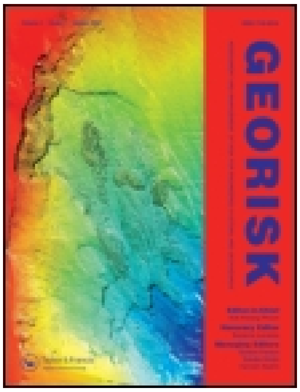

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Determination of partial factors for the verification of the bearing capacity of shallow foundations under open channels

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Determination of partial factors for the verification of the bearing capacity of shallow foundations under open channels

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The limit state design method has been introduced into the design criteria for geotechnical structures. The current paper attempts to apply the reliability-based design method, at Level II, to the bearing capacity of the foundations of open channels from the viewpoint of the limit state design. To examine the applicability of the proposed procedure for practical structures, the reliability index is computed for evaluating the stability of the foundations of existing open channels designed by the conventional method. The conventional design procedure makes excessively safe side design. We applied the first order reliability methods (FORM) to the existing open channels designed by the conventional design procedure, and consequently, large values of reliability index, 3 and 5 were obtained for clayey and sandy soils, respectively. Finally, the partial factors for the soil parameters have been determined, corresponding to the target reliability indices $\beta_t = 1$, 3 and 4.

Keywords: open channel; bearing capacity; limit state design; first order reliability methods (FORM); reliability index; statistical property of soil parameters; foundations; geotechnical reliability; reliability analysis

1. Introduction

The formation of the World Trade Organization (WTO) and the subsequent adoption of the Technical Barriers to Trade (TBT) agreement placed an obligation on the International Organization for Standardization (ISO) to ensure that international standards would be globally relevant. For this purpose, the structural design code for agricultural facilities is required to fulfill international standards such as ISO2394 (Japan Society of Irrigation, Drainage and Reclamation Engineering [JSIDRE] 2008). The use of a performance-based design approach is widely recognised as supporting the development of globally relevant ISO standards.

The Japanese Geotechnical Society (JGS) published the Japanese Geotechnical Standard, i.e. JGS4001-2004, entitled 'Principles for Foundation Designs Grounded on a Performance-based Design Concept' (JGS 2006) to introduce a performance-based design for foundation structures and developed a design code for geotechnical structures. In this standard, the limit state design is introduced into their design criteria.

The current paper attempts to apply the first order reliability method (FORM) to the bearing capacity of spread foundations for open channels, following the performance-based design framework. Reliability analysis can be classified into three different levels, and the approach in this paper is categorised into Level II, which is based on the reliability index β.

Several studies have been conducted for the reliability analysis of the bearing capacity of shallow foundations. For example, Babu *et al.* (2006) carried out a reliability analysis of a foundation resting on cohesive soil based on Prandtl's solution. Larkin (2006) proposed a reliability analysis for foundations subjected to multidirectional seismic loading, by considering the shear strength and the probabilistic distribution of ground acceleration. FORM is applied to the bearing capacity of strip footings under the assumption that the shear strength is variable (Massih *et al.* 2008). Griffiths and Fenton (2001), Griffiths *et al.* (2002) and Fenton and Griffiths (2003) carried out a probabilistic study on the bearing capacity of a rough rigid strip footing on a weightless cohesive soil, to assess the influence of randomly distributed undrained shear strength.

In this paper, a modified Terzaghi's bearing capacity formula is employed as an evaluation method for the bearing capacity. The shear strength

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parameters, undrained shear strength c_u for clay, the effective internal friction angle ϕ' for sand and soil density γ are dealt with as probabilistic parameters. The load due to the self-weight of the concrete channel and the inside water is considered as a deterministic parameter and is assumed to be a static parameter, since the impact of the loading stress is relatively small for the design of open channels. The statistical moments of the soil parameters are determined from the published data records, and the database of the soil test results is organised for the design of agricultural infrastructures in Japan.

The partial factors to satisfy the target reliability indices are then determined for existing open channels. The determined partial factors for c_u, effective internal friction angle ϕ' and γ are averaged for 16 cases. Target reliability indices of $\beta = 2.0$, 3.0 and 4.0 are herein adopted. The determined partial factors are calibrated with the re-calculation of the reliability indices for the 16 channels and verified to be appropriate for the new design code for shallow foundations under open channels.

The presented paper is organised as follows. In the next section, the reliability analysis method for the spread foundations of open channels is briefly reviewed. In Section 3, the reliability index is computed for evaluating the stability of the foundations of 16 existing open channels designed by the conventional design code to examine the applicability of the proposed procedure to practical cases. The paper ends with a summary of the main conclusions.

