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Research Paper

Variability of N-SPT-Correlated Undrained Shear Strength of Alluvial Deposit in Doplang Region, Central Java, Indonesia

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Abstract

The need to better quantify the variability of soil shear strength and its relations with the factor of safety is increasing in Indonesia. However, this aspect has not yet been studied thoroughly. This paper presents an attempt to quantify the variability of undrained shear strength in relation with the factor of safety of an alluvial deposit in the Doplang region, Central Java, Indonesia. A relationship between the undrained shear strength, s_u, and N-SPT for the deposit was found as s_u = 3.4 N-SPT. The variability of the undrained shear strength was quantified utilizing the coefficient of variance, s/μ (the sample standard deviation, s over the mean, μ) of the N-SPT correlated undrained shear strength. The variability of the undrained shear strength was investigated for the soil near ground surface. The deposit had a value of s/μ ranging from 0.15 to 0.25 near ground surface. The variation of s/μ tended to follow normal and lognormal distributions. Relationships among the coefficient of variance, the probability of failure, and the factor of safety in terms of soil strength for normal and lognormal distributions were developed. For the value of s/μ near ground surface, the relationship between the probability of failure and factor of safety was obtained.

Keywords: *alluvial deposit; factor of safety; probability of failure; standard penetration test; variability of shear strength.*

Introduction

In nature, soil varies in shear strength and compressibility (Lumb [1], Holtz *et al*. [2]). This variability, particularly the shear strength, needs to be incorporated in geotechnical design (Bishop and Henkel [3]). Several attempts have been made to incorporate the variability of soil in geotechnical design (e.g., Casagrande [4], Phoon [5], Ching *et al*. [6], Phoon *et al*. [7]). In line with this, the need to better quantify the variability of soil shear strength is increasing in Indonesia (e.g., in Prakoso [8],[9]).

Despite the need to quantify the variability of soil shear strength in the design process in Indonesia, to the authors' knowledge, this aspect has not been studied thoroughly in Indonesia. Particularly, the quantification of the relationship among the variability of soil shear strength, the probability of failure, and the factor of safety, *FS*, has not been comprehensively investigated for soils in Indonesia.

The standard penetration number (N-SPT value) (ASTM D1586-84 [10], Terzaghi *et al*. [11]) is a popular method in Indonesia to obtain information regarding the undrained shear strength of soil (Building Construction Advisory Committee of Jakarta [12], Indonesian Bridge and Tunnel Road Safety Committee [13], Krisnanto *et al*. [14]).

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Also, N-SPT has been utilized to characterize the site class (e.g., in BSN [15], Irsyam *et al*. [16]). This means that the long experience from engineering practice in Indonesia has formed a database that relates N-SPT with undrained shear strength. Therefore, the N-SPT correlated undrained shear strength can be utilized in the study of the variability of soil undrained shear strength in Indonesia.

Major geotechnical structures have been constructed on alluvial soil deposits (e.g., Wurjanto *et al*. [17], Mahmood *et al*. [18], and Sungkono *et al*. [19]). The understanding of the variability of shear strength is essential in the design of such geotechnical structures. Therefore, the quantification of the variability of shear strength in alluvial deposits is very important.

This paper presents an attempt to quantify the variability of the undrained soil shear strength of an alluvial deposit in the Doplang region, Central Java, Indonesia (Figure 1). Using the variability of the undrained shear strength, the relationships among the variability shear strength, the probability of failure, and the factor of safety in terms of soil undrained shear strength was obtained for the alluvial soil deposit in the studied area.

Figure 1 Location of Doplang region.

Methodology

Deep borings (30 meter in depth) were performed in three locations (BH1, BH2, and BH3 in Figure 2) [20,21]. In each borehole, a standard penetration test (SPT) was performed at a two-meter depth interval. Disturbed and undisturbed samples were obtained from each borehole. Index properties and unconsolidated-undrained (UU) triaxial tests were performed for the disturbed and undisturbed samples, respectively.

The general soil type in the region was analyzed from the regional geological information, visual description of soil samples, and index properties. The relationship between the undrained shear strength and N-SPT was developed for the alluvial deposit in the studied area. The variability of N-SPT correlated undrained shear strength values was quantified utilizing the coefficient of variance, s/μ (the sample standard deviation, s over the mean, μ) of the N-SPT correlated undrained shear strength. Uniform, normal, lognormal, exponential, and gamma distribution functions were considered as possible distribution functions to describe the variation of shear strength at the same depth. A goodness-of-fit test was performed to assess which distribution could appropriately describe the variation of the shear strength data. For the appropriate distribution function, the relation among the s/μ ratio in the distribution function, the probability of failure, and the factor of safety in terms of soil undrained strength were derived. The relationship between the probability of failure and factor of safety was developed for the values of the coefficient of variance of the alluvial soil deposit in the studied area. In this paper the variability of the undrained shear strength near ground surface for a shallow foundation was considered.

