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A GENERAL FORMULATION TO DESCRIBE THE EMPIRICAL PREDICTION OF THE CRITICAL AREA OF A LANDSLIDE

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Abstract

This study aims to find a general formulation for prediction of the critical area of a landslide through the analysis of the critical slip surface of embankment landslides in various conditions. This study uses several variables such as the typical soil subgrade and the embankment dimension. This research was conducted by the limit equilibrium method to obtain the value of the Safety Factor, circular centre, resistance moment, and several other landslide variables. The results obtained from this study show that the minimum landslide Safety Factor does not produce the largest amount of reinforcement requirements. In addition, the landslide area which produces the largest amount of reinforcing needs is different for each variable used. Empirical formulations were generated in this study to help construction designers to carry out reinforcement of road embankments to provide safety from landslides.

Keywords: Bishop method, Critical area of landslide, Embankment reinforcement, Landslide empirical prediction, Limit equilibrium method.

1. Introduction

Slope stability analysis is a very important issue in the engineering speciality in the geotechnical field. This topic has attracted extensive attention across the world [1, 2]. Many researches have been conducted on sliding surface searching technology in slope stability analyses since the 1970s [3, 4]. The Limit Equilibrium Method (LEM) is one very common method that is widely used and applied to analyse the stability of slopes with various slope dimensions and engineering geological conditions in the field [5, 6].

Basically, two steps are undertaken to analyse the slope stability using this method: in the first step, the position of the slope's potential sliding surface is found, while the slope stability of this surface is analysed in the second step.

To obtain the values of the Safety Factor (SF) and other variables of slope stability, some formulas in the LEM have been developed in the most recent research. In addition, many formulations have been developed to obtain the critical slip surface of the landslide on the embankment. The critical slip surface calculation is performed to determine the area of landslides on embankment bodies in the field to carry out the reinforcement design. Generally, the smallest SF values provide a standard for calculating the reinforcement requirements. In fact, the smallest value of the SF does not necessarily produce the greatest number of reinforcement needs [7]. In addition, areas of landslides that occur in the field could not be ascertained. The uncertainty of the landslides that occur in this field certainly makes it difficult for construction designers to determine the best reinforcement to ensure the safety of the embankment in relation to sliding.

This study concentrates on analysing the issues and difficulties of embankment reinforcement designs that generally occur in the field. Many problems and difficulties happen during the process of analysing the stability of a road embankment with high elevation. In the process of embankment stability analysis for roads, construction designers do not yet know a definite location where the critical slip surface may occur in the field. In fact, an analysis of the critical slip surface will be a standard used by designers to determine the required amount of embankment reinforcements if necessary. Thus, the designers assign the landslide location by the trial of a slip surface that is considered to occur in the field. Moreover, there are many methods of calculating the embankment stability as a standard for reinforcement design, but the most commonly used and accurate method is the limit equilibrium. However, the use of this method becomes very "uncertain" because designers need to carry out the determination of slip surface that is considered possible in the field.

To achieve accuracy in the determination of the slip surface in the field, the designer conducts the trial several times. The more trials it takes to get the critical landslide approach, the more accurate the result, but it is time-consuming. Furthermore, as a standard of embankment reinforcement design, the designer often looks for the smallest SF value of a field trial of landslides. However, according to [7], landslides are not necessarily possible in the field at the minimum SF, so the smallest SF value is not necessarily the critical condition in the field. Thus, to reduce the duration of the stability analysis, designers generally conduct only 5-10 trials to obtain 5-10 landslide fields and SF values. However, these values do not necessarily represent the most critical condition in a field that requires the highest amount of reinforcement. A design with a minimum number of trials leads to an incorrect amount of embankment reinforcement requirements. Those inaccuracies in the calculation design are likely to be one of the causes of the occurrence of a landslide in a reinforced embankment. Research on the amount of geotextile reinforcement needs on the embankment that produced the suggested graph has been done by [8], but the number of landslide trials was conducted with a very minimal number. Thus, it is necessary to carry out a further study to obtain the proposed solutions to reduce the problems and difficulties experienced by designers.

The main purpose of this study is to develop an empirical formulation to find the critical surface area of the landslide for reinforcement design which is appropriate under the various conditions of the embankment and soil subgrade in the field. The results obtained are expected to help construction designers to determine the need for the safest reinforcement against landslide disaster.

2. Methods and Data Analysis

This research was conducted with many variations of the data and field trials of landslides. The basic method used for all these calculations in this research is the LEM (Bishop Method). The stages of analysis in this research are as follows:

- Determination of the variation of data in accordance with field conditions.
- Determination of a slope stability analysis with 30 variations; 180 landslide trials were conducted for each variation.
- Calculation of the amount of reinforcement requirements for all results.
- Determination of the landslide area requiring the greatest amount of reinforcement.
- Analysis of the result of the stability calculation of the embankment. The analysis consists of determining the relationship between the SF value and the amount of geotextile needs and the relationship between the SF and the sliding moment, circular centre, and sliding area.
- Determination of the empirical formula for obtaining the critical landslide area under some conditions that may occur in the field.

