Design and Observation of Steep Reinforced Embankments

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Proceedings: Third International Conference on Case Histories in Geotechnical Engineering, St. Louis, Missouri, June 1-4, 1993, SOA 7

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Synopsis: Using the design method proposed by R. A. Jewell et al. numerous steep reinforced embankments have been constructed in the authors' home country since the year 1984. In fact these soil structures are built with the reinforcement of polymer grids (the so-called geogrids invented by F. B. Mercer of U.K.) which have a unique structural composition with high-tensile and low-ductility characteristics. This paper deals with first the development of steep reinforced soil structures and their design method, and then introduces a well-documented case history of steep reinforced embankment. The authors propose a current design method developed on the basis of the findings obtained from the observations at several steep reinforced embankments including the present one of the case history. And finally an ultimate seismic-design method for steep reinforced embankment adopted recently in Japan is presented.

INTRODUCTION

Building for various purposes steep-slope high embankment structures has been a persistent desire of the people throughout the history of mankind. At present the technology of embankment construction has become a very important branch of civil engineering. The remains of large embankments, the soil structures which had been built in ancient time using natural materials as reinforcements, were found at Ziggurat of Mesopotamia and at palaces and royal grave yards or castle walls of China (Yamanouchi, 1992). The Terre Armee (the registered English name is Reinforced Earth) invented by H. Vidal of France in the year 1963 is a method of applying the frictional resistance between the sandy soil and galvanized metal strips to retaining of soil wall-structure and this method is increasingly popular among the engineers of various countries since the beauty of the concrete wall-surface is highly appreciated. The impact of this method on the future development of steep reinforced soil structures is considerably great.

Since the beginning of 1980s the reinforced soil structures have been built using polymer grids of high tensile strength and low ductility as reinforcements. These materials have been invented by F. B. Mercer of U.K. and the application of their tensile strength to earth reinforcement is quite different from that of the Terre Armee.

Reinforced soil structures can be classified into two categories: (a) construction of a steep slope embankment by repeated process of wrapping the earth with geogrids or a vertical concrete-face reinforced wall with geogrids, (b) reinforcing the back-fill soil or increasing the bearing capacity of foundation with the polymer grids so as to keep the embankment stable.

In the former method the uniaxially oriented polymer grids of SR type are used and in the latter the biaxially oriented polymer grids of SS type are employed. The reinforced embankment was accepted by all engineers as it has, for its easy vegetation on the embankment slope, some merits from the point of view of environmental protection. The vertical front-face reinforced embankment is considered, in fact, as a counter-technology of U.K. against the Terre Armee. The earth wall of the latter method is a competition as stated in the Godfrey's report "Retaining walls; competition or anarchy?". At present the former technology, as it does not hamper the embankment's unique independent nature, has attracted the attention of the author and consequently here in this report the case history of embankment construction by this method is presented.

The epoch-making design method developed by Jewell et al. (1984) is a very valuable design tool accompanied by practical and easy-to-use charts. The method also makes use of the two-part wedge concept for the analysis of ultimate equilibrium condition. These features lead to the wide-spread use of the method. Since the polymer-grid reinforced embankments are introduced the retaining walls having concrete-block faces that look like the form of Terre Armee are tested in large number and then steep or near vertical high embankments are built and

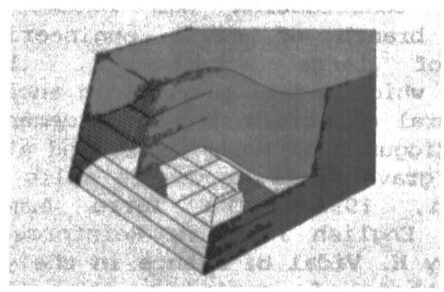
TABLE Ⅰ. Development of Steep Reinforced Soil Structures and Their Environs

Year	Events
1962	Development of Nonwoven Fabrics of Continuous Fibers by Spun Bond Method
1963	Development of Terre Armee (France) by Vidal (Patent 1966)
1977	Int.Conf.The Use of Geotextiles (1st Int.Geotextile Conf.)
1978	Symp.Reinforced Soil Structures by ASCE
1980	Successful Construction of Reinforced Soil Wall Structure Using Web-form Reinforcement
1981	Various Lectures on Geotextiles (J.K.Mitchell et al.), 10th Int.Conf.SMFE (Stockholm)
	"Construction and Geotechnical Engineering Using Synthetic Fabrics" by R.M.Koerner
1982	2nd Int.Geotextile Conf. (Las Vegas)
1983	Session on Reinforced Soil Structures, 8th Symp.European Society of SMFE (Helsinki)
	Case Study of Embankment Construction Using Polymer Grids, UK's Success Story
1984	Symp. Polymer Grid Reinforcement in Civil Engineering (London)
	Announcement of Steep Reinforced Embankment Design by Jewell, et al. (Ditto)
	Symp. Geomembranes (Denver)
	Int.Geotextile Society (IGS) founded (Presided by J.P.Giroud)
	"Geotextiles and Geomembranes", an Int.Jour.Publication (Ed.by T.S.Ingold)
	"Geotextile and Geomembrane" by Int.Information Source (Ed.by J.D.Scott and E.A.Ricards)
1985	Technical Committee on Geotextiles, ISSMFE (TC9) Established
	11th Int.Conf.SMFE, Geotextiles Sectional Meeting (Chaired by J.P.Giroud, San Francisco)
	"Earth Reinforcement and Soil Structures" (Ed. by C.J.F.P.Jones)
1986	3rd Int.Geotextile Conf. (Vienna)
	"Geotextile Testing, an Inventory of Current Geotextile Test Method and Standard" (Ed.IGS)
1988	Publication of "ASTM Standards on Geosynthetics" (1st Ed., 2nd Ed.: in 1991)
	"Report on Strengthened Reinforced Soils and Other Fills" by British Standards Institute
	Continuous Synthetic Fiber Method for Granular Soil Reinforcement by E.Leflaive
1990	Successful Construction of Steep Embankment of height 12m + Surcharge of 6m-Equivalent Height Using High Tensile Fabrics (Seattle)
	"Specification for the Use of Geotextiles and Related Materials" by Ground Engineering Group Board, ICE
	4th Int.Geotextile Conf. (Hague)
1991	Successful Construction of Geosynthetic-reinforced Soil Retaining Wall for Railway by Japan Railway Research Institute

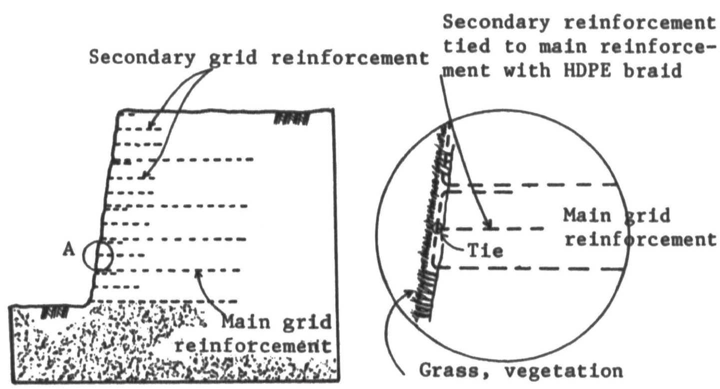
developed by laying high tensile woven and nonwoven fabrics as reinforcements. In this state-of-the-art paper the historical background, the development of design method, introduction of Japan's well-documented case histories, the recent design method together with the seismic design procedure, etc. mainly on the steep polymer-grid reinforced embankments are presented.

DEVELOPMENT OF STEEP REINFORCED SOIL STRUCTURES

The construction technique of various kinds of steep-slope reinforced soil structures including the polymer-grid reinforced embankment have undergone rapid progress as accompanied by the quality improvement of the reinforcing material, the holding of geogrid related symposia and the publication of techological books. The historical events of reinforced soil structures are listed in TABLE I. The steep-slope reinforced embankment shown in Fig.1 uses the uniaxially oriented geogrids (Fig.2) as reinforcements. The practice of such polymer-grid reinforced embankments has been repeatedly carried out first in U.K. since the early 1980s and then spread to other technologically advanced countries after the 1984 London symposium on polymer grids.

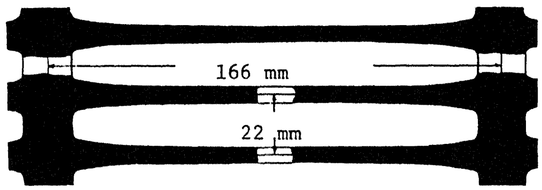


(a) General feature after completion



(b) Sectional feature

Fig.1. The Original Design of Polymer-grid Reinforced Embankment Proposed by Netlon Limited in 1984



SR 55 (54 kN/m), SR 80 (69 kN/m), SR 110 (98 kN/m)

Fig.2. Unioriented Polymer Grid Produced in Japan (1990)

The steep-slope embankments or earth walls constructed by using reinforcing materials excluding the polymer grids that are taken up in this report are mostly designed with a view to achieve cost reduction. The design method of Jewell et al. (1984) has been used or referred to from time to time for the design of reinforced embankments. A brief current history of the reinforced soil structures is described as follows.

(a) The materials such as metallic grid, metallic ring chain, etc. which are cheaper than the polymer grids have been used as reinforcements for the construction of steep-slope soil structures in Japan. The developer of the metallic reinforcements has devised a kind of rust protection by coating the metal bars with rust-proof coating and the creep characteristics of the polymer grid that cannot be found in the metal is considered as a demerit. Even if the metal is rusted the structure will, it is expected, remain stable as it can adjust itself to the new state of the reinforcing metal. The long-term creep strength that may result in after 120 years' polymer-grid's life is also considered negligible. The metallic materials are to some extent encouraged as reinforcements; this is due to the fact that these materials are accepted as reinforcements in Manual for Earth Work (1987) of Japan Road Association.

