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EFFECT OF BASE REINFORCEMENT ON THE BEHAVIOR OF EMBANKMENT OVER SOFT SUBSOIL

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ABSTRACT: A case history of both reinforced and unreinforced embankments on soft subsoil built to failure is described. The results of a laboratory model test about the behavior of embankment on soft soil are referred during discussions. The effect of geotextile reinforcement on embankment behavior is discussed by comparing the field data of with and without reinforcement cases. The field data as well as analysis indicate that the reinforcement had a certain effect on embankment stability. However, at working state (normally with a factor of safety of 1.2 to 1.3), the reinforcement did not have an obvious effect on subsoil response. Only under the condition that unreinforced embankment approached to failure, the effect of reinforcement on subsoil could be noticed. The laboratory model test results indicated that if the reinforcement is strong enough, the effect of reinforcement is considerable. It is suggested that although the geotextile certainly has a beneficial effect on embankment over soft subsoil, due to the relative lower stiffness of geotextile, to achieve a substantial improvement on embankment behavior, the multi-layer geotextile reinforcements or high strength geogrid may be needed. This case history also demonstrated that the rate of lateral displacement and excess pore pressure development are sensitive indexes for the stability of embankment on soft subsoil.

INTRODUCTION

In the case of constructing an embankment over soft subsoil with low strength and high compressibility, the engineering task is to prevent the failure of embankment and control the subsoil deformation. Several methods have been developed for economically and safely constructing embankment on soft subsoil. Placing a layer of reinforcement at the base of embankment is one of the methods.

The mechanisms of reinforcing an embankment over soft subsoil are discussed elsewhere (Bonapare and Christopher 1987; Jewell 1988). The functions of a reinforcement are: (1) tensile force mobilized in the reinforcement contributing to the stability of embankment, and (2) reinforcement tensile force providing a confinement to embankment fill and foundation soil adjacent to reinforcement. This confining effect can reduce the lateral distortion to subsoil due to embankment load, and therefore, the shear stress level in subsoil. The magnitude of the reinforcement effect is related to the strength of reinforcement, the relative stiffness of reinforcement and

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subsoil, embankment geometry, and the factor of safety of a embankment. Chai and Bergado (1993) showed that at working state, the geogrid placed at the base of a full scale embankment only had a minor effect on embankment stability, and a negligible effect on subsoil deformation. However, there are some reported case histories in which the reinforcement had a considerable effect on embankment behavior (e.g. Rowe et al. 1984). Regarding to the magnitude of reinforcement effect, one of uncertainties is that some reported cases lack the comparison with unreinforced case at the same site. During analysis, any under or overestimation on soil properties will over or underestimate the effect of reinforcement. Also, the effect of reinforcement is strongly stress level dependent. Low and Ducan (1985) using finite element analysis showed that when the factor of safety of unreinforced case is less than 1.0, the reinforcement can improve the performance of the embankment significantly. Therefore, to have a better understanding on the magnitude of reinforcement effect, directly comparing the behavior of reinforced and unreinforced embankments at the same site is desirable, especially, for those built to failure cases.

From April 1986 to June 1986, a test section with 6 subsections was constructed over a soft deposit, Lian-Yun-Gang, China. Among 6 subsections, one was on natural subsoil, and one was reinforced by a layer of geotextile. Both with and without reinforcement cases were built to failure. Other 4 subsections were built on prefabricated vertical drain (PVD) improved subsoil. This paper describes test results of two built to failure subsections. The subsoil condition and embankment construction is presented first. The behavior of the two built to failure embankments is compared and analyzed to demonstrate the effect of reinforcement on (1) embankment stability and (2) subsoil response. During discussions, the results of a laboratory model test conducted at Saga University, Japan, are referred.

TEST EMBANKMENTS AT LIAN-YUN-GANG

Subsoil Condition

The test site is located in an alluvial plain near the Yellow-sea, Lian-Yun-Gang area, Jiangsu province, China. The region belongs to lowland with an altitude of 2.6 m. The subsoil is uniform with a soft deposit about 10.5 m thick. The ground water level was fluctuated between 0.5~1.0 m below ground surface. The soil profile consists of 2.0 m thick mucky clay crust underlain 8.5 m thick soft muck layer. Below the soft layer there are medium to stiff sandy clay and silt sand layers. The physical and mechanical properties of the soft deposit are summarized in Table 1. The water content, plasticity index, void ratio, vane shear strength, and sensitivity distribution with depth are

