A Full Scale Model Test of a Polymer Grid Reinforced Embankment with Pumice Soil

T. Yamanouchi
Kyushu Sangyo University, Fukuoka, Japan

Y. Takenaka
Ministry of Construction, Kurume, Japan

T. Ohtake & E. Hamada
Kiso-Jiban Consultants Co Ltd, Fukuoka, Japan

K. Orihara
Kiso-Jiban Consultants Co Ltd, Singapore

ABSTRACT : For the design of reinforced embankment with a pumice soil "Shirasu", it is required to take into account the characteristics of the soil, especially its erosiveness. The authors constructed a test embankment with the pumice soil and reinforced by polymer grid. Based on the observation of this embankment, the authors assessed the applicability of the two design methods currently used in Japan. One method is the two-part wedge design method proposed by Jewell et al. (1984) and Geogrid Research Board, Japan (1990) and the other is a limit equilibrium method using a slip circle analysis proposed by Doboku Research Centre, Japan (1993). The comparison of the design and the observations indicated that the shear strength of soil reduced by a factor of safety needs to be used in determination of the required length of polymer grid, while the strength without reduction can be used for determining the required tensile strength of the grid.

1. INTRODUCTION

There is widely distributed an unwelded pumice flow deposit called "Shirasu" in the southern part of Kyushu island, Japan.

The pumice soil has been popularly used in the area as the fill material inevitably and the soil is of high gain in strength after compaction. Recently, there is an increase in the use of pumice soil for the reinforced embankment, too. However, since the pumice soil is extremely erosive, further studies need for designing reinforced embankment using this soil.

The purposes of the present study are to assess the suitability of Shirasu for reinforced embankment and to study the appropriate design method referring to the manuals by Geogrid Research Board (1990) and Doboku Research Centre (1993). To achieve these purposes, an embankment with a steep slope reinforced by polymer grid was constructed. The behaviour of this test embankment was monitored for a year from November 1991 to November 1992. The tension generated in the polymer grid, the degree of saturation of the fill material, horizontal stress induced in the embankment and displacement of the embankment were monitored.

2. FAVOURABLE PERFORMANCE AGAINST EROSION

A slip failure took place in the part of embankment without reinforcement due to a heavy rain (450 mm/day) in August 1993, while no defect was observed in the reinforced part as shown in Figure 1. Displacement of the reinforced embankment was very small during the monitoring period as shown in Figure 2. It proved that the reinforced embankment could work favourably against erosion.