2. Reliability analysis for open channels

2.1 Formulation of reliability index

Most bearing capacity predictions involve a relationship of the form (Terzaghi 1943)

$$q_u = c \cdot N_c + \frac{1}{2}\gamma_1 \cdot B \cdot N_\gamma + \gamma_2 \cdot D_f \cdot N_q \tag{1}$$

where c, cohesion of the soil below the foundation (kPa); γ_1, unit weight of the soil below the foundation (kN/m^3); γ_2, unit weight of the soil in the embedment portion (kN/m^3); N_c, N_γ and N_q, coefficients of the bearing capacity; D_f, the embedment depth of the foundation (m); B, the length of the foundation's shorter side (m). Equation (1) follows from the Japanese Geotechnical Standard, JGS (2006). The concrete forms of N_c, N_γ and N_q are given as

$$\begin{cases} N_c = (N_q - 1)\ \cot\phi\ (\phi \neq 0) \\ N_c = 5.14 \quad (\phi = 0) \end{cases} \tag{2}$$

$$N_\gamma = (N_q - 1)\ \tan(1.4\phi) \tag{3}$$

$$N_q = \frac{1 + \sin\phi}{1 - \sin\phi}\exp(\pi \cdot \tan\phi) \tag{4}$$

The performance function is defined using the following equations.

$$g_q = q_u(c, \varphi, \gamma_1, \gamma_2) - q_{\max} \tag{5}$$

where, $q_{\max}$, the maximum loading stress due to self-weights of the concrete structure, of the channel and the inside water, which is treated as a deterministic variable. The probabilistic variables for analysis are c, $\varphi = \tan\phi$, γ_1 and γ_2. The definitions of variables are presented in Figure 1. Because the loading stress q is different in each case as shown in Table 4 and greatly variable, the maximum and deterministic value, $q_{\max}$, is employed as an assumption on the safety side design.

The Taylor series expansion of the performance function at design points, e.g. c^*, φ^*, γ_1^* and γ_2^*, is obtained as

$$\begin{aligned} \hat{g}_q = & \left.\frac{\partial g_q}{\partial c}\right|_{c=c^*}(c - c^*) + \left.\frac{\partial g_q}{\partial \varphi}\right|_{\varphi=\varphi^*}(\varphi - \varphi^*) \\ & + \left.\frac{\partial g_q}{\partial \gamma_1}\right|_{\gamma_1=\gamma_1^*}(\gamma_1 - \gamma_1^*) + \left.\frac{\partial g_q}{\partial \gamma_2}\right|_{\gamma_2=\gamma_2^*}(\gamma_2 - \gamma_2^*) \end{aligned} \tag{6}$$

$$\frac{\partial g_q}{\partial c} = N_c \tag{7}$$

$$\frac{\partial g_q}{\partial \varphi} = c\frac{\partial N_c}{\partial \varphi} + \frac{1}{2}\gamma_1 \cdot B \cdot \frac{\partial N_\gamma}{\partial \varphi} + \gamma_2 \cdot D_f \frac{\partial N_q}{\partial \varphi} \tag{8}$$

$$\frac{\partial g_q}{\partial \gamma_1} = \frac{1}{2}B \cdot N_\gamma \tag{9}$$

$$\frac{\partial g_q}{\partial \gamma_2} = D_f \cdot N_q \tag{10}$$

Four probabilistic variables are normalised as defined in the following equation and have a normal

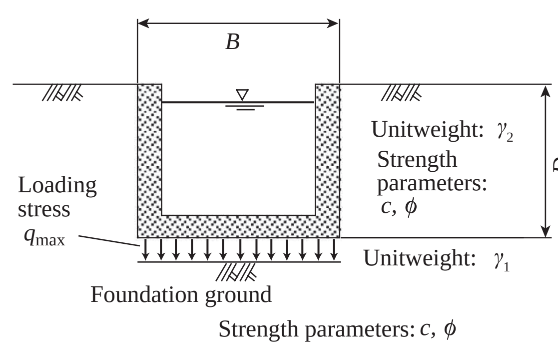

Figure 1. Definition of variables of an open channel in Equation (1).

distribution of N (0, 1) when c, $\varphi = \tan\phi$, γ_1 and γ_2 follow the normal distribution:

$$X_c = \frac{c - \mu_c}{\sigma_c},\ X_\varphi = \frac{\varphi - \mu_\varphi}{\sigma_\varphi},\ X_{\gamma_1} = \frac{\gamma_1 - \mu_{\gamma_1}}{\sigma_{\gamma_1}},$$
$$X_{\gamma_2} = \frac{\gamma_2 - \mu_{\gamma_2}}{\sigma_{\gamma_2}} \tag{11}$$

The soil parameters are found to be normally distributed [according to JGS (1988) and Matsuo (1984)]. When their variability is great, a lognormal distribution may be a better choice. The method to treat the parameters as lognormal variables is described in Section 3.1(2).