Table 1 Physical and mechanical properties of the subsoil

Soil Layer	Thickness (m)	Water Content (%)	Unit Weight (kN/m^3)	Void Ratio e	Plasticity Index I_p	Liquidity Index I_L	Coefficient of Consolidation (10^{-3}m^2/day) C_h	C_v	Compression Index C_c	Vane Shear Strength (kPa)
Crust	2.0	39.8~58.5	16.0~18.7	1.13~1.64	19.6~27.2	0.95~1.59			0.27	4.5~37.6
		48.9	17.0	1.4	23.9					20.1
Muck	8.5	51.1~68.4	15.5~17.7	1.44~1.99	19.7~33.2	1.04~1.95	3.7~8.4	2.7~4.8	0.36~0.62	4.5~25.9
		69.3	16.3	1.68	25.8		6.0	3.6	0.50	14.8

shown in Fig. 1, which indicates that a weak zone is in a depth of 2 m to 6 m. The top crust is in an overconsolidated state due to weathering and aging. Below 2.0 m, the soil is in a normal to slightly overconsolidated state with an overconsolidation ratio (*OCR*) of 1.04~1.07.

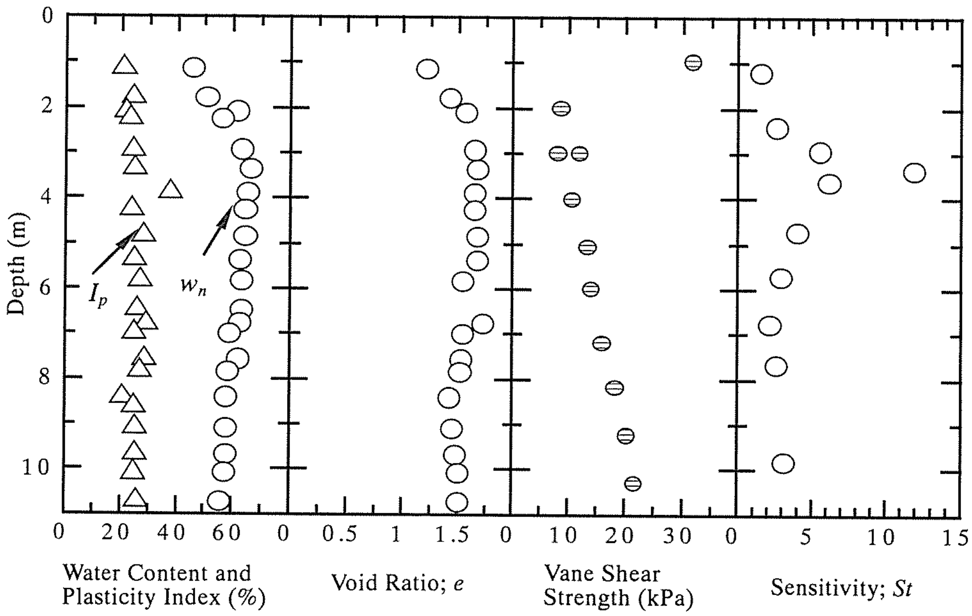


Fig. 1 Index and mechanical properties of subsoil

Test Embankment Construction

Total length of the test section was 270 m and each subsection had a length of 45 m. The subsection with a layer of geotextile reinforcement at the base of embankment, and the subsection on natural subsoil were built to failure. The results of two built to failure subsections will be presented in detail, and the results of other subsections will be referred during discussion if necessary.

Two built to failure embankments had a base width of 42 m. After placing a 0.5 m thick sand mat, a berm with a width of about 8.0 m was left on both sides and the embankments were built to failure. The embankments had a 1V:1.75H slope. The fill material was sandy clay. The unit weight of compacted fill material was about 18 kN/m^3. The average filling rate was about 0.1 m/day. The embankment on natural subsoil failed at a fill thickness of 4.04 m and 4.35 m for geotextile reinforced case. The embankments were instrumented with surface settlement gages, piezometer points, and casing for lateral displacement measurement. The embankment geometry, the location of reinforcement, and the main instrumentation points for geotextile reinforced case are illustrated in Fig. 2. Two types of geotextiles were used. One was a woven polypropylene geotextile with a unit weight of 303 g/m^2. In-air tensile strength from wide strip test was 40 kN/m and failure strain was about 18% at a strain rate of 2%/min. Another was a heat-bounded non-woven geotextile with a unit weight of 260.8 g/m^2. In-air tensile strength from wide strip test was 38.5 kN/m and failure strain was about 20%

