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Numerical analysis for bearing capacity improvement of soft clay ground reinforced with geotextiles

Y.Tanabashi
Nagasaki University, Japan

K.Takaoka
Mitsui Industrial Material Co., Ltd, Tokyo, Japan

K.Kamuro
Construction Technology Inst., Fukuoka, Japan

J.Nishimura
Mitsui Petrochemical Co., Ltd, Tokyo, Japan

K.Hirao
Nishinippon Institute of Technology, Fukuoka, Japan

K.Yasuhara
Ibaraki University, Hitachi, Japan

ABSTRACT: This paper proposes a new analytical procedure for estimating the effect of bearing capacity improvement and the deformation characteristics of soft clay ground reinforced with geotextiles. The proposed finite element technique is organized of rational modeling for each soil and reinforcement with inclusion of the interaction between soil and reinforcement. Comparing the analytical results and the observed values of the laboratory model footing tests carried out by Hirao and Yasuhara(1991),the applicability of the proposed technique for estimating the bearing capacity was proved. A numerical analysis has been carried out under the combination of the ratio of loading width to clay layer thickness,kind of reinforcements and supporting manner of the geotextile's end. Based on the results of analysis, a reasonable method for evaluating the bearing capacity improvement of the soft clay ground reinforced with geotextiles has been proposed.

1. INTRODUCTION

Soft alluvial clay deposits, which are unable to insure trafficability, are very popular in Japan. The earth spreading work with polypropylene sheet or plastic net has been developed from the late of 1960's in Japan for security of trafficability.

Recently, the earth spreading execution with geotextiles has gradually increased. However,a rational design for the earth spreading work with reinforced materials has not been established, because of the variety of the ground condition,the tensile stress within geotextiles and the thickness of covering layer according with execution procedure.

Finite element analyses on this subject have been proposed by Kutara et al.(1985), Yamaoka et al.(1986) and Ochiai et al. (1988). However, they adopted a linear or nonlinear elastic model as the soft clay ground.It is obvious that those models are unable to represent the critical mode of the soil behavior accompanied with extreme settlement, heaving and plastic lateral flow of ground.

This paper proposes a new analytical procedure for the judgment of the effectiveness of geotextiles as the reinforced materials, and for the establishment of new design.

The proposed finite element technique is organized of rational modeling for each soil and geotextile with inclusion of the interaction between soil and geotextile. This technique can represent the critical mode of the soil behavior and also takes into consideration the frictional force on the boundaries between soil and geotextile.

2. ANALYTICAL METHOD

2.1 Adopted models

The proposed finite element technique is organized of rational modeling for soil,geotextile and the interaction between those materials.

Each model is as follows.

(1) Soil model is the elasto-viscoplastic model proposed by Sekiguchi and Ohta(1979) which is able to represent the critical mode of the soil behavior.

An yield function of the constitutive model is followed by:

$$f=\frac{\lambda-\kappa}{1+e_0}\ln\frac{p}{p_0}+D\eta^{*} \qquad (1)$$

Visco-plastic strain rate is:

$$\dot{\varepsilon}_{ij}^{p} = [\{1-\exp(-\frac{V^p}{\alpha})\}(\frac{\lambda-\kappa}{1+e_0}\frac{\dot{p}}{p}+D\dot{\eta}^{*}) + \dot{V}_0\exp(\frac{f-V^p}{\alpha})\}\frac{\partial f}{\partial \sigma_{ij}}/\frac{\partial f}{\partial p} \quad (2)$$

where, $p=\sigma_{ii}$, $q=\sqrt{3/2(s_{ij}s_{ij})}$

$$\eta = p/q,\ \eta_{ij}=s_{ij}/p,$$

$$\eta^{*}=\sqrt{3/2(\eta_{ij}-\eta_{ij0})(\eta_{ij}-\eta_{ij0})} \quad (3)$$

Most of the parameters in above equations are explained in Table.1.

(2) A geotextile model consist of a plane truss element with pin connections, which is able to take into consideration the nonlinearity of tensile stress-strain curves, proposed by Yamaoka et al.(1986). Although a geotextile has a slight bending stiffness, a plane truss element is adopted as a geotextile model because of its higher resistance against tensile stresses.

