Confining Effect in Geogrid-Reinforced Soil related to Soil Dilatancy

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ABSTRACT: The reinforcing effects of geogrid-reinforced soil are generally evaluated by the tensile effect due to the tensile force of a geogrid. However, we have experimentally examined the existence of the confining effect, which is one component of the reinforcing effects and is independent on the tensile force of a geogrid. An evaluation method in which the reinforcing effect can be divided into tensile effect and confining effect is proposed related to the dilatancy rate of reinforced soil mass. The mobilized confining effect is given by a function of the dilatancy rate, in which the basic idea is in the assumption of the dissipated energy done by unit volume of the reinforced soil mass. The confining effect is introduced into the tie-back wedge method of geogrid-reinforced retaining walls in a practical design. A new formula for calculating the maximum tensile force of geogrid mobilized on the sliding plane was derived on the basis of Rankine’s active earth pressure theory.

1 INTRODUCTION

The reinforced effects of geogrid-reinforced soil are generally evaluated by the tensile effect alone due to the tensile force of a geogrid. Some researchers reported, based on the in-situ measurements, that the tensile force of a geogrid, which should be mobilized for the stability of a structure, was not fully mobilized in soil, although the structure maintained the sufficient stability. The other researchers reported that a strong earthquake inflicted little damage on a reinforced structure. These studies suggest the existence of an additional reinforcing effect other than the tensile effect due to tensile force of a geogrid. In a previous study, the reinforcing effects of geogrid-reinforced soil were experimentally examined. As an important result, the existence of an additional reinforcing effect in laboratory and model tests was confirmed, and the additional effect was defined as the confining effect (Ochiai et al., 1996, 1998; Kawamura et al., 2000; Yasufuku et al., 2002).
It has been found from a series of laboratory tests that the confining effect mobilized in the reinforced soil mass exists, which is independent on the tensile force of a geogrid. The quantitative evaluation is requested to apply the confining effects to the general geogrid-reinforced soil structure. In the present study, the confining effect was evaluated quantitatively by experimental and theoretical considerations. A parameter for estimating the confining effect is derived as a function of the dilatancy angle of the reinforced soil mass in which the important key idea is in an assumption of the dissipated energy done by unit volume of reinforced soil mass. It was introduced into the tie-back wedge method, which is one of the most widely used method in the world, based on Rankine’s earth pressure theory as shown in Figure 1. A new formula that takes into account the confining effect related to the soil dilatancy behaviour is proposed for checking the safety of each layer of a geogrid. Finally, critical height of geogrid reinforced retaining wall, which are calculated by tie-back wedge method with and without consideration of the confining effect, are discussed.

2 REINFORCING EFFECTS IN GEOGRID-REINFORCED SOIL

The reinforcing effects of geogrid-reinforced soil are often evaluated by only the tensile effect due to the tensile force of a geogrid. Jewell and Wroth (1987) carried out direct shear tests on reinforced soil, as shown in Figure 2, and they showed that the shear resistance of reinforced soil increased by

\[ \tau_{\text{EXT}} = \frac{P_R}{A_S} \left( \sin \theta \tan \phi + \cos \theta \right) \]  

(1)

where \( \tau_{\text{EXT}} \) is the increment of shear resistance, \( P_R \) is the mobilized tensile force of the reinforcement, \( A_S \) is the area of the sliding plane, \( \phi \) is the internal friction angle of soil and \( \theta \) is the angle between the reinforcement and the sliding plane.

Equation (1) can be considered as the tensile effect due to the tensile force of a geogrid. As shown in Equation (1), \( \tau_{\text{EXT}} \) does not depend on the normal stress on the sliding plane and it is equal to the increment of apparent cohesion of soil, \( c_T \), in Figure 3. Considering this tensile effect, the relationship between the shear strength of reinforced soil, \( s_R \), and the normal stress, \( \sigma_n \), can be expressed as

\[ s_R = c + c_T + \sigma_n \tan \phi \]

\[ = c + \frac{P_R}{A_S} \left( \sin \theta \tan \phi + \cos \theta \right) + \sigma_n \tan \phi \]  

(2)

Fukushima et al. (1988) carried out the large-scale triaxial compression tests using geogrid-reinforced sand, and their results clearly showed the internal friction angle increases.
We have defined the confining effect, which will reflect an apparent incremental confining stress in reinforced soil mass, as an effect that is independent on the tensile effect, and also we have proposed an evaluation method that takes into account both the tensile effect and the confining effect, as shown in Figure 4. “$\beta \tan \phi$” in Figure 4 is the increment of the slope of the reinforced line of the $s$-$\sigma_n$ relationship. However, in order to simply introduce the confining effect into a design method, the shear strength of reinforced soil should be evaluated not as the increment of the internal friction angle, $\beta \tan \phi$, but as the increment of the normal stress, $\beta \sigma_n$, as follows (Ochiai et al., 1996, 1998):

$$s = c + \frac{Pr}{As} (\sin \theta \tan \phi + \cos \theta) + (1 + \beta)\sigma_n \tan \phi$$