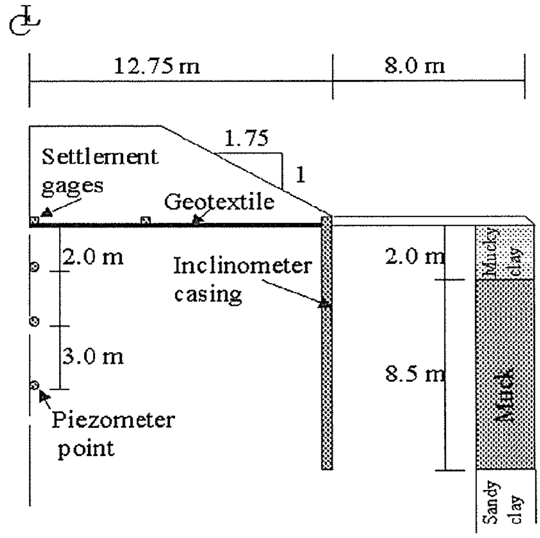


Fig. 2 Embankment geometry and the main instrumentation points

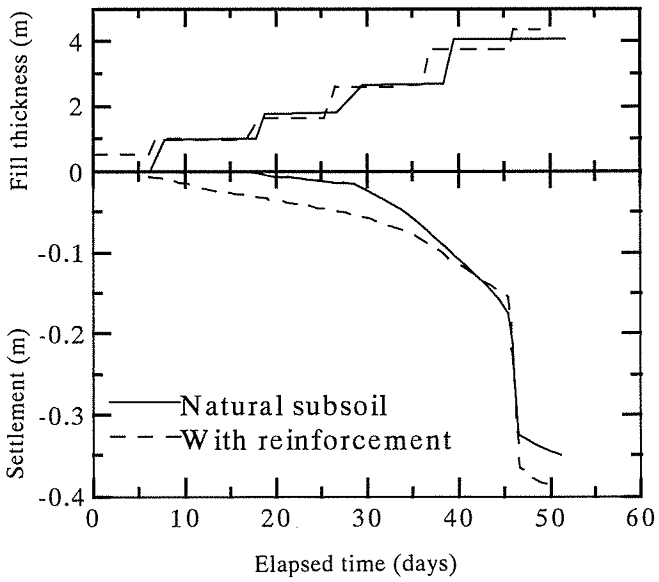


Fig. 3 Surface settlement versus elapsed time curves

Field Observed Results

(1) Surface settlement: The measured surface settlement-time curves under embankment centerline together with embankment construction histories are shown in Fig. 3. Following observations can be made from the figure. (a) Before fill thickness exceeded 2.0 m, the settlement was small. The top crust is stiffer and when the surcharge load was less than the yielding stress of the crust, the crust limited the subsoil deformation. (b) When the embankment approaching failure-state, the settlement was rapidly increased due to lateral distortion of the subsoil. (c) Just before failure-state, the geotextile reinforcement slightly reduced the settlement rate. It can be partially explained as that the combination of geotextile (strong in tension) and top

crust (strong in compression) strengthened the effect of the top crust.

(2) Lateral displacement: At the time of writing this paper, only the lateral displacement data for geotextile reinforced embankment are available. The lateral displacement profiles of reinforced case are given in Fig. 4. It indicates that the lateral displacement was small before the fill thickness reached the 4.35 m (failure occurred). Figure 5 gives the variation of the ratio of lateral displacement/settlement with embankment fill thickness. This case history also clearly indicates that when the embankment approaches failure condition, the lateral displacement/settlement ratio increased rapidly. The results support the proposal made by Matsuo and Kawamura (1977).