(b) The steep-slope high embankments were successfully constructed in USA using the geogrids of low cost and wide-range tensile strength. The reinforcing materials are the so-called "fiber grids" which are made up of polymer filaments and PVC coated materials and the "combined aramid fiber grids coated with HDPE" manufactured in Japan. The competition of the medium-strength polymer grids on one side and the extremely high above mentioned polymer grids on the other will, it is considered, continue for the next several years.

(c) The vertical block-face wall with anchor resistance of steel bars with plate anchor has been already in practice in Japan (multiple anchored wall system). Just recently the method of using the web made of polyethylene coated polyester fiber connecting the wall with anchor inside the backfill has been successfully introduced to Japan in 1992 (Websol system developed in U.K. around 1980). By this method it is not necessary to lay web throughout the back-fill and the on-site construction works are made easy.

Out of these steep reinforced soil structures the one that is taken up in this report i.e. construction by wrapping the soil with polymer grids is evaluated by the authors as an innovative design that has been practiced after the Terre Armee.

DESIGN METHODS OF REINFORCED EMBANKMENTS

Development of Design Methods

As to the design method of steep embankment reinforced by polymeric materials various research papers have been published since the year 1982 and these papers are described in TABLE II. The common procedure of the design is that the reinforcing materials are laid in parallel in same length with an exceptional case of layer of polymer grids with different lengths. Out of these design methods the design guidelines laid down by Netlon Limited in accordance with the method by Jewell et al. (1984) includes the practical design charts which are the important factors that leads to the wide-spread use of the method.

The Design Method by Jewell et al.

Jewell et al. proposed the two-part wedge analysis model for the ultimate equilibrium condition assuming the slope of the stable embankment on a foundation of adequate bearing capacity as 30° ~ 80° in slope angle and the dynamic or seismic load is not taken into consideration. The design parameters are specified as shown in Fig. 3. The design procedure and the design charts are described as follows with the following computations.

(1) Determination of Geogrid Laying Length

The maximum polymer-grid laying length determined by the following three standards is taken as a required length and a definite length (vertical spacing) is adopted in the vertical direction.

Pattern A

The slope failure pattern is denoted by bilinear sliding failure as shown in Fig.4 (this is

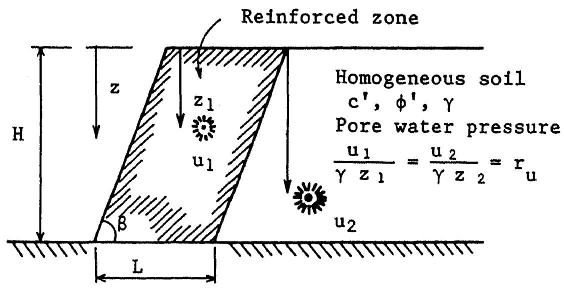


Fig.3. Conditions in the Design

called the two-part wedge failure). The critical slope- failure plane that gives the maximum horizontal pressure while maintaining the equilibrium condition is covered by polymer grids and the required length that may not cause the pull-out of polymer grid is taken as L (Fig.5 (a).

Pattern B

Take the required length L_b (Fig.5(b)) so that the sliding failure at the boundary between the soil and polymer grid may not occur.

Pattern C

Take the length L_c that is required so that the value of σ_{min}, one of the foundation reactions acting against the pressure of the back-fill soil above the base plane of the reinforced zone, may not be negative. Refer to Fig.5(c).

By composing the figures in Fig.5 the char

TABLE II. Development of Design Methods for Embankment Structures Reinforced with Polymer Grids or Other Geotextiles

Author	Reinforce Material (R.M.)	Length of R.M.	Strength of R.M.	Soil Constant	Pore Water Pressure $\gamma_w = u/\gamma z$	Interacting Friction Coefficient	Slope Angle (°)	Ultimate Equilibrium Model	Layer Spacing	Crest Surcharge	Notes
Ingold (1982) (Design chart)	Geotextile	Parallel Same length	- - - -	ϕ'	- - - -	- - - -	30~80	Endless slope Slip circle	Constant	- - - - -	Safety of slope
Jewell et al. (1984) (Design chart)	Geogrid	Same length	Parallel design strength at end of design	Safe ϕ_s'	0,025,0.5	$\delta=0.5\phi_s'$ (Pull-out) $\delta=0.8\phi c'$	30~80	Two-part wedge	Arbitrary	Uniform distribution	Most utilized chart (Design method) Safe design strength = specified inservice-strength/safety factor = $f_k/\gamma_m F_s$
Jones et al. (1984) (Design chart)	Geogrid	Parallel Same length	Same as Jewell's	ϕ'	- - - -	*Refer to Notes	90	Slip plane	Arbitrary	Uniform distribution	Design procedure Coherent gravity and Tie-back wedge
Yamanouchi et al.(1986)	Geogrid	Parallel Same length	40% of tensile strength	Shirasu ϕ' $\tan^{-1}\left[\frac{\phi'}{1.5}\right]$	- - - -	Same as Jewell's	30~80	Two-part wedge	Arbitrary	Uniform distribution	•Basically same as Jewell's method •Apply Richardson's method for seismic design •Tensile strength during earthquake = 1.4Xthat of static condition
Bonoparte et al. (1986) (Design chart)	Geogrid Geotextile	Parallel Same length	50% of maximum strength	ϕ_{cr}'	- - - -	Determine by shear box test	45~90	Two-part wedge	Constant	- - - -	Seismic Design based on static design method (Charting the ratio of dynamic force/static force)
Hirota,others (1986) (Design chart)	Geotextile	Parallel	Longterm tensile strength	ϕ'	Arbitrary γ_w	$\delta=2/3\phi'$	Steep slope	Slip circle log-spirial	Constant	Uniform distribution	Proposed the safety factor map
Sheneider et al. (1986) (Design chart)	Geogrid Geotextile	Parallel	Strength determined from test	ϕ' c'	0.35	$0.5\phi' < \delta < \phi'$	0~40	Two-Part wedge	- - - -	- - - -	Extension of Murray's research, consider the cohesion
Leshchinsky et al.(1987) (Design chart)	Geotextile	Parallel	25~50% of tensile strength	ϕ'	- - - -	$\delta=2/3\phi'$	15~90	Plane slip log-spirial	Constant	Uniform distribution	Refer to the report of Delaware University,1985
Schmertmann et al.(1987) (Design chart)	Geogrid	Parallel Different length	<20~40% of tensile strength	ϕ_t' $\tan^{-1}\left[\frac{\phi'}{1.5}\right]$	- - - -	0.9 times shear strength of soil	30~80	Two-part wedge slip plane	Arbitrary	Uniform distribution	Extension of Jewell's research

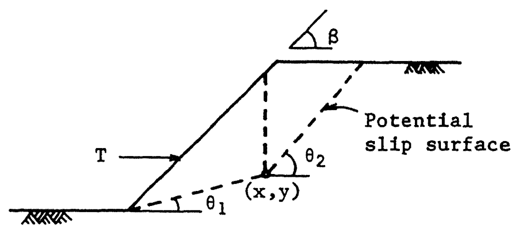
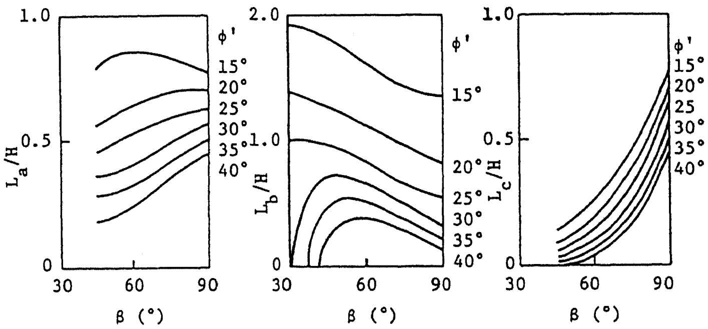


Fig.4. The Two-part Wedge Concept

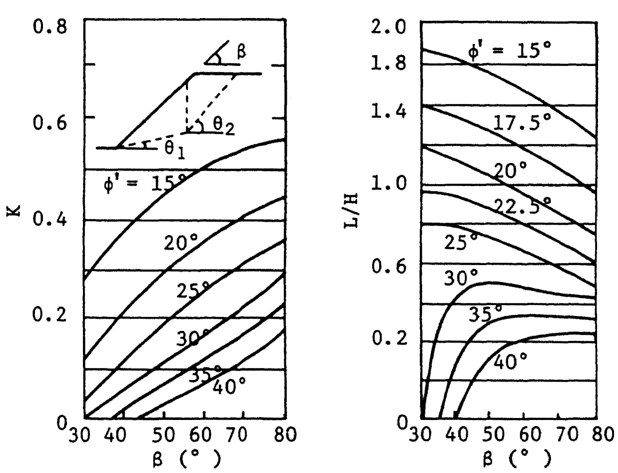


(a) Required length to contain the critical two-part wedge mechanism (Pattern A)

(b) Required length to prevent outward sliding (Pattern B)

(c) Required length to prevent tensile effective stresses on the reinforced zone base (Pattern C)

Fig.5. Charts Used to Determine Required length of Grid on the Basis of Slope Angle



(a) Coefficient of earth pressure

(b) Required length of grid

Fig.6. Charts Used to Determine Coefficient of Earth Pressure and Required Length of Grid on the Basis of β and ϕ'

shown in Fig.6 (b) results in. Moreover the coefficient of earth pressure is taken from Fig.6 (a). From these studies the internal friction angle of pull-out resistance between the soil and polymer grid is taken as $0.5\phi'$ and that of the resistance against sliding is assumed as $0.8\phi'$.

