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COMPARISON OF SAFETY FACTOR AND GEOSYNTETIC REINFORCEMENT REQUIREMENT FOR SLOPE STABILITY USING 2-D AND 3-D ANALYSIS METHOD

Putu Tantri K.Sari[1]*, Yudhi Lastiasih[1], Nur 'Arfiati Shoffiana[2]

[1] Civil Engineering Department, Institute Technology of Sepuluh Nopember,Surabaya, Indonesia
[2] Magister student in Civil Engineering Department, Institute Technology of Sepuluh Nopember,Surabaya, Indonesia
* tantrigeoteknik@gmail.com

The analysis of landslide slope stability since 1960s is the development of a 2-D structure proposed by various experts, through the 3-D method. Most of these previous studies stated that the ratio of 3-D and 2-D safety factors was more than one for cohesive and less than one for non-cohesive soils. These were because several required slope reinforcements were affected by the safety factors, with the analytical differences of the 2-D and 3-D methods causing a distinction in the requirements. These differences further cause problems by underestimating or overestimating the design. Therefore, this study aims to determine a comparative analysis of 2-D and 3-D slope stability on several required reinforcements. The analyses of the 2-D and 3-D structures were carried out using the LEM proposed by Fellenius and Hovland, respectively. The comparison of the several required reinforcements was also conducted using geotextile with Tult = 200 kN/m. The results showed that the reinforcements required with geotextile between 2-D and 3-D analysis were relatively similar on homogeneous soils. Meanwhile, the geotextile reinforcement needs were different for heterogeneous soils. Under different certain conditions, the need for 2-D reinforcement was greater and lesser than 3-D. In addition, the difference in the reinforcement required for the analysis of these structures was between 1-8 layers of geotextile, depending on soil parameters, slope, and length of the landslide field.

Keywords: Slope stability, 2-D slope, 3-D slope, Limit Equilibrium Method (LEM), safety factor, geotextile reinforcement

1 INTRODUCTION

Landslide is a natural disaster with a relatively high fatality rate. The results study from [1] showed that approximately 80% of the previous studies found that landslides are one of the impacts of climate change. A similar result was also stated by [2] which showed that climate change naturally and artificially affected the stability of the canal. Furthermore, [3] stated that climate change altered the pattern of rain, causing less rain frequency and more intense duration of the phenomenon (rain). These changes altered the infiltration and evapotranspiration pattern of rainwater, which in turn affected the pore pressure in the soil. This is believed to be one of the major causes of high landslide occurrences in several regions and countries [3].

Landslides are one of the 3 major disasters that often occurs in Indonesia between 2012-2014. It also increasingly occurs in other countries, as shown by several previous studies, such as [4] in Singapore, [5 in Indonesia], [6] in South Korea, [7] in European country, [8] etc. With several increasing occurrences and discussions in global scientific forums, this is a very important problem to study, based on determining effective handling efforts.

Slope stability is further analyzed using several balance methods, to determine the safety effectiveness and efficiency against landslides. These include the limit equilibrium, finite element and difference, as well as discrete element methods (LEM, FEM, FDM, and DEM), respectively, which are conducted with two and three-dimensional (2-D and 3-D) approaches. Most embankment stability analyses against landslides are often carried out using the two-dimensional (2-D) approach with the limit equilibrium method. This is conducted by calculating the number of safety factor against landslides, through the assumption of several plane-strain conditions. This assumption is based on field-occurring landslides having infinite lengths, towards eliminating the 3-D effect. These phenomena are not infinite, for the assumption of 2-D calculations to be unsuitable in this condition. According to [9], the 2-D analysis was suitable for slope design, due to producing a conservative estimate for the factor of safety. This was because the final effect for the estimated factor of safety was not included. In addition, landslide analysis using the 3-D method is recommended for performing back analysis [10].

Several studies are being increasingly conducted on the soil and rock slope stabilities, using a 3-dimensional approach that was initially introduced by Anagnosti (1969). This was due to the development of the 2-D stability analysis (N.R., Morgenstern; V.W., 1965). The approach is carried out using the limit equilibrium method (LEM). After this, the 2-D slope stability approach was further developed by several experts, to calculate the 3-dimensional technique. The difference from each of these studies is the assumption of the landslide field in 3 dimensions. This led to several assumptions based on the slip plane being a circular and cylindrical cross-section.

