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In-site investigation and numerical estimation for bearing capacity improvement of very soft ground reinforced with geotextiles

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ABSTRACT: This paper first describes the field investigation on the effect of bearing capacity improvement of the very soft ground reinforced with the composite geotextiles(non-woven fabric and geogrid) at the Chofu reclamation site near Shimonoseki, Yamaguchi Prefecture in Japan. Comparing the observed values of the mobilizing strain in geogrid with the filling work procedure with the analytical results by using the finite element technique proposed by Tanabashi et. al.(1992), the numerical simulation taking account of the earth spreading speed was carried out under the combination with the bulldozer's weight and the covered soil's thickness. Furthermore, the statical numerical simulation of real scale footing test was carried out varying the ratio of loading width to soft clay layer thickness, both with and without reinforcement and also both with and without the sand-mat layer. Finally, based on the results of these simulations, a rational estimation of the bearing capacity improvement and the deformation characteristics of the very soft ground reinforced with geotextiles and sand-mat layer has been proposed.

1 INTRODUCTION

New reclamation land just after dredging or soft alluvial clay deposits, which are unable to insure trafficability, is very popular in Japan.

In order to secure trafficability, soft ground reinforced with geotextiles and spreading sand mat is very popular execution in Japan. However, a rational estimation of bearing capacity improvement of very soft ground reinforced with geotextiles has not been established, because of the variety of the ground condition and the lack of information about both tensile stress mobilized within reinforcement material and the effect of sand mat layer.

The authors proposed a new finite element technique including of rational modeling for soil, geotextile and interaction between both materials (Tanabashi et.al., 1992)

This paper first describes the field investigation on the effect of bearing capacity improvement of the very soft ground reinforced with the composite (non-woven fabric and geogrid) at the Chofu reclamation site near Shimonoseki, Yamaguchi Prefecture in Japan. Furthermore, the statical numerical simulation was carried out varying the ratio of loading width to soft layer thickness, both with and without reinforcement and also both with and without sand mat layer. Finally, based on the results of these simulations, a rational method of estimating the bearing capacity improvement and the deformation characteristics of the very soft ground reinforced with and sand mat layer has been proposed.

2 IN-SITU INVESTIGATION

2.1 Outline of the testing site

The field testing site is located in the Chofu reclamation area near Shimonoseki, Yamaguchi Prefecture, Japan. The site is reclaimed by dredging the soft soil on the sea bed for construction of harbor facilities. After three months's self-consolidation of dredging soil, the reclamation site was reinforced with composite geotextiles (non-woven fabric (TF) and geogrid with opening 40 mm × 28 mm (GS2)) and completed primary earth filling work for six months from November 1991 to April 1992.

2.2 In-situ testing procedure

The in-situ testing procedure of the site was as follows:

(1) excavation of primary fill whose height was 0.64 m.

(2) exposure of the geogrid (GS2) used for reinforcement.

(3) replacement of another geogrid with opening 60 mm x 62 mm (GM4) with strain gage. Because the geogrid (GS2) 's rib was too slender to attach the strain gage and replaced area was 50 cm x 80 cm in square.

(4) measurement of mobilizing strain in the geogrid (GM4) during back filling of the primary fill.

(5) measurement of mobilizing strain in the geogrid (GM4) during the secondary filling in 1 m height after one month later of the back filling.

2.3 Result of in-situ testing

Fig.1 indicates the result of the mobilizing strain in both longitudinal and latitude ribs of the geogrid (GM4) during the buck filling and the secondary filling.

The mobilizing strain increases with increase of the back filling's height and the strain is almost stable after the back filling. The maximum mobilized strain during the back filling is 0.13% and this value is only 0.53% of the strain at the failure of the geogrid (GM4). The mobilized strain at instance after the secondary filling is about 1.6 times of the maximum value during the back filling. The strain after the secondary filling has the tendency of decrease with elapsed time. Moreover, the mobilizing strain in the longitudinal rib is always about twice of that in the latitude rib during every testing procedures.

