Applicability of Seismic Design Methods to Geogrid Reinforced Embankment

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ABSTRACT: This paper deals with the results from stability analyses of geogrid reinforced road embankment that remains without any damage during Kushiro Offshore Earthquake (1993; M7.8) in Japan. The earthquake's epicenter was located approximately 33 km from the embankment, and the horizontal acceleration at the embankment site is estimated at 310 gal. The embankment under study has the following features: height: 5.5 m, angle of face slope: 73.3°, reinforcement: geogrid of type SR–55, wall face material: expanded metal, height of upper unreinforced embankment section: 1.2 m and designed by Jewell et al.'s method at ordinary condition. The analyses are conducted according to the seismic design method of the Public Works Research Institute compared with the method of Geogrid Research Board. From these analyses, the seismic efficiency of geogrid reinforced structure is evaluated.

1 INTRODUCTION

In Japan, seismic design and construction manuals of reinforced earthworks have recently been published by the Geogrid Research Board (GRB,1990), the Public Works Research Institute (PWRI,1992), and the Railway Technical Research Institute (1992). However, the applicability of these design methods has not yet been verified, since the embankments designed by these methods have never been struck by actual earthquake.

On the other hand, after the Loma Prieta Earthquake (M7.1) struck San Francisco in 1989, Collin et al. (1992) surveyed and reported that geogrid reinforced structures had a satisfactory earthquake–resistant performance.

On January 15, 1993 Japan was struck by the Kushiro Offshore Earthquake (M7.8). This earthquake seriously affected a geogrid reinforced steep–slope road embankment, with a total height of 6.7 m (replaced depth: 1 m, height of upper unreinforced embankment section: 1.2 m). The earthquake's epicenter was located approximately 33 km from the embankment, and the horizontal acceleration is estimated at 310 gal based on a created–space attenuation characteristic curves for horizontal ground acceleration (Coordinating Committee for Promotion of the Strong Motion Earthquake Observation Project, 1993).

Though the embankment was designed under ordinary condition by using Jewell et al.'s method (1984), it was capable of maintaining stability even during the above mentioned earthquake. On the basis of this fact the seismic design methods described in PWRI design manual or proposed by GRB, this paper discusses and evaluates the applicability of these design methods and the earthquake–resistant performance of ordinarily designed geogrid reinforced embankment.

2 ORIGINAL DESIGN AND CONSTRUCTION

2.1 Ground condition and filling material

As shown in Fig. 1, the reinforced embankment is constructed on the foundation covered by topsoil of 1 m thickness. This foundation is underlain by the alluvium on rock, which consists of a 1 m–thick gravelly soil layer with a SPT *N*–value of approximately 5 to 10.

On the foundation formed by stripping off 1 m–thick topsoil, the embankment was constructed to a height of 5.5 m with the face slope 73.3° and the height of upper unreinforced embankment section of 1.2 m. The front area of the embankment was backfilled (1 m–thick) to the previous surface level after the construction of embankment.

The filling material is of volcanic cohesive soil(SV). The characteristics of this material are: natural moisture

