



Failure Mechanism of a Rainfall-Induced MSE Wall Embankment Revealed by Two-Stage Numerical Back-Analysis



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Abstract

Rainfall infiltration is one of the main causes of embankment failure because it changes the engineering properties of soil and reduces slope stability. Conventional stability analyses commonly assume uniform soil conditions, which may not represent the actual field condition after prolonged rainfall. This study investigates the failure mechanism of an MSE wall embankment at Kedungmlati 2 Overpass, Mojokerto–Kertosono Toll Road, using a two-stage numerical back-analysis. Laboratory tests were performed on embankment soils under different degrees of saturation ($S_r = 50\%$, 62.5% , 75% , 87.5% , and 100%) to determine the corresponding engineering properties. The numerical analyses were conducted using the finite element method implemented in PLAXIS 2D. Stage 1 evaluated six numerical models with uniform saturation conditions throughout the embankment. None of these models reproduced the observed failure mechanism. Stage 2 introduced a non-uniform saturation distribution to represent localized rainfall infiltration along the embankment slope. The modified model successfully reproduced the observed failure surface. The results demonstrate that the non-uniform saturation scenario adopted in Stage 2 provided a closer agreement with the observed failure mechanism than the uniform saturation scenarios evaluated in Stage 1. In this scenario, a saturated near-surface zone and a relatively unsaturated embankment core were assigned as part of the back-analysis procedure to reproduce the observed failure pattern. The proposed rehabilitation measures further increased the safety factor from 1.085 to 1.765, effectively restoring embankment stability. These findings suggest that considering spatial variations in the degree of saturation can provide a more representative prediction of rainfall-induced failure mechanisms than assuming a uniform saturation distribution. The novelty of this study lies in the proposed practical back-analysis approach, which combines a non-uniform saturation scenario with experimentally determined shear strength parameters at different degrees of saturation to reproduce the observed failure mechanism and evaluate rehabilitation strategies for MSE wall embankments without explicitly modeling transient seepage.

Keywords: rainfall infiltration; back-analysis; MSE wall; degree of saturation; PLAXIS; failure mechanism

1. Introduction

Toll roads require stable embankment slopes and retaining structures to ensure safe and reliable operation. In Indonesia, high rainfall frequently promotes water infiltration into embankment soils. In unsaturated soils, rainfall infiltration may reduce matric suction and shear strength, thereby decreasing slope stability [14]. In this study, however, transient seepage and matric suction were not modeled explicitly. Instead, different degrees of saturation were introduced as back-analysis scenarios, with shear strength parameters obtained from laboratory direct shear tests, to reproduce the observed failure mechanism of an MSE wall embankment. The risk becomes greater when the drainage system does not function properly, allowing water to accumulate within the embankment. Previous studies have shown that rainfall infiltration and inadequate drainage reduce slope stability [1], [2], [3]. However, slope stability analyses are commonly performed using soil properties obtained from site investigations without considering changes in soil properties associated with increasing soil saturation. As a result, the calculated stability may differ from actual field conditions, particularly for embankments reinforced with Mechanically Stabilized Earth (MSE) walls. In unsaturated soils, rainfall infiltration reduces matric suction, leading to a decrease in shear strength and slope stability [14].

Although these hydro-mechanical processes provide the theoretical basis for rainfall-induced failures, the present study does not explicitly model transient seepage or matric suction. Instead, different degrees of saturation are introduced as back-analysis scenarios, with shear strength parameters obtained from laboratory direct shear tests at each saturation level to reproduce the observed failure mechanism.

Previous studies have shown that rainfall infiltration reduces the factor of safety (FoS) of slopes by increasing pore water pressure and decreasing soil shear strength [4], [5]. Previous studies have investigated rainfall-induced slope failures using transient seepage analyses or spatially varying hydrological conditions [15]. In contrast, this study proposes a practical back-analysis approach that employs a non-uniform saturation scenario and laboratory-derived shear strength parameters at different degrees of saturation (Sr) to reproduce the observed failure mechanism of an MSE wall embankment. Such an assumption may not represent localized rainfall infiltration and may lead to an unrealistic prediction of the failure mechanism. Therefore, saturated conditions and pore water pressure variations should be considered in the design of slope reinforcement systems. One such case occurred at Kedungmlati 2 Overpass (STA 0+950) on the Mojokerto–Kertosono Toll Road, where the MSE wall, geogrid reinforcement, and drainage system were damaged following a high-intensity rainfall event [6], [7]. However, limited studies have evaluated whether a non-uniform distribution of soil saturation can better reproduce the observed failure mechanism through numerical back-analysis. This study aims to identify the slope failure mechanism and evaluate remedial measures using numerical modeling with PLAXIS [8], [9]. The analysis integrates previous and recent site investigation data together with laboratory testing conducted under different degrees of saturation.