The studies on 3-D landslides have reportedly been carried out by several experts during the 60s till date. The majority that used the 2-D theoretical-based slope stability by Spencer (1967), includes [11], [12] and [13] using ordinary limit equilibrium method; [14] and [15] using computational and earthquake load. Meanwhile, the studies that developed the 2D slope stability theory by Fellenius, (1936), were [16] and [17] for cohesive soil, [18] and [19] with ordinary slice method, and [20] with Concave Slopes in Plain View. In addition, the 2-D slope stability study by Bishop (1955), was also developed by several 3-D analysis experts, namely [19],[21]and [22] before 90's; [23] and [24] after 2000's. Furthermore, Anagnosti [25] and O. Hungr [26] developed a 2-D theoretical base by NR, Morgenstern, & VW (1965). [27] , [28] and [24] also uses the same method in their research; while [22] , [23] and [24] improved the simplified and generalized Janbu methods in their studies. Furthermore, not many researchers have developed a formula to calculate 3D slope stability.

The study on the 3-dimensional slope stability in cohesive soils was initially presented by [16] based on the circular arc method, where the landslide plane was assumed to be a combination of cylindrical centre points with conical ends. Chen and Chameaut [29], also presented a 3-D method to analyze slope stability in homogeneous cohesive soils and frictional slopes. This study considered force and moment equilibrium with different water pore pressure conditions, as results showed that the value of safety was greater than 2-D. In addition, [19] assumed that the landslide field was a combination of cylindrical center points with curve ends, to calculate the safety factor value. The result showed that the 3-D safety value was greater than 2-D at a ratio of 1.03 to 1.30. Another study conducted by [30], stated that the ratio of the 3-D and 2-D safety numbers was 1.07-1.30.

Bahsan and Fakhriyyanti [31], also studied the natural slope and obtained the ratio of 3-D and 2-D safety numbers at an average of 1.44. Using the limit analysis method, another study obtained a ratio of 1.76, 1.15, and 1.04 when L/H = 5, on an undrained uniform, cut, and natural slopes, respectively [32]. Furthermore, [33], conducted a comparative analysis of 3-D and 2-D SF values within open-pit mines, with results showing that the ratio in all analyzed conditions produced a value of more than 1. The SF3D/2D ratio was 1.29 and 1.17 for the steep and gentle slopes, respectively.

Based on these explanations, the majority of the previous studies stated that the ratio of 3-D and 2-D security numbers was more than one under certain conditions. Baligh [17] , further concluded that the SF of 3D was higher and lesser than 2D for cohesive and non-cohesive soils, respectively. Similar results were also obtained by [29] and [34]. The longer and steeper the landslide field length and the slope, the lower the safety ratio between the 3-D and 2-D. Moreover, [34], stated that higher water pore pressure caused the SF ratio of 3-D/2-D in cohesive soils to decrease with increasing cylinder length. Although these results are debated by other experts, they are still used as references in future studies.

Most of the previous results further showed that the values of safety obtained from the 3-D and 2-D methods were different, due to the soil types, landslide area assumptions, and slope dimensions being analyzed. These differences are found to affect the treatment of the embankment when designing the reinforcement requirements [35], [36]. Therefore, this study aims to analyze the effect of the SF differences in conducting the design of reinforcement requirements. The ratio of 3-D and 2-D safety numbers in several previous studies is to be used in obtaining different reinforcement requirements. The study is also a continuation of the research conducted by [35].

Based on the background descriptions above, this study aims to obtain a comparison between the 2-D and 3-D slope stability methods. The safety factors derived are further used to calculate the amount of geotextile reinforcement needed. For planning purposes, the differences in the reinforcement requirements are also analyzed to determine the effectiveness of the 2-D analysis on the 3-D approach. Based on the description above, the main problem to be solved is the effects of the SF differences between 2-D and 3-D on the reinforcements needed for slope stability design. The details of this problem include, (1) The difference in the safety factor between the 2-D and 3-D at similar subgrade conditions, (2) The effect of embankment height on safety factor, (3) The embankment reinforcement differences against landslides in similar soil conditions.

2 MATERIAL AND METHODS

The 2-D slope stability analysis in this study used one of the LEM methods (the ordinary slice) proposed by [37]. The assumption in this method is that only moment balance is considered, as all inter-slice forces are ignored. The plane of failure in the ordinary slice method is a circular arc, as the calculation of the safety factor is shown as follows,

$$F = \frac{\sum_{n=1}^{n=p}(c.\Delta L_n + W_n.\cos\alpha_n.\tan\phi)}{\Sigma W_n.\sin\alpha_n} \quad (1)$$

$$F = \frac{\sum_{n=1}^{n=p}\left(c.\frac{(b_n)}{\cos\alpha_n} + W_n.\cos\alpha_n.\tan\phi\right)}{\Sigma W_n.\sin\alpha_n} \quad (2)$$

When the slope is affected by the groundwater table, the equation becomes,

$$F = \frac{\sum_{n=1}^{n=p}(c.\Delta L_n + [W_n.\cos\alpha_n - U.\Delta L_n]\tan\phi)}{\Sigma W_n.\sin\alpha_n} \quad (3)$$

Where c = soil cohesion, ϕ = angle of friction, b_n = width of n slices, W_n = weight of n slices, α_n = slip plane angle of n slices. The commonly used 2-D analysis has a disadvantage based on the ignorance of the 3-D slope. Furthermore, field landslides often have a limited length, as the assumption of the 3-D slope is more suitable for planning. In recent decades, various experts developed 2-D stability analysis into 3-D, to eliminate the shortcomings of the two-dimensional measurement. The results of these studies were found to vary, with the majority stating that the SF ratio of the analysis was more than one under certain conditions.

