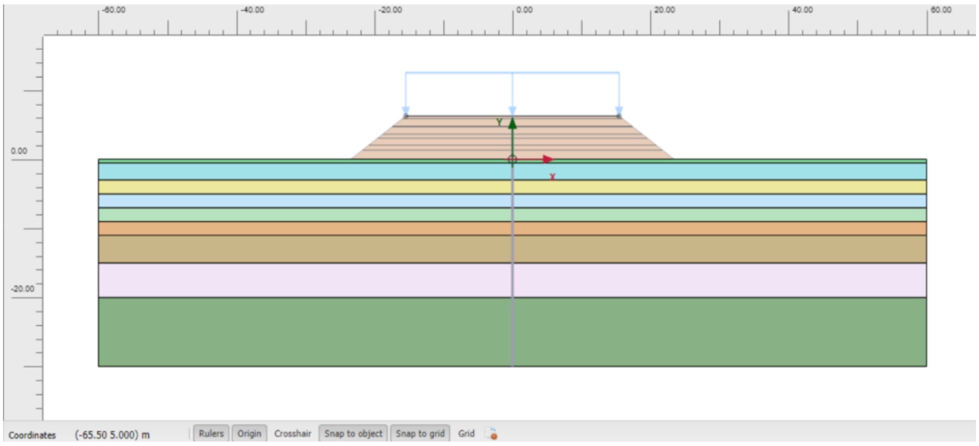

Figure 8. Soil stratification with 9 layers of soil and geometry of embankment according to Settlement Plate data

After trial and error with the parameter variations, the input parameters that are closest to the field are obtained according to Table 3. This input parameter value is based on the combination of secondary data from the laboratory with primary data from individual tests in which a sample is taken on the depth of organic soil.

Table 6. Input soil parameters that best match the field condition

Layer Model	Elevation (m)	Soil type	Consistency	Tipe Model		γ unsat	γ sat	Modulus Young - E	Poisson Ratio- v'	C'ref	Ø	Cc	Cs	eo	K
				Material Model	Drainage	kN/m3	kN/m3	kN/m2		kN/m2	degrees				m/hari
1	0 - -0.5	Silty Clay	Soft	Soft Soil	Undrained A	8.8	15.14	-	0.3	5.06	6.32	0.8175	0.1204	3.713791	0.00000864
2	-0.5 - -3	Organic	Soft	Soft Soil	Undrained A	10.7	10.7	-	0.3	8.17	19.68	0.8175	0.1204	3.713791	0.00864
3	-3 - -5	Silty Sand	Soft	Soft Soil	Undrained A	15.75	20.45	-	0.3	16.78	24.96	0.121	0.02	0.676	0.00864
4	-5 - -7	Silty Clay	Soft	Soft Soil	Undrained A	8.8	15.14	-	0.3	5.06	6.32	0.75704	0.115286	1.957	0.0000864
5	-7 - -9	Silty Clay	Medium	Mohr Coulomb	Undrained A	8.8	15.14	13800	0.3	5.06	6.32	-	-	1.957	0.0000864
6	-9 - -11	Clayey Sand	Stiff	Hardening Soil	Drained	20.99	20.99	65280	0.3	10.41	26.37	-	-	0.569	0.000864
7	-11 - -15	Silty Sand	Stiff	Hardening Soil	Drained	20.99	20.99	68544	0.3	10.41	26.37	-	-	0.569	0.00864
8	-15 - -20	Silty Sand	Very Stiff	Hardening Soil	Drained	20.99	20.99	66827	0.3	10.41	26.37	-	-	0.569	0.00864
9	-20 - -30	Silty Sand	Very Stiff	Hardening Soil	Drained	20.99	20.99	69000	0.3	22.9	30.03	-	-	1.13	0.00864
Common Fill	0 - 6.29	Borrow Material	-	Hardening Soil	Drained	17	18.5	10000	0.3	10	35	-	-	0.5	0.00864

From the modeling results using the above parameters, several points are obtained that are close to the existing conditions in the field, i.e.

