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DESIGN AND TECHNIQUES OF STEEP REINFORCED EMBANKMENTS WITHOUT EDGE SUPPORTINGS

DIMENSIONNEMENT ET TECHNIQUES DE REMBLAIS A FORTE PENTE RENFORCES SANS SUPPORT LATERAL

BEMESSUNG UND KONSTRUKTION STEILER GEOTEXTILVERSTÄRKER ERDDÄMME OHNE RANDVERSTÄRKUNG

This paper describes with a steep slope reinforcing embankment of non-wall face built in Japan for the first time. This first part of the paper treats of the design strength of polymer grid reinforcement based on a series of creep tests adopted the effect of strain characteristics not only for the ordinary condition but also for seismic (short term) condition. The second part of this reports a case study regarding a steep slope embankment built using a pumice soil called *shirasu*, an extremely erosive soil. Also, results of a series of observation were compared with results of an analysis by means of the finite element method on the embankment was approved to be appropriate.

Diese Abhandlung behandelt Verstärkungsauftragarbeit mit steilem Hang ohne Seitenwände, die erstmalig in Japan durchgeführt worden ist. Zuerst wird eine Serie von Dauerstandversuchen mit Hinblick auf den Effekt der Spannungsgeschwindikeit behandelt, die durchgeführt wurden, um die Entwurfsfestigkeit von polymergittern nicht nur im normalen Zustand, sondern auch während eines Erdbebens (kurzzeitig) zu erhalten. Anschließend wird über eine Fallstudie in Bezug auf Verstärkungsauftragarbeit mit steilem Hang unter Verwendung eines *Shirasu* genannten Bimssteins mit sehr hoher Erosion berichtet. Die Ergebnisse einer Serie von Messungen in Bezug auf die Auftragarbeit wurden mit den Analyseergebnissen nach der Finite Element Methode verglichen, und die Angemessenheit dieser Analysemethode wurde bestätigt.

1. INTRODUCTION

In Japan early research on the method of reinforcing soil with geotexitiles has been in progress and it has been practically used as a surface treatment for weak foundations(1967),(1) and also as a reinforcement for railway embankments (1978),(2). On the other hand, reinforcing methods for retaining walls with geotextiles (1977),(3) and steep slope embankments in Europe and the United States,(4),(5).

This paper deals with the design, construction and analysis of polymer grid reinforced steep slope embankments in Japan. The first application of this kind in Japan that makes use of polymer grid developed in the United Kingdom. A series of creep tests has been conducted in the laboratory to determine the design strength of the grid. The results are used in the designs for both ordinary condition and seismic condition. In the practical construction of the steep embankments, a simple method which, with the use of sand-bags, eliminates the construction of formworks is used. Moreover, the suitability of the design is evaluated and checked by means of the finite element analysis of the system compared with the instrumented data obtained during and after the construction.

2. DESIGN STRENGTH OF POLYMER GRIDS

2.1 Ordinary Loading Condition

In using plastic materials as reinforcements in semipermanent earth structures, the problem of how to fix the design strength (α_d) arises on account of its visco - elastical stress - strain behaviour. The yield tensile strength of the material (α_f) is largely dependent upon the temperature and speed of testing (MacGown, A. et al.,1984) , (6). Consequently on the basis of rational thought that the strength should be determined from creep characteristics, uniaxial and biaxial stretched polymer grids are tested in the laboratory (Yamanouchi et al.,1985),(7). The creep loads are taken to be 0.3 ∿ 0.6 times that of the standard tensile strength (α_{f50}) which is obtained from testing of the polymer grid at 20°C and at a strain rate of 50% of the length per minute.

In order to evaluate the trend of changes in creep strain, the relationship between the creep strain rate $\dot{\varepsilon}$ and elapsed time is plotted in Figs. 1(a)and (b). From these figures, it is quite obvious that $\dot{\varepsilon}$ is decreasing when a loading ratio of R ($= \alpha_c/\alpha_{f50}$) $\leqq$ 0.4 for the case of a uniaxial stretched polymer grid (Fig. 1 (a)). Consequently the design strength α_d of the reinforcement of this type is determined at $0.4\alpha_{f50}$ (i.e. R_d = 0.4). However in the case of a biaxial stretched grid, the strain rate $\dot{\varepsilon}$ is increasing even before R ≦ 0.4, hence α_d cannot be derived for the design condition (Fig. 1 (b)).