The analysis of the 3-D slope stability in this study used the method proposed by [17], which extended the ordinary slice approach [37]. Moreover, the safety factor is defined as the ratio between the total available resistance and mobilization stress along the failure surface and plane, respectively. In simplifying the analysis, the slice method was used. Therefore, the inter-column forces were neglected, as the normal and shear stresses of each column were obtained from the weight.

Similar to the 2-D case, the soil mass above the failure surface was divided into several vertical soil columns in the 3-D analysis. The assumptions used in this division were the horizontal XY plane, as well as the vertical and downward slope Z and Y axes, as shown in Figure 1. The calculation of the 3D safety factor is as follows,

$$F_3 = \frac{\Sigma_x \Sigma_y \left\{\frac{c.\Delta x.\Delta y \sin\theta}{\cos\alpha_{xz}\cos\alpha_{yz}} + \rho z.\Delta x.\Delta y \cos(DIP) tan\phi\right\}}{\Sigma_x \Sigma_y \, \rho z.\Delta x.\Delta y \sin\alpha_{yz}} \tag{4}$$

$$\cos(DIP) = \left(1 + tan^2\alpha_{xz} + tan^2\alpha_{yz}\right)^{-1/2} \tag{5}$$

$$\sin\theta = \left(1 - sin^2\alpha_{xz} . sin^2\alpha_{yz}\right)^{1/2} \tag{6}$$

Where F_3 = 3-D safety factor, c = soil cohesion, ϕ = angle of friction, Δx = column weight, Δy = length of the column, z = height of the column, α_{xz} = angle of slip surface in the x-direction, α_{yz} = angle of slip surface in the y-direction, and ρ = soil density.

The 3-D landslide field used in this study was a central cylinder with an ellipsoid tip, which had lengths of 2lc and ls, respectively. The dimensions used in this analysis are, (1) lc/H = 0.5 and ls/H = 1, (2) lc/H = 0.5 and ls/H = 2, (3) lc/H = 0.5 and ls/H = 4. Where, H is the embankment height, while lc and ls are the dimension parameters, as shown in Figure 2. This illustration shows the front view of the landslide plane on a 3-D slope.

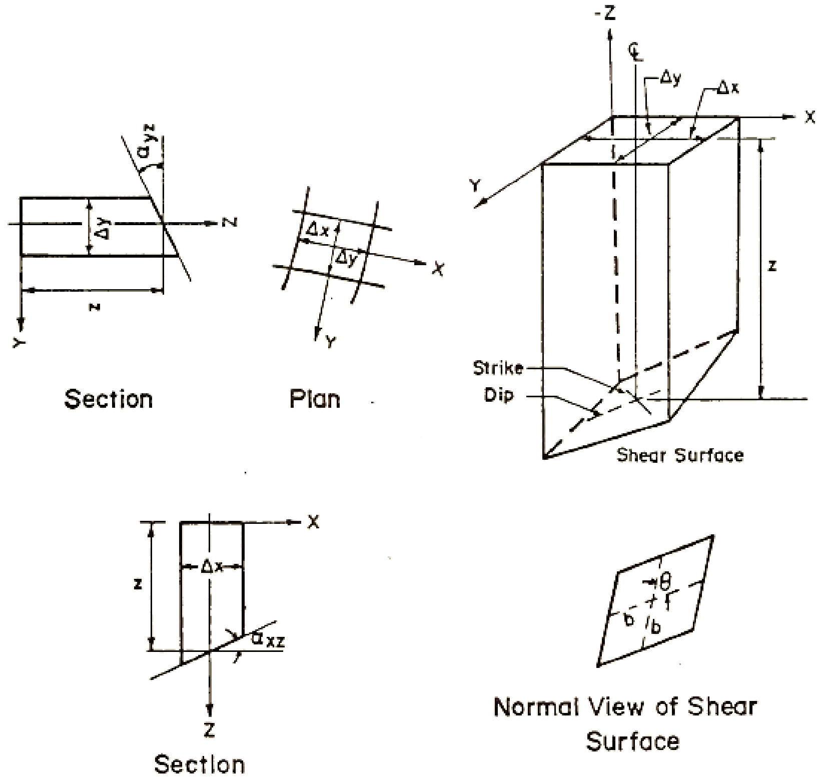

Fig 1. The cross-sectional shape and three-dimensional view of one soil column

Source: [29]