The tensile stress-strain curve of the geogrid shown in Fig.1 is approximated the third polynomial eq.(4).

$$\sigma = C_1\varepsilon + C_2\varepsilon^2 + C_3\varepsilon^3 \quad (4)$$

where, σ is tensile stress and ε is extensional strain. The tangential Young's modulus E_i (eq.(4)) can be obtained as eq.(5).

$$E_i = \partial\sigma/\partial\varepsilon = C_1 + 2C_2\varepsilon + 3C_3\varepsilon^2 \quad (5)$$

(3) The interaction model between soil and geotextile is a Goodman's joint element considering deformation dependency

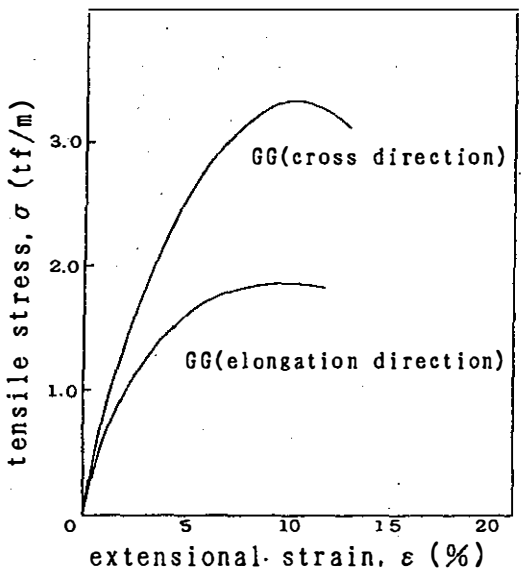


Fig.1 Stress-strain curves of geotextile

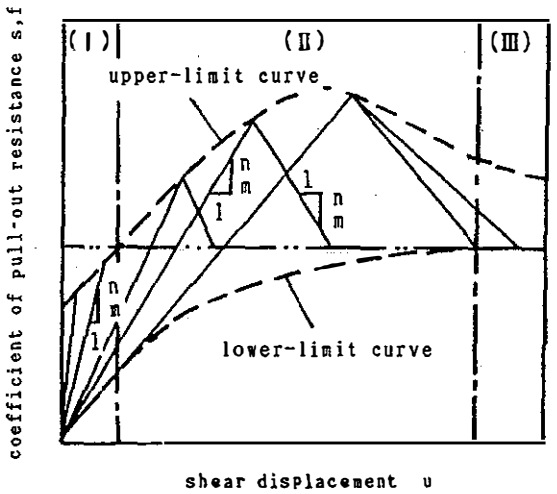


Fig.2 Determination of shear stiffness of joint element

of pull-out resistance based on the results of Hayashi et al.(1985). The pull-out resistance is changed with increasing the shear displacement. Fig.2 shows the mobilizing process of the pull-out resistance and its range. Fig.2 are used to determine the varying shear stiffness k_s of the joint element with shear displacement mobilized.

2.2 Analytical Procedure

The finite element program "DACSAR" developed by Ohta and Iizuka(1985), which is able to treat the coupling problems of soil skeleton and pore water, is used.
The analytical procedure for this study is as follows:

1) Analysis of the soil behavior and the bearing capacity of the model footing test is made.

2) The iteration process of the parameters determination is made to fit the analytical values with the observed values in the model tests within the range of the parameters.

3) After fixing the parameters, in -situ footing tests are performed with numerical simulation.

4) Extracting the results of the numerical simulation, data is offered to the establishment of the design for earth spreading work on the soft clay ground reinforced with geotextiles.

3. ANALYTICAL CONDITION

3.1 Outline of the model footing test

The outline of the model footing tests carried out by Hirao and Yasuhara(1991) is

described in detail in their literature. The model clay ground was made with 40 cm and 80 cm depth whose ratio of the loading width (B=10cm) to the clay layer thickness(D),D/B are 4.0 and 8.0 respectively. Index properties of the clay are: specific gravity G_s=2.62, initial water content w_i=130 %, liquid limit w_L = 107% and plasticity index PI=66. Applying incremental load of 0.01 kgf/cm^2 for 15 minutes.One of the geotextiles is a geogrid with opening 28 mm 40 mm (named GG hereafter). From the third polynomial approximation eq.(4), the parameters of geotextile GG obtained are following.