The result of this study is an empirical formulation that can enable the planner to determine the critical landslide area that can occur in the field.

2.1. Limit equilibrium approach using the Bishop method

Determination of the SF of an embankment is generally performed to analyse the embankment's stability. The SF is defined as the ratio of the average shear strength of the soil (τ_f) to the average shear stress working along the plane of the landslide (τ_d).

$$Sf = \frac{\tau_f}{\tau_d}. \tag{1}$$

The soil shear strength consists of two components, namely cohesion and shear.

$$\tau_f = c + \sigma.tan\emptyset \tag{2}$$

The scope of this study is to evaluate the slope stability using the LEM. The LEM is one method of determining the SF of an infinite slope (natural slope) and a finite slope (embankment). This method does not consider the stress-strain relationship and deformation in the soil. The principles of sliding analysis with this method are as follows:

- A cinematically feasible sliding surface is assumed to define the mechanism of failure.
- The available shear strength along the assumed slip surface is obtained by using the application of static principles. Two static principles that are applied are the assumption of plastic behaviour of the soil mass and the validity of the Mohr-Coulomb failure criterion.
- A comparison of the available shear strength and the shear resistance required to bring the equilibrium into the limiting condition is carried out in terms of the SF.
- The satisfying value of the SF is determined through an iterative process.

There are many published methods that use the limit equilibrium approach [9], each of which has positive and negative impacts. These methods are generally based on the slice method, where the slope is divided into several vertical slices. The formula used in this research is based on the Bishop slice method [10]. The SF can be defined using the following formula:

$$SF = \frac{\sum_{i=1}^{n}[c_i.\Delta x_i + (W_i - u_i.\Delta x_i)tan\emptyset_i]\frac{sec\alpha_i}{1+tan\varphi_i.tan\alpha_i/FOS}}{\sum_{i=1}^{n} W_i sin\alpha_i} \tag{3}$$

2.2. Data analysis

This research was carried out only on a finite slope of road embankment. Analyses were conducted with some variations of soil subgrade data under the embankment and variation of embankment dimensions. The soil subgrade data variations used in this study are based on differences in soil consistency in each layer reviewed. The correlation of consistency values for clay-dominant soils is based on [11], as can be seen in Table 1. The variation of soil subgrade characteristics used in this research is shown in Tables 2 and 3 and Figs. 1(a) and (b). The embankment height varied from 6 to 8 m with a slope ratio of 1:1.5.

Table 1. Soil consistency for cohesive soil (silt and clay).

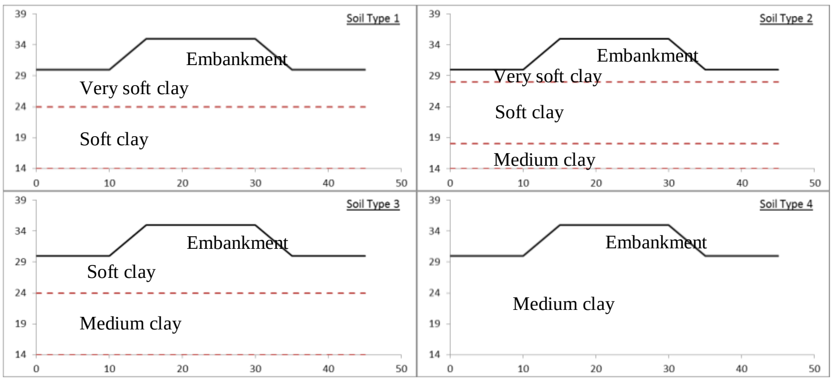
Soil consistency	Undrained cohesion (kPa)	*N-SPT*	q_c from conus (kPa)
Very soft	0-12.5	0-2.5	0-10
Soft	12.5-25	2.5-5	10-20
Medium stiff	25-50	5-10	20-40
Stiff	50-100	10-20	40-75
Very stiff	100-200	20-40	75-100
Hard	>200	>40	

Table 2. Soil subgrade variation with 100%-clay soil subgrade.

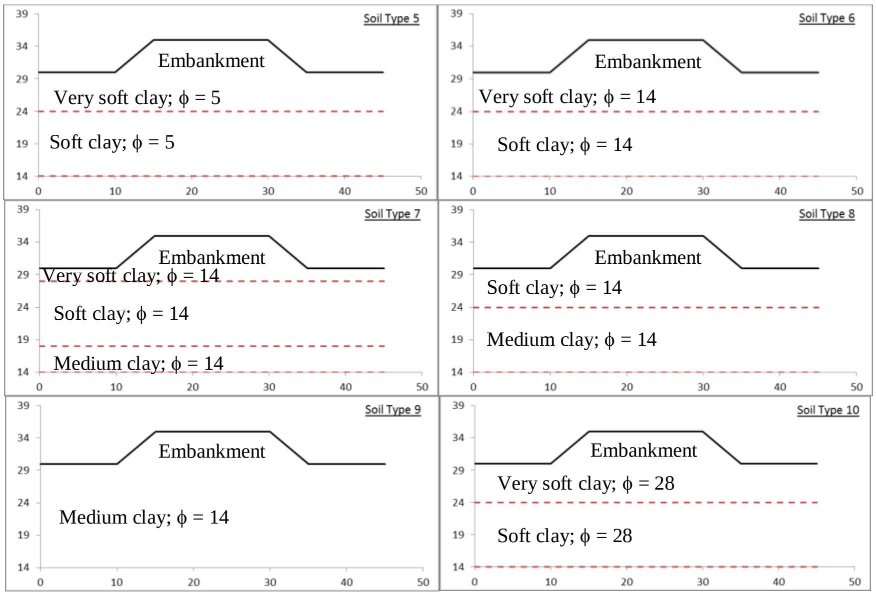
Soil type	Very soft depth (m)	Soft depth (m)	Medium depth (m)
Type 1	6	10	4
Type 2	2	10	8
Type 3	-	6	14
Type 4	-	-	20

