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Analysis of Excavated Slope Stability on the TRSS of Blitar Regency with a Cracked Soil Approach

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ABSTRACT

The construction of the Trans South-South Java Road Project (TRSS) in East Java Province is part of the National Strategic Project which aims to reduce the disparity between the northern and southern regions of Java Island. In STA 17+850 of the Bululawang–Sidomulyo–Tambakrejo section, a ±30 m high excavated slope composed of limestone experienced two landslides at the same segment (slope 2 and slope 3), although slope geometry has been improved through regrading from 71.6° to 45°. This study aims to analyze the main causes of landslides at the location using the cracked soil approach, as well as evaluate the effect of slope angle on the safety factor (SF). The concept of cracked soil refers to the presence of cracks, which can be fine cracks that are not visible or wide cracks, these cracks can function as rainwater infiltration pathways, potentially trigger collapse. The soil condition at the slope is modeled as a material "behaving like sand" under "drained" condition (c = 0 and φ = φ). The analysis method was carried out using the Finite Element Method (FEM) with the Plaxis 2D auxiliary program through two main models. Model 1 simulates the soil material as "behaving like sand" (c = 0 and φ = φ), where failure is governed entirely by the internal friction angle without cohesion. Model 2 includes a polygon such as ground cracks placed on or near a sliding area, thus facilitating a specific c = 0 on that plane with variations in the width and depth of the cracks. Stability analysis at the initial conditions (after excavation) showed an SF>> 1, which did not yet represent the landslide conditions in the field. Through the cracked soil approach, an SF< 1 were obtained for both slope variations, both in model 1 and model 2. These results show that the cracked soil approach can be applied to explain the mechanism of landslides on the slope of STA 17 + 850 of the Bululawang – Sidomulyo – Tambakrejo Section.

Keywords:
slope stability, cracked soil, safety factor, landslide, plaxis 2D

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INTRODUCTION

Road infrastructure on the island of Java is a vital asset that supports regional connectivity and drives the economic activity, one of which is the existence of the North Coast Line (Pantura), which has an impact on improving the economy of the people in the northern region of Java Island. However, the existence of this route has indirectly created a disparity between the northern region and the southern region of Java Island. This gap factor then became the background for the construction of the South Coast Line (Pansela), also known as the Trans South-South Java Road Project (TRSS). The construction of TRSS in East Java Province is part of the National Strategic Project (PSN) which aims to develop the tourism sector on the south coast so that it can improve the economy of the surrounding communities.

The TRSS in East Java Province has a total length of 628.38 km. One of the packages currently under construction is the Bululawang–Sidomulyo–Tambakrejo Road and Bridge

Construction with a handling length of 12,855 km (STA 5+100 – STA 17+990) which is in Blitar Regency, East Java Province, as shown in Figure 1. Geographically, this section is in a hilly landscape, so this condition affects the design of the road route which requires a combination of excavation work (cut), fill work, and bridge structure.

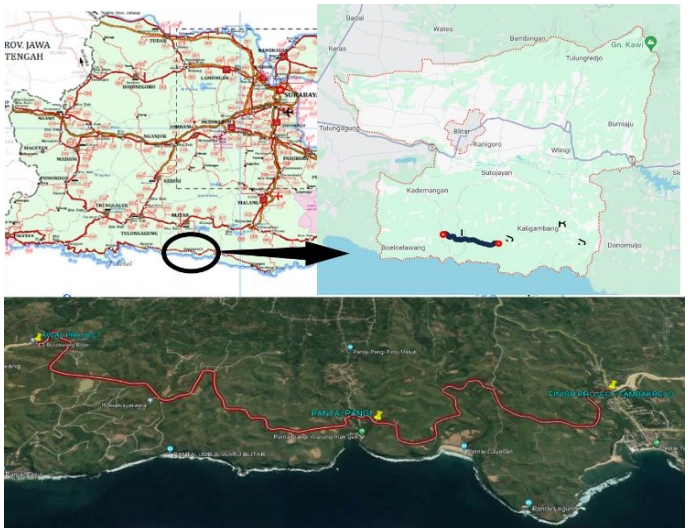


Figure 1. Location of the Bululawang–Sidomulyo–Tambakrejo Road and Bridge Construction Package (Bina Marga, 2025)

Excavation work on slopes is often related to slope stability problems. The stability of the cut on the slope is determined by the geological conditions, the technical properties of the soil, the water pressure due to seepage, and the method of cutting. Important aspects of excavated slope stability consist of the shear strength of the excavated material, the weight of the soil content, slope height, slope inclination, and pore water pressure (SNI 8460, 2017). One of the locations that has a high excavation slope is STA 17+850, where the cut slope reaches approximately 30 m. The slope design at this location consists of 4 slopes with inclinations of 3 Vertical: 1 Horizontal (71.6º) and 4 Vertical: 1 Horizontal (76º). At the stage of excavation work at STA 17 + 850, slope failure has occurred on the 2nd slope and 3rd slope. As a handling action, the slope geometry was modified through regrading, by changing the initial inclination from 3V:1H to 1V:1H, consequently, the initial slope height of ± 30 m increased to ± 40 m. Following this regrading effort, landslide recurred in the same zone (2nd slope and 3rd slope). Documentation of the landslide conditions is shown in Figure 2. This phenomenon of recurring landslides indicates that the underlying failure mechanism is more complex. Regrading was ineffective in increasing the resultant resisting force, or insufficiently significant in reducing the driving force to achieve equilibrium, resulting in continued slope failure. The landslide occurred because the driving force exceeded the shear strength of the soil along the failure plane.

