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Numerical analysis of the behavior of clay foundation beneath reinforced embankment

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ABSTRACT: The performance of an embankment with a geogrid is dependent on soil-structure interaction between foundation, the embankment fill and the geogrid reinforcement. In the present paper, a series of finite element analyses for the stability and deformation of an embankment reinforced at the base and overlaying on a clay foundation have been carried out.

1 INTRODUCTION

A dramatic increase in the use of high tensile strength plastic materials as reinforcement to ensure the stability of embankments constructed on soft foundations are seen recently. These reinforcements are being used to secure steep slopes of embankments and control differential settlement.

Most of the design methods for reinforced soil structures, currently being employed are based on the theory of rigid-plasticity which takes no account of displacements and deformations of reinforcing material and the soil (Oikawa, 1988; Leshchinsky, 1987; Miki,H. et al., 1986). The performance of an embankment with a geogrid is, however, highly dependent on soil-structure interaction between foundation, the embankment fill and the geogrid reinforcement. It is also well known that the failure of a soft ground is closely related to the magnitude and history of the deformation before the final failure (Matsuo and Kawamura, 1977; Shibata, 1982). The deformation is an important index of the failure of a soft ground. Therefore, the interaction between the embankment fill, the geogrid and the clay foundation needs to be investigated. The number of studies carried out (eg. Ochiai et al., 1987; Koga et al. 1988; Mylleville et al. 1988) on the interaction between the geogrid and the foundation ground are very small.

Since, a soft sensitive clay is regarded as a strain-hardening plastic and rate sensitive material with dilatancy, the constitutive model must be the one which can describe the behavior due to those properties. The purpose of this paper is to evaluate the effectiveness of the geogrid method as the reinforcement for embankments on the basis of the consideration of both stability and deformation of clay foundations. In the present paper, the performance of an embankment with the geogrid (SR2) on a soft sensitive clay foundation is analyzed by use of an elasto-viscoplastic constitutive model and a finite element method considering the consolidation phenomena.

It is found from the numerical results that the mode of failure and degree of improvement in embankment stability may vary substantially depending on the amount of the reinforcement used, the rigidity of the embankment fill and the properties of the foundation soil.

2 FINITE ELEMENT ANALYSIS

In the analysis, an elasto-viscoplastic constitutive model developed by Adachi and Oka (1982) based on the overstress type viscoplastic model and Cam-clay model is used for the soft sensitive clay foundation. The viscoplastic flow rule is given by

$$\dot{\epsilon}_{ij}^{vp} = < \Phi(F) > \frac{\partial f}{\partial \sigma'_{ij}} \tag{1}$$

where $\dot{\epsilon}_{ij}^{vp}$ is the viscoplastic strain rate, σ'_{ij} is the effective stress tensor, f is the dynamic yield function, Φ is the material function for strain rate effect, $F = 0$ denotes the static yield function and $<>$ is the Macaulay's brackets.

The yield function is expressed as follows,

$$f = \frac{\bar{\eta}^*}{M^*} + ln\frac{\sigma'_m}{\sigma'_{my}} = 0 \tag{2}$$

where σ'_{my} is the hardening parameter and $\bar{\eta}^*$ is the stress ratio invariant expressed by

$$\bar{\eta}^* = \sqrt{(\eta_{ij} - \eta_{ij(0)})(\eta_{ij} - \eta_{ij(0)})} \tag{3}$$

$$\eta_{ij} = s_{ij}/\sigma'_m \tag{4}$$

in which s_{ij} is the deviatoric stress tensor, σ'_m is the mean effective stress and $\eta_{ij(0)}$ is the value of η_{ij} at the initial state.

Material function $\Phi(F)$ is given by

$$\Phi(F) = C \cdot exp\{m'(\frac{\bar{\eta}^*}{M^*} + ln\frac{\sigma'_m}{\sigma'_{me}} - \frac{1+e}{\lambda-\kappa}v^{vp})\} \tag{5}$$

where M^*, m' and C are material constants, σ'_{me} is the initial value of σ'_m, λ is the compression index, κ is the swelling index, e is the void ratio and v^{vp} is the viscoplastic volumetric strain.