(2) Determination of Polymer Grid Spacing

The tensile force T_i acting on the polymer grids laid at a distance z_i from the crest of the embankment is given by the following equation (1) when polymer grid spacing is V_i.

$$T_i = \sigma_h V_i = K \gamma z_i V_i \tag{1}$$

The maximum polymer grid spacing V_{imax} will be the ideal value when the value of T_i is equal to the designed strength of polymer grid T_D. In other words this means that

$$V_{imax} = T_D / (K \gamma z_i) \tag{2}$$

Here it is convenient to take the spacing of polymer grid laying (V_i) as an integer times the lift height (v) of the embankment compaction. The spacing constant Q is given by the following equation (3) so as to make it possible to construct the embankment of maximum height with the lift height (v).

$$Q = T_D / (K \gamma v) \tag{3}$$

Consequently the possible height of the embankment becomes Q/2 when the spacing of polymer grid laying is 2v and it means that the polymer grids will be laid at a spacing V=v in the portion from Q to Q/2. Similarly the portion from Q/2 to Q/3 will be determined by the spacing V=2v and the portion from Q/n to Q/(n-1) by nv.

Such a design procedure carried out in accordance with Jewell et al. is considered as an excellent design method that leads to the rapid popularity of the construction of unique steep reinforced embankments using the polymer grids wrapping around the fill-material.

A DOCUMENTED CASE HISTORY

Design and Construction

Until now more than 400 embankments including steep-slope ones reinforced by polymer grids and designed in accordance with the design method of Jewell et al. have been successfully constructed in Japan.

Here the general outline of design and construction of two types of steep reinforced embankment will be discussed. These embankments have the salient features shown in TABLE III. Type A embankment is performed in Kagoshima, Kyushu, in 1985 for the first time in Japan.

TABLE III. Specifications of Trial and Prototype Reinforced Embankments

Item	Type A	Type B
Purpose	Road Embank.	Test Embank.
Height, H (m)	6.0 (in design) 7.0 (in practice)	4.5
Slope angle, β	78° (1:2)	90°
Surcharge load, q (kN/m^2)	9.8	0
Fill material	Pumice sandy soil (Shirasu)	Sea sand
Unit weight, γ (kN/m^3)	14.7	15.2
Cohesion, c' (kN/m^2)	24.5	4.9
Angle of internal friction, ϕ '	45.2°	39.0°
Specific gravity, G_s	2.44	2.63
Natural moisture content, ω (%)	19.6	21.8
Temporary works for construction of face	Sand bags	Steel form-work, short geogrid, fabric

(1) General outline of design

The design for steep reinforced embankments is based on the method proposed by Jewell et al. (1984). One consideration is the analysis of the stability of the reinforced zone against external forces, i.e. the external analysis. In this the reinforced zone is considered as a rigid body, and the stability analysis consists of checking safety factors for slipping, sliding, overturning, and bearing capacity against external forces. The other is known as internal stability analysis, in which tensile failure and pull-out failure of the grid against the earth pressure are checked.

Strength parameters in the design were determined as follows.

$$c' = 0\ \mathrm{kN/m^2};\ \phi_d' = \tan^{-1}(\phi'/1.5) = 30°\ (\phi' = 45°)$$

Considering an increase of moisture content due to rain water, the unit weight γ was taken as high as 17.7 kN/m^3.

The design strength of grid T_D was taken to be $0.4T_f$ = 31.4 kN/m. This value of T_D takes into account creep deformation and is less than the short-term ultimate tensile strength T_f of 78.5 kN/m determined from tensile tests run at 50 mm/min.

The design of the type A embankment was based on Jewell's method. In this method the earth pressure is determined by the use of Fig.6(a), and the minimum required length of the grid L_{min} is determined from Fig.6(b). The earth pressure coefficient K in Fig. 6(a), based on the assumption of a bilinear sliding plane, is several % higher than that of the active earth pressure by Coulomb's theory in which the sliding plane is assumed to be linear.

When Fig.6(b) was applied for design, the required length of grid was found to be 4.1 m; the design grid length was taken as 4.5 m. Moreover the vertical spacing V of grids is computed by considering the equilibrium condition of the tensile design strength of the grid and earth pressure forces:

$$V = \frac{T_D}{K\gamma\,(H + q/\gamma)} \tag{3}$$

The vertical reinforcement spacing at the lowest part of the embankment was given by:

$$V = \frac{31.4}{0.277 \times 17.7 \times 6.6} = 0.97$$

Baced on this computation, the grid spacing was taken to be 1 m throughout the whole height of the embankment.

As to the design of the type B embankment, Rankine's active earth pressure coefficient K_a was applied, that is

$$K_a = \frac{1 - \sin\phi'}{1 + \sin\phi'} = \frac{1 - \sin 30°}{1 + \sin 30°} = 0.333 \tag{4}$$

The total grid lengths required for stability against sliding and overturning were 2.8 m and 1.9 m respectively, as computed from considerations of external stability. However a 4.0 m grid length was used in construction. Pull-out failure was analysed for the uppermost grid layer with the bond length L_R selected to give equilibrium between pull-out resistance of the grid and the thrust developed by the earth pressure.

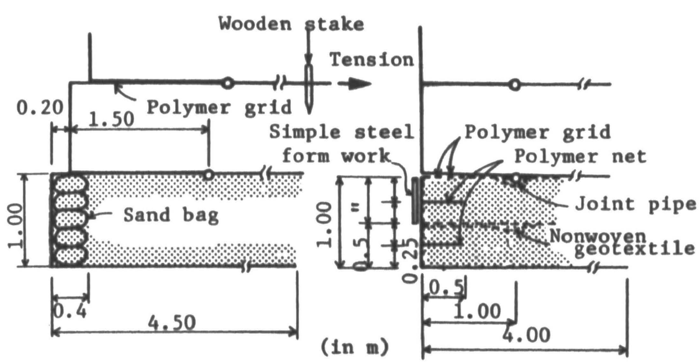
The pull-out resistance T_R is given by a simplified equation:

$$T_R = 2\alpha\,\sigma_v \tan\phi' L_R \tag{5}$$

where the coefficient 2 accounts for friction being developed on both sides of the grid and σ_v is the overburden stress acting on the grid. The factor α is an interaction coefficient multiplied by $\tan\phi'$. Normally α is less than unity and based on laboratory test results it was found to be 0.9. The required grid length L_{req} based on the bond length L_R from equation (5) and the length of active zone L_a was 2.5 m which is less than the length obtained from the external stability analysis.

(2) Construction

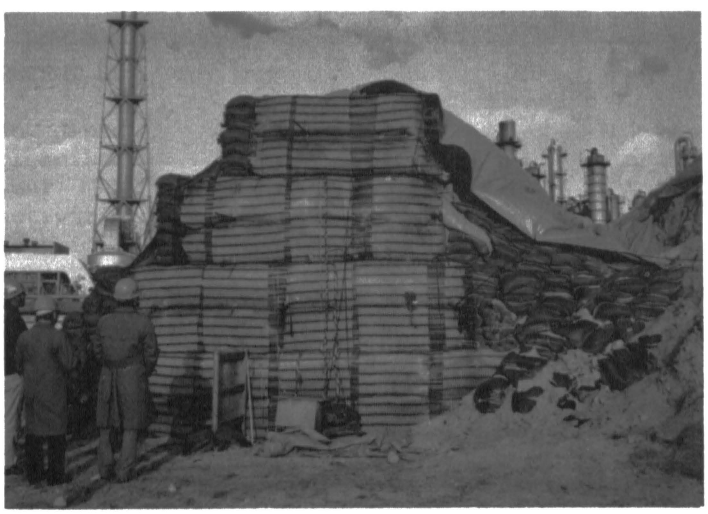
Regarding construction, the finish of the slope face is considered to be a governing factor in the stability of the embankment. In the type A embankment the slope was formed by piling up sand bags (Fig.7(a)), but in type B a simple steel form-work of 0.5 m height was used as temporary support. The slope reinforcing technique for type B is shown in Fig.7(b). In both cases the free end of the grid was stretched out and fixed at the ground by a small wooden stake. Neither method of construction requires working from the front of the embankment; hence speed and safty of



(a) Type A (b) Type B

Fig.7. Types of Slope Formation

(a) Type A, slope angle $\beta = 78°$ (Kagoshima, 1985)



(b) Type B, slope angle $\beta = 90°$ (Iwakuni, 1985)

Fig.8. After Completion of Embankments

construction were ensured by using this type of construction. Fig.8 shows the embankments after construction.