C_1 =660, C_2 = - 8800, C_3 = 40000

3.2 Analytical condition of the model test

An example of the finite element mesh for D/B=8.0 is shown in Fig.3.Both mechanical and hydraulic boundary conditions are also designated in Fig.3.

Numerical analysis has done with free end of the geotextile for restrained condition, and also under the plane strain condition.

The initial parameters of the model clay ground are determined with only plasticity index,PI following the determination flow chart proposed by Iizuka & Ohta(1987). The final fixed values of the parameters for the following numerical simulation are determined as in Table.1.

4. ANALYTICAL RESULTS

Fig.4 shows a comparison of the model ground deformation between analytical results and measured values for the case of D/B=8.0 and load intensity, p =0.0655 kgf/cm^2. The calculation values can represent the effectiveness of the geogrid as a reinforcement, and also sufficiently estimate the observed values of maximum settlement and heaving both with and without reinforcement. For the unreinforced case,the lateral displacement of the ground surface has a tendency to move outward. On the other hand, for the

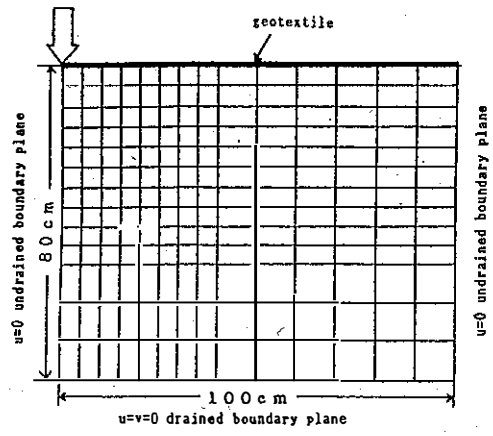
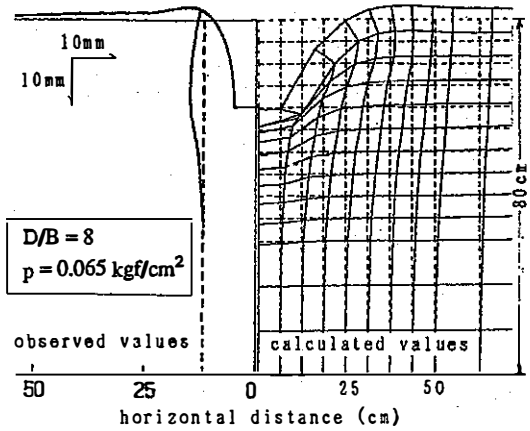