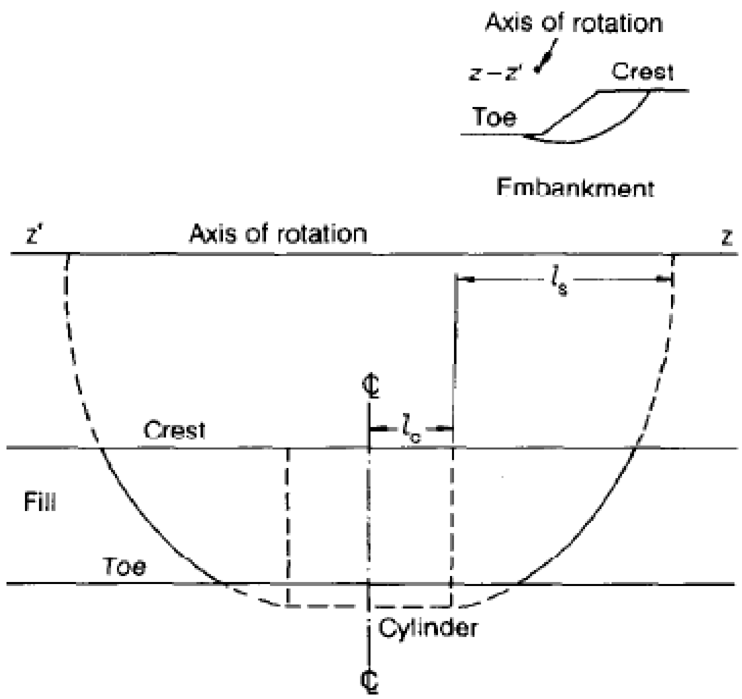

Fig 2. The front view of a 3D landslide

Source: [19]

Based on this study, the soil parameters used were divided into two conditions, namely homogeneous soil and heterogeneous. Several previous studies were used as the basis for the parameters in this study. Parameter A represented non-cohesive soil conditions, with a value of c = 0 Kpa and 40°. Meanwhile, parameters B and C represented the mixture between non-cohesive and cohesive soils, with values at c = 14.4 and 28.7 Kpa, as well as 25° and 15°, respectively. In addition, heterogeneous soils had parameters representing cohesive (Data 1) and non-cohesive (Data 2) conditions, which were obtained at 2 different locations in East Java, Indonesia. The data analysis of the field soil was carried out through the process of correlation, to obtain several parameters. After this, stratigraphy was conducted from the soil data.

Besides the 5 and 3 variations of subgrade parameters and 3-D landslide modeling, respectively, this study also used three different embankment dimensions. The embankment height varied due to the slope, which was adjusted to that of previous studies. The width was also determined at 10.5 m, for the height variations in Dimensions A, B, and C to be 7, 4.2, and 3 m, at a slope of 1:1.5, 1:2.5, and 1:3.5, respectively. Moreover, all dimension and parameter variations of the soil were used in the stability analysis of the embankment, to calculate the geotextile reinforcements needed in this study.

This calculation was only carried out on one type of ultimate geotextile tensile strength at 200 KN/m'. In this study, the calculation of the reinforcement needs should consider the tensile strength of the material, to accept or carry the shear force that occurred during landslides. This calculation is shown as follows,

$$T_{all} = T_{ult}\left(\frac{1}{FS_{ID}.FS_{CR}.FS_{CD}.FS_{BD}}\right) \quad (7)$$

Where, T_{all} = geotextile strength based on specification, T_{ult} = ultimate strength of geotextile, FS_{ID} = safety factor due to installation error, FS_{CR} = safety factor due to creep, FS_{CD} = safety factor due to chemical effect, FS_{BD} = safety factor due to biological effect.

3 RESULT AND DISCUSSION

3.1 2-D stability analysis

The two-dimensional (2-D) slope stability analysis was carried out using the GeoStudio program, where several landslide areas and safety factors were obtained. In this modeling, the centre (X and Y) and radius (R) of the slide, as well as the moment of resistance (Mres) were also obtained. The 2-D slope stability analysis was further conducted for each soil and slope parameter, as the summary of safety factors is shown in Table 1. From the results, the landslide field and safety factor with the highest reinforcement requirements were selected through several experiments.

Table 1. Summary of safety factor using 2-D slope stability analysis

Soil Parameter	Slope	SF 2D
Soil A (c = 0 ; ϕ = 40°)	1:1,5	1,431
	1:2,5	2,256
	1:3,5	3,167
Soil B (c = 14.4 kPa ;ϕ = 25°)	1:1,5	1,873
	1:2,5	3,079
	1:3,5	4,300
Soil C (c = 28,7 kPa ;ϕ = 15°)	1:1,5	2,173
	1:2,5	3,601
	1:3,5	5,046
Data 1	1:1,5	0,976
	1:2,5	1,496
	1:3,5	2,029
Data 2	1:1,5	0,854
	1:2,5	0,907
	1:3,5	1,21

Based on Table 1, soil parameters B/C and A had SF 2-D above (for all slopes) and below (for 1:1.5) 1.5 (SF >/< 1.5), respectively. However, the SF > 1.5 was observed for the slopes of 1:2.5 and 1:3.5. Data 1 also had better soil parameters than Data 2. This was because the SF for Data 2 and 1 had SF < 1.5 for all slopes and only 1:1.5, respectively. Based on Table 3, the steeper the slope or the higher the embankment, the lower the safety factor for all soil parameters.