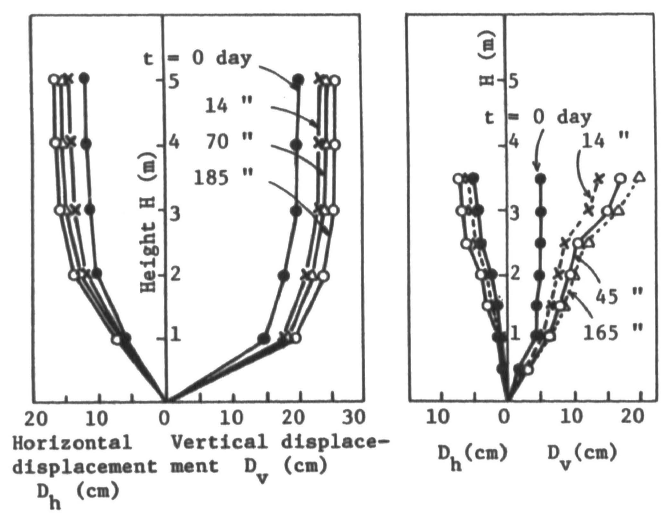
Results of Measurement

(1) Embankment deformation

Fig.9 shows the vertical and horizontal deformations of both embankments. In the type A embankment the displacements in the lowest 2m section were a maximum for both vertical and horizontal directions. On completion of construction these deformation amounted to 90% of the total displacement recorded during monitoring. The vertical displacement at H=1.0m is considered to include the settlement of the initial earth fill layer, of about 2m thickness, at the bottom of the embankment. On the other hand, in type B the trend was the higher the embankemnt the more the vertical and horizontal displacements increase, with the vertical displacement increasing rapidly in upper portion. This was due to the compressive displacement that occured after 18 days' heavy rainfall of 1050mm after completion of construction.

(2) Strain distribution of grid

Fig.10 shows the strain distributions measured by foil strain gauges attached to the grid. In type A the maximum strain was as low as 0.15 ~ 0.3%. Changes in strain after completion of the embankment were insignificant. On the other hand, the maximum strain in type B was 0.05 ~ 0.28% at the end of construction, increasing to 0.19 ~ 0.50% after heavy rainfall. However, the absolute values of strain in both cases were similar and found to be rather small.



(a) Type A (b) Type B

Fig.9. Observed Displacements of Embankment Body after Completion

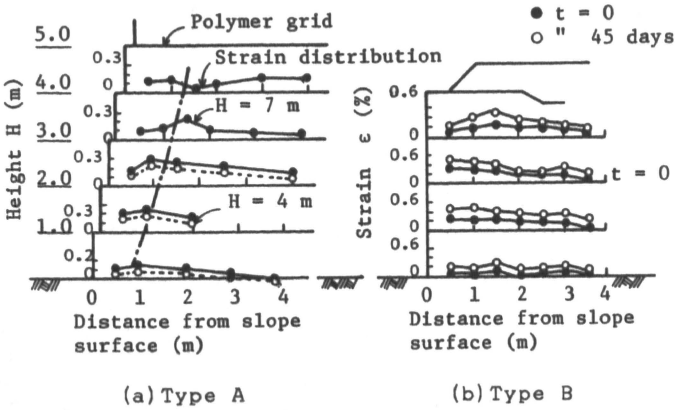


Fig.10. Measured Strain Distribution in Grids

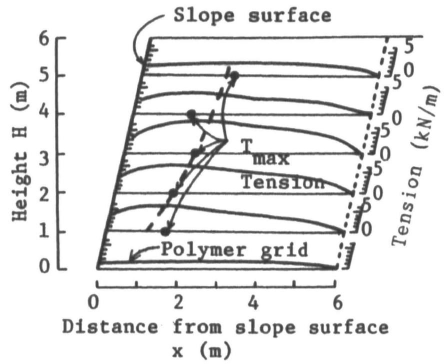


Fig.11. Results Obtained from the Analysis of Finite Element Method (Grid Length=6m)

Comparison of Design and Practice

The reinforcing effect of the grid will be discussed here along with a comparison of design and practice. In analysing the results from the trial embankment it is convenient to convert observed grid strains into tensile forces. In so doing it should be noted that the grid shows visco-elastic bahaviour and consequently the higher the rate of strain used in the tension test the higher the observed tensile strength and tensile stiffness of the grid. The tensile stiffness J = 700 kN/m, at a strain of 1.0%, was obtained from the relationship between tensile force and strain taken from uniaxial tensile tests run at a rate of strain of 2%/min. Thus it is computed that, in type A, T_{max} = 1.1～2.1 kN/m (H = 1～4 m) and in type B, T_{max} = 3.3～3.5 kN/m (H = 1～3 m).

Here tension at each grid will be computed in accordance with the basic design principle. Thus T_{max} becomes

$$T_{max} = K(\gamma z_i + q) V_i \qquad (6)$$

where z_i is the distance between the grid in the ith layer and the top of the embankment. For a vertical grid spacing V_i = 1.0m the grid tension (T_{max}) is computed as follows:

Type A: H = 1～4m; T_{max} = 14.7～29.4 kN/m
Type B: H = 1～3m; T_{max} = 8.8～20.6 kN/m

When referred to the strength parameters from soil test results, T_{max} becomes 4.9～9.7 kN/m and 5.1～12.1 kN/m for type A and type B respectively. In this computation c′ is zero and the earth pressure coefficient K for type A is estimated from the chart for ϕ'=45° (Fig.5(a)). The ratio of the tension computed from the measured strains to the design tensions is 9～26% in type A and 27～67% in type B.

A finite element analysis was carried out for type A before the construction works for the embankment started, taking into account the elasto-plastic properties of the fill material and the friction between grid and fill material (Fig.11). From the results of the above analysis maximum tensions were 3.9～5.9 kN/m for H = 1～4m. These are 50～120% of the maximum tension that is obtained from the computation by using the practical soil properties. One problem is the difficulty of determining the tensile stiffness J from the grid tension computed by the use of measured data. The results from these computations indicate a higher order of reinforcing effect than those derived from the present design method. Such a reinforcing effect can be considered due to the integration effect of grids and soil. This is confirmed by Fig.12 which shows the condition of type B, 4 months after removing the soil in the rear portion of the embankment.

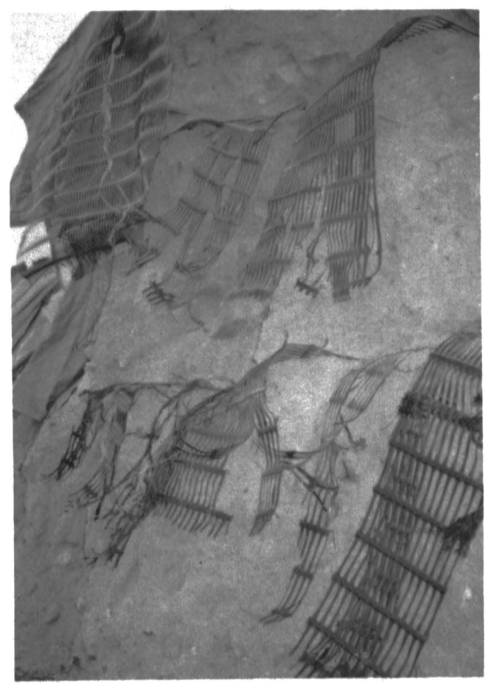


Fig.12. Integration Effect of Grids and Soil after Removing the Rear Back-fill Soil (Type B Embankment)

Concluding Remarks

Results from instrumented full-scale embankements verify some of the assumptions made in design. The following points can be concluded.

(1) Steep slope (slope angles 78° and 90°) reinforced embankments were constructed and the reinforcing effects were thoroughly observed. The structure is, with a small amount of displacement, is resisitant to external forces as conventional gravity type structures. It was noticed that a steep slope, less than 90°, has the effect of lowering the tension on the grid as well as the displacement of the structure.

(2) From the recorded data of the strain distribution it was found that the tension in the grid is much lower than the computed result. Difficulty arises in computing the tensile stiffness of the grid. In other words, it can be concluded that soil and grid are integrated into a rigid body by laying several layers of grids in the fill material. It may be assumed that the whole embankment is completely reinforced.

(3) The designed tensile strength of polymer grids designated by Netlon Limited (1984) is the 50% of 120-year creep strength (40% for wall) for the construction of reinforced embankment. The creep strength taken for the design considering such a long period seems to be as a strength that is not really reflecting the effective life of civil engineering structures.

(4) Furthermore the results of observations of the tensile strength and deformation in the case history reveal that they are rather small when compared with those of the designed values. This topic will not be brought up in this report. The same thing can equally be said of the case history or case histories. Jewell et al. seems to be convinced of this fact.

RECENT DESIGN METHOD IN JAPAN

In Japan a meeting on geogrids (1983) was organised by the author and it developed in the year 1987 into the Geogrid Research Board the activities of which ended in the year 1990. The Geogrid Research Board, represented by the author, Yamanouchi, has published the Guidelines for Geogrids in 1990. As one of these research activities the design method for the steep reinforced embankment is brought under discussion. Accordingly the revised design method was suggested (Fukuda et al. 1989).

Discussions on the Conventional Method

The comments and suggestions on the embankment design made by Jewell et al. are shown in TABLE IV. These comments are, it is expected, fully convinced by Jewell et al. The modified method of reinforced embankment design proposed by Jewell himself (1991) is stated in Appendix A.

Proposal for the Modified Design Method

The general flow chart of the design is as shown in Fig.13. And as to the design parameters that are described in Fig.14 their design considerations can be explained as follows.

(1) Computation for Earth Pressure Coefficient

The earth pressure coefficient is determined by the bilinear sliding failure method. This means that the horizontal earth pressure P_{h1} of zone ① is determined by force polygon method as shown in Fig.15 and the total horizontal earth pressure P_{h2} is also determined as acting on the zone ② similarly by the force polygon method. And the earth pressure coefficient is determined on the basis of equation (7).