Table 3. Soil subgrade variation with sandy clay soil subgrade consistency (ϕ = 5, 14, and 28).

Soil type	Very soft depth (m)	Soft depth (m)	Medium depth (m)
Type 5 (ϕ = 5)	6	10	4
Type 6 (ϕ = 14)	6	10	4
Type 7 (ϕ = 14)	2	10	8
Type 8 (ϕ = 14)	-	6	14
Type 9 (ϕ = 14)	-	-	20
Type 10 (ϕ = 28)	6	10	4



(a) Variation of 100% clay soil subgrade.



(b) Variation of the sandy clay soil subgrade.

Fig. 1. Variation of the soil subgrade characteristic used in this study.

The variation of data used in this analysis is expected to represent soil data in various locations, especially in Indonesia, where the dominant characteristic of soil subgrade is soft clay consistency. The varied data are used to perform the stability analysis of the embankment. The analysis of embankment stability for each variation was done using 180 landslide trial areas. From the trial results obtained, it is expected that the characteristics of the landslide area under various SFs will be seen. Based on the many parameters of the landslide produced, the most critical landslide area that requires the most embankment reinforcement can be seen.

The result of slope stability analysis with 180 trials for each variation will result in different moment resistance values, different circular centres of sliding, different landslide fields, and also different SF values. The difference in the sliding parameters in one of the variations used in this study is shown in Fig. 2. The difference will affect the amount of reinforcement requirements to enable that will be used to enable the embankment to withstand landslide. The reinforcement used in this research is geotextile with an ultimate tensile strength value of 200 kN/m.

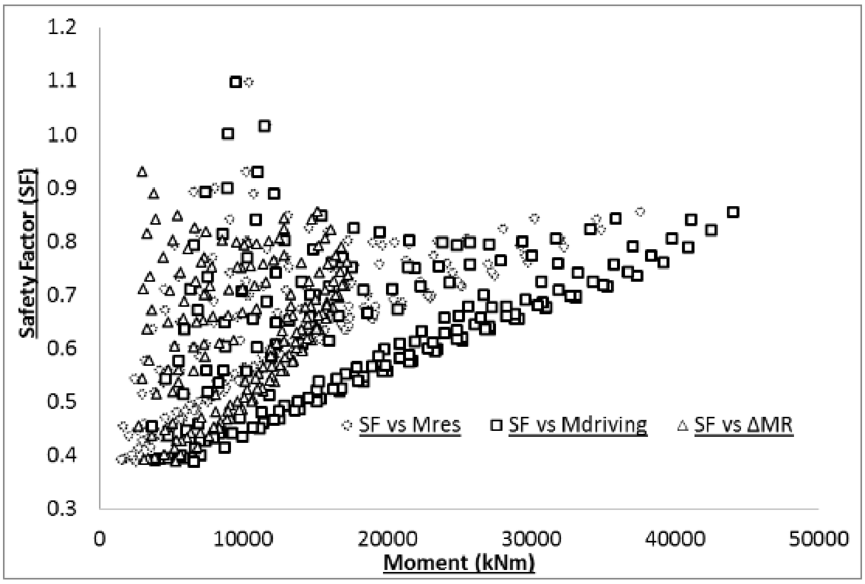


Fig. 2. The difference in sliding parameters in one of the variations used.

2.3. Geotextile design

The allowable stress of the geotextile used for the embankment reinforcement is defined as the ultimate tensile strength divided by the reduction factor. The allowable stress values of the geotextile are in accordance with the following equation:

$$\sigma_{all} = \sigma_c \left(\frac{1}{f_d}.\frac{1}{f_{env}}.\frac{1}{f_m}.\frac{1}{f_c}\right) \quad (4)$$

Based on AASHTO [12], the tensile capacity of the reinforcement determined from constant-load laboratory testing must also be adjusted using reduction factors to account for the site-specific potential load of strength due to chemical and biological degradation (RF_d) and mechanical damage during installation (RF_{ID}). The allowable tensile strength of the reinforcement (T_{allow}) is calculated as:

$$T_{allow} = \frac{T_{ult}}{RF} = \frac{T_{ult}}{RF_D x RF_{ID} x RF_{CR}} \tag{5}$$