2.2 Short Time Loading Condition

The tests mentioned above were carried out with the aim to evaluate the design strength of polymer grid under ordinary loading conditions. Tests were also conducted in order to determine the increase factor of design strength for the case of increased load on the polymer grids for a short duration (Yamanouchi et al., 1985), (8). These additional tests were aimed at finding out the increase factor of design strength from the deformation characteristics of the polymer grid in case it is subjected to a load increase of $\Delta\alpha_c$ for a duration of 30 minutes and then reduced to the original load of α_c of the creep tests mentioned above. Loading ratio increases from 0.3 to 0.65 were used.

Fig. 2 shows the results obtained from the tests

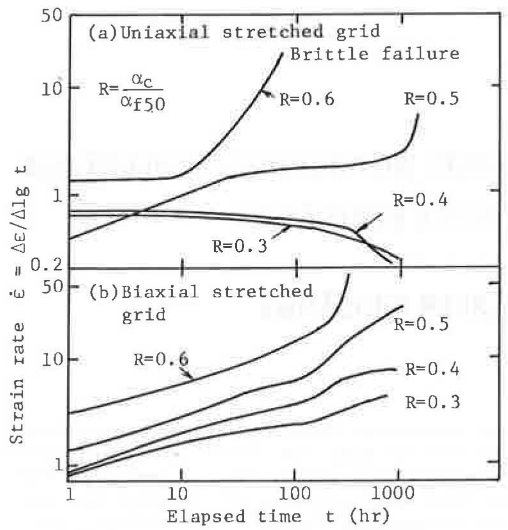


Fig.1 Relationship between $\dot{\varepsilon}$ and lg t

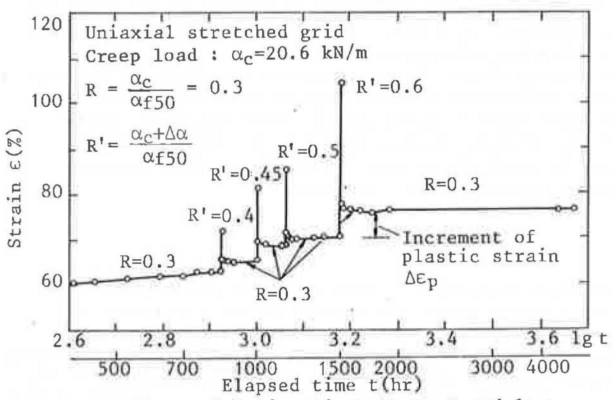


Fig. 2 Relationship between ε and lg t under short-interval loading

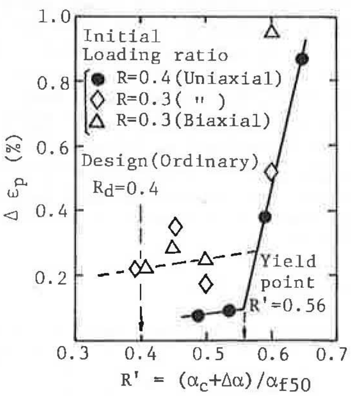


Fig. 3 Increment of plastic strain under short term loading.

for the uniaxial polymer grids in the case of R = 0.3. Changes of creep strain rate $\dot{\varepsilon}$ before and after the short-interval loading can hardly be noticed. However the increment of plastic strain $\Delta\varepsilon_p$, in cases of $R' = (\alpha_c + \Delta\alpha)/\alpha_{f50} \geq 0.6$ which is equivalent to the load ratio at the brittle failure under creep tests, is rather significant.

Fig. 3 shows the relationship between R' and $\Delta\varepsilon_p$. According to this figure, it proves that there is an influence on the deformation characteristics of the grids due to the increase of short-interval loading for the case of $R' \geq 0.56$. Hence, it is suggested that the increase factor of design strength of polymer grid for short-interval loading should be $R'/R_d = 0.56/0.4 = 1.4$.