(a) (b)

Figure 2. Landslide Documentation at STA 17+850 (a) Before Regrading (b) After Regrading (Bina Marga, 2025)

Soil investigations carried out at the landslide site indicate that N-SPT value of 60 had been reached at a depth of 2 m. Generally, high N-SPT value indicates that the layer is very strong, dense, or hard. This is also proven by bore log results, who show that most of them are rock layers. Almost all soils with a medium to hard consistency already have a lot of cracks and discontinuities, the cracks that occur range from very fine hairy cracks to wider and more visible cracks, often undetectable from the ground surface, these cracks allow rainwater to enter and flow through the subsurface channels, which may subsequently extend deeper and longer over time. This combination of cracks and water infiltration becomes the main factor controlling slope stability, because the cracks can ultimately act as slip surfaces that trigger landslides. The condition of the soil/rocks on the slope is like "behaving like sand", with a "drained" condition of c=0 and φ = φ (Mochtar, 2023).

In this study, the excavation slope is modeled using the Finite Element Method (FEM) using the Plaxis 2D auxiliary program. The analysis produces the Safety Factor (SF) and slope deformation values. The objectives of this study are to analyze slope stability using cracked soil approach, to evaluate the effect of slope inclination variations on slope stability, and to determine the cause of the landslide at STA 17+850.

RESEARCH METHOD

A. Research Location

This study took samples from the excavation slope of the Trans South-South Java Road Project in Blitar Regency, East Java Province. The research location is focused on single excavation slope point where the landslide occurred, located at STA 17+850 of of the Bululawang-Sidomulyo-Tambakrejo section. The slope at this point has an excavation height of up to 40 m.

B. Geological Conditions

Geologically, this section is in the Wonosari Formation (Tmwl) with its main composition consist of coral limestone, clayey-tuffaceous-sandy limestone, marl, peaty black claystone, and calcirudite, as shown in Figure 3. Although carbonate rocks generally have a moderate to high Uniaxial Compressive Strength (UCS), the slope strength at STA 17+850 is more strongly influenced by rock mass conditions such as the degree of fracturing, weathering, and discontinuity quality than by the strength of the intact rock itself.

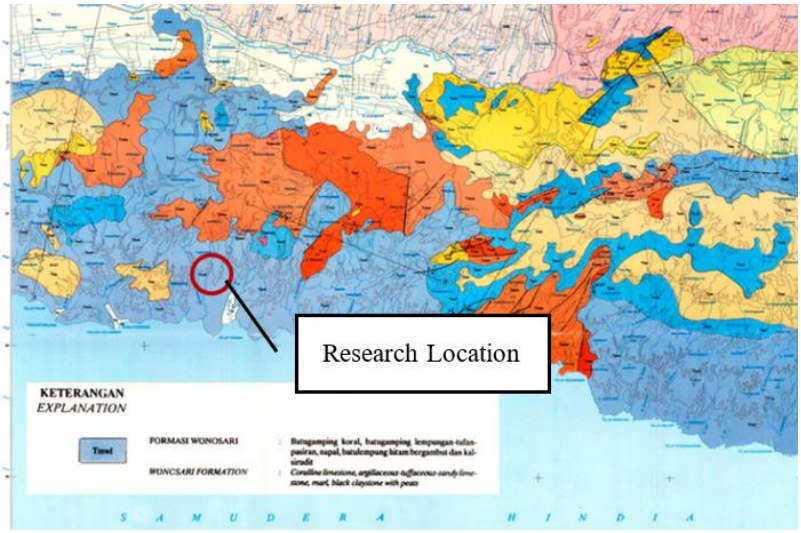


Figure 3. Geological Map of Blitar Regency (Sjarifudin & Hamidi, 1992)

According to Bina Marga (2020), the classification of excavations based on UCS refers to SNI 2825:2008, where soft rock excavation includes materials with *a* UCS of 0.6–12.5 MPa, while rock excavation refers to materials with *a* UCS of >12.5 MPa, and block size ≥ 1 m³. Therefore, the UCS value of the limestone at the study site can be used to determine whether the material is classified as soft rock or hard rock, serving as an initial reference before considering the condition of the rock mass.

C. Slope Stability

Slope stability can be predicted by the value of Safety Factor (SF) which is a comparison between the shear strength of the soil and the shear stress required for equilibrium (Duncan & Wright, 2005).

$$SF = \frac{Resisting\ Force}{Driving\ Force} \qquad (1)$$

The classification of slope stability based on SF values is shown in Table 1 as follows:

Table 1. Classification of Slope Stability by Safety Factor (Van Westen & Terlien, 1996)

Safety Factors	Slope Stability Class	Information
SF > 1.5	Stable	Only large triggers of instability (destabilization) can cause collapse
1.25 < SF < 1.5	Quite Stable	Factors that trigger moderate instability can lead to collapse
1 < SF < 1.25	Critical/Quasi Stable	A small instability trigger can lead to collapse
SF < 1	Unstable	Reinforcement measures are required (for the slope to be stable

The process of evaluating the balance between the driving force and the resisting force is

referred to as slope stability analysis (Hardiyatmo, 2002). The required safety factors for the stability analysis of rock slopes, considering both permanent or temporary conditions are shown in Table 2 as follows:

Table 2. Recommended Safety Factor values for Rock Slopes (SNI 8460, 2017)

Rock Slope Conditions	SF Recommendations
Permanent Conditions	1,5
Temporary Conditions	1,3

D. Rock Mass Quality

Rock slopes stability is greatly influenced by geological structures called discontinuities (such as joints, faults, and bedding planes). These structures act as weak planes within the rock mass, making them the primary locations where failure or landslides occur. One method to determine the quality of rock mass is to use the Geological Strength Index (GSI). GSI is widely used to estimate the reduction of rock mass strength under different geological conditions. The GSI value is determined based on a visual assessment of two fundamental characteristics of the rock mass observed at the outcrop (Hoek et al., 2002). The GSI assessment of the visual method was carried out using the table in Figure 4.