2. Method

The study obtained data from field investigations and laboratory testing of embankment soil samples, historical and recent site investigation reports, and records of slope failure. The engineering properties of the embankment soil were obtained through laboratory testing performed in accordance with the Indonesian National Standards (SNI). Physical properties, including unit weight, were determined following SNI 03-3637-1994, while water content and specific gravity were measured according to SNI 03-1971-2011 and SNI 1964:2008, respectively. Compaction characteristics, including optimum moisture content and maximum dry density, were obtained using the Standard Proctor test in accordance with SNI 1742:2008. The shear strength parameters were determined using triaxial compression testing (SNI 4813:2015) and direct shear testing (SNI 03-3420-1994). The Laboratory tests were conducted to determine the engineering properties of embankment soils at different degrees of saturation (Sr) of 50%, 62.5%, 75%, 87.5%, and 100%, which were subsequently used as input parameters for the numerical model. Slope stability was analyzed using the finite element method (FEM) implemented in PLAXIS. The initial analysis employed the soil parameters obtained from the recent site investigation to evaluate the existing slope condition. A back-analysis [7], [10] was then performed by incorporating different degrees of saturation together with historical site investigation data, recent field investigation data, and laboratory test results until the predicted failure surface closely matched the observed slope failure.

The failure mechanism was investigated using a finite element model developed from the latest site investigation. Undisturbed soil samples were collected from three boreholes along the embankment, including BH3 and BH1 in the embankment fill and BH2 in the foundation soil (Fig. 1). Laboratory testing was conducted to determine the unit weight, saturated unit weight, Young's modulus, Poisson's ratio, cohesion, internal friction angle, porosity, and degree of saturation (Sr) of the embankment soil. The calculated Sr values ranged from 55% to 98%. Therefore, five saturation scenarios (Sr = 50%, 62.5%, 75%, 87.5%, and 100%) were adopted for the numerical analysis. The laboratory tests showed that the engineering properties of the embankment soil varied with the degree of saturation (Sr) [11].

Therefore, the corresponding values of unit weight, Young's modulus, Poisson's ratio, cohesion, internal friction angle, and saturated unit weight obtained for each saturation condition were assigned to the numerical model. The porosity was assumed to remain constant because no significant volumetric change of the soil specimens was observed during testing.

One of the shear strength parameters that exhibited a noticeable change with increasing degree of saturation (Sr) was cohesion. The laboratory test results showed that cohesion gradually decreased as the degree of saturation

increased. The cohesion values for each saturation level are presented in Table X, while the corresponding decreasing trend is illustrated in Fig. X.

Table 1. Cohesion of Embankment Soil at Different Degrees of Saturation (Sr)

Soil Properties	Unit	Soil Parameter Values				
Degree of Saturation SR	%	50	62,5	75	87,5	100
Cohesion Embankment 1	kN/m ²	29,13	22,85	15,89	4,81	11,38
Cohesion Embankment 2	kN/m ²	31,19	28,44	22,16	14,91	13,83
Cohesion Embankment 3	kN/m ²	10,69	19,42	17,36	15,89	17,36
Cohesion Embankment 4	kN/m ²	53,64	53,45	24,22	10,40	55,51
Cohesion Embankment 5	kN/m ²	47,66	55,80	24,22	12,16	1,77
Cohesion Embankment 6/7	kN/m ²	24,61	10,10	13,14	76,79	2,45

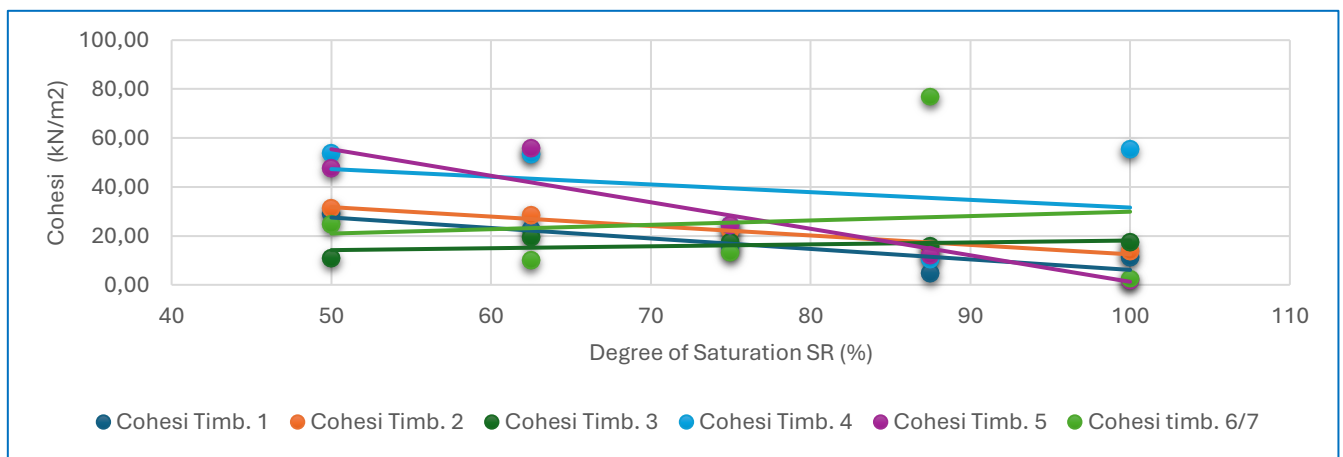


Figure 1. Trend of Cohesion Reduction with Increasing Degree of Saturation (Sr)

