Bearing Capacity of Geogrid Reinforced Grounds

J. Otani
Kumamoto University, Japan

H. Ochiai
Kyushu University, Fukuoka, Japan

Y. Miyata
National Defense Academy, Japan

ABSTRACT: This paper describes bearing capacity analysis of geogrid reinforced foundation ground using rigid plastic finite element method. In order to take into account the reinforcing effect in the analysis, a composite type model including geogrid and the surrounding soil is proposed. The effectiveness of proposed method is firstly verified by conducting comparative studies with both theoretical and test results. Then, a series of parametric study by changing the reinforcement conditions, i.e. length, depth and strength of geogrid, is conducted. Finally, a simple design chart based on the analysis results is proposed for the bearing capacity of geogrid reinforced soft foundation ground.

1 INTRODUCTION

Most design methods for earth reinforcement structures are based on limit equilibrium method in which the assumptions of the location and shape of failure surface are the most important task. And besides, the one who designs these structures requires his intuition and ample experiments in order to obtain a good approximate solutions. A bearing capacity of geogrid reinforced foundation ground usually depends on the width and length of the embedded geogrids as well as the number of geogrid layers in the ground. A rigid plastic finite element technique is one of the method which the limit load is obtained without setting up the location and shape of the failure surface.

The objective of the paper is to analyze the bearing capacity of geogrid reinforced grounds using rigid plastic finite element method. First of all, the loading test of model reinforced embankment is analyzed in order to verify the proposed analysis method. Then, a series of parametric studies is conducted by changing the length and width of the geogrid in the ground. Thus, the effect of the reinforcement on the bearing capacity of soft ground is evaluated.

2 METHOD OF ANALYSIS AND REINFORCED SOIL MODEL

2.1 Method of analysis

Tamura et al.(1984) have formulated a rigid plastic finite element method(RPFEM) to the stability analysis in geotechnical engineering field, and this method is used for calculating bearing capacity of geogrid reinforced ground.

The formulation of RPFEM is summarized briefly on the basis of the references by Tamura et al.(1984) and Otani et al.(1994). A uniform strip load is taken into consideration on this formulation. This is based on the upper bound theorem : If a compatible mechanism of plastic deformation is assumed, which satisfies the boundary condition of the displacement; then load determined by equating the rate at which the external force do work to the rate of internal dissipation will be either higher or equal to the actual limit load.

This theorem is formulated as

$$\overline{\rho} \int_{S_\sigma} T_i \dot{u}_i \, dS = \int D(\dot{\varepsilon}_{ij}) \, dV \tag{1}$$

with

$$\rho^* \leq \overline{\rho} \tag{2}$$

where ρ^* and $\overline{\rho}$ are load factors of limit state and upper bound. T_i and $\dot{u}_i$ are traction force and velocity, respectively, and $\dot{\varepsilon}_{ij}$ is a plastic strain rate. The left hand side of Eq.(1) shows an external work, while the right hand side is an internal dissipation by plastic strain energy.

The formulation of RPFEM is finally expressed with the conditions of no volumetric strain and constraint for the work at the limit state:

$$\left. \begin{array}{l} \text{Minimize} \int D(\dot{\varepsilon}_{ij}) \, dV \\[2mm] \qquad \text{subject to the constraint} \\[2mm] \int_{S_\sigma} T_i \dot{u}_i \, dS = 1 \text{ and } \dot{\varepsilon}_v = 0 \end{array} \right\} \tag{3}$$

This is exactly a constrained problem in variational method and is usually solved by using Lagrange multipliers. Thus, with using finite element technique, Eq.(3) is equivalent to the following matrix equations :

$$\left. \begin{array}{r} \int_v \mathbf{B}^T \mathbf{s} \ dV + \mathbf{L}^T \lambda = \mu \ \mathbf{F} \\ \mathbf{F}^T \mathbf{u} = 1 \\ \mathbf{L}\mathbf{u} = 0 \end{array} \right\} \tag{4}$$

where
  $\mathbf{B}$: matrix defined such as $\dot{\boldsymbol{\varepsilon}} = \mathbf{B}\dot{\mathbf{u}}$
  $\mathbf{L}$: matrix defined such as $\dot{\mathbf{v}} = \mathbf{L}\dot{\mathbf{u}}$
  $\mathbf{s}$: vector of deviatoric stresses sij
  $\mathbf{F}$: vector of nodal forces
  $\dot{\mathbf{u}}$: vector of nodal velocities
  $\dot{\mathbf{v}}$: vector of rates of volumetric change
  λ : indeterminate isotropic stress
      (Lagrange multiplier)
  μ : load factor of external force
      (Lagrange multiplier)