Design Parameters
Embankment : height, slope angle
Loading : surcharge, seismic load, etc.
Soil : ϕ', c', γ, etc.
Designed safety factor : F_s

↓

Check Safety Against Sliding
Check if the sliding safety factor satisfies the designed value for total system

↓

Select Geogrid
Designed strength : $T_D = 0.4T_f$ *
Select the geogrid type

↓

Characteristics of Friction Between Soil and Geogrid
$\tau = \alpha\,\sigma_v \tan\phi' + \beta c'$

↓

Determine Geogrid Laying Length
$L = Max(L_a, L_b, L_c)$

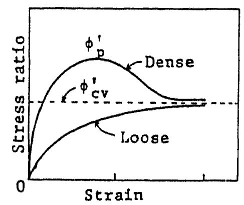
↓

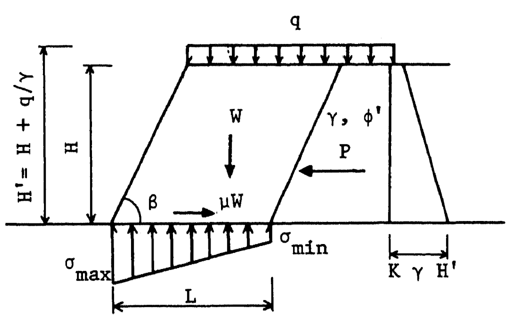
Determine Geogrid Spacing
$Q = T_D/(K\gamma v)$
Make spacing = nV between Q/n~ Q/(n+1)

* Consider whenever necessary the safety of material due to damage during construction, the durability and the strength decrease at joint.

Fig.13. Basic Design Flow-chart

TABLE IV. Jewell's Design Method and Counterproposals by Geogrid Research Board of Japan

Item	Concepts of Jewell's method	Proposal
Way of taking soil constants, ϕ'	Internal friction angle at ultimate condition by direct shear test  	ϕ' by triaxial test, normally the relationship between stress and strain is as shown in the left figure Loose case for compacted soil (ϕ' from direct shear test is too much higher against ϕ' from triaxial test)
Friction characteristics between soil and geogrid	Pull-out : $\mu=\tan(0.5\phi')$ Sliding : $\mu=\tan(0.8\phi')$ μ: Friction coefficient	From test results shearing stress is $\tau=\alpha\sigma_v\tan\phi'+\beta c'$ (α, β change with the kind of soil, for sandy soil, α=0.8 β=0)
Designed strength of geogrid, T_D	$T_D=f_K/\gamma_m \cdot F_s$ f_K : Specified strength with creep consideration γ_m: Partial safety factor considering during-construction damages (sand 1.1~1.4) F_s : Safety factor during service (1.35) For SR2 T_D = 29/1.25X1.35 = 17.2 kN/m	T_D = 0.4 T_f T_f : Peak tensile strength Coefficient 0.4 : Stress level is fixed by considering creep characteristicsp γ_m F_s = 1.0 from practical results For SR2 T_D = 0.4X78.4 = 31.4 kN/m
Factors that determine spacing — Safety factors against • Sliding • Pull-out • Foundation reaction	Not clearly indicated, to refer to Waggle program $\sigma_{min} \geqq 0$	1.5 refer to Road Design Manual 2.0 Same as the left equation
Factors that determine spacing — L/H chart	Not clear as to the composite L/H chart obtained from three charts	Determine the maximum spacing by using three charts separately
Earth pressure coefficient at failure mode	By two-part wedge method	Same as the left method



H : Embankment height q : Surcharge
H' : Equivalent embankment height (= H + q/γ)
β : Angle of inclination of slope
L : Grid laying length
W : Weight of the reinforced zone (= γ L H)
K : Soil pressure coefficient
P : Resultant of horizontal forces (= $0.5K\gamma H^2$)
σ_{max}, σ_{min} : Foundation reactions

Fig.14. Design Parameters of External Forces

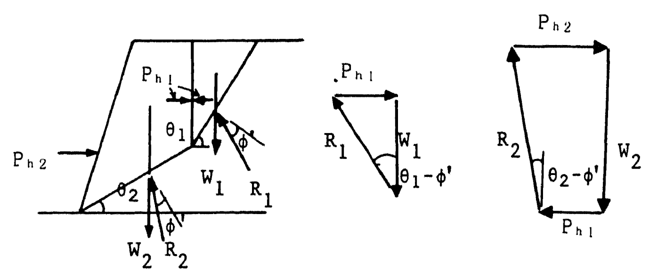


Fig.15. Determination of Horizontal Earth Pressure by Force Polygon Method

$$K = P_{h2}/0.5\ \gamma\ H^2 \tag{7}$$

Moreover in the case of homogeneous fill-material the value of θ becomes: $\theta=45°+\phi'/2$. It is confirmed that the relationship between ϕ' and β according to the above mentioned consideration can be obtained just the same as the one obtained from the Jewell's chart.

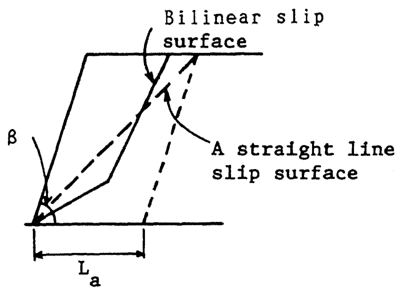
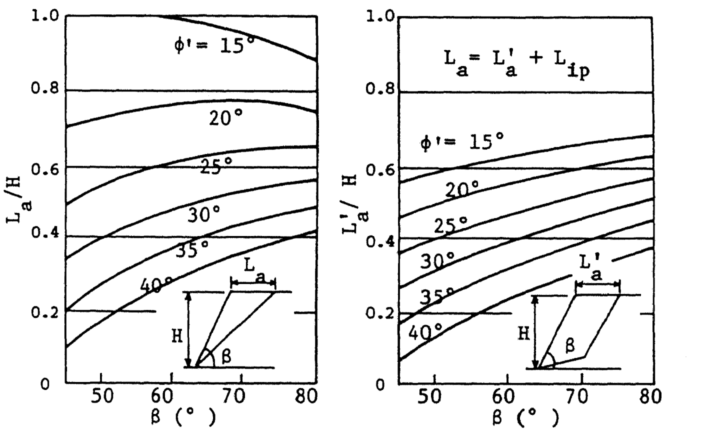


Fig.16. Determination of Grid Length for the Case of Pattern A



(a) Length under the assumption of straight slip surface

(b) Length under the assumption of the two-part-wedge slip surface

Fig.17. Determination of Grid Length for the Case of Pattern A (at Pull-out Failure Mode)

(2) Design on the Laying Length of Polymer Grids

Checking of the laying lenth L_a by the pattern A (Pull-out failure mode)

The length L_a from the point of intersection of the straight failure line and the crest plane to the slope (refer to Fig.16) is taken as the required polymer grid laying length since it covers the bilinear sliding failure zone. The relationship between L_a, ϕ' and β is shown in Fig.17 (a). Moreover the uppermost layer of the reinforcing polymer grids has a problem with respect to the pull-out forces. Hence the bonding length L_{ip} is given by the following equation (8) that is the case the polymer grids are laid at equal spacing (V_i) in the upper portion of the embankment. See Fig.18.

$$L_{ip} = \frac{T_{max}\ F_s}{2\ \mu\ (\gamma\ V_i + q)} = \frac{0.5\ K\ \gamma\ (1.5\ V_i + q/\gamma)^2 \times 2}{2 \times 0.8 \times \tan\phi'\ (\gamma\ V_i + q)} \tag{8}$$

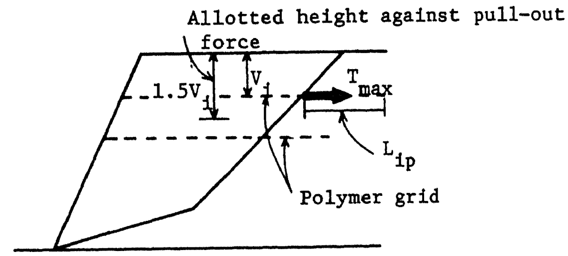


Fig.18. Basic Concept of the Fixed Length

Take $\phi'=30°$, $\gamma=1.8tf/m^3$, $q=0tf/m^2$, $V_i=1.0m$; then $L_{ip}=0.7$ m when $\beta=78°$ (i.e.1:0.2). This means that the bonding length is sufficient if it is 1m long. Hence if the sum of L_{ip} and the distance between the slope and the bilinear failure plane is greater than L_a (the length determined by Fig.16's method), then the length of L_a should be revised. And the length L_a can be determined from the sum of L_a' and L_{ip} (from Fig.17 (b)).

Checking of L_b by the pattern B (Direct sliding failure mode)

The polymer-grid laying length required for securing safety against sliding between polymer grid and soil is determined by the following procedure.

In fact the equation (9) is derived from the equilibrium condition of the resultant of horizontal soil pressures acting in the rear of the reinforced zone (overturning force) P_h and the friction resistance P_r as determined from the dead weight of the soil in the reinforced zone.

$$F_s = \frac{P_r}{P_h} = \frac{\mu\ (\gamma\ L_a\ H)}{(1/2)\ K\ \gamma\ H^2} \tag{9}$$

Here if $\mu=0.8\ \tan\phi'$, $F_s=1.5$, then the polymer grid laying length L_b and H bear the relationship as stated in equation (10).

$$\frac{L_b}{H} = \frac{0.94\ K}{\tan\phi'} \tag{10}$$

K can be obtained from Fig.6 (a) and L_b/H, ϕ' and β bear the relationship as shown in Fig.19.

(c) Checking L_c by the pattern C (Overturning failure mode)

As to checking the pattern C or the bearing capacity of foundation the minimum polymer grid laying length L_c necessary to make $\sigma_{min}=0$ is determined by the following procedure. And the allowable bearing capacity q_a is here assumed to be great enough to make $\sigma_{min} \leq q_a$.