In the present paper, the clay foundation at Saint Alban, west of Quebec city, Canada where the geotechnical group of Laval University constructed a test embankment is modeled as the typical soft sensitive clay foundation (Oka et al. 1991).

Fig.1 shows the geometrical configuration of the embankment and the clay foundation analysed using the geogrid as the reinforcement. Soft sensitive clay deposits of depth equal to 13.8 m is examined and is underlain by a rigid base. Considering the previous works on the geotechnical properties of the site (Tavenas et al., 1974; Kabbaj et al., 1988), the soil profile has been defined. The coefficient of the earth pressure at rest in the overconsolidated region at Saint Alban is 0.8. It is approximately 0.5 in the normally consolidated region. The density of the embankment fill material is 1.857 tf/m^3. Other material parameters are summarized in Table 1. Parameters, λ, m' and C are assumed to depend on the viscoplastic volumetric strain as shown in Table 2. The viscoplastic parameter, C, is assumed to be especially related to the preconsolidation pressure, σ'_p, as

$$C = C_0 exp(\sigma'_{v0}/\sigma'_p) \tag{6}$$

where σ'_{v0} is the initial vertical effective stress. Oka et al. (1991) succeeded in predicting the performence of the test embankment on this site.

The geogrid is located on the embankment fill/clay foundation interface. The interface between the soil and the geogrid is represented by the joint elements to allow for the relative displacement between the soil and the geogrid if the mobilized shear stress at the interface equals or exceeds that obtained from Mohr-Coulomb failure criterion.

The bottom and upper surfaces are permeable boundary and excess pore water pressure is initially zero in the clay deposit. Horizontal displacement is constrained at the vertical boundaries which are assumed to be impermeable.

For the solution procedure by FEM, a modified Christian's method in which finite element and finite difference methods are combined. The governing equation for the flow of pore water is discretized by the finite difference method. A four-noded isoparametric element is used. In the time integration process of viscoplastic strain, an implicit technique given by Owen et al. (1982) has been used.

In order to investigate the effect of the geogrid, a series of numerical analyses are carried out. All analytical cases are listed in Table 3. In the analyses, the loading history of the embankment, the stiffness of the embankment fill and number of layers of the geogrid are changed. The coefficient of permeability depends on the void ratio (Oka et al. 1991).

Table 1 Material parameters.

No. of layer	Depth (m)	λ_0	λ_1	λ_2	e_0	σ'_p (tf/m^2)	G (tf/m^2)
1	0.00~0.66	0.020	0.300	0.100	1.10	7.39	89.7
2	0.66~1.50	0.523	1.000	0.363	1.70	5.82	153
3	1.50~3.00	0.072	1.140	0.495	2.30	4.69	212
4	3.00~4.80	0.039	1.040	0.411	1.80	7.20	307
5	4.80~6.70	0.246	0.560	0.282	1.80	9.00	412
6	6.70~9.60	0.010	0.409	0.175	1.40	14.0	571
7	9.60~13.5	0.008	0.409	0.100	1.40	18.0	1120

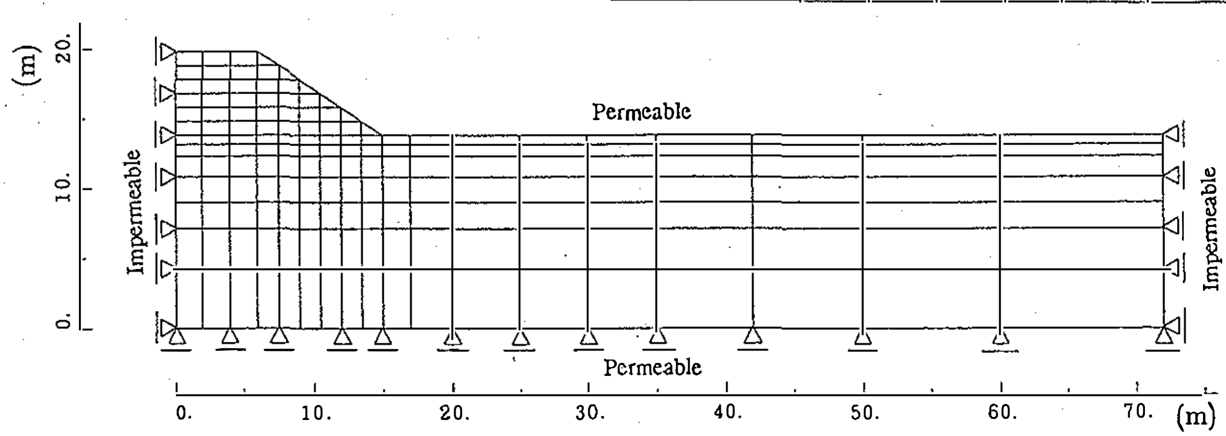

Fig.1 Finite element mesh.