All reduction factors must be based on product-specific testing. The values for RF_D and RF_{ID} are less than 1.1. In the absence of such data, AASHTO [12] recommends that RF should not be less than 7 or 3.5 for a permanent and or a temporary wall structure, respectively. The magnitude of the creep reduction factor (RF_{CR}) varies with the design life. Typical values may range from 1.5 to 3.0, with the lowest value corresponding to a short lifetime. The maximum design load for a geosynthetic layer in a permanent reinforced wall application is typically reduced to a long-term allowable design load T_{des}, where:

$$T_{des} = \frac{T_{allow}}{FS} \tag{6}$$

SF is an overall factor of safety to account for uncertainty in the problem geometry, soil variability, and applied loads and has a minimum value of 1.2. For slope reinforcement, $FS = 1$ since the overall factor of safety is accounted for in the stability analyses. A solution for the factor of safety using the Bishop method of analysis was obtained using the following equation:

$$FS = \left(\frac{Mr}{MD}\right)_{unreinforced} + \frac{\sum T_{allow} x R_t}{MD} \tag{7}$$

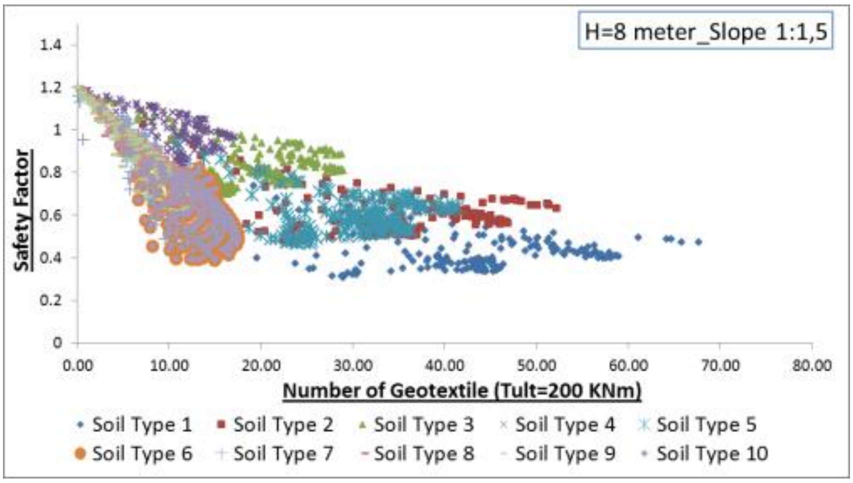
The selection of geotextile for the reinforcement is influenced by two types of factors: internal and external. Internal factors consist of geotextile tensile strength, extension properties (creep), geotextile structure, and resistance to environmental factors. However, not all the available geotextile tensile strength can be utilized in planning and construction reinforcement. This study used a geotextile with an ultimate tensile strength equivalent to 200 kN/m.

3. Result and Discussion

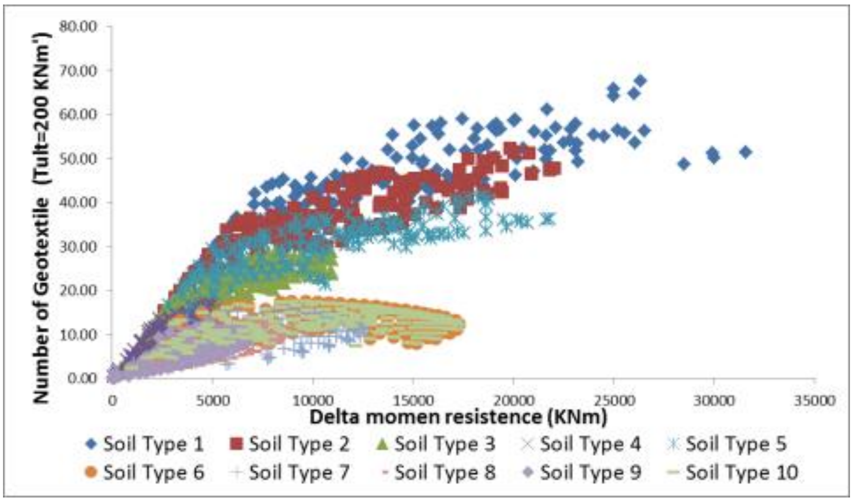
The analysis in this research was done by 180 trials for each variation used. Analysis of embankment stability with the Bishop method (LEM) produced the SF value, moment resistance, circular centre of the landslide, landslide area that occurred. Based on the 180-stability result, the amount of reinforcement required to prevent the landslide was calculated. The landslide area which required the largest amount of reinforcement was then used for analysis in this research to obtain the empirical formulation.

Prior to further analysis, it is necessary to analyse the relationship between the SF value and the amount of reinforcement needs in each variation. Based on the result analysis of three different dimensions of the embankment, the relatively similar results were obtained. The result shows that the minimum SF value does not necessarily generate the most amount of reinforcement. In addition, the largest delta of the moment resistance value (ΔM_{res}) also does not necessarily produce the largest amount of reinforcement requirements. One of the summary results of the SF, ΔM_{res} and amount of geotextile reinforcement needs can be seen in Figs. 3(a) and (b). This pattern of results also occurs for different embankment elevations.