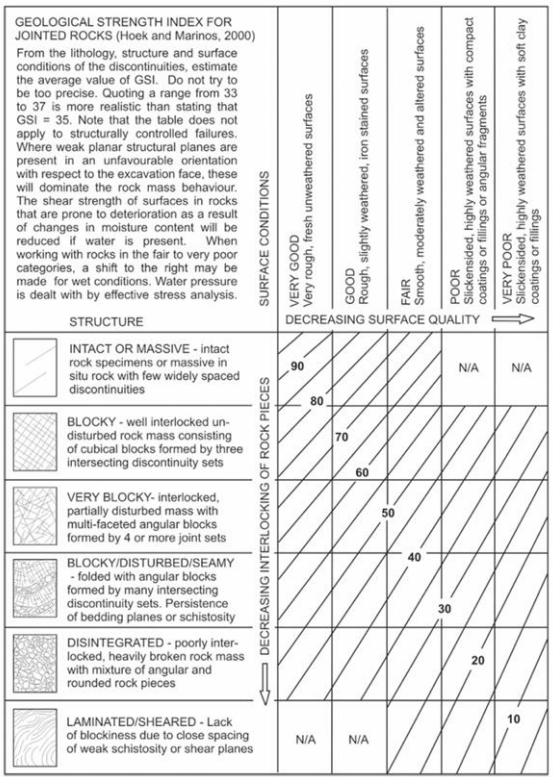


Figure 4. GSI value base chart (Marinos & Hoek, 2000)

In addition to the GSI value table, rock slope analysis requires the intact rock material constant (m_i), calculated based on Figure 5, as well as the value of the disturbance factor (D), calculated based on Figure 6.

Guidelines for the selection of modulus ratio (MR) values in Eq. (3)—based on Deere [24] and Palmstrom and Singh [15]

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
Sedimentary	Clastic		Conglomerates 300–400 Breccias 230–350	Sandstones 200–350	Siltstones 350–400 Greywackes 350	Claystones 200–300 Shales 150–250[a] Marls 150–200
	Non-clastic	Carbonates	Crystalline limestones 400–600	Sparitic limestones 600–800	Micritic Limestones 800–1000	Dolomites 350–500
		Evaporites		Gypsum (350)[b]	Anhydrite (350)[b]	
		Organic				Chalk 1000 +
Metamorphic	Non-foliated		Marble 700–1000	Hornfels 400–700 Metasandstone 200–300	Quartzites 300–450	
	Slightly foliated		Migmatite 350–400	Amphibolites 400–500	Gneiss 300–750[a]	
	Foliated[a]			Schists 250–1100[a]	Phyllites/Mica Schist 300–800[a]	Slates 400–600[a]
Igneous	Plutonic	Light	Granite[c] 300–550 Granodiorite[c] 400–450	Diorite[c] 300–350		
		Dark	Gabbro 400–500 Norite 350–400	Dolerite 300–400		
	Hypabyssal		Porphyries (400)[b]		Diabase 300–350	Peridotite 250–300
	Volcanic	Lava		Rhyolite 300–500 Andesite 300–500	Dacite 350–450 Basalt 250–450	
		Pyroclastic	Agglomerate 400–600	Volcanic breccia (500)[b]	Tuff 200–400	

[a]Highly anisotropic rocks: the value of MR will be significantly different if normal strain and/or loading occurs parallel (high MR) or perpendicular (low MR) to a weakness plane. Uniaxial test loading direction should be equivalent to field application.
[b]No data available, estimated on the basis of geological logic.
[c]Felsic Granitoids: coarse grained or altered (high MR), fined grained (low MR).

Figure 5. Value of m_i constant (Marinos & Hoek, 2000)

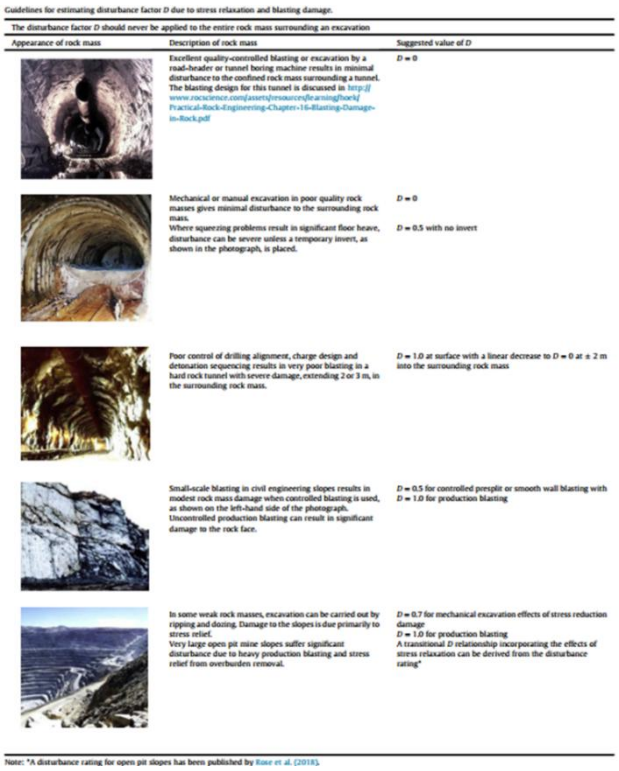


Figure 6. Value of the constant D (Marinos & Hoek, 2000)