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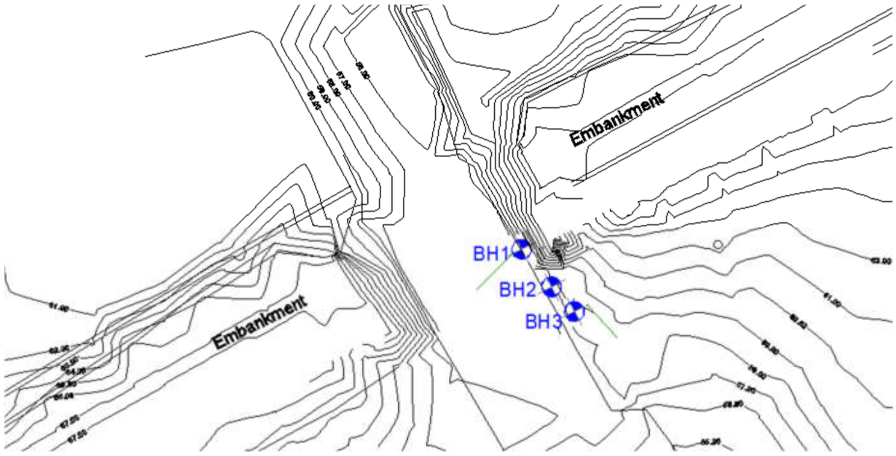


Note:
Universal Transverse Mercator (UTM) coordinate of BH1: 535032, 9206427

Figure 2 Layout of the boreholes.

Results and Discussions

The soil type in the studied area is clay, which extends from the ground surface to a depth of 30 m (end of the borehole). The soil has a brown color from the ground surface to a depth of 8 m and a greyish color from a depth of 8 m to a depth of 30 m. The values of N-SPT range from 4 to 8 at ground surface to a depth of 6 m (Figure 4(a)) and is categorized as medium clay in Terzaghi *et al*. [11]. At a depth of 6 m to 11m, the N-SPT values range from 8 to 13 and the soil is categorized as stiff clay; at a depth of 11 m to 30 m, the N-SPT values range from 17 to 33 and the soil is categorized as very stiff clay.

Geologically, Doplang region consists of a Quatenary alluvial (Qa) deposit of the Wulung river (Figure 3). It was deposited on the flood plains of the Wulung and Bengawan Solo rivers. To the north of the studied area lies an east-west anticline consisting of Tertiary limestone rock. An Upper Miocene Ledok limestone formation forms the axis of the anticline with an overlying younger Early Pliocene Mundu formation and becoming less carbonate. The Tambakromo Formation (QTpt) was deposited in the Late Pliocene to Pleistocene and rests conformably above the Mundu Formation. The anticline gently dips around 5° to 10° toward the south and forms the base for the alluvial deposition. Hiatus occurred from Tambakromo formation with neritic (shallow marine) facies to deposition of fluvial deposits of Wulung and Bengawan Solo rivers with terrestrial facies. In this location, alluvium was deposited further away from the main mountainous sources and therefore has low energy, which leads to less erosion in the Tambakromo Formation, which is positioned around 100 m below the current surface. Datun *et al*. [22] provides some basis for this information.

From the index properties tests, all the soil samples had percent fines higher than 90%. The plasticity index, *PI*, ranged from 40% to 68%. According to the unified soil classification system (USCS), the soil samples were classified as clay with high plasticity (CH). The variation of the N-SPT values with depth is shown in Figure 4(a). There were 14 N-SPT data. Among them, five were complemented with the undrained shear strength. Due to the small number of undrained shear strength values that could be acquired as compared to the number of N-SPT data, a relationship between undrained shear strength and N-SPT was formulated. The relationship between N-SPT and undrained shear strength s_u is shown in Figure 4(b), which has the following relationship:

$$s_u = 3.4 \text{ N-SPT} \quad (1)$$

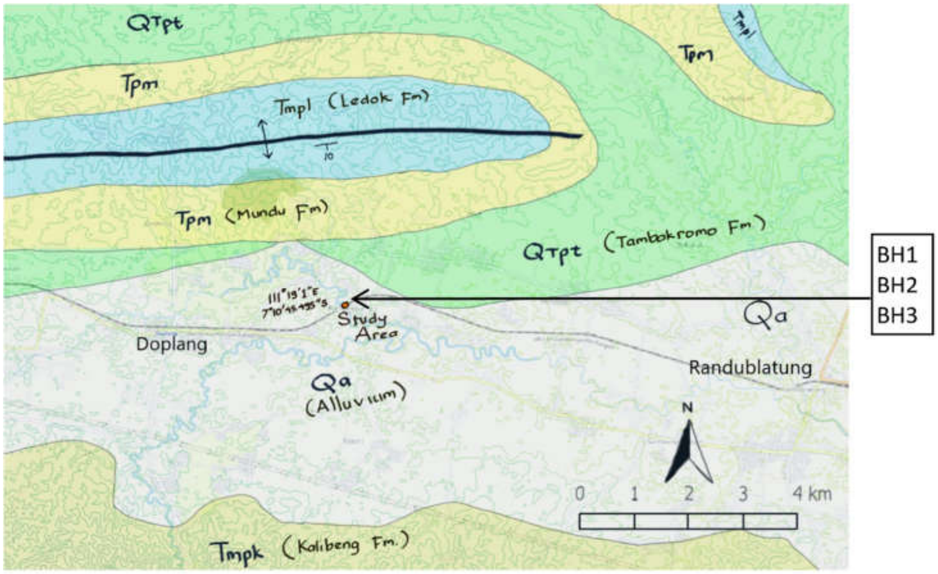
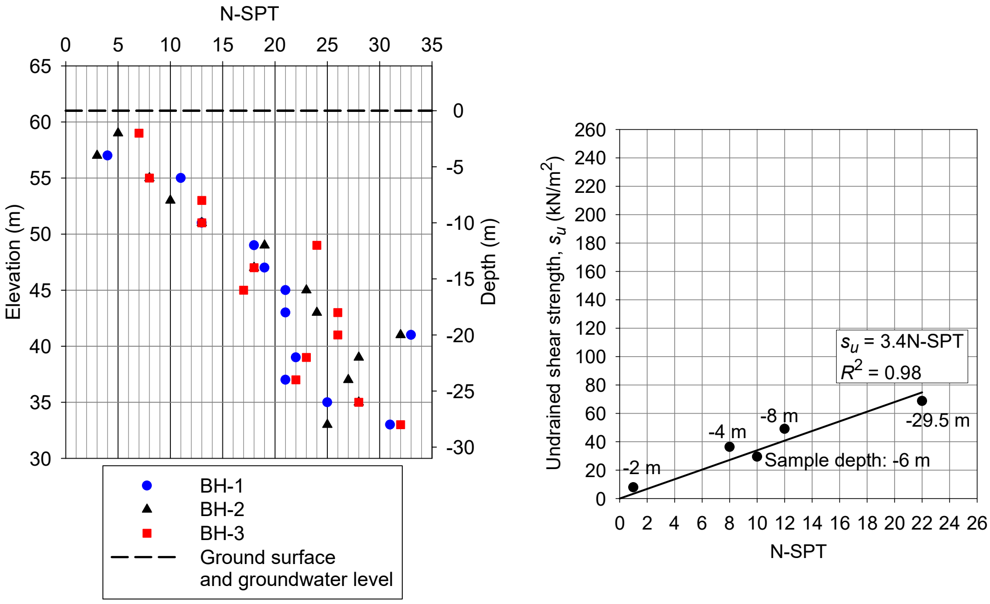


Figure 3 Regional geological condition.



(a) (b)

Figure 4 (a) Variation of N-SPT values with depth, (b) relationship between N-SPT and undrained shear strength, s_u.

The mean and the sample standard deviation of the undrained shear strength, s_u of the samples from the same elevation were then calculated. There were 14 N-SPT data, and a plot of the calculation results is shown in Figure 5(a). The plot in Figure 5(a) describes the variation of the shear strength in the horizontal as well as the vertical direction. The variation in the horizontal direction refers to the variation of the N-SPT correlated undrained shear strength at the same depth among the undrained shear strengths from BH1, BH2, and BH3. The variation in the vertical direction refers to the variation of the N-SPT correlated undrained shear strength at different depths in the borehole. To quantify the relative value between the standard deviation and its corresponding mean value,

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the ratio of s/μ (i.e., the coefficient of variance (Benjamin and Cornell [23], Ang and Tang [24], Montgomery and Runger [25])) was used. A plot of the s/μ ratio is shown in Figure 5(b).

In this paper, the variability of undrained shear strength was considered for the soil near ground surface. This is applicable for shallow foundations. Figure 5(b) shows that from the ground surface to a depth of 15 m, the value of s/μ ranged from 0.15 to 0.25. A value of s/μ equal to 0.20 is considered a representative value in this study to quantify the variability of the undrained shear strength near the ground surface. The values of the ratio of s/μ in Figure 5(b) also indicated that the upper layer has higher variability than the lower layer.