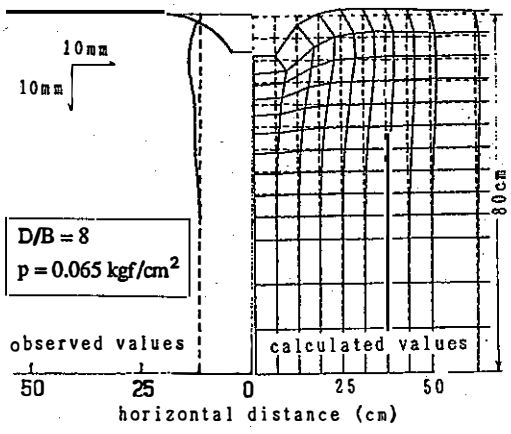
Fig.3 Finite element meshes for D/B=8.0

Table.1 Input values of the parameters for soil and geotextiles

Ground Material Properties	
D : coefficient of dilatancy	0.0477
Λ : irreversibility reatio=1-κ/λ	0.506
M : critical state parameter	0.886
ν' : effective Poisson ratio	0.4176
k_{xo}/γ_w : coefficient of permeability of x direction	0.354～7.51x10^{-4} cm/min
k_{yo}/γ_w : coefficient of permeability of y direction	0.354～7.51x10^{-4} cm/min
σ_{vo}' : pre-consolidation pressure	0.0014～0.0431 kgf/cm^2
K_o : coefficient of earth pressure at rest	0.717
σ_{vi}' : effective overburden pressure	0.0014～0.0431 kgf/cm^2
K_i : coefficient of earth pressure at rest in-situ	0.216～0.630
α : coefficient of secondary compression	7.51x10^{-3}
v_o : initial volumetric strain rate	2.88x10^{-8}～2.59x10^{-6}
λ : compression index	0.317
e_o : void ratio (at pre-consolidation)	2.80



(a) Unreinforced case



(b) Reinforced case

Fig.4 Ground deformation in comparison with analytical results and measured values

reinforced case, the lateral displacement of the ground surface is obviously restrained with geogrid. the calculated values can explain the above mentioned tendency.

Fig.5 shows a comparison of the load-settlement curves observed at the center of the load with the analytical results for the case of D/B= 8.0 both with and without geogrid.

It is clear that the calculation values have proved the settlement restrained effect of the geogrid, and also reasonably estimate the observed p versus S curves. The bearing capacities can be determined from the intersection point of two tangential lines of both initial and ultimate region in p versus S curves. The bearing capacity without geogrid, p_{yun}, and with geogrid, pyr and its increase ratio, p_{yr} / p_{yun}, obtained from Fig.5, are shown in Table.2.

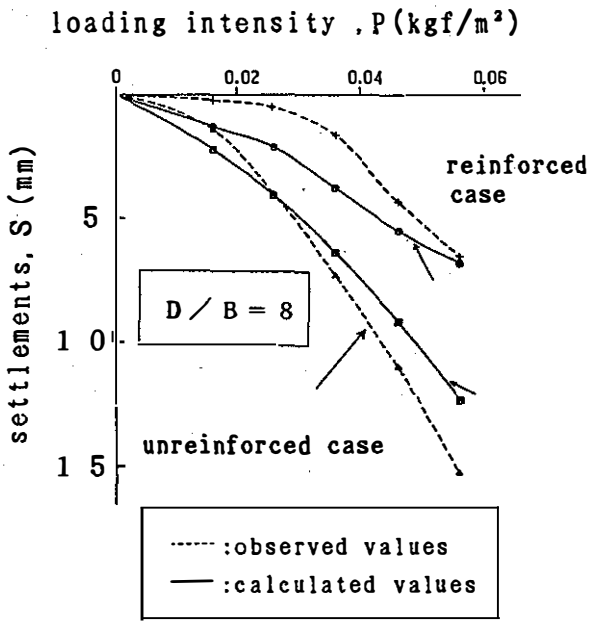


Fig.5 Load intensity,p versus settlement,S curves for D/B=8.0

Table.2 Comparison with observed and calculated bearing capacity

	Observed value	Calculated value
reinforced case Pyn(kgf/cm²)	0.032	0.030
unreinforced case Pyun(kgf/cm²)	0.0253	0.0247
the increase ratio Pyr/Prun	1.265	1.215

The values in Table 2 almost coincide with both observed and calculated values, and also indicate that the bearing capacity in reinforced case is improved by 22% to 27% higher than that in unreinforced case. Therefore, it may be concluded that the proposed numerical analysis can estimate the deformation and the bearing capacity improvement of the soft clay ground reinforced with geotextile.

5. NUMERICAL SIMULATION OF IN-SITU FOOTING TEST

5.1 Analytical conditions

Most of the analytical conditions of the in-situ footing test are the same of that for the model footing test.

The numerical simulation has done under the following conditions.

The footing width,B is 3.0 m and the clay layer thickness,D is 2, 4, 8, 12, 16 m,which is correspondence to D/B = 0.67, 1.33, 2.67, 4.00, 5.33 respectably.

Reinforcements used are the geogrid (GG), and a polymer net with a 15 mm netting opening and a 6.9kN/m yield strength (called PN hereafter). The geotextile parameters of PN are C_1 =111, C_2 = -150, C_3 = 2564.