The following definitions are also used at design points (c^*, φ^*, γ_1^* and γ_2^*):

$$X_c^* = \frac{c^* - \mu_c}{\sigma_c},\ X_\varphi^* = \frac{\varphi^* - \mu_\varphi}{\sigma_\varphi},$$
$$X_{\gamma_1}^* = \frac{\gamma_1^* - \mu_{\gamma_1}}{\sigma_{\gamma_1}},\quad X_{\gamma_2}^* = \frac{\gamma_2^* - \mu_{\gamma_2}}{\sigma_{\gamma_2}} \tag{12}$$

where μ_c, μ_φ, $\mu_{\gamma1}$ and $\mu_{\gamma2}$ are the mean values for c, $\tan\phi$, γ_1 and γ_2; σ_c, σ_φ, $\sigma_{\gamma1}$ and $\sigma_{\gamma2}$ are the standard deviations for c, $\tan\phi$, γ_1 and γ_2.

The derivative of the performance function should satisfy the following equation from Equation (11):

$$\frac{\partial g_q}{\partial c} = \frac{1}{\sigma_c}\frac{\partial g_q}{\partial X_c},\ \frac{\partial g_q}{\partial \varphi} = \frac{1}{\sigma_\varphi}\frac{\partial g_q}{\partial X_\varphi},$$
$$\frac{\partial g_q}{\partial \gamma_1} = \frac{1}{\sigma_{\gamma_1}}\frac{\partial g_q}{\partial X_{\gamma_1}},\ \frac{\partial g_q}{\partial \gamma_2} = \frac{1}{\sigma_{\gamma_2}}\frac{\partial g_q}{\partial X_{\gamma_2}} \tag{13}$$

The expected value of the performance function is approximated with Equation (14) derived from Equation (6) as

$$\begin{aligned}\mu_g &= \left.\frac{\partial g_q}{\partial c}\right|_{c=c^*}(\mu_c - c^*) + \left.\frac{\partial g_q}{\partial \varphi}\right|_{\varphi=\varphi^*}(\mu_\varphi - \varphi^*)\\ &\quad + \left.\frac{\partial g_q}{\partial \gamma_1}\right|_{\gamma_1=\gamma_1^*}(\mu_{\gamma_1} - \gamma_1^*) + \left.\frac{\partial g_q}{\partial \gamma_2}\right|_{\gamma_2=\gamma_2^*}(\mu_{\gamma_2} - \gamma_2^*)\\ &= -\left.\frac{\partial g_q}{\partial X_c}\right|_{c=c^*}X_c^* - \left.\frac{\partial g_q}{\partial X_\varphi}\right|_{\varphi=\varphi^*}X_\varphi^*\\ &\quad - \left.\frac{\partial g_q}{\partial X_{\gamma_1}}\right|_{\gamma_1=\gamma_1^*}X_{\gamma_1}^* - \left.\frac{\partial g_q}{\partial X_{\gamma_2}}\right|_{\gamma_2=\gamma_2^*}X_{\gamma_2}^*\end{aligned} \tag{14}$$

The standard deviation of the performance function is written in Equation (15) as

$$\begin{aligned}\sigma_g &= \left\{\left(\left.\frac{\partial g_q}{\partial c}\right|_{c=c^*}\right)^2\sigma_c^2 + \left(\left.\frac{\partial g_q}{\partial \varphi}\right|_{\varphi=\varphi^*}\right)^2\sigma_\varphi^2\right.\\ &\quad \left. + \left(\left.\frac{\partial g_q}{\partial \gamma_1}\right|_{\gamma_1=\gamma_1^*}\right)^2\sigma_{\gamma_1}^2 + \left(\left.\frac{\partial g_q}{\partial \gamma_2}\right|_{\gamma_2=\gamma_2^*}\right)^2\sigma_{\gamma_2}^2\right\}^{1/2}\\ &= \left\{\left(\left.\frac{\partial g_q}{\partial X_c}\right|_{c=c^*}\right)^2 + \left(\left.\frac{\partial g_q}{\partial X_\varphi}\right|_{\varphi=\varphi^*}\right)^2\right.\\ &\quad \left. + \left(\left.\frac{\partial g_q}{\partial X_{\gamma_1}}\right|_{\gamma_1=\gamma_1^*}\right)^2 + \left(\left.\frac{\partial g_q}{\partial X_{\gamma_2}}\right|_{\gamma_2=\gamma_2^*}\right)^2\right\}^{1/2}\end{aligned} \tag{15}$$

The reliability index is computed using the mean value and the standard deviation from Equation (16).