Table 2 Dependency of parameters on volumetric plastic strain.

Volumetric viscoplastic strain v^{vp}(%)	m'	λ	C'_0(1/sec.)
v^{vp} <0.027	17.8	λ_0	1.2×10^{-12}
0.027≤ v^{vp} <4.2	26.7	λ_1	5.9×10^{-11}
4.2< v^{vp}	26.7	λ_2	5.9×10^{-11}

Table 3 Analytical cases.

Anal. case	Filling rate of emb. (m/day)	Stiffness of emb. (tf/m^2)	Num. of layers of geogrid
1	0.2	100.0	none
2	0.2	100.0	1
3	0.2	100.0	3
4	0.2	1000.0	none
5	0.2	1000.0	1
6	0.2	1000.0	3
7	0.2	10000.0	none
8	0.2	10000.0	1
9	0.2	10000.0	3
10	1.0	100.0	none
11	1.0	100.0	1
12	1.0	100.0	3
13	1.0	1000.0	none
14	1.0	1000.0	1
15	1.0	1000.0	3
16	1.0	10000.0	none
17	1.0	10000.0	1
18	1.0	10000.0	3

Table 4 Summary of numerical results.

Analytical case	δ_v(cm)	δ_v/δ_v^*	δ_h(cm)	δ_h/δ_h^*	δ_h/δ_v
1	11.64	1	7.48	1	0.643
2	11.60	0.997	7.39	0.988	0.637
3	10.72	0.921	6.60	0.882	0.616
4	10.84	1	5.24	1	0.483
5	10.24	0.945	4.93	0.941	0.481
6	9.93	0.916	4.69	0.895	0.472
7	8.50	1	2.21	1	0.260
8	8.66	1.019	2.25	1.018	0.260
9	8.43	0.992	2.16	0.977	0.256
10	8.65	1	8.11	1	0.938
11	8.53	0.986	7.90	0.974	0.926
12	8.34	0.964	7.50	0.925	0.899
13	7.74	1	5.36	1	0.693
14	7.39	0.955	5.10	0.951	0.690
15	7.48	0.966	5.02	0.937	0.671
16	6.41	1	2.26	1	0.353
17	6.32	0.986	2.22	0.982	0.351
18	6.40	0.998	2.23	0.987	0.348

δ_v^*, δ_h^*: without geogrid

3 NUMERICAL RESULTS

Table 4 shows the displacement of the embankment at a hight of embankment equal to 6 m. In this table, δ_v is the vertical settlement just below the center of the embankment and δ_h is the lateral displacement at the toe of the slope. The positive value for δ_h means an outward displacement from the embankment center. From this table, it is indicative that the geogrid reduces the lateral displacement more significantly than the reduction of vertical settlement. Hence, there is no major advantage in using the geogrid regarding the reduction of settlement of the embankment. It can be noted that if the rigidity of the embankment fill is too stiff, the improvement by the geogrid is not to be expected. Further, the improvement by the geogrid with slower rate of construction of embankment is more than that for the case with faster construction rate.

Fig.2 shows the contours of equal lateral displacement at the hight of embankment equal to 6 m for the case with the stiffness of the embankment fill of 100 tf/m^2. It is found from these figures that the presence of geogrid reduces the lateral displacement in the upper portion of the clay foundation. On the other hand, the geogrid does not contribute to the reduction of the lateral displacement at lower depths of the clay foundation. The geogrid also contributes to the improvement of the safety in the upper portion of the clay foundation.