The distance from the toe of slope to the resultant is

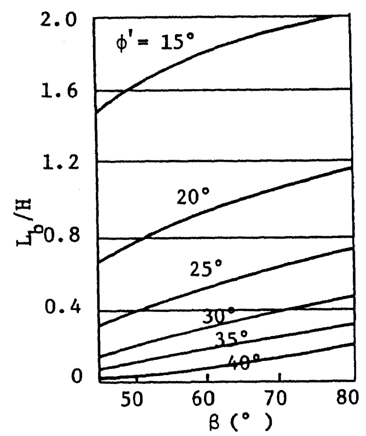


Fig.19. Grid Length for Pattern B

$$d = \{\frac{1}{2}\gamma HL_c (L_c + H\cot\beta) - \frac{1}{6}K\gamma H^3\}\frac{1}{\gamma HL_c}$$

$$= \frac{1}{2}(L_c + H\cot\beta) - \frac{KH^2}{6L_c} \quad (11)$$

When the distance of eccentricity e is given by

$$e = L_c/2 - d = L_c/6 \quad (12)$$

then $\sigma_{min} = 0$.

Hence $d = L_c/3$ (13)

Consequently if L_c/H is compiled from equation (11) and (13) the following equation can be set up.

$$\frac{L_c}{H} = \frac{3}{2}\cot\beta\ \{\sqrt{1 + \frac{4}{9}K\tan^2\beta} - 1\} \quad (14)$$

When the relationship between L_c/H, ϕ' and β is given in the form of a graph then Fig.20 is obtained.

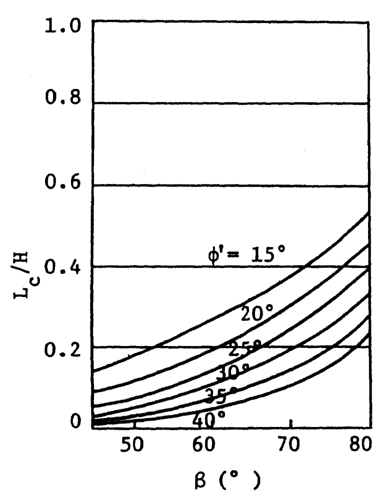


Fig.20. Grid Length for Pattern C

(3) Checking of the polymer-grid laying spacing

The determination of V_i is done in accordance with the Jewell's method. In that the value of V_{imax} should be restricted to 1.0m as viewed from the point of construction. Hence an economical design can be achieved by using the polymer grids of low tensile strength in the area where V_{imax} is rather big.

Additional Remarks

The design method of grid-reinforced steep embankment is revised as mentioned above with a view to assist the designer to secure a check list including the procedure for taking safety factors since the Jewell's chart for the determination of L/H is something like a black-box that controls the whole story. In fact this design method is basically an analysis for ultimate equilibrium condition and the effect of integration of polymer grids and earth into one unit is not taken into consideration. Hence it is deemed necessary to upgrade the method to a rational one by conducting the instrumented observation of dynamic characteristics and the structural analysis of the embankment.

SEISMIC DESIGN METHOD

Basic Concept

The steep reinforced embankments are constructed in large number in Japan by using Jewell et al.'s method, but that method can not be introduced in its original form to the authors' home country since it has the problems such as the way of taking safety factors is different and the safety consideration against seicmic forces is not sufficiently discussed, etc. As to the former problem Fukuda et al. (1989) has proposed some design method. In this report the design method with consideration of seismic forces on the basis of bilinear sliding failure is suggested. Basically the suggested method is for the design of steep reinforced embankment.

The design of steep reinforced embankment with

TABLE V .Stability Factors and Designed Strength of Polymer Grid

Checking Item		Safety Factor or Values	
		Normal	Seismic
External Stability	Slipping	1.2~1.3	1.0
	Sliding	1.5	1.2
	Bearing Capacity	3.0	2.0
	Overturning	e≦L/6	e≦L/3
Internal Stability	Pull-out*1	2.0	1.2
Designed strength of Polymer Grid		$T_D=0.4T_f$	$T_{DE}=1.5T_D$ *2

Note/ e:Eccentricity, L:Length of Reinforced Zone, T_f:Tensile Strength of Grid, *1:Same as Terre Armee,*2:Yamanouchi et al. (1986)

consideration of seismic forces is to be done when the embankment height is more than 8m; but this is not necessarily applied to important structures. The design is conducted by computing earth pressure during earthquake (under seismic condition) based on the seismicity coefficient method. Checking the external stability against sliding and overturning and checking internal stability against pull-out and rapture are done and the normal polymer-grid laying plan will be modified. Moreover the horizontal seismic forces are computed by using k_h ($=\alpha/g$ where α: seismic acceleration, g: acceleration due to gravity) as specified in the Earth Works for Road Structure (JRA 1989).

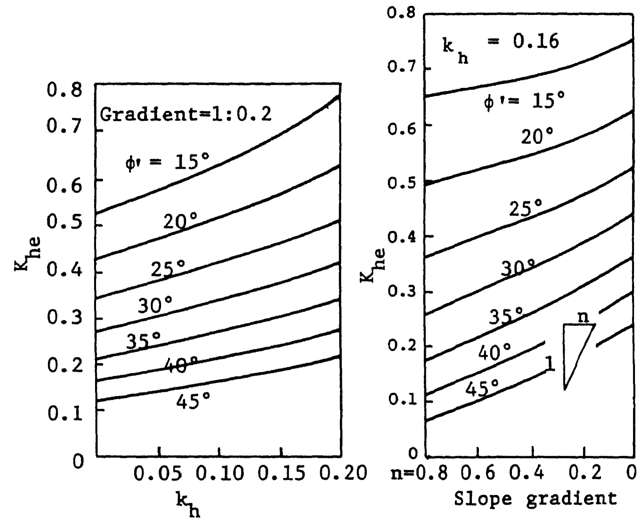
Safety Factor in the Design

The designed safety factors and the designed strength of polymer grid are as shown in TABLE V.

Calculation of Seismic Earth Pressure

The steep reinforced embankment will be assumed as a pseudo retaining wall and the horizontal earth pressure acting on the embankment under seismic forces is computed according to the force polygon method assuming a bilinear sliding failure (see Fig.21). This is the case when the sliding mass inside the bilinear sliding zone is divided into two zones, zone ① and zone ② and determine the horizontal earth pressure P_{h1} that is acting on the boundary section so as to make the force polygon close in equilibrium under the weight W_1 of zone ①, the horizontal inertial force $k_h W_1$ and the resultant force R_1. And then determine the horizontal earth pressure of zone ② P_{h2} (P_{he}) that is acting on the pseudo retaining wall so as to make the force polygon close in equilibrium under the forces of zone ②, W_2, $k_h W_2$, R_2, P_{h1}. The series of computation aim at finding the horizontal pressure when P_{he} is maximum by changing the sliding angles θ_1 and θ_2.

Fig.22 is the example showing the relationship between the horizontal seismicity coefficient k_h, angle of internal friction ϕ' and earth pressure coefficient in horizontal direction



(a) Seismic coefficient of earth pressure vs. seismicity coefficient

(b) Seismic coefficient of earth pressure vs. slope gradient

Fig.22. Curves showing the relationships between k_h and K_{he}, k_h and slope gradient

K_{he} ($=P_{he}/0.5\gamma H^2$) when the slope of the embankment is 1:0.2.

Determination of Laying Length of Reinforcement

The laying length of polymer grids L is determined as the maximum legth out of L_a, L_b and L_c which are found necessary from the point of view of both internal and external stability analyses. And the load condition is as shown in Fig.23.

Checking the External Stability

∘ Provide the laying length L_a with which the sliding of the base of the reinforced zone may not occur under the horizontal inertial force of reinforced zone ($k_h W$) and the earth pressure due to the back-fill during earthquake (P_{he}= 0.5 $K_{he}\gamma H^2$).

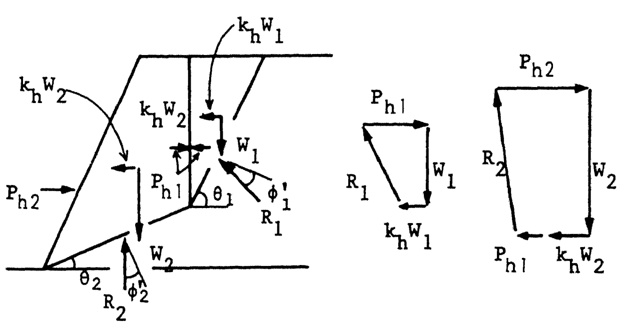


Fig.21. Determination of Earth Pressure Assuming the Two-part-wedge Slip Surface

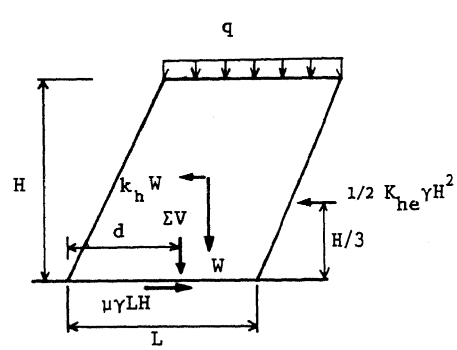


Fig.23. Forces Acting on the Reinforced Body During Earthquake

◦ Provide the laying length L_b with which the overturning may not occur due to the composition of moments of the above two and the gravitational forces.

