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New method for internal stability analysis of geogrid-reinforced soil wall – Slice method considering distribution of confining effect

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ABSTRACT: It has been shown experimentally that confining pressure in the ground increases locally because of pull-out of reinforcement. This paper proposes a distribution model of the confining effect in the ground considering pull-out behavior. The model was introduced into the equation to calculate the safety factor for internal stability of geogrid-reinforced soil wall and embankment. In order to introduce the confining effect, methods are shown to equalize the distribution model. Furthermore, the applicability of the safety factor obtained using the proposed method was verified using results of previous a full-scale failure experiment of a geogrid- reinforced soil wall.

1 INTRODUCTION

Shear strength increment due to tensile force developed in reinforcement is considered in design method of reinforced soil wall. However, an additional increment of shear strength exists that is independent of the tensile force. If this increment is introduced into design method, more reasonable design method can be established.

Ochiai et al. (1996, 1998) and Hirai (1997) carried out shear tests of geogrid-reinforced soil. They described that a confining effect increased the shear strength independent of the tensile force. The confining effect was interpreted that normal stress on a sliding surface increases from initial normal stress, σ_{n0}, to $\sigma_{n0} + \Delta\sigma_n = (1 + \beta)\sigma_{n0}$ through confinement of soil deformation by reinforcement, where $\Delta\sigma_n$ is the increment of normal stress and β is the confining effect parameter. In addition, Yamaji et al. (1997) and Hirai (1997) proposed internal stability analyses based on slice method considering that confining effect. In the proposed method, the uniform confining effect is considered for the entire sliding surface in the reinforced zone. The effect between reinforcement layers is not examined.

Umezaki et al. (2005) used pull-out test results to show that vertical stress increases because of reinforcement pull-out. They reported that the vertical stress increment could be approximated by exponential distribution, which decreases with distance from the reinforcement.

In this study, in the case where sliding failure passing through the reinforced zone occurs, a distribution model of the confining effect in the ground is proposed by considering pull-out behavior. In addition, an equation is proposed to calculate safety factors for internal stability of reinforced soil wall and embankment. The distribution model is introduced into that equation. Safety factors calculated using the proposed method are verified using results of a previous failure experiment of geogrid-reinforced soil wall.

2 CONFINING EFFECT BASED ON PULL-OUT BEHAVIOR

Umezaki et al. (2005) modeled stress conditions near sliding surface in a reinforced-soil structure, as shown in Figs. 1(a)–1(d), based on pull-out test results for a dense reinforced specimen, which consisted of a stack of aluminum rods, and reinforcement, which simulated geogrid. During sliding failure, pull-out of reinforcement occurs and vertical stress in the ground, σ_v, increases. The vertical stress increment, $\Delta\sigma_v$, becomes maximum, $\Delta\sigma_{vmax}$, near the reinforcement, and it decreases with distance from the reinforcement, as shown in Fig. 1(a). The following exponential function approximates the distribution:

$$\Delta\sigma_v(x) = \Delta\sigma_{vmax} \exp(bx), \tag{1}$$

where b is a constant and x is the distance from the reinforcement. The value of b is about –0.1 (Umezaki et al. 2005) and that of $\Delta\sigma_{vmax}$ is about 0.2-0.4 (Ogisako et al. 1989, Mitachi et al. 1992).

