Geosynthetics, J. Kuwano & J. Koseki (eds)
 ISBN 90 5966 044 7

Confining effect of geogrid-reinforced soil linked with soil dilatancy and its application to practical design method

Yasufuku, N. & Ochiai, H.
Department of Civil Engineering, Kyushu University, Japan

Kaneshige, M.
Division of Civil Engineer, Local Government of Yamaguchi Prefecture, Japan

Kawamura, T.
Department of Civil Engineering, Shinshu University, Japan

Keywords: geogrid reinforced soil, reinforcing effect, confining effect, soil dilatancy, practical design method

ABSTRACT: An evaluation method in which the reinforcing effect can be divided into tensile effect and confining effect is carefully discussed in relation to the dilatancy rate of reinforced soil mass. The mobilized confining effect is given by a function of the dilatancy rate and the internal friction angle at critical state. The effectiveness of the proposed equation is discussed by comparing with the experimental data obtained from the biaxial compression tests results. Then, the confining effect is introduced into a current design method, which can directly reflect the soil dilatancy and strength properties. It is found from the computed results on realistic steep slope reinforced embankment that the total amount of reinforcements may be reduced by 5 to 20(%).

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 (e.g., Tatsuoka et al. 1991). Further, as a good deal of evidence, a strong earthquake inflicted little damage to 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, 2004).

In this study, the reinforcing effects in geogrid reinforced soil are first briefly reviewed with the confining effect of reinforcement. Then, it is indicated that the mobilized confining effect is given by a function of the dilatancy rate, in which the basic idea lies in the dissipated of energy by unit volume of the reinforced soil mass. The confining effect is introduced into the practical design method in steep slope reinforced embankment, and the contributions of the confining effect are discussed on the stability of the slope.

2 REINFORCING EFFECT IN GEOGRID REINFORCED SOIL

In general, dealing with the tensile effect in geogrid reinforced soil, the relationship between the shear strength of reinforced soil, s_R, and the normal stress, σ_n, can be expressed as (e.g., Jewell and Wroth, 1987):

$$s_R = c + c_T + \sigma_n \tan \phi$$
$$= c + \frac{P_R}{A_S}(\sin\theta \tan\phi + \cos) + \sigma_n \tan\phi \quad (1)$$

where P_R is the mobilized tensile force of the reinforcement, A_S is the area of the sliding plane, ϕ is the internal friction angle of soil and θ is the angle between the reinforcement and the sliding plane. From the characteristics of Eq. (1), s_R does not depend on the normal stress on the sliding plane and it is equal to the increment of apparent cohesion of reinforced soil, c_T.

We have defined the confining effect, which will reflect an apparent incremental confining stress in reinforced soil mass, as an effect that is independent of 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

Fig. 1. "$\beta \tan \phi$" in Fig. 1 is an 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, it is considered that the shear strength of reinforced soil should be evaluated not as an increment of the internal friction angle, $\beta \tan \phi$, but as an increment of the normal stress, $\beta\sigma_n$, as follows (Ochiai et al.,1996, 1998):

$$s = c + \frac{P_R}{A_S}(\sin\theta\tan\phi + \cos\theta) + (1+\beta)\sigma_n \tan\phi \tag{2}$$

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

3 CONFINING EFFECT LINKED WITH SOIL DILATANCY

The confining effect of reinforced soil mass is supposed to be the effect of restriction of soil mass by the geogrid around it, and thus the confining stress around the geogrid apparently increases. 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 β in Eq. (2), the work equations applied to a simple shear test sample were discussed for reinforced and non-reinforced soil mass (Yasufuku et al., 2004). We shall suppose that a small shear stress increase $d\tau$ causes a shear deformation, so that the shear strain $d\gamma > 0$ 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 Fig. 2, we now can deduce the magnitude of the plastic work which is fed into the element and presumably dissipated such that

$$dW = \sigma_n d\varepsilon_y + \tau d\gamma_{xy} \tag{3}$$

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 β, is supposed to be given by

$$dW_r = \sigma_n \sqrt{(\tan\phi_c' d\gamma_{xy})^2 + (d\varepsilon_y)^2} \tag{4}$$

where ϕ_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, which is equivalent to an apparent incremental normal stress is generally mobilized by restricting the smooth