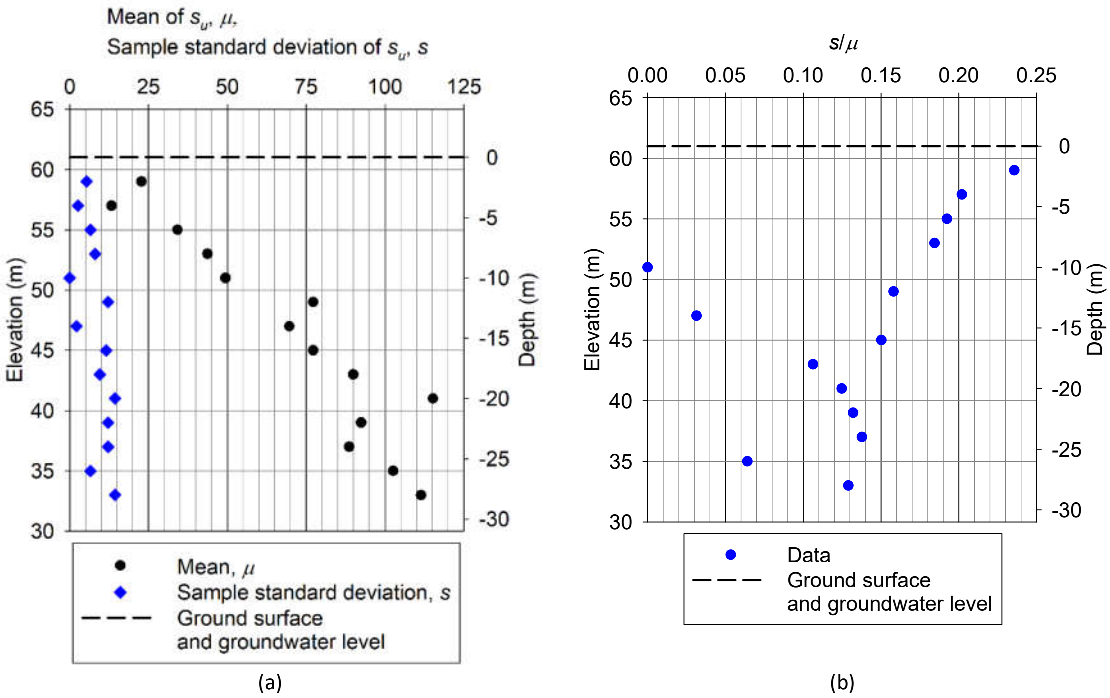

Figure 5 Horizontal variability of N-SPT values with depth: (a) mean and sample standard deviation of N-SPT at the same depth, (b) ratio of sample standard deviation and mean of N-SPT at the same depth, s/μ.

In addition to the mean and the standard deviation, a distribution function is required to describe the distribution of the undrained shear strength values. To select the appropriate distribution, a goodness-of-fit test was performed. Commonly used methods are Chi Squared, Kolmogorov-Smirnov, and Anderson-Darling tests (Benjamin and Cornell [23], Ang and Tang [24]). In the Chi Squared test, the data need to be put into several data interval bins. The Kolmogorov-Smirnov test does not need the data to be put into several data interval bins and therefore the small number of data is not an issue when using this method (Ang and Tang [24]). The Anderson-Darling test is recommended for distributions with a large tail (Ang and Tang [24]). Considering the available data (three undrained shear strength data at each depth) and the condition that the tail was not the focus of this study, the Kolmogorov-Smirnov test was considered the most suitable for this study. The different distribution functions considered in the calculation were uniform, normal, lognormal, exponential, and gamma distribution functions. The performance was quantified using the significance level, α. The significance level is defined as the probability of rejecting the null hypothesis, H_o (Ang and Tang [24]). In this paper, the null hypothesis consists of the value of the N-SPT correlated undrained shear strength, s_u, obtained from the considered distribution function (uniform, normal, lognormal, exponential, and gamma distribution functions). In addition to the goodness-of-fit test, the sum of difference, ΣD_n, was calculated for the data of each depth to compare the performance of each distribution function. The results shown in Table 1 indicate that for a significance of 5%, 10%, and 20%, the normal, lognormal, and exponential distribution functions worked well for 13 of the 14 data. Further comparison using the sum of difference showed that normal and lognormal distribution functions gave the two smallest values of ΣD_n. In this condition, the normal and lognormal distribution functions were considered able to describe the distribution of the undrained shear strength data best among the other distribution functions.

Table 1 Results of goodness-of-fit test using Kolmogorov-Smirnov.