A load is supplied by separating 6 stages with each increment of 1.6 tf/m^2 for 1 day.

5.2 Numerical results

The ratio of the footing width to the clay layer thickness, D/B versus both maximum settlement, Smax and heaving,Hmax curves as indices of the deformation of the soft clay ground are shown in Figs 6 and 7. Figs 6 and 7 indicate that both maximum

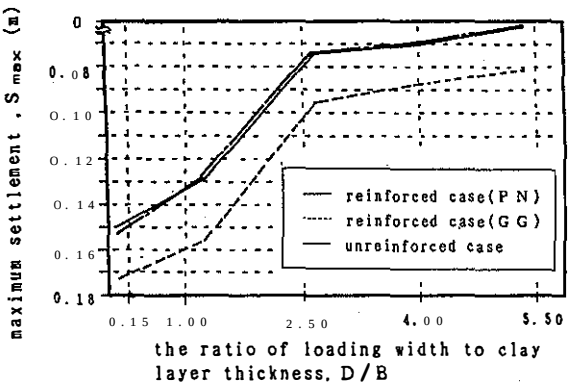


Fig.6 Effect of the D/B ratio on maximum settlement

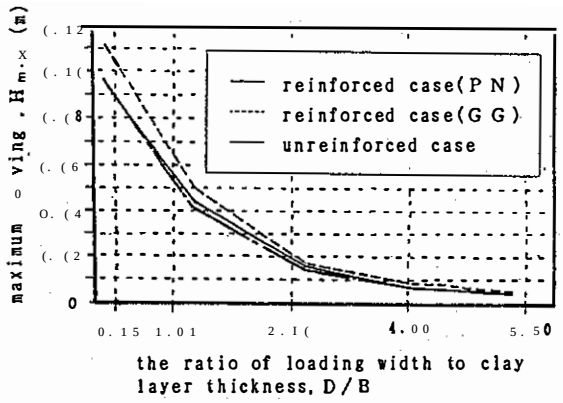


Fig.7 Effect of the D/B ratio on the maximum heaving

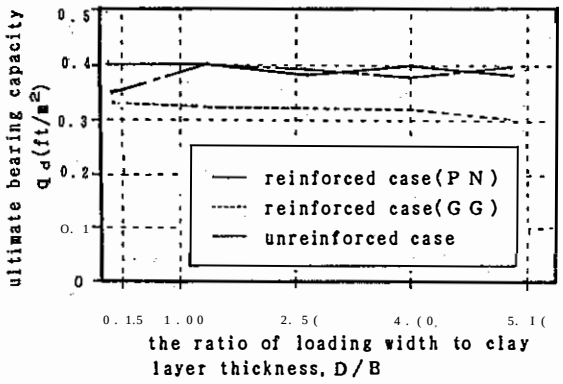


Fig.8 Effect of the D/B ratio on the bearing capacity

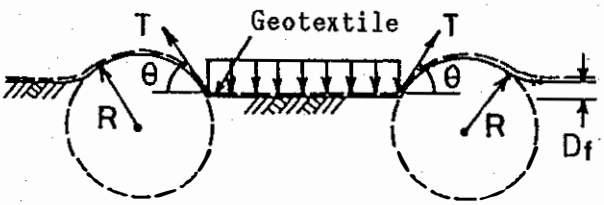


Fig.9 Schematic diagram of the effect of geotextile

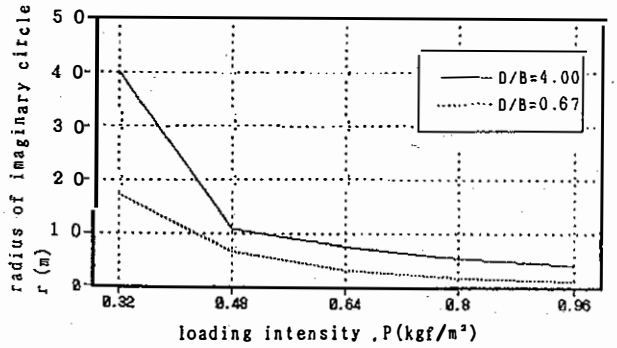


Fig.10 The radius of imaginary circle versus load intensity

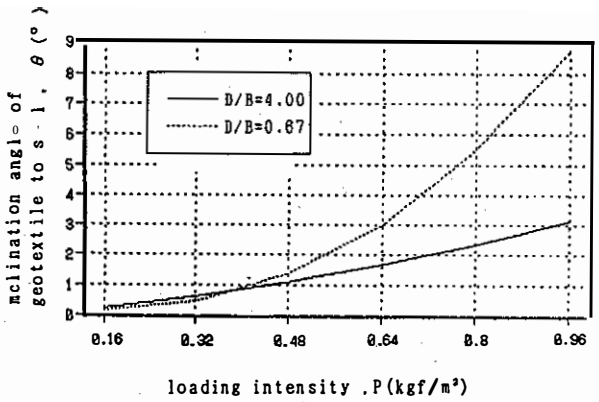


Fig.11 The inclination angle of geogrid versus load intensity

settlement, S_{max} and maximum heaving, H_{max} increases with decreasing the value of D/B. This means that more remarkable deformation can be seen in accordance with the decrease of D/B ratio less than 2.5.

Fig.8 shows the bearing capacity for various D/B ratios to investigate geotextile effect on the bearing capacity of surface loading. Fig.8 indicates that the effect of the ratio D/B on the bearing capacity of reinforced soft clay ground is greater than that of the unreinforced ground. Fig.8 also shows that D/B ratio gives no effect on the bearing capacity improvement of the reinforced soft ground with both geotextiles GG and PN. This result was also observed in the laboratory model footing test carried out by Hirao and Yasuhara(1991). It can be seen from Fig.8 that the bearing capacity with and without reinforcement is about 0.4 tf/m^2 and 0.325 tf/m^2 respectably. Therefore, the rate of calculated increase of the bearing capacity is 23% , nearly the same value both observed and calculated for the model footing test mentioned before.

Yamanouchi and Gotoh(1979) proposed the following formula eq.(6) to calculate the bearing capacity of reinforced soft ground,by modifying the Terzaghi bearing capacity theory.