The GSI value can also be calculated based on the Rock Mass Rating (RMR). The RMR value is determined using Bieniawski's (1989) equation, as shown in Equation 2. The RMR value is calculated on five main parameters, namely the Uniaxial Compressive Strength (UCS), Rock Quality Designation (RQD), discontinuity spacing, discontinuity condition, and groundwater condition. Once all parameters are obtained, each is assigned a rating, and the sum of these ratings produces the final RMR value.

$$GSI = RMR - 5 \qquad (2)$$

E. Determination of Material Models in Plaxis 2D

The 2002 Hoek-Brown Failure equation can be used as an approach to determine the Mohr–Coulomb (MC) parameters in performing rock slope stability analysis. The values of the safety factors produced by HB and MC modeling have a similar tendency, with a difference of less than 10% (Pratiwi, 2022). Stability analysis for weathered rocks is performed using the Generalized Hoek-Brown (GHB) and Mohr-Coulomb Equivalence approaches, the results of the analysis showed that the equivalent MC criterion produces higher or overly optimistic SF values compared with the GHB criterion. The difference is quite significant, at approximately 50.5%, indicating that the GHB criterion is more sensitive in identifying shallow (near-surface) failure potential, making it more suitable for analyzing weathered or weak rock slopes (Mulyanto, 2022).

The soil material model used in Plaxis 2D is the Hardening Soil (HS) model, however, for rock layers, using soil-based failure criteria is less appropriate because they do not account for the strength of discontinuity planes that may control rock mass failure. Therefore, in Plaxis 2D, rock layers can be more appropriately represented using the Hoek–Brown (HB) material model.

F. Research Flow Chart

The flow chart of this study covers the scope of the research shown in Figure 7.

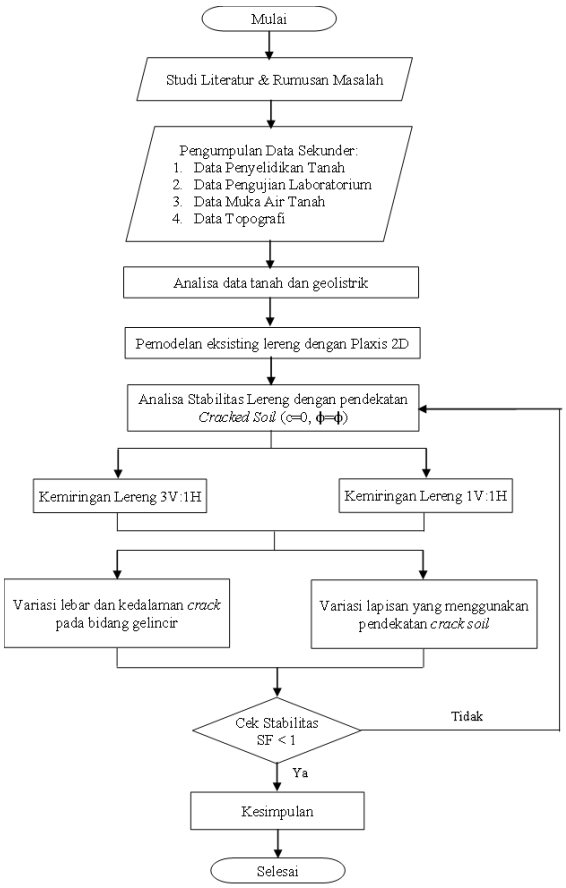


Figure 7. Research Flow Chart

Slope stability analysis was carried out to determine the safety factor (SF) and deformation values resulting from the excavation slope cutting, using two slope inclinations based on the existing design: 3V:1H (before regrading) and 1V:1H (after regrading). The cracked soil approach used in this study consists of two modeling methods:

1. "Behaving like sand″ modeling, which assumes that the soil layer behaves like sand. This condition occurs because the presence of cracks eliminates the cohesion within a particular soil layer (c = 0), while the internal friction angle (φ) remains the same as the field test results (φ = φ).
2. Crack modeling along the slip surface, which simulates a crack using a polygon placed at the slip surface location, with variations in crack width and depth. The soil cohesion within the polygon is removed (c = 0), while the internal friction angle (φ) remains consistent with the field test results (φ = φ).

RESULTS AND DISCUSSION

A. Stratigraphic Analysis

Stratigraphic analysis is used to define the soil and rock layers and their geological conditions both vertically and horizontally, ensuring that the geotechnical model accurately represents the actual field conditions. The soil data used in this study were obtained from a single borehole located at the top of the slope at STA 17+850 after the second landslide event. The Standard Penetration Test (SPT) N-values range from 43 to 60, as shown in Figure 8.

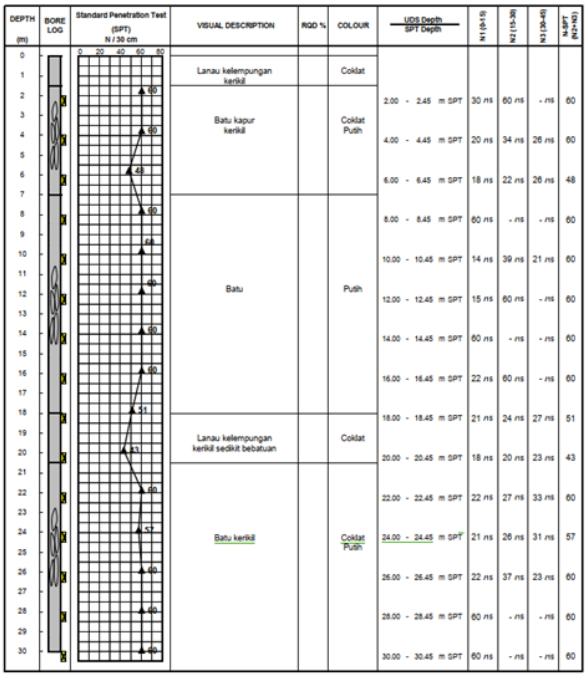


Figure 8. Results of Borelog STA 17+850 (Bina Marga, 2025)