3 ANALYSIS OF IN-SITU TESTING PROCEDURE

3.1 Analytical method

The authors proposed a new coupled stress finite element technique organized of rational modeling for

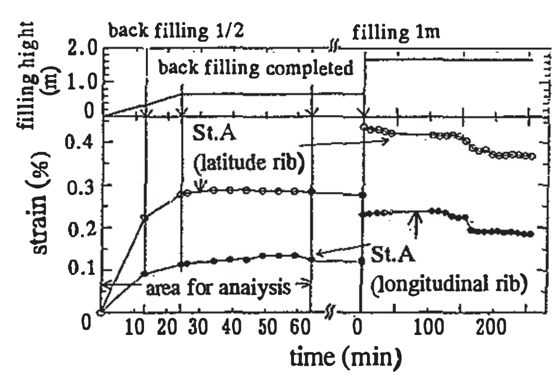

Fig.1 The observed mobilizing strain in both longitudinal and latitude ribs of the geogrid (GM4) during the back filling and the secondary filling

soil and geotextile with including interaction between those materials (Tanabashi et. al., 1992). A soil model was adopted the elasto/visco-plastic model proposed by Sekiguchi and Ohta (1979) which was able to represent the critical mode of the soil behavior. A geotextile model consists of a plane beam element which was able to take the slight bending stiffness and the non-linearity of tensile stress-strain curve of geogrid into consideration. The interaction model between soil and geotextile was a Goodman's joint element considering deformation dependency. The analytical method for in-situ testing is the same one as mentioned above.

3.2 Analytical model and condition

The analytical model simulates 12.6 m thickness of soft soil at the site and the width is four times of the thickness of soft soil, 50.4 m and its mesh consists of total 420 quadrilateral elements and total 403 nodes.

Geometrical boundary condition is settled as that both vertical side is restrained in vertical direction and base is restrained both vertical and horizontal direction. Hydraulic condition is settled as that both vertical side is undrained and both surface and base is drained conditions. Numerical analysis has been done under the plane strain condition. The initial input parameters of the soft ground are determined with only plasticity index, PI following the determination flow chart proposed by Iizuka & Ohta (1987). The in-situ testing during only the back filling is analyzed.

3.3 Analytical result

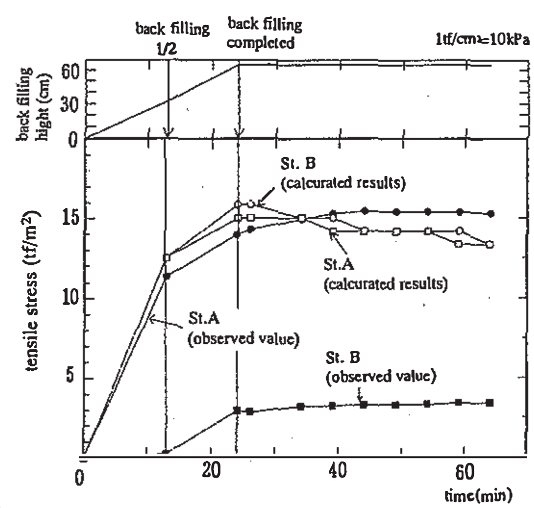

Fig.2 The analytical result of mobilized tensile stress in the geogrid (GM4) during and 40 minutes after the back filling

Analytical result of mobilized tensile stress in the rib of the geogrid (GM4) during and 40 minutes after the back filling is shown in Fig.2. The mobilized tensile stress in Fig.2 is calculated from the analytical result and a stress-strain curve of the geogrid (GM4) in a tensile test. Fig.2 indicates that the calculated value is able to estimate sufficiently the observed value during the all testing period at the station A. The distance between station A and B in Fig.2 is only 4 m at the site. Therefore, the measuament trouble at the station B was supposed to occur for any reason, especially during the half back filling. In spite of this fact, the calculated value can well represent the tendency of the observed value after the half back filling.