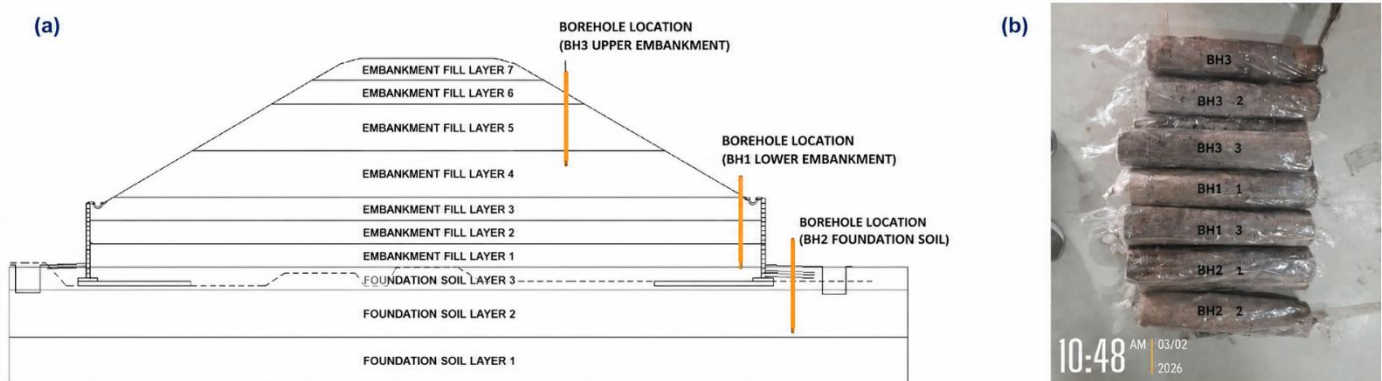


Figure 2. Borehole locations and undisturbed soil samples used for laboratory testing: (a) Cross-section of the existing embankment showing the borehole locations (vertical orange lines). BH3 and BH1 were drilled in the embankment fill, whereas BH2 was drilled in the foundation soil, and (b) Undisturbed soil samples collected from boreholes BH3, BH1, and BH2.

Three boreholes (BH1, BH2, and BH3) were selected to represent the geotechnical conditions of the failed MSE wall embankment. BH1 was drilled to a depth of 3 m at the center of the approximately 20 m-long landslide zone, extending from the upper slope to the embankment toe, to characterize the soil properties within the critical failure region. BH3, drilled to a depth of 6 m in the upper embankment fill outside the main failure zone, was used to characterize the engineering properties of the surface fill directly exposed to rainfall infiltration. BH2 was advanced to a depth of 5 m below the original ground surface to characterize the underlying natural foundation soil supporting the embankment. The three boreholes were strategically positioned to represent the upper embankment fill, the failed

embankment zone, and the foundation soil, thereby capturing the principal spatial variability of the embankment materials for laboratory testing and numerical back-analysis.

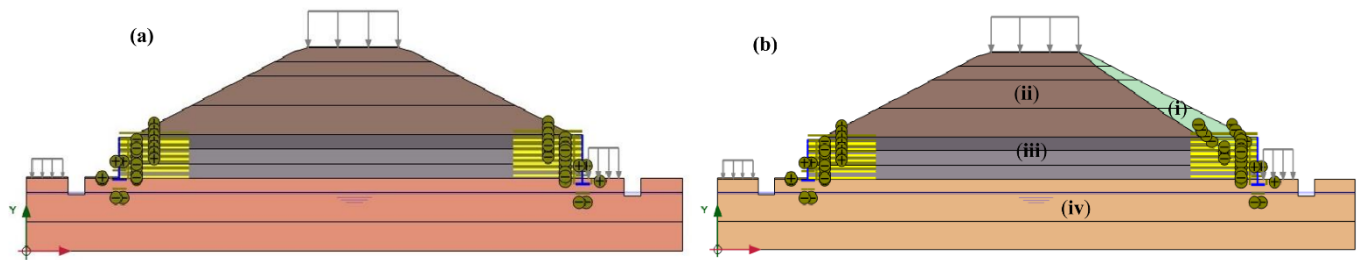


Figure 3. Numerical models used for back-analysis. (a) Stage 1: Initial numerical model with a uniform degree of saturation assigned to all embankment soil layers, (b) Stage 2: Modified numerical model with non-uniform soil saturation, where region (i) was assigned $S_r = 100\%$, regions (ii) and (iii) were assigned $S_r = 50\%$, and region (iv) retained the initial saturation condition.