For the constitutive equations, Von-Mises and Drucker-Prager models are used for clayey and sandy ground, respectively. Eq.(4) is solved iteratively by replacing $\dot{\mathbf{u}}$ to $\dot{\mathbf{u}} + \alpha \ \Delta\dot{\mathbf{u}}$ using Newton-Raphson method because Eq.(4) has nonlinear characteristic where α is a convergence parameter. Note that $\alpha = 0.2$ for unreinforced case and $\alpha = 0.01 \sim 0.05$ for reinforced one are used here in this study. Finally, the both Lagrange multipliers λ and μ, and the velocity field $\dot{\mathbf{u}}$ are obtained at the limit state. The bearing capacity is evaluated by the load factor μ multiplied by the strength of soil C_u and the resultant velocity field may be considered to be a plastic flow in the failure zone.

2.2 Reinforced soil model (Otani et al., 1994)

There are two basic ideas of modeling the interaction behavior between soil and geogrid. One is that the soil and geogrid are individually modeled, and the other is that the geogrid and the surrounding soil are unified in the model. When the former is used to solve nonlinear differential equation, the large difference of the stiffness between soil and geogrid is very crucial for the conversion of the solution by Newton-Raphson method. The sand seam is usually constructed around the geogrid in the clayey ground for the purpose of increasing the friction between the geogrid and the ground. Here in the paper, the latter is used as the model of earth reinforcement, in which the geogrid and the surrounding sand seams are assumed to be a unique material as an earth reinforcement. The equivalent cohesion, C_{uR} for this earth reinforcement is evaluated by using following equation:

$$C_{uR} = \frac{T \ \sqrt{K_p}}{2 \ \Delta H} \tag{5}$$

where T and K_p are tensile strength of the geogrid and passive earth pressure coefficient of sand, respectively.

ΔH is a spacing of each two geogrids. This equation was originally developed by Schlosser et al.(1973) based on the laboratory test results. The assumption on this equation is that the average strain at the direction of the geogrid surface in sand is always equal to the elongation of the geogrid, so that the stress increment due to the reinforcement is obtained from the tensile strength of the geogrid at the limit state with the confining stress by passive earth pressure. It is noted that the relative displacement between geogrid and sand is neglected in this model.

3 ANALYSIS OF MODEL LOADING TEST

3.1 Summary of loading test

Fig.1 shows a sketch of test embankment made by sand in the soil box. Three layers of geogrid were placed in this embankment. The vertical pressure by loading plate was applied by controlling the loading speed, and the resultant settlement of the plate was measured using dial gauge. In the test, the effect of different geogrid strength on the bearing capacity of the ground was observed. Here, the tensile strength of geogrids was changed by means of cutting some of the libs in the geogrid.

Fig.1 Model embankment in loading test.

Fig.2 Results of test and analysis.

3.2 Test and analysis results

Fig.2 shows a test results of vertical pressure and settlement relations for two different tensile strengths (T = 40 and 130 kN/m). The test result for the case of without reinforcement was also included in Fig.2 for the purpose of the comparison with the reinforced case. In the analysis, in order to make comparisons quantitatively, the soil parameters such as cohesion c and friction angle ϕ are determined by back analysis using test result of bearing capacity for unreinforced case. These values are shown in Fig.3 with the resultant plastic flow. The case of reinforced ground is also analyzed using these parameters with proposed reinforced soil model and the result for the case of T = 40 kN/m is shown in Fig.4. As clearly shown from these results, the area of existing plastic flow for the reinforced case is extended widely towards the bottom of embankment comparing that for the unreinforced case, and the resultant bearing capacity is improved(q_R = 52 kN/m^2). These bearing capacities are also plotted in Fig.2 with that of the other test case (T = 130 kN/m). Comparing these analysis results with the test results, the analysis results are fairly close to those of the residual values in the test and this is acceptable as far as the plastic theory is concerned. It may be, therefore, concluded that the proposed analysis method is capable of expressing the test results.