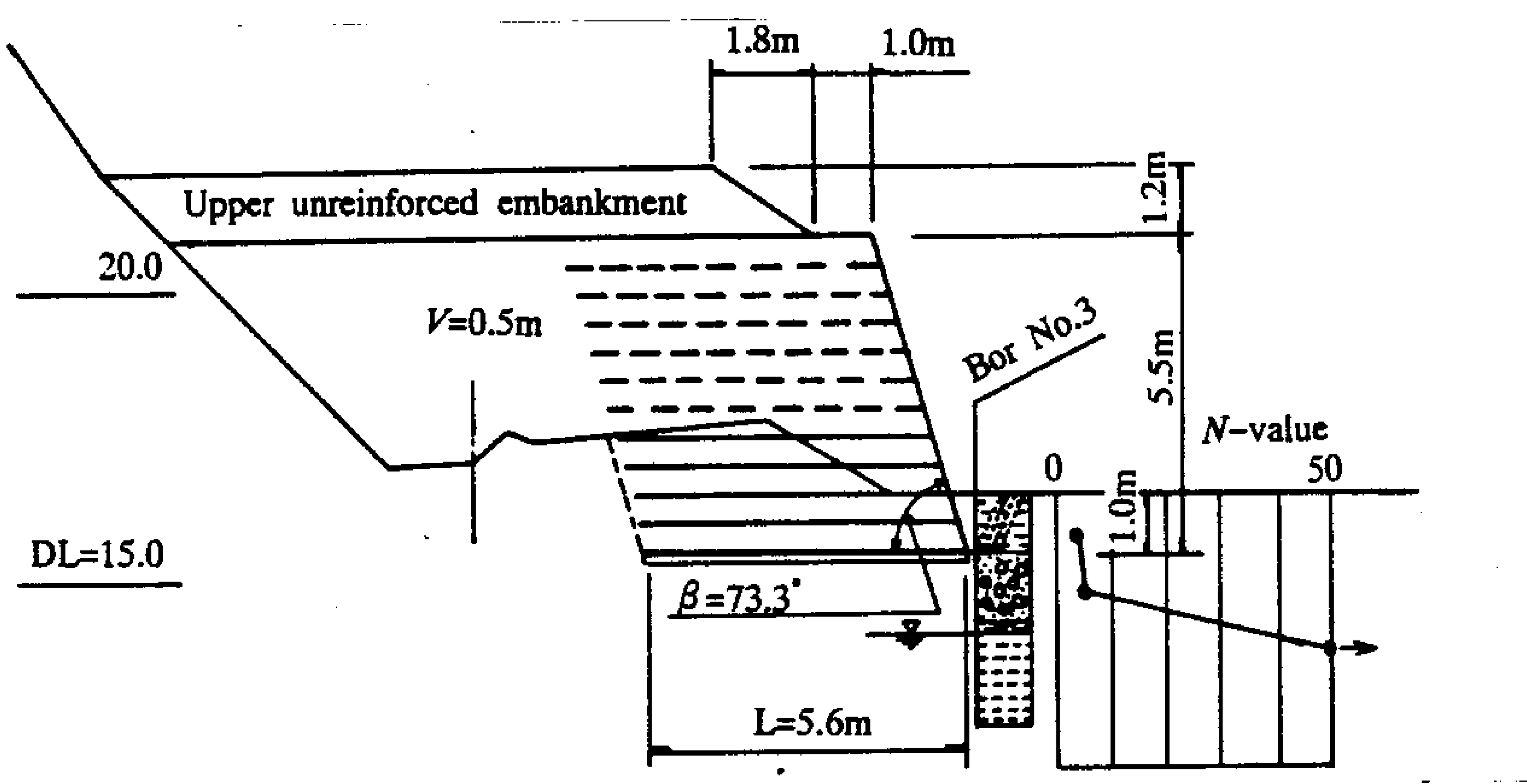

Fig. 1 Standard cross section of reinforced embankment.

content w_n = 40.6%, liquid limit w_L = 48%, plasticity index I_p = 15.3, and a fine fraction F = 49%. The designed shear strength parameters of the filling material are: internal friction angle ϕ' = 25°, cohesion c' = 0 kN/m^2, and unit weight γ = 17.6 kN/m^3 although the results obtained from the test are ϕ' = 27.0° and c' = 6.9 kN/m^2.

2.2 Original design and construction

In Japan the steep-slope reinforced embankments mostly have been designed in accordance with the Jewell et al.'s method (1984). This method is based on the principle of two-part-wedge, and the required length of reinforcement L_{req} is determined from the proposed diagram of slope gradient β and internal friction angle ϕ'. The reinforcement laying interval is determined from the force coefficient K which is obtained from the correlation diagram of β and ϕ' taken as parameters. This method, however, can only be applied to design for ordinary condition.