Fig.3 shows the relationships between the settlement just below the center of the embankment and time for the cases without geogrid. Fig.4 shows the relationships between the lateral displacement at the toe of the embankment and time for the cases without geogrid. It is easily found out from these figures that the deformation of the embankment strongly depends on the filling rate of the embankment and the rigidity of the embankment fill.

The relationship between δ_v and δ_h/δ_v proposed by Matsuo and Kawamura (1977) has been often used as the embankment construction control. Fig.5 shows these relationships for all cases. Since, in this investigation, the foundation soil has been modelled as "overconsolidated structured clay", being different from those assumed in the work of Matsuo and Kawamura, the calculated vertical settlement values of the embankment are much smaller than those indicated by them. From these diagrams, the safety of the embankment is found to be dependent on the filling rate of the embankment, the rigidity of the embankment fill (Ohtsuka and Asaoka, 1989). In the present study, these effects appear to be more significant than the effect of the geogrid.

The tensile force distribution along the geogrid is shown in Fig.6. If the elastic modulus of the embankment material is 10000 tf/m^2, the compressive force does not occur along the geogrid.

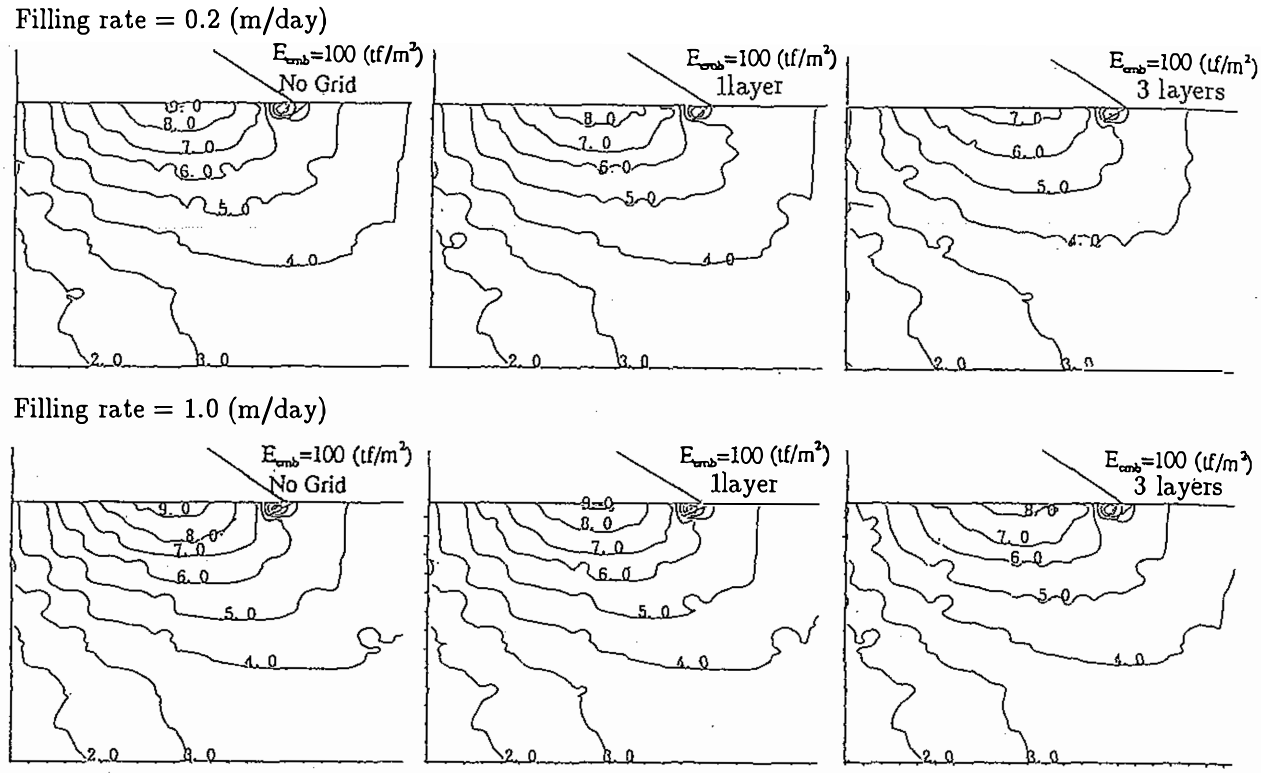

Fig.2 Contours of equal lateral displacement. (unit : cm)