q_0 = 15 kN/m^2 C = 0.1 kN/m^2 ϕ = 30°
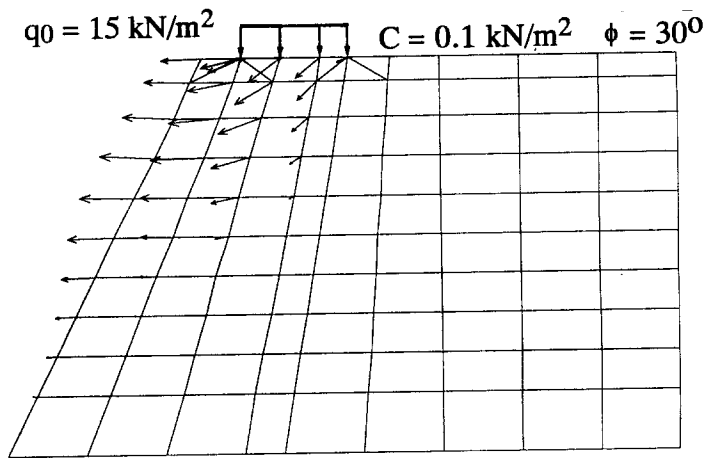
Fig.3 Resultant soil parameters and plastic flow for the unreinforced case.

q_R = 52 kN/m^2 T = 40 kN/m
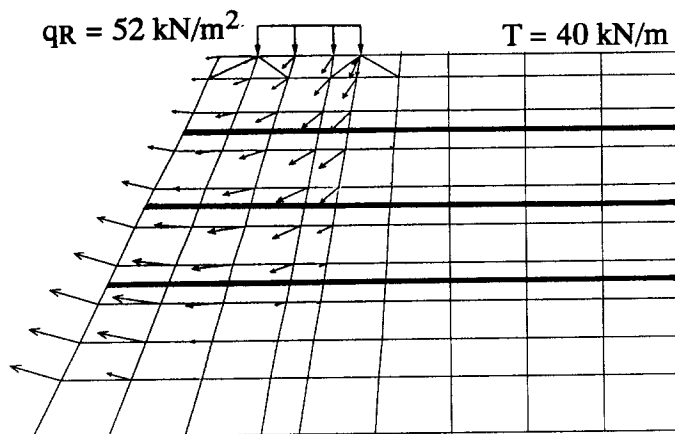
Fig.4 Bearing capacity and plastic flow for the model test (reinforced case T = 40 kN/m)

4 BEARING CAPACITY OF REINFORCED SOFT GROUND

4.1 Unreinforced foundation ground

In order to verify the rigid plastic finite element method, the bearing capacity of the ground without the reinforcement is firstly solved. The improvement on the accuracy of the solution is also examined by using the treatment of the singular point at the edge of the load.

Fig.5 shows the result of plastic flow by vector description and their bearing capacity. The technique on the treatment of singularity used here was firstly used by Levy et al. (1971) for the analysis on fracture mechanics. The capacity by using the singularity treatment is clearly improved and is very close to rigorous solution(q_u = 5.14Cu).

Thus, it is concluded that the RPFEM is capable of obtaining the fairly good solution. The singular treatment as shown in Fig.5 is also used for the case of the reinforced foundation ground.

4.2 Geogrid foundation ground

Using the model of the reinforced soil described above, a series of rigid plastic finite element analysis for geogrid foundation ground is conducted for various cases of different depth, D and the length, L of geogrid. The parameters for the earth reinforcement model shown in Eq.(5) are:
•Rankine's passive earth pressure coefficient of sand:

K_p = (1 + sinϕ)/(1 - sinϕ) (ϕ = 30°) and
•tensile strength of geogrid: T = 550 kN/m.