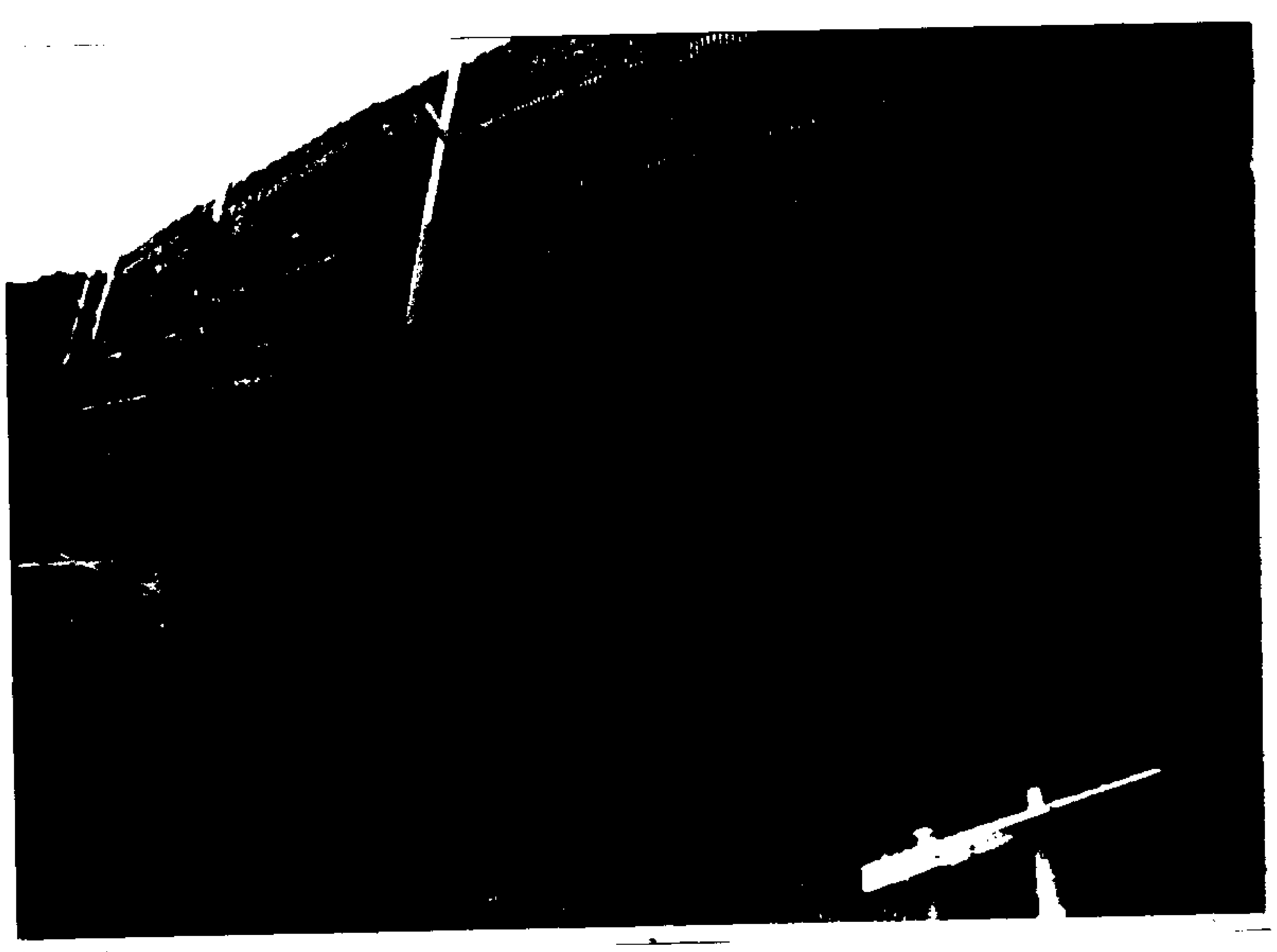

Fig. 2 Front view of the geogrid reinforced embankment under construction.