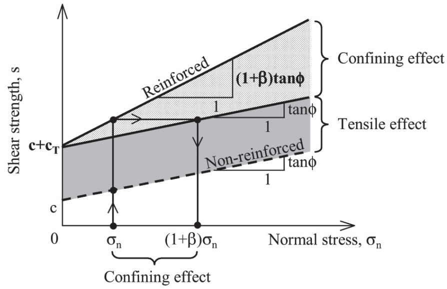

Figure 1. Relationship between shear strength and normal stress considering both of tensile and confining effects.

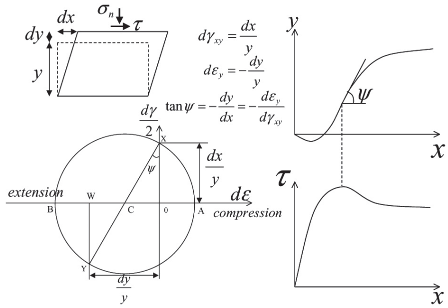

Figure 2. Stress-strain system in direct shearing with soil dilatancy.

movement of soils around the geogrid, it is reasonable to understand that the source of the confining effect is due to the restriction of soil dilative behaviour due to shearing. Assuming that the reinforced soil mass can be homogenized in average, it is believed that the alternative dissipated work in reinforced soils, which directly reflect the confining effect, can be expressed as

$$dW_r = (\sigma_n + \Delta\sigma_n)\sqrt{(\tan\phi_c' d\gamma_{xy})^2} \tag{5}$$

where, $\Delta\sigma_n$ is an apparent incremental normal stress corresponding to the confining effect. Furthermore when remembering that $\Delta\sigma_n$ is equal to $\beta\sigma_n$ as shown in Eq. (2), Eq. (5) is rewritten as

$$dW_r = (\sigma_n + \beta\sigma_n)\sqrt{(\tan\phi_c' d\gamma_{xy})^2} \tag{6}$$

After assuming that the dissipated work equations in Eq. (4) is equivalent to that in Eq. (6), and by merging Eq. (4) with Eq. (6), we can get the following equation:

$$\beta = \sqrt{1 + \left(\frac{d\varepsilon_y}{\tan\phi_c' d\gamma_{xy}}\right)^2} - 1 = \sqrt{1 + \left(\frac{\tan\psi}{\tan\phi_c'}\right)^2} - 1 \tag{7}$$

where, ψ is dilatancy angle which is defined as $\tan\psi = -d\varepsilon_y/d\gamma_{xy}$ shown in Fig. 2. For simplicity, we assume that the value of $\tan\psi$ can be determined from the dilatancy angle at peak state in the shearing test for non-reinforced soil. Figure 3 shows the characteristics of β against the internal friction angles ϕ_c' at residual strength state for the simple shear under the plain strain conditions in terms of dilatancy angle ψ, which are calculated by Eq. (7). In addition, experimental values of β are also depicted in this figure, which is obtained by biaxial compression test results for the reinforced aluminum rods samples. It is characterized that the values of β decrease with the increasing friction angle and increase with the increasing dilatancy angle. It is also found that the predicted β gives a roughly good agreement with the experimental results. The detail of test procedures and arrangements of the data obtained has been reported by Kaneshige et al., (2004).