As a comparative reference for determining the stratigraphy of the soil layers, previously collected geoelectrical data were used, as shown in Figure 9. At STA 17+850, most sections indicate layers with resistivity values greater than 78.6 Ωm, although lower resistivity values of 16.3–27.6 Ωm are also present at elevations of 16–24 m. This lower-resistivity layer corresponds to the borehole data, which identifies it as an interbedded layer of clayey silt with gravel and minor rock fragments.

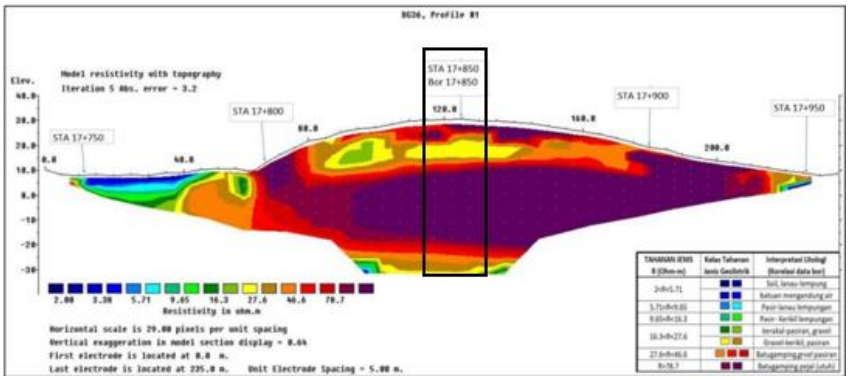


Figure 9. STA Geoelectric Test Results 17+750 – 17+950 (Bina Marga, 2025)

Based on these two datasets, the soil layer is divided into 6 layers. Layer 1 at a depth of 0 – 1.5 m consist of clayey silt with gravel, layer 2 at a depth of 1.5 m – 7 m is limestone with gravel, layer 3 at a depth of 7 m – 13 m is limestone, layer 4 at a depth of 13 m – 18 m is limestone, layer 5 at a depth of 18 m – 20 m consist of silt with gravel and minor rock fragments, and layer 6 at a depth of 20 m – 30 m is gravelly rock. Accordingly, layer 1 and layer 5 are classified as soil, while layer 2, 3, 4, and 6 are classified as rocks.

B. Parameters Used

The parameters obtained were used as input for the Plaxis 2D modeling. The summary of the soil parameters used is presented in Table 3, while the summary of the rock parameters is presented in Table 4.

Table 3. Values of the soil parameters used

Parameters	Layer 1	Layer 5
γ_{sat} (kN/m^3)	21,00	21,00
γ_{unsat} (kN/m^3)	21,00	21,00
c' (kN/m^2)	27,00	25,00
φ' (°)	22	22
E_{50} (kN/m^2)	18.000	18.933
E_{oed} (kN/m^2)	20.250	19.350
E_{ur} (kN/m^2)	54.000	56.800
v_{ur}	0,3	0,3

Table 4. Values of the rock parameters used

Parameters	Layer 2	Layer 3	Layer 4	Layer 6
γ_{sat} (kN/m^3)	17,33	18,49	19,99	22,44
γ_{unsat} (kN/m^3)	17,33	18,49	19,99	22,44
m_i	11	11	11	11
GSI	55	55	45	65
D	0,5	0,5	0,5	0,5
E (kN/m^2)	1.087.000	461.000	555.000	2.648.000

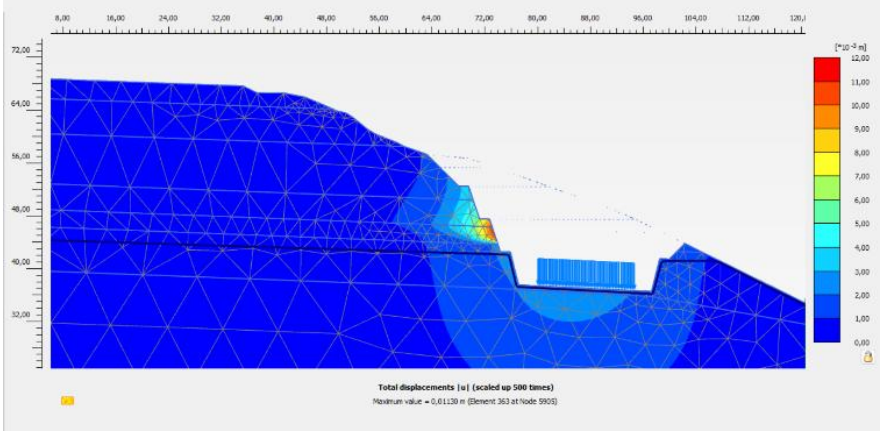
C. Analysis of Slope Stability of Initial Conditions (Excavated)

The results of the slope stability analysis in the initial condition (excavated), including

