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Application of geosynthetics to embankment on soft ground and reclamation using soft soils

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ABSTRACT: This paper first outlines how to apply geosynthetics to embankments on soft ground and then positions this reinforced earth method in the array of measures used to improve soft ground for embankments based on a feasibility study. Concerning the two most suitable methods of application, namely, the soft ground surface layer reinforcing method using geogrid in combination with vertical drains and the method of preventing settlement using geosynthetics in combination with the deep mixing method, the design concept and method of estimating deformation are outlined. In addition, as a method of reclaiming and quickly using land made of low-quality soils from construction sites, the concept, design and execution of a new method of reclamation using horizontal drains in combination with sand mounds to lower the water level are outlined.

1 OUTLINE OF METHOD FOR APPLICATION OF GEOSYNTHETICS TO EMBANKMENTS ON SOFT GROUND

It has been said that the Japanese geotextile method originated with the method of reinforcing soft ground surface layers by means of an earth cover; the sheet method began to be used as a treatment method for soft ground surface layers for land reclaimed by means of pump sledging in the 1960s.

In the West, on the other hand, non-woven geotextiles were first widely used for preparation of roads on soft ground located in wetlands. Non-woven geotextiles were used more often in wetlands than in reclaimed land, and the method of using non-woven geotextile—which functioned as an excellent boundary separator—was developed.

In Japan, the method of using sheets in combination with rope nets and the method for net-matting embankments using geonets were developed subsequent to the sheet method; the method using sheets in combination with the traditional bamboo frame has proven valuable as a method for reinforcing embankments on extremely soft ground. These methods have also been used to reinforce soft ground under the sea, for example, when harbor facilities such as breakwaters and seawalls are built

Applications of geosynthetics for embankments on soft ground, roughly classified into four types, are listed in Table 1.

1.1 *Soft ground surface layer reinforcing method for earth cover*

(1) Outline of the reinforcing method

In the soft ground surface layer reinforcing method for earth cover, in order to ensure trafficability on soft ground where it is almost impossible to put people or construction machines, geosynthetic reinforcing materials are laid on the ground surface, and the ground is then reinforced using the horizontal continuity and tensile strength of the materials; earth is then spread over the ground. The tensile force generated by the load of covered earth will either serve as the tension member of the beam or alternatively, will produce a hammock effect as the part where the earth is placed sinks and the ground level is raised on the periphery because the edges are supported, resulting in reinforcement of the ground.

In the sheet, net matting, and rope/sheet methods, the tensile force produced at the edges is handled through friction between the ground and the reinforcing materials or by fixing it to an anchor pile embedded in the embankment. The bamboo frame/sheet method is one in which the covered earth is supported by allowing the rigidity of the bamboo frame to disperse the load of covered earth so that the load is evenly distributed.

The reinforcement work consists of installing reinforcing materials and spreading the earth used to cover the ground. Even on ground where people are not able to stand, reinforcing materials may be laid by a variety of methods, including one that requires reinforcing materials to be laid manually after they are spread and scaffolding plates are placed on them, a

Table 1. Application of geosynthetics to soft ground

Applications	Method and materials used	
Reinforcement of soft ground surface layers for earth covers • To ensure trafficability on soft ground where it is difficult for people and motor vehicles to pass • To ensure sufficient bearing capacity to support light-weight objects	Sheet method Net matting method Sheet/rope net method Sheet/bamboo frame method	(Woven geotextile, etc.) (Geonet) (Geo grid) (Woven geotextile, etc., ropes) (Woven geotextile, etc., bamboo)
Reinforcement of soft ground surface layers for embankments • To stabilize embankments on soft ground requiring more strength than that afforded by soft ground with a reinforcing earth cover; and also for control of differential settlement.	Plane reinforcement Single reinforcement material Multiple reinforcing materials Solid reinforcing	(Geotextile or geogrid) (Reinforcing bars with anchor plates) (Reinforcing bars with anchor plates) (Mattress), (Geocell)
Reinforcement of soft ground surface layers by means of geo-synthetics used in combination with piles • For high, permanent embankments on soft ground having the same strength as soft ground that needs earth cover for reinforcement	Pile net method Deep mixing pile/net method	(Wire mesh, geotextile, etc.) (Geotextile, etc.)
Reinforcement of soft ground for base courses and subgrade	(Roads) Reinforcement of bearing capacity of base courses Reinforcement of bearing capacity of subgrade (Railroads) Reinforcement of bearing capacity of base courses Prevention of mud pumping on base courses	 (Geogrid and woven geotextile) (Geo-cells, non-woven geotextile, woven geotextile, geomembrane)

method in which a winch is installed on the opposite bank to pull and lay the reinforcing materials, and a method in which the ground is covered with water, and reinforcing materials are loaded onto a barge and are subsequently placed at the bottom of the water as the barge moves about.

(2) Design concept

If reinforcing materials and a load of earth cover are applied to soft ground with a high water content (such as reclaimed land where cohesive soil dredged from the bottom of the sea is used as reclamation material) the part directly below the load tends to sink and the ground around the part rises. Models have been developed to analyze such ground deformation phenomena, and different design methods have been proposed as a result. However, since deformation phenomena differ greatly depending on soil and working conditions at the site, it is difficult to fully incorporate all the details underlying the supporting mechanism, and therefore various constants used in the formulae to calculate bearing capacity are determined based on measurements of behavior at the site and on the results of indoor experiments.

The following proposals have been made with regard to formulae used to calculate bearing capacity and settlement.

- Method based on the theory of ground bearing capacity: The tensile force and the effect of pressing down on the ground and the effects resulting from settlement and rising on embedment are taken into consideration, based on the ground bearing capacity formula.
- Method using a combination of soil reaction coefficients and the cable theory: After a load is applied, a basic formula is obtained from the deformed shape of the stabilized reinforcing material using cable theory, and the tensile force of the reinforcing material is calculated from its relation to the amount of settlement.
- Method based on slab theory: In this method, the slab theory is applied on the assumption that sandy soil—used as earth cover— integrated with reinforcing material with relatively large meshes assumes a rigid slab-like shape.
- Method based on pneumatic membrane theory: In this method, an analytical theory is applied when extremely soft ground is considered as a liquid, and air pressure acts on the membrane structure whose periphery is confined.

Of all these methods, the method based on the bearing capacity theory is used comparatively often. In this method, a model is developed for the ground, and the bearing capacity is expressed as the sum of four components.

Bearing capacity of conventional ground:

$$q_1 = c \cdot Nc \tag{1.1}$$

Bearing capacity resulting from tensile force generated at both ends of the reinforcing material:

$$q_2 = 2 \cdot T \cdot \sin\theta / B \tag{1.2}$$

Effect of reinforcing material pressing down the ground:

$$q_3 = T \cdot Nq / r \tag{1.3}$$

Embedment effect resulting from settlement and rising:

$$q_4 = r_t \cdot Df \tag{1.4}$$

Where c: denotes the cohesion of soft ground (kPa) {tf/m^2}, Nc, Nq: the bearing capacity factor, T: the tensile force of the reinforced material, θ the angle formed by the reinforcing material and the horizontal surface at the end of the load (°), B: the load width (m), r: the radius of the deformed shape of the ground near the load when the shape is considered as circular (m), r_t: the weight per unit volume (kPa){tf/m^2}, and Df: the amount of settlement of the soft ground (m).

Thus, the ultimate bearing capacity is calculated as follows.

$$q_d = q_1 + q_2 + q_3 + q_4$$
$$= c \cdot Nc + 2 \cdot T \cdot \sin\theta/B + T \cdot N_q/r + r_t \cdot Df \quad (1.5)$$

T in this formula is determined based on the measurements taken at the site and the indoor experimental values as there are three unknown constants generated by the state of deformation of the ground, namely, r, θ, and Df if the strength of the material to be used is employed.

As to the design procedure, ground strength is determined first of all, and then the partition embankment and the earth cover—in the block surrounded by the partition embankment—are designed. When designing the earth cover, a combination of both the earth covering method and reinforcing materials suitable to the site conditions are selected; the earth cover, the layer thickness in each state, the reinforcing materials and work method are all determined.

(3) Tensile strength of reinforcing materials reviewed in examples of reinforcing work

Figure 1 shows the relationship between ground cohesion and the tensile strength of the reinforcing material used at the site with all data cited from examples of reinforcing work. For extremely soft ground with a the cohesion $c \leqq 0.1$ tf/m^2, a reinforcing material with the tensile strength $T = 7 \sim 11$ tf/m is used. The type of reinforcing material is rope, bamboo frame and sheet, or geogrid; there is no case in which a sheet alone is used. This is because it is assumed that since the sheet lacks rigidity, part of the ground sinks when earth is spread, and this settlement cannot be controlled.

On the other hand, if cohesion $c > 0.1$tf/m,the tensile strength of the reinforcing material ranges very widely from 1tf/m to 11tf/m.This indicates that a particular reinforcing material is apparently determined considering not only the ground cohesion but also the shape of each block of earth cover surrounded by the partition embankment as well as the method used for spreading the earth. As for the use of the sheet, a sheet with a smaller tensile strength tends to be used as the ground cohesion increases, but a geonet with a relatively narrow range of cohesion from 0.1tf/m^2 to 0.5tf/m^2 is used regardless of the maximum tensile strength of 2.0tf/m or so. It is assumed that this is closely related also to the large tensile elongation.

1.2 *Soft ground surface layer reinforcing method for embankments*

(1) Outline of the reinforcing work

The main objective of the above-mentioned "soft ground reinforcing for earth cover" is chiefly to ensure trafficability for motor vehicles used for the reinforcing work or for placing light-weight structures on the ground by utilizing a stabilizing earth cover of good

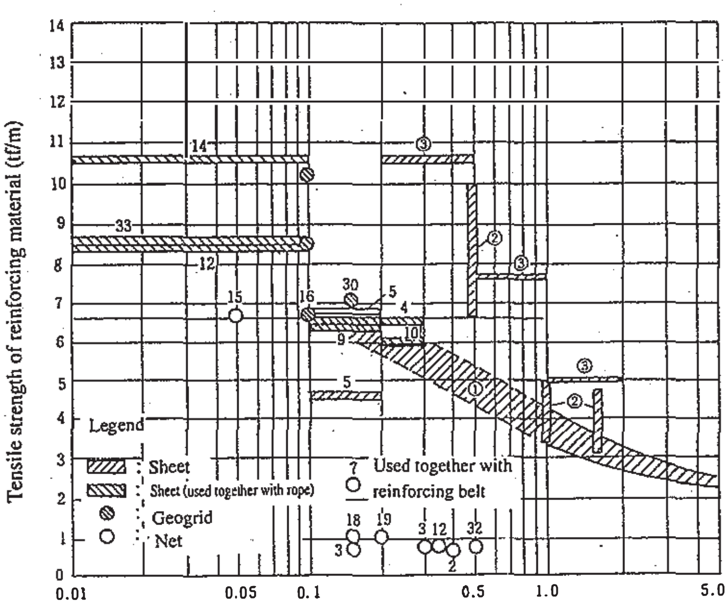


Fig. 1. Relation between ground strength and tensile strength of reinforcing material [1)]

quality over the geotextile laid on the soft ground surface. Therefore, the uneven thickness of the embankment or any differential settlement is tolerated because of its temporary nature.

On the other hand, the main objective of "soft ground reinforcing for embankment" is to stabilize the embankment and minimize the possibility of differential settlement by reinforcing the surface layer of the soft ground as well as the part closer to the bottom of the embankment. Therefore, the soft ground is required to have a certain degree of strength.

Application methods for geotextile can be roughly classified into 1) the method in which geogrid or some other material is laid on the surface layer of the foundation ground or on the lower layer of the embankment to ensure stability of the embankment with respect to sliding failure that occurs in the foundation ground (reinforcing material laying method); and 2) the method in which the foundation ground is reinforced by using a geogrid to construct grid frames in the surface layer of the foundation ground and putting filling material into the frames (mattress method).