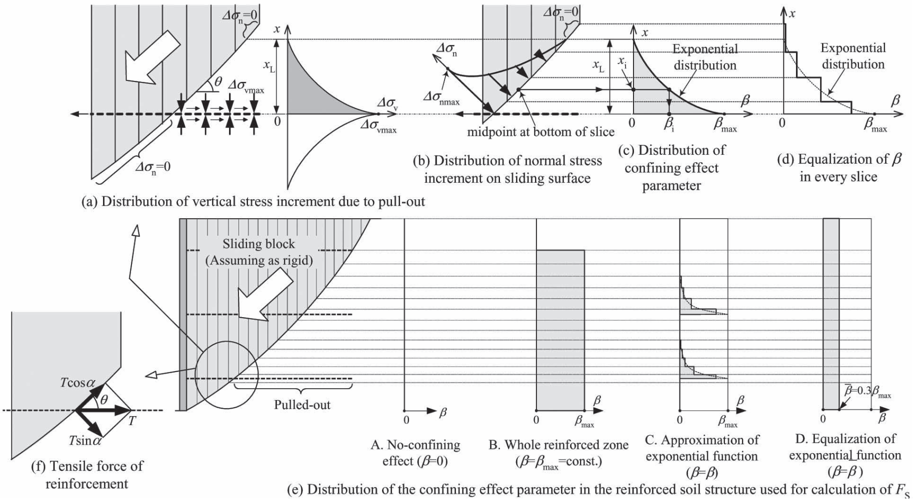

Figure 1. Model of confining effect in reinforced soil structure and its distribution for internal stability analysis.

Because the sliding surface does not exist below the reinforcement, $\Delta\sigma_v$ below the pulled out reinforcement is disregarded as assumption on the safe side. As shown in Fig. 1(b), the distribution of the normal stress increment, $\Delta\sigma_n$ (= $\Delta\sigma_v\cos^2\theta$), on the sliding surface can also be approximated as an exponential distribution, where θ is the angle between the sliding surface and the reinforcement. The distribution of the confining effect can be modeled based on the distribution of $\Delta\sigma_v$. Here, β is expressed as $\beta = \Delta\sigma_n/\sigma_{n0} = \Delta\sigma_v/\sigma_{v0}$. Therefore, as with $\Delta\sigma_v$, the distribution of β becomes maximum β_{max} (= $\Delta\sigma_{nmax}/\sigma_{n0}$) near the reinforcement, and it decreases with distance from the reinforcement. Moreover, β can be expressed as an exponential distribution, as shown in Eq. (2), and Fig. 1(c) and 2. Therefore, shear strength on the sliding surface increases by $\Delta\tau_D = \Delta\sigma_n \tan\phi = \beta\sigma_{n0} \tan\phi$ due to pull-out, where ϕ represents the internal friction angle of soil.

$$\beta(x) = \beta_{max} \exp(bx) \tag{2}$$

3 INTERNAL STABILITY ANALYSIS CONSIDERING CONFINING EFFECT

3.1 *Introducing the model into equation of safety factor*

The equation to calculate the safety factor, F_s, for internal stability of reinforced soil wall based on the slice method is expressed as (PWRI 2000):

$$F_S = \frac{M_R + \Delta M_R}{M_D} = \frac{R\sum\{cl_i + W_i\cos\theta_i\tan\phi + T_i(\cos\theta_i + \sin\theta_i\tan\phi)\}}{R\sum(W_i\sin\theta_i)} \tag{3}$$

where M_R is the moment of resistance in non-reinforced soil, M_D is the sliding moment and ΔM_R is the moment of resistance attributable to the tensile force of reinforcement (see Fig. 1(f)). Subscript i indicates the i-th slice. R is the radius of the sliding circle, l_i is the slice arc length, W_i is the weight of the soil in a slice, c represents soil cohesion, and T_i is the tensile force of reinforcement.

Figure 2 portrays the distribution of β obtained from pull-out tests, which Umezaki et al. (2005) carried out. In this figure, the distribution obtained from substituting b = –0.1 into Eq. (2) is also indicated. We propose that a representative value of β is determined as value of the midpoint at bottom of each slice, β_i, as shown in Fig. 1 (c). Different values for each slice as a step-like distribution are used, as shown in Fig. 1(d). Thus, the shear strength increment due to the confining effect in a slice, $\Delta\tau_{Di}$, is expressed as follows.