Distribution function	Number of Fit Set of Data			Sum of Difference
	α = 5%	α = 10%	α = 20%	ΣD_n
Uniform distribution	13 of 14	11 of 14	7 of 14	6.10
Normal distribution	13 of 14	13 of 14	13 of 14	1.91
Lognormal distribution	13 of 14	13 of 14	13 of 14	1.81
Exponential distribution	13 of 14	13 of 14	13 of 14	5.14
Gamma distribution	1 of 14	1 of 14	1 of 14	10.23

The relationship between factor of safety versus probability of failure (Meyerhof [26], Bathurst *et al*. [27], Vu *et al*. [28]) is shown in Figure 6. The factor of safety, *FS*, can then be calculated as follows:

$$FS = \frac{\mu_R}{\mu_Q} \quad (2)$$

where μ_R is the mean of the soil resistance and μ_Q is the mean of the load. In this paper, the load was considered to be deterministic. Therefore, the probability of failure in terms of load and the probability of failure in terms of resistance were simplified as shown in Figure 7. The factor of safety in terms of soil resistance can be calculated as follows:

$$FS_{\text{in terms of soil resistance}} = \frac{\mu_R}{x_{P(X \le Q)}} \quad (3)$$

where $X_{P(X \le Q)}$ is the load correspond to the probability of failure, P_f.

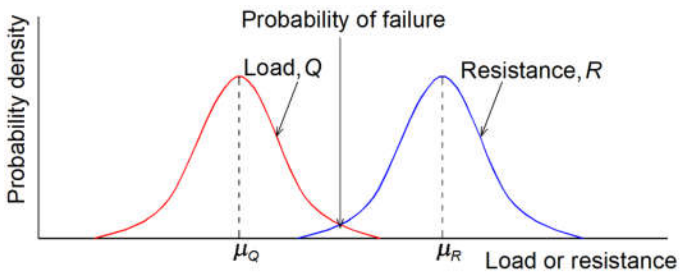

Figure 6 Relationship among distribution, probability of failure, and factor of safety, *FS* (adapted from Meyerhof [26], Bathurst *et al*. [27], Vu *et al*. [28]).

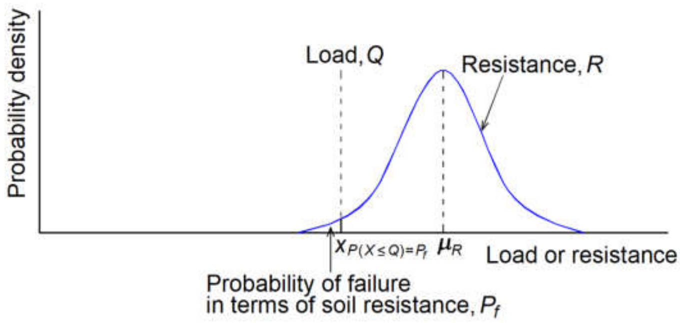

Figure 7 Simplified relationship among distribution, probability of failure, and factor of safety, *FS*, in terms of soil resistance used in this paper.

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Factor of Safety in Terms of Soil Strength for Normally Distributed Data

For normally distributed data of *X*, the relationship among the inverse standard normal distribution function for a corresponding probability of failure, $Z_{P(X\leq Q)}$, the inverse normal distribution function for a corresponding probability of failure, $X_{P(X\leq Q)}$, the mean of the undrained shear strength data, μ, and the standard deviation of the undrained shear strength data, σ is (Benjamin and Cornell [23], Ang and Tang [24], Montgomery and Runger [25]) is as follows:

$$Z_{P(X\leq Q)} = \frac{X_{P(X\leq Q)}-\mu}{\sigma} \tag{4}$$

Using the ratio of σ/μ, Eq. (4) can be written as:

$$Z_{P(X\leq Q)} = \frac{X_{P(X\leq Q)}-\mu}{\left(\frac{\sigma}{\mu}\right)\mu} \tag{5}$$

$$X_{P(X\leq Q)} = \mu\left(Z_{P(X\leq Q)}\left(\frac{\sigma}{\mu}\right)+1\right) \tag{6}$$

Substituting Eq. (6) into Eq. (3) gives:

$$FS_{\text{in terms of soil resistance}} = \frac{\mu}{\mu\left(Z_{P(X\leq Q)}\left(\frac{\sigma}{\mu}\right)+1\right)} \tag{7}$$

Therefore, the factor of safety, *FS*, in terms of soil shear strength for normally distributed data can be calculated as:

$$FS_{\text{in terms of soil resistance}} = \frac{1}{Z_{P(X\leq Q)}\left(\frac{\sigma}{\mu}\right)+1} \tag{8}$$

For the linear relationship between the shear strength, s_u, and the resistance:

$$\mu_R = k\mu_S \tag{9}$$

and

$$\sigma_R = k\sigma_S \tag{10}$$

where μ_R is the mean of the soil resistance, σ_R is the standard deviation of the soil resistance, μ_S is the mean of the shear strength, σ_S is the standard deviation of the shear strength, *k* is a constant. From Eqs. (9) and (10):

$$\left(\frac{\sigma}{\mu}\right)_R = \left(\frac{\sigma}{\mu}\right)_S \tag{11}$$

and the value of the ratio of (σ/μ) of the shear strength can be used in place of that of the resistance (Equations (3) and (8)).