(2) Design concepts

In designing 1) the reinforcing material laying method, stability with respect to the following three general failure modes is examined.[2)]

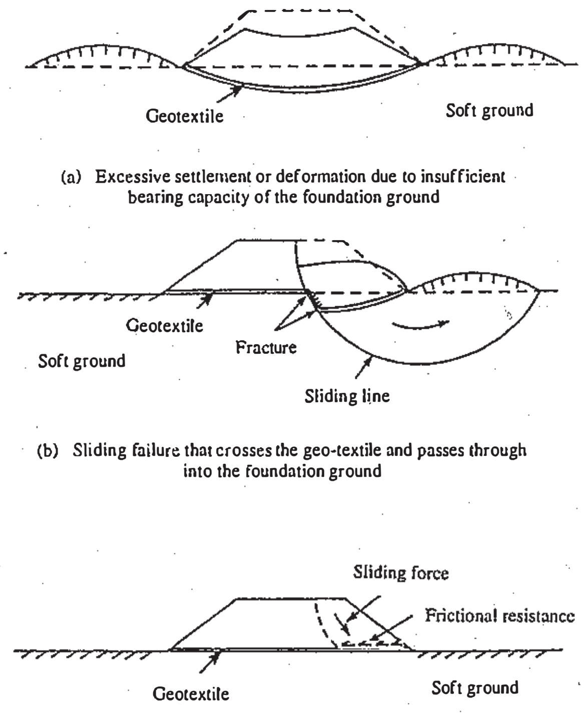
A) Excessive settlement or deformation due to insufficient bearing capacity of the foundation ground (see Fig. 2(a))
B) Sliding failure that cuts through the geotextile and passes into the foundation ground (see Fig. 2(b))
C) Embankment Sliding on geo-textile (see Fig.2 (c))

The basic design procedure is shown in Fig. 3.

First of all, whether excessive settlement or deformation is likely to occur is determined based on the value of safety factor Fs used in the circular arc sliding calculation at the time of reinforcing. It is generally accepted that if the value of Fs is below 1.0, excessive settlement or deformation as shown in Fig. 2(a) occurs and the effects of geotextiles are not significant in many cases. Therefore, improvements in the foundation ground itself must be examined initially.

In cases where a type A) failure is not likely to occur, but a type B) failure is more likely to occur ($1.0 \leqq Fs < 1.2 \sim 1.3$), geotextiles can be laid on the surface of the foundation ground (or in the lower section of the embankment) to compensate for the insufficient safety factor with the tensile strength of geotextiles.

The safety factor with respect to sliding failure in considering geo-textiles is obtained from the following formula. (See Fig. 4)



(a) Excessive settlement or deformation due to insufficient bearing capacity of the foundation ground

(b) Sliding failure that crosses the geo-textile and passes through into the foundation ground

(c) Sliding of embankment on geotextile

Fig. 2. Modes of failure in embankment on soft ground reinforced with geotextile

$$Fs = \frac{Mr + \Delta Mr}{M_d} \tag{1.6}$$

$$M_d = R \cdot \Sigma\,(W \cdot \sin\theta) \tag{1.7}$$

$$Mr = R \cdot \Sigma\,(c \cdot 1 + W \cdot \cos\theta \cdot \tan\phi) \tag{1.8}$$

$$\Delta Mr = R \cdot T_A \cdot \cos\alpha \text{ or } R \cdot T_A \tag{1.9}$$

Where, M_d : Sliding moment
Mr : Resisting moment of soil
ΔMr : Resisting moment of geotextile
T_A : Design tensile strength of geotextile, which must be determined from the tensile test but should be such that the tensile elongation in consideration of creep is kept below 10% or so.

A type C) failure is likely to occur when the frictional resistance between the embankment and the geotextile is small. Therefore, it is necessary to determine if the frictional resistance between the geotextile and the embankment is sufficient with respect to the sliding force of the embankment.

The safety factor with respect to embankment sliding on geotextile is determined from the following formula. (See Fig. 5)

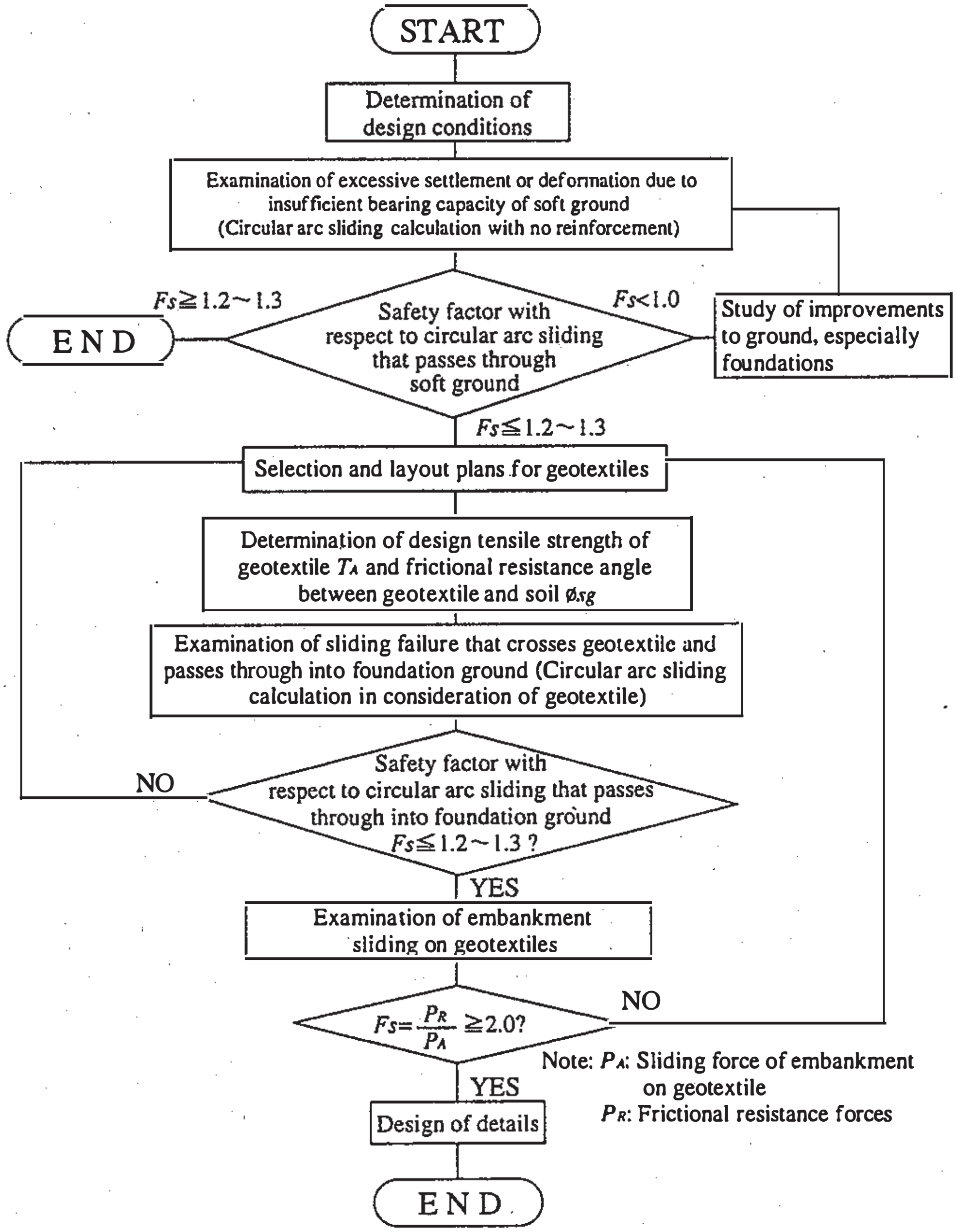


Fig. 3. Examination procedure undertaken when geotextiles are used to reinforce embankments on soft ground[2)]

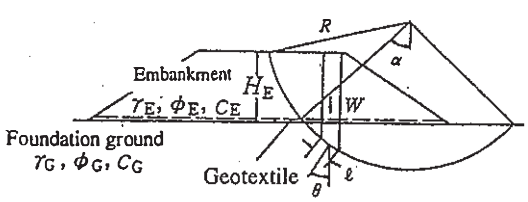


Fig. 4. Stability calculation with respect to sliding failures which pass through into the foundation ground[2)]

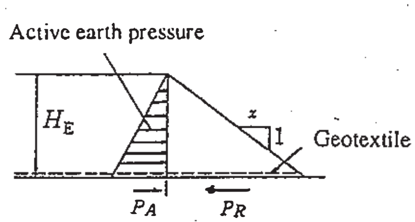


Fig. 5. Stability calculation with respect to sliding of embankment on geotextiles[2)]

$$Fs = \frac{P_R}{P_A} \quad (1.10)$$

$$P_A = \frac{1}{2} r_E \cdot H_E^2 \cdot K_A \quad (1.11)$$

$$P_R = \frac{1}{2} r_E \cdot x \cdot H_E^2 \cdot \tan \phi_{SE} \quad (1.12)$$

Where, P_A : Sliding force of embankment on geotextile (resultant force of active earth pressure)
P_R : Frictional resistance force
K_A : Coefficient of active earth pressure
ϕ_{sg} : Frictional resistance angle between the geotextile and the soil
x : Slope gradient

Formulas (1.10), (1.11) and (1.12) are used to determine whether the safety factor with respect to sliding of embankment on geotextile is greater or less than 2.0.

1.3 *Soft ground surface layer reinforcing method using geo-textiles with piles*

The above-mentioned soft ground surface layer reinforcing method for embankments is applied to soft ground with a relatively high strength, and reinforcing methods used to build full-scale, permanent embankments on soft ground with extremely low strength include the pilenet method in which geotextiles are used in combination with piles, or piles used in the deep mixing method of soil stabilization and a method in which geogrid and the deep mixing method of soil stabilization are used together.

(1) Pile net method

The pilenet method was developed to be applied to the peat layer and soft cohesive soil layer that have a high water content, high compressibility and low strength, as found in Hokkaido and along the Ariake coast in Kyushu. As shown in Fig. 6, wooden piles are driven into soft ground in groups; the top of each pile is linked by reinforcing bars, and a sand mat is spread over them. One or more geotextiles are then laid inside the sand mat in the form of a sandwich to increase the confining effect, and a full-scale embankment is constructed.[3)~5)]

Since this method uses long piles to enhance rigidity quite deep into the soft layer, it prevents or greatly reduces the lateral flow.

In the pile net method, the distance between piles D appropriate to produce the group pile effect is determined from the following formula.

$$D = 1.5\sqrt{r \times l} \quad (1.13)$$

Where r is the radius of the pile, and l the length of the pile.

From the above formula, if the radius of the pile is 10 cm, the distance between piles will be about 1.1 meters when the length of the pile is 5 meters, and 1.6 meters when the length of the pile is 10 meters. The length of the pile l must be determined such that the ultimate bearing capacity per pile Rd is obtained for a group of piles, and the value of the safety factor, which is the ratio of the ultimate bearing capacity for a group of piles to the load applied to each pile Rw, is more than 1.3. The connecting reinforcing bars must have a deflection angle of 30 degrees (θ=30°) so that T equals P as shown in Fig. 7. The tensile force T applied to the reinforcing bar is obtained from the following formula, and the type and the diameter of the reinforcing bar are determined so that S is equal to or greater than T.

$$T = \frac{0.5 \cdot Rw}{2 \cdot m} \quad (1.14)$$

Where, T : the tensile force per reinforcing bar,
Rw : the load applied to each pile, and
m : the number of reinforcing bars used (single m = 1, double m = 2).

$$S = \tau_t \cdot \frac{\pi d^2}{4} \quad (1.15)$$

Where, S : the allowable stress of the reinforcing bar,
τ_t : the allowable stress
SR24: 1400kgf/cm^2
SD30: 1600kgf/cm^2), and
d : the diameter of the reinforcing bar.

$$S \geqq T \quad (1.16)$$

Geotextile laid on the upper part of the piles prevents the embankment material from slipping through and distributes the load of the embankment evenly to the lower ground and the reinforcing bars; geotextiles with excellent contact with the ground, air permeability and weather-resistance are used. In some cases, the tensile force T_g applied to the geotextile is determined from the following formula for conveniences' sake since the actual behavior is not known.

$$T_g = \frac{\text{Overburden load applied to the space between the reinforcing bars}}{\text{The total length of the reinforcing bars supporting the geotextile}}$$

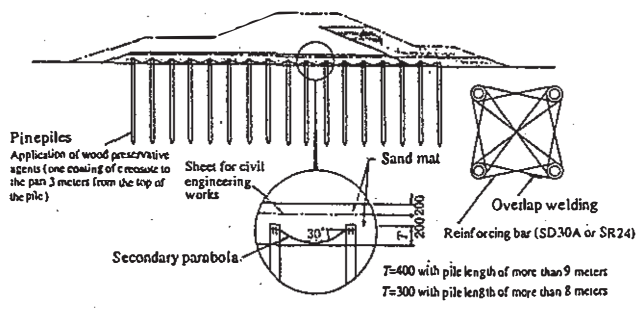


Fig. 6. Concept of the pile net method

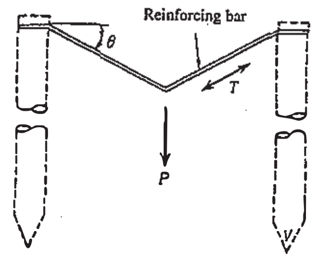


Fig. 7 Deflection angle θ of connecting reinforcing bars

When selecting a geotextile, a geotextile with a tensile strength of more than the value obtained in consideration of the safety factor for T_g (normally 1.2) is used.