4 NUMERICAL SIMULATION OF REAL EXECUTION PROCEDURE

The analysis of the in-situ testing procedure did not take a real execution procedure of filling work into consideration, for example, real filling work and weight of bulldozer and those movement. Therefore, a numerical simulation of real execution procedure of filling work on soft ground reinforced with composite geotextiles was carried out for the examination of the effectiveness with regard to bulldozer's weight and filling height.

4.1 Simulation model and condition

The soft ground, the geotextiles and the interaction between those materials are used the same models for analyzing the in-situ testing procedure. The numerical simulation taking account of the earth spreading speed was carried out under the combination with the bulldozer's weight, W_b =4 tf, 10 tf and the fill's height, H_f = 30 cm, 50 cm (refer to Table 1). The bulldozer's speed is postulated 1.7 m/d and the speed of the filling front is also postulated the same of the bulldozer. The bulldozer's weight is represented as the equivalent nodal force in the finite element procedure. The unit weight, γ_t of fill is given 1.7 tf/m^3, and γ_t of the elements before the filling front is given zero.

Table 1 The combination with the bulldozer's weight and the fill's height

	fill's thickness, H_f (m)	bulldozer's weight, W_b (tf)
Case.1	0.3	4
Case.2	0.3	10
Case.3	0.5	4
Case.4	0.5	10

The above postulations are illustrated as the schematic diagram of Case 3 for example in Fig.3.

4.2 Result of numerical simulation

The relationships between absolute vertical displacements and the extends of the filling front, L_f are shown in Fig.4 under the combination with the bulldozer's weight and the fill's height, where, absolute vertical displacement means the total amount of maximum both settlement and heaving. Fig.4 indicates that the absolute vertical displacement is affected by more fill's height (fill's weight) than the bulldozer's weight and the maximum absolute vertical displacement is 22 cm for Case 4, i.e., under the combination with the bulldozer's weight, Wb = 10 tf and the fill's height, H_f = 50 cm. Maximum mobilized tensile stress in the rib of the geogrid for Case 1-4 with the combination of the bulldozer's weight and fill's height is illustrated in Fig.5.

Fig.5 indicates that the shorter the extend of the filling front is, the larger the maximum mobilized

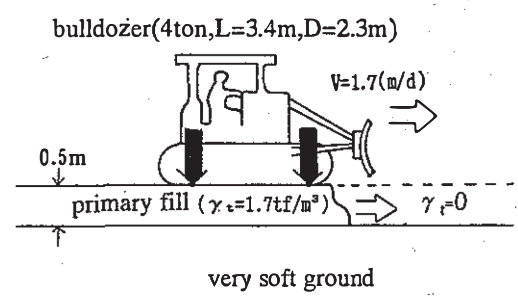

Fig.3 The schematic diagram of adopted postulations

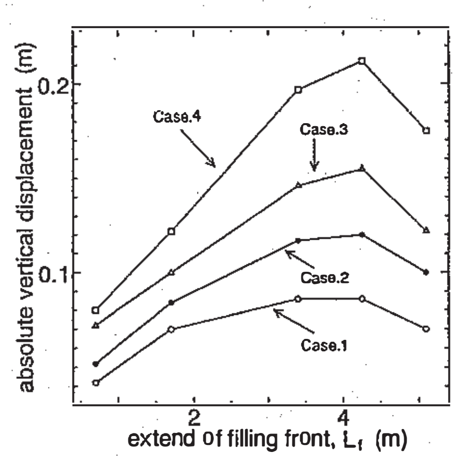

Fig.4 The relationships between absolute vertical displacements and the extends of the filling front