3. DESIGN OF STEEP POLYMER GRID REINFORCED EMBANKMENT

3.1 Soil Characteristics of Fill Material

In the city of Kagoshima, which is situated in southern Kyushu of Japan, a large land development for a housing estate is in progress and there arises due to the limited available flat land, a necessity of constructing an earth fill embankment with a steep slope. After a comparative study of different construction methods, a steep geogrid reinforced embankment, the first of its kind in Japan, was selected for the site in question due to its economic validity , construction efficiency and partly a motive of construction innovation.

Shirasu, a pumice-flow deposit soil, is widely distributed throughout this region and is considered to be a difficult unusual soil due to its frequent damages (gully erosion and slip failure) mainly from rainfalls and earthquakes. Here it became necessary to use *Shirasu* as a fill material for the construction site. *Shirasu* is classified as a well-graded volcanic sand. The results of the triaxial compression tests under drained condition of *Shirasu*, compacted to an initial void ratio of $e_0 = 0.94$, indicates that $c' = 2.45$ kN/m^2 and $\phi' = 45.2°$.

3.2 Fundamental Design under Ordinary Condition

The fundamental design of the reinforced embankment is done in accordance with the design standard of Jewell,(8). The basic geometry of the embankment is, H = 6.0 m, slope 1 : 0.2,under the condition of 9.8 kN/m^2 surcharge. The shear constants for the design of the fill material are taken as $c' = 0$, and the angle of intenal friction $\phi' = \tan^{-1}(\tan\phi'/1.5) = 33.9° = 30°$ with the safety factor of soil strength at 1.5, and the unit weight of soil γ is taken as 17.7 kN/m^3 with an assumption that the soil is in a completely saturated state due to heavy rain. As to the reinforcement material, uniaxial stretched grid with a strength of $\alpha_{f50} = 78.5$ kN/m is utilized and the design strength is taken, on the basis of the creep tests as $\alpha_d = 0.4\alpha_{f50} = 31.4$ kN/m.

Jewell has provided design charts based on two-part wedge mechanisms (Fig. 4 (a) and (b)). Fig. 4 (a) is the chart used for the determination of the apparent

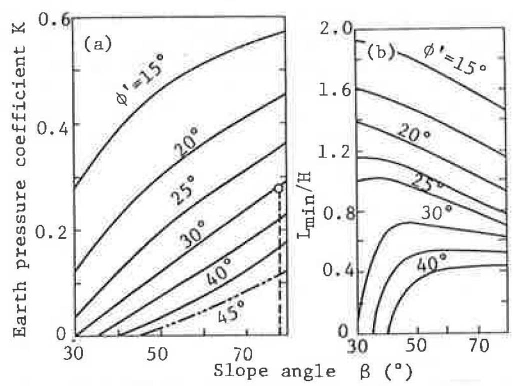


Fig. 4 Design chart by Jewell et al.(8)

values of earth pressure coefficient K, required for computing the horizontal stresses that stabilize the steep slope embankment with the parameters of angle of internal friction ϕ' and slope inclination β. Fig. 6 (b) is the chart from which the required ratio of minimum length (L_{min}) to height (L_{min}/H) of the grid is determined.

Length of Polymer Grid

From Fig . 4(b) the value of L_{min}/H is found to be 0.63 when β = 78° and ϕ'= 30° the design length (L) is determined from the following formula, and the same length (L) was taken for whole embankment.

$$L = (\frac{L}{H})\ (H + \frac{w}{\gamma}) = 0.63 \times (6.0 + \frac{9.8}{17.7}) = 4.13 \text{ m} \qquad (1)$$

and rounded up 4.5 m.