4 INTRODUCTION OF CONFINING EFFECT INTO SAFETY FACTOR IN REINFORCED STEEP SLOPE EMBANKMENTS

According to Japanese design guideline in 1993, safety factor in reinforced state of geotextile reinforced steep slope embankment has been expressed as:

$$F_s = \frac{M_R + \Delta M_R}{M_D} \tag{8}$$

where, M_R: resistance moment of soil mass in reinforced state, M_D: sliding moment of soil mass in reinforced state, ΔM_R: resistant moment due to tensile force of reinforcement, in which the concrete formula is derived by a slip circle method. When considering the confining effect mentioned above, ΔM_R in current guideline is given by:

$$\Delta M_R = R\sum \{\beta W \cos\theta \tan\phi + (T\cos\theta + T\sin\theta\tan\phi)\} \tag{9}$$

where, W: weight of each sliding mass element. The first term reflects the confining effect in the resistant moment. This formula is same, except that β is given by Eq. (7), as one which has been reported by Yasufuku et al., (2002). It is pointed out that, based on the design procedure including the confining effect, the required total tensile forces in design is reduced, and the amount of reduction is expressed as a function of the first term, $\sum\beta W\cos\theta\tan\phi$. It means that the reasonable estimation of the confining effect in practical design is important to establish an economical and rational design procedure in geotextile reinforced embankments.

Figure 4 shows the maximum dilatancy angles ψ against the confining pressure for a typical silica sand with loose, medium dense and dense states in triaxial compression tests. It is clear that ψ increases with the increasing relative density and also gradually decreases with the increasing confining pressure. It is noted that the friction angle of this sand at critical state is roughly 34 degrees. Figure 5 shows the

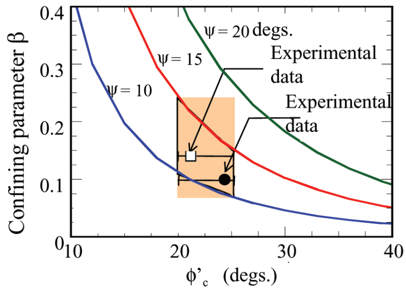

Figure 3. Comparison of predicted and experimental confining effect associated with frictional and dilatancy.

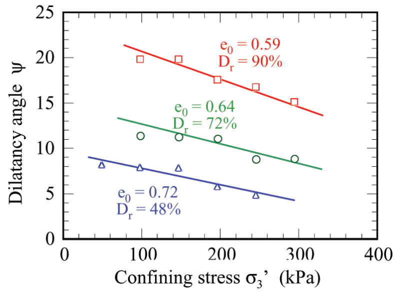

Figure 4. Relationships between dilatancy angle and confining stresses of typical silica sand with various relative density.

expected confining parameters in reinforced soil wall, which is backfilled by sandy soils shown in Fig. 4, against the calculated depth from the surface. It is clear that the mobilized confining effect is strongly dependent on the relative density, in turn, it becomes greater when the relative density becomes larger. In addition, the confining parameter becomes smaller when the depth from the surface becomes deeper. According to the results for dense sand, the confining parameters changes in the range from 0.1 to 0.2 at the height of reinforced wall from 2 m to 12 m.

In order to indicate a typical example for a reinforced steep slope embankment designed by introducing the confining effect presented here, average β value of 0.15 is used, assuming a well compacted embankment based on the result in Fig. 5. The cross section of the embankment slope and the design condition are shown in Fig. 6. Three kinds of heights and slopes are selected in calculations and also the factor of safety is fixed as 1.2. Figure 7 shows the calculated reduction rate of reinforcements against the height of embankment. It is found that, when introducing the recommended confining effect, the amount of reinforcement can be reduced which ranges from 5% to 20%, depending on the height of embankment.

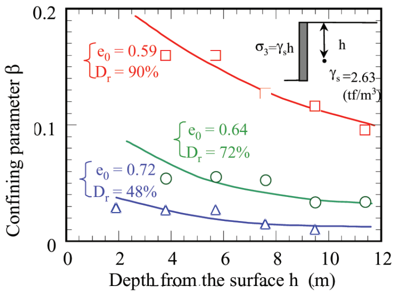

Figure 5. Predicted confining effect parameters related to the depth from the ground surface.