3.2 3-D stability analysis and ratio of safety factor

The three-dimensional (3-D) slope stability analysis was carried out using the equation proposed by [17]. The landslide fields obtained from the 2-D analysis were also used as the basis for making 3-D structures. After construction using the Autocad program, the columns were divided and derived from the slide in the 2-D field. The dimensions of the 3-D area were further varied for ls/H = 1, 2, and 4. Figure 3 is an example of several slopes and landslide fields for 3-D stability. The results of the safety factor ratio on the variations of soil data and landslide dimensions are shown in Table 2.

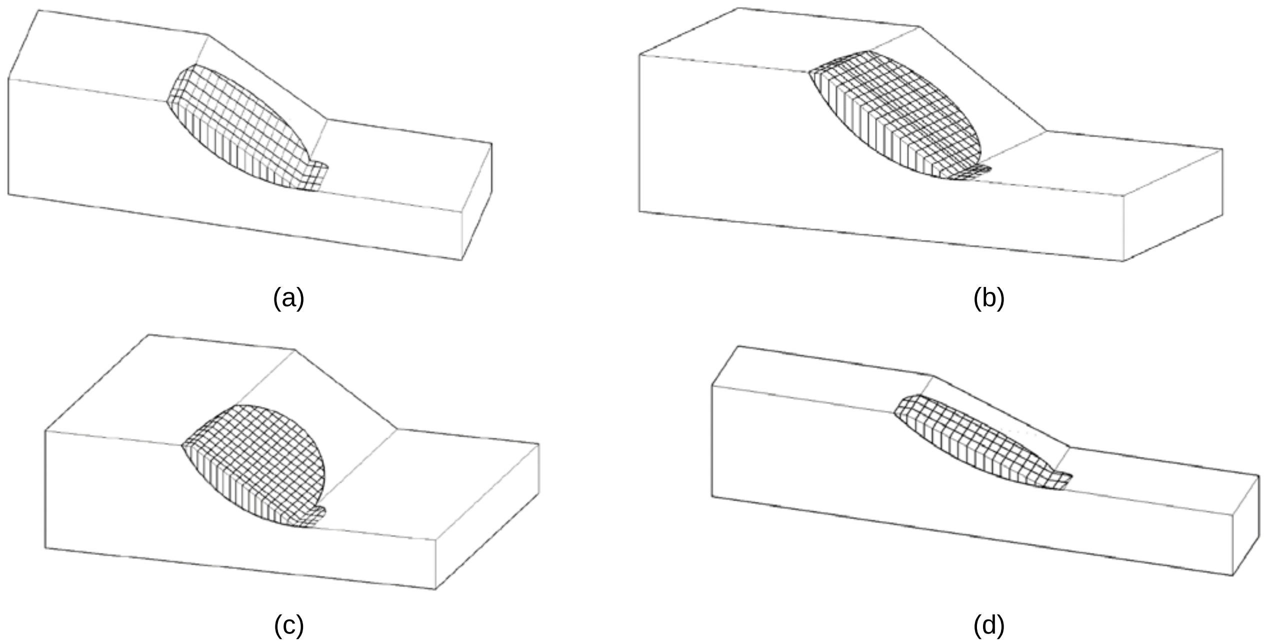

Fig 3. Some examples of 3D landslide fields with data variations used in this study. a. slope 1:1.5 and ls/H = 1; b. slope 1:1.5 and ls/H=2; c. slope 1:1.5 and ls/H=4; d. Slope 1:2.5 and Ls/H=1.

Based on Table 2, the 3-D and 2-D SF ratio of soil parameter A were less than 1 (SF3-D/SF2-D < 1) for all slopes and landslide dimensions. Meanwhile, soil parameters B and C had a 3-D and 2-D SF ratio of more than 1 (SF3-D/SF2-D > 1) for all slopes and landslide dimensions. Soil Data 1 (for all slopes and dimensions) and 2 (for 1:1.5) also had a 3-D and 2-D SF ratio of more than 1. The ratio of 3-D and 2-D SF obtained was within the range of 0.75-1.9, depending on the soil parameters, slope, and landslide dimensions.