Number of Grid Layers

Applying the various parameters of the embankment to Fig. 4 (a), the apparent composite earth pressure required for stability is given by

$$T = \frac{1}{2} K\gamma (H + \frac{w}{\gamma})^2$$
$$= \frac{1}{2} \times 0.277 \times 17.7 \times (6 + \frac{9.8}{17.7})^2 = 105.1 \text{ kN/m} \qquad (2)$$

The minimum required number of grid layer is calculated as

$$N_{min} = \frac{T}{\alpha_d} = 3.4 \qquad (3)$$

and rounded up to 4 layers, since the design strength (α_d) of the grid is 31.2 kN/m^2.

Vertical Spacing of Grids

In case of composite earth pressure - the product of the grid vertical spacing (v) and the apparent horizontal earth pressure $p_h = K\gamma z$, with the grid in such a position that the height of earth fill is z - it becomes equal to α_d, the grid layers will be the most efficient position. In the present analysis all the grid layers were placed in equal spacing as required for spacing the base plane of the embankment. The vertical spacing (v) is 1 m according to the following equation.

$$v = \frac{\alpha_d}{K\gamma (H + \frac{w}{\gamma})} = \frac{31.4}{0.277 \times 1717 \times 6.6} = 0.97 \qquad (4)$$

3.3 Fundamental Design under Seismic Condition

The techniques of earthquake resistant design for polymer grid reinforced embankments have not been well established as yet. Here the analysis will be done in accordance with the Teree Armée design and construction manual which has been standardized in Japan, on the basis of soil-retainig test results of Richardson et al. (1977),(9).

Therefore, Fig. 5 is the schematic diagram of the increase of horizontal earth pressure during earthquake. And Fig. 6 shows the relationship between seismic design coefficient (E) and input uniform base acceleration (a/g). The stress increase $\Delta\sigma$ is given by the following equation.

$$\Delta\sigma = \frac{1}{2} (1 + \frac{z}{H}) E \sigma_0 \qquad (5)$$

Where, σ_0: Horizontal earth pressure of the lowest end of the embankment at ordinary condition = $K\gamma H$, E : The earthquake design coefficient = $1.4a/g = 1.4\ k_h$, k_h : Horizontal seismic coefficient for the design of road structures in Japan. For Kagoshima city, it is calculates to be 0.10.

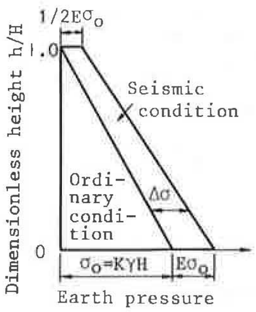


Fig. 5 Schematic diagram of the increment of earth pressure under seismic condition (9)

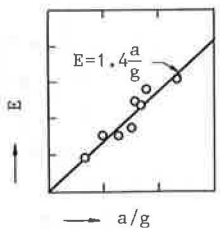


Fig.6 relationship between seismic design coefficient and input uniform base acceleration

The Required Number Of Grid Layers

As to the design conditions, the surcharge loading w is assumed to be absent during an earthquake. In the case the total horizontal stress T_E is computed by equation (3) and (5).

$$T_E = \frac{1}{2} K\gamma H^2 + \int_0^H \Delta\sigma dz = \frac{1}{2} K\gamma H^2 (1 + 2.1\ k_h)$$
$$= \frac{1}{2} \times 0.277 \times 17.7 \times 6^2 \times (1 + 2.1 \times 0.10)$$
$$= 106.8 \text{ kN/m} \qquad (6)$$

Accordingly if the design strength of the polymer grid α_{dE} (short duration) = $(R'/R_d)\alpha_d$ (refer to 2.2), the minimum required number (N_{min}) of grid is given by

$$N_{min} = \frac{T_E}{\alpha_{dE}} = 2.4 \qquad (7)$$

i.e. 3 layers where R'/R_d = 1.4.

The Vertical Spacing of Grids

On the basis of Fig. 5, the vertical spacing of grid at the lowest of the embankment will be subjected to analysis. The horizontal earth pressure p_{hE} at the base plane of the embankment is given by

$$p_{hE} = (1 + E)\ K\gamma H \qquad (8)$$

The vertical spacing v is computed in accordance with the equation (4).

$$v = \frac{R'\alpha_d}{R_d(1 + E)K\gamma H} = \frac{1.4 \times 31.4}{1.14 \times 0.277 \times 17.7 \times 6} = 1.31 \text{ m} \qquad (9)$$

As is obvious from the results of the general computations, the design section is determined under ordinary loading condition. However, the aseismic design should be, in fact, established by conducting vibration tests for the polymer grid reinforced embankment.