A plot of the factor of safety in terms of soil resistance for normally distributed data calculated from Eq. (8) is shown in Figure 8(a). For the calculation of the factor of safety utilizing Eq. (8), the inverse standard normal distribution function $Z_{P(X\leq Q)}$ was obtained from the table of standard normal distribution functions (Benjamin and Cornell [23], Ang and Tang [24], Montgomery and Runger [25]). The curve of the factor of safety in terms of soil strength versus probability of failure corresponding to the ratio of s/μ of the undrained shear strength equal to 0.20 is shown in Figure 8(b). In the calculation of the coefficient of variance, the standard deviation is calculated from the samples of the undrained shear strength, s_u. Therefore, the symbol *s* is used to represent the sample standard deviation.

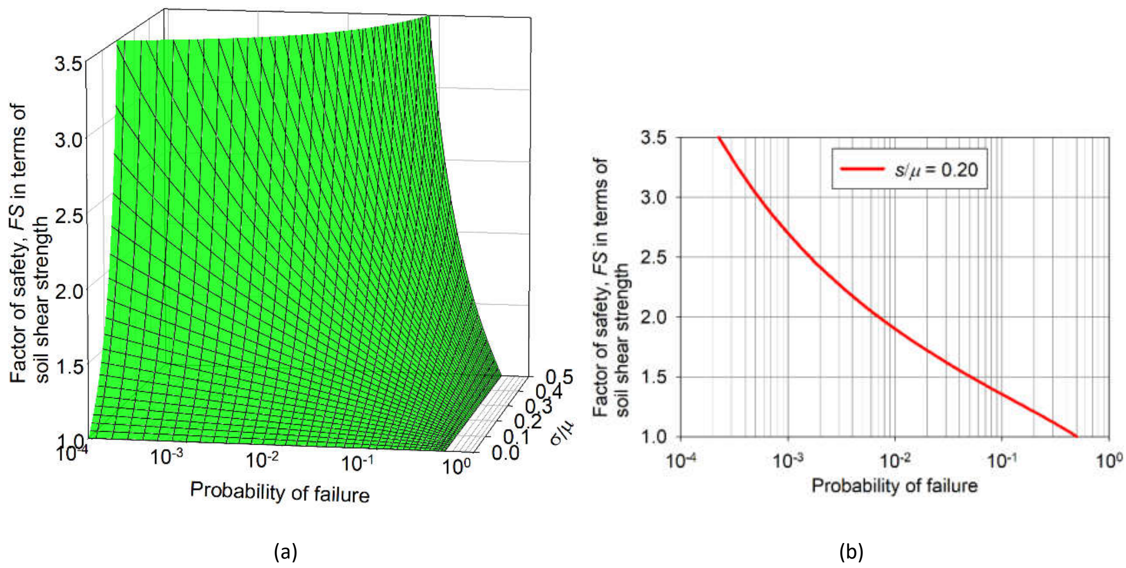

Figure 8 (a) 3D surface of distribution, probability of failure, and factor of safety, *FS* in terms of soil strength for normal distribution, (b) plane of factor of safety, *FS* in terms of soil shear strength vs probability of failure for the ratio of s/μ equal to 0.20.

Factor of Safety in Terms of Soil Strength for Lognormally Distributed Data

For lognormally distributed data of *X*, ln (*X*) is normally distributed with mean θ and standard deviation ω. The relationships among the mean of *X*, μ, the standard deviation of *X*, σ, the mean of ln (*X*), θ, and the standard deviation of ln (*X*), ω are (Montgomery and Runger [25]):

$$\mu = e^{\theta+\frac{\omega^2}{2}} \tag{12}$$

$$\sigma^2 = e^{2\theta+\omega^2}\left(e^{\omega^2}-1\right) \tag{13}$$

Re-written of Eq. (13) gives:

$$ln\,\mu = \theta + \frac{\omega^2}{2} \tag{14}$$

$$\theta = ln\,\mu - \frac{\omega^2}{2} \tag{15}$$

Substituting Eq. (15) into Eq. (13) gives:

$$\sigma^2 = e^{2\,ln\,\mu-\omega^2+\omega^2}\left(e^{\omega^2}-1\right) \tag{16}$$

$$\sigma^2 = \mu^2\left(e^{\omega^2}-1\right) \tag{17}$$

$$\left(\frac{\sigma}{\mu}\right)^2 = e^{\omega^2}-1 \tag{18}$$

$$ln\left(e^{\omega^2}\right) = ln\left(\left(\frac{\sigma}{\mu}\right)^2+1\right) \tag{19}$$

$$\omega^2 = ln\left(\left(\frac{\sigma}{\mu}\right)^2+1\right) \tag{20}$$

$$\omega = \sqrt{ln\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)} \tag{21}$$