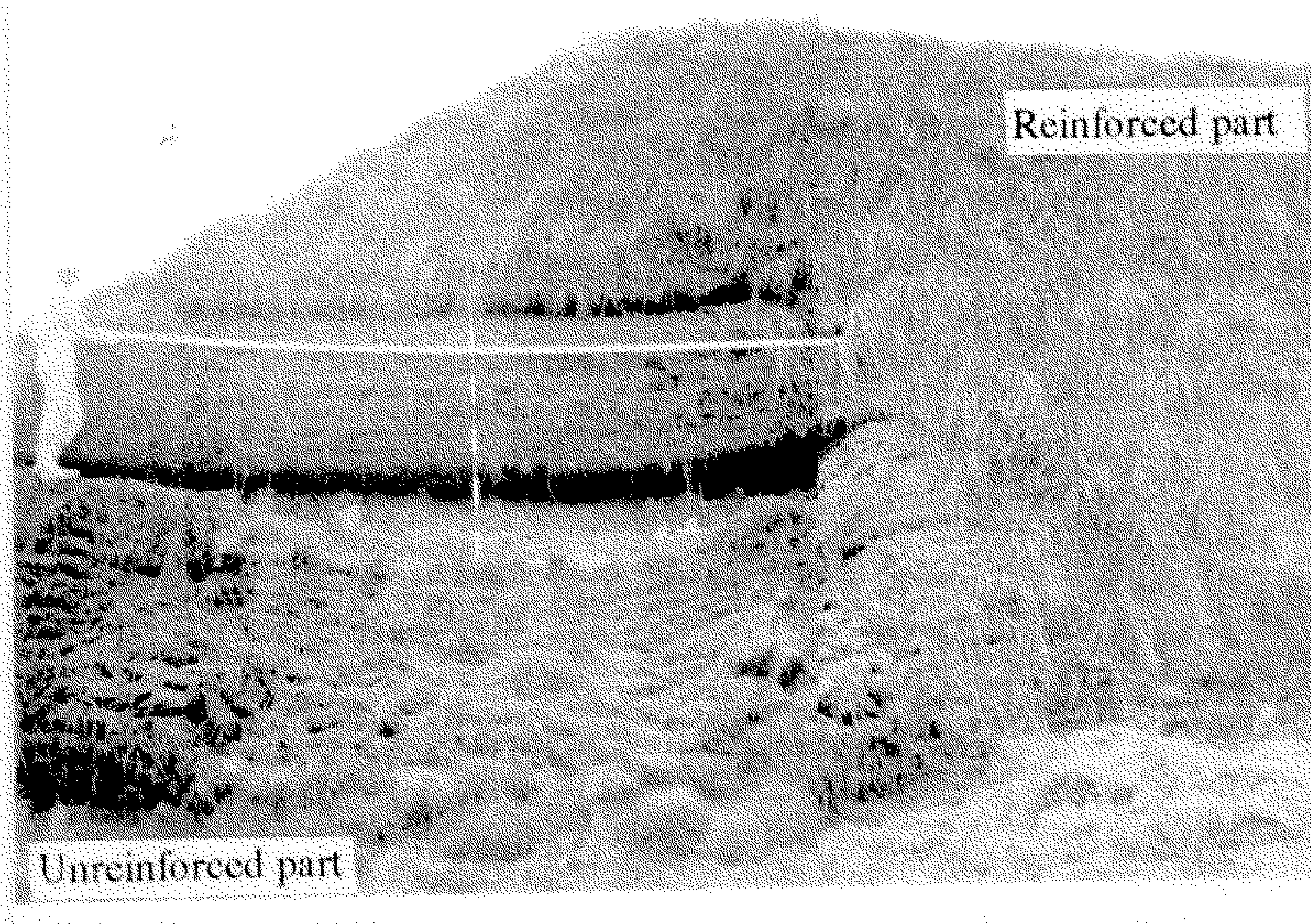


Figure 1 Slip failure in unreinforced part due to heavy rain in August 1993

3. DESIGN METHOD

3.1 Design Parameters Used

The properties of Shirasu used in the present study are presented in Table 1. The design strengths are determined as follows:

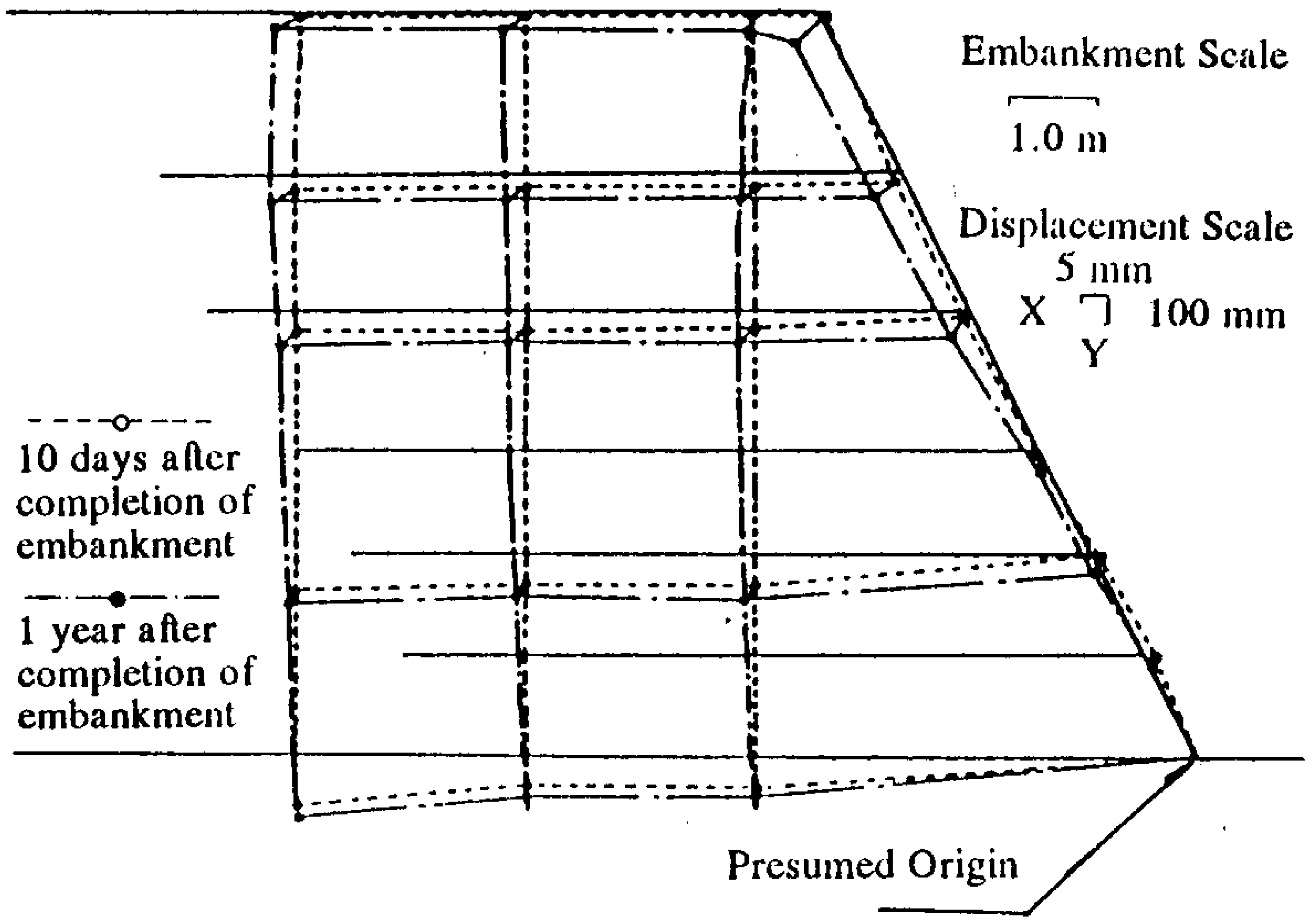


Figure 2 Displacement of test embankment

$$\tan \phi_F = \tan \phi'/F_s$$

$$\phi_{design} = \phi_F = \tan^{-1}(\tan \phi_d'/F_s)$$

We applied $\phi' = \phi_d = 40$ degrees and $F_s = 1.5$ in the above equation as follows:

$$\phi_{design} = \phi_F = \tan^{-1}(\tan 40°/1.5) = 29 \text{ degrees}$$

We neglected cohesion in the design, i.e., $c_{design} = 0$ as proposed by Jewell et al., (1984), instead of using $c_{design} = c_d/F_s = 44/1.5 = 29$ kN/m^2.

Table 1 Properties of the fill material "Shirasu"

Grading of Shirasu used	
Gravel (%)	18
Sand (%)	67
Silt (%)	11
Clay (%)	4
Coefficient of Uniformity	20.9
Coefficient of Curvature	1.90
Japanese Soil Classification	SV (Sandy and Volcanic)
Specific Gravity of Particles, G_S	2.43
Natural Water Content, w_n (%)	16.8
Max. Dry Unit Weight, γ_{dmax} (kN/m^3)	13.8
Optimum Moisture Content, w_{opt} (%)	24.7
In-Situ Wet Unit Weight, γ_t (kN/m^3)	15.5
Drained Peak Strength	
Cohesion, c_d (kN/m^2)	44
Friction Angle, ϕ_d (degree)	40

3.2 The Test Embankment

The test embankment was constructed as a berm adjacent to the newly-constructed road embankment. The embankment has a slope gradient at 1 in vertical to 0.5 in horizontal (63.4°). It has a height of 6.5 m, a length of 20 m, and a width of 17 m at the top. Behaviour of the embankment was monitored by geotechnical instruments installed as shown in Figure 3.

Polymer grid SR55 with design strength $T_d = 22$ kN/m was used for the reinforcement.