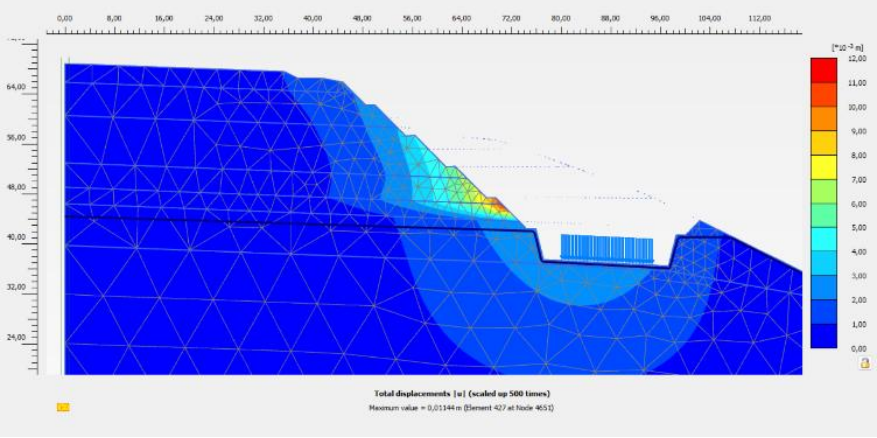
the slip surface scheme, along with the summary of the Safety Factor (SF) values and deformation values for both slope inclination models, are presented in Table 5.

Table 5. SF values and initial condition displacement (excavated)

Slope Inclination	SF	Displacement |u| (m)
3V:1H	1,51	0,0113
1V:1H	1,89	0,0114



(a)



(b)

Figure 10. Slip surface of the excavation slope under initial conditions (a) slope 3V:1H SF 1.51 (b) slope 1V:1H SF 1.89

The slip surface shown in Figure 10 is located on the second slope, which corresponds to the area where the landslide occurred (consistent with the actual field conditions). The SF values for both slope inclinations indicate SF >> 1, with the 3V:1H slope having an SF of 1.51 and the 1V:1H slope having an SF of 1.89. An SF value greater than 1.5 indicates that the slope is stable. Therefore, the initial (excavation) condition does not yet reflect the actual failure condition observed in the field, indicating the need for an alternative approach to determine the cause of the landslide.

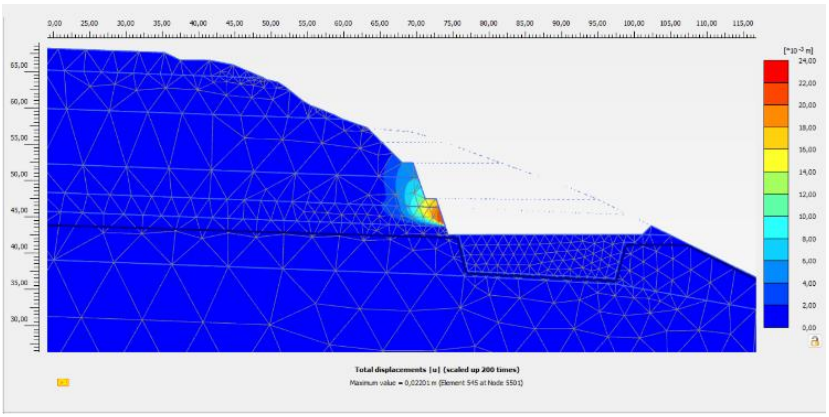
D. Slope Stability Analysis of the Cracked Soil Model 1 Approach

Slope stability analysis was performed using the cracked soil approach for Model 1, in which all soil layers are assumed to behave like sand ("behaving like sand"). In this model, soil cohesion is disregarded/eliminated (c = 0), while the internal friction angle remains considered (φ = φ). A summary of the analysis results, including the Safety Factor (SF) and deformation values, is presented in the following Table 6:

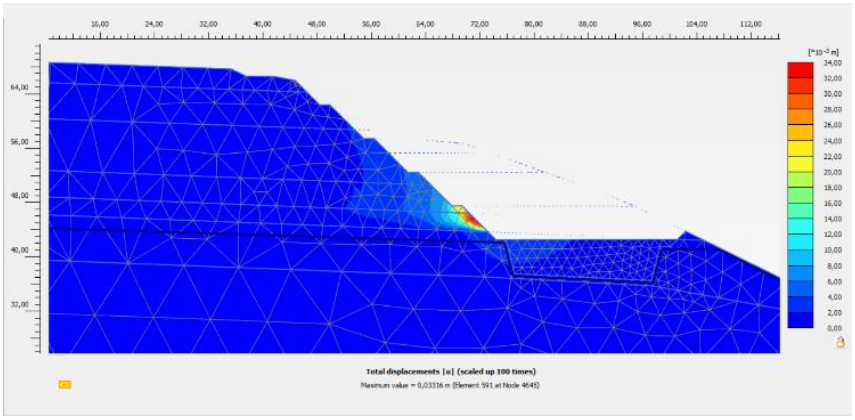
Table 6. SF and displacement values with the cracked soil model 1 approach

Type	Slope inclination	SF	Displacement |u| (m)
Behaving like sand	3V:1H	0,90	0,0220
	1V:1H	0,93	0,0332

The analysis results indicate that the Safety Factor (SF) is less than 1 for both slope inclinations (3V:1H and 1V:1H). Slope failure occurs after the excavation stage on the second slope, with the slip surface located precisely on the second slope, consistent with the initial (excavation) condition, as shown in Figure 11. This condition demonstrates that the modeling approach using cracked soil Model 1 is in good agreement with the actual field conditions.