4. CONSTRUCTION TECHNIQUES

The construction of polymer grid reinforced embankment, making use of Shirasu soil as fill material, has been put into practice as shown in the schematic diagram Fig. 7. Due to the topographic condition of the site, it was required to construct a portion of the embankment with a height of 7 m. As the stability of whole embankment is easily affected by the degree of completion at the slope portion, sand bags were piled up each 1 m high, to retain compacted earth fill, before the whole layer is wrapped up by polymer grids. In the embankment the grid portion is taken to be 1 m vertical and the slope is of the form of stairs with

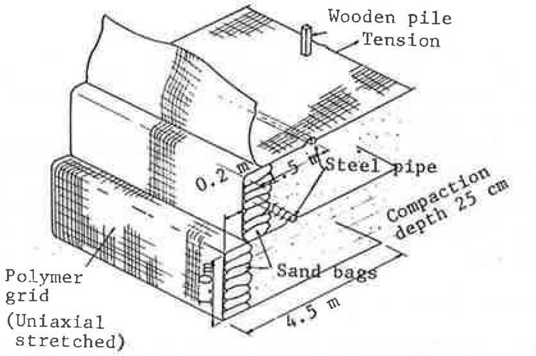


Fig. 7 Techniques to construct the polymer grid reinforced embankment

a berm width of 0.2 m. And with respect to the grid joint between upper and lower layers, the length of inter-wrapping is 1.5 m at the lower step, and both layers are jointed by steel pipes at the upper step as shown in Fig. 7. In order to eliminate the relaxation of the grid during construction, tension is added to the free end of the grid and temporarily retained through a small wooden pile, earth-filling is done. This technique of construction do not require working from the front side of the embankment and is easy to construct with a high degree of finishability, and it is found to be speedier than an ordinary conventional work, e.g. the concrete retainig wall. Fig. 8 shows the completed view of the reinforced embankment.

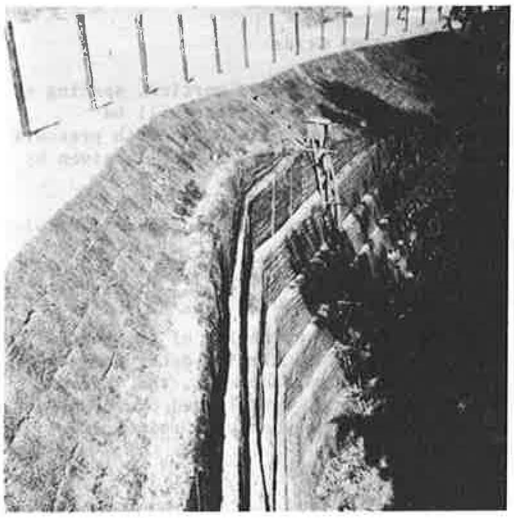


Fig. 8 Completed view of reinforced embankment

5. INSTRUMENTATION

5.1 Setting-up of Instruments

It had been decided upon at the beginning of works to measure the horizontal and vertical displacements of slope and the distribution of grid strains. With the aim of studying the stability of the reinforced embankments and also comparing the results of design with those of practical construction. The instruments and their layout are as shown in Fig. 9 . The measurements are done at two cross section. The displacement measuring unit is made of L shaped iron members and a member is placed at every one meter height of the slope. From the differences of scaled readings of the upper and lower intersection points, shown in Fig. 9, horizontal and vertical displacements at every layer are measured. and again the strains of the grid are measured by strain gauges directly attached to the grid ribs.