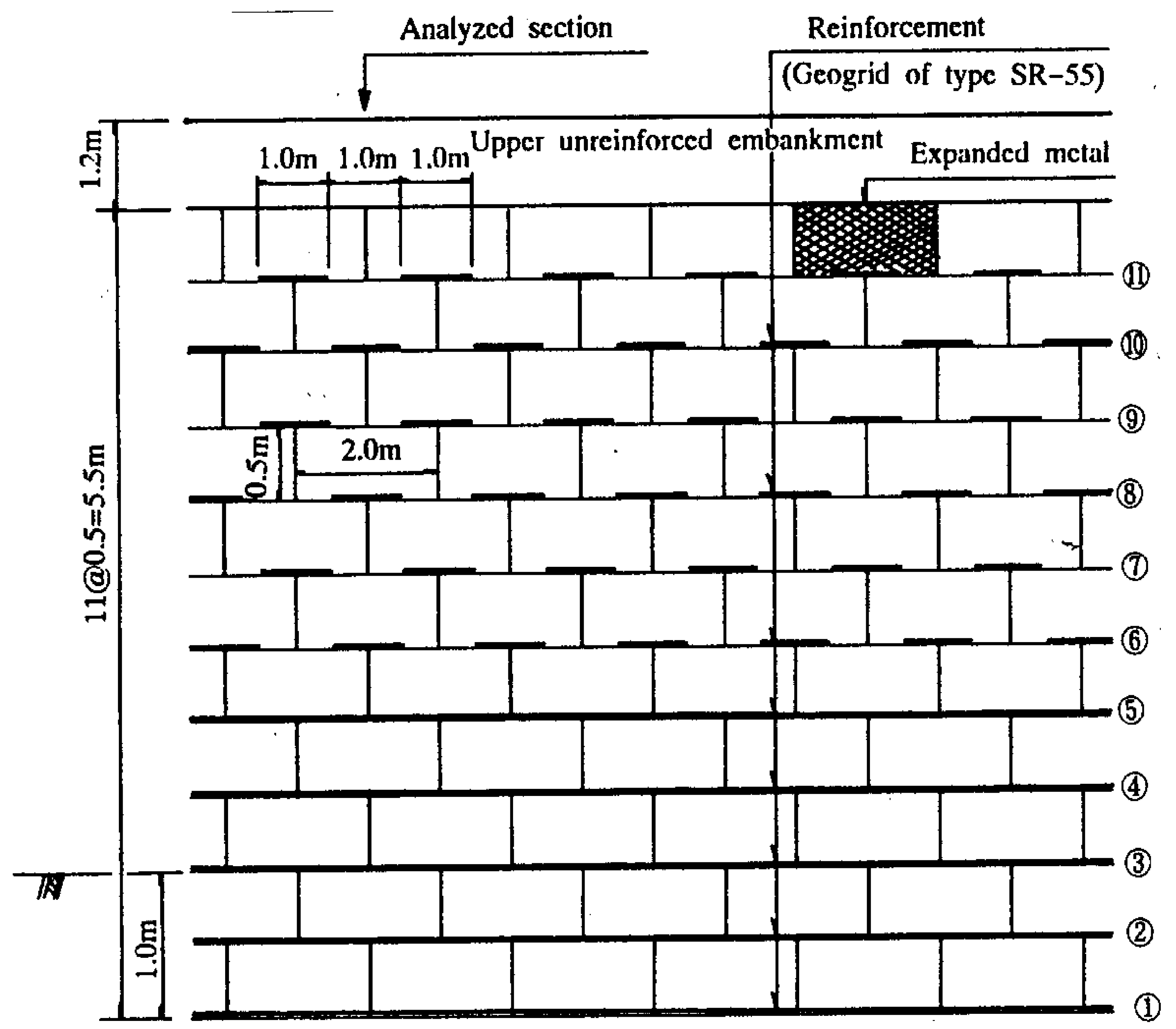

Fig. 3 Specification of geogrid laying (front view).

In the design calculation, the upper unreinforced embankment section is converted into uniform distributed loads, and geogrid used as the reinforcement material and having a tensile strength of T_f = 49.0 kN/m and an allowable tensile strength of T_A = 29.4 kN/m are laid with the same length. The required length is L_{req} = 5.6 m, and the vertical spacing of V = 0.5 m. However, in the sections represented by broken lines in Fig. 1, 1 m-wide geogrids are laid in a staggered pattern at 1 m horizontal intervals, since the strength of the reinforcements exceeds the required tension.

The slope face of the geogrid reinforced embankment was protected by applying expanded metal units of 0.5 m-high and 2.0 m-wide as shown in Figs. 2 and 3.

3 SEISMIC ANALYSIS BY PROPOSED DESIGN METHOD

3.1 Outline of analysis

In this paper, the authors evaluate the stability of steep-slope reinforced embankment to which a horizontal seismic intensity k_h = 0.31 is applied for comparison of two design methods, one proposed by PWRI and the other by GRB. Here the failure mode is different from the slip circle in PWRI method or two-part-wedge in the GRB method (Yamanouchi and Fukuda, 1993, 1994).

Design parameters for stability during earthquake are adopted by the PWRI method. During this process the applicability of the seismic design method is reviewed, referring to the GRB method. Although the height of the actually constructed reinforced embankment is 6.7 m, here it is assumed to be 5.7 m for this analysis, since the lower part has been already backfilled by 1 m.