Checking the Internal Stability

◦ Provide the laying length L_c with which the pull-out of the polymer grids may not take place and the length covers the ultimate sliding plane.

a) Determination of the Laying Length L_a Required against Sliding

The safety factor against sliding during earthquake is given by equation (15).

$$F_s = \frac{\text{the resisting forces against sliding}}{(\text{ inertial force of reinforced zone + earth pressure of backfill) during earthquake}}$$

$$= \frac{\mu\ \gamma\ L_a\ H}{k_h\ \gamma\ L_a\ H + 0.5\ K_{he}\ \gamma\ H^2} \qquad (15)$$

$$\therefore \frac{L_a}{H} = \frac{1}{2}\ \frac{F_a\ K_{he}}{\alpha \tan\phi' - F_a k_h} \qquad (16)$$

Here
- F_s : safety factor against sliding (1.5)
- k_h : horizontal seismic intensity
- K_{he} : earth pressure coefficient during earthquake
- α : interaction coefficient (the correction factor to the friction between the earth and polymer grid, it is 0.8 from experience)
- ϕ' : angle of internal friction of earth
- H : height of embankment
- γ : weight per unit volume of embankment

b) Determination of the Laying Length L_b Required against Overturning

The moment around the toe of reinforced slope is as given below.

Overturning moment :

$$\Sigma M_o = \frac{1}{6}\gamma\ H^3\ \{K_{he} + 3k_h(\frac{L_b}{H})\} \qquad (17)$$

Resisting moment :

$$\Sigma M_R = \frac{1}{2}\gamma\ H^2 L_b\ \{\frac{L_b}{H} + \cot\beta\} \qquad (18)$$

Direct load : $\Sigma V = \gamma\ L_b\ H$ (19)

The ΣV's point of application from toe of the slope is:

$$d = (\Sigma M_R - \Sigma M_o)/\Sigma V \qquad (20)$$

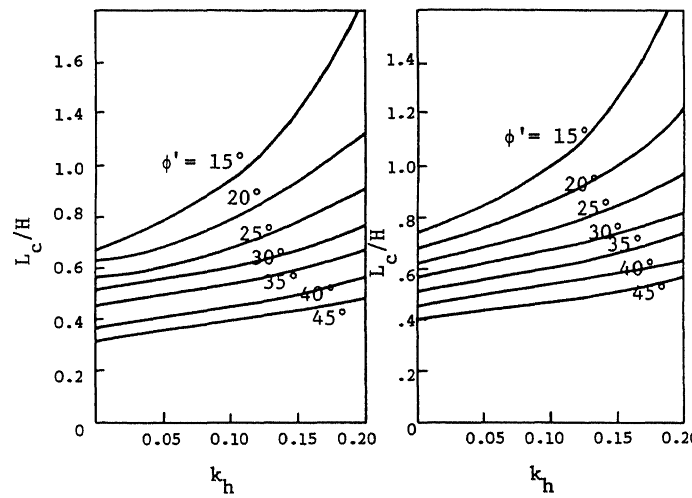
Consequently the eccentricity distance: $e = L_b/2 - d = L_b/3$ (during earthquake) and the value of d becomes $d = L_b/6$

When this value is substituted in equation (20) then L_b becomes as shown in equation (21).

$$\frac{L_b}{H} = \frac{1}{4}\ \{\sqrt{9(\cot\beta - k_h)^2 + 8K_{he}} - 3(\cot\beta - k_h)\} \qquad (21)$$

c) Determination of the Laying Length L_c Required against Pull-out

Fig.24 displays the relationship between L_c/H, k_h and ϕ' after the value of L_c, the distance from the tip of the slope to the point of intersection of the crest and the ultimate failure plane that gives rise to the maximum earth pressure under the condition of linear sliding failure or bilinear sliding failure. L_c is, in case of bilinear sliding failure, the computed result plus the bonding length (normal condition about 1m and during earthquake 0.6m) and for simplicity the case will be sufficient for the linear sliding failure.



(a) The case of the two-part-wedge slip surface patern

(b) The case of the straight slip surface patern

Fig.24. Determination of Length from Earth Pressure Coefficient (Slope=1:0.2)

Determination of Vertical Spacing of Reinforcements

The checking of the vertical spacing of polymer grids is done according to the normal Jewell's design method. This is when, ss shown in Fig.25, the tensional force T_i (equation (22)) that is acting on the polymer grids laid by the vertical spacing V_i at a distance z_i from the crest of the embankment is equal to the designed tensional strength T_{De}, then the grid spacing is the ideal spacing V_{imax} as stated in equation (22).

$$T_i = K_{he}\ \gamma\ z_i\ V_i \qquad (22)$$

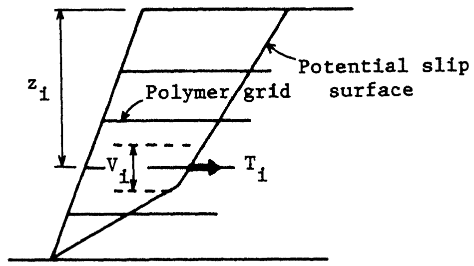


Fig.25. Figure Showing Length Determination

$$V_{imax} = T_{De}/(K_{he}\,\gamma\;\;z_i) \qquad (23)$$

The polymer-grid laying spacing should, however, generally be taken as an integer times the lift v of the embankment compaction (nv) considering the conveniency of construction works. Here take the spacing constant equal to Q_e, the laying spacing V_i equal to v then the maximum embankment height possible to construct will be defined as equation (24).

$$\text{Seismic condition} \quad Q_e = T_{De}/(K_{he}\,\gamma\;\;v) \qquad (24)$$

$$\text{Normal condition} \quad Q = T_D/(K_h\,\gamma\;\;v) \qquad (25)$$

From this equation the possible embankment height is $Q_e/2$ if the value of V_i is taken equal to 2v. Consequently the vertical laying spacing V_i=v in the zone between $Q_e \sim Q_e/2$. Similarly the polymer grids are to be laid with spacing nv for the zone between $z_i = Q_e/n \sim Q_e/(n+1)$. The spacing constant for normal condition will be as defined by equation (25). For both equations T_{De}/T_D=1.5 and for almost all cases the spacing is determined with respect to normal condition since $K_{he}/K_h \leqq 1.5$.

Additional Remarks

Here the seismicity resistant design method which is in accord with the bilinear sliding failure is proposed for the steep reinforced embankment. In using this method for checking sliding failure the earth pressure of backfill during earthquake will be considerably big when ϕ' is small. That is when ϕ'=25°, L_a=20.9m. Moreover as to computation for sliding the working group of Geogrid Research Board has decided to sanction the use of earth pressure coefficient for normal condition as prescribed by Manual for Earth Works, JRA (1989).

In Japan the method of computation for seismicity resistant design is first formulated classically by R. Sano in the year 1914 for such structures as slope retaining walls and it was modified by Mononobe in 1933. Since then the method has not undergone any further progress. And the model testings are, even now, mostly limited to conducting by seismic or vibrational forces in horizontal direction. The design and testing are insufficient for the case of the so-called vertical earthquake that involves vertical seismic forces.

Japan has not had any experience of destruction of steep reinforced soil structures under seismic forces. Hence the fact-finding on-site report by J.G. Collin (1992) on the Loma Prieta earthquake of 1989 that involved HDPE steep reinforced slopes and walls is of great value to Japanese engineers. According to this report the withstanding of the steep grid-reinforced structures under seismic forces is a desirable information to the authors who are working to promote the construction of these structures.

CONCLUSION

From the above mentioned state-of-the-art design for the steep embankment reinforced by polymer grids the following points can be concluded.

(1) Out of the various reinforced soil structures that have been developed since Terre Armee the state of the art of steep reinforced embankment is fully described.

(2) The design method by Jewell et al.,the most popular method of the steep reinforced embankment designs, is introduced.

(3) A documented case history, a representative or standard one in Japan, is introduced. The results of observation are compared with those of the design by Jewell et al.

(4) Based on the results of the above comparison and other case studies the problems that may be encountered in using the Jewell et al. method are summarized. In addition to this the modified design method adopted in Japan is presented.

(5) The design method that takes into consideration the dynamic loads (these are not considered in Jewell et al. method) is very much important as viewed from a country frequently hit by earthquake. The recent seismicity resistant design method adopted in Japan is introduced.

ACKNOWLEDGEMENT

The authors express sincere thanks to the Geogrid Research Board (1987~1990) for permitting them to make use of the invaluable information and data obtained as the results of research and discussion of the Board Members on the topic of polymer grids.