Figure 2 shows the initial numerical model of the existing embankment. The embankment consists of a 6 m high sloping section with a slope inclination of 1V:2H, supported by a 3 m high mechanically stabilized earth (MSE) wall on both sides. The embankment fill comprises seven compacted soil layers overlying three foundation soil layers. The predicted failure mechanism, factor of safety (FoS), and slope deformation were compared with the observed landslide to identify the initial back-analysis model.

The embankment geometry and soil stratigraphy were modeled in PLAXIS using the laboratory-derived soil properties. The degree of saturation (S_r) was calculated from the measured water content, unit weight, and specific gravity using the soil phase relationships. Therefore, five saturation scenarios ($S_r = 50\%$, 62.5%, 75%, 87.5%, and 100%) were adopted for the numerical analysis. Stage 1 consisted of six numerical models. The first model was developed using the engineering properties obtained from the laboratory tests under the initial field condition. Five additional models were then generated by assigning uniform degrees of saturation ($S_r = 50\%$, 62.5%, 75%, 87.5%, and 100%) to all embankment fill layers. The predicted failure mechanism from each model was compared with the observed landslide to identify the model that best represented the field condition.

The embankment geometry and soil stratigraphy remained unchanged for all simulations. Only the engineering properties of the embankment soil were updated according to the laboratory test results for each degree of saturation (S_r). Each saturation condition was simulated separately to evaluate the corresponding failure mechanism, factor of safety (FoS), and slope deformation. The predicted failure mechanism was evaluated by qualitative visual comparison with the observed landslide, considering the location, shape, depth, and extent of the failure surface. A simulation was accepted when it closely matched the observed failure pattern. Owing to the lack of quantitative field monitoring data, this assessment was based solely on visual interpretation.

When the initial model failed to reproduce the observed failure mechanism, a second-stage model was developed by introducing a non-uniform distribution of the degree of saturation (S_r) (Fig. 2b). The embankment was divided into four zones based on the expected influence of rainfall infiltration. The Stage 2 model was developed to represent the expected rainfall infiltration pattern in the embankment. Rainfall infiltration was assumed to occur mainly through the exposed slope. Therefore, Region (i) was assigned $S_r = 100\%$ to represent the saturated zone, whereas Regions (ii) and (iii) were assigned $S_r = 50\%$ to represent the relatively unsaturated embankment core. This saturation distribution was introduced as a back-analysis scenario to reproduce the observed failure mechanism and should not be interpreted as a directly verified rainfall infiltration pattern in the field. The foundation soil (region iv) retained the initial saturation condition because no evidence of foundation instability was observed during the field investigation. The modified model was subsequently analyzed to determine whether the simulated failure mechanism more closely matched the observed landslide.

3. Results and Discussion

The numerical model was developed in PLAXIS 2D 2024 using the finite element method (FEM). The embankment and foundation soils were modeled using the Mohr–Coulomb constitutive model. The engineering properties were assigned based on the laboratory test results.

Table 2. MSE Wall Parameters

No	Properties	Value
1	w	4,8 kN/m/m
2	EA1	5.000.000 kN/m
3	EI	16.666,67 kN/m ² /m
4	Poisson ratio	0,2

Table 3. Geogrid Parameters

No	Properties	Vakue
1	Tipe	Tensar SS30
2	Sifat Material	<i>Elastoplastic</i>
3	EA	30
4	Np	525 kN/m

Table 4. Initial Geotechnical Parameters of the Embankment Soil

Soil Properties	Unit	Foundation Soil		
		Layer 1	Layer 2	Layer 3
Unit weight	Y_{unsat} kN/m ³	17,82	16,48	16,54
Saturation Unit Weight	Y_{sat} kN/m ³	17,85	16,71	16,59
Modulus Elastisitas	E	3666,95	2839,37	2096,91
Poisson ratio	v	0,30	0,30	0,30
Kohesi	c kN/m ²	58,15	40,40	29,52
Friction Angle	ϕ degree	3,80	3,10	1,90
Porosity	n	0,4628	0,5528	0,5552