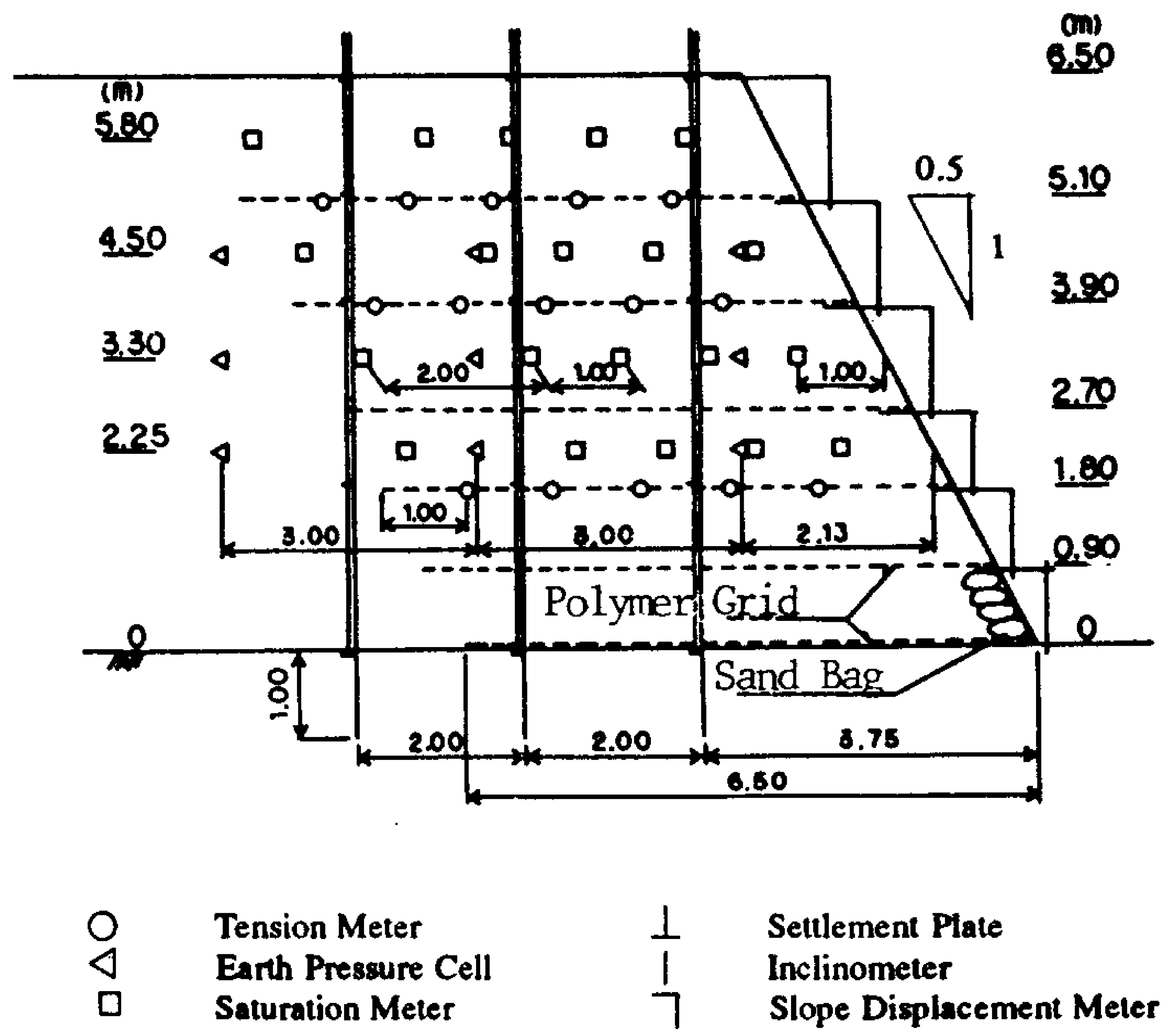


Figure 3 Geotechnical instruments installed in the test embankment

3.3 Comparison of the Design Method

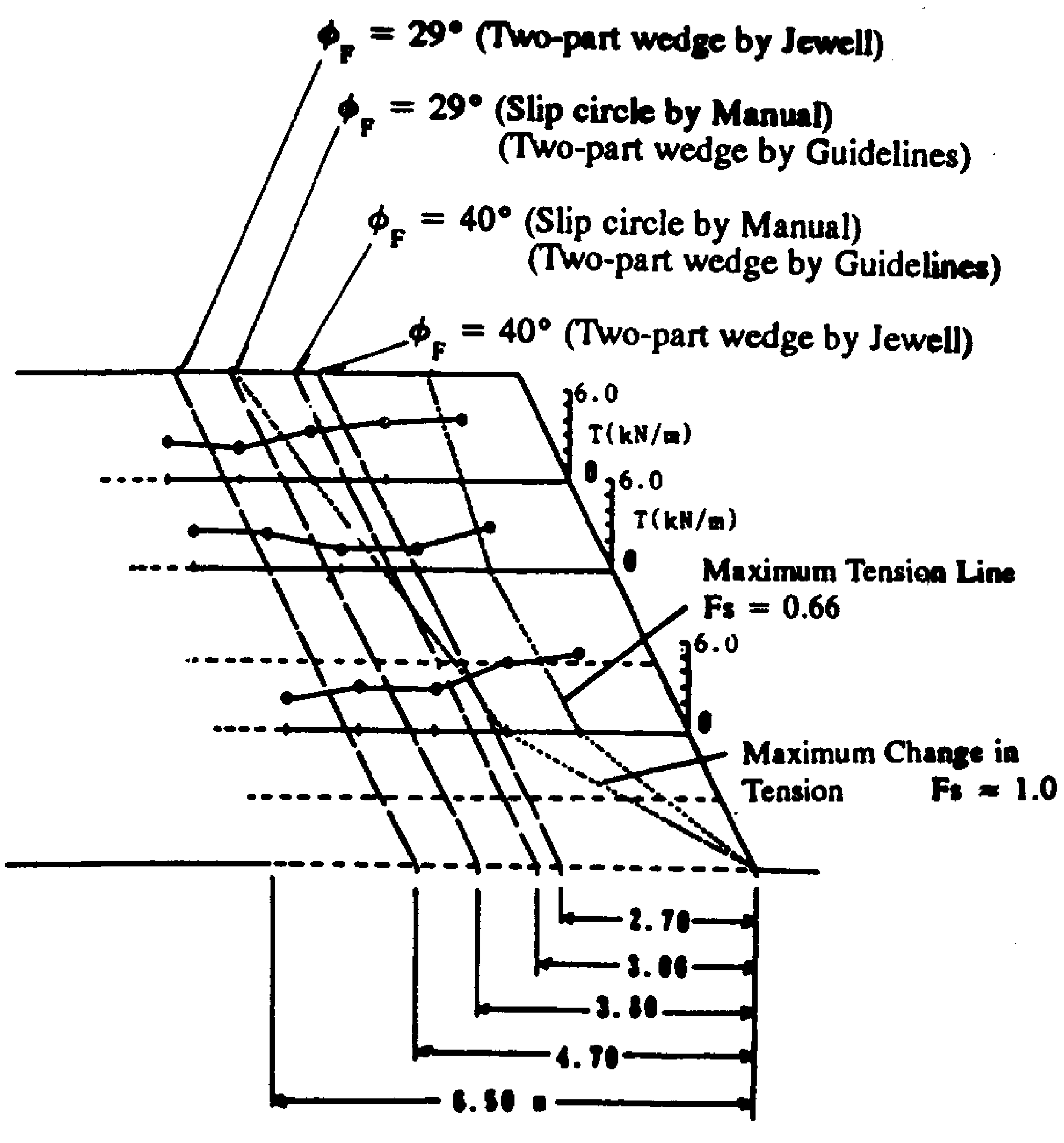
For designing the embankment reinforced by the polymer grid, the design charts by Jewell et al. were first introduced into Japan by Netlon Ltd., UK. Geogrid Research Board, Japan, published "Guidelines for Geogrids". Doboku Research Centre published "Design and Construction Manual of the Reinforced Embankment by Geogrid". Table 2 compares these three methods. The design in the present study was based on the method proposed by the Guidelines.

4. REINFORCEMENT LENGTH

Figure 4 shows tensile force observed in each layer of the reinforcement. Also shown in Figure 4 are a line which chains the maximum tensile force observed in each layer of reinforcement and a line which chains the maximum change in tension, i.e., the line that divides the reinforced area into low and high tension zones.

Table 2 Comparison of three design methods

Item	Jewell et al. (1984)	Guidelines (1990)	Manual (1993)
Method of Analysis	Two−Part Wedge	Two−Part Wedge	Slip Circle
Required Tensile Reinforcement Force	66.6 kN/m	65.2 kN/m	81.1 kN/m
Bond Coefficient	−	0.8	1.0
Location of Critical Failure Plane	−	2.9 m	2.9 m
Required Bond Length of Reinforcement	−	0.6 m < 1.0 m	0.5 m < 1.0 m
Required Reinforcement Length	4.7 m	3.8 m	3.8 m
No. of Layer of Reinforcement	7	7	8



* Fs = a factor of safety obtained for unreinforced embankment

Figure 4 Observed tensile force in reinforcement and required reinforcement zones estimated by various design methods

The maximum tension line is located about 2 m inward from the slope surface. We assume that this line is a potential slip surface. For the low tension zone, we assume that there is a sufficient bonding between the reinforcement and soil because the observed tension is small. The factors of safety obtained by the stability analysis along the lines of the maximum tension and the maximum change in tension are 0.66 and 1.0 respectively when no reinforcement is assumed to be provided and the shear strength of $\phi' = 40°$ is used.