Based on Table 2, the soil parameters A and B/C showed that the longer the landslide field (ls/H is greater), the higher and lower the SF ratios of 3-D and 2-D to 1, respectively. In addition, soil data 1 and 2 had 3-D and 2-D SF ratios that decreased with the length of the landslide field. According to the effect of the landslide field, the SF ratios of parameters A, B, and C (c = 0, 14.4, and 28.7 kPa) were less, approximately equal, and more than 1, respectively. This showed that greater cohesion values (c) led to higher SF ratios of 3-D and 2-D, further indicating more security for the three-dimensional factor than the 2-D SF. Furthermore, similar results were still observed based on Table 2, where greater ls/H for parameters A and B/C led to higher and lower 3-D SF, respectively. For soil data 1 and 2, the 3-D safety factor decreased as the landslide area length increased.

Table 2 Comparison of Safety Factors for 2D and 3D Slope Stability Recapitulation

Soil parameter	Slope	SF 2D	SF 3D		
			ls/H=1	ls/H=2	ls/H=4
Soil A (c = 0 ; ϕ = 40°)	1:1,5	1,431	1,305	1,359	1,360
	1:2,5	2,256	1,973	2,085	2,136
	1:3,5	3,167	2,408	2,838	2,956
Soil B (c = 14,4 kPa ; ϕ = 25°)	1:1,5	1,873	1,890	1,878	1,877
	1:2,5	3,079	3,150	3,143	3,107
	1:3,5	4,300	4,694	4,403	4,305
Soil C (c = 28,7 kPa ; ϕ = 15°)	1:1,5	2,173	2,486	2,402	4,550
	1:2,5	3,601	4,510	3,928	3,747
	1:3,5	5,046	6,887	5,691	5,237
Data 1	1:1,5	0,976	1,158	1,093	1,089
	1:2,5	1,496	2,147	1,783	1,767
	1:3,5	2,029	3,815	3,025	2,417
Data 2	1:1,5	0,854	0,853	0,814	0,810
	1:2,5	0,907	1,101	0,973	0,963
	1:3,5	1,21	1,547	1,357	1,295

The differences between the analysis and correction factor of 3-D and 2-D SF were also found to be similar. This correction factor was based on the smallest SF ratio of 3-D and 2-D. According to Table 2, parameter A with a slope of 1:3.5 had the smallest 3-D and 2-D SF ratio at 0.765, which is further used as a reference in determining the correction factor. Therefore, the 2-D SF should be multiplied by a correction factor of 1/0.765 or 1.31, to obtain similar two and three-dimensional SF. Slope stability was also related to the concept of cracked soil, which was developed by [38], [39] , [40], as well as [41]. Moreover, soil conditions on cracked soil were assumed to be drained where the cohesion value was lost (c = 0). Based on this result, the ratio of 3-D and 2D SF was less than 1 for non-cohesive soils (soil parameter A), indicating that the safety factor of 3-D was more critical than 2-D. Therefore, precautions should be performed in conducting slope stability analysis for cracked soil conditions, where the 2-D SF was multiplied by a correction factor of 1.31.

3.3 Comparison with previous studies

The results of the 3-D slope stability analysis were compared with the study conducted by [29]. This was carried out by graphing the relationship between the ratios of SF 3-D/2-D and ls/H at a slope and soil parameters of 1:2.5 and c = 28.7 kPa and 15°, as shown in Figures 4 and 5. Based on Figure 4, the SF ratios of 3-D and 2-D in this study and [29] were similar, where soil parameters A and B/C were less and more than 1, respectively. However, the results of the previous research and this present study had different values. This was caused by the differences in the utilized methods, where Chen & Chameau and this present study used the LEM approaches developed by Spencer and Hovland, respectively.

Another difference was further observed between the previous results and this study, based on soil parameter A (c=0 kPa and = 40^{o}). Although the 3-D and 2-D SF of both studies were less than 1, the results of previous study showed that a longer landslide field led to a lower ratio. This was different from the results in this study, where longer landslide fields increased ratios close to 1. Based on Figure 5, similar trend curves were observed for both studies, where longer landslide areas decreased the SF ratios of 3-D and 2-D. Meanwhile, the ratios of 3-D and 2-D obtained in this study were greater than those in [29]. In addition, a difference was observed at 1:1.5, where the SF results of previous research and this present study were less and more than the ratios of 1:2.5 slopes, respectively.