1. Staged Construction

   From the modeling of stage construction, a critical SF value of 1.028 is obtained in the condition where the embankment reaches the maximum height of 6.291 meters shown in Figure 5. 15. Then the SF value is less than 1 under the condition that the embankment soil is passed by the traffic load. As known in the background this embankment is one of the mobility paths for project vehicles that are shown in Figure 9.

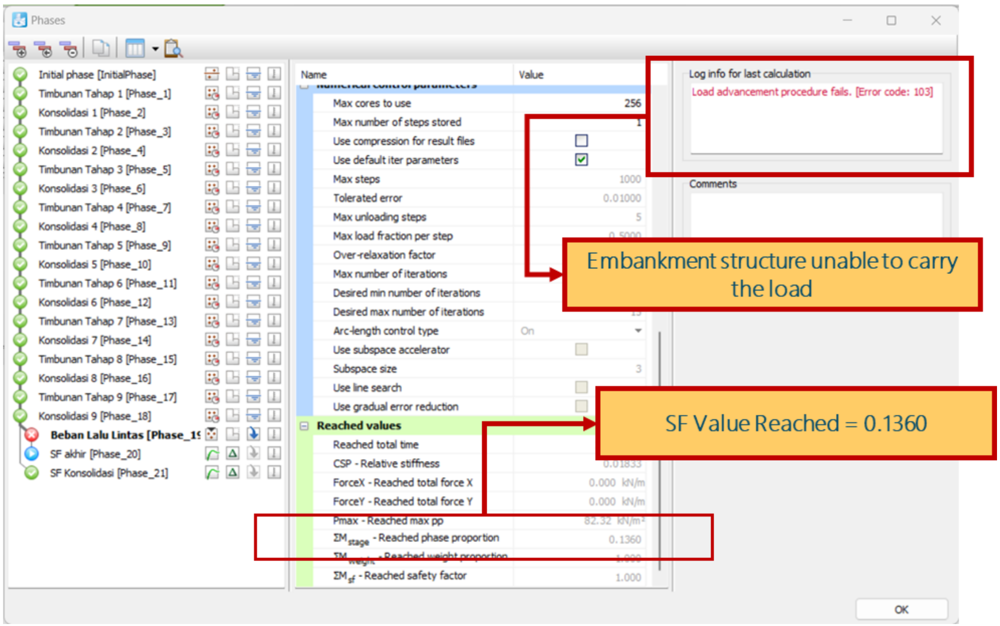

Figure 9. Staged construction on the modelling program indicated the embankment failure at the traffic load stage

2. Consolidation Settlement

Based on the settlement results, the settlement value is almost the same as the condition in the field. Monitoring of settlement in the field was carried out with geotechnical instrumentation media in the form of settlement plates. The decline that occurred in the field monitored using the settlement plate was read in the period November 26, 2022, to July 8, 2023. The last monitoring result, on July 8, 2023, obtained the value of the decline that occurred was 1,456 meters. While the results of modeling with finite elements obtained a settlement value of 1,455 meters. A comparison between the results of this settlement can be seen in Figure 10.