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NOTATION

D_h	Horizontal displacement
D_v	Vertical displacement
F_s	Safety factor
G_s	Specific gravity of soil particles
H	Height of embankment
H'	Equivalent height of embankment including surchrage load $(H+q/\gamma)$
J	Tensile stiffness of grid
K	Earth pressure coefficient
K_a	Ditto in active state
K_d	Designed value of earth pressure coefficient $(=K_{req}/(1-L_B/L_R))$
K_h	Horizontal earth pressure coefficient
K_{he}	Horizontal earth pressure coefficient during earthquake
K_{req}	Earth pressure coefficient giving minimum required reinforcement force
L	Length of reinforced body
L_B	Bond length for reinforcement at the base of the slope
L_R	Reinforcement length
L_a	Required length of grid against pull-out failure of reinforced body
L_a'	$=L_a+L_{ip}$
L_b	Required length of grid against direct-sliding of reinforced body
L_c	Required length of grid against over-turning of reinforced body
L_{ip}	Bond length of grid
L_{min}	Minimum length of grid
L_{req}	Required length of grid
M_d	Driving moment on reinforced body
M_r	Resisting moment of reinforced body
P_{all}	Allowable reinforcement force
P_h	Horizontal force of earth pressure
P_{h1}	Ditto from zone ①

P_{h2} Ditto from zone ②
P_{he} Horizontal force of earth pressure during earthquake
P_r Friction resistance
Q Spacing constant
Q_e Ditto during earthquake
R_1 Resultant force in zone ①
R_2 Ditto in zone ②
T_D Designed tensile strength of grid
T_{De} Ditto during earthquake
T_f Tensile strength of grid at failure
T_R Pull-out resistance of grid
T_i Tensile force on the ith grid member
T_{max} Maximum tensile force on grid
T_{min} Minimum tensile force on grid
V Vertical spacing between grid layers
V_i Ditto of the ith grid layer
V_{imax} Maximum (ideal) ditto
W Weight of reinforced body
W_1 Weight of zone ①
W_2 Ditto of zone ②
W_r Width of reinforcement
c' Cohesion in terms of effective stresses
d Distance from the toe of slope to the resultant
e Eccentricity distance
f_{ds} Direct shearing coefficient of soil over a reinforcement layer
f_k Characteristic strength of grid with consideration of creep
k_h Seismicity coefficient (α/g, α: seismic acceleration, g: acceleration due to gravity
n Integer (number)
q Surcharge load
q_a Allowable bearing capacity of foundation
r_u Ratio of pore water pressure and γz
s_h Horizontal spacing between reinforcements
s_v Vertical spacing between reinforcements
u Pore water pressure
v Lift or vertical depth of compacted soil layer
w Water content ratio
x a coordinate in x direction
y Ditto in y direction
z Ditto in z direction
z_{crit} Critical depth from the slope crest
z_i Depth of the ith grid from the crest
z' Equivalent depth ($=z+q/\gamma_d$) of grid
α Coefficient of interaction between grid and soil for friction component
β Ditto for cohesion component or slope angle of embankment
γ Unit weight of soil
γ_d Design value of unit weight
δ Angle of shearing resistance between soil and reinforcement surface
θ_1 Variable angle of sliding plane in zone ①
θ_2 Ditto in zone ②
μ Coefficient of friction
σ_{max} Maximum reaction of foundation
σ_{min} Minimum reaction of foundation
σ_{req} Required stress in the soil to be provided by reinforcement
σ_{av} Available stress in the soil from the reinforced body ($=P_{all}/s_v s_h$)
σ_h Horizontal stress
σ_v Vertical stress
τ shearing stress between soil and reinforcement ($=\alpha\sigma_v\tan\phi'+\beta c'$)
ϕ' Effective angle of internal friction
ϕ_d' Ditto in terms of design value
ϕ_{cs}' Ditto in terms of critical state or large deformation (shearing strain)
ϕ_p' Ditto in terms of peak state

APPENDIX A REVISED DESIGN CHART BY R.A. JEWELL

Background

As introduced in the main text the method has been in use by Jewell et al. and Netlon Limited since 1980s. As the actual measured values of the tensile strength and the deformation of the reinforcing grids are smaller than the expected or designed ones some considerations as to the method are made so as to reduce the reinforcements as much less as possible. And as a result an economical and easy-to-use design chart was submitted by R.A. Jewell (1991).

Procedure for Simplified Design

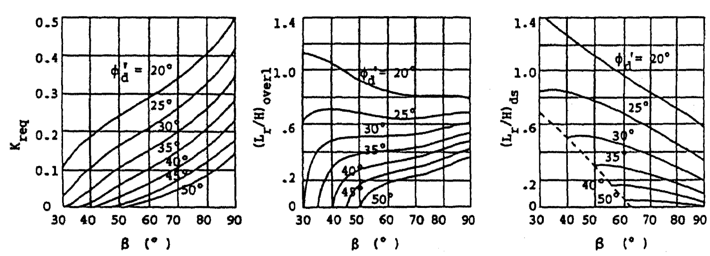
Fig.A-1 is the design chart for the case of the coefficient of pore water pressure γ_u = 0.0 with the assumption that the direct shear coefficient f_{ds}=0.8. The charts for γ_u=0.2 and γ_u=0.5 are also included in the main text.

(Step 1) Determination of Design Parameters

(1) Fix the design parameters and determine the required earth pressure coefficient K_{req} and the required length factor for overall stability $(L_R/H)_{ovrl}$ and direct shear stability $(L_R/H)_{ds}$. If the additional safety factor $f_o \geq 1$, increase each parameter proportionately.

(2) Ensure that the required length $(L_R/H)_{ds}$ is valid by checking that $f_{ds} \geq 0.8$. If this is not the case, increase the required reinforcement length $(L_R/H)_{ds}$ by a factor 0.8/ f_{ds}.

(3) The length of the reinforcement is to be determined in accordance with the following procedure.



(a)Min. required force K_{req} (b)Min. required length$(L_R/H)_{ovrl}$ (c)Min. required length$(L_R/H)_{ds}$

Fig.A-1. Design Charts Used to Determine the Required Minimum Length from Slope Angle

Select the reinforcement length arrangement as follows.

(a) Where $(L_R/H)_{ovrl} > (L_R/H)_{ds}$ choose reinforcement with a constant length $L_R/H = (L_R/H)_{ovrl}$

(b) Where $(L_R/H)_{ds} > (L_R/H)_{ovrl}$ either
  (i) choose reinforcement with a constant length $L_R/H = (L_R/H)_{ds}$, or
  (ii) choose reinforcement with a length varying uniformly from $(L_R/H)_{base} = (L_R/H)_{ds}$ at the base to $(L_R/H)_{crest} = (L_R/H)_{ovrl}$ at the crest.

(4) Determine the value of L_B/H by the following equation (A1).

$$\frac{L_B}{H} = \left(\frac{P_{all}}{\gamma H^2 2W_r}\right)\left(\frac{1}{f_b \tan\phi_d'}\right)\left(\frac{1}{1-\gamma_u}\right) \qquad \text{(A1)}$$

(Step 2) The Distribution of the Maximum Required Earth Pressure

(1) Computation of the required earth pressure at the distance z from the crest

$$\sigma_{req} = \gamma_d \ z \ K_{req} \qquad \text{(Fig.A-2(a))} \qquad \text{(A2)}$$

From the absolute safety consideration the designed earth pressure coefficient will be increased by the following equation.

$$K_d = K_{req}/(1-L_B/L_R) \qquad \text{(A3)}$$

(3) The additional reinforcements are required when viewed from the necessity of bonding strength for the stability of the zone from the crest to the vicinity of the slope. The minimum required stress at the embankment crest σ_{min} is given by the following equation.

$$\sigma_{min} = \gamma_d \ z_{crit} K_{req} = \gamma_d H (L_B/L_R) \ K_{req} \qquad \text{(A4)}$$

(Step 3) The location of Point of Application of Minimum Stress of Reinforcement

(1) The laying of reinforcement is fixed so as to make the minimum reinforcement's stress $\sigma_{av} = P_{all}/(S_v S_h)$ big enough with respect to the distributed value of the maximum required earth pressure where S_v and S_h are the vertical spacing and the horizontal spacing of the reinforcements respectively.

(2) The laying of polymer grids with a constant spacing and layer-wise system is adopted here. The location where the reinforcement stress is maximum will be the lowest layer and the following inequality equation must be satisfied.

$$P_{all}/(S_v \ S_h) \geqq \gamma \ H \ K_d \qquad \text{(A6)}$$

(3) When the laying spacing is changed to z_2 from the crest the depth z_2 will have to satisfy the above inequality equation.

(4) Plot the envelope of available stress and determine the height of the reinforcement layer by showing the maximum depth where the spacing is changed. The boundary of the position of change of layer spacing is at the positi the lowest layer situated at z_2 from the ba shown in Fig.A-2(b).

(5) The maximum value of S_v for the desi specified by the following equation.

$$(S_v)_{max} \leqq \text{Minimum } (H/8, 1m)$$

Moreover, in case the deformation of the wrapped-up slope of the reinforced embankme restricted the recommended maximum spacir the polymer grids is 0.5m.

(6) For the case of the uniform surcharge embankment height $H' = H + q/\gamma_d$ and equivalent depth $z' = z + q/\gamma_d$ and use the mentioned procedure.

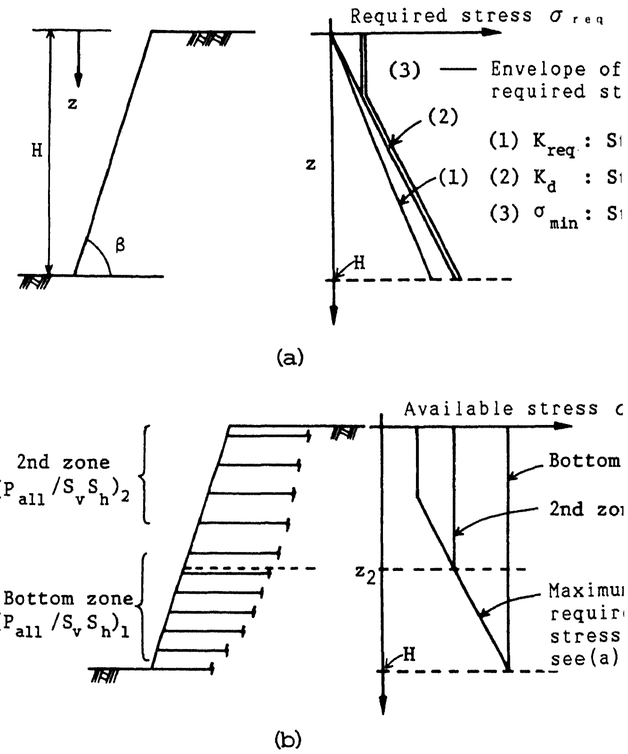


Fig.A-2. Envelopes of (a) Maximum Required Stress and (b) Minimum Available S in a Steep Reinforced Slope