3.2 Internal stability analysis under seismic condition

3.2.1 Analysis by using designed soil parameters

Table 1 shows the results of internal stability analysis which assumes the strength parameters of the embankment material as designed values, and the seismic designed strength of the reinforcement material is the same as that of PWRI method under ordinary condition and 1.5 times that of GRB method under ordinary condition.

The calculation proves that the internal stability predicted by PWRI method is lower than the designed safety factor. If it is true, it would eventually lead to the structures' instability, which is a different case from the reality.

3.2.2 Back analysis

Since it is difficult to explain the stability of the reinforced embankment by the analysis described in 3.2.1, the authors review the shearing strength of the embankment material and to estimate the minimum value of cohesion c' that would secure the safety factor by slip circle method FS equal to 1.0. By trial calculation, c' is obtained to be 4.9 kN/m^2. This value is almost the same as the experimental value. In the analysis ϕ' is taken at 27.0° as an experimental value.

The results of calculation are shown in Table 1. As to some reinforcements, the reinforcement tensions are generated higher than the predicted maximum tensile strength, which indicates that the embankment nearly reached the critical level during the earthquake. However, the stability could be maintained by stress redistribution in the upper staggered sections.

3.3 External stability analysis

As to the external stability, the safety factor of direct sliding FS_d = 0.93 which is lower than the allowable value of 1.2 as computed in equation (1). This value actually means that the embankment is unstable. Previously, it is assumed that both inertia k_hW_1 in the reinforced body and seismic earth pressure P_{HE} originating from the backfill were operating simultaneously as sliding forces during an earthquake. Refer to Fig. 4(a). However, here it is assumed that the horizontal seismic forces observed in the two zones

Table 1 Results of stability analysis by PWRI method.

Design condition			Designed parameters	Back analysis parameters
c' (kN/m^2)			0.0	4.9
ϕ' (degree)			25.0	27.0
k_h			0.31	0.31
T_f (kN/m)			49.0	49.0
T_A (kN/m)			29.4	29.4
T_{AE} (kN/m)			29.4 (44.1 = 1.5T_A)	44.1 (= 1.5T_A)
Internal stability	FS of slip circle		FS = 0.77 (0.80) < 1.0	FS = 1.02 > 1.0
		Layer No.*	Tension T_i (kN/m)	Tension T_i (kN/m)
	Tension at reinforcement	⑩	49.0 << 106.5	49.0 << 56.1
		⑧	49.0 << 81.3	29.4 < 40.4 < 44.1
		⑥	49.0 << 68.7	29.4 < 32.6 < 44.1
		⑤	24.2 < 29.4	11.3 < 29.4
		④	25.5 < 29.4	11.6 < 29.4
		③	13.4 < 29.4	6.0 < 29.4
External stability	FS_d of direct sliding		FS_d = 0.67 < 1.2	FS_d = 0.93 < 1.2
	Overturning (m)		e = 0.91 < $L/3$ = 1.87	e = 0.53 < $L/3$ = 1.87
	Bearing capacity (kN/m^2)		q_{max} = 167.6 < 294.0	q_{max} = 138.2 < 294

Notes : The figure in ( ) is acording to GRB method. * Refer to Fig. 3.

would not overlap each other, since the entire embankment is subjected to seismic vibration. If both the seismic inertia $k_h W_1$ of the reinforced zone and the backfill are assumed to maintain an ordinary earth pressure P_H, FS_d = 1.23 can be obtained as computed in equation (2). This concept is shown in Fig. 4(b). This allows the embankment to properly maintain its stability.

• Case 1: In the case of overlapping seismic horizontal forces (design assumption),

$$FS_d = \frac{\tan\phi'(W_1 + P_{VE})}{k_h W_1 + P_{HE}} = \frac{0.510 \times (526 - 32)}{0.31 \times 526 + 109} = 0.93 \qquad (1)$$

• Case 2: In the case of taking into account the seismic inertia of the reinforced zone and the ordinary earth pressure of the backfill,

$$FS_d = \frac{\tan\phi'(W_1 + P_V)}{k_h W_1 + P_H} = \frac{0.510 \times (526 - 15)}{0.31 \times 526 + 49} = 1.23 \qquad (2)$$