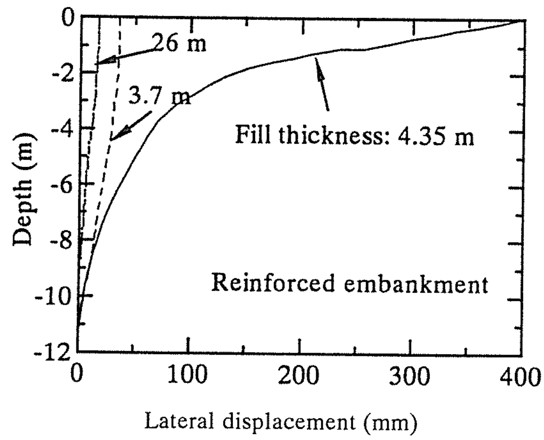


Fig. 4 Lateral displacement profiles

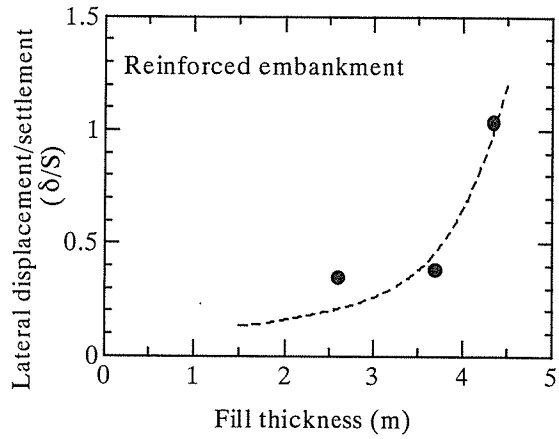
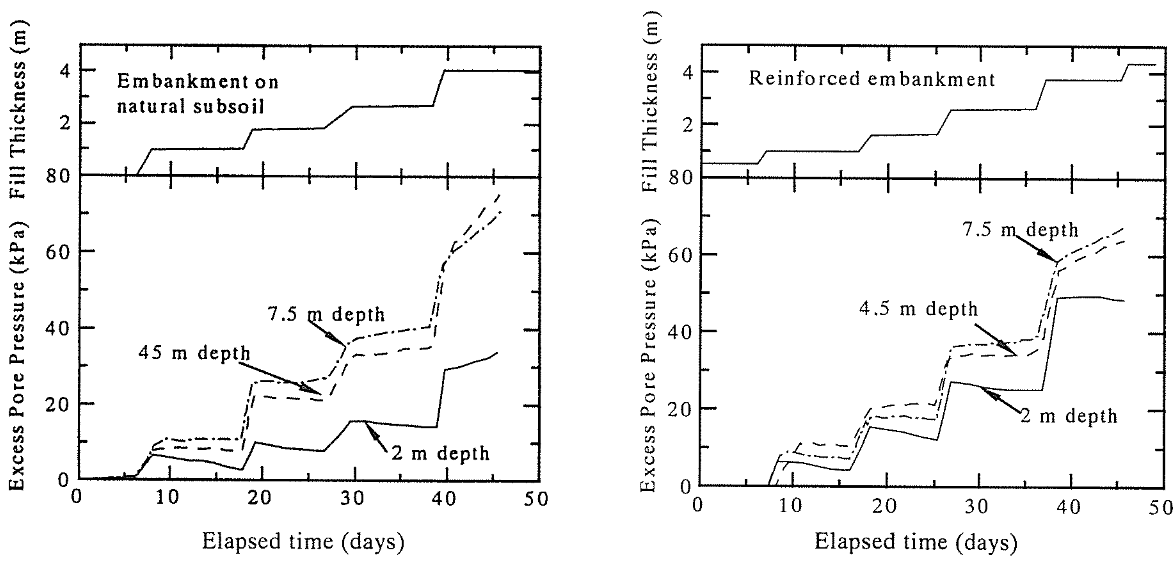


Fig. 5 Lateral displacement over Settlement ratio



(a) Embankment on natural subsoil

(b) Embankment with reinforcement

Fig. 6 Excess pore pressure variation

(3) Excess pore pressures: Variations of excess pore pressure at 2.0 m, 4.5 m, and 7.5 m depths are given in Figs. 6 (a) and (b) for with and without reinforcement cases, respectively. When fill thickness was less than about 3.0 m, at rest period between two load increments, there was a tendency of dissipation of excess pore pressure due to

consolidation effect. However, when fill thickness was approaching or more than 3.0 m, even at rest period, there was no reduction or an increase in excess pore pressure at 4.5 m and 7.5 m depths. When the fill thickness approached 4.0 m, there was an obvious increase of excess pore pressure during rest period. It can be explained that due to the progressive development of shear strain in subsoil, the shear induced excess pore pressure increment was larger than partial dissipation effect. Also, when the subsoil approaches failure state, the coefficient of consolidation is reduced due to reduction of soil stiffness, which will reduce the dissipation rate of excess pore pressure. The excess pore pressure versus fill thickness is given in Fig. 7 for 4.5 m depth. The rate of excess pore pressure development is increased with the increase of fill thickness, which can be explained by using the inserted figure in Fig. 7. The shear induced excess pore pressure will increase rapidly with the increase of shear stress level. The continuous development of excess pore pressure under constant surcharge loading can be considered as an index of embankment instability. The tendency shown in Fig. 7 partially supports the proposal made by Tavenas and Leroueil (1980) on excess pore pressure development in soft subsoil under embankment loading.