Substituting Eq. (20) into Eq. (15) gives:

$$\theta = ln\,\mu - \frac{1}{2}ln\left(\left(\frac{\sigma}{\mu}\right)^2+1\right) \tag{22}$$

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$$\theta = ln\,\mu - ln\sqrt{\left(\left(\frac{\sigma}{\mu}\right)^2 + 1\right)} \tag{23}$$

$$\theta = ln\left(\frac{\mu}{\sqrt{\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)}}\right) \tag{24}$$

For lognormally distribution data of *X*, the relationship among the inverse standard lognormal distribution function for a corresponding probability of failure, $X_{P(X\leq Q)}$, the inverse lognormal distribution function for a corresponding probability of failure, $\ln X_{P(X\leq Q)}$, the mean of the ln of the data, θ, and the standard deviation of the ln of the data, ω is (Benjamin and Cornell [23], Ang and Tang [24], Montgomery and Runger [25]):

$$Z_{P(X\leq Q)} = \frac{ln\,X_{(P\leq Q)} - \theta}{\omega} \tag{25}$$

$$ln\,X_{(P\leq Q)} = Z_{P(X\leq Q)}\omega + \theta \tag{26}$$

$$X_{(P\leq Q)} = exp\left(Z_{P(X\leq Q)}\omega + \theta\right) \tag{27}$$

Substituting Eq. (27) into Eq. (3) gives:

$$FS_{\text{in terms of soil resistance}} = \frac{\mu}{exp\left(Z_{P(X\leq Q)}\omega + \theta\right)} \tag{28}$$

Substituting Eqs. (21) and (24) into Eq. (28) gives:

$$FS_{\text{in terms of soil resistance}} = \frac{\mu}{exp\left(Z_{P(X\leq Q)}\sqrt{ln\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)} + ln\left(\frac{\mu}{\sqrt{\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)}}\right)\right)} \tag{29}$$

$$FS_{\text{in terms of soil resistance}} = \frac{\mu\sqrt{\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)}}{\mu\,exp\left(Z_{P(X\leq Q)}\sqrt{ln\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)}\right)} \tag{30}$$

Therefore, the factor of safety, *FS*, in terms of soil resistance for lognormally distributed data can be calculated as follows:

$$FS_{\text{in terms of soil resistance}} = \frac{\sqrt{\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)}}{exp\left(Z_{P(X\leq Q)}\sqrt{ln\left(\left(\frac{\sigma}{\mu}\right)^2+1\right)}\right)} \tag{31}$$

A plot of factor of safety in terms of soil shear strength for lognormally distributed data calculated from Eq. (31) is shown in Figure 9(a). The curve of factor of safety in terms of soil resistance versus probability of failure for an s/μ ratio of undrained shear strength equal to 0.20 is shown in Figure 9(b).

The relationship between the undrained shear strength and N-SPT can be categorized as a transformation model. A transformation model has model uncertainty (Prakoso [9]). Therefore, the use of the relationship between the undrained shear strength and N-SPT introduces an additional uncertainty to the undrained shear strength values. This additional uncertainty is one of the limitations of this study. It can be reduced by performing more triaxial tests to obtain more undrained shear strength data. The relationship between the undrained shear strength and N-SPT used in this paper served merely as a method to obtain the shear strength. The method presented in this paper focuses on the variability of the undrained shear strength assuming that the undrained shear strength is determined accurately through the respective triaxial tests. This emphasize that more laboratory tests are required in Indonesia.