Table 5. Initial Embankment Soil Parameters

Soil Properties	Unit	Embankment Initial					
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6/7
Unit weight	Y_{unsat} kN/m ³	16,43	16,64	16,02	16,44	13,95	15,57
Saturation Unit Weight	Y_{sat} kN/m ³	16,51	17,37	18,22	16,66	16,44	16,08
Modulus Elastisitas	E	6.771,64	4.524,63	7.626,24	6.771,64	7.626,24	6.150,93
Poisson ratio	v	0,30	0,30	0,30	0,30	0,30	0,30
Kohesi	c kN/m ²	11,38	14,91	10,69	55,51	47,66	76,79
Friction Angel	ϕ degree	40,80	47,45	43,18	7,60	13,70	5,60
Porosity	n	0,5215	0,4790	0,4488	0,5411	0,5735	0,5950

Table 6. Embankment Soil Parameters at Sr = 50%

Soil Properties	Unit	Embankment SR 50%					
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6/7
Unit weight	Y_{unsat} kN/m ³	13,95	15,02	16,02	14,32	13,95	13,16
Saturation Unit Weight	Y_{sat} kN/m ³	16,50	17,37	18,22	16,66	16,44	16,07
Modulus Elastisitas	E	3.173,2	6.192,6	7.626,2	3.050,5	7.626,2	2.563,3
Poisson ratio	v	0,30	0,30	0,30	0,30	0,30	0,30
Kohesi	c kN/m ²	29,13	31,19	10,69	53,64	47,66	24,61
Friction Angel	ϕ degree	36,90	38,40	43,18	12,30	13,70	5,80
Porosity	n	0,5214	0,4790	0,4488	0,5411	0,5735	0,5950

Table 7. Embankment Soil Parameters at Sr = 62.5%

Soil Properties	Unit	Embankment SR 62,5%					
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6/7
Unit weight	Y_{unsat} kN/m ³	13,95	15,02	16,02	14,32	13,95	13,16
Saturation Unit Weight	Y_{sat} kN/m ³	16,50	17,37	18,22	16,66	16,44	16,07
Modulus Elastisitas	E	2.621,4	4.761,8	4.472,4	2.960,6	2.801,9	2.383,1
Poisson ratio	v	0,30	0,30	0,30	0,30	0,30	0,30
Kohesi	c kN/m ²	22,85	28,44	19,42	53,45	55,80	10,10
Friction Angel	ϕ degree	25,50	34,20	50,20	8,50	9,20	8,50
Porosity	n	0,5214	0,4790	0,4488	0,5411	0,5735	0,5950

Table 8. Embankment Soil Parameters at Sr = 75%

Soil Properties	Unit	Embankment SR 75%					
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6/7
Unit weight	Y_{unsat} kN/m ³	13,95	15,02	16,02	14,32	13,95	13,16
Saturation Unit Weight	Y_{sat} kN/m ³	16,50	17,37	18,22	16,66	16,44	16,07
Modulus Elastisitas	E	2.956,0	3.016,4	4.041,1	3.487,3	3.809,1	3.966,6
Poisson ratio	v	0,30	0,30	0,30	0,30	0,30	0,30
Kohesi	c kN/m ²	15,89	22,16	17,36	24,22	24,22	13,14
Friction Angel	ϕ degree	30,50	33,00	42,70	37,30	43,70	29,20

Table 9. Embankment Soil Parameters at Sr = 87.5%

Soil Properties	Unit	Embankment SR 87,5%					
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6/7
Unit weight	Y_{unsat} kN/m ³	13,95	15,02	16,02	14,32	13,95	13,16
Saturation Unit Weigh	Y_{sat} kN/m ³	16,50	17,37	18,22	16,66	16,44	16,07
Modulus Elastisitas	E	3618,3	4524,6	5431,0	2988,0	4285,9	6150,9
Poisson ratio	v	0,30	0,30	0,30	0,30	0,30	0,30
Kohesi	c kN/m ²	4,81	14,91	15,89	10,40	12,16	76,79
Friction Angel	ϕ degree	30,90	47,45	39,40	40,80	35,60	5,60
Porosity	n	0,52	0,4790	0,4488	0,5411	0,5735	0,5950

Table 10. Embankment Soil Parameters at Sr = 100%

Soil Properties	Unit	Embankment SR 100%					
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6/7
Unit weight	Y_{unsat} kN/m ³	13,95	15,02	16,02	14,32	13,95	13,16
Saturation Unit Weight	Y_{sat} kN/m ³	16,50	17,37	18,22	16,66	16,44	16,07
Modulus Elastisitas	E	6.771,6	3.189,1	5.323,6	6.771,6	2.027,7	2.669,7
Poisson ratio	v	0,30	0,30	0,30	0,30	0,30	0,30
Kohesi	c kN/m ²	11,38	13,83	17,36	55,51	1,77	2,45
Friction Angel	ϕ degree	40,80	32,20	35,00	7,60	44,10	28,50
Porosity	n	0,5214	0,4790	0,4488	0,5411	0,5735	0,5950