$$\Delta\tau_{Di} = \beta_i\sigma_{n0i} \text{ tab } \phi \tag{4}$$

$$\beta_i = \beta_{max} \exp(bx_i) \tag{5}$$

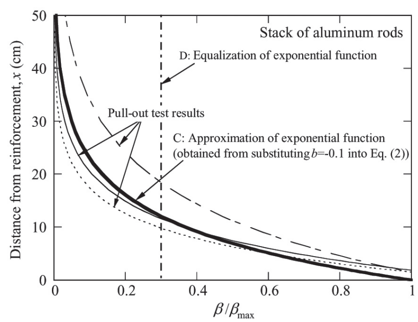

Figure 2. Distribution of confining effect parameter.

Where x_i is distance between reinforcement and midpoint of bottom of a slice, as shown in Fig. 1 (c). Therein, $\Delta\tau_{Di}$ is changed into force from stress and is introduced into ΔM_R. Consequently, the following equation is obtainable.

$$\Delta M_R = R \sum \{T_i(\cos\theta_i + \sin\theta_i \tan\phi)$$

$$+ \beta_i W_i \cos\theta_i \tan\phi\} \quad (6)$$

The equation for safety factor based on slice method is expressed as follows.

$$F_S = \frac{R\Sigma\{cl_i + W_i\cos\theta_i \tan\phi(1+\beta_i) + T_i(\cos\theta_i + \sin\theta_i \tan\phi)\}}{R\Sigma(W_i \sin\theta_i)} \quad (7)$$

Equation (7) has the same shape as the equation that Yamaji et al. (1997) and Hirai (1997) proposed in consideration of the confining effect. However, the distribution of β was not mentioned in those studies, and the confining effect was introduced into the whole reinforced zone, as shown in Fig. 1(e). Herein, the equation for F_S is proposed using β_i.

3.2 *Determination of confining effect parameter*

Kawamura and Umezaki (2004) examined results of shear tests of reinforced soil performed by Ochiai et al. (1996, 1998) and Hirai (1997), and those of pull-out tests performed by Ogisako et al. (1989) and Mitachi et al. (1992). Results showed that values of β and vertical stress increment ratio $\beta^* = \Delta\sigma_v/\sigma_{v0}$ in pull-out tests were related to the shape index $J = nS_D/U_L$, which Hirai (1997) proposed. In that equation, S_D is the reinforcing transverse-rib thickness, n is the number of transverse-ribs per unit width, and U_L is the unit length. In this paper, values of β and β^* obtained from tests were interpreted as those maxima. Figure 3 shows relationships between β_{max}, β^*_{max} and J. Values of β_{max} and β^*_{max} are similar. These relations indicate the value of β_{max}.

4 APPLICABILITY OF PROPOSED METHOD

4.1 *Outline of full-scale failure experiments*

Miyatake et al. (1995) and Tajiri et al. (1996) reported full-scale failure experiments using geogrid-reinforced 6.0-m-high soil walls. Based on results of experiments using concrete panels for facing materials, this study assesses the applicability of F_S calculated using the proposed method. The influences of weight and rigidity of facing on structural stability seem to be small.

The laying specifications of reinforcements were determined according to the standard design method in Japan (PWRI, 2000) assuming $F_S = 1.0$. The F_S against sliding failure was $F_S = 1.32$. Figure 4 shows an outline of the reinforced soil wall. Six reinforcement layers were laid (geogrid (SR-55), peak strength, $T_f = 56.4$ kN/m and design strength, $T_A = 29.4$ kN/m). Soil parameters of the filling material were wet density,

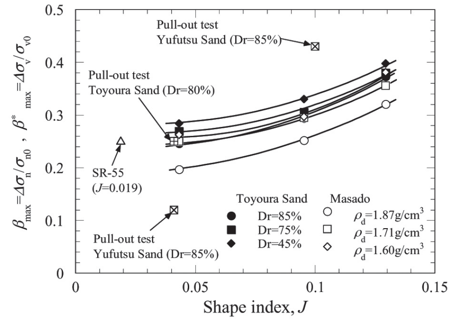

Figure 3. Determination of the confining effect parameter.