As for the amount of settlement of the ground on which the pilenet is constructed, the portion surrounded by the piles is considered as a form of integrated caisson, and the ground settlement caused by the embankment load must be determined only for the section deeper than the tip of the pile inside the soft layer.

As for the drainage side required in the calculation of consolidation, the tip of the wooden pile is regarded as the drainage side in some cases on the assumption that, although the wooden pile itself does not have a drainage function, water will drain through the peripheral sides of the wooden pile. A simple calculation method such as this tends to produce a larger value than the actual measured amount of settlement, but this does not pose any problem for practical purposes, and is considered as generally acceptable.

(2) Method using geogrid in combination with deep mixing method of soil stabilization

Based on a concept similar to the pilenet method, a method is proposed in which a pile-type improvement is made to the entire section below the embankment by means of the deep mixing method of soil stabilization that offers a low improvement rate (about 10~30%). The lack of stability with respect to circular arc sliding is compensated for by a high-strength geogrid; geotextile is used to solve the problem of differential settlement that develops between the improved columns and the non-improved sections.

The design method is not yet fully established, but it is anticipated that this method will be made practicable as an economical and new solution to soft ground reinforcing. This method will be discussed in detail in Section 4.

1.4 *Soft ground reinforcing method for base courses and subgrade*

According to the guidelines governing asphalt pavement, if the design CBR for the subgrade section is less than 3, the soil is to be replaced with soil of good quality and a stabilization method is applied in many cases. When geotextile is used, geotextile is laid between the base course and subgrade (or in the base course) to reinforce the ground by preventing separation and confinement effects and thus compensating for insufficient subgrade bearing capacity. As to the design method, when geotextile is laid between the subgrade and the base course, the thickness of the lower base course must be determined so that the total amount of deformation of the pavement structure and the pavement surface will be the same as that of the normal subgrade (CBR $\geqq$ 3). When geotextile is laid inside the base course, the fundamental assumption is that the deformation of the paved road surface decreases because the elastic modulus of the base course around the geotextile increases, and thus the design must be predicated based on the multi-layer elasticity theory. Both cases use the results of indoor experiments, and verification through experiments at the site are desirable before the methods are used. Geotextile is used for unpaved roads such as temporary roads and access roads with low traffic volumes, and various design methods have been proposed. Basically, the thickness of the base course is determined so that the ultimate bearing capacity of the subgrade and the stress applied to the base course are well-balanced.

The objective in using geotextile in the reinforcement of railroad track beds is to keep the ballast well maintained. It is used to prevent intrusion and mud-pumping on the ballast track bed as well as to prevent deformation of the ballast track bed due to running trains. The surface of the track bed is covered by geotextile, such as a sheet, to prevent rain water from intruding on the track bed, and thus preventing the mud-pumping phenomenon. Replacement methods and stabilization methods are also often used to improve the bearing capacity of the track bed as in the case of roads when the value of K_{30} (obtained in the plate loading test) is less than 7 kgf/cm^3. There is also a method using geotextile in which the track bed is reinforced with geocell. Tests have proven that a track bed reinforced with geocell is effective with respect to repeated loading from running trains.

2 POSITIONING OF GEOSYNTHETIC REINFORCING METHODS IN THE ENTIRE SOFT GROUND IMPROVEMENT MEASURES

2.1 *Examination of low embankments and high embankments*

(1) Introduction

In recent years, the use of new soft ground improvement methods, such as the lightweight embankment method, and the embankment reinforcing method using geosynthetics, is increasing. However, it is not yet known how such new methods and conventional methods compare in terms of total cost including maintenance and repair expenses. Therefore, the authors designed various soft ground improvement methods for embankments on a trial basis, and examined applications of these different methods by comparing the construction cost including maintenance expenses, the work period, and the standing period by the end of which the residual settlement falls within the allowance.

(2) Outline of trial calculations

In the trial design, the improvement methods listed in Table 4 are designed in accordance with the design conditions shown in Table 3 for the specific cases shown in Fig. 8 and Table 2, and a comparative economic study of them was made. In the case of a low embankment (He=2m), the magnitude of the traffic equivalent load is set to 2.2tf/m^3, considering the settlement due to traffic loads.

(3) Results of the trial calculations

a) For low embankments with a relatively thin soft layer (see Fig. 9)

When the thickness of the soft layer is small, the surcharge method offers advantages since the method is inexpensive and has a short period during which the residual settlement falls within the allowance. However, the period during which the residual settlement falls within the allowance increases as the thickness of the soft layer increases, and thus it is better to promote the settlement by using a sand drain.

In addition, the surface layer stabilization method is effective if applied to ground with a thin soft layer because the depth of ground that can be improved is limited and the consolidation settlement below the improved layer cannot be controlled.

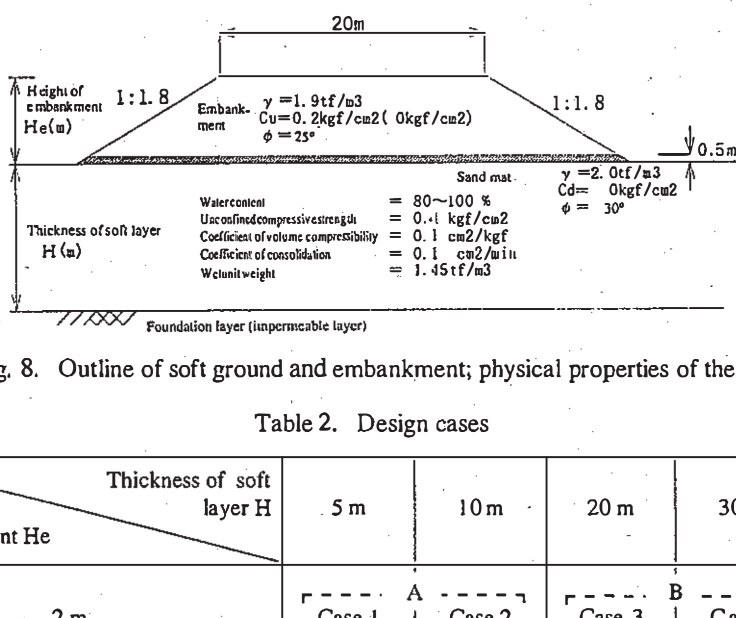


Fig. 8. Outline of soft ground and embankment; physical properties of the soil

Table 2. Design cases

<table>
<tr><th>Height of embankment He \ Thickness of soft layer H</th><th>5 m</th><th>10 m</th><th>20 m</th><th>30 m</th></tr>
<tr><td rowspan="2">2 m</td><td colspan="2">A</td><td colspan="2">B</td></tr>
<tr><td>Case 1</td><td>Case 2</td><td>Case 3</td><td>Case 4</td></tr>
<tr><td rowspan="2">5 m</td><td rowspan="2">-</td><td rowspan="2">-</td><td colspan="2">C</td></tr>
<tr><td>Case 5</td><td>Case 7</td></tr>
<tr><td>8 m</td><td>-</td><td>-</td><td>Case 6</td><td>Case 8</td></tr>
</table>

b) For low embankments with a thick soft layer (see Fig. 10)

The surcharge method was found to be the most inexpensive, but the time required for the residual settlement to fall within the allowance is extremely long especially in a case involving thick soft layers, such as this case. Therefore, it is better to use the surcharge together with a sand drain because the time required for the settlement to fall within the allowance can be significantly reduced.

In the case where a non-treated embankment requires maintenance in many locations, the expense for overlay increases and thus the surcharge used in combination with the sand drain is less expensive. In addition, the time required for the residual settlement to fall within the allowance is much shorter if the surcharge is used together with the sand drain. Therefore, it is advisable that the residual settlement be controlled from the beginning by means of a sand drain, etc.

Although the work cost is on the high side due to material costs, the method is considered effective for roads requiring severe allowances for residual settlement and also for roads where it is difficult to block traffic to carry out repairs.

c) For high embankment with thick soft layer (see Fig. 11)

If the embankment is as high as about 5 meters, the sand drain method and the sand drain method used in combination with the embankment reinforcing method or the counterweight fill method is comparatively inexpensive and offers a number of advantages.

Table 3. Design conditions

Item	Design conditions
(1) Examination of sliding failure	Safety factor $Fs \geqq 1.1$ (at the completion of embankment) $Fs \geqq 1.2$ (after the completion of embankment) Earthquakes have not been factored in.
(2) Allowable residual settlement	Less than 30cm
(3) Consolidation settlement	The consolidation settlement is calculated at the place below the embankment center, and settlement is considered as a one-dimensional consolidation.
(4) Embankment work speed	5cm/day However, the speed is 30 cm/day for the deep mixing method of soil stabilization and sand compaction pile method, and 150 m^3/day for the lightweight embankment (EPS) method.
(5) Strength increase rate for soft layer	m=0.3. The maximum value for the degree of consolidation U that increases the strength due to consolidation is 90%.
(6) Overlay standard for non-treated embankment	The number of areas that need maintenance per 1 km is set to a maximum of 20 and a minimum of 6. An overlay is required once for every 3 cm of settlement.