Table 11. Embankment Soil Parameters at Sr = 50% + 100%

Soil Properties	Unit	Embankment SR 50%						Sr (100 %)
		Layer 1	Layer 2	Layer 3	Layer 4	Layer 5	Layer 6/7	
Unit weight	Y_{unsat} kN/m ³	13,95	15,02	16,02	14,32	13,95	13,16	13,16
Saturation Unit Weight	Y_{sat} kN/m ³	16,50	17,37	18,22	16,66	16,44	16,07	16,07
Modulus Elastisitas	E	3.173,2	6.192,6	4.682,9	3.050,5	7.626,2	2.563,3	2.669,7
Poisson ratio	v	0,30	0,30	0,30	0,30	0,30	0,30	0,30
Kohesi	c kN/m ²	29,13	31,19	10,69	53,64	47,66	24,61	2,45
Friction Angel	ϕ degree	36,90	38,40	43,18	12,30	13,70	5,80	28,50
Porosity	n	0,5214	0,4790	0,4488	0,5411	0,5735	0,5950	0,60

Figure 3 shows the incremental displacement patterns obtained from the Stage 1 back-analysis. The contour color represents the magnitude of incremental displacement, where blue indicates negligible displacement, green indicates low displacement, yellow indicates moderate displacement, and red indicates the highest incremental displacement. The red zone represents the area where deformation is concentrated during failure development.

Figures 3(a), 3(b), and 3(d) show similar incremental displacement patterns. The foundation soil and most of the embankment exhibit negligible incremental displacement (blue). The highest incremental displacement develops beneath the embankment crest and extends toward the right slope. The deformation decreases gradually from red to yellow, green, and blue. The maximum incremental displacement is 0.393 m, 0.630 m, and 0.659 m for Figs. 3(a), 3(b), and 3(d), respectively. However, the predicted deformation pattern does not coincide with the observed failure surface. Figure 3(c) shows a different deformation pattern. Figure 3(c) shows an incremental displacement of approximately 800 m, which greatly exceeds the embankment height (9 m). This value is not physically meaningful but reflects numerical divergence after failure during the Strength Reduction analysis. Therefore, the discussion focuses on the failure mechanism and the corresponding factor of safety (ΣMsf). The highest incremental displacement is concentrated beneath the traffic load, while the embankment slope and foundation remain almost entirely blue. No continuous deformation zone develops toward the observed failure surface. The maximum displacement is not discussed because its value is considerably larger than those of the other models. Figures 3(e) and 3(f) show localized deformation with relatively small incremental displacement. The maximum displacement is only 0.833×10^{-3} m and 1.178×10^{-3} m, respectively. Most of the embankment and foundation remain blue, indicating negligible displacement. The predicted deformation is limited to the upper embankment and does not follow the observed failure surface. None of these deformation patterns agrees with the observed failure surface.

The uniform saturation scenarios considered in Stage 1 did not reproduce the observed failure mechanism. Therefore, the back-analysis was continued by introducing a non-uniform distribution of the degree of saturation (S_r), assuming that rainfall infiltration occurred primarily along the embankment slope.

Figure 4 shows the predicted incremental displacement pattern of the Stage 2 back-analysis model. The highest incremental displacement is concentrated within the saturated zone (region i), where $S_r = 100\%$ was assigned to represent rainfall-induced saturation along the embankment slope. The displacement propagates downslope through zones of moderate and low incremental displacement, whereas the remaining embankment and foundation soil remain dominated by negligible incremental displacement. The predicted deformation is confined to the upper slope and does not extend into the embankment core or the foundation soil.

The predicted deformation pattern closely follows the observed failure surface (red dashed line). The failure initiates near the shoulder of the embankment and propagates along the saturated zone toward the toe of the MSE wall. In contrast, the embankment core (regions ii and iii), assigned $S_r = 50\%$, exhibits only negligible incremental displacement, indicating that the relatively unsaturated fill remains stable. Likewise, the foundation soil (region iv), which retained the initial saturation condition, shows insignificant deformation and does not participate in the failure mechanism.