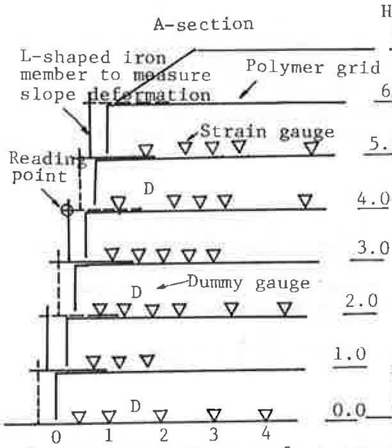


Fig.9 Layout of instruments

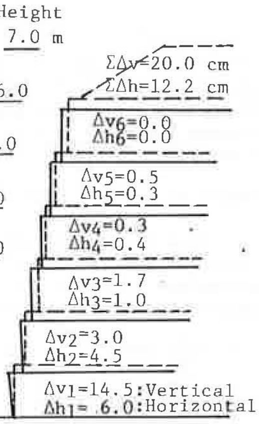


Fig.10 Displacement of polymer grid reinforced embankment

5.2 Deformation of Embankment Body

Fig. 10 shows the deformed conditions of the embankment body after the completion of construction works. In both measured sections it was found that the more the overburden pressure increases above the bottom, the more horizontal and vertical displacements increase. It is to be considered that the reason why the vertical displacement of the lowest layer shows a high value is due to the settlement of the initial earth fill layer of about 2 m height at the bottom of the embankment. Besides it was found out that the horizontal or vertical displacement of the first two layers from the bottom composes 86∿88% of the total. During embankment construction, both displacements increase slightly after construction, but that was considered a negligible amount.

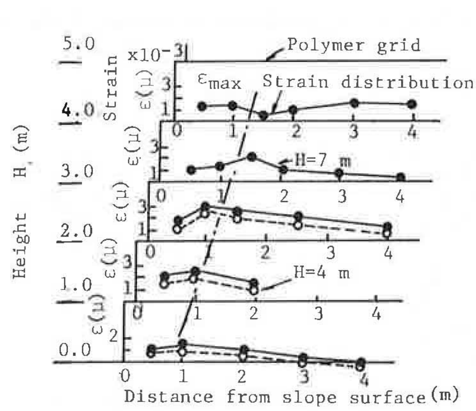


Fig.11 Measured strain distributions of polymer grid. (B-section)

5.3 Strain of Grid

Fig. 11 shows the distribution of strain of each grid reinforcement in the embankment. The maximum strain of each strain is found to be about 3000 μ (0.3%). Here it is required to convert the strain of the polymer grid into tensional force and for that the relationship between tensional force and strain, as shown in Fig. 12, is used. According to Fig. 12, the greater the strain rate the higher the tensional strength as well as the stiffness coefficient will be. Accordingly, to convert correctly the actual recorded strain into grid tensional force, the maximum tensional force is estimated at around 2.9 ∿ 6.9 kN/m referring to Fig. 12. This value is equivalent to 9 ∿ 22% of the design strength of the grid (i.e. α_d = 31.4 kN/m) and is considered to be small, only 17 ∿ 39%, compared with the average grid tensional strength of 17.6 kN/m (Eqn. (2): 105.1 ÷ 6 layers) of the present embankment. Roughly converting in the backward order the actual recorded values into the resistance force T_R of the grid, even the maximum is found to be 6.9 kN/m x 6 layers = 41.2 kN/m. Here comparing the original design conditions, the suitability of T should be checked. That is for the embankment height H = 7 m (the original design was H = 6.6 m), the field unit weight of *Shirasu* γ = 14.6 kN/m3 (design 17.7 kN/m^3) and the angle of internal friction ϕ'= 45° (design 30°) are taken. For Fig. 6, K = 0.11 is obtained from the estimated curve of ϕ'= 45°, and from equation (2).

$$T = \frac{1}{2} \times 0.11 \times 14.6 \times 7^2 = 39.4 \text{ kN/m} \qquad (10)$$

In other words, this value is found to just coincide with 41.2 kN/m which was derived in the backward computation from the actual observed strains. From these results the present design conditions are considered to be on the safe side against those originally assumed.