Accordingly, the reinforcement must extend beyond the line of the maximum change in tension. Figure 4 shows required reinforcement zones determined by the two-part wedge and the slip circle analyses. If the parameter of $\phi' = 40°$ (ϕ_d) is used in the design, both analyses provide smaller reinforcement zones than the high tension zone determined by the observed maximum change in tension. Hence, the design lengths determined by these methods are insufficient.

If the reduced parameter of $\phi' = 29°$ (ϕ_F) is used, the required reinforcement zones determined by both methods can cover the high tension zone. This friction angle ($\phi' = 29°$) is almost equal to a low mobilized shear strength of $\phi_{mob} = 28°$ which can be expected at the low strain level estimated by the observed strain level in the reinforcements. It is therefore concluded that the strength reduced by a factor of safety (ϕ_F) must be used to determine the reinforcement length.

5. TENSILE FORCE IN REINFORCEMENT

Figure 5 shows the calculated horizontal force R_H based on the two-part wedge method and ϕ' of 40° (ϕ_d). This is an additional resisting force that is required for supporting the unreinforced embankment.

The calculated R_H is 27.3 kN/m for the slip surface at the maximum tension line and is -3.4 kN/m (approximately zero) for the slip surface at the maximum change in tension. The calculated R_H of 27.3 kN/m is equal to a total of the maximum tensile forces observed in each layer of the reinforcement.