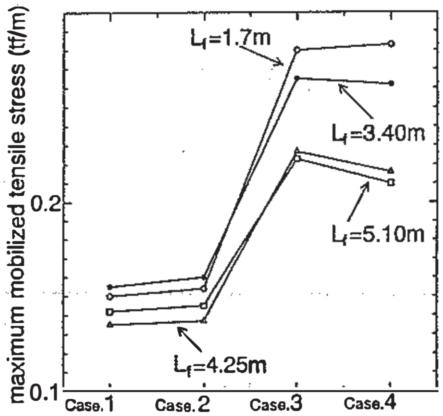

Fig.5. The maximum mobilized tensile stress in the rib of the geogrid for Case 1-4

tensile stress is. This can be explained by the adopted postulation that the horizontal displacement of starting point of filling work is restrained. Further, Fig.5 shows that the maximum mobilized tensile stress is more affected fill's height (fill's weight) more than the bulldozer's weight notwithstanding the extend of the filling front.

5 NUMERICAL SIMULATION OF REAL SCALE FOOTING TEST

5.1 Outline of the previous paper by Tanabashi et.al. (1992)

The outline of the model footing tests carried out by Hirao et.al. (1992) is described in detail in their literature. 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 Kanda clay used are: specific gravity G_S = 2.62, initial water content w_i=130 %, liquid limit w_L = 107% and plasticity index PI=66. Incremental load of 0.01 kgf/cm² is applied for every 15 minutes. The model footing tests were carried out with various combination of woven fabrics, non-woven fabrics, geogrids under the condition with and without sand mat layer.

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

5.2 Analytical condition of real scale footing test

The scale of in-situ footing test is assumed 20 times

Table 2 The final fixed values of the in-put parameters for the following numerical simulation

D:coefficient of dilatancy	0.112~0.124
Λ:irreversibility ratio=1-κ/λ	0.85
M:critical state parameter	0.923~1.02
ν':effective Poisson ratio	0.378~0.405
k_{x0}/γ_w:coefficient of permeability of x direction (cm/min)	1.09×10^{-4}
k_{y0}/γ_w:coefficient of permeability of y direction (cm/min)	1.09×10^{-4}
σ'_{v0}:pre-consolidation pressure (kgf/cm²)	0.54~8.68
K_0:coefficient of earth pressure at rest	0.608~0.645
σ'_{vi}:effective overdurden pressure (kgf/cm²)	0.54~8.68
K_i:coefficient of earth pressure at rest in-situ	0.54~8.68
α:coefficient of secondary compression	0.0067
V_0:initial volumetric strain rate	1.1×10^{-7}
λ:compression index	0.485
e_0:void ratio (at pre-consolidation)	2.60

of that of the model footing test carried out by Hirao et.al. (1992). Both geometrical and hydraulic boundary conditions are the same of those for the analysis of the in-situ testing procedure. Numerical analysis has been done with free end of the geotextile as restrained condition. The final fixed values of the parameters for the following numerical simulation are determined as shown in Table 2.

5.3 Analytical result of real scale footing test

Yamanouchi and Gotoh (1979) proposed the following formula Eq.(1) to calculate the bearing capacity of reinforced soft ground, by modifying the Terzaghi's bearing capacity theory.

$$q_d = \alpha \cdot c_u \cdot N_c + \frac{2T \cdot \sin\theta}{B} + \frac{T}{r} N_q + \gamma_t \cdot N_q \qquad (1)$$

where c_u, γ_t : undrained shear strength and unit weight of clay, N_c and N_q : bearing capacity factors, D_f: Maximum settlement, T : mobilizing tensile stress in geotextile, r : radius of imaginary circle, θ: inclined angle of geotextile to the horizontal surface. All parameters are given in Fig.6, which schematically shows the effect of geotextile as expressed by Eq.(1).