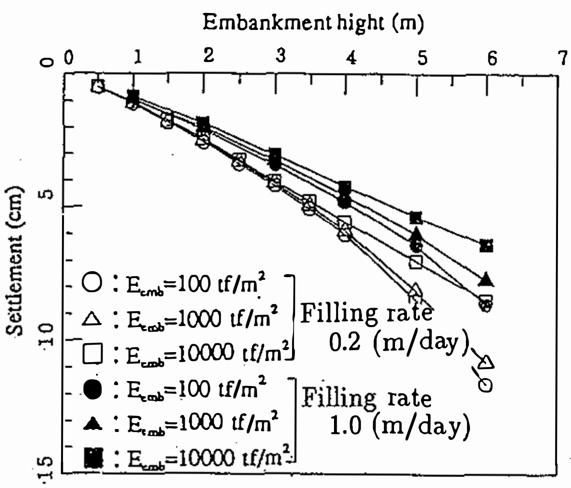

Fig.3 Relationship between the settlement and time.

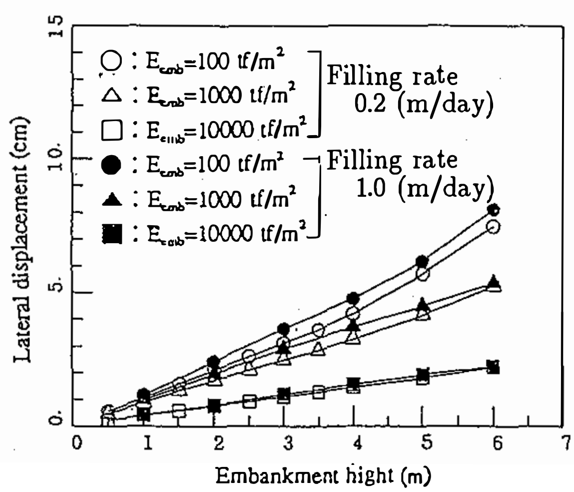

Fig.4 Relationship between the lateral displacement and time.

On the other hand, the compressive force occurs near the toe of the embankment if the stiffness of the embankment becomes softer.

4 CONCLUSIONS

A series of finite element analyses for the stability and deformation of an embankment reinforced at the base and overlaying on a clay foundation have been carried out. The principal conclusions