(3)

The confining effect is believed to be the effect of restriction of soil around the geogrid by the geogrid, and the confining stress around the geogrid apparently increases. In other words, this idea indicates that the confining effect is closely connected with the dilatancy behaviour of reinforced soil mass.

3 EXPERIMENTAL OBSERVATIONS OF CONFINING EFFECTS

3.1 Test Apparatus and Test Procedure

A new shear test apparatus was developed to investigate the reinforcing effects on the sliding plane of geogrid-reinforced soil mass, as shown in Figure 5 (Ochiai et al., 1996). The shear box is rectangular in shape and is 200 mm wide, 200 mm long, and 380 mm high. The shear box is divided into two equally sized upper and lower parts by sliding plane inclined by an angle of $\theta$. One end of the geogrid is fixed to the upper part of the shear box, so that the sliding soil mass with geogrids moves as a rigid block. This is the central feature. A con-
3.2 Test results and existing of confining effect

Figures 6 (a) and (b) shows the typical relationships between shear strength, \( s \), and normal stress, \( \sigma_n \), obtained from the results of a series of tests conducted under the condition of various tensile forces of the geogrid. The shear strength, \( s \), is defined as the maximum value of shear stress, \( \tau = (P/A) \sin \theta \), until the shear displacement reaches 10 mm. “\( P \)” is the vertical load, “\( A \)” is the area of the sliding plane and \( \theta \) is the sliding angle. The normal stress, \( \sigma_n \), is the normal component of the overburden pressure, \( \sigma_0 \), against the sliding plane and is expressed as \( \sigma_n = \sigma_0 \cos \theta \). The relationships between \( s \) and \( \sigma_n \) for both non-reinforced and reinforced soils are expressed by straight lines. Similar relationships were obtained under different conditions of the sliding angle, \( \theta \), shape index, \( R \), relative density of Toyoura sand, \( D_r \), and dry unit weight of Masado, \( \gamma_d \). The relationships can therefore schematically be modeled as shown in Figure 7. For Toyoura sand, the intercept of the relationship of non-reinforced sand is zero. The evaluation method that takes into account both the tensile effect and the confining effect was verified by laboratory tests in which the sliding plane in the geogrid-reinforced soil mass was simulated (see Figure 7).

Considering the stress condition near the sliding plane of a reinforced retaining walls as shown in Figure 5, Eq.(3) can be rewritten as Eq.(4).

\[
s_R = c + \frac{T}{A} (\sin \theta \tan \phi + \cos \theta) + \sigma_n \tan \phi + \beta \cdot \sigma_n \tan \phi
\]

The fourth term in Eq.(4), \( \beta \sigma_n \tan \phi \), is the confining effect independent of the tensile force of the geogrid. This term means that the normal stress on the sliding plane, \( \sigma_n \), apparently increases due to the confining effect, as illustrated in Figure 8. The coefficient \( \beta \) is a parameter for evaluating the confining effect which is considered to be dependent on the dilatancy behaviours of the reinforced soil mass.

4 EVALUATION OF CONFINING EFFECT RELATED TO SOIL DILATANCY

The confining effect of reinforced soil mass is supposed to be the effect of restriction of soil around the geogrid by the geogrid, and thus the confining stress around the geogrid apparently increases shown in Figure 8. In other words, it is considered that the confining effect of reinforced soil mass is closely related to the soil dilatancy behaviours during shearing. In turn, in order to rationally evaluate the confining effect parameter \( \beta \) in
Eq.(4), the work equations applied to a simple shear test sample are discussed for reinforced and non-reinforced soil mass. We shall suppose that a small shear stress increase $d\tau$ causes a shear deformation, so that the shear strain $d\gamma_y > 0$ and likewise a small normal stress $d\sigma_n$ causes a vertical compression so that the direct strain $d\varepsilon_y > 0$. According to the stress-strain system in Figure 9, we now can deduce the magnitude of the plastic work which is fed into the element and presumably dissipated such that

$$dW = \sigma_n d\varepsilon_y + \tau d\gamma_{xy}$$  \hspace{1cm} (5)$$

We assume that this work has been dissipated by friction and dilation due to shearing. Then, the general form of the dissipation energy equation for reinforced soils, which doesn’t directly include the confining effect parameter $\beta$, is supposed to be given by