Table 4. Improvement methods examined

Group	Improvement methods
A	• Surcharge method • Surcharge + sand drain method (hereinafter referred to as SD) • Surcharge + surface layer stabilization method
B	• Surcharge method • Surcharge + SD • Lightweight embankment method (using EPS blocks) • Non-treatment (overlay after service is considered, maximum maintenance) • Non-treatment (overlay after service is considered, minimum maintenance)
C	• Lightweight embankment method (using EPS blocks) • Deep mixing method of soil stabilization } SD used together • Counterweight fill method } • Sand compaction pile method (SCP)

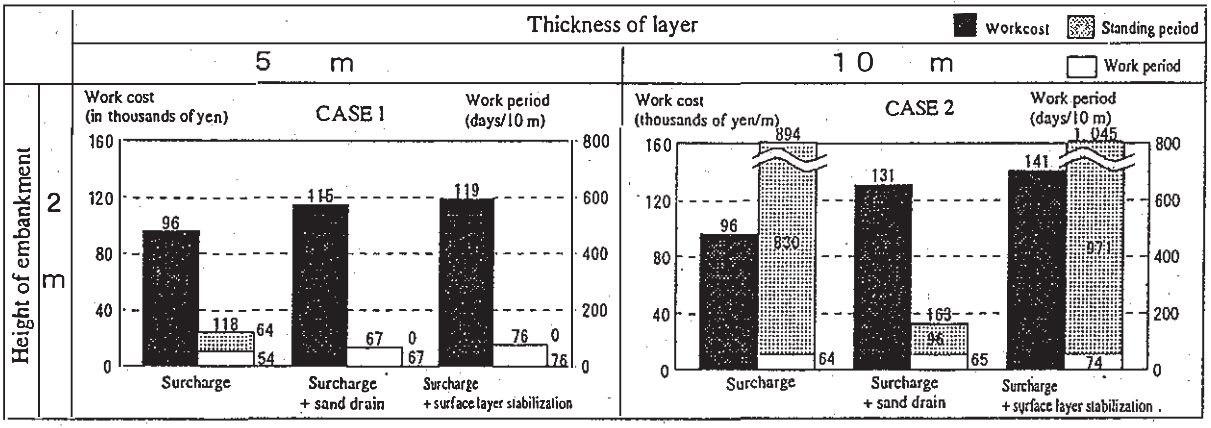


Fig. 9. Comparison group A (for low embankments with a relatively thin soft layer)

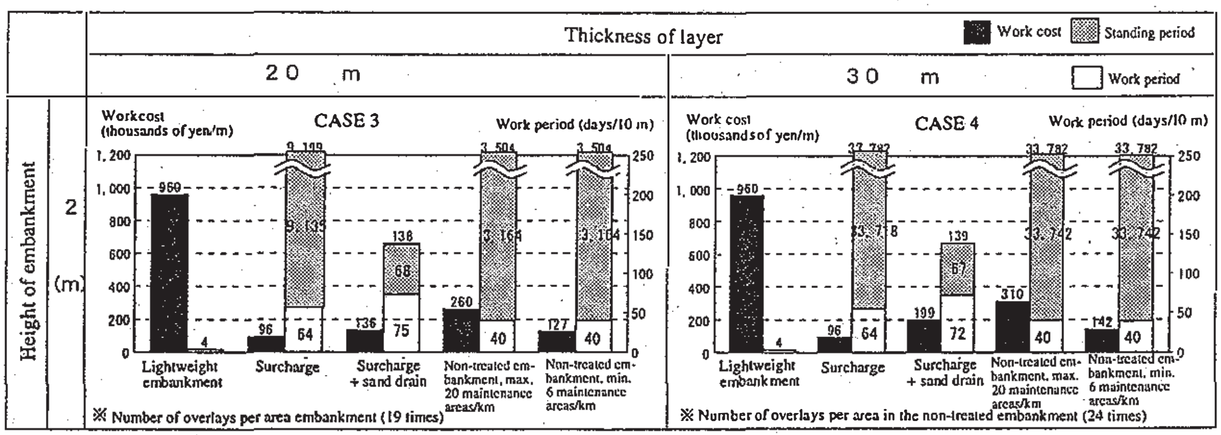


Fig. 10. Comparison group B (for low embankment with thick soft layer)

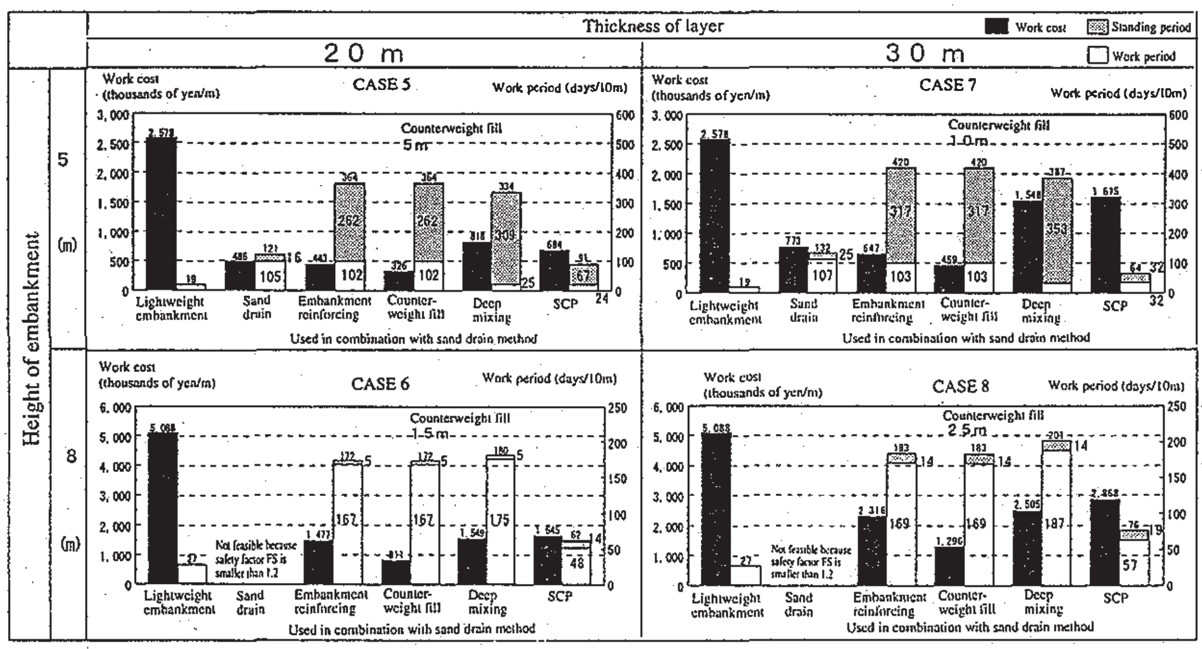


Fig. 11. Comparison group C (for high embankments with a thick soft layer)

However, if the sand drain method alone is used, the time required for the residual settlement to fall within the allowance can be reduced because drains are driven more densely compared with other combined methods (sand drain + embankment reinforcing, sand drain + counterweight fill), but the construction cost is somewhat higher. As the height of the embankment increases further, there are cases where the embankment cannot be stabilized by the sand drain method which enhances the strength of the embankment. Therefore, the sand drain method must be used together with another method (embankment reinforcing, counterweight fill, deep mixing method). In such cases, if sufficient land is secured, the sand drain + counterweight fill method is the most inexpensive and thus is preferred.

The embankment reinforcing method is the next most inexpensive method (as compared only with the counterweight fill), and is effective when used as a supporting method in cases where the safety factor offered by the sand drain method alone is insufficient.

The sand compaction pile (SCP) method offers a number of advantages, in that its work period is relatively short while the deep mixing method is also a superb method that has a limited effect on the surrounding environment. The lightweight embankment method is quite expensive compared with other methods, but the total amount of settlement is small and the work period is also very short. Therefore, this method can be used when the height of the embankment and the thickness of the soft layer are quite large, and a maintenance-free installation is required.

(4) Summary

For a low embankment with a relatively thin soft layer, the surcharge method is advantageous. However, if the thickness of the soft layer increases, the surcharge method should be used together with a sand drain to reduce the settlement period. For an embankment as high as about 5 meters, the sand drain method and the sand drain method combined with embankment reinforcing or counterweight fill are relatively inexpensive and offer advantages. As the height of the embankment increases, the sand drain method combined with other methods (embankment reinforcing, counterweight fill, deep mixing method) or the SCP method is required.

The work cost of the lightweight embankment method is rather high. However, this method is preferable when the height of the embankment and the thickness of the soft layer are very large, and a maintenance-free installation is required. The embankment reinforcing method is very effective when used in combination with the sand drain method for high embankments in cases where the safety factor provided by the drain method alone is not sufficient.

3 SOFT GROUND SURFACE LAYER REINFORCING METHOD USING GEOGRID IN COMBINATION WITH VERTICAL DRAIN

As the results of the feasibility study presented in Chapter 2 show, the soft ground surface layer reinforcing method is a good supporting method when the vertical drain method alone fails to provide a sufficient safety factor. This chapter gives an outline of embankment work for the Niigata-Nishi Bypass, in which the soft ground surface layer reinforcing method using geogrid was used to support the vertical drain. We also describe how the deformation behavior of the embankment and the tension of the geogrid were predicted through FEM elastoplastic consolidation analysis, and examine the applicability of the analysis method.

3.1 *Introduction*

The method features a simple calculation based on the rotational slip calculation method, though some aspects of the tension of the geogrid and the deformation behavior of the reinforced embankment have not been clarified yet. Field observations made at various sites have shown that the tension applied to the geogrid is quite small and the current design method allows us to design with a large safety margin. Therefore, if the tension applied to the geogrid can be estimated with practical accuracy in advance, a more reasonable and economical design method can be established.

This report describes the applicability of the analysis method, which we assessed by simulating the deformation behavior of a real embankment and the tension of the geogrid, using the FEM elastoplastic-consolidation analysis method for the Niigata-Nishi Bypass, where the vertical drain method is combined with a reinforcing method using geogrid.

3.2 *Outline of embankment work*

(1) Conditions of ground and embankment soil

The Niigata-Nishi Bypass is planned as part of Route 116, an ordinary national highway, and is a major expressway connecting the City of Niigata and the City of Kashiwazaki. The planned construction site is located on the flood plain of the Shinano River and is on soft ground. In addition, since most of the planned route is to be built on a high embankment, sliding failure and residual settlement after the highway is brought into service will be major problems.

Figure 12 shows the ground condition and the embankment shape of the cross-section of the Niigata-Nishi Bypass No. 113 which was subjected to the FEM elastoplastic-consolidation analysis.

The ground is soft ground with an N value ranging from 0 to 2 up to a depth of 6.5 meters, and a cone

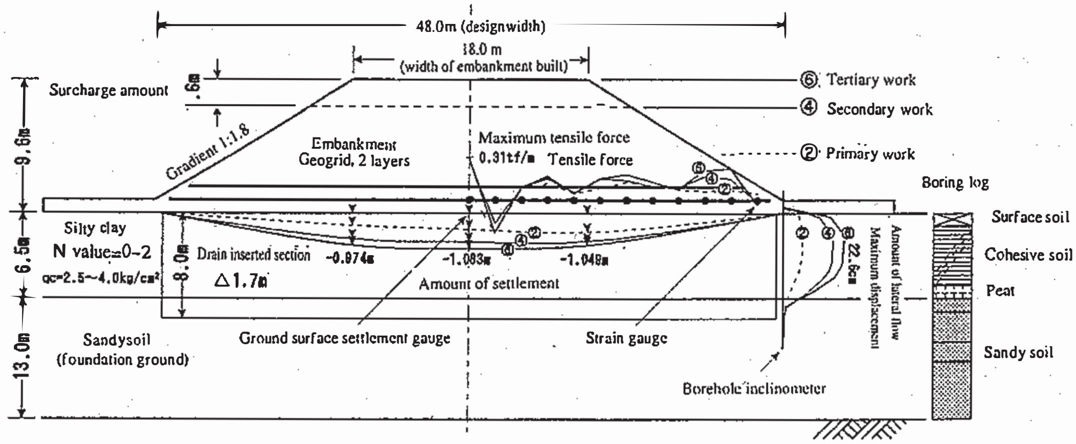


Fig. 12 Outline of cross-section of the Niigata-Nishi Bypass measurement point No.113, location of instruments and results of observation

penetration resistance ranging from 2.5 to 4.0 Kgf/cm^2. The embankment is high with a planned height of 8.0 meters, and if the embankment is built without reinforcement, the safety factor Fs for rotational slip that passes through the foundation ground will be 0.89, far below the required safety factor of 1.25.

We therefore studied the vertical drain method (1.7 m spacing), stage embankment method (5 cm/day) and the surcharge method (extra-fill 1.6 m) to reinforce the embankment. Even if these methods are used, Fs will be 1.15, slightly below the required factor, and thus the embankment cannot be fully stabilized. Therefore, we also used the geogrid reinforcing method, which gave the required safety factor Fs of 1.25. The safety factor of an embankment reinforced by geogrid is given by the following formula.

$$Fs = \frac{Mr + \Delta Mr}{M_d} \tag{3.1}$$

Where, M_d : Sliding moment
Mr : Resisting moment of soil
ΔMr : Resisting moment of geogrid
$\Delta Mr = R \cdot T_A \cdot \cos\theta \cdot n$
(see Fig. 13)

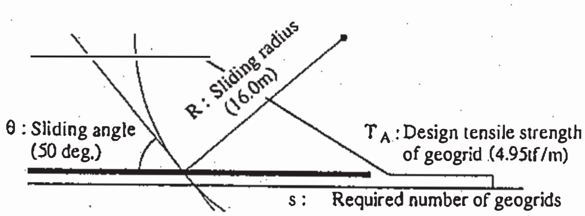


Fig. 13. Design of reinforcement method using geogrid

The required number of geo-grids is given by the following calculation:

$$1.25 = \frac{1033.467 + (16.0 \times 4.95 \times \cos 50 \times n)}{901.416} \tag{3.2}$$

$n = 1.84$, and thus the required number of geogrids is 2.