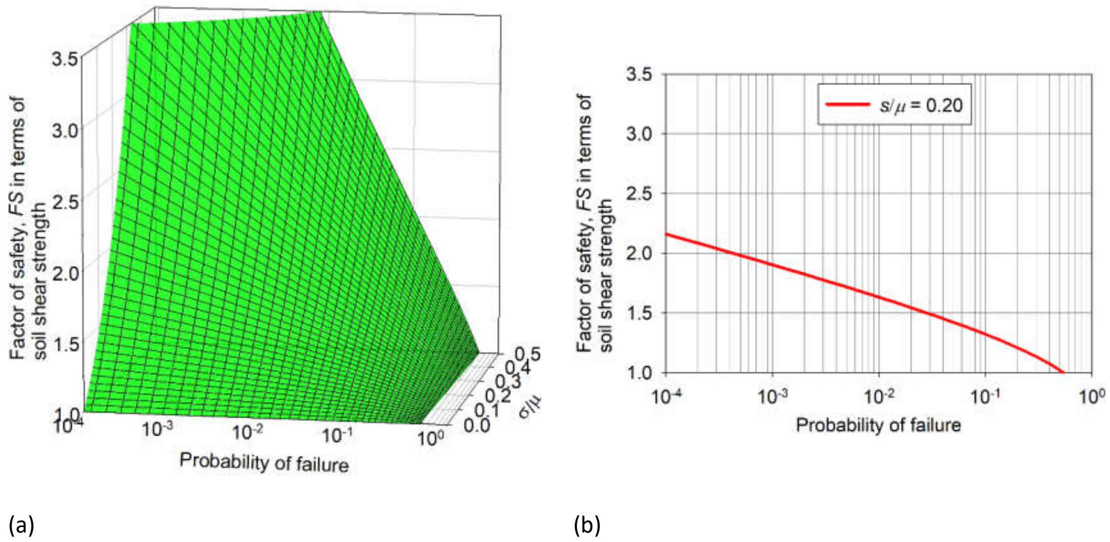

Figure 9 (a) 3D surface of distribution, probability of failure, and factor of safety, *FS* in terms of soil strength for lognormal distribution, (b) plane of factor of safety, *FS*, in terms of soil shear strength vs probability of failure for an s/μ ratio equal to 0.20.

The use of a linear relationship between the shear strength and the soil resistance (Eqs. (9) and (10)) can be explained by considering the bearing capacity of shallow foundations for soils in undrained condition. In undrained condition (the friction angle, ϕ = 0), the formula to calculate the bearing capacity of a shallow foundation, q_u (e.g., Terzaghi [29], Meyerhof [30], Vesic [31]), is reduced to a linear function of undrained cohesion, *c*. The undrained cohesion, *c*, is equal to the undrained shear strength, s_u (Carter and Bentley [32]). Figure 5(b) shows that the ratio of s/μ for soil near ground surface (at depth of 0 to 15 meter) was higher than for a deeper layer. This indicates that the variability in the undrained shear strength for soil near ground surface was higher than that for a deeper layer. This condition underlines the need for a higher factor safety for shallow foundations (≥ 3.0) (Indonesian Bridge and Tunnel Road Safety Committee [13], Das [33]) than the factor of safety for deep foundations (≥ 2.5) (Tomlinson [34]). The failure surface of a shallow foundation extends to a distance relatively close to the ground surface while the failure surface of a deep foundation extends to a distance further from the ground surface.

The variation of the factor of safety versus the probability of failure shown in Figures 8(b) and 9(b) indicates that: (i) this variation depends on the distribution function of the shear strength, and (ii) the lower the probability of failure, the higher factor of safety has to be. Firstly, this emphasizes the need of an adequate number of measurement data to quantify the variability of shear strength in the investigated deposit. Secondly, it shows that the probability of failure is the fundamental requirement in the safety of a geotechnical construction. Any geotechnical construction needs to have adequate safety against failure (Ching *et al*. [6], Indonesian Bridge and Tunnel Road Safety Committee [13], Meyerhof [26], Bathurst *et al*. [27], Vu *et al*. [28], Burland [35]). Therefore, the factor of safety needs to be determined to fulfill a required probability of failure. This emphasizes the need to obtain the relationship between the factor of safety and the probability of failure.

In engineering practice, the relationship among the variability in the undrained shear strength, the probability of failure, and the factor of safety, can be utilized to make a design based on a particular probability of failure for a geotechnical structure. The undrained shear strength can be obtained from soil investigation. The variability in the undrained shear strength can be quantified using the s/μ ratio. Thus, the s/μ ratio for the undrained shear strength of a soil deposit can be calculated using soil investigation results. The probability of failure for a particular geotechnical structure can then be selected. Finally, the factor of safety can be calculated utilizing the s/μ ratio and the selected probability of failure.

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Conclusions

Geologically, the Doplang region consists of a Quatenary alluvial (Qa) deposit of the Wulung river. The relationship between undrained shear strength, s_u, and the N-SPT values for the studied area was s_u = 3.4 N-SPT. At the near ground surface (i.e., from the ground surface to a depth of 15 m), the value of s/μ ranged from 0.15 to 0.25. A value of s/μ equal to 0.20 was considered a representative value in this study to quantify the variability of undrained shear strength near ground surface.

Based on the results of the goodness-of-fit test (Table 1), the lognormal and normal distribution functions described the distribution of the undrained shear strength data best among other distribution functions considered in this study (i.e., uniform, exponential, and gamma distribution functions). Using these two distribution functions, the relationships among the distribution of data, the probability of failure, and the factor of safety in terms of soil resistance were developed. For the value of s/μ near the ground surface, the relationship between the probability of failure and factor of safety was obtained for the considered distribution functions. The variation of the factor of safety versus the probability of failure indicated that: (i) the variation between the factor of safety versus the probability of failure depends on the distribution function, and (ii) the lower the probability of failure, the higher factor of safety has to be.

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