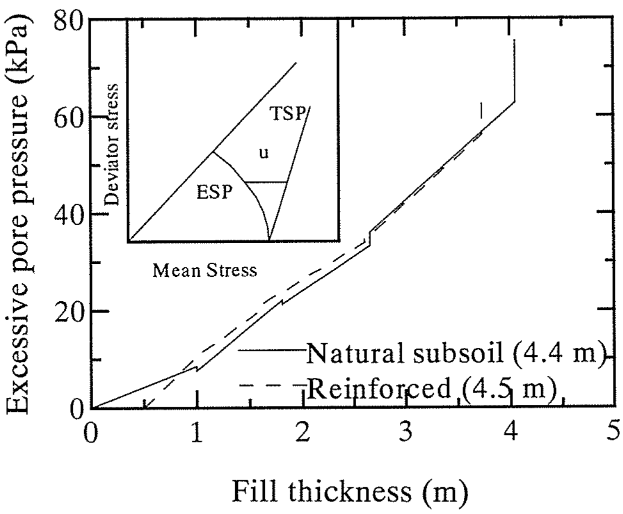


Fig. 7 Excess pore pressure versus fill

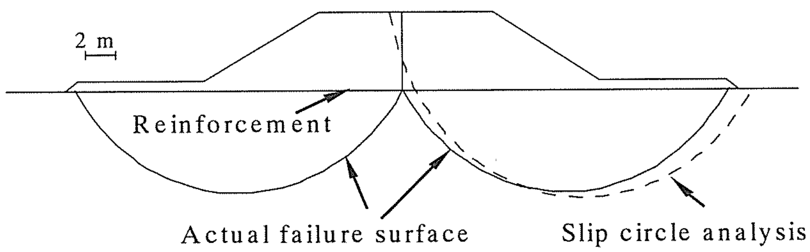


Fig. 8 Failure surfaces

(4) Failure surface: Symmetric failure pattern was observed in the field, i.e. the embankments failed at both sides. The observed failure surfaces are shown in Fig. 8. The estimated depth of failure surface is about 5~6 m. The observed failure surfaces

are not much difference for with and without geotextile reinforcement. It supports the assumption made for conventional stability analysis that the existence of geotextile reinforcement does not change the failure surface much.

DISCUSSIONS

Mobilized Tensile Force in Reinforcement

When discussing the mobilized tensile force, the strain compatibility between reinforcement and soil must be considered. Rowe and Soderman (1985) pointed out that the allowable compatible strain is a function of both subsoil properties and the geometry of embankment. From their finite element analysis results, the allowable compatible strain is in a range of 1% to 9%. By review some of the published data, Bonapare and Christopher (1987) suggested that the allowable strain is mainly controlled by subsoil properties. The recommended values are 2~3% for sensitive subsoil, 4~6% for medium to low sensitive subsoil, and 6~10% for non-sensitive plastic subsoil. According to this suggestion, a limiting strain of about 5% can be considered for the case presented here. Another factor should be considered on determining the mobilized tensile force is the in-soil stiffness of a geotextile. It is generally agreed that the soil confinement will increase the friction resistance between fabrics of geotextile and therefore, the stiffness of geotextile. Some of the methods proposed for determining the in-soil stiffness of geotextile seem not properly considering the friction resistance between soil and geotextile. Miura and Chai (1999) proposed a method for determining the in-soil stiffness of geotextile by combining a small-scale soil/geotextile interface shear test and large-scale pullout test results. The reported data showed that for tensile strain within 3~5%, the in-soil stiffness was 2~3 times of in-air one. Beyond this limiting strain, the in-soil tangent stiffness was the same as in-air value. For the case discussed here, assuming that the in-soil stiffness is twice of in-air value and with 5% limiting strain, the estimated mobilized tensile force just before embankment failure would be 20~25 kN/m. Considering the creep effect, Rowe and Soderman (1985) suggested that the allowable tensile force should not exceed 60% of the ultimate capacity of geotextile determined from a wide strip tension test at a strain rate of 2%/min. The value estimated satisfies this requirement also.