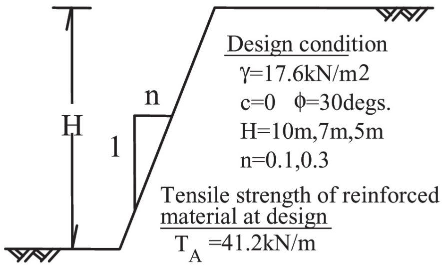

Figure 6. Cross section and condition in design.

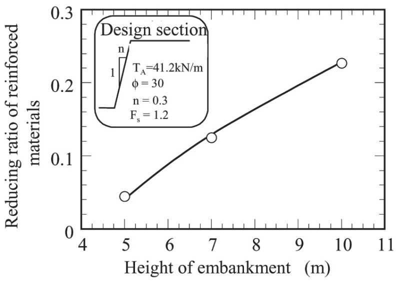

Figure 7. Expected reducing ratio of reinforced materials due to confining effect.

5 CONCLUSIONS

In this study, the confining effect in reinforced soil is discussed and linked with the soil dilatancy. The parameter which reflects the confining effect is given by a function of internal friction angle at critical state and the maximum dilatancy angle, which is expected to change in the ranges from roughly 0.1 to 0.2 for well compacted embankment. It is found from the computed results in steep slope reinforced embankment that the total amount of reinforcement may be reduced by 5 up to 20(%), when the confining effect is properly reflected in the design.

ACKNOWLEDGEMENTS

The authors wish to extend special thanks to: Mr. M. Nakashima of Kyushu university for his support.

REFERENCES

Jewell, R.A. and Wroth, C.P. (1987). “Direct shear test on reinforced sand”, Geotechnique, 37, No.1 pp. 53-68.

Kaneshige, M., Ochiai, H., Yasufuku, N. and Kawamura, T. (2004). “Confining effect in georid-reinforced soil on consideration of dilatancycharacteristics”, Geosynthetics Engineering Jounal, Vol. 19, pp. 217-224 (in Japanese).

Kawamura, T. Ochiai, H. Yasufuku, N. and Hirai, T. (1999a). “Introduction of Confining Effect of Geogrid-Reinforced Soil into Design Method”, Geosynthetics Engineering Jounal, Vol. 14, pp. 211-220 (in Japanese).

Ochiai, H., Yasufuku, N. Yamaji, T., Guang-Li Xu and Hirai, T. (1996). “Experimental Evaluation of Reinforcement in Geogrid-soil Structure”, Proc. of Int. Symp. on Earth Reinforcement (IS Kyushu 96), Vol. 1, pp. 249-254.

Ochiai, H., Yasufuku, N. Kawamura, T., Hirai, T. and Yamaji, T. (1988). “Effect of End-Restraint in Geogrid-Soil Structures”, Proc. of 6th Int. Conf. on Geosynthetics, pp. 545-550.

Public Works Research Center, (1993). “Design and construction manual of geotextile-reinforced embankment”, Technical memorandum of Pubric Works Research Center, pp. 404 (in Japanese).

Tatsuoka, F., Koseki, J. and Tateyama, M. (1996). “Performance of reinforced soil structures during the 1995 Hyogo-ken Nanbu Earthquake”, Proc. of Int. Symp. on Earth Reinforcement (IS Kyushu 96), Fukuoka, Balkema, Vol. 1 pp. 973-1008.

Yasufuku, N., Ochiai, H., Omine, K., Ninomiya, Y. and Kawamura, T. (2002). “Evaluation of Confining Effect in Geogrid-Reinforced Retaining Wall related to the Practical Application”, Proc. of the 7th International Conference on Geosynthetics, Vol. 4, pp. 1289-1294.

Yasufuku, N., Ochiai, H., Kaneshige, M., Kawamura, T. and HIrai, T. (2004). “Confining Effect in Geogrid-Reinforced soil related to soil dilatancy”, Proc. of GeoAsia04, Seoul, pp. 385-394.