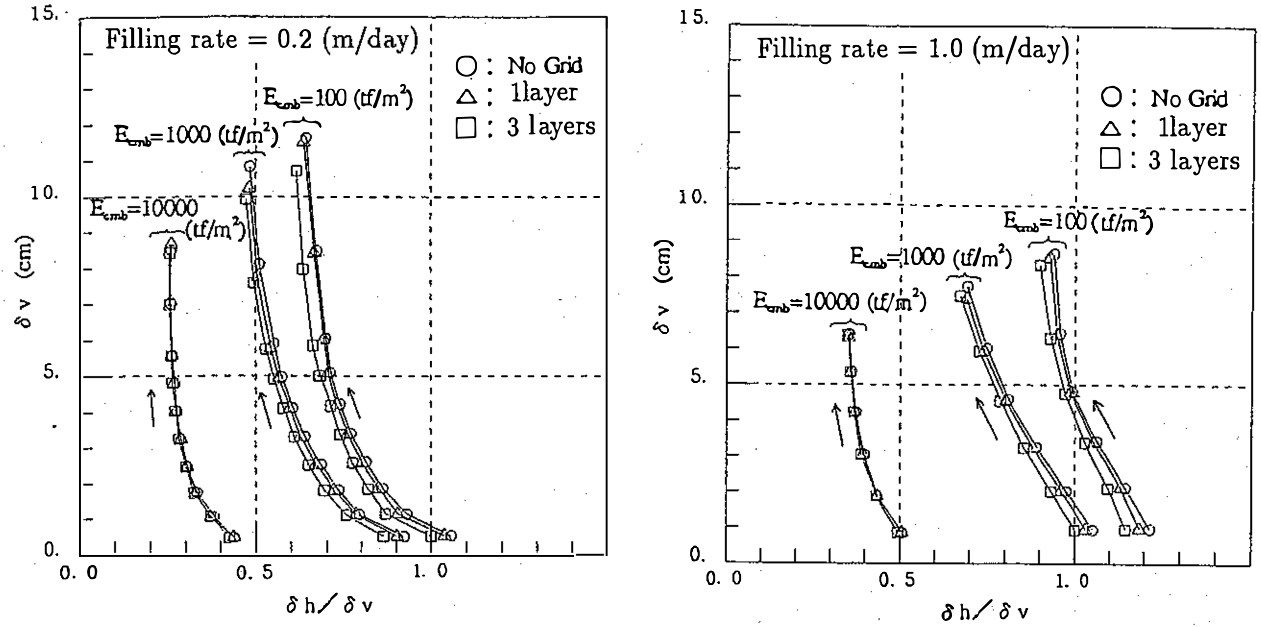

Fig.5 ($\delta_h/\delta_v \sim \delta_v$) diagram.

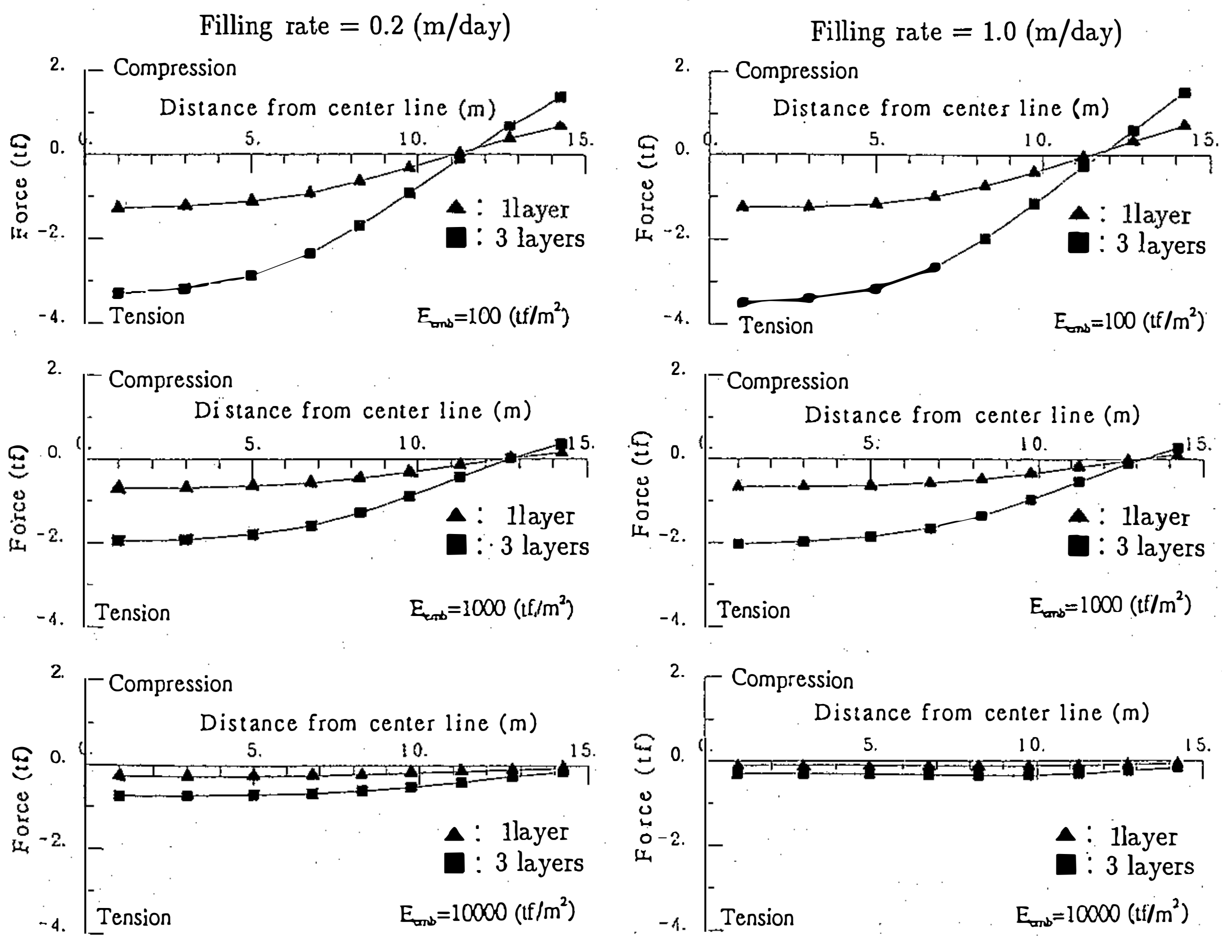

Fig.6 Force distribution along the geogrid.