(2) Instrumentation for field observation

Various instruments for field observation over time are used. These instruments include ground surface settlement gauges installed at the natural ground surface (at three points; the center of the embankment, right and left top of the slope), a borehole inclinometer directly below the toe of the embankment slope (up to a depth of 12 meters at the toe of the embankment slope), and strain gauges placed on the surface of geogrid (at 12 locations at intervals of 2 meters).

(3) Results of field observation

The results of field observation are shown in Fig. 12. The figure indicates that settlement, lateral displacement and tension of the geogrid corresponding to approximately 80% of the final consolidation have developed by the secondary work. As for the tension, a maximum tension of only about 0.31 tf/m was generated, and this reflects the fact that the embankment was designed with a sufficient safety margin by this design method.

3.3 *Analysis of field observation results*

(1) FEM elastoplastic-consolidation analysis

The computer program (LADIS) was used to perform the FEM elastoplastic-consolidation analysis.

(2) Modeling of ground

The ground models used for the analysis was the Cam-

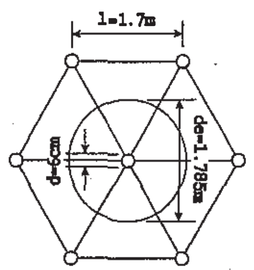
Table 5. Soft ground input constants

Name of stratum	Constitutive equation	Elastic modulus E (tf/m²)	Poisson's ratio γ	Cohesion C (tf/m²)	Internal friction angle ϕ	Compression index λ	Expansion index κ	Wet density (tf/m³)	Converted coefficient of permeability k (m/day)	Converted coefficient of permeability k (m/day)
Embankment	D.P.	1000	0.263	4.0	40.0	-	-	1.70	8.64×10^{-1}	→
Clay stratum 1	C.C.	-	0.400	-	-	0.600	0.060	1.59	2.16×10^{-4}	1.00×10^{-2}
Clay stratum 2	C.C.	-	0.400	-	-	0.680	0.068	1.56	1.40×10^{-4}	1.00×10^{-2}
Clay stratum 3	C.C.	-	0.336	-	-	4.800	0.480	1.35	2.16×10^{-4}	1.00×10^{-2}
Sand stratum 1	D.P.	340	0.300	0.0	35.0	-	-	2.00	8.64×10^{-2}	→
Sand stratum 2	D.P.	820	0.300	0.0	35.0	-	-	2.00	8.64×10^{-2}	→
Sand stratum 3	D.P.	1000	0.300	0.0	35.0	-	-	2.00	8.64×10^{-2}	→
Sand stratum 4	D.P.	1600	0.300	0.0	35.0	-	-	2.00	8.64×10^{-2}	→

Constitutive equation: D.P. = Drucker-Prager, C.C. = Cam-Clay
Coefficient of permeability: k = Coefficient of permeability after insertion of drain materials

Clay model for cohesive soil and the Drucker-Prager model for sandy soil, and the strength constant and other constants were determined by various soil tests (Table 5).

As for the section containing the vertical drains, assuming that the coefficient of permeability for the entire drain section will be improved by the insertion of drains, the average value of the coefficients of permeability calculated for the drain material and the natural ground based on the insertion interval is used as the coefficient of permeability (Fig. 14).



Interval at which drains are positioned: l = 1.7 (m)
Sand column equivalent diameter of drain: d = 0.05 (m)
Effective diameter of drain: de = 1.05d = 1.785 (m)
Coefficient of permeability for sand column: k' 8.64 (m/day)
Coefficient of permeability for soil: ka (m/day)

$$k' = \frac{(d^2\pi/4)\times ks + \{(de^2\pi/4)-(d^2\pi/4)\}\times kc}{(de^2\pi/4)}$$

Fig. 14. Calculation of the average coefficient of, permeability for ground with vertical drains

(3) Modeling of characteristics of friction between soil and geogrid

a) Modeling of geogrid

The geogrid, which is defined as a one-dimensional truss element that provides tension only, is regarded as having friction with the soil material around the geogrid (Fig. 15), and the elastic modulus, etc. were obtained from tensile tests (Table 6). The tension in the geogrid generated by slight sliding on the surface could thus be assessed.

Table 6. Geogrid input constants

Name of material	Elastic modulus E(tf/m²)	Yield stress (tf/m)	Sectional area m²
Adem F-10	80000	9.00	0.001

b) Joint element

To demonstrate the above-mentioned frictional relationship, a Goodman-type joint element was used (Fig. 15). This element is characterized by the fact that the shear modulus (frictional force) and the shear yield stress in the direction of axis increase in proportion to the overburden pressure (earth pressure) that acts vertically in the axial direction (Fig. 16).

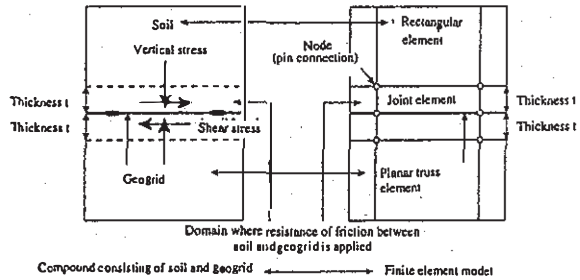


Fig. 15. Goodman-type joint element

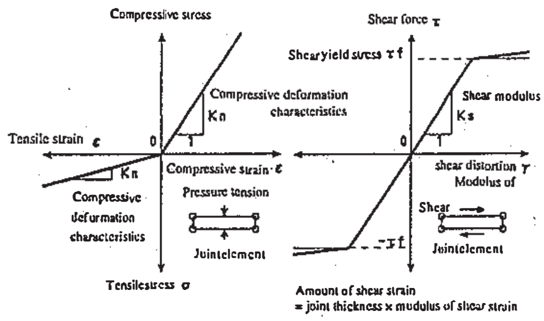


Fig. 16. Deformation characteristics of joint element

c) In-the-ground geo-grid pull-out test

To examine the frictional characteristics of the joint element, we conducted a geogrid pull-out test in the ground. An outline of the testing apparatus is shown below.

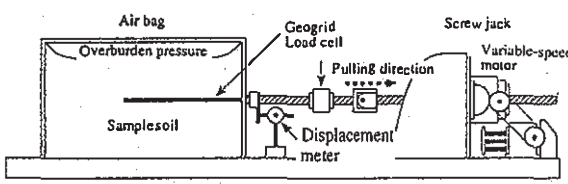


Fig. 17 In-the-ground geogrid pull-out testing apparatus

Assuming that the maximum height of the embankment at the site is just under 10 meters, a test was conducted for six cases, each with different overburden pressure, namely, 0.2, 0.6, 1.0, 1.4, 1.6 and 1.8 kg/cm^2, and geogrid with an embedded length of 45 cm was pulled out at a speed of 1 mm/min in each case. During each test, the strain of the geogrid was measured at 10 locations while real-time measurements of the amount and force of pull-out were taken. The relationship between the pull-out force on one side of the geogrid and the shear displacement for each overburden pressure is shown in Fig. 18. To plot this graph, the effective area method was used, in which only the part where friction is actually generated is evaluated.

The effective area method allows us to evaluate frictional force more accurately compared with the total area method in which the entire contact area is evaluated. From the displacement graph shown in Fig. 18, the values for the initial tangential gradient and the shear yield point were taken, and correlated with the overburden pressure as shown in Fig. 19. The figure shows the relationship between the overburden pressure *sn* and the shear modulus *Ks* as well as the shear yield stress *tf*.

(4) Results of analysis

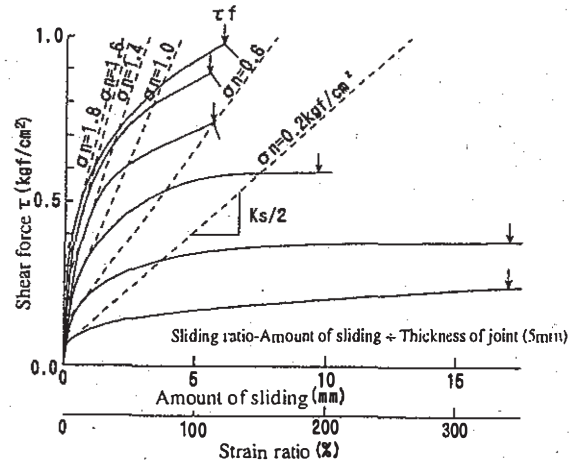


Fig. 18. Relationship between shear force and strain

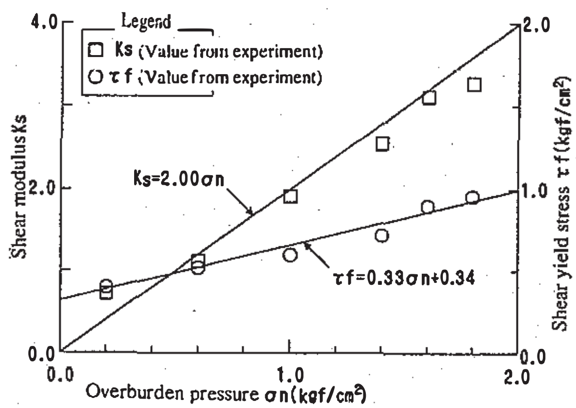


Fig. 19. Deformation characteristics of joint element

Figure 20 shows the amount of ground settlement for each stage of the embankment measured over time, Fig. 21 the state of lateral flow directly below the toe of the embankment slope, and Fig. 22 the distribution of tension generated in the geogrid. The circled numbers in the figures are for each work stage of the embankment in Fig. 20.

The settlement behavior evident in Fig. 20 is caused by the low coefficient of permeability for the ground, but the amount of the settlement did not sharply increase immediately after the embankment work in each state. Therefore, the amount of settlement remained constant through the standing period. The amount of settlement measured in the middle of the embankment showed a tendency very similar to the value obtained from the analysis, indicating that the actual settlement behavior can be predicted accurately in advance by using this analysis method.

As for the lateral flow shown in Fig. 21, lateral flow is likely to occur immediately after embankment work in each stage. A comparison of the predicted values with the values actually measured at stage (4) showed excellent correspondence, except that the predicted value was somewhat larger than the measured value. The analysis was thus fairly accurate.

As for the distribution of tension in Fig. 22, the measured values varied greatly from one measurement point to another, but the peak value of close to 0.3 tf/m of tension generated at the center of the embankment, which is important in the design, almost corresponded to the predicted value. We therefore consider that the maximum tension can be accurately predicted by the analysis. The reason for the variation of the measured values is thought to be that local stress concentration had a large effect because the field observation was conducted with strain gauges placed at intervals of 2 meters.

The above results showed that the behavior of the actual embankment can be modeled accurately through FEM analysis by appropriately representing the drained zone, etc., by evaluating the frictional characteristics of the geogrid by means of in-ground pull-out tests, and by using a joint element.