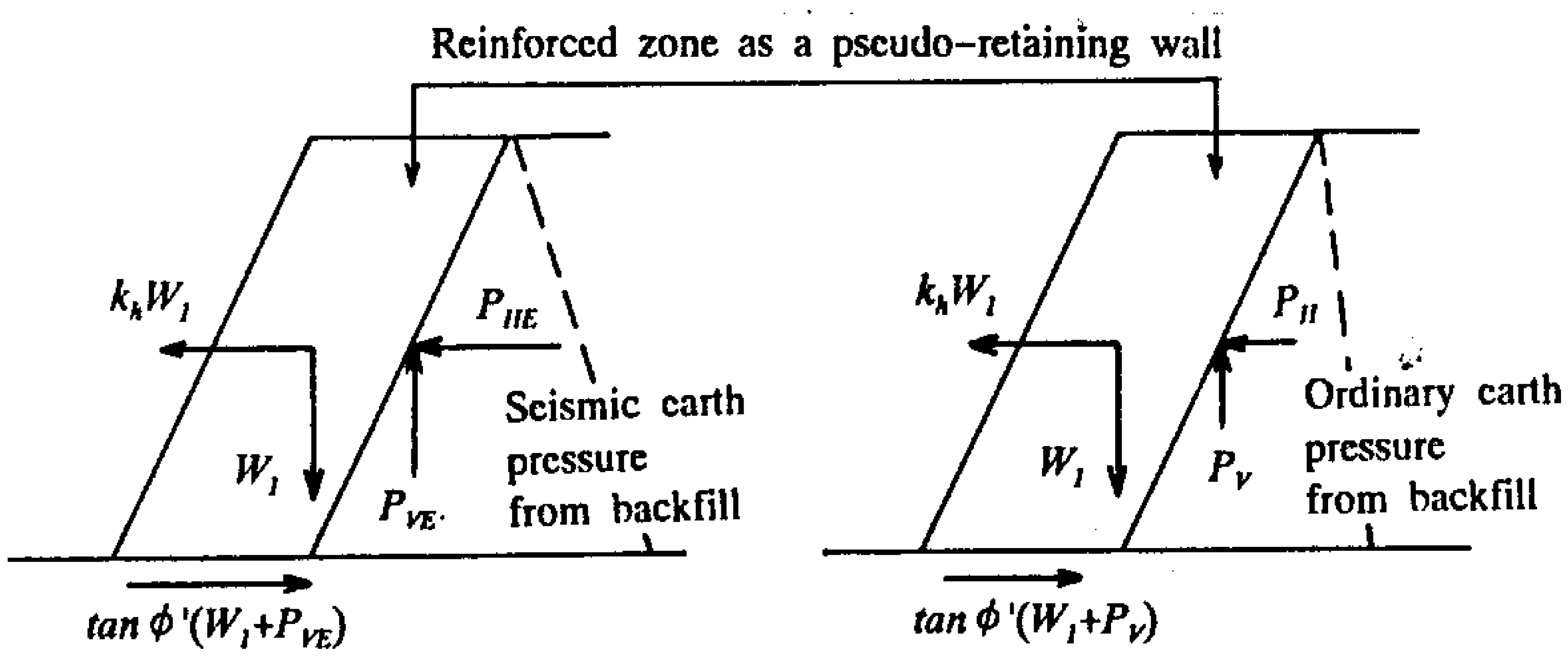

Fig. 4 Concept of earth pressure from the backfill for external stability under seismic condition.

4 CONCLUSION

The paper presents the results of the seismic analysis through currently proposed design methods for the geogrid reinforced steep–slope road embankment that maintains its stability even during a severe earthquake (M7.8). The analyzed results are concluded as follows.
(1) The steep–slope reinforced road embankment remained stable during high–intensity earthquake although it was not designed taking into account the seismic condition. Therefore it can be concluded that the design by Jewell et al.'s method (1984) is also efficient for the seismic condition.
(2) From the analyzed results the design strength of reinforcement during earthquake is required to be increased to T_{AE} = $1.5T_A$ as proposed by the GRB method and that the filling material should have at least c' = 4.9 kN/m^2 to ensure the reinforced embankment's stability.
(3) The analysis of the external stability yields a safety factor of direct sliding FS_d = 0.93. But the steep–slope reinforced road embankment remains stable during earthquake. This is to be concerned due to the fact that the seismic inertia of the two zones, the reinforced zone and the backfill zone, are acting as an integrated unit giving FS_d = 1.23 and acting as if they are under ordinary earth pressure. However further detailed studies are required.

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