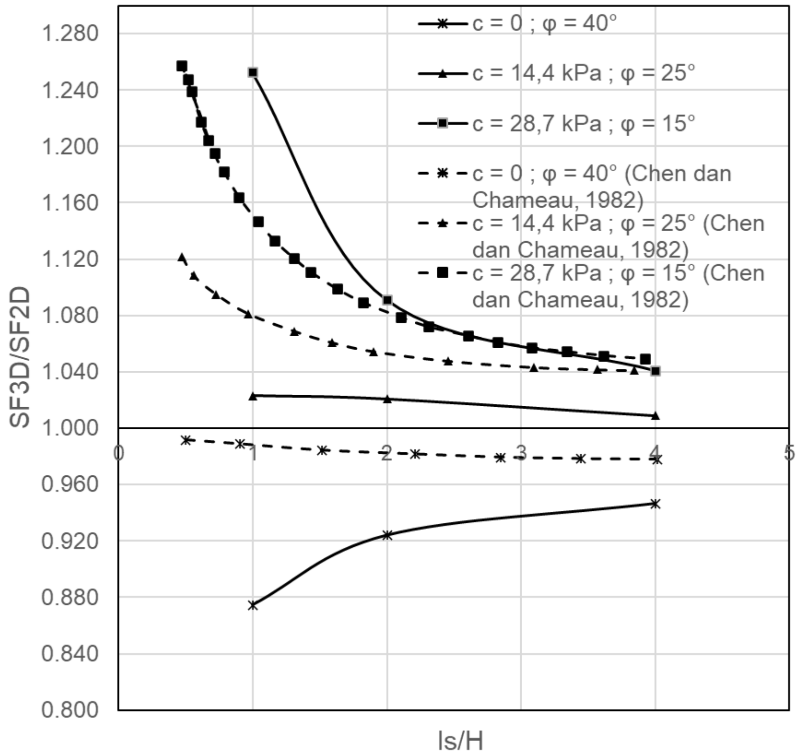

Fig 4. The graphical representation of the comparisons between this study and [13], based on the ratio of SF3-D/2-D and ls/H for lc/H = 0.5 and 1:2.5 slope.

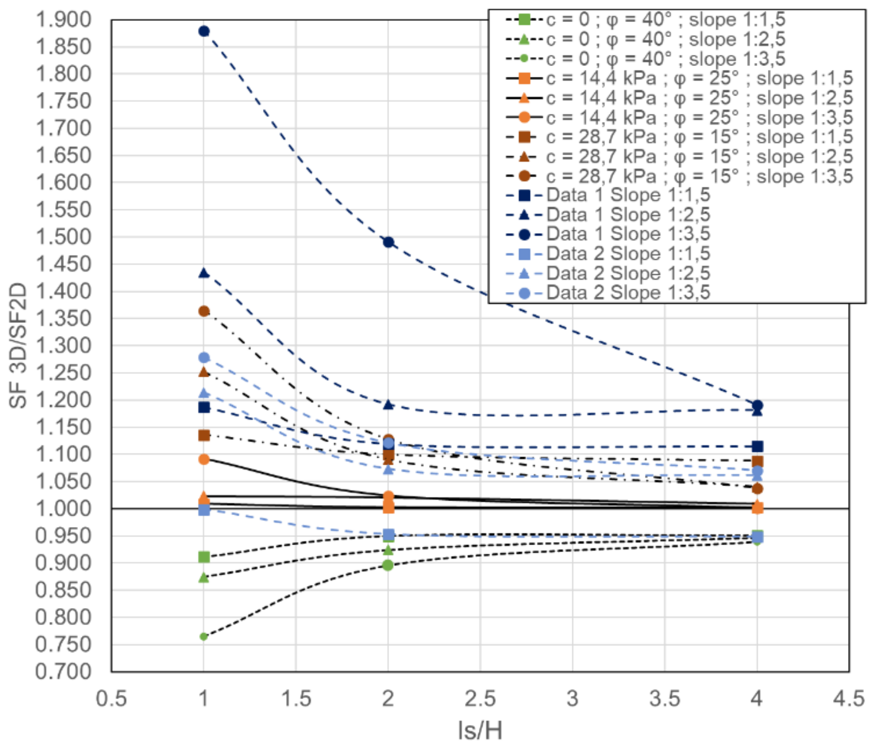

Fig 5. The graphical representation of the comparisons between this study and [13], based on the ratios of SF3-D/2-D and ls/H for lc/H = 0.5, as well as c = 28.7 kPa and 15° for various slopes.

3.4 Geotextile reinforcement calculation

Several factors should be considered in geotextiles designs, including the reinforcements used in planning. Field geotextile reinforcement is often installed on non-cohesive soils such as sandstone, based on the ease of compaction. This indicates the rare installation of geotextile on clay soils. Based on Tables 3-5, the reinforcement required with 2-D and 3-D geotextiles was relatively similar and different for homogeneous and heterogeneous soils (Soil A-C and data 1-2), respectively. In soil data 1 (slope of 1:1.5 and 1:2.5), the amount of reinforcement needed for 3-D geotextiles was less than 2-D. However, the need for the three-dimensional structures was approximately half less than 2-D at 1:1.5. Based on soil data with slopes of 1:1.5 and 1:2.5/1:3.5, the needs for 3-D geotextile reinforcement were more and less (insignificant) than 2-D, respectively. These differences were caused by the alterations in the subgrade conditions and slopes.