The calculated correlation between three undetermined parameters, D_f, q, r in Eq.(1) for geogrid (GS2) is shown in Fig.7. Fig.8 indicates the same correlation obtained by 1g model footing test reported by Hirao et. al. (1993). However, maximum settlement, D_f is normalized by dividing by the equivalent fill's height, H to the load intensity, p (refer to Fig.6,7 and 8).

The inclined angle of geotextile to the horizontal surface, θ increases up to about 70 degrees with increase of D_f or q in both Fig.7 and Fig.8. The radius of imaginary circle, r decreases from about 2 m to 0.2 m with increase of D_f or q in both Fig.7 and Fig.8. These similarities of Fig.7 and Fig.8 may indicate the validity of the previous proposed numerical analysis.

Four undetermined parameters in Eq.(1), D_f, r, q, T can be apploximated for quadratic equation of both load intensity, p, and the ratio of footing width (B) to clay layer thickness (D), D/B for the geogrid (GS2) with sufficient accuracy. Therefore, the bearing capacity can easily be estimated if the information on undrained shear strength, c_u, clay layer thickness, D, width of footing or embankment, B and unit weight, γ_t for any clay ground can be obtained. Furthermore, contribution degree of each four term in the right side of Eq.(1) to bearing capacity improvement can be also easily estimated.

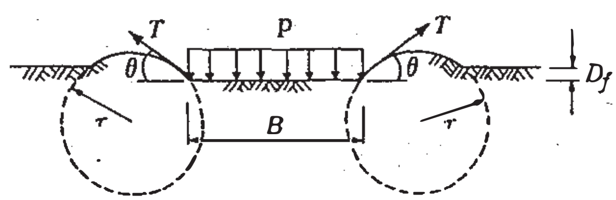

Fig.6 The schematically diagram of the effect of geotextile as expressed by Eq.(1)

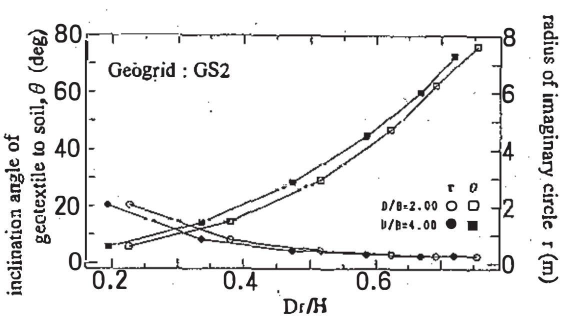

Fig.7 The calculated correlation between three undetermined parameters, D_f, θ, r in Eq.(1) for geogrid (GS2)

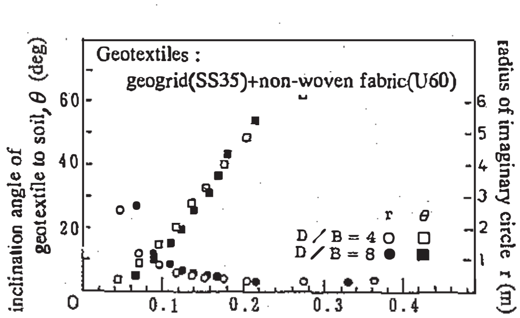

Fig.8 The observed correlation between three parameters, Df, θ, r in Eq.(1) for geogrid (SS35) + non-woven fabric (U60)

6 ESTIMATION OF SAND MAT EFFECT

6.1 Simulation of model footing test

Footing test of model clay ground reinforced with geogrid and with sand mat were carried out by HIrao et.al.(1996) by using the same Kanda clay and the same test equipment.

Fig.9 shows a comparison of the load-settlement curves of the model footing test with analytical results for the case of D/B= 4.0. for the Kanda clay ground both with and without sand mat. It is clear that the calculated values have proved the settlement restrained effect of the sand mat, and also reasonably have estimated the observed load intensity, p, versus settlement, S curves.