$$\begin{aligned}\beta = \frac{\mu_g}{\sigma_g} &= -\frac{1}{\sigma_g}\left(\left.\frac{\partial g_q}{\partial X_c}\right|_{c=c^*}X_c^* + \left.\frac{\partial g_q}{\partial X_\varphi}\right|_{\varphi=\varphi^*}X_\varphi^*\right.\\ &\quad \left. + \left.\frac{\partial g_q}{\partial X_{\gamma_1}}\right|_{\gamma_1=\gamma_1^*}X_{\gamma_1}^* + \left.\frac{\partial g_q}{\partial X_{\gamma_2}}\right|_{\gamma_2=\gamma_2^*}X_{\gamma_2}^*\right)\\ &= -\alpha_c X_c^* - \alpha_\varphi X_\varphi^* - \alpha_{\gamma_1} X_{\gamma_1}^* - \alpha_{\gamma_2} X_{\gamma_2}^*\end{aligned} \tag{16}$$

where α_c, α_φ, $\alpha_{\gamma1}$ and $\alpha_{\gamma2}$ are called the sensitivity, which expresses the effect of each parameters on the performance function, g_q, and are defined as follows:

$$\alpha_c = \frac{1}{\sigma_g}\left(\left.\frac{\partial g_q}{\partial X_c}\right|_{c=c^*}\right),\ \alpha_\varphi = \frac{1}{\sigma_g}\left(\left.\frac{\partial g_q}{\partial X_\varphi}\right|_{\varphi=\varphi^*}\right),$$
$$\alpha_{\gamma_1} = \frac{1}{\sigma_g}\left(\left.\frac{\partial g_q}{\partial X_{\gamma_1}}\right|_{\gamma_1=\gamma_1^*}\right),\ \alpha_{\gamma_2} = \frac{1}{\sigma_g}\left(\left.\frac{\partial g_q}{\partial X_{\gamma_2}}\right|_{\gamma_2=\gamma_2^*}\right) \tag{17}$$

2.2 Statistics of parameters

The statistical values of soil parameters, e.g. c, $\tan\phi$, γ_1 and γ_2 are obtained by collecting data from the literature, for example, Matsuo (1984) and JGS (1988). The statistical values of the internal friction angles $\tan\phi$ result from direct shear tests for the sandy material, whereas the statistical values for the undrained shear strength c_u were determined based on unconfined compression tests on saturated clayey soils. The unit weights for the sandy and clayey material are given as the statistical values derived from the wide range of soil conditions. In the design calculation, the coefficients of variation are used as generic values, while the mean values must be evaluated for each site. Table 1 lists the mean, the standard deviation and the coefficient of variation obtained through a statistical analysis of the data.

The statistical properties of c_u and $\tan\phi$ at the sites of the open channels were also investigated, and the results shown in Tables 2 and 3. Table 2 lists the statistical properties of the undrained shear strength of three sites in Japan. The subsoils of Ryoso and Ohigawa are categorised as alluvial clay, and the soil of Kasumigaura as Kanto-loam clay. The clay samples from these sites were saturated. The test results are derived from unconsolidated and undrained (UU) triaxial compression tests on undisturbed clay samples. Figure 2 (a)–(c) shows the spatial distributions of the undrained shear strength

Table 1. Statistical values of the soil parameters.

Parameter	Mean	Standard deviation	Coefficient of variation
c_u (kPa)	25.0	7.35	0.302
tan ϕ'	0.65	0.10	0.153
γ_1, γ_2 (kN/m^3)	16.9	0.98	0.055

Table 2. Statistical values of undrained shear strength from UU triaxial compression tests.

Sampling site	Trend (kPa)	Coefficient of variation
Ryoso	15.3+2.28z	0.327
Kasumigaura	11.4+8.01z	0.475
Ohigawa	11.0+7.72z	0.426

Note: z, Depth (m).

in the three sites. The trends and the coefficients of variation, which are assumed to be constant with the depth, are presented in Figure 2 and Table 2. Beside the mean values, the standard deviation is indicated by the σ-limits. The values of the coefficient of variation, for c_u are in the range 0.3–0.5, and larger than the published values above mentioned.

Table 3 shows the statistical values of the internal friction angles derived from the results of consolidated and drained (CD) triaxial compression tests. The samples of the sandy material from two sites were tested. The sampled sand of Ryoso is alluvial sand, and Nogata sand belongs to the volcanic ash sand. The mean values are almost the same with about 35° at the two sites, and the coefficients of variation at the sites, Ryoso and Nogata, are 0.10 and 0.17, respectively, which are very similar to the published values (e.g. Matsuo 1984)

The following coefficients of variation, V for different variables are adopted for subsequent analyses from Table 1.

Unit weight, γ_1 and γ_2: $V = 0.06$ ($\cong 0.055$)
Coefficient of friction, tanϕ': $V = 0.15$ ($\cong 0.153$)
Undrained shear strength, c_u: $V = 0.30$ ($\cong 0.302$)

In Table 1, the data were collected from the wide range of soil conditions, whereas the statistical values of the specific sites are treated as shown in Tables 2 and 3. Although, in the actual design calculations, the statistical values of the objective sites are required, the amount of data is usually not enough. The coefficient of variations in Table 1 can be conveniently used as the generic values in such cases; however, the values might be overestimated, since the statistical values come from a wide range of data. Therefore, the coefficient of variations in Table 1 and those in Tables 2 and 3 has been compared to check

Table 3. Statistical values of internal friction angle from CD triaxial compression test.