drawn from these analyses may be summarized as follows.

1) The performance of an embankment with geogrid as reinforcement, highly depends on the soil-structure interaction between the foundation, the embankment fill and the geogrid reinforcement. Therefore, the soil-reinforcement interaction has to be investigated to evaluate the effectiveness of the geogrid reinforcement.

2) The geogrid reinforcement can reduce the lateral displacement more significantly than the vertical settlement. When the rigidity of the embankment is too stiffer, the improvement in the embankment behavior due to geogrid reinforcement is negligible. It is found that if the filling rate of embankment is slower, the improvement in the behavior due to geogrid becomes more significant.

3) The geogrid contributes to the improvement of the safety in the upper portion of the clay foundation. On the other hand, the geogrid does not contribute to the reduction of the lateral displacement at lower depths of the clay foundation.

4) The tensile force distribution along the geogrid is strongly dependent on the stiffness of the embankment.

REFERENCES

[1] Adachi,T. and F.Oka. 1982. Constitutive equations for normally consolidated clay based on elasto-viscoplasticity. Soils and Foundations, 22(4), 57-70.

[2] Kabbaj,M., F.Tavenas and S.Leroueil. 1988. In situ and laboratory stress-strain relationships. Geotechnique, 38, 83-100.

[3] Koga,K., G.Aramaki and S.Valliappan. 1988. Finite element analysis of grid reinforcement. Proc. Int. Geo. Symp. Theory and Practice of Earth Reinforcement, 407-411.

[4] Leshchinsky,D. 1987. Short-term stability of reinforced embankment over clayey foundation, Soils and Foundations, 27(3), 43-57.

[5] Matsuo,M. and K.Kawamura. 1977. Diagram for construction control of embankment on soft ground. Soils and Foundations, 17(3), 37-52.

[6] Miki,H. et al. 1986. Large scale experiments on the behavior of embankment reinforced with polymer grids. Proc. 1st Geotextile symp., J.P.IGS, 77-82. (in Japanese)

[7] Mylleville,B.L.J. and R.K.Rowe. 1988. Steel reinforced embankments on soft clay foundations. Proc. Int. Geo. Symp. Theory and Practice of Earth Reinforcement, 437-442.

[8] Ochiai,H. and A.Sakai. 1987. Analytical method for geogrid-reinforced soil structures. Proc. 8th ARCSMFE, 483-486.

[9] Oikawa,H. 1988. Stability of reinforced embankment over soft cohesive foundation, Proc. Int. Geo. Symp. Theory and Practice of Earth Reinforcement, 455-460.

[10] Ohtsuka,S. and A.Asaoka. 1989. Bearing capacity analysis of soft clays taking account of rigidity of footings. Proc. 34th JSSMFE Sympo. 77-82. (in Japanese)

[11] Oka,F., F.Tavenas and S.Leroueil. 1991. An elasto-viscoplastic FEM analysis of sensitive clay foundation beneath embankment. Computer Methods and Advances in Geomechanics, Balkema, 1023-1028.

[12] Owen,D.R.J. and F.Damjanic. 1982. Visco-plastic analysis of soils, Recent advances in nonlinear computational mechanics, edt. by Hinton et al., Pineridge Press Limited, 225-254.

[13] Shibata,T. 1982. Lateral displacements and stability of embankments. Tsuchi-to-Kiso, 30(5), 3-6. (in Japanese)

[14] Tavenas,F., C.Chapeau, P.Larochelle and M.Roy. 1974. Immediate settlements of three test embankments on Champlain clay. Can. Geotech. J., 11(1), 109-141.