Effect of Reinforcement on Stability

Practically, the limiting equilibrium method is used in analyzing the factor of safety of embankment on soft subsoil. The key point is how to select a proper mobilized strength of subsoil as well as fill material. In field, the failure surface is formed gradually, i.e. progressive failure. Also, due to strain softening characteristics of most natural clay deposit and cracks occurring in fill material, determining the mobilized strength at failure is a complicated task. For the purpose of discussing the effect of reinforcement on embankment stability, the values of soil strength listed in Table 2 were used. The subsoil strength was vane shear strength multiplied by a factor of 0.8. With the strength given in Table 2, the factors of safety of 1.0 and 1.01 were obtained for embankments with and without reinforcement, respectively. The most proposed methods for analyzing the stability of reinforced embankment on soft subsoil (e.g. Milligan and Rochelle 1984; Rowe and Soderman 1985), only consider the restoring moment due to mobilized reinforcement tensile force. This is because with a total stress analysis, the reinforcement may not increase the undrained shear strength of subsoil much. It may have certain confining effect on the granular fill material, but its magnitude cannot be quantified without a numerical analysis. The same assumption was adopted in analyzing the reinforced embankment here. The mobilized tensile strength of 25 kN/m with horizontal orientation was assumed. The analysis revealed the followings. (1) A tensile force of 25 kN/m can only increase the factor of safety about 0.035. With in-air tensile strength from wide strip test of 40 kN/m, an improvement of 0.055 was resulted. The absolute value of the increase on factor of safety due to reinforcement is related to the magnitude of sliding mass. If the amount of mobilized tensile force is the same, the smaller the sliding mass, the larger the effect. (2) The same increase on the factor of safety can be obtained by increasing the strength of subsoil about 0.5 kPa (5% increase). Ladd (1991) proposed an empirical equation for estimating the undrained shear strength (S_u) of clay as follows:

$$S_u = S(OCR)^m \sigma'_v \tag{1}$$

where S and m are constants, OCR is overconsolidation ratio, and σ'_v is vertical effective stress. Normally, the value of S is in a range of 0.2~0.3, and m is from 0.75 to 1.0. Using S of 0.3 and m of 0.8 and OCR of 1.0, 0.5 kPa increasing in strength requires an increase in effective vertical stress of about 2 kPa or increase the degree of consolidation by 5% (assume 40 kPa average vertical stress increase in subsoil due to embankment loading). This amount of increase in the degree of consolidation can be obtained by reducing the construction speed, or using vertical drain improvement. At the same site, PVD improved subsections with a fill thickness of 6.5 m, did not show any sign of embankment instability.

Table 2 Strengths for stability analysis

Depth (m)	Cohesion (kPa)	Friction angle (Degree)
0.0-1.0	24.0	0
1.0-2.0	16.0	0
2.0-3.0	7.6	0
3.0-4.0	8.0	0
4.0-5.0	9.2	0
5.0-6.0	10.6	0
6.0-7.0	12.0	0
Fill material	5.0	20

Confining Effect of Reinforcement to the Soil Adjacent to It

Although in stability analysis, the effect of reinforcement to the strength of soil is normally not considered, it is generally agreed that the reinforcement has certain functions on reducing the lateral displacement of subsoil. Milligan and La Rochelle (1984) illustrated that the existence of reinforcement can reduce the fill material induced shear stress on ground surface, and then the shear stress level in subsoil, which contributes to reduce the lateral displacement. However, the amount of soil/reinforcement interface shear stress can be mobilized is controlled by the relative stiffness of soil and reinforcement, depth of soft subsoil, and the factor of safety of an embankment. There is no simple close form solution for this soil/reinforcement interaction problem. Generally, the weaker the subsoil, the more effect of reinforcement on subsoil deformation. The field data of this case history indicated that only under the condition of unreinforced case approached to failure, there was a noticeable difference on subsoil deformation for with and without reinforcement cases. In practice, a factor of safety of 1.2 to 1.3 is normally required for embankment over soft subsoil. For the case studied, a factor of safety of 1.2 corresponds to an embankment height of 3.2~3.5 m. Field observation showed at that stage, there was no obvious difference on subsoil response between two cases. As shown in Fig. 9, normally the stress-strain curve of soil is non-linear. Assuming that the reinforcement can reduce certain amount of shear stress ($\Delta\tau$). At point 1, with a higher factor of safety (about 1.2), the reduction of shear stress $\Delta\tau$ can result in a reduction of shear strain of $\Delta\varepsilon_1$. However, at point 2 with a factor of safety close to unit, $\Delta\tau$ can result in a larger shear strain reduction $\Delta\varepsilon_2$ ($\Delta\varepsilon_2 >> \Delta\varepsilon_1$). Therefore, at working state, the main benefit of using reinforcement is to increase the stability of an embankment.