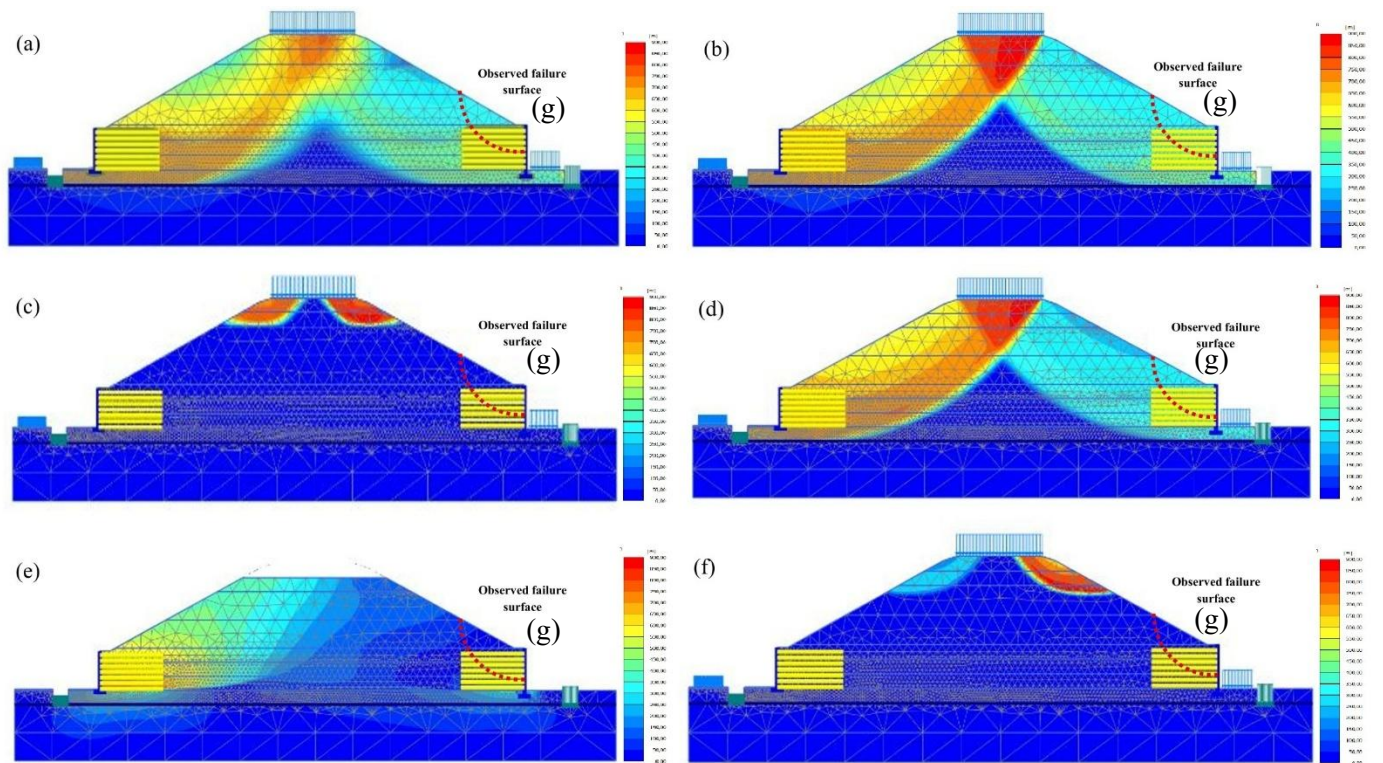


Figure 4. Predicted incremental displacement patterns obtained from the Stage 1 back-analysis under different degrees of soil saturation: (a) initial field condition, (b) $S_r = 50\%$, (c) $S_r = 62.5\%$, (d) $S_r = 75\%$, (e) $S_r = 87.5\%$, and (f) $S_r = 100\%$. The dashed line (g) indicates the observed failure surface.

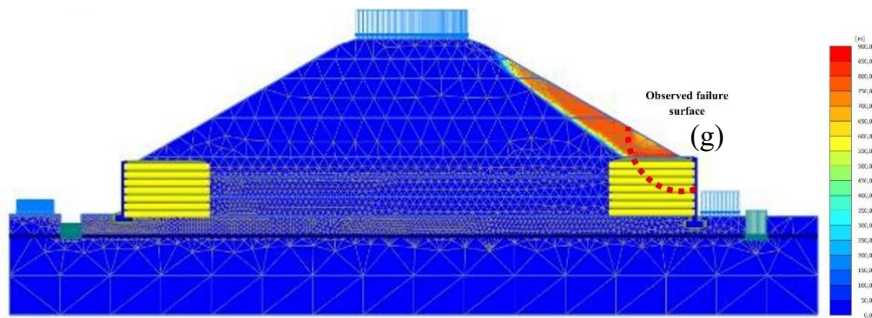


Figure 5. Predicted incremental displacement pattern of the final Stage 2 back-analysis model. The predicted failure mechanism closely follows the observed failure surface (red dashed line) identified from the field investigation.

These results indicate that the localized increase in the degree of saturation along the embankment slope is sufficient to reproduce the observed failure mechanism. The Stage 2 model provides a closer agreement with the field observations than the Stage 1 models, where uniform saturation conditions failed to capture the location and extent of the observed landslide. The comparison indicates that the failure mechanism was governed by localized rainfall infiltration rather than uniform saturation of the entire embankment. Assigning a saturated condition only to the near-surface slope successfully reproduced the observed failure surface, whereas the uniform saturation scenarios in Stage 1 did not.