Sampling sites	Mean of tan ϕ' (ϕ'°)	Coefficient of variation
Ryoso	0.72 (35.8)	0.100
Nogata	0.70 (35.0)	0.170

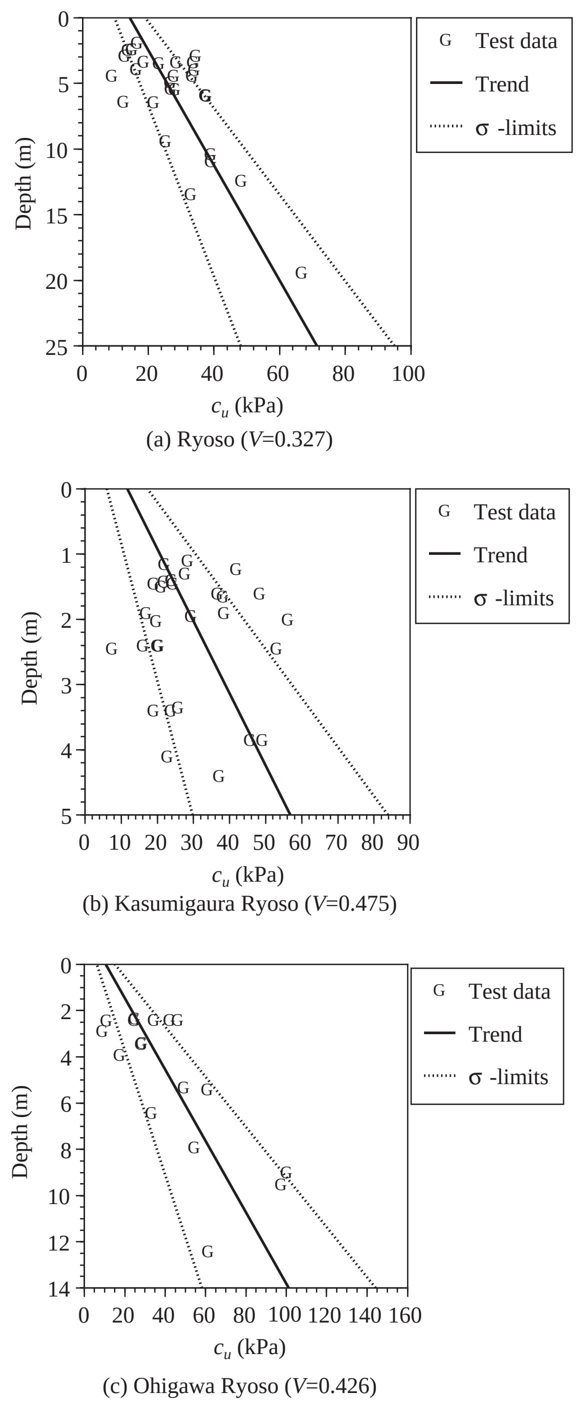

Figure 2. Spatial variations of undrained shear strength from UU triaxial compression tests.

Table 4. Profiles of the open channels.

No.	Width B of open channel (m)	Height H of open channels	Soil type	Unit weight (kN/m^3)	Strength parameter c_u (kPa)	Strength parameter ϕ'(°)	Loading stress q_{max} (kPa)
1	2.96	1.62	S	19.8	0	35	22
2	3.00	1.80	S	18.0	0	23	27
3	2.32	1.20	C	14.0	13	0	21
4	1.70	0.90	C	19.8	18	0	19
5	1.90	1.65	S	20.0	0	25	25
6	2.00	1.70	S	20.0	0	25	33
7	2.00	1.20	S	20.0	0	25	13
8	4.50	1.80	S	19.8	0	23	22
9	2.85	1.65	S	18.8	0	29	24
10	7.90	2.48	S	20.0	0	25	37
11	2.80	1.00	S	20.0	0	25	15
12	2.00	2.20	S	20.0	0	30	21
13	3.30	1.80	S	18.0	0	15	23
14	3.30	1.80	C	15.0	8.0	0	23
15	3.40	1.00	S	20.0	0	20	16
16	2.20	1.20	S	20.0	0	20	16

Note: S, Sand; C, Clay.

that the coefficients of variation in Table 1 are really usable as design values.

The maximum bearing stress, q_{max}, is assumed to be static and deterministic. Since the mean value of q_{max} is relatively small in this problem, compared with the q_u value, the variability of this quantity does not significantly affect the results of the computation.

2.3 Dimensions of open channels to be analysed

Table 4 lists the dimensions of 16 open channels and the mean strength parameters and loading stress. Figure 3 shows a definition of the variables used in the equation for the bearing capacity using the data for Case 1. The 16 cases comprises 13 cases in sandy soil assuming zero cohesion and 3 cases for clayey soil assuming $\phi = 0$. The submerged unit weight is considered below the water level.