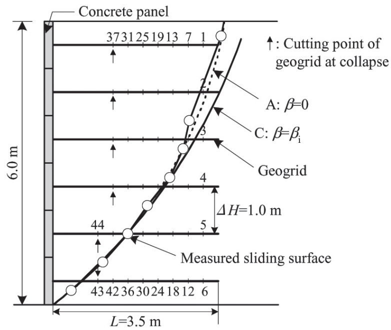

Figure 4. Outline of a geogrid-reinforcement soil wall.

$\gamma_t = 16.0$ kN/m^3, $c = 0.0$ and $\phi = 39.4°$. Horizontal displacement of the wall, earth pressure against the wall, and so on were measured at every step of filling and until failure of the wall by cutting of reinforcements. The numbers in Fig. 4 indicate cutting processes. The wall collapsed instantaneously after cutting No. 44. Arrows in Fig. 4 indicate the cutting point at collapse. The sliding surface after failure is also shown in Fig. 4.

4.2 *Comparison of design calculation results and test results*

Figures 5(a) and 5(b) show changes of horizontal displacement of the wall and F_S calculated using Eq. (7) in the cutting process of reinforcements. Based on the shape index, $J = 0.019$, of the reinforcement, $\beta_{max} = 0.25$ was determined using Fig. 3. The slice width, B, where the reinforcement and sliding surface intersect, was set as $B = 20$ cm. That in other parts were set to $B < 20$ cm. Values of F_S were calculated for the following four cases, as shown in Fig. 1(e). Case A: The confinement effect is ignored ($\beta = 0$). Case B: $\beta = \beta_{max} = 0.25$ is input uniformly into the whole reinforced zone. Case C: β is equalized to a step-like distribution ($\beta = \beta_i$). Case D: $\bar{\beta}$ is equalized in the whole sliding surface. Herein, $\bar{\beta} = 0.3\beta_{max} = 0.3 \times 0.25 = 0.075$ is applied, considering a 1.0 m interval of reinforcements, as shown in Fig. 2.

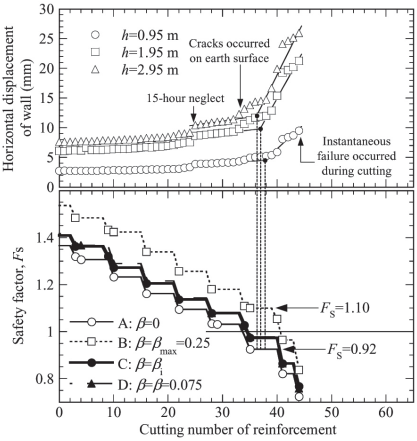

Figure 5. Change of wall-surface horizontal displacement and safety factor in the cutting process of reinforcement.

In Fig. 5(a), two parts, which displacement caused only slightly and which displacement caused rapidly, were approximated using straight lines. The intersection of these lines indicates a failure point that is assumed to be F_S = 1.0. The value of F_S of Case A at failure is F_S = 0.92. The stability of the reinforced soil wall is underestimated. On the other hand, the value of Case B is F_S = 1.10 and the stability is overestimated. The values in Cases C and D are F_S = 0.97 and 0.96, respectively, and F_S is evaluated appropriately using the proposed method. The sliding surfaces in Cases A and C are also shown in Fig. 4. No difference exists between both cases.

5 CONCLUSIONS

Considering the pull-out behavior of reinforcement, the distribution of the confining effect in the ground at sliding failure of reinforced soil wall and embankment was modeled. Furthermore, this distribution was introduced into the equation to calculate the safety factor, F_S, for internal stability based on slice method. The applicability of F_S, which was calculated using the proposed method, was verified using previous full-size failure experiment of geogrid-reinforced soil wall. The main conclusions are as follows.