$$dW_{\text{non-r}} = \sigma_n \sqrt{\left(\tan \phi_c \cdot d\gamma_{xy}\right)^2 + \left(d\varepsilon_y\right)^2}$$  \hspace{1cm} (6)$$

where $\phi_c'$ is defined as an internal friction angle at critical state, and note that this equation is similar to that of modified Cam-clay in which both friction and volumetric terms are included. On the other hand, when considering that the confining effect, that is, an apparent incremental direct stress is generally mobilized by the restriction of smooth movement of soils around the geogrid, it is essential to understand that the confining effect can be mobilized by being restrictive of soil dilative behaviour due to shearing. Assuming that the reinforced soil mass can be homogenized in average, it is believed that the dissipated work in reinforced soils, which directly reflect the confining effect, can be expressed as

$$dW_r = \left(\sigma_n + \Delta\sigma_n\right) \sqrt{\left(\tan \phi_c \cdot d\gamma_{xy}\right)^2 + \left(d\varepsilon_y\right)^2}$$  \hspace{1cm} (7)$$

where, $\Delta\sigma_n$ is an apparent incremental normal stress corresponding to the confining effect. It is clear that this type of equation is similar to that of Cam-clay. Furthermore when remembering that $\Delta\sigma_n$ is equal to $\beta\sigma_n$ as shown in Eq.(4), Eq.(7) is rewritten as

$$dW_r = \left(\sigma_n + \beta\sigma_n\right) \sqrt{\left(\tan \phi_c \cdot d\gamma_{xy}\right)^2}$$  \hspace{1cm} (8)$$

![Figure 9 Dilatancy angle at peak strength state in direct shearing, and the corresponding Mohr circle of strain increment](image)
We presumed that the work done by the stresses at the boundaries in Eq.(5) must have been internally dissipated. Thus, Eqs.(7) and (8) represent an internal energy dissipation equation which contains 1) an internal friction angle $\phi'_c$ for non-reinforced soil, 2) the normal stress with a confining effect for reinforced soil mass and 3) an internal shear strain which defines the magnitude of the change of the shape of the element.

After assuming that the dissipated work equations at peak strength state in Eq.(6) is equivalent to that in Eq.(8), so that $dW_{\text{non-r}}=dW_r$ and by merging Eq.(6) with Eq.(8) we obtained the following equation

$$
\sigma_n\left[\tan\phi'_c d\gamma_{xy}\right]^2 + \left(d\varepsilon_y\right)^2 = \left(\sigma_n + \beta\sigma_n\right)\left[\tan\phi'_c d\gamma_{xy}\right]^2
$$  \hspace{1cm} (9)

After brief calculations in terms of the confining effect parameter $\beta$, we can get the following equation

$$
\frac{\sigma_n + \beta\sigma_n}{\sigma_n} = \sqrt{\left[\tan\phi'_c d\gamma_{xy}\right]^2 + \left(d\varepsilon_y\right)^2} \quad \frac{\sigma_n + \beta\sigma_n}{\sigma_n} = \frac{\left[\tan\phi'_c d\gamma_{xy}\right]^2}{\left[\tan\phi'_c d\gamma_{xy}\right]^2}
$$  \hspace{1cm} (10)

Thus,

$$
\beta = \sqrt{1 + \left(\frac{d\varepsilon_y}{\tan\phi'_c d\gamma_{xy}}\right)^2} - 1 = \sqrt{1 + \left(\frac{\tan\psi}{\tan\phi'_c}\right)^2} - 1
$$  \hspace{1cm} (11)

It is clear that $\beta$ is given as a function of the dilatancy rate defined by $d\varepsilon_y/d\gamma_{xy}$ at peak direct shear strength for the specimen. For simplicity, we assume that $d\varepsilon_y/d\gamma_{xy}$ at peak strength state can be obtained from the simple direct shear tests of the non-reinforced soil mass associated with a reinforced soil mass. Eq.(11) represents that $\beta$ becomes greater with the increasing dilatancy rate at peak state. It is noted that, based on the definition of dilatancy angle $\psi$ shown in Figure 9, $d\varepsilon_y/d\gamma_{xy}$ is easily connected with $\psi$, that is, $\tan\psi = d\varepsilon_y/d\gamma_{xy}$. Further the dilatancy rate $d\varepsilon_v/d\varepsilon_s$ under plain strain condition is often used to indicate the dilatancy behaviours where $d\varepsilon_v = d\varepsilon_1 + d\varepsilon_3$ and $d\varepsilon_s = d\varepsilon_1 - d\varepsilon_3$. In this case, considering the characteristics of Mohr circle of strain shown in Figure 9, the dilatancy angle $\psi$ is also related to $d\varepsilon_v/d\varepsilon_s$, such that $\sin\psi = d\varepsilon_v/d\varepsilon_s$. Therefore, the mutual relationship between $d\varepsilon_y/d\gamma_{xy}$ and $d\varepsilon_v/d\varepsilon_s$ is connected through the dilatancy angle $\psi$ such that