2.4 Reliability analysis and discussion

Table 5 shows the reliability indices and sensitivities for each soil parameter for the 16 cases studied. The results

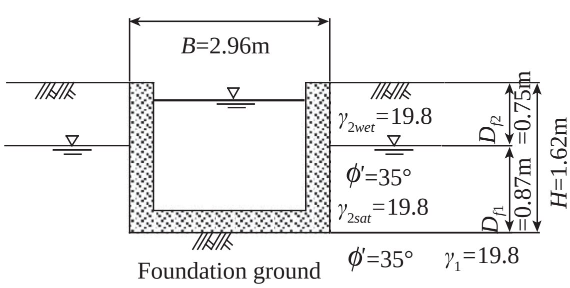

Figure 3. Example of an open channel.

reveal that the reliability indices for sandy soil are in the range of 5.2 and 17.6. The reasons are as follows.

Since parameters N_q and N_γ in Equation (1) are very sensitive to the internal friction angle and the maximum load q_{max} is relatively small compared to the value of q_u in the problems of the open channels, the value of the performance function becomes extremely large. In the cases of the clayey soil, friction angle ϕ is zero and the bearing capacity, q_u has a linear relationship with undrained shear strength c_u, namely, there is no extreme change in bearing capacity q_u for the change in c_u. Consequently, the reliability indices are very similar among three cases.

The sensitivity of the internal friction angle is dominant for the sandy subsoils, and undrained shear strength, c_u, has dominant sensitivity for the clayey subsoils. The unit weights, γ_1 and γ_2, have small sensitivities. Since unit weight γ_1 is usually treated as a submerged unit weight, the sensitivity to the bearing capacity is smaller than for unit weight γ_2.

The reliability indices for sandy subsoils are comparably large for the presented conditions. The reliability indices for the clayey soil are almost 3.0, which corresponds to a probability of failure of 0.1%, and thus, the structures on the ground are sufficiently stable.

3. Determination of partial factors for the foundation of open channels

3.1 Determination of partial factors

In this research, partial factors are adopted to the material parameters (material factor approach). The following methods are listed in ISO2394:

Table 5. Reliability indices and sensitivities of the soil parameters.

No.	β	Soil type	Sensitivity of parameters			
			c_u	tan ϕ'	γ_1	γ_2
1	11.0	S	–	0.985	0.000	0.171
2	6.1	S	–	0.963	0.003	0.268
3	3.2	C	0.997	–	0.000	0.071
4	3.2	C	0.999	–	0.000	0.041
5	10.0	S	–	0.963	0.000	0.268
6	5.9	S	–	0.975	0.003	0.224
7	12.7	S	–	0.956	0.000	0.294
8	5.2	S	–	0.988	0.016	0.154
9	17.0	S	–	0.986	0.000	0.167
10	7.5	S	–	0.968	0.000	0.251
11	9.5	S	–	0.963	0.000	0.271
12	17.6	S	–	0.961	0.000	0.275
13	6.5	S	–	0.887	0.003	0.462
14	2.9	C	0.990	–	0.000	0.140
15	6.3	S	–	0.957	0.003	0.290
16	14.0	S	–	0.838	0.000	0.545

Note: S, Sand; C, Clay.

(1) Cases where the soil parameters follow a normal distribution

When probabilistic variables for the soil parameters follow a normal distribution and their characteristic value is the mean, partial factor ρ is defined as follows:

$$\rho = f_k/f_d = 1/(1 - V\alpha\beta_t) \tag{18}$$

where f_k, the characteristic value of the parameter, usually, $f_k = \mu$; f_d, the design value of the parameter for the reliability analysis; V, coefficient of the variation of the parameter; α, sensitivity of the parameter and β_t, target reliability index.

(2) Cases where the soil parameters follow a lognormal distribution

When probabilistic variables follow a lognormal distribution, partial factor ρ is written as follows:

$$\begin{gathered} \rho = f_k/f_d \\ f_k = \exp(\lambda) \\ f_k = \exp(\lambda - \alpha\beta, \varsigma) \end{gathered} \tag{19}$$

where λ, the mean of the logarithms of the probabilistic variables, $\lambda = \ln(\mu/\sqrt{1+V^2})$, ζ, standard deviation of the logarithms of the probabilistic variables, $\zeta = \sqrt{\ln\ (1+V^2)}$, μ, the mean of the probabilistic variables and V, the coefficient of variation of the probabilistic variables.

3.2 Calibration of the partial factors

A series of partial factors, ρ, are defined for each of the four parameters, and m sets of partial factors, ρ_j ($j = 1, 2,\ldots, m$) are prepared. For each of the 16 cases listed in Table 4, the reliability indices, $\beta_{ij} = \beta_{ij}(\rho_j)$ ($i = 1, 2,\ldots,16$), are calculated with partial factors ρ_j, ($j = 1, 2,\ldots, m$). Consequently, $16 \times m$ reliability indices are obtained and the summations of the squared deviation for β_{ij} from the target reliability index, β_t. The sum of squared deviation between the examined and target reliability indices, D_j ($j = 1, 2,\ldots, m$) are computed as follows:

$$D_j = \sum_{i=1}^{n}[\beta_{ij}(\rho_j) - \beta_t]^2 \quad (j = 1, 2, \ldots, m) \tag{20}$$

The optimum partial factors are selected for the minimum D among D_j ($j = 1, 2,\ldots, m$), so that the calculated reliability index closely approaches the target reliability index β_t.