From 180 slope stability analyses for each soil subgrade type with one embankment height, the various frequencies of the range of SF values are shown. The range of SF values that mostly appeared was a landslide area that produced a larger amount of reinforcement compared to the range of SFs with a small number of occurrences. However, the number of occurrences of the range of SF values cannot directly be the standard in determining the critical landslide area of the embankment. The patterns of the result in each variation were not the same. The result of frequency analysis of the number of occurrences of the SF range in some variations analysed can be seen in Figs. 4(a) and (b).

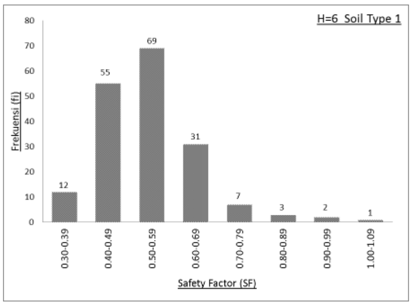


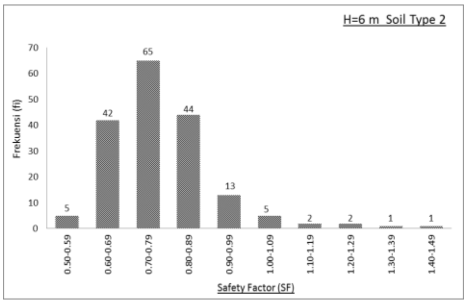
(a) Relationship of safety factor and the number of geotextile.



(b) Relationship of safety factor and delta moment of resistance.

Fig. 3. Summary results of the SF, ΔM_{res} and amount of geotextile reinforcement needs.





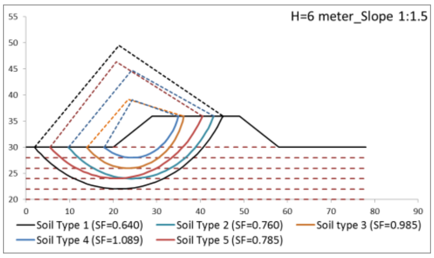
(a) *H*=6 m, soil type 1. **(b) *H*=6m, soil type 2.**

Fig. 4. The frequency of occurrence of SFs.

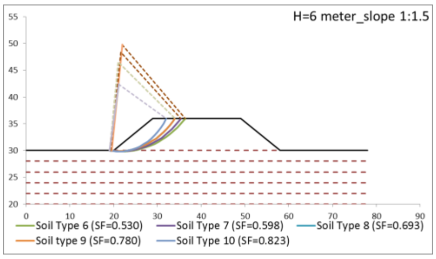
The 180 trials were used to perform analysis of the landslide areas that generate the most amounts of maximum reinforcement needs. Summary of the landslide area and landslide data for each variation used is given in graphical form. The graph of the landslide area that produced the largest amount of reinforcement requirements for the variation can be seen in Figs. 5(a) to (f). These figures show that the softer the soil subgrade, the bigger the landslide that occurs. The depth of the landslide area will be even greater, and the circular centre of the landslide will be further from the ground level elevation with a relatively soft clay soil consistency of the soil subgrade. The depth of the landslide will decrease as the depth of soft soil under the embankment becomes shallower. Otherwise, the soil subgrade that has sandy clay consistency (ϕ = 14-28^o) produces a landslide with internal stability.

Based on the recapitulation result, it can be seen that the maximum landslide depth is 8 m from the ground surface level. The maximum depth of landslide area occurs where the depth of soft soil clay is 6 m and the depth of soft clay is 10 m. The landslide also happens when the soil subgrade under the embankment is medium stiff clay with a *Cu* value close to the *Cu* value of clay soil with a soft consistency. The depth of landslide in this condition is relatively shallow. Summary of the landslide depth for each height variation of embankment used in this research can be seen in Fig. 6. The empirical formula using linear regression used to calculate the landslide depth that requires the greatest amount of reinforcement for the variations used in this research can be seen in Table 4.

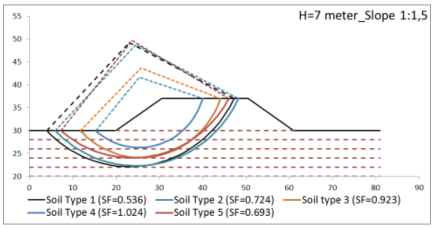
Moreover, the other results that were obtained from this study were the position of the circular centre of the landslide. The location of the circular centre that requires the greatest amount of reinforcement in all variations was always above the slope of the embankment. This condition occurs for both the clay-dominant soil subgrade consistency and the sandy clay soil consistency. The distance of the circular centre of the landslide from the surface of the soil subgrade elevation varies between 8 and 22 m, while the distance between the circular centre and the embankment toe was between 1 and 5 meters. Summary of circular centres of a landslide for the variations used can be seen in Figs. 7(a) and (b). The empirical formulation used to determine the location of the central point of the landslide with the variations used could be seen in Table 5. The formulations listed on the table are obtained using the regression method (linear and polynomial) with *R*-squared value almost the same as 1.