Fig.6 shows the plastic flows and the bearing capacities for the case of L/B = 1.0 and 2.0 with the depth of D = 1/10B where B is the width of the applied load. And Fig.7 shows the same results for the depth of two times as large as those shown in Fig.6. Although the flow for the case of unreinforced ground shown in Fig.5 was concentrated around the edge of the load, this for the case of geogrid foundation ground is distributed widely beneath the load. As a results, the bearing capacity is increased. As far as these results are concerned, the bearing capacity is larger for long and deep reinforcements.

In order to investigate the reinforcing effects on various reinforcement conditions, one-layer geogrid foundation ground is analyzed with changing its length and depth as well as the tensile strength of geogrid. Fig.8 shows the results, in which the resultant reinforcing effect (q_{uR}/q_u - 1) is plotted against the depth of the reinforcement D/B for various lengths of the geogrid L/B. It is noted that the reinforcing effect is evaluated by an increment of the bearing capacity for that of unreinforced ground, q_u. All the curves have a maximum reinforcing effect at some depth, and this depth is changed with different length and tensile strength of geogrid.

Ochiai et al. (1992) have conducted a series of model loading tests on geogrid foundation ground and they have concluded that there is an optimum depth in order to mobilize the maximum reinforcing effect, and the results here in the analysis represent these test results.
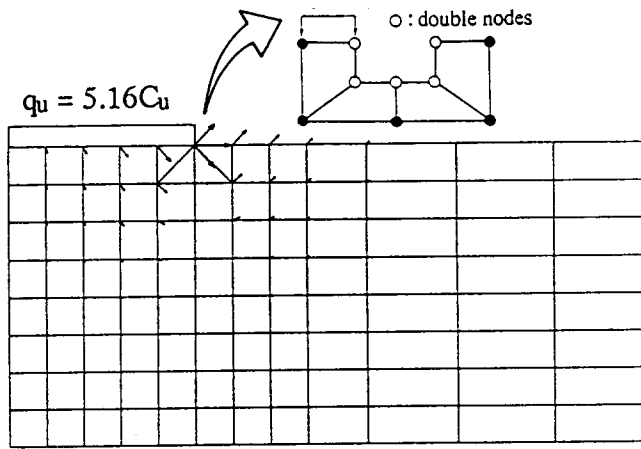
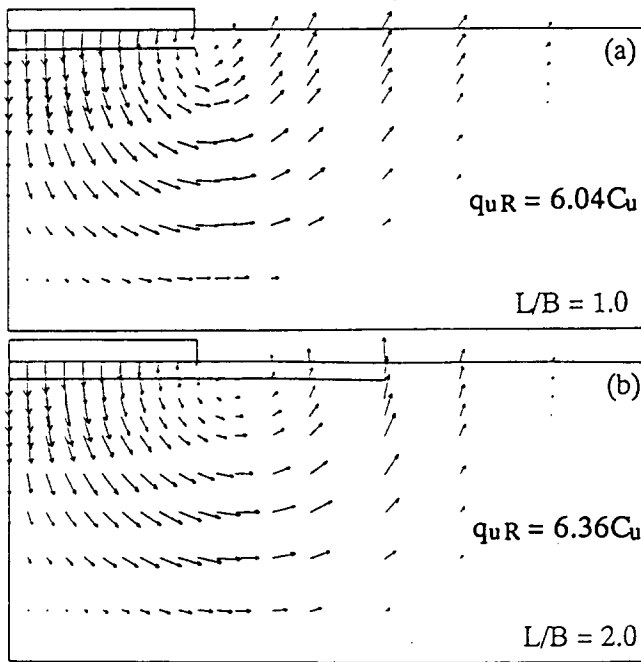Fig.5 Bearing capacity and plastic flow for the case of unreinforced case