For each site, the different sensitivity α is calculated. Corresponding to α, the partial factor is obtained for the target reliability index, β, the sensitivity affects the D values. Selecting the partial factor corresponding to the minimum D means obtaining the mean value of the partial factors.

In the reliability analysis in this section, coefficient of internal friction tanϕ follows the normal distribution, while the undrained shear strength c_u is assumed to lognormally distribute. Though the coefficient of variation for c_u was assumed to be 0.3 in this study, the value is too large to be applied to Equation (18), in which the soil parameters must be assumed to normally distribute. If the coefficient of variation of 0.3 and the target reliability index of $\beta_t = 4.0$ are substituted into Equation (18) simultaneously, the partial factor takes negative value. Consequently, the lognormal distribution is better assumption for the undrained shear strength, since the negative partial factors can not be defined.

3.3 Performance function for calibration analysis

In the conventional design code for open channels (Ministry of Agriculture, Forestry and Fisheries 2001), the following equation is employed to check the stability of the foundations, in which a safety factor of 3.0 is considered for the bearing capacity. The safety factor of 3.0 is supposed to be very conservative in general.

$$\frac{1}{3}q_u - q_{\max} \geq 0 \tag{21}$$

The maximum loading stress, $q_{\max}$, is relatively small as seen in Table 4. As a result, the calculated reliability indices for open channels designed with the conventional design method have great values as shown in Table 5. The values of $q_{\max}$ are different for each site, and therefore, the actual $q_{\max}$ values are not used for the determination of the partial factors. As a performance function, Equation (22) is employed in following sections instead of Equation (5).

$$g_q = q_u - q_d \tag{22}$$

where q_d is the design bearing capacity and adjusted so that the computed reliability index obtained with Equation (22) exactly coincides with the target reliability index in the calibration analysis. Another reason why $q_{\max}$ is not used here is that the highest allowable load needs to be considered here. The highest allowable load corresponds to the design bearing capacity for the respective target reliability index.

3.4 Computation of sensitivities by design values

The partial factors for each case listed in Table 4 are computed for the target reliability indices of $\beta_t = 2$, 3 and 4, based on Equations (18) and (19). The partial factors for unit weight γ and coefficient of friction, $\tan\phi'$ are computed based on Equation (18), by adopting the coefficients of the variation in Table 1, assuming a normal distribution. As for the undrained shear strength, the partial factors for each case are evaluated by Equation (19) for lognormally distributed variables, because undrained shear strength c_u follows a lognormal distribution.

In Tables 6 and 7, the mean values and standard deviations of the sensitivities calculated based on Equation (17) are listed for the target reliability indices of $\beta_t = 2$, 3 and 4, respectively. The sensitivity values of the coefficient of friction and the undrained shear strength are >0.98, while those of the unit weights γ_1 and γ_2 are <0.15. Table 6 shows a value of γ_2 of 0.263. The results of Table 6 highlight that the strength parameters, $\tan\phi'$ and c_u, are high and are the dominant parameters in the determination of β_t. Table 7 shows that the standard deviations of the sensitivity for each parameter are small and therefore confidence in the values in Table 6 is high.

Table 6. Expected values of sensitivities for 16 cases ($\beta_t = 2$, 3 and 4).

Target reliability index β_t	Expected value of sensitivity α					
	Sand			Clay		
	$\tan\phi'$	γ_1	γ_2	c_u	γ_1	γ_2
$\beta_t = 2$	0.981	0.047	0.179	0.984	0.000	0.160
$\beta_t = 3$	0.979	0.035	0.189	0.974	0.000	0.208
$\beta_t = 4$	0.977	0.023	0.201	0.958	0.000	0.263

3.5 Computation of the partial factors by the design values

The expected values and the standard deviation of the partial factors for the target reliability indices of $\beta_t =$ 2, 3 and 4 are listed in Tables 8 and 9, respectively.

(1) The obtained partial factors of unit weights γ_1 and γ_2 are comparatively small between 1.00 and 1.07.
(2) The standard deviation of the partial factors is quite small for every case. This leads to the adoption of the expected value of the partial factors for each case.

3.6 Examination of partial factors by calibration

In order to examine an optimal set of partial factors, based on their expected values evaluated in the previous subsection, trial partial factors are proposed as multiples of 0.05 to cover the expected value of the

Table 7. Standard deviations of sensitivities for 16 cases ($\beta_t = 2$, 3 and 4).