The predicted failure surface closely matched the observed landslide geometry. The results indicate that a non-uniform saturation distribution, represented by a saturated near-surface zone and a relatively unsaturated embankment core, provided a more realistic representation of the observed failure mechanism than the uniform saturation scenarios. It should be noted that this saturation distribution was introduced as a back-analysis scenario rather than a directly verified rainfall infiltration pattern. The shear strength parameters assigned to each region were obtained from laboratory direct shear tests performed on soil specimens prepared at the corresponding degree of saturation (S_r).

Consequently, the analysis directly incorporated the measured shear strength parameters for each S_r condition without explicitly modeling rainfall infiltration or matric suction. Under this scenario, the predicted failure surface developed above the MSE wall and did not intersect the reinforced zone, indicating that the MSE wall remained outside the primary failure mass.

The Stage 2 model reproduced the observed failure mechanism more closely than the Stage 1 models. The predicted failure surface developed within the saturated zone and followed the observed failure surface along the embankment slope. In contrast, the embankment core and the foundation soil remained stable because only negligible incremental displacement was observed. These results indicate that rainfall infiltration was concentrated near the embankment slope rather than uniformly distributed throughout the embankment.

Table 12. Safety Factor (SF) Values at Different Degrees of Saturation (S_r)

No	Conditions	SF Embankment. 7	SF Load	SF Seismic
1	Initial	1,823	1,602	1,081
2	SR 50%	1,929	1,677	1,274
3	SR 62,5%	1,881	1,424	1,186
4	SR 75%	1,648	1,411	1,180
5	SR 87,5%	1,544	1,380	1,019
6	SR 100%	1,162	Soil Body Collapse	Soil Body Collapse
7	SR 50%+SR100%	1,085	1,084	Soil Body Collapse

The non-uniform saturation scenario successfully reproduced the observed failure surface, whereas the uniform saturation scenarios did not. In this back-analysis, the near-surface slope was assigned $S_r = 100\%$ and the embankment core $S_r = 50\%$. The predicted failure surface developed above the MSE wall, while the embankment core and foundation soil remained stable, indicating that the reinforced zone was outside the primary failure mass. These findings are consistent with the principles of unsaturated soil mechanics and previous studies on rainfall-induced slope failures considering spatially varying hydrological conditions[13];[14]; [15].

The two-stage back-analysis successfully reproduced the observed failure mechanism with a safety factor of 1.085. Rainfall infiltration increased the degree of saturation in the near-surface embankment, reducing matric suction, effective stress, and shear strength, with cohesion decreasing from 23.67 to 11.38 kPa and the friction angle from 39.49° to 33.60° . This localized degradation triggered progressive failure above the MSE wall, while the proposed rehabilitation measures increased the safety factor to 1.765

4. Conclusions

The Stage 1 back-analysis showed that the uniform saturation scenario could not reproduce the observed failure mechanism, although increasing the degree of saturation (S_r) consistently reduced the safety factor (SF) from 1.677 ($S_r = 50\%$) to 1.424 ($S_r = 62.5\%$), 1.411 ($S_r = 75\%$), and 1.380 ($S_r = 87.5\%$), which is below the allowable value of 1.5. A global body collapse occurred only at full saturation ($S_r = 100\%$), and none of the six numerical models reproduced the observed failure surface. These results indicate that assuming uniform saturation is insufficient to represent the field conditions and that a non-uniform saturation distribution is required to realistically reproduce the observed failure mechanism.

A closer agreement with the observed failure mechanism was obtained in Stage 2 by introducing a non-uniform degree of saturation within the embankment. Assigning a saturated condition ($S_r = 100\%$) to the near-surface slope and a relatively unsaturated condition ($S_r = 50\%$) to the embankment core successfully reproduced the observed failure surface, while the foundation soil remained stable. These results indicate that localized rainfall infiltration governed the failure mechanism. Therefore, the spatial variation of soil saturation and the corresponding changes in engineering properties should be considered in the back-analysis and stability assessment of MSE wall embankments subjected to

rainfall infiltration. The proposed procedure can be used as a reference for the back-analysis of rainfall-induced failures in MSE wall embankments with similar geological conditions.

The proposed back-analysis approach provides practical guidance in evaluating the instability of Mechanically Stabilized Earth (MSE) wall embankments due to rainfall infiltration. For field conditions with similar characteristics, the use of shotcrete can be considered to reduce rainwater infiltration, while gabion structures can be applied at the toe of the slope to enhance stability and control erosion.

Acknowledgment

The authors would like to express their sincere gratitude to the Institut Teknologi Sepuluh Nopember (ITS) Project Team for providing the research data and for their valuable technical support throughout this study. The authors also gratefully acknowledge the Soil Mechanics Laboratory, Department of Civil Engineering, Institut Teknologi Sepuluh Nopember (ITS), for conducting the laboratory testing of the soil samples. Special appreciation is extended to the landowner for granting permission to collect the data required for this research. The authors further thank all individuals and institutions whose support and cooperation contributed to the successful completion of this study.

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