(a)



(b)

Figure 11. Slip surfaces of the excavation slope modeled using cracked soil approach model 1 (a) slope 3V:1H SF 0.90 (b) slope 1V:1H SF 0.93

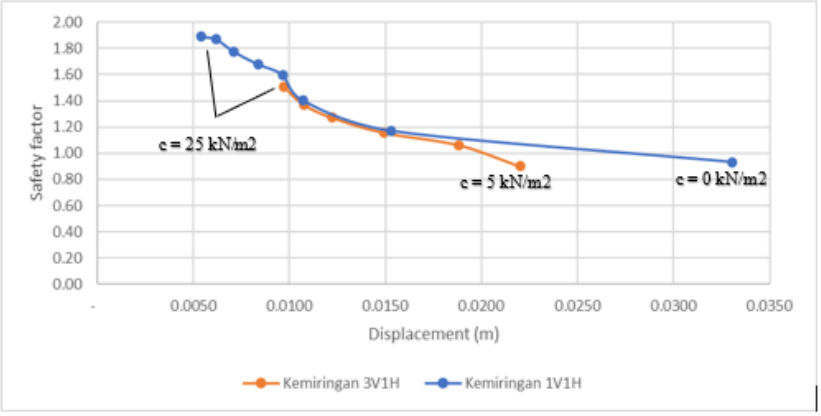


Figure 12. Relationship between the Safety Factor (SF) and deformation resulting from the reduction of cohesion (c) from the initial condition

The graph in Figure 12. is the relationship between Safety Factor (SF) and deformation (displacement) for soil experiencing a reduction in cohesion (c) using the cracked soil model 1 for both slope inclinations. The graph shows that as the cohesion decreases, the SF also decreases, while the deformation increases. Soil with higher cohesion exhibits smaller deformation because its structure is still able to resist loading, whereas soil with low or zero cohesion shows larger deformation due to the soil becoming more susceptible to movement. For the 3V:1H slope, SF < 1 (slope failure) is reached at c = 5 kN/m², while for the 1V:1H slope failure is reached at c = 0 kN/m². The graph indicates that the 1V:1H slope has higher stability compared to the 3V:1H slope. However, this slope design is still insufficient to withstand the working load induced by the presence of cracked soil.

E. Slope Stability Analysis of the Cracked Soil Model 2 Approach

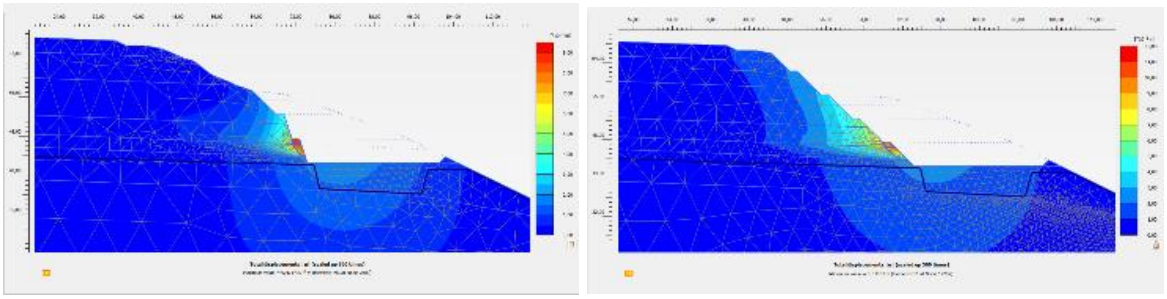
Slope stability analysis was carried out using the cracked soil model 2 approach, in which the soil located around the potential slip surface is assumed to behaving like sand with c = 0 and φ = φ while the soil outside the slip surface is considered to remain in accordance with the field conditions, with c = c and φ = φ. The slip surface was defined using a polygon whose width and depth were varied. A summary of the analysis results, including the Safety Factor (SF) and deformation, is presented in the following Table 7:

Table 7. SF and displacement values with the cracked soil model 2 approach

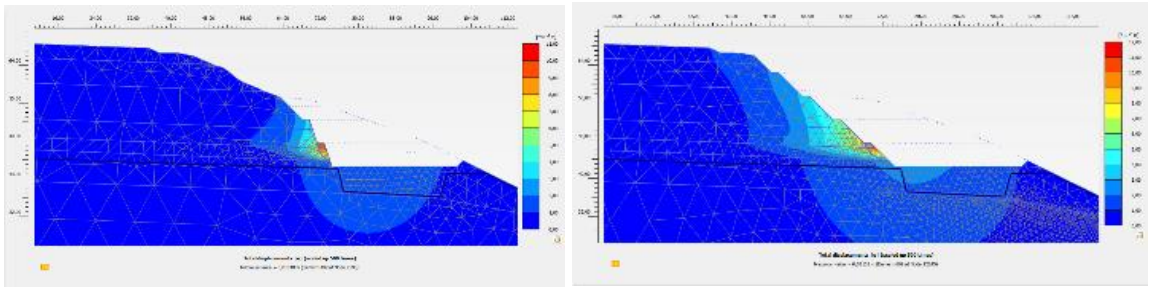
Crack Variations		**Slope 3V:1H**		**Slope 1V:1H**	
Width (m)	**Depth (m)**	**SF**	**|u| (m)**	**SF**	**|u| (m)**
0,025	2	1,50	0,011	1,89	0,011
	2,5	1,39	0,011	1,89	0,011
	3	1,17	0,011	1,88	0,011
	3,5	0,80	0,011	1,34	0,011
	4	0,18	0,010	0,57	0,011
0,05	2	1,48	0,011	1,89	0,011
	2,5	1,38	0,011	1,89	0,011
	3	1,14	0,011	1,86	0,011