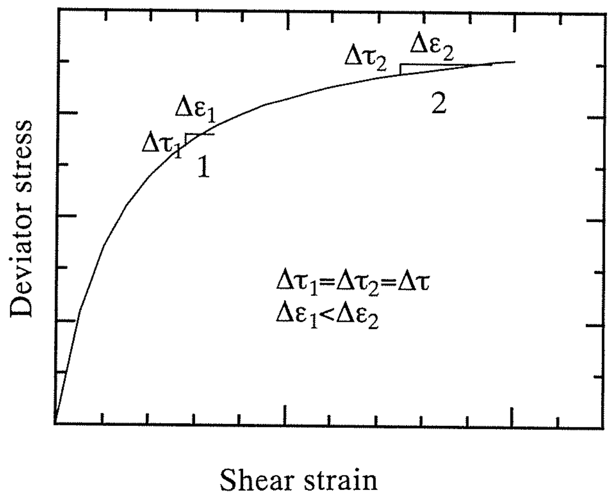


Fig. 9 Illustration of the effect of stress level

Since the measured field data are limited and the field conditions for reinforced and unreinforced cases are not completely identical; a laboratory model test was conducted at Saga University, Japan, to further clarify the effect of reinforcement on subsoil response. The set-up of the model test is illustrated in Fig. 10. A model box made by transparent acrylic has an inner dimension of 1.5 m in length, 0.6 m in width, and 0.8 m height. A acrylic plate was fixed at the middle of the box along length direction to form two separated sub-model boxes with a width of 0.3 m. The 2 layers of geotextiles were placed at the bottom and two end vertical boundaries as drainage layers. To measure the lateral displacement, the paper grids were put on the inner side of both side-walls. Three (3) clay layers sandwiched two (2) thin sand layers formed the model ground. The thickness of each clay layer was about 150 mm, and about 30 mm for each sand layer. The clay used was remolded Ariake clay with a water content of about 130~140% (larger than liquid limit of about 105%). The plastic limit is about 57%. The model ground was consolidated under 10 kPa dead load for about 2 months. After the primary consolidation was finished, the dead load was removed and the soil was sampled for strength and compressibility test. The test results indicate that the model ground had a compression index (C_c) of 0.8, void ratio (e) of 3.0, and undrained shear strength (Su) of 4.5 kPa to 5.5 kPa (laboratory vane shear test).

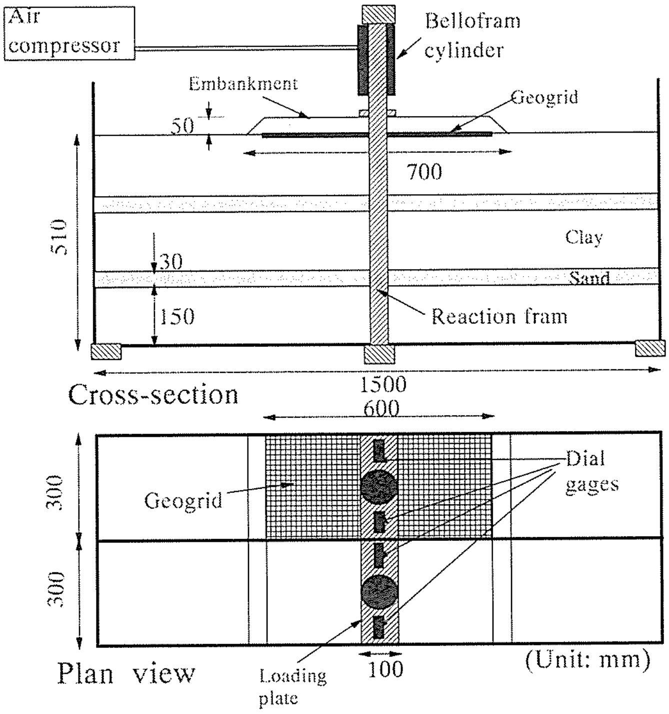


Fig. 10 Illustration of model test set-up

Model embankments with a height of 50 mm and base width of 700 mm were built on both sub-models using sand as fill materials. However, one sub-model had one layer of geogrid between the model ground and the embankment. The geogrid used has a grid size of 6 mm by 6 mm, tensile strength of 5.2 kN/m (strain rate 1%/min). The stiffness is about 300 kN/m for less than 1% tensile strain condition. Considering the scale of the model (about 1/20 to 1/30 of prototype), the reinforcement was very strong and it can be regarded as "fully reinforced" (Jewell 1988). On top of the model embankment, a 100 mm wide loading plate was settled at the center. The load was applied stepwise by air pressure though bello-frame cylinder with an increment of 15 kPa and the loading duration under each increment was about one week. The same loading condition was maintained for both sub-models.