(1) Considering pull-out behavior of reinforcement, the distribution of the confining effect in the ground was modeled. The distribution of the confining effect parameter, β, is expressed as an exponential function, which decreases with distance from the reinforcement.

(2) The equation to calculate F_S, in which parameter β_i is introduced into every slice, was proposed. In addition, a method to equalize the distribution of the confining effect was shown for every slice as a step-like distribution.

(3) Failure in previous full-scale experiments (F_S = 1.0) was defined based on the wall's horizontal displacement. Values of F_s, as calculated by methods that did or did not incorporate the distribution of the confining effect, were compared. The structural stability is underestimated by methods that neglect the confining effect. On the other hand, stability is overestimated slightly by methods that consider the confining effect in the entire reinforced zone. The value of F_s, considering the distribution, is close to F_S = 1.0. Results of this study support the applicability of this method.

REFERENCES

Hirai, T. 1997. Reinforcing effect of soil-geogrid system, and its application to steep reinforced embankment, Kyushu University Doctoral Thesis, (in Japanese).

Kawamura, Y. and Umezaki T. 2004. Method of Determining Confining Effect Parameter in Geogrid-Reinforced Walls, *Geosynthetics Engineering Journal*, pp. 21-28, Vol. 20, (in Japanese).

Mitachi, T., Muraki, S. and Matsui, E. 1992. Mechanism of in-situ resistance of geogrid with various properties, *Proc. of 27th Japan National Conf. on Geotech. Eng.*, pp. 2395-2398, (in Japanese).

Miyatake, H., Ochiai, Y., Dobashi K., Tajiri, N., Fukuda, N., Nishimura, J. and Sasaki, H. 1995. Full Scale Failure Experiments on Reinforced Earth Wall with Geotextiles (Part 1) – Facing with Concrete Panels-, *Proc. of 34th Japan National Conf. on Geotech. Eng.*, pp.2423-2426, (in Japanese).

Ochiai, H., Yasufuku, N. Yamaji, T., Guang-Li Xu and Hirai, T. 1996. Experimental evaluation of reinforcement in geogrid-soil structure, *Earth Reinforcementt*, Vol. 1, pp. 249-254.

Ochiai, H., Yasufuku, N., Kawamura, T., Yamaji, T. and Hirai, T. 1998. Effect of end-restraint in geogrids-soil structure, *Proc of 6th Int. Conf. on Geosynthetics*, pp. 545-550.

Ogisako, E., Ochiai, H. and Hayashi, S. 1989. Properties of Pull-out Resistance of Geogrid in Soil, *Proc. of 24th Japan National Conf. on Geotech. Eng*., pp. 13-16, (in Japanese).

Public Works Research Institute, Ministry of Construction. 2000. Design and Construction manual of Geotextile-reinforced Embankments, *Technical Memorandum of Public Works Research Institute*, No. 404p, (in Japanese).

Tajiri, N., Sasaki, H., Nishimura, J., Ochiai, Y. and Dobashi, K. 1996. Full-scale failure experiments of geotextile-reinforced soil walls with different facings, *Earth Reinforcement*, pp. 525-530.

Umezaki, T. and Kawamura, T. 2004. Pull-out Behavior of Reinforcement in Soil Subjected to Several Conditions, *Proc. of 3rd Asian Regional Conference on Geosynthetics*, pp. 996-1003.

Umezaki, T., Kawamura, T. and Ochiai, H. 2005. Increment of Confining Pressure due to Pull-out of Reinforcement and Modeling of Reinforcing mechanisms, *Geosynthetics Engineering Journal*, pp. 241-248, Vol. 20, (in Japanese).

Yamaji, T., Kawamura, T., Ochiai, H., Yasufuku, N. and Hirai, T. 1997. Confining effect in geogrid-reinforced embankment and its application to design method, *Proc. of 32nd Japan National Conf. on Geotech. Eng*., pp. 2529-2530, (in Japanese).

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