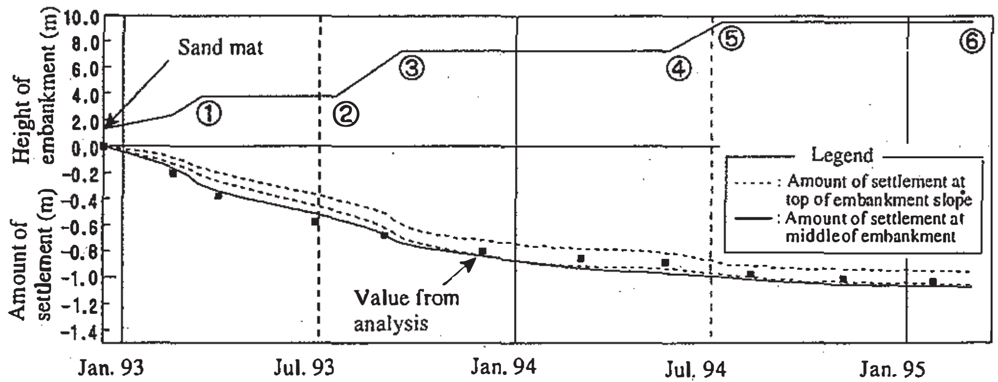


Fig. 20. Height of embankment and settlement curve

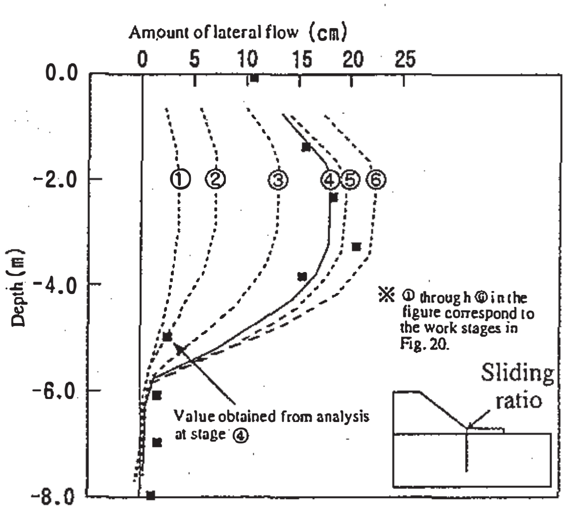


Fig. 21. Lateral flow directly below toe of embankment slope

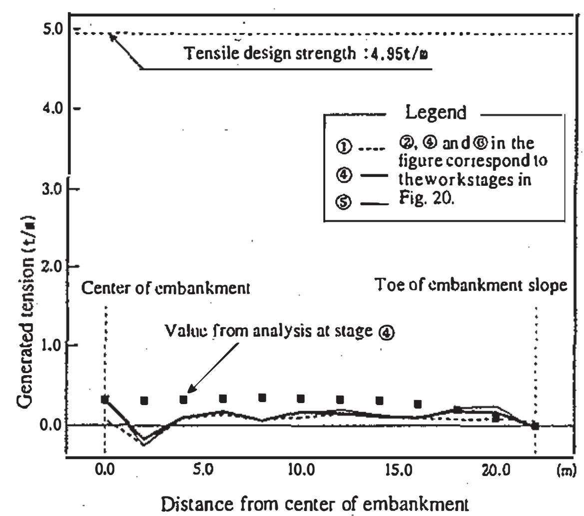


Fig. 22. Distribution of tension generated on geogrid

3.4 *Summary*

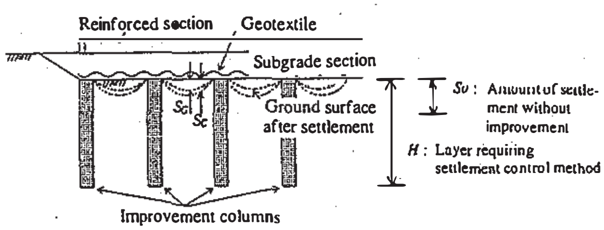
We attempted to reproduce the behavior of the site through FEM elastoplastic-consolidation analysis for the Niigata-Nishi Bypass in which a reinforcing method using geogrid is applied. For the analysis, we represented the coefficient of permeability for the natural ground by an average calculation for the intervals at which drains are embedded, conducted an in-the-ground pull-out test, and represented the geogrid using a joint element. The results showed that the actual measured values were similar to the analysis values for the settlement, lateral flow and generated tension, indicating that the behavior of the actual embankment is accurately predicted by the analysis method, and that the tension actually applied to the geogrid can be predicted.

In the future, we will apply this method to other sites, accumulate case study data, and study the possibility of establishing a more reasonable and economical design method by reflecting these results in the current design method, and thus design with a small safety margin.

4 SETTLEMENT CONTROL METHOD USING GEOGRID IN COMBINATION WITH DEEP-MIXING METHOD OF SOIL STABILIZATION

4.1 Outline of the method

Foundation ground is sometimes improved at a low improvement rate by the deep-mixing method of soil stabilization as shown in Fig.23 to reduce residual settlement in road embankments on soft ground. However, differential settlement sometimes develops between the improvement columns used for the deep-mixing method of soil stabilization and the unimproved part between the columns, causing cracks or unevenness at the top of the embankment. In this chapter, we propose a tentative design concept for laying geotextiles to reduce such differential settlement.[2)]



S_G : Differential settlement between improvement columns and unimproved section between columns (with geotextile)
S_C : Differential settlement between improvement columns and unimproved section between columns (without geotextile)

Fig. 23. Example of method of controlling differential settlement between improvement columns and unimproved part between columns by using geotextile

4.2 *Design method*

(1) Selection of materials

For the geotextile material, geogrid or a material with a smaller amount of tensile strain is desirable. Since this design method allows the tensile strain of the geotextile to be less than 1%, creep may not need to be considered for the safety factor of the material. As for the soil material around the place where the geotextile is laid, a material with large frictional resistance is desirable so that the tensile force of the geotextile can be fully utilized. Improper materials include those containing a large portion of fine particles or those that are so sharp and large grained that the geotextile may be damaged.

(2) Design conditions

1) Allowable settlement: The allowable values for total settlement *Sco* and differential settlement *Sco* are assumed to be 10~20 cm or smaller and 5 cm or smaller, respectively, based on the various guidelines.

2) Design load: Design load comprises embankment load and traffic load. The traffic load on a low-embankment road on soft ground will be evaluated in terms of embankment load equivalent to the portion affected by repeated traffic load based on long-term settlement (see the Road Earthwork Manual on soft ground improvement works).

3) Improvement columns: Improvement columns used for the deep-mixing method of soil stabilization will be positioned in a staggered manner or in a grid form, and provide column-based improvement. The distance between column centers should be about 1.5 ~ 3 m, and the columns must be embedded down to the bearing ground.

(3) Design procedure

The design procedure for the differential settlement control method using the deep-mixing method of soil stabilization in combination with geotextile is as follows.

1) Design conditions and load conditions such as design constants of the soft ground are determined.

2) The amount of settlement without the deep-mixing method of soil stabilization S_0 is examined, and if the required allowable value is not satisfied, improvement based on the deep-mixing method of soil stabilization is studied.

3) The load bearing capacity of the improvement column used for the deep-mixing method of soil

stabilization is examined, and the specification of the improvement column (strength of pile and improvement rate) is determined.

4) Based on the specification for ground improvement determined in step 3), the average amount of settlement of the improved ground S_T is reviewed. If the average settlement does not meet the target allowance, the improvement specification of the deep-mixing method of soil stabilization is re-examined.

5) The amount of differential settlement for the application of only the deep-mixing method of soil stabilization Sc is examined. If the allowance is not satisfied, the amount of differential settlement for combined application of the deep-mixing method and geotextile is examined, and the laying specification of the geotextile is determined.

6) The case where the improvement rate of the deep-mixing method is increased and the amount of differential settlement is reduced is compared with the case where geotextile and the deep-mixing method are used together for examination, and an economical method is then finalized.

(4) Design calculation method

This section explains the principles of the design calculation.

1) Examination of load-bearing capacity of improvement column and determination of specification of improvement column

The specification of the improvement column is examined using the following formula, assuming that all the overburden load above the top of the improvement column (embankment load and traffic load) is supported by the improvement column.

$$Fs \leqq \frac{q_{uck}}{\frac{\Delta p}{a_p}} \tag{4.1}$$

Where, Fs : Target safety factor (1.0~1.5),

q_{uck} : Design standard strength of pile (kgf/cm^2)

Δp : Overburden load (kgf/cm^2)

a_p : Improvement rate (0.1~0.3)

2) Method of calculating amount of settlement

a) Average amount of settlement of ground where deep-mixing method is used

The average amount of settlement of the composite ground S_T where the deep-mixing method is applied is obtained from the following formula, in consideration of the stress concentration on the improvement column, and assuming that the stress share ratio of the improvement column n is equal to the ratio of the coefficient of volume compressibility of the improvement column to the unimproved soil n.

$$S_T = \beta c \cdot So = \frac{1}{1 + a_p\,(n-1)} \cdot So \tag{4.2}$$

Where, S_T : Average amount of settlement of improved ground (m)

βc : Settlement reduction ratio (ratio of reduction of increased stress applied to unimproved section)

a_p : Improvement rate

n : Stress share ratio($=\sigma_p/\sigma_c=n'=mv/m_p$=10~20)

mv : Coefficient of volume compressibility of unimproved section

m_p : Coefficient of volume compressibility of improvement column

b) Amount of differential settlement of composite ground where the deep-mixing method is used

The amount of differential settlement of the improvement column for the deep-mixing method and the unimproved section Sc is obtained from the following formula, using the amount of settlement of the pile top of the improvement column Sp and the amount of settlement of the unimproved section Sd. In this formula, n is 1/2 of the value used in 1) for simplicity, obtained from the comparison between FEM and actual measurements.

$$Sc = Sd - Sp \tag{4.3}$$

$$Sd = \beta c \cdot mv \cdot H \cdot \sigma \tag{4.4}$$

$$Sp = \beta c \cdot m_p \cdot H \cdot n \cdot \sigma \tag{4.5}$$

3) Amount of differential settlement for combined application of geotextile and deep-mixing method

When the deep-mixing method of soil stabilization and geotextile are used together, the amount of settlement that occurs between the improvement column and the unimproved section between columns S_G is calculated as follows.

$$S_G = \frac{Sc}{1 + 2 \cdot \alpha \cdot (Sc/\sigma)} \tag{4.6}$$

The proportional factor α in the above formula is determined from Fig. 24. Namely, it is determined from the tension stiffness of the geotextile EA, which will be assumed first, and from the pile pitch equivalent to the improvement rate a_p. The value of α is then substituted in Equation (4.6), and a trial calculation is continuously made until the standard value S_G is obtained.

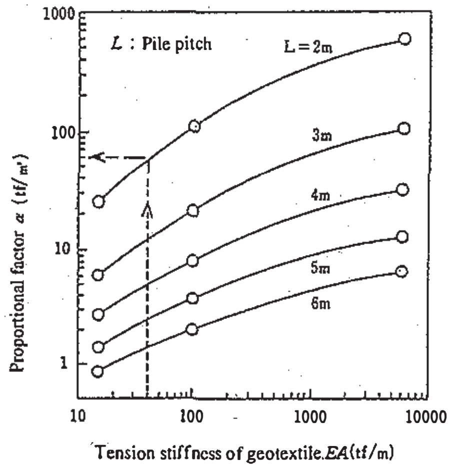


Fig. 24 Relationship between tension stiffness of geotextile and proportional factor α[2]

4) Selection of material for geotextile

In this design method, the material is selected not based on the design tensile strength but on the stiffness obtained when the strain is about 1% because the strain level of the geotextile is low.

The amount of strain generated in the geotextile ε is estimated from the amount of differential settlement based on field observations, using the following formula.

$$\varepsilon = 0.15 \cdot S_G \tag{4.7}$$

Then, from the amount of strain, the required tensile strength of the geotextile T is calculated, using the following formula, and geotextile with the required tensile strength or the number of layers requiring geotextile is determined.

$$T = \frac{E \cdot A \cdot \varepsilon}{100} \tag{4.8}$$

4.3 *Future tasks*

(1) The formula (4.2) used to calculate the average settlement of the ground where the deep-mixing method is applied leads to excessive settlement compared with the measured values, and therefore needs to be reexamined. The values of S_T where the columns are embedded down to the bearing ground are so small that they can be ignored. Instead, a formula for calculating the settlement of the frictional pile type is required.

(2) An appropriate method of evaluating the stress share ratio, which is important when calculating the differential settlement between the improvement column used for the deep-mixing method and unimproved section, is required. Measurements taken so far indicate that the differential settlement Sc is so small that it can be ignored if the improvement rate is larger than 15%, even when geotextile is not used in combination with the deep-mixing method.

(3) From the above, if the improvement rate of the deep-mixing method is larger than about 15%, the differential settlement between the improvement column and unimproved section is fairly small in many cases even when geotextile is not used. The possible combined use of geotextile should therefore be considered only when the improvement rate is less than 15%.