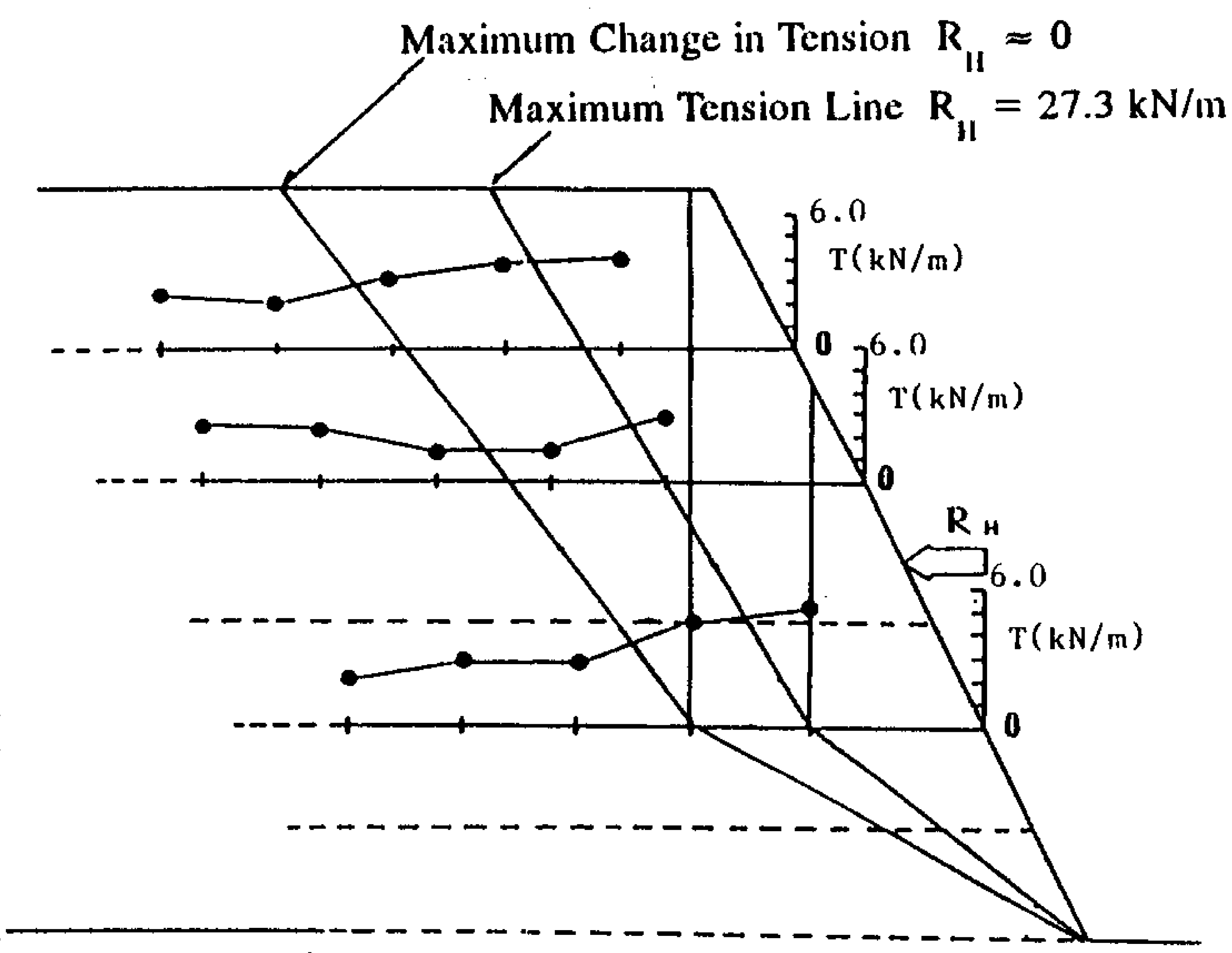


Figure 5 Required horizontal tension R_H calculated by two-part wedge method

The tension calculated based on the two-part wedge method and ϕ_d agrees well with the observed tension.

Figure 6 shows a factor of safety obtained by the limit equilibrium method using a slip circle analysis. The equation used in the stability analysis is as follows using common symbols:

$$F_s = \frac{R\cdot\Sigma(c'\cdot l + W\cdot\cos\Theta\cdot\tan\phi' + T\cdot\sin\Theta\cdot\sin\phi' + T\cdot\cos\Theta)}{R\cdot\Sigma W\cdot\sin\Theta}$$