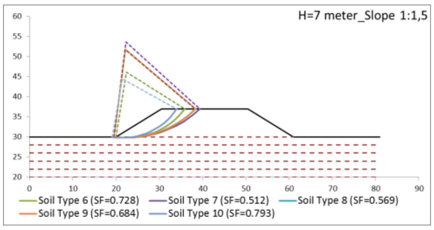
(a) *H*=6 m, slope 1:1.5, clay subgrade.



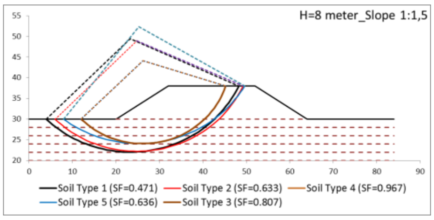
(b) *H*=6 m, slope 1:1.5, sandy clay subgrade.



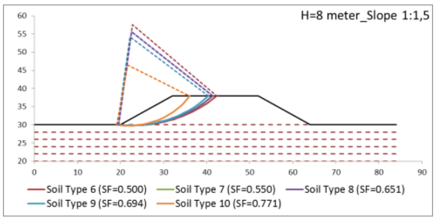
(c) *H*=7 m, slope 1:1.5, clay subgrade.



(d) *H*=7 m, slope 1:1.5, sandy clay subgrade.



(e) *H*=8 m, slope 1:1.5, clay subgrade.



(f) *H*=8 m, slope 1:1.5, sandy clay subgrade.

Fig. 5. The landslide area that results the largest amount of reinforcement requirements.

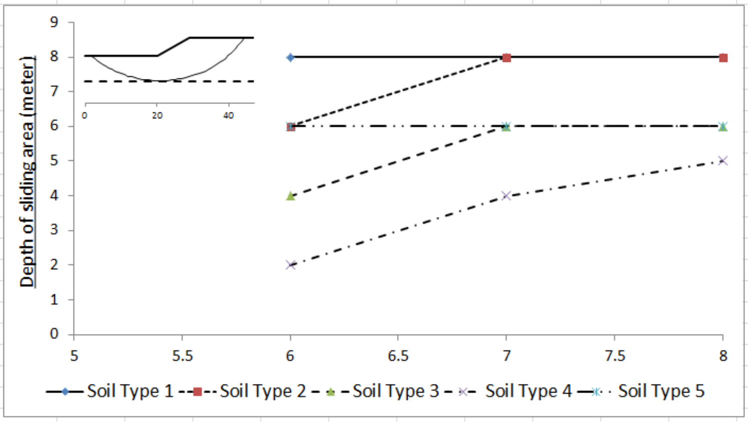
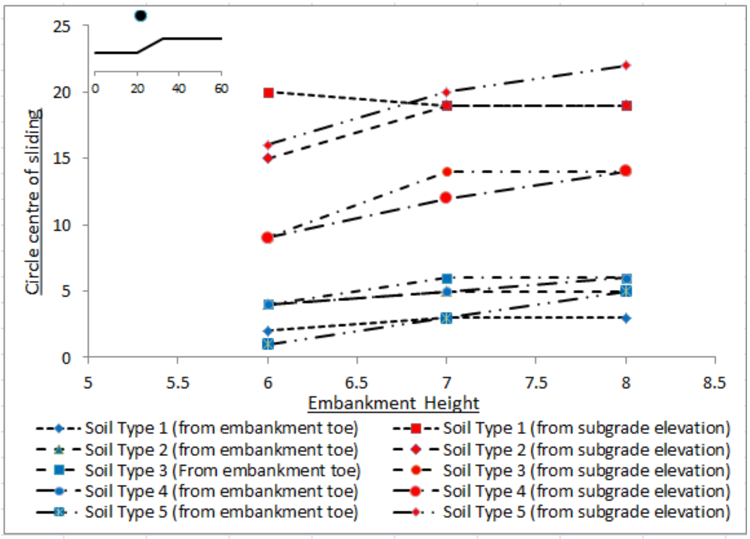


Fig. 6. The depth of sliding area for various embankment dimensions and soil types.

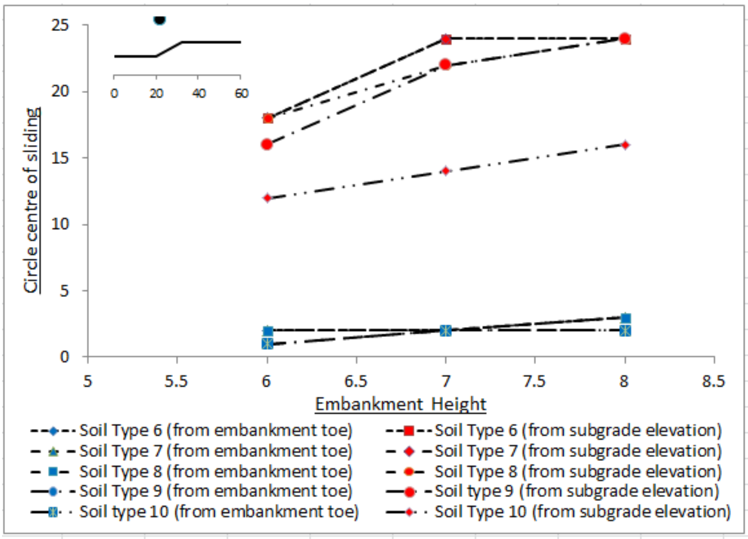
Table 4. Empirical formulation of sliding area depth.