The measured settlement variation and lateral displacement profiles for after applying 60 kPa loading are given in Figs. 11 and 12 respectively. Figure 11 shows that before load increased to 45 kPa, the settlements for both with and without reinforcement are identical. After the load reached 45 kPa, the settlement difference was clearly appeared. At the surface of the model ground, the undrained shear strength was about 5.2 kPa, and the plasticity theory gives an undrained bearing capacity of about 27 kPa. Take the effect of partial consolidation during loading period and the stress spreading effect of model embankment, a bearing capacity of about 40 to 50 kPa can be roughly estimated. When load increased to 45 kPa, for the case without reinforcement, the subsoil was close to failure. For with reinforcement case, due to the restriction effect of geogrid, the surface soil still not failed. Therefore, the settlement difference of two cases appeared. For further increasing the load to 60 kPa, the settlement difference increased rapidly, and which was mainly due to the difference of lateral displacement as indicated in Fig. 12. The model test results clearly illustrated that: (1) when the factor of safety is large, even with very strong reinforcement, there is no noticeable effect on subsoil deformation. (2) When the factor of safety is close to 1.0 for unreinforced case, the reinforcement can have considerable benefit on reducing the lateral deformation of subsoil if the reinforcement is strong enough.

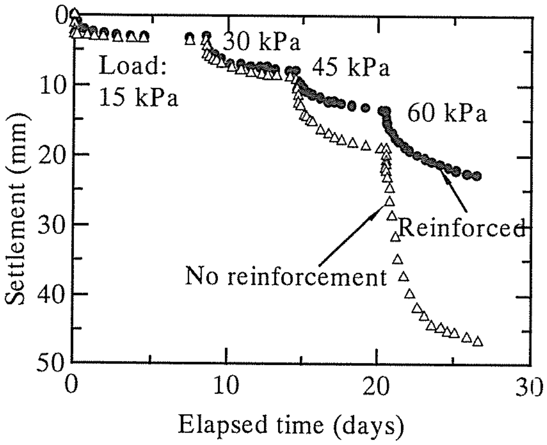


Fig. 11 Settlement versus elapsed time curves of model test

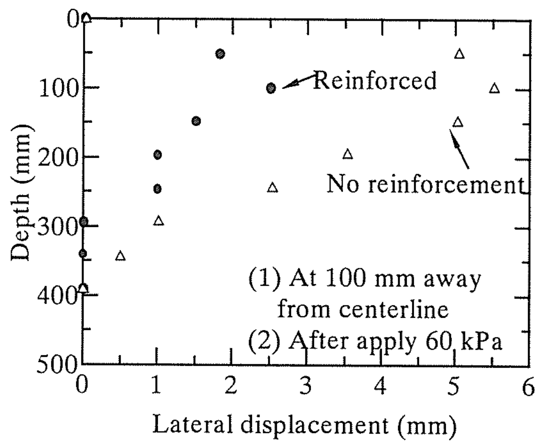


Fig. 12 Lateral displacement profiles of model test

CONCLUSIONS

The results of test embankments both with and without geotextile reinforcement on soft muck deposit, Lian-Yun-Gang, China, are reported. The field data together with some analyses revealed the followings:

1. The geotextile reinforcement certainly can increase the stability of embankment on soft deposit. The amount of effect is a function of mobilized tensile force in reinforcement and the geometry of the embankment. For the case studied here, the geotextile reinforcement might only increase the factor of safety about 0.035. The field measurements indicate that the rate of lateral displacement and excess pore pressure development in subsoil are sensitive indexes for embankment stability.
2. The base reinforcement will only have a beneficial effect on subsoil deformation when the unreinforced case is close to failure. At working state with a factor of safety of 1.2 to 1.3, both field full-scale test and laboratory model test results indicate that there was no obvious effect of reinforcement on subsoil deformation. The laboratory model test shows that when unreinforced case approached to failure, reinforcement can have considerable constriction effect on subsoil lateral deformation if the reinforcement is strong enough.
3. It is suggested that to obtain a substantial improvement on embankment behavior, the multi-layer geotextile reinforcements or high strength and stiffness geogrid need to be used.

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