6. FINITE ELEMENT ANALYSIS

6.1 Method of Analysis

With respect to the model shown in Fig. 13 the analysis by the finite element method is carried out with an aim to verify the mechanism of steep grid reinforced embankment design. The models of elements

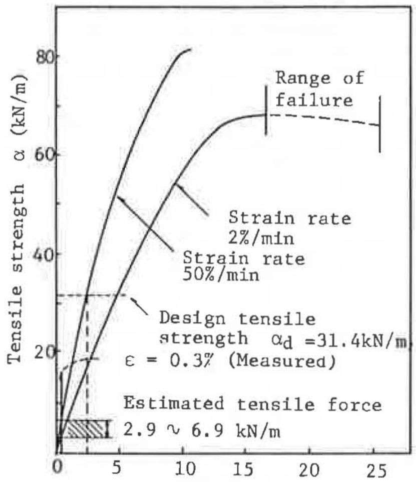


Fig. 12 Tensile strength and strain characteristics of uniaxial stretched polymer grid. (10)

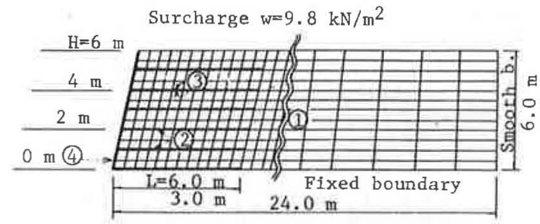
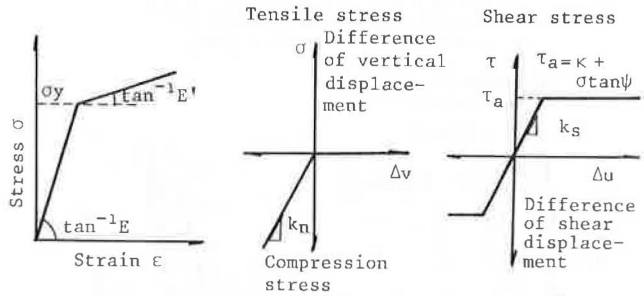


Fig. 13 Model of finite element analysis



(a) Model of fill material (b) Model of joint element

Fig. 14 Model of analysis

Table 1 Values for finite element method

No.	Material	Element	Values for analysis
①	Fill	Drucker-praper model	γ_d=14.6kN/m^3,E=1960kN/m^2 E'=19.5",c'=0",ϕ'=42° ν=0.25
②	Polymer grid (in the fill-body)	Spring element (Elasto-plastic model)	E=4.3x10^{5}kN/m^2,E'=4.2x10^3" σy=5.6x10^4",A=1.4x10^{-5}m^2
③	Boundary between ① and ②	Joint elememt (Frictional model)	k_n=98.1kN/m^3,k_s=1.1x10^{-9}" κ=0.54kN/m^2,ψ=35°
④	Polymer grid (slope surface)	Beam element (Elastic model)	E=4.3x10^{5}kN/m ,A=1.4x10^{-5} m^2,I=2.4x10^{-12}m^4,ν=0.3

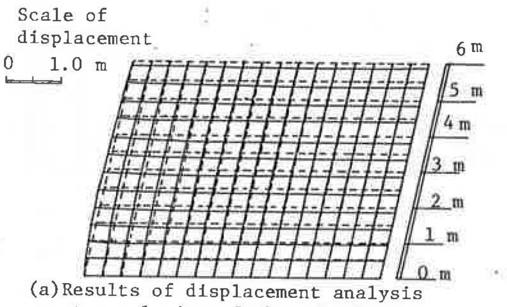
k_n: Estimated value

are illustrated as follows. That is as shown in Fig. 14(a) ; the model of Drucker - Prager for embankment material, Fig. 14(b) : the joint elements of frictional type inserted between the grid and fill material. Moreover the grids laid horizontally in layers in the fill-body is considered as a spring element of the elasto-plastic type and the grid at the slope as a beam element.