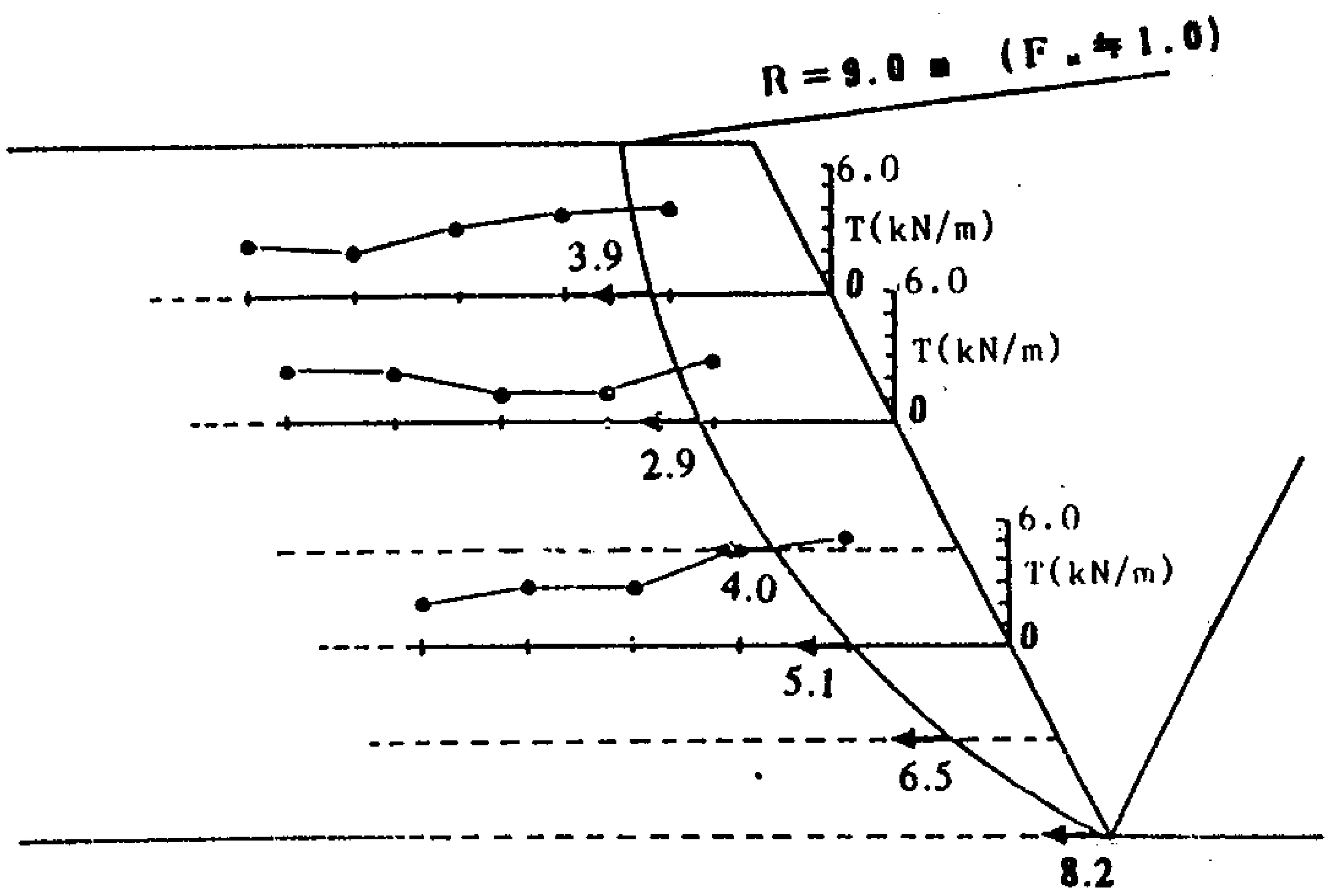


Figure 6 Slip circle passing the observed maximum tension line and factor of safety using observed tensile force

The factor of safety was calculated for a slip circle passing the observed maximum tension line. A sum of the observed maximum tension along the tension line was used for the horizontal tension T in the calculation.

If we assume that ϕ_d will be fully mobilized when the maximum tension takes place, the factor of safety of 1.0 is calculated by using $\phi' = 40°$ and $c' = 0$. This is the same factor of safety as obtained by the two-part wedge method.

In addition, the factor of safety of 1.0 is also calculated by using a mobilized cohesion of Shirasu material, $c'_{mob} = 4$ kN/m^2 and a low mobilized shear strength of $\phi_{mob} = 28°$.

In summary, the peak shearing resistance of ϕ_d (neglecting cohesion as $c' = 0$) can be used to determine the required tensile strength of reinforcement.

Figure 6 shows the maximum tension line. The maximum tension, T_{max} observed is 5.1 kN/m which is only 23% of the allowable force of polymer grid used. For the Shirasu embankment, the reinforcement with a lower characteristic strength can be used. One of the reasons why we observed the low tension may be due to an apparent cohesion induced by the interlocking effect of Shirasu particles which is its characteristic property.

6. CONCLUSIONS

(1) The reinforcement length determined using the peak or large strain strength of $\phi' = \phi_d$ is insufficient when compared with the observed tension in the reinforcement. This was understood in the analyses using the two-part wedge method and the slip circle analysis. For the determination of reinforcement length, the strength of ϕ' of Shirasu should be reduced using a factor of safety.

(2) The tensile force observed in the reinforcement in Shirasu embankment agrees fairly well with the required horizontal tension calculated by the two-part wedge method using $\phi' = \phi_d$ without reduction. It means that the shear strength ϕ' of Shirasu was fully mobilized in order to support the reinforced slope.

(3) The tensile force observed in the reinforcement was only 23% of the characteristic tensile strength of the polymer grid used. The reinforcement with lower characteristic force can be used for the Shirasu embankment.

ACKNOWLEDGEMENT

The authors are grateful to all staff in the Kagoshima National Road Construction Office, Ministry of Construction, for their helps in conducting the present study.

REFERENCES

Doboku Research Centre (1993) *Design and construction manual of the reinforced Embankment by Geotextile (in Japanese).*

Geogrid Research Board (1990) *Guidelines for Geogrids (in Japanese).*

Haruyama M. (1969) Effect of water content on the shear characteristics of granular soils such as Shirasu, *Soils and Foundations*, Vol. 9, No. 3, 35 - 57.

Jewell R.A., Paine N. and Woods R.I. (1984) Design methods for steep reinforced embankments, *Symp. Polymer Grid Reinforcement in Civil Engineering*, London, No. 12, 1 - 7.

Yamanouchi, T., Fukuda N. and Ikegami M. (1986) Design and techniques of steep reinforced embankments without edge supportings. *Proc. 3rd Int. Conf. Geotextiles*, Vienna, Vol. 1, 199 - 204.