5 NEW RECLAMATION METHOD USING HORIZONTAL DRAINS IN COMBINATION WITH SAND MOUNDS TO LOWER WATER LEVEL

5.1 *Outline and characteristics of the method*

(1) Outline of the method

This method makes it possible to quickly start using land reclaimed with cohesive soil (class 3 or 4) from construction sites by accelerating the consolidation of the reclaimed ground with horizontal drains that are installed in it during the reclamation.

In order to load the ground, a multilayered sand mound is constructed inside the land-facing side of the revetment of the reclaimed land and a wellpoint or the like is used to lower the water level within the mound. This is the key characteristic of the method (see Figure 25).

The reclaimed ground should be an artificial island connected to the land with a road. Of the revetment surrounding it, the side facing the land should be of a cutoff type and the other three sides should be of optional sections. The revetment and the connecting road should all be completed before the reclamation starts.

In the reclamation, which is executed layer by layer, a multilayered sand mound is constructed inside the land-facing side of the revetment and, on each of the layers of fixed depth, horizontal drains are installed. On top, earth brought from another place is placed in a thinner layer. Finally, a wellpoint is driven into the multilayered sand mound from above the revetment facing the land.

In the loading procedure, first the inside water level is lowered as much as possible by opening the conduits passing through the land-facing side of the revetment. Then, the wellpoint is used to lower the water level within the mound so that a hydraulic gradient occurs in the horizontal drains, thus enabling drainage from the layers of reclamation soil.

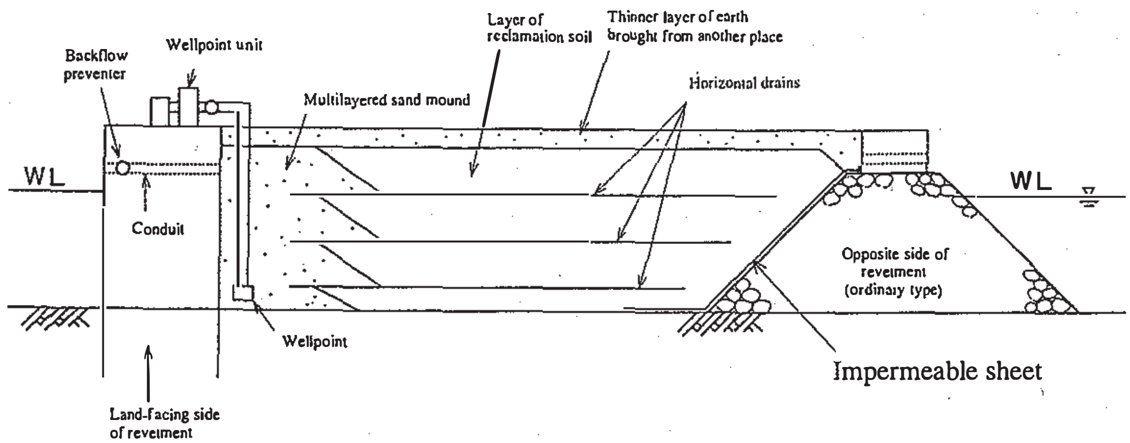


Fig. 25. Reclamation Method Using Horizontal Drains in Combination with Sand Mounds to Lower Water Level: Conceptual Illustration

(2) Characteristics of the method

The following are the characteristics of the method:

1) Most of the soil used in this method is low-quality soil from construction sites.
2) Horizontal drains are installed in the process of reclamation, and are quick and easy to install.
3) Preliminary drainage with conduits enhances the effect of the wellpoint.
4) The loading is executed by lowering the water level: it is not necessary to spread geomembranes or perform any extra surface treatment.
5) The multilayered sand mound not only functions as a drainage layer but also reduces the earth pressure on the revetment.
6) The earth brought from another place and placed in a thinner layer on top not only functions as a drainage surface but also as a surcharge and allows traffic to pass over the reclaimed ground in order to bring the earth from the other place for the last time.

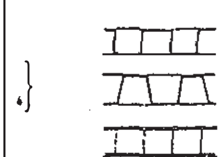
5.2 *Design method*

(1) Materials

The drains must be able to convey all the water that drains into them from the soft soil. The examples of drains shown in Table 6 are each composed of a corrugated core enclosed in a filter. Polyethylene resin is used as the core, and nonwoven polyester fabric as the filter. The core is structured so that it is scarcely deformed under the pressure of the soil, and the filter has a high water permeability to prevent clogging. These drains are very flexible and can be wound into rolls, and so can be easily transported and installed continuously over a long distance.

Drains other than these can be used provided that they can let all the water run through themselves, are sufficiently resistant to deformation under the pressure of the soil, and are easy to install.

Table 6. Drains: Examples

Section size B × t (mm)	Area of empty parts of section S (cm^2)	Diameter obtained by conversion dw (mm)	Section form
100 × 3	1	65.5	
100 × 6	5	67.5	
100 × 10	8	70.0	
150 × 6	8	99.3	
150 × 12	12	103.1	

(Note) The above "diameter obtained by conversion" is calculated using the formula, $d_w = 2(B + t)/\pi$, supposing the external circumference of each drain is circular.

(2) Flowchart of design

The following is a flowchart of the design of soil improvement by this method.

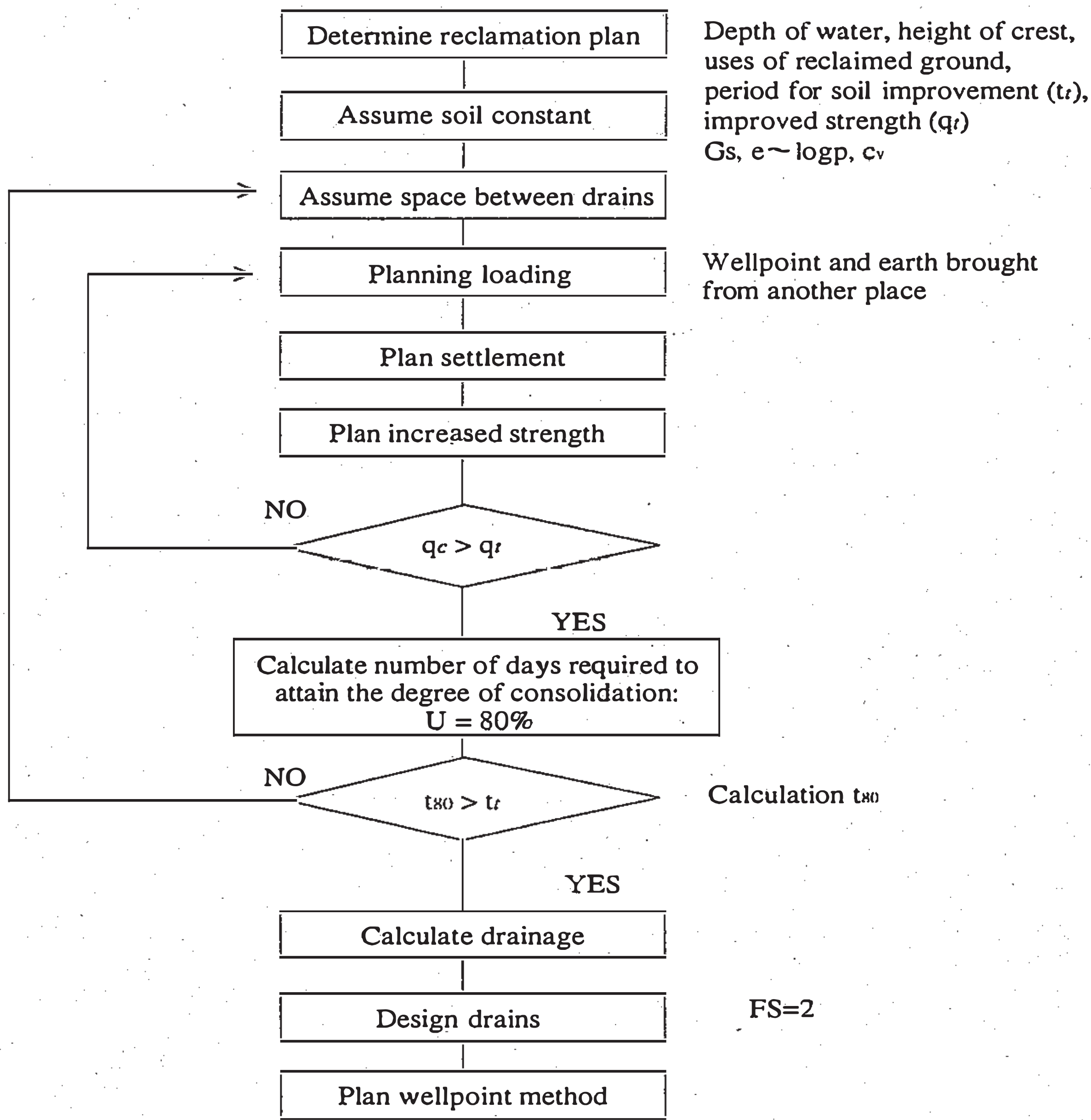


Fig. 26. Flowchart of design

(3) Design steps

1) Calculating the settlement

In calculating the final settlement due to consolidation, use the appropriate equation as follows:

$$S = \frac{e_0 - e}{1 + e_0} H \tag{5.1}$$

$$S = m_v \Delta p H \tag{5.2}$$

$$S = H \frac{C_c}{1 + e_0} \log \frac{p_0 + \Delta p}{p_0} \tag{5.3}$$

where,
- S : settlement due to consolidation
- H : target thickness of layer of soil
- e_0 : initial void ratio
- e : target void ratio
- m_v : volume change rate
- Δp : load change
- C_c : compression index
- p_0 : yield consolidation load

2) Calculating the loading period

Assuming that one-dimensional consolidation applies to the consolidation, calculate the degree of consolidation, which corresponds to the loading period and the time factor, using the following formula:

$$t = \frac{H_d^2}{c_v} T \tag{5.4}$$

$$U' = \{T^3/(T^3 + 0.5)\}^{1/6} \tag{5.5}$$

where,
- t : loading period
- H_d : drainage distance; in case of draining to both sides: $H_d = H/2$
- c_v : coefficient of consolidation
- T : time factor
- U : degree of consolidation

We use the degree of consolidation in this formula similar to that of Terzaghi's theoretical formula, because they are precise and simplify the calculation. For reference, when $T_{70} = 0.403$, $T_{80} = 0.567$ and $T_{90} = 0.848$, then the formula gives $U = 0.698$, 0.802 and 0.905, respectively.

3) Selecting the drain material

Calculate the maximum drainage speed per 1 m of drain width from the drainage when the degree of consolidation is 10%, as follows:

$$v = \frac{HL\,\varepsilon_{10}}{t} \tag{5.6}$$

where;
- v : maximum drainage speed
- H : thickness of soil layer to be improved covered by drains on each level
- L : length of layer covered by the drains
- ε_{10} : volume compression strain at 10% consolidation
- t : loading period till the degree of consolidation of 10%

Figure 27 shows the relationship between the area of inner empty section of each drain and the average drainage speed per drain, which we have obtained by a field experiment. This figure shows that, when selecting the drain, we should fix the area of the inner empty section of the drain so that v, the average drainage speed obtained by the above calculation, is positioned in the lower right of the broken line in the figure. Obtain the required area of the inner empty section of the drain from this figure, and thus select the drain. Note that the drains shown in the figure are only examples; other drains can be used provided that their average drainage speeds corresponding to section size are examined.