Soil type	Depth of landslide	Soil type	Depth of landslide
Type 1	y=8	**Type 6**	Internal stability
Type 2	$y=-x^2+15x-48$	**Type 7**	Internal stability
Type 3	$y=-x^2+15x-50$	**Type 8**	Internal stability
Type 4	$y=-0.5x^2+8.5x-31$	**Type 9**	Internal stability
Type 5	y=6	**Type 10**	Internal stability

x=height of embankment, y=sliding area depth



(a) Result of clay soil subgrade.



(b) Result of sandy clay soil subgrade.

Fig. 7. The circular centre of the landslide area for various embankment dimensions and soil types.

Table 5. Empirical formulation of circular centre of landslide.

Soil type	Distance from toe-right side	Distance from subgrade elevation
Type 1	y=-0.5x^2 +7.5x-25	y=0.5x^2-7.5x+47
Type 2	y=-0.5x^2+7.5x-23	y=-2x^2+30x-93
Type 3	y=-x^2+15x-50	y=-2.5x^2+37.5x-126
Type 4	y=x-2	y=2.5x-5.833
Type 5	y=2x-11	y=3x-1.67
Type 6	y=-0.5x^2+7.5x-26	y=-3x^2+45x-144
Type 7	y=-0.5x^2+7.5x-26	y=-3x^2+45x-144
Type 8	y=-0.5x^2+7.5x-26	y=-1x^2+17x-48
Type 9	y=0.5x^2-6.5x-23	y=-2x^2+32x-104
Type 10	y=0.5x^2-6.5x-23	y=2x

y=circular centre, x=embankment height

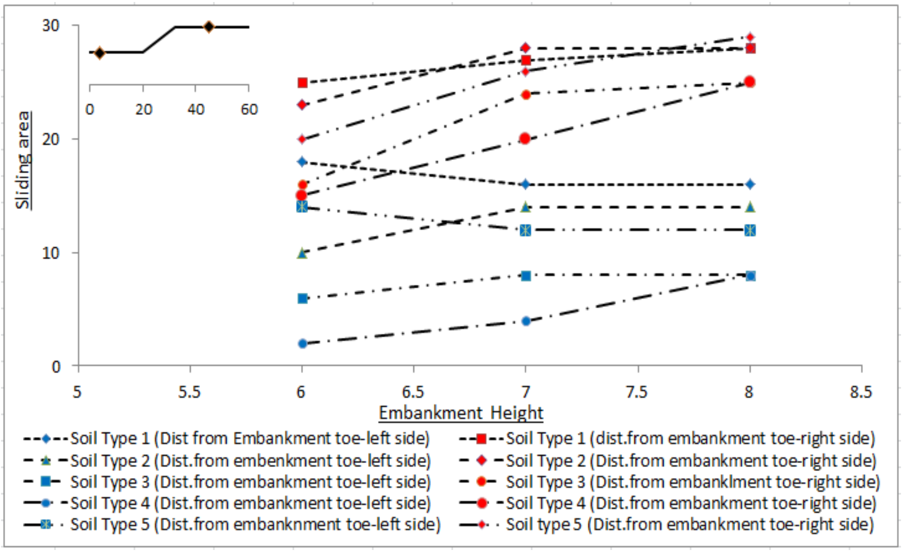
The position of the critical landslide areas has also been obtained in this study. The position of the landslide area was divided into two coordinates of the area, that is, the initial position of the landslide area (calculated as the distance from the embankment toe to the right side of the embankment) and the end of the landslide (calculated as the distance from the embankment toe to the left side of the embankment). This analysis generated varied results on embankments on clay-dominated soil subgrade. However, on the soil subgrade with sandy clay consistency, the landslide field that occurs is likely to have internal stability. Moreover, the end of the landslide was almost at the embankment toe. The recapitulation result of the landslide area obtained for the variations used in this study can be seen in Figs. 8(a) and (b). The empirical formulation using linear and polynomial regression used to obtain the critical landslide area can be seen in Table 6.

There is an interesting result in the empirical formulation that can be shown in the table. It is shown that the landslide area of an embankment on the soil type 6 to type 10 is relatively shallow. Thus, the distance from embankment toe-left side is relatively very small, 1 meter. This causes the empirical formula obtained is relatively simple that is y=1. Other empirical formulation showed excellent results that can also easily be used.