$$q_d = \alpha\, c\, N_c + T\left(\frac{2\sin\theta}{B} + \frac{N_q}{r}\right) + \gamma_t D_f N_q \qquad (6)$$

where c, γ_t = undrained shear strength and unit weight of clay, N_c and N_q = bearing capacity factor, T = tensile stress of geotextile and r, θ =parameters given in Fig.9, which schematically shows the effect of geotextile as expressed by eq.(6).

Figs 10 and 11 shows that the undetermined parameters r and θ in eq.(6) versus load intensity, p curves calculated for geogrid GG in the case of D/B=4.00 and 0.67. It can be seen from Fig.10 that the parameter, θ increases with the increased load intensity, p. Also another interesting feature in Fig.10 is that the parameter, θ increases with the decreased D/B ratio and this tendency become more remarkable in accordance with the increased load, p. On the contrary, the parameter, r decreases with increasing the load intensity, p as clearly shown in Fig.11. This tendency was also recognized in the laboratory model footing test of the clay ground carried out by Yamanouch and Gotoh. However, the calculated Parameter, r is too large and θ is too small in comparison with the values measured in both the model test by Yamanouchi et al. and the field test by Watari et al.(1988).

6. CONCLUSIONS

The main Conclusions from the results of numerical analysis indicating good agreement with the observed values, are as follows:

1) The rate of the increase of the bearing capacity of the reinforced ground is about 25 %.

2) The ratio of the footing width to the clay layer thickness, D/B influences on the deformation remarkably, but gives no effect on the bearing capacity improvement of the reinforced soft ground.

3) The parameter, θ increases with the decreased D/B ratio and this tendency become more remarkable in accordance with increased load, p.

4) The parameter, r decreases with increasing of load intensity, p.

From the foregoing discussion of the numerical results related to the laboratory and in-situ footing test, the proposed numerical analysis can sufficiently estimate the deformation and the bearing capacity improvement of the soft clay ground reinforced with geotextiles.

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