Target reliability index β_t	Standard deviation of sensitivity α					
	Sand			Clay		
	$\tan\phi'$	γ_1	γ_2	c_u	γ_1	γ_2
$\beta_t = 2$	0.014	0.012	0.062	0.016	0.000	0.090
$\beta_t = 3$	0.016	0.009	0.065	0.026	0.000	0.112
$\beta_t = 4$	0.018	0.006	0.068	0.039	0.000	0.133

Table 8. Expected values of partial factors for 16 cases.

Target reliability index β_t	Expected values of partial factor, ρ					
	Sandy soil			Clayey soil		
	tan ϕ'	γ_1	γ_2	c_u	γ_1	γ_2
$\beta_t = 2$	1.42	1.01	1.02	1.86	1.00	1.02
$\beta_t = 3$	1.79	1.01	1.03	2.46	1.00	1.04
$\beta_t = 4$	2.42	1.01	1.05	3.22	1.00	1.07

Table 9. Standard deviations of partial factors for 16 cases.

Target reliability index β_t	Standard deviation of partial factor, ρ					
	Sandy soil			Clayey soil		
	tan ϕ'	γ_1	γ_2	c_u	γ_1	γ_2
$\beta_t = 2$	0.009	0.005	0.009	0.021	0.000	0.010
$\beta_t = 3$	0.023	0.004	0.013	0.057	0.000	0.020
$\beta_t = 4$	0.059	0.003	0.018	0.014	0.000	0.040

partial factors appearing in Table 8. Table 10 shows a set of partial factors for each target reliability index, β_t.

The calibration has been made based on the set of partial factors in Table 8 using Equation (20), and resultant deviation of the reliability index from the target reliability index, β_t, is obtained in each case. As a result of such a calibration, Table 11 lists an optimal set of partial factors to minimise the sum of the square of the deviations in each case for different target reliability indices.

Since the conventional design code for open channels results in very conservative designs, almost all open channels including these 16 cases have been overdesigned. Because the bearing stress has a great uncertainty, whose level cannot reliably be predicted, the design must be conservative to prevail the uncertainty. Therefore, relatively great values of the partial factors are proposed to accommodate the uncertainty in the bearing stress. However, the proposed partial factors may decrease when the design becomes more reliable.

4. Conclusions

This paper has evaluated reliability indices for the foundations of existing open channels in order to examine the safety of the current design method for the bearing capacity and the effect of the uncertainty of soil parameters. The concluding remarks are as follows.

(1) The statistical properties of the soil parameters were investigated based on the published data and the results of tests conducted at several sites. The coefficients of variation have been determined to be 0.3, 0.15 and 0.06 for the cohesion, the internal friction angle and the unit weight, respectively.

(2) Reliability analyses have been performed for the 16 sites designed with the current design

Table 10. Set of partial factors for the cases ($\beta_t = 2$, 3 and 4).

Parameters	$\beta_t = 2$		$\beta_t = 3$		$\beta_t = 4$	
	Average factors	Examined factors	Average factors	Examined factors	Average factors	Examined factors
Sandy soil						
Unit weight, γ_1,γ_2	1.01, 1.02	1.00, 1.05	1.01, 1.03	1.00, 1.05	1.00, 1.05	1.00, 1.05
Coefficient of friction, tan ϕ'	1.42	1.40, 1.45	1.79	1.75, 1.80	2.42	2.40, 2.45, 2.50
Clayey soil						
Unit weight, γ_1,γ_2	1.00, 1.02	1.00, 1.05	1.00, 1.04	1.00, 1.05	1.00, 1.07	1.00, 1.05, 1.10
Undrained shear strength, c_u	1.86	1.85, 1.90	2.46	2.45, 2.50	3.22	3.20, 3.25, 3.30

Table 11. Optimum set of partial factors for the cases ($\beta_t = 2$, 3 and 4).

Parameters	$\beta_t = 2$		$\beta_t = 3$		$\beta_t = 4$	
	Sandy soil	Clayey soil	Sandy soil	Clayey soil	Sandy soil	Clayey soil
Unit weight, γ_1,γ_2	1.05	1.05	1.05	1.05	1.05	1.10
Coefficient of friction, tan ϕ'	1.40	–	1.80	–	2.50	–
Undrained shear strength, c_u	–	1.85	–	2.50	–	3.30

code. Consequently, it has been revealed that the current code presents a conservative design for the bearing capacity of foundations with a reliability index of >3.0, and that the internal friction angle and the cohesion are the dominant parameters that affect the safety.

(3) The reliability index obtained for sandy soil is >5.0 for the sites of open channels constructed based on the conventional design code. In the authors' opinion, the value of 5.0 is too conservative. The β index for clayey soil is approximately 3.0.

(4) The partial factors have been obtained by considering the sensitivity of the variability in the soil parameters for three target reliability indices, namely, 2, 3 and 4. Finally, design partial factors corresponding to the target reliability indices have been proposed as the mean values for the 16 cases. The results obtained in the current paper are limited to shallow foundations under open channels, but the evaluation of the reliability indices shown herein is effective for any type of structure.

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