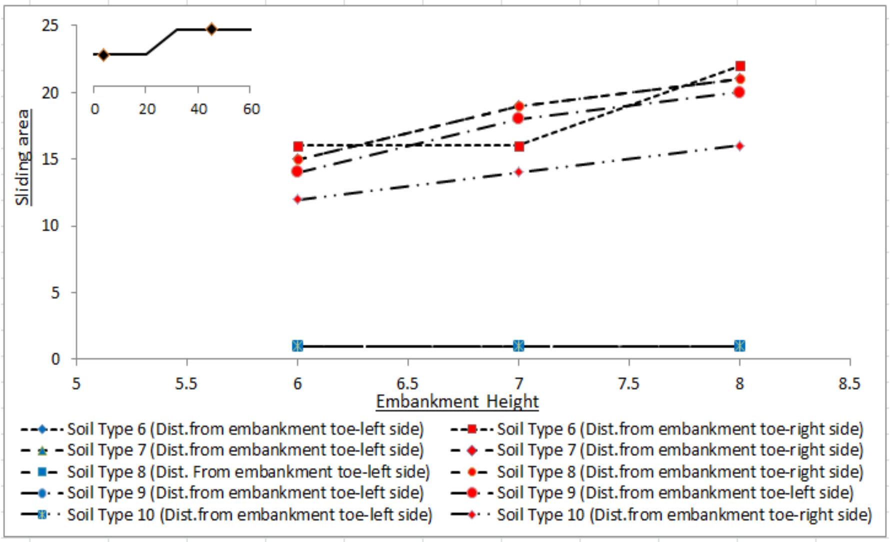
Table 6. Empirical formulation of landslide area.

Soil type	Distance from embankment toe-left side	Distance from embankment toe-right side
Type 1	y=x^2-15x+72	y=-0.5x^2+8.5x-8
Type 2	y=-2x^2+30x-98	y=-2.5x^2+37.5x-112
Type 3	y=-x^2+15x-48	y=-3.5x^2+53.5x-179
Type 4	y=x^2-11x+32	y=5x-15
Type 5	y=x^2-15x+68	y=-1.5x^2+25.5x-79
Type 6	y=1	y=3x^2-39x+142
Type 7	y=1	y=-x^2+17x-51
Type 8	y=1	y=-x^2+17x-51
Type 9	y=1	y=-x^2+17x-52
Type 10	y=1	y=2x

x=height of embankment, y=distance from embankment toe-left side, y=Distance from embankment toe-right side



(a) Result of clay soil subgrade.



(b) Result of sandy clay soil subgrade.

Fig. 8. The landslide area coordinates for various embankment dimensions and soil types.

The empirical formulation produced in this research can be used as a standard in analysing the embankment stability using the LEM either by manual calculation or by using an auxiliary program. This empirical formulation will then also be used to develop an auxiliary program to calculate the stability of the embankment without performing too many landslides. Further analysis is needed to develop an auxiliary program using the empirical formulation from the results of this study.

4. Conclusions

The main conclusions obtained after analysis of the variations considered in this research are as follows:

- The lowest SF value does not necessarily result in the largest number of reinforcement requirements. The largest delta moment resistance value does not necessarily produce the largest amount of reinforcement needs.
- Landslides that produce the largest number of reinforcement needs always occur when the circular centre of the landslide is above the slope of the embankment. This condition occurs in all variations used.
- The depth of the landslide field is affected by the type of soil subgrade and the height of embankment. The depth of the landslide area is relatively deep (approximately as high as the embankment elevation) on a 100%-clay soil subgrade with a soft or very soft consistency.
- The empirical formulation of the results of this study can be used to find the value of the SF and other landslide parameters. This study is still limited to soil types that correspond to the data variations used in this study.

To advance this research, further analysis is needed in field verification and auxiliary programming to facilitate the results of the proposed empirical formulas obtained from this study. This empirical formula can already be used for the stability analysis of an embankment using manual calculation.

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Nomenclature

C	Soil cohesion, kPa
C_i	Cohesion of the soil on the base of the i^{th} slice, kPa
Fc	Secure construction factor
f_d	Reduction factor for mechanical damage
f_{env}	Reduction factor for environmental conditions
f_m	Reduction factor for the extrapolation of geotextile tensile strength data
q_c	Conus resistance of soil subgrade, kPa
H	Embankment height, meter
MD	Driving moments for the unreinforced slope, kNm
MR	Resisting moments for the unreinforced slope, kNm
$N\text{-}SPT$	Standard penetration resistance of soil subgrade
RF_{CR}	Creep reduction factor
RF_d	Chemical and biological degradation
RF_{ID}	Mechanical damage during installation
Rt	Distance between the circular centre and the location of the geotextile layer

T_{allow}	Allowable tensile strength of the reinforcement, kN/m
T_{des}	Allowable design load, kN/m
u_i	Pore water pressure at the base of the i^{th} slice, kN/m^2
W_i	Weight of the i^{th} slice, kN

Greek Symbols

σ_{all}	Allowable stress of the geotextile, kN/m^2
ϕ	Friction angle of soil, degree
ϕi	Friction angle of the soil at the base of the i^{th} slice, degree
τ_f	Shear strength, kN/m^2
τ_d	Shear stress working along the plane of the landslide, kN/m^2
αi	Tangential angle of the base of the i^{th} slice, degree
σc	Ultimate tensile strength according to the age of geotextile construction, kN/m^2
Δxi	Width of the i^{th} slice, m

Abbreviations

AASHTO	American Association of State Highway and Transportation Officials
LEM	Limit Equilibrium Method
SF	Safety Factor
FOS	Factor of Safety

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