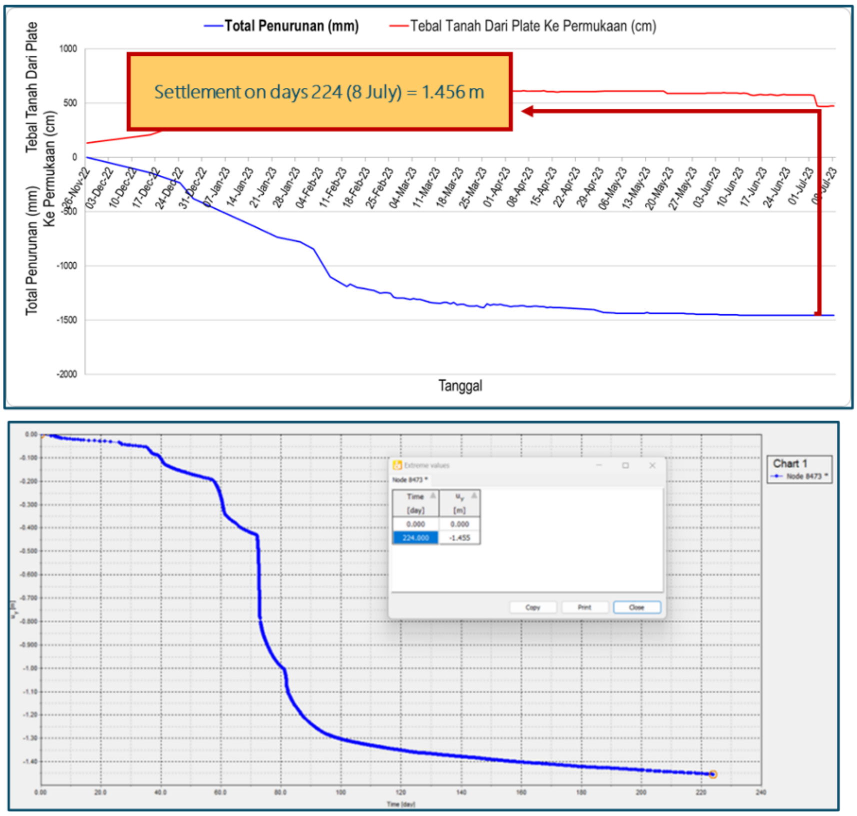

Figure 10. Comparison settlement from field data and modelling

3. Sliding Plane

One of the parameters used as a counter-analysis reference is the representation of landslide conditions in the field in the modeling program. From the modeling results, the landslide line is obtained which is compared with the crack line that occurs in the field. The landslide line in this program is seen in the modeling output in the incremental deviatoric strain display mode and in the plastic point display mode by reviewing the failure point.