$$
\tan^{-1}\left(\frac{d\varepsilon_y}{d\gamma_{xy}}\right) = \sin^{-1}\left(\frac{d\varepsilon_n}{d\varepsilon_s}\right) = \psi
$$  \hspace{1cm} (12)

Figure 10 shows the characteristics of $\beta$ against the dilatancy angle $\psi$ directly linked with $d\varepsilon_y/d\gamma_{xy}$ and/or $d\varepsilon_v/d\varepsilon_s$ in terms of internal friction angle $\phi'_c$ at peak strength state for the simple shear under the plain strain conditions which are calculated by Eqs (11) and (12). It is found that $\beta$ roughly changes from 0 to 0.3 with increasing the dilatancy angle $\psi$ from 0 to 30 degrees as a function of $\phi'_c$ which are believed to be an expected
range of $\psi$ and also that the increasing rate of $\beta$ becomes higher with the increasing dilatancy angle. This tendency asserts that the compaction of reinforced soil mass is important to obtain the proper confining effect. It is further pointed out that the predicted range of $\beta$ gives a good agreement with the experimental results which have been reported by the authors (Ochiai et al. 1996; Kawamura et al., 2000; Yasufuku et al., 2002).

5 INTRODUCTION OF CONFINING EFFECT INTO CURRENT DESIGN GUIDELINE

5.1 Tie-Back Wedge Method

In this section, the confining effect is introduced into the tie-back wedge method recommended in the Canadian Foundation Engineering Manual (1992: see Figure 1), which is characterized to be simple to use for applications. Internal stability includes the failure mechanism such that 1) rupture of the geogrid due to tensile overstressing, 2) pullout of the geogrid within the reinforced soil mass and 3) failure of the facing connection. Among those, we focused on the first of the above mechanisms, thus, the confining effect mobilized in the reinforced soil mass is introduced into the internal stability analysis related to the rupture of the geogrid. In the case of the external stability in which the reinforced soil mass is regarded as a rigid block, the confining effect may not be mobilized.

In the tie-back wedge method, the safety of each layer of the geogrid is referenced to an internal Rankine active plane propagating from the toe of the wall at an angle of $45^\circ + \phi/2$ degrees to horizontal. The essential features are summarized in Figure 11. The maximum tensile force of the geogrid, $T_{\text{max}}$, is mobilized at the sliding plane in the reinforced soil mass. Rankine’s earth pressure theory gives the horizontal earth pressure as a triangular distribution. Horizontal active earth pressure, $\sigma_h(z)$, at the depth of $z$ from the top of the wall can be expressed as

$$\sigma_h(z) = K_a \gamma \gamma + K_a q$$

where $K_a$ is the coefficient of Rankine’s active earth pressure, $\gamma$ is unit weight of soil and $q$ is the surcharge load.

The tensile force of the geogrid is thought to resist the horizontal earth pressure directly, and the sum of tensile forces of each layer of the geogrid must be larger than that of the horizontal earth pressure.

The maximum tensile force, $T_{\text{max}}$, of the geogrid at the depth of $z$ from the top of the wall is expressed as the area of the gray zone in Figure 11 and it is computed as

$$T_{\text{max}} = \sigma_h(z) \times S_v = (K_a \gamma \gamma + K_a q) \times S_v$$

where $S_v$ is the contributory area about each geogrid layer. As a general treatment, $S_v$ is often regarded as the length of the vertical space of geogrid. The allowable design tensile force of the geogrid, $T_A$, must exceed the maximum tensile force, $T_{\text{max}}$, as follows:
Equation (15) is used to check the safety of each layer of the geogrid, and the vertical spacing of each layer of geogrid is determined. The length of each layer of the geogrid is thus obtained by considering the pullout resistance of the geogrid. However, the determination of the length is not discussed in this paper.