	3,5	0,79	0,010	1,35	0,011
	4	0,20	0,010	0,58	0,011
	2	1,47	0,011	1,89	0,011
	2,5	1,35	0,011	1,89	0,011
0,1	3	1,11	0,011	1,86	0,011
	3,5	0,77	0,010	1,31	0,011
	4	0,19	0,095	0,58	0,011

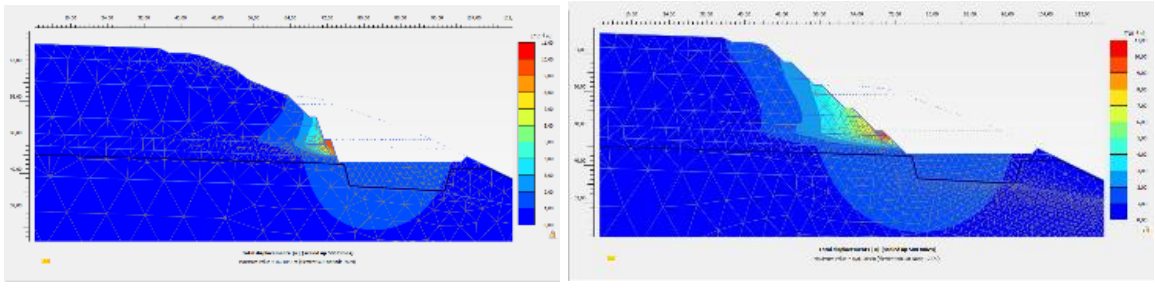
The analysis results indicate that for the 3V:1H slope, SF < 1 occurs at a crack depth of 3.5 m for all crack-width variations, while for the 1V:1H slope, SF < 1 occurs at a crack depth of 4 m for all crack-width variations. Failure occurred after the excavation stage on the second slope, and the slip surface was located precisely on the second slope, as shown in Figure 13. These findings demonstrate that the cracked soil model 2 approach accurately represents the actual field conditions.



L=0.1 m and Z=4 m



L=0.05 m and Z=4 m



L=0.025 m and Z=4 m

Figure 13. Slip surfaces of the excavated slope with 3V:1H and 1V:1H using cracked soil approach model 2

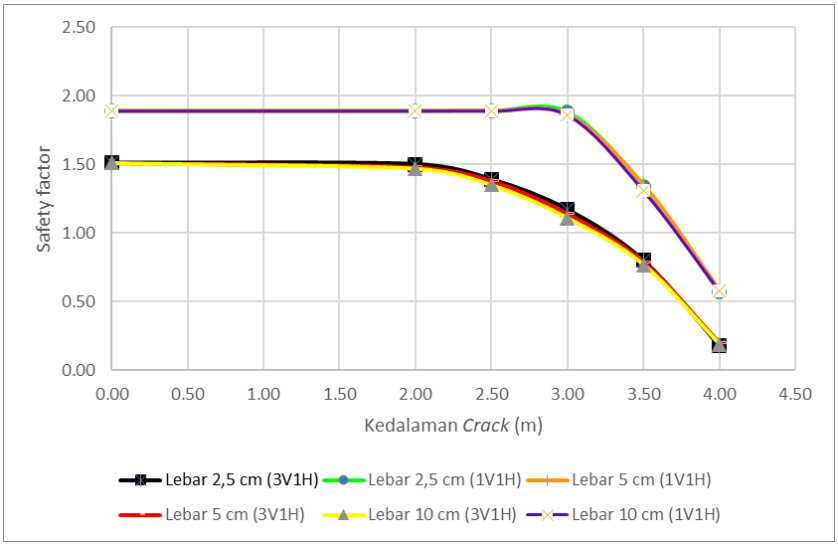


Figure 14. The relationship *of SF* values to crack width and depth variations

The graph in Figure 14. is the relationship between the Safety Factor (SF) and the variations in crack width and depth under the cracked soil model 2 approach for both slope inclinations. The graph shows that the deeper the crack, the lower the SF value; however, variations in crack width have only a minor influence on the reduction of the SF. For the 3V:1H slope, SF < 1—or slope failure—occurs at a crack depth of 3.5 m for all crack width variations, whereas for the 1V:1H slope, SF < 1 occurs at a crack depth of 4 m for all width variations. These results indicate that crack depth has a more significant effect on reducing the SF compared to crack width.

CONCLUSION

The conclusions of this study are:

1. The "cracked soil" approach where the soil “behaving like sand” using both model 1 (entire soil layers assumed to behave as sand with c = 0) and model 2 (only the zone along the slip surface assumed to behave as sand with c = 0) has been proven appropriate for the landslide case at STA 17+850. The analysis results, which show SF < 1 (indicating slope failure) for both slope inclinations of 3V:1H and 1V:1H, are consistent with the actual field conditions.
2. The Safety Factor decreases as the crack depth increases. This occurs because deeper cracks extend closer to or even intersect the potential slip surface, causing the loss of cohesion in the critical zone and allowing pore-water pressure to increase more easily due to water infiltration. Meanwhile, crack width does not significantly affect the reduction of the Safety Factor, because the width of the crack does not alter the geometry of the shear surface or the extent of the soil zone that loses cohesion. Thus, variations in crack width only produce local deformation at the ground surface but do not influence the overall global stability of the slope.
3. The main factor causing the landslide was the presence of a cracked-soil zone within the interbedded layer of limestone, specifically within the silty clayey gravel layer containing minor rock fragments at a depth of 18–21 m. Heavy to extremely heavy rainfall at the study site accelerated water infiltration into these cracks, thereby increasing pore-water

pressure, eliminating effective cohesion, and significantly weakening the layer. This condition triggered failure on the second slope, which subsequently propagated and caused additional failure on the third slope.

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