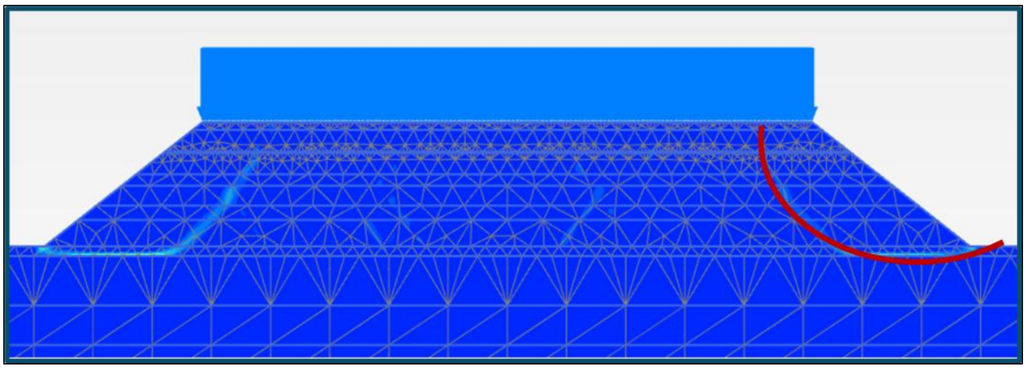

Figure 11. Sliding line of incremental deviatoric strain display mode

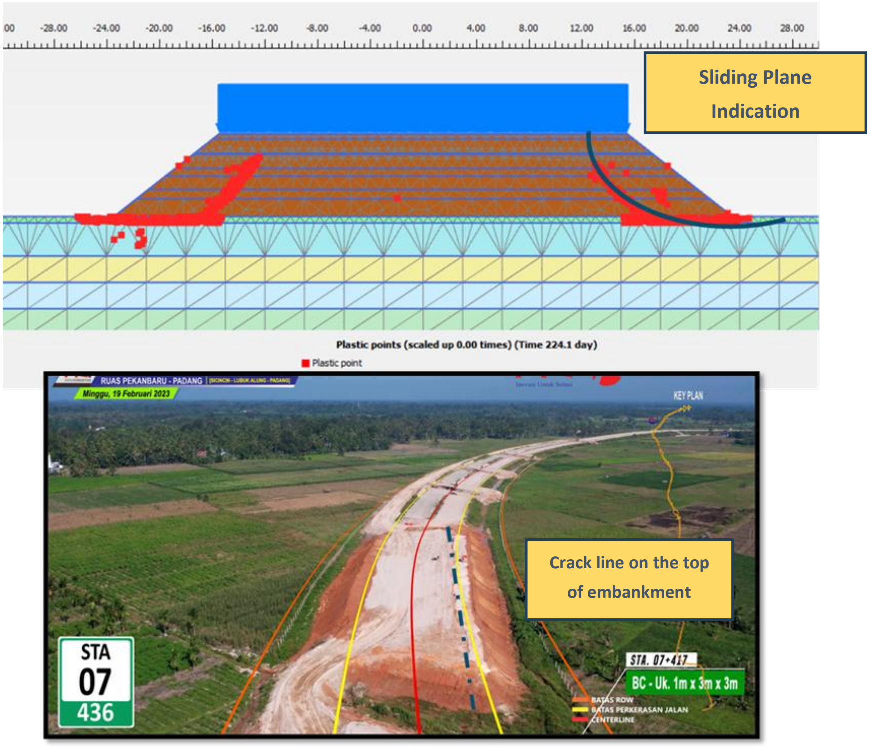

Figure 12. Comparison between the sliding plane and crack on the top of the embankment

4. Pore Excess Pressure

With consideration of pore water pressure as a validation of the back analysis results, readings from geotechnical instrumentation in the form of piezometers installed in the field at a depth of 2 meters below the subgrade are used here. Based on the modeling results, the maximum pore water pressure value is on day 73 with a value of Pexcess = 74.04 kN/m2 according to the graphical output of the finite element program with the same observation point as the piezometer location shown in Figure 13. As for the piezometer readings, the readings obtained on day 73, on February 7, 2023, have a value of 78.86 kN/m2 shown in Table 4. From both modeling results and piezometer readings in the field have similar pore water pressure values, this indicates that the soil model and parameters used in modeling are close to field conditions.

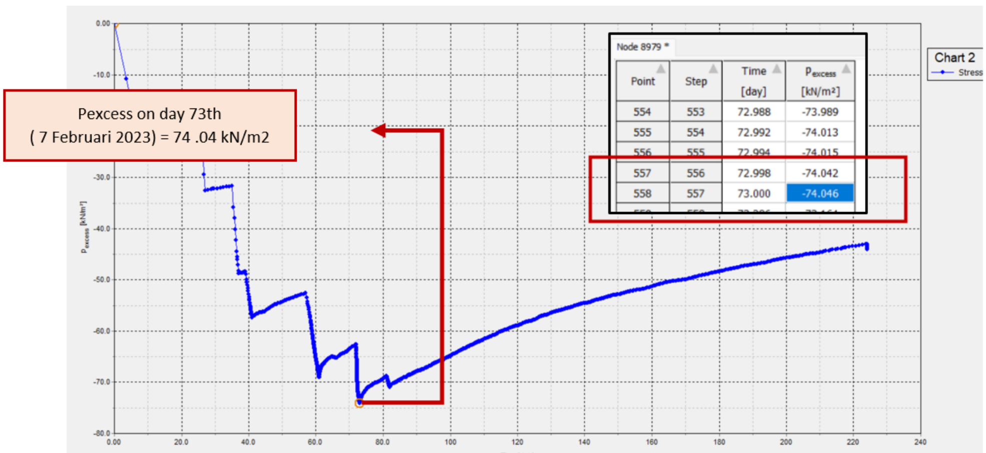

Figure 13. Pexcess output from modeling

Table 7. Pore pressure result from Piezometer

Calibration Data Sheet		Tip 2	depth :	2	m fr WFL
Serial number		577271		5.008	Tip 2 (Depth 2m)
Thermal Coefficient (Tct)		0.002132	kg/cm²/ºC	A factor	-3.0838E-06
Cal Temp		22.0	ºC	B factor	1.7750E-03
Baro. Pressure		1016.0	mb	C factor	2.2361E+01
Date	Day	Hidrostatik Pressure (kPa)	Reading (Hz)	Temperature (ºC)	Pore Water Pressure (kPa)
7 Feb 23	73	-0.54	2947.6	27.4	78.86
8 Feb 23	74	-0.77	2948.4	27.2	77.59
9 Feb 23	75	0.22	2948.5	27.2	77.42

Based on the 4 verification parameters that are close to the field situation, it can be said that the soil model and the input parameters are representative of the field situation. The initial soil parameter values tend to change due to the landslide that occurred, this result is in accordance with the research by Irsyam et al (2006) who analyzed changes in soil parameters in the landslide at Cipularang Toll Road, Indonesia caused by the presence of a layer of silty clay and weathered clay shale with the parameters of the back analysis results are c = 5 kPa and pi = 13 degrees.