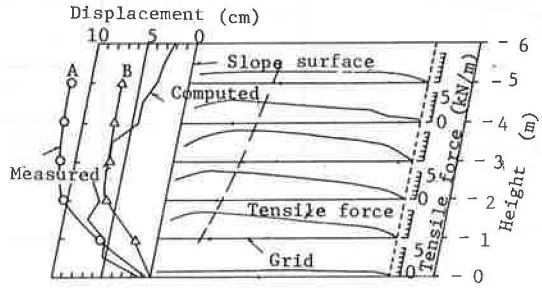
The notations in Table 1 are as indicated in Fig. 14 and they are as follows. ν: poisson's ratio, A: cross-sectional area of the grid, I: moment of inertia of grid area, and the values of κ and ψ are adhesion and angle of friction between the grid and soil respectively.

6.2 Comparison of Observed Data and Computed Results

The finite element analysis was performed for two cases of grid lengths L = 3.0 m and L = 6.0 m prior to the practical construction. In practice the length was 4.5 m, and the results were compared with the practical recorded data. In the analysis the vertical displacement, of the top portion of the embankment was estimated to be 7.1 cm (Fig. 15(a)) , whereas it is of the order of 15.8 ∿ 20.0 cm in the practical construction. Considering the possibility of settlement of the base layer as mentioned earlier the results of the



(a) Results of displacement analysis at completion of the embankment



(b) Horizontal displacement of slope surface and tensile force of grids(L=6 m).

Fig. 15 Results of the finite element anlysis

analysis is found to be quite satisfactory. Fig. 15 (b) shows the distributions of polymer grid tensile strength and the horizontal displacement of slope. In the analysis the tendency of construction of the upper portion of embankment, accompanied by the internal settlement is clearly observed and the values of this case is found to coincide though the pattern is slightly different from the practically measured data. Moreover, the measured distribution pattern of grid tension is approximately equal to that of the strain distribution of Fig. 11. And it is clearly understood that the results of analysis (3.9 ∿ 5.9 kN/m for H = 1 ∿ 4 m grid) is exactly in coincidence with the maximum tension (2.9 ∿ 6.9 kN/m) computed from measured strains. Moreover, results of the finite element analysis for the case of grid length L = 3.0 m were also the same as the case of L = 6.0 m.

7. CONCLUSIONS

The reinforcing of steep embankment with polymer grid is a new technique, primarily developed in the United Kingdom. The authors believe that this method of construction will enjoy a wide application in future following this first and foremost practical undertaking in Japan. With respect to it, the following points can herewith be concluded.

1) Polymer grids have the properties of visco-elastic materials. Thus the design strength of polymer grids should be determined by conducting creep tests, especially when they are applied to this type of earth structures. From a series of tests under ordinary and seismic conditions it is found that the design strength should be 40% of the tensile strength for 50%/min at 20°C, and that to increase the design strength for seismic conditions an increase factor of 1.4 could be applied to the ordinary design strength.

2) The design of the steep reinforced embankment is done in accordance with Jewell's method under the ordinary condition and a conceptional design under the seismic condition is also done by applying Richardson's testing results, with due consideration of design strength increase of short duration. And design section for H = 6 ∿ 7 m embankment as this paper is likely to determined under the ordinary condition.

3) The completion of the slope portion of steep embankment is especially important in constructing steep reinforced embankments. Here sand bags are piled up and used as a provisional slope retaining structure. This method of construction was adopted for two advantages, one is, formworks are not necessary, and the other is to require less amount of work on the slope portion.

4) The strain on the polymer grid and deformation of embankment are measured during and after construction. The embankment deformation that was measured during construction was quite small. The tensile strength computed from grid strains was found to be much lower than the design strength. However, irrespective of the fact that the various coefficients of embankment material are taken on the safe side at the primary design stage, the designed model is found to be in similitude with the prototype. This suitability of the design is well confirmed.

5) The analysis by the finite element method is carried out with an aim to verify the mechanism of steep grid reinforced embankment design. Here the actual recorded data are found to be in accordance with the results of visco-elastic analysis of the model with joint elements between grids and fill material. For future this analysis is considered to be effective to use for the analysis and design of reinforced earthfill structures.

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