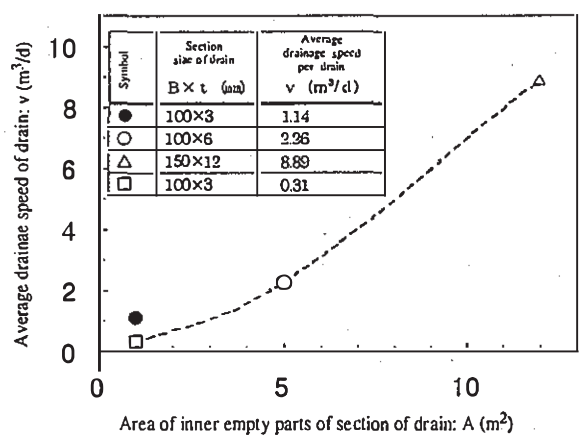


Fig. 27. Relationship between area of inner empty parts of section and average drainage speed of drain

4) Calculating the strength increase

Calculate the increased strength of the improved soil using the following formula:

$$\Delta c = c_u/p \times U \times \Delta p \tag{5.7}$$

where ;
- Δc : increased strength
- c_u/p : strength increase ratio
- U : degree of consolidation
- Δp : load change

5) Calculating the cone index (q_c)

Assume that the cone index (q_c) and the unconfined compressive strength of cohesive soil (q_u) are related to each other as expressed by Muromachi's formula:

$$q_c = 5\,q_u = 10\,cu \qquad (5.8)$$

where, q_c : cone index (kgf/cm^2)
q_u : unconfined compressive strength (kgf/cm^2)
cu : cohesion (kgf/cm^2)

5.3 *Method of reclamation*

(1) Machines

1) Floating barge for installation of horizontal drains
The floating barge for installation of horizontal drains in the ground should be a catamaran with two floating bodies (made from styrofoam), each with the following dimensions: 1.0 m (w) × 1.0 m (l) × 0.9 m (h) (and with a bottom coated with fiber reinforced plastic (FRP), a friction reducing agent), arranged parallel with each other with a space of 1.2 m in between. There is a mandrel between the two bodies and, by operating the winches, it is possible to wind them up onto the barge and install the drains on the ground. On the barge, there is also a reel unit on which to mount the drains. The barge is structured so that when it is moved back and forth, the reel rotates to automatically pay out the drain onto the ground. The gross weight of this barge is about 4 t and it has a 0.3 m draft. However, the draft can be adjusted by increasing the part of the barge above the water level.

Figure 28 outlines the floating barge for installation of horizontal drains.

2) Wellpoint unit
Figure 29 shows how the wellpoint unit should be installed.

(2) Procedure

In order to securely cut off the water, use steel sheet pile cellular type or steel sheet pile double-wall type when constructing the land-facing side of the revetment. Use soil from construction sites of class 1 or 2 as the soil packing for the side of the revetment. Optional but appropriate sections may be used for the other three sides of the revetment. In the case of the rubble-mound type, however, measures must be taken to prevent reclamation soil from being drawn out, such as by constructing a preventive structure.

a) Building a sand mound
Build a sand mound with a crest of the required width inside the land-facing side of the revetment using a clamshell, etc.

b) Filling soil from construction sites in layers
Fill in soil brought from construction sites systematically layer by layer, because horizontal drains are to be installed on it later. Properly locate each filled in layer, and measure the section of reclaimed ground each time. Use a floating bridge to guide the vehicles to where the soil is to be placed.

c) Installing the horizontal drains
When the depth of the layer of soil reaches the fixed level, install the horizontal drains. The width of the drains should be about 1.5 m. In the case of a band-type drain, mount multiple reels at regular intervals on the shaft aboard the floating barge . First, pull out one end of the drain and attach spacers to ensure adequate space between its neighboring drains. As the spacer material, geogrid, etc. may be used because it is easy to handle and relatively stiff.

From this initial state, operate the mandrel of the base to put the drain below the surface of the water. Then, start installing it in the longitudinal direction by winching the wire line, which has been installed beforehand. Install the horizontal drain so that about 2 m from one end of it lies on the sand mound and the other end is positioned several meters away from the structure to prevent the reclamation soil from being drawn out.

In order to securely position the drain, attach spacers at appropriate intervals to the middle part of it. Drive in bamboo spits through it as needed to prevent it from rising. Having reached the opposite side of the reclaimed ground, cut the drain and finish off the end appropriately. Then, return to the initial side of the ground and repeat the same work until all of the ground is covered by drains.

d) Repeating steps a), b), and c)
Repeat the cycle consisting of building the sand mound, placing the reclamation soil and installing the horizontal drains until the top of the crest is approached.

e) Placing a thinner layer of earth brought from another site
As the top layer, place earth brought from another site in a thinner layer of about 1 m. Perform this process by the floating conveyor method or the like. This layer prevents rainwater from permeating the reclaimed ground and encourages it to promptly drain off from the surface by preventing the surface of the soft reclamation soil from drying.

f) Loading by the wellpoint
Drive the wellpoint down into the multilayered sand mound and install the wellpoint unit on the land-facing side of the revetment. Immediately after completing reclamation of all parts of the land, the water level in the reclaimed ground is fairly high. Therefore, first open the conduits that pass through the land-facing

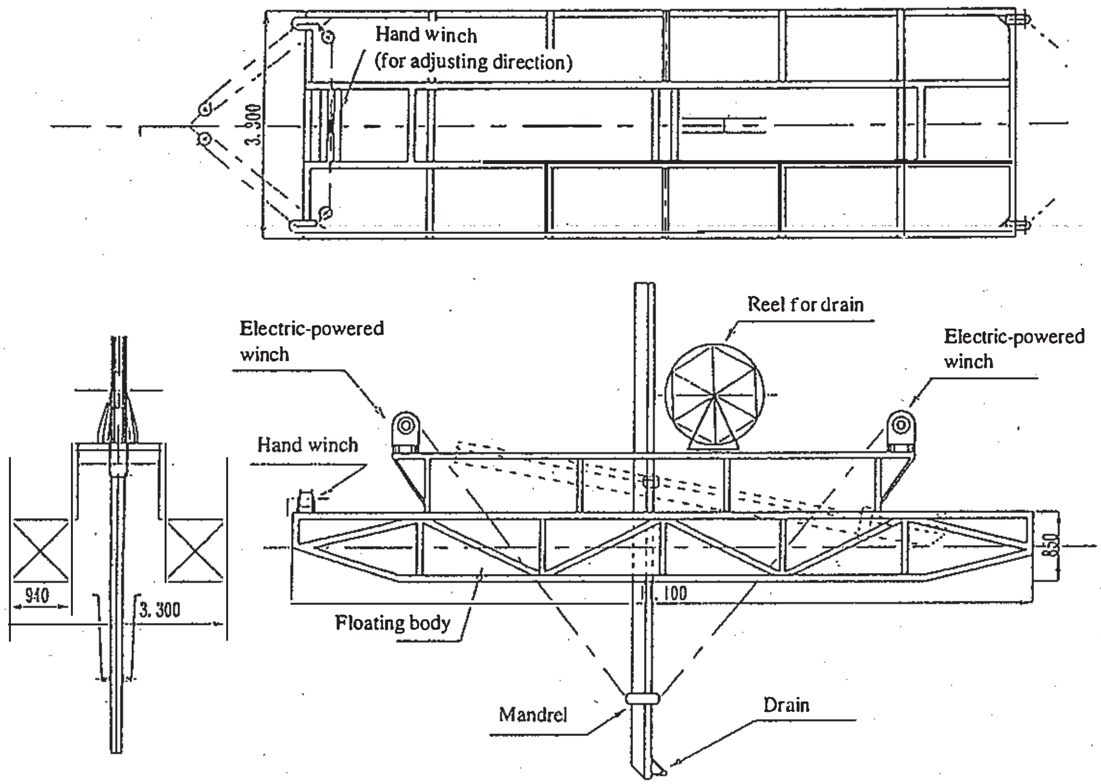


Fig. 28 Floating Barge for Installation of Drains

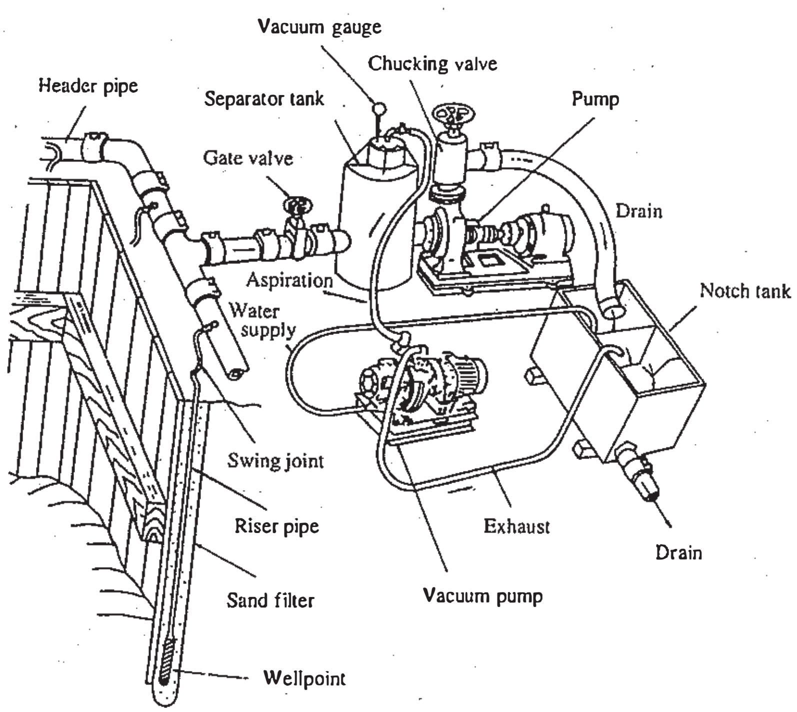


Fig. 29 How to Install the Wellpoint Unit

side of the revetment at appropriate intervals at a suitable time when the tide is right to lower the water level within the multilayered sand mound as much as possible. This is the preliminary loading. In order to make the wellpoint lower the water level as much as possible later, do not operate the wellpoint at this stage.

When the degree of consolidation by this preliminary loading has reached about 80%, operate the wellpoint to lower the water level further.

g) Placing earth brought from another place for the final time

When the degree of consolidation due to loading by the wellpoint reaches about 80%, the reclamation soil should have settled by several meters. Then put earth brought from another place (class 1 or 2) for the final time to offset the settlement. The earth thus placed acts as a load and also as the surface ground of the reclaimed land for parks, housing land and so on.

6 AFTERWORD

This paper explained the techniques of applying geosynthetics to embankments on soft ground mainly by presenting studies in which I have been involved. The results of many other valuable studies made in Japan and overseas have thus hardly been mentioned.

In conclusion, the following tasks remain to be addressed in this field.

In the case of reinforcement of soft ground surface for earth cover, a variety of design concepts that take the bearing capacity mechanism into consideration have been proposed. However, because the method of work execution has a great effect, the design tensile strengths of reinforcing materials have actually been fixed so far based on past experience. Therefore, a more rational design method should be established through an analysis of works executed to date.

On the other hand, in the case of reinforcement of soft ground surface for embankments, when the soft ground is too deep or the embankment too high to secure the stability against sliding failure with sand drains alone, a feasibility study that we conducted has shown that it is advantageous to use synthetic geogrid as a supplement to the sand drains, as another possible way of applying geosynthetics. In order to extend the use of geosynthetics, a limited number of specific uses that can fully compete with existing methods, such as this one, should be developed.

Concerning the combined use of geogrid and the deep mixing method, various methods have been proposed based on concepts similar to that of the pilenet method: conducting piling improvement up to a low improvement rate (about 10 ~ 30%) for all the surface below the embankment through the deep mixing method and then compensating for the insufficient stability against sliding with geogrid, and using geotextile to prevent differential settlement of the improved and non-improved parts. The combined use of the deep mixing method with geogrid to realize a low improvement rate has not yet fully been established as a design method. However, this new and economical attempt at improving soft ground is likely to find practical use.

Further, as a measure to resist earthquakes, we shall study the application of the reinforced earth method using geosynthetics to control settlement during an earthquake of embankments on loose sandy ground that could be liquefied.

Concerning the effective use of low-quality soils generated at construction sites, a design method has been proposed based on the theory for saturated soil, so the current method of reinforcing the embankment using cohesive soil with high water content by draining it can only be applied to saturated soil. It is thus necessary to develop and establish a theoretical method of reinforcement that can be applied not only to saturated soil but also to partially saturated soil. In addition, a new execution method should be developed that enables this method to be used not only for embankments on land but also for those that have foundations under water such as for reclamation works.

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