Reinforcement with Full Displacement Column Analysis

After the residual soil parameters from the back analysis process are obtained, the next step is to plan the reinforcement at the observation location. In this study, reinforcement with rigid inclusion type full Displacement Column will be used which has also been applied at STA 4+600. The application of this reinforcement will certainly accelerate construction progress because all the necessary resources are already available in the field. The geometry design of the Full Displacement Column at STA 7+450 is shown in Figure 13.

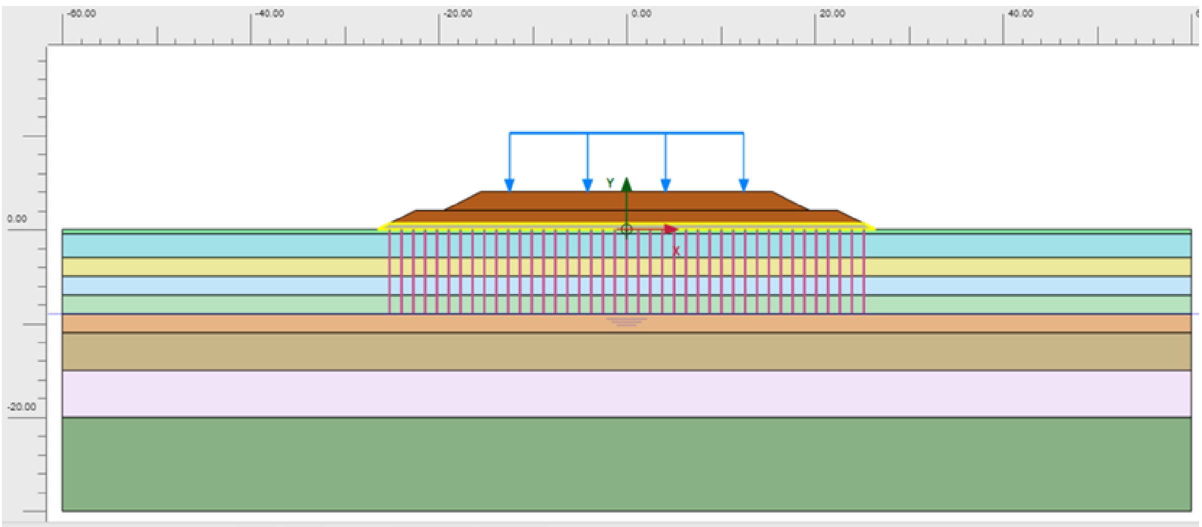

Figure 13. Geometry of Embankment and Full Displacement Column

Modeling analysis with several variations of reinforcement applied is then analyzed for effectiveness based on the value of safety factor and settlement. From the modeling results with 18 variations of the model as shown in Table 5.

Table 8. Variation model of full displacement column reinforcement

Model Code	Material LTP	Column Spacing (Diameter of column = 0.42 m)	Thickness of LTP (m)
FDC 1	Geogrid dan granular	3 x Diameter	0.6
FDC 2	Geogrid dan granular	3 x Diameter	1.2
FDC 3	Geogrid dan granular	3 x Diameter	1.8
FDC 4	Geogrid dan granular	4 x Diameter	0.6
FDC 5	Geogrid dan granular	4 x Diameter	1.2
FDC 6	Geogrid dan granular	4 x Diameter	1.8
FDC 7	Geogrid dan granular	5 x Diameter	0.6
FDC 8	Geogrid dan granular	5 x Diameter	1.2
FDC 9	Geogrid dan granular	5 x Diameter	1.8
FDC 10	Geotextile dan granular	3 x Diameter	0.6
FDC 11	Geotextile dan granular	3 x Diameter	1.2
FDC 12	Geotextile dan granular	3 x Diameter	1.8
FDC 13	Geotextile dan granular	4 x Diameter	0.6
FDC 14	Geotextile dan granular	4 x Diameter	1.2
FDC 15	Geotextile dan granular	4 x Diameter	1.8
FDC 16	Geotextile dan granular	5 x Diameter	0.6
FDC 17	Geotextile dan granular	5 x Diameter	1.2
FDC 18	Geotextile dan granular	5 x Diameter	1.8