The confining effect can be expressed as “$\beta \sigma_n \tan \phi$”, as shown in Eq.(4), and also it reflects the effect of an additional normal stress, $\beta \sigma_n$, apparently induced on the sliding plane. Thus, the overburden pressure on the horizontal surface of the geogrid, $\sigma_0$, which can be computed as $\sigma_0 = \frac{\sigma_n}{\cos \theta}$, apparently increases by the amount of $\beta \sigma_0$ due to the confining effect. It is assumed that the ultimate earth pressure of the reinforced retaining wall under plastic equilibrium conditions could be changed by the amount of $K_a \beta \sigma_0 = K_a \beta \gamma z$ due to the confining effect. This assumption is based on the following simple idea. The sliding angle of the ultimate slip failure, $45^\circ + \phi/2$ degrees, and the coefficient of Rankine’s active earth pressure, $K_a$, is regarded as a constant, because the confining effect is evaluated independently against the internal friction angle, $\phi$, as shown in Figure 4. Therefore, the maximum tensile force that directly resists to the horizontal earth pressure can be reduced by the amount of $K_a \beta \gamma z$ as schematically shown in Figure 12.

Finally, considering the confining effect, Eq.(15) of the design method recommended in the Canadian Foundation Engineering Manual (1992) can be rewritten as

$$T_A \geq T_{\text{max}} = (K_a \gamma z + K_a q) \times S_V$$

(15)

where $\beta$ is directly related to the dilatancy angle $\psi$ when introducing Eq.(12) into Eq.(11), such that:

$$\beta = \sqrt{1 + \left(\frac{\tan \psi}{\tan \phi_c}\right)^2} - 1$$

(17)

Eq.(16) with Eq.(17) is used to check the safety of each layer of the geogrid in the reinforced retaining wall.
A cross-sectional diagram of a geogrid-reinforced retaining wall and the parameters for design are shown in Figure 13 and Table 2, respectively. The values of the confining effect parameters, $\beta$, used for the computation are estimated by using Eq.(17), where, the dilatancy angles with $\beta$ are chosen from 0 to 30 degrees as the expected values in practice.

Figure 14 shows comparisons of calculated values of the critical height of the retaining wall, $H_{cr}$, under the condition of various confining effect parameters and the various vertical spacings of geogrids. In the case of $\beta=0$ which reflects that $\psi=0$, the confining effect is not considered in the design method. The critical height, $H_{cr}$, increases with the decreases of the vertical spacing of geogrids and the increases of the confining effect parameter, $\beta$ which is directly linked with the dilatancy angles of reinforced soil mass. In the proposed design method, the critical height of wall, $H_{cr}$, increases by roughly 5-20% in the expected practical ranges of the vertical space of geogrids from 0.5m to 1.0m, depending on the magnitude of the confining effect parameter, $\beta$, which was linked with the dilatancy behaviours of the reinforced soil mass.

5.2 Examples of calculations using the proposed design method

Table 2 Design parameters used

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Unit weight of soil, $\gamma$</td>
<td>17.7 kN/m$^3$</td>
</tr>
<tr>
<td>Internal friction angle, $\phi$</td>
<td>30 degrees</td>
</tr>
<tr>
<td>Allowable tensile strength of geogrid, $T_A$</td>
<td>39.8 kN/m$^2$</td>
</tr>
<tr>
<td>Surcharge load, $q$</td>
<td>9.8 kN/m$^2$</td>
</tr>
<tr>
<td>Dilatancy angle, $\psi$</td>
<td>0, 10, 20, 30 degrees</td>
</tr>
</tbody>
</table>

Figure 14 Critical height of geogrid-reinforced wall, $H_{cr}$ against vertical space of geogrid at various dilatancy angles reflecting the confining effect
CONCLUSIONS

An evaluation method in which the reinforcing effects can be divided into tensile effect and confining effect is proposed related to the dilatancy behaviour of reinforced soil mass, in order to introduce the confining effect into design methods.

The mobilized confining effect was given by a function of the dilatancy rate, in which the key idea is in the assumption of the dissipated energy done by the unit volume of the soil mass. The proposed model indicated that the confining effect parameter, which defines the degree of the apparent incremental confining stress to the current normal stress on the expected sliding plain, was in the range of 0 to 0.3 against the supposed dilatancy angles from 0 to 30 degrees. The expected range had a good agreement with the experimental ones which have been reported by the authors.

The confining effect closely linked with the dilatancy angle of reinforced soil mass was introduced into the tie-back wedge method of geogrid-reinforced retaining wall based on Rankine’s active earth pressure theory. For instance, in the proposed design method, the critical height of the reinforced wall increased by roughly 5-20% in the expected practical ranges of the vertical space of geogrids from 0.5m to 1.0m, depending on the magnitude of the confining effect parameter, $\beta$, which was linked with the dilatancy behaviours of the reinforced soil mass.

REFERENCES