Fig.6 Bearing capacity and plastic flow for the case of D = 1/10 B.
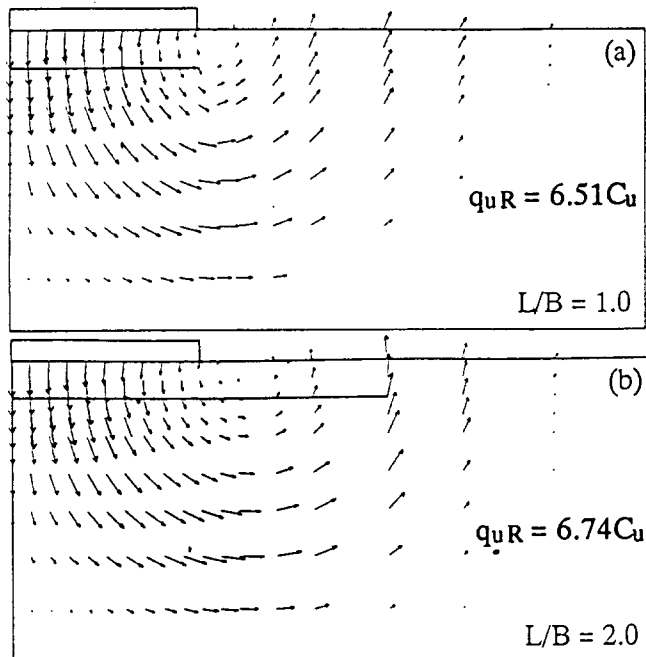

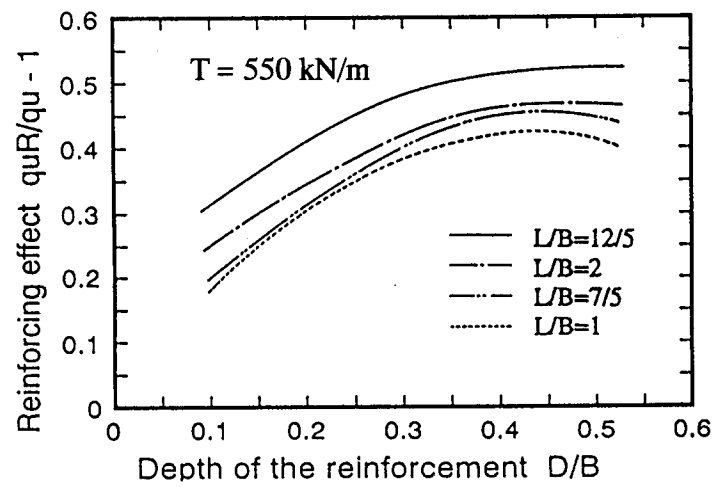Fig.7 Bearing capacity and plastic flow for the case of D = 1/5 B.

Fig.8 Reinforcing effects of geogrid foundation ground.

5 CONCLUSIONS

The bearing capacity for geogrid foundation ground was analyzed by using rigid plastic finite element method. In order to model the reinforced soil by geogrid, the idea of the unification between geogrid and the surrounding soils was introduced in the analysis.

The conclusions drawn from this study are summarized as follows:

(1) The effectiveness of the method(RPFEM) proposed in this study was evaluated by conducting the analysis of model loading test.

(2) The treatment of singular point at the edge of the load is effective in order to obtain a good agreement with Prandtl's explicit solution.

(3) The bearing capacity of geogrid foundation ground is increased as the depth and length of the reinforcement are increased, but there exists an optimum depth in order to mobilize the maximum reinforcing effect.

(4) As a results, Fig.8 shown in this paper may be a simple design chart for calculating bearing capacity of geogrid reinforced foundation ground.

REFERENCES

Levy, N., Marcal, P.V., Ostergren, W.J. and Rice, J.R. (1971) Small scale yielding near a crack in plane strain: a finite element analysis. International Journal of Fracture Mechanics, Vol.7, No.2, pp.143 - 156.

Ochiai, H., Hayashi,S., Yang, J. and Otani, J. (1992) Reinforcing effects of foundation ground with geogrids. Proc. of Int. Symposium on Soil Improvement and Pile Foundation, Nanjing China, pp.310-315.

Otani, J. and Ochiai, H. (1994) Bearing capacity analysis of reinforced foundation by geosynthetics, 8th Int. Conf. on Numerical Method in Geomechanics, May, West Virginia, U.S.A.

Schlosser, F. and Long, N.T. (1973) Etude dn comportement du materiaux Terre Armee. Annales de L' Institut Technique de Batiment et des Travaux Public, Supplement No.304, Series Materiaux No.45.

Tamura, T., Kobayashi, S. and Sumi, T. (1984) Limit analysis of soil structure by rigid plastic finite element method. S and F,Vol.24, No.1, pp.34 - 42.