The safety factor value and the settlement with a traffic load of 15 kPa result from modeling are plotted into a graph to see the effect of each variation applied. The modeling results of each variation analyzed are as follows.

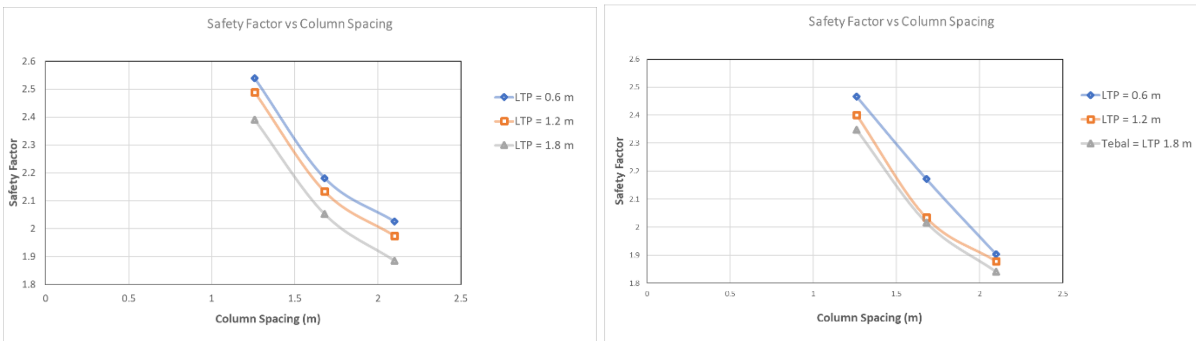

Figure 14. Safety factor vs Column Spacing Graphic reinforced on load transfer platform material: geogrid (left), and geotextile (right)

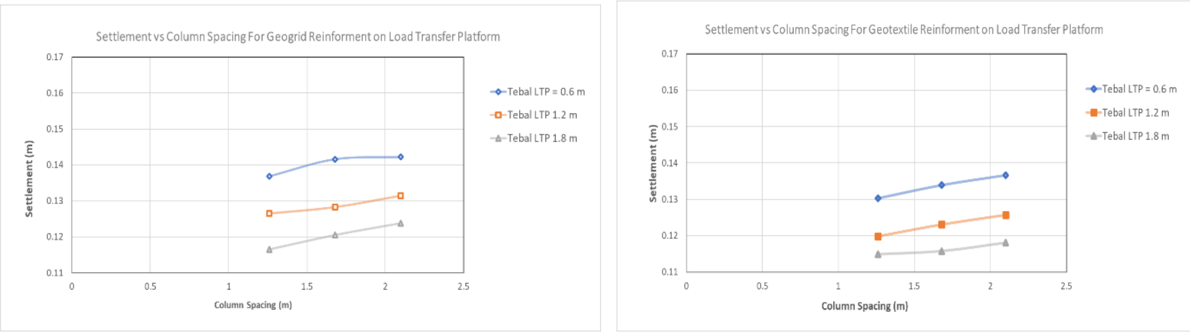

Figure 15. Settlement vs Column Spacing Graphic reinforced on load transfer platform material: geogrid (left), and geotextile (right)