6.2 Simulation of real scale footing test

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. Hereafter, these symbols are used for the bearing capacity without geogrid, p_{yun}, with geogrid (GS2), p_{yr}, with geogrid and sand mat, p_{yrs}, and these

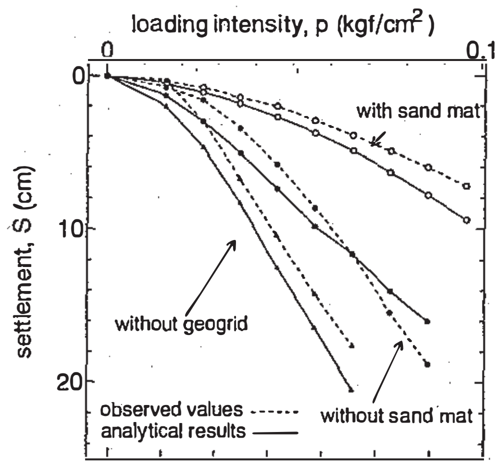

Fig.9 Comparison of the load-settlement curves of the model footing test with and without sand mat

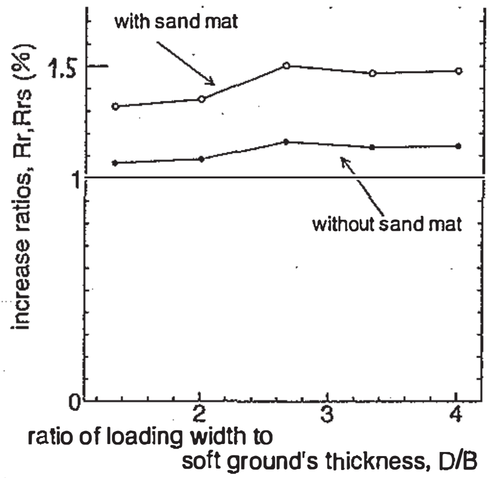

Fig.10 Correlation between the increase ratios, *Rr*, *Rrs* and the ratio of footing width to clay layer thickness, D/B

increase ratios, $Rr = p_{yr} / p_{yun}$, $Rrs = p_{yrs} / p_{yun}$, respectively. The increase ratios, *Rr*, *Rrs* concerning the ratio of footing width to clay layer thickness, D/B are shown in Fig.10

It is understood from the value of *Rr*, *Rrs* in Fig.10 that the bearing capacity of reinforced case is improved by 15 % and that of reinforced case with sand mat is improved by 40 % higher than that in unreinforced case.

From the result of the simulation, it has been clarified that settlement, heaving and lateral displacement restrained rate of reinforced case are 25%, 58 % and 50 % and those of reinforced case with sand mat are 45 %, 83 % and 75 % respectively, with regard to those in unreinforced case. From the above results, it can be concluded that the placement of sand mat combined with geogrid is most effective for bearing capacity improvement and its effect for bearing capacity improvement is calculated by <40 % / 15 % = 2.67>. In the similar manner, sand mat effects for settlement, heaving and lateral displacement are calculated 1.80, 1.43 and 1.50 times, respectively, as those with no sand mat.

7 CONCLUSIONS

The main conclusions obtained from this study are as follows:

(1) The proposed analytical method sufficiently explains the mobilizing maximum tensile stress in the geogrid and its time-dependent behavior.

(2) Both maximum absolute vertical displacement and mobilizing tensile stress in the geogrid depend on the fill's thickness than the bulldozer's weight.

(3) All four undetermined parameters, D_f, r, q, T in Eq.(1) are able to be estimated as the function of the current loading intensity, p and the ratio of loading width to soft clay layer thickness, D/B. Therefore, the bearing capacity can easily be estimated if the values of c_u, D, , B , γ_t are obtained for a given clay.

(4) The bearing capacity improvement rate of the very soft ground reinforced with geotextiles with and without sand mat layer of 30 cm thickness is 15 % and 40 %, respectively. The sand mat is effective for bearing capacity improvement and its effect can be estimated 2.7 in comparison with the case without sand mat.

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