Table 3. Recapitulation of the total requirements for 2-D and 3-D geotextile layer reinforcement (ls/H = 1)

Soil parameter	Slope	SF 2-D	Layer 2-D	SF 3-D	Layer 3-D
Soil A	1:1,5	1,431	1	1,305	1
	1:2,5	2,256	-	1.973	-
	1:3,5	3,167	-	2,408	-
Soil B	1:1,5	1,873	-	1,890	-
	1:2,5	3,079	-	3,150	-
	1:3,5	4,300	-	4,694	-
Soil C	1:1,5	2,173	-	2,486	-
	1:2,5	3,601	-	4.510	-
	1:3,5	5,046	-	6.887	-
Data 1	1:1,5	0,976	16	1,159	8
	1:2,5	1,496	1	2.147	-
	1:3,5	2,029	-	3,815	-
Data 2	1:1,5	0,854	21	0,853	21
	1:2,5	0,907	6	1.101	5
	1:3,5	1,21	2	1,547	-

Table 4. Recapitulation of the total requirements for 2-D and 3-D geotextile layer reinforcement (ls/H = 2)

Soil parameter	Slope	SF 2-D	Layer 2-D	SF 3-D	Layer 3-D
Soil A	1:1,5	1,431	1	1,359	1
	1:2,5	2,256	-	2,085	-
	1:3,5	3,167	-	2,838	-
Soil B	1:1,5	1,873	-	1,878	-
	1:2,5	3,079	-	3,143	-
	1:3,5	4,300	-	4,403	-
Soil C	1:1,5	2,173	-	2,402	-
	1:2,5	3,601	-	3,928	-
	1:3,5	5,046	-	5,691	-
Data 1	1:1,5	0,976	16	1,093	10
	1:2,5	1,496	1	1,783	-
	1:3,5	2,029	-	3,025	-
Data 2	1:1,5	0,854	21	0,814	23
	1:2,5	0,907	6	0,973	5
	1:3,5	1,21	2	1,547	-

Table 5. Recapitulation of the total requirements for 2-D and 3-D geotextile layer reinforcement (ls/H = 4)

Soil parameter	Slope	SF 2-D	Layer 2-D	SF 3-D	Layer 3-D
Soil A	1:1,5	1,431	1	1,360	1
	1:2,5	2,256	-	2,136	-
	1:3,5	3,167	-	2,956	-
Soil B	1:1,5	1,873	-	1,877	-
	1:2,5	3,079	-	3,107	-
	1:3,5	4,300	-	4,305	-
Soil C	1:1,5	2,173	-	4,550	-
	1:2,5	3,601	-	3,747	-
	1:3,5	5,046	-	5,237	-
Data 1	1:1,5	0,976	16	1,089	10
	1:2,5	1,496	1	1,767	-
	1:3,5	2,029	-	2,417	-
Data 2	1:1,5	0,854	21	0,810	23
	1:2,5	0,907	6	0,963	5
	1:3,5	1,21	2	1,295	1

4 CONCLUSIONS

Based on the comparative results of the 2-D and 3-D slope stability analysis, the following conclusions were obtained,

1. For similar soil types, the safety factors obtained from the 2-D and 3-D analyses were different. The magnitude of the differences also varied depending on each soil data, with the SF ratio found to be between 0.75–1.9. In homogeneous soils, non-cohesive and cohesive soils had safety factor ratios less and more than 1, respectively. Meanwhile, the ratios of the 3-D and 2-D safety factors were mostly more than 1 in heterogeneous soils. These differences were due to the type of soil and slope.
2. The gentler the slope, the greater the safety factor for both 2-D and 3-D. In homogeneous fields, longer 3-D landslide area (ls/H) led to the decrease and increase of safety factors in cohesive and non-cohesive soils, respectively. This indicated that the longer landslide field caused the closeness of the 3-D safety factor to the 2-D. However, longer landslide fields caused the decrease of 3-D safety factors in heterogeneous soils.
3. The geotextile reinforcement requirements for homogeneous soils were relatively similar between 2-D and 3-D analyses. As for heterogeneous soils, the reinforcement required when analyzed with 3-D was more than the 2-D analysis under certain conditions. However, the reinforcement required when analyzed with 3-D was less than the 2-D analysis in other soil conditions.

Based on the slope stability analysis, the results were uncertain on heterogeneous soils, where the reinforcement requirements of 2-D were more than 3-D in certain conditions. This indicated that the 2-D analysis was overestimated. Meanwhile, the need for 2-D reinforcement was less than 3-D in other conditions, indicating that the analysis was underestimated. Therefore, the need for further analysis is suggested, based on the variations of soil data and embankment heights on heterogeneous soils. It is also useful to determine the overestimation or underestimation levels of 2-D analysis performed on heterogeneous soil parameters.

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