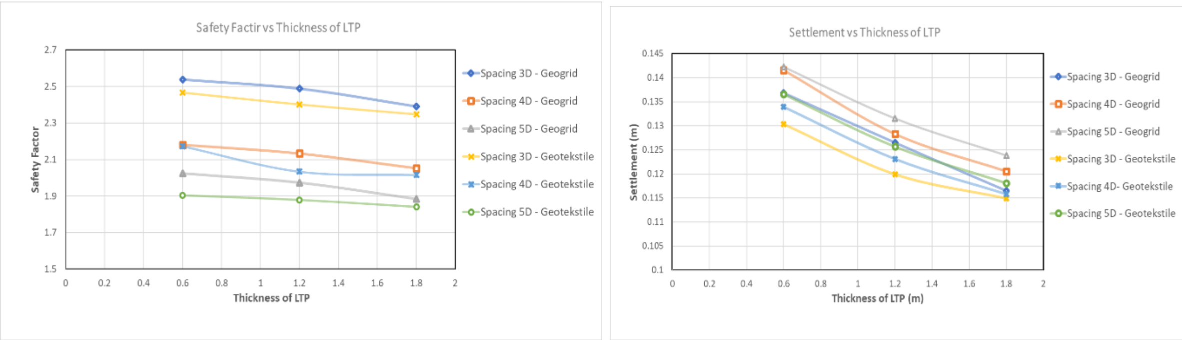

Figure. 16. Graph of SF vs thickness of LTP (left), settlement vs thickness of LTP (right)

Based on the graph, the Load Transfer Platform thickness that is effective in transferring loads to the subgrade is t = 1.8 meters for reinforcement using geogrids. As for reinforcement using geotextile, the optimum Load Transfer Platform thickness is t = 1.8 meters because the settlement is smaller than others. Based on research conducted by Lauzon et al (2009), it was found that reinforcement with the Full Displacement Column type as a tank foundation under soft soil obtained maximum results. Where the settlement that occurs due to this reinforcement reduces the differential settlement at the edge of the tank and the center of the tank up to 25 mm. For the full displacement column spacing, from the analysis, it is found that the closer the spacing between FDC columns, the SF value increases both for LTP reinforcement using geotextile or geogrid. With 3 - 5 times the diameter of distance, the safety factor value is obtained that reached the SF> 1.5 specification. So the most effective reinforcement design is to use a distance of 3 times the column diameter with an LTP thickness of 1.2 - 1.8 meters for LTP reinforcement with geotextile and geogrid.

CONCLUSION

The existing safety factor value of the preloading embankment before the collapse was SF = 1.028, the collapse in the field occurred due to the organic subgrade with a soft consistency. In addition, the landslide occurred due to the traffic load at the location of the embankment which was used as a mobility lane for project vehicles. Based on the results of the back analysis with modeling, the SF value before the landslide was considered critical (Staged Construction: Consolidation 9), namely SF = 1.028, and after being given a traffic load the SF dropped to 0.1360 (Collapse).

The results of the subgrade parameters that are suitable for the soil condition during the landslide are in the first layer of depth 0 - 0.5 meters which is soft silty clay soil with residual parameters C = 5.06 kN/m2, Ø = 6.32, cc = 0.8175 and cs = 0.1204. Then the second layer of depth 0.5 - 3 meters is organic soft soil with residual parameters C = 8.17 kN/m2, Ø = 19.68, cc = 0.8175 and cs = 0.1204. For the third layer of 3 - 5 meters depth obtained C = 16.78 kN/m2 and 24.96, cc = 0.121 and cs = 0.02. The last for the layer depth of 5-7 meters obtained parameter values C = 5.06 kN/m2, Ø = 6.32, cc = 0.757, and cs = 0.115.

Based on the modeling analysis, the Load Transfer Platform thickness that is effective in transferring loads to the subgrade is t = 1.8 meters for reinforcement using geogrids. As for reinforcement using geotextile, the optimum Load Transfer Platform thickness is t = 1.8 meters. For the full displacement column spacing, from the analysis, it is found that the closer the spacing between FDC columns, the SF value increases both for LTP reinforcement using geotextile or geogrid. With 3 - 5 times the diameter of distance, the safety factor value is obtained that meets the SF> 1.5 specification. So the most effective reinforcement design is to use a distance of 3 times the column diameter with an LTP thickness of 1.8 meters for LTP reinforcement with geotextile and geogrid.

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