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FEM comparative analysis of facing rigidity of geotextile-reinforced soil walls

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ABSTRACT: The deformation behavior of geotextile-reinforced soil walls with two types of comparatively rigid facings (made of concrete panels and concrete blocks) and that of a geotextile-reinforced wall with a wrap-type facing, the latter having many records of construction to date, were numerically analyzed through a two-dimensional elasto-plastic FEM, and the effect of the facings was evaluated through comparative study from various aspects. As the result, some findings were obtained about the constraining conditions for comparatively rigid facings to exhibit the effect of restraining the deformation of reinforced walls and about constructional measures to satisfy such constraining conditions.

1 INTRODUCTION

In case of geotextile-reinforced soil walls which consist of a facing and an embankment arranged in parallel, ascertained from construction cases to date is that comparatively rigid facings have the deformation-restraining effect such as the restraint of wall-surface displacement and the reduction of tensile force required of geotextile (Tatsuoka, 1993; Nakajima et al., 1996; Tajiri et al., 1996). To evaluate this effect, it would be necessary to grasp the deformation behavior mainly from the viewpoint of the interaction among the facing, the geotextile, and the soil. Accordingly, a reinforced soil-wall model 10 meters high with a wrap-type facing and its two versions with concrete-panel and -block facings were assumed. The construction process taken into consideration, numerical analyses using a two-dimensional elasto-plastic FEM were performed to compare and study the deformation behavior of the models and the deformation-restraining effect of their facings.

2 RELIABILITY OF FEM ANALYSIS AND ITS VERIFICATION

To verify the reliability of the results of the analyses, a simulative analysis was performed on a full-scale failure experiment of a reinforced soil wall 6 meters high with a concrete-block facing constructed in a test laboratory (Tajiri et al., 1996).

2.1 *Outlines of full-scale experiment and FEM analysis*

Fig. 1 shows the section of the wall subjected to the failure experiment (Tajiri et al., 1996). This reinforced wall was designed, given a safety factor of 1.2 under the current design manual (PWRI, 1992). The verification of the FEM analysis' results was performed in comparison with the data of the wall-surface deformation, the normal earth pressure on the back of the facing, the strain of the geotextile, and the subgrade reaction, all measured during the construction process.

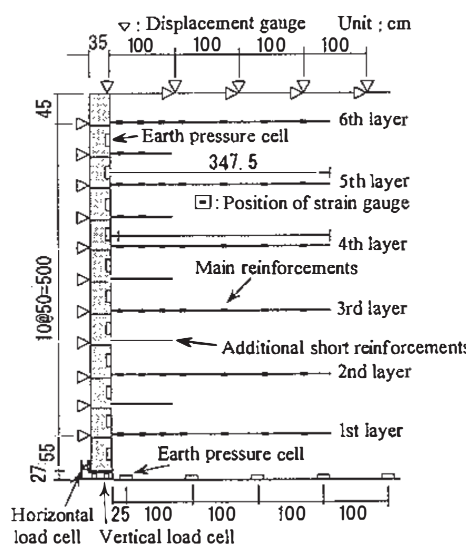

Fig. 1 Sectional view of experimental wall

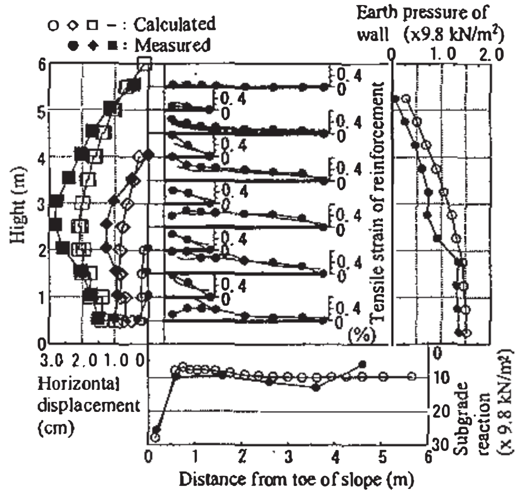

Fig. 2 Comparison of results of experiment and analysis

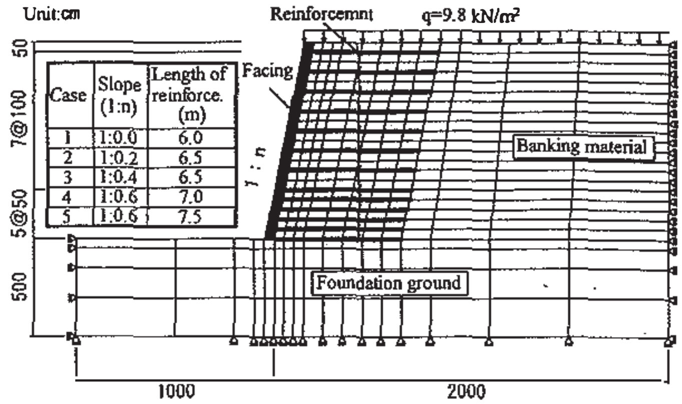

Fig. 3 Analytical model

The FEM analysis was based on an elasto-plastic analysis with a gradually increasing load, of which the details are described in Section 3.

2.2 *Evaluation of analytical precision*

Fig. 2 shows the results of comparison between the measured values and calculated ones for the wall-surface horizontal displacement, the geotextile strain, the normal earth pressure on the back of the facing, and the subgrade reaction. It can be observed from this figure that they are close to each other and, hence, the calculated values well represent the experimental results qualitatively and quantitatively. In particular as for problems where the design safety factors are high enough and the advent of plastic areas in the elasto-plastic analyses has small effects upon the results, the analytical precision can be assured, though the separate problem of how to set various material properties has to be addressed properly.

3 ANALYTICAL MODELS AND SETTING OF MATERIAL PROPERTIES

To cover the applicable range of the current design manual (1:0.0 -1.0), five wall models 10 meters high with different slopes, i.e., 1:0.0, 0.2, 0.4, 0.6, and 0.8, were assumed. The arrangement of reinforcements in each wall was designed in accordance with the current design manual, and the general stability analyses using the slip circle method showed that each wall met the safety-factor condition of Fs = 1.2 or more.

As shown in Fig. 3, the analytical models were composed of foundation ground, banking material, reinforcements, and a wall facing. The foundation ground was treated as an elastic body to assume enough bearing capacity. The Mohr-Coulomb's yield criterion was applied to the banking material to assume a bilinear, elasto-perfectly-plastic body. Because tensile forces to be mobilized in the reinforcements were not to exceed the design strength, they were represented as bar models.

As shown in Fig. 4, a facing was composed of concrete blocks, 100 cm (W) x 50 cm (H) x 50 cm (D) and another of concrete panels, 100 cm (W) x

Table 1 Material properties

Soil

	Banking material	Foundation ground
Young's modulus E (x 9.8 kN/m²)	1,000	5,000
Poisson's ration ν	0.3	0.3
Cohesion c (x 9.8 kN/m²)	0.1	—
Internal friction angle ϕ (°)	30.0	—
Unit weight γ (x 9.8 kN/m³)	1.9	2.2

Reinforcement

	Geotextile
Young's modulus E (x 9.8 kN/m²)	2.7 x 10^5
Sectional area A (m²)	3.71 x 10^{-4}
Extensional rigidity EA (x 9.8 kN)	100

Facing

	Panel type	Block type	Wrap type
Young's modulus E (x 9.8 kN/m²)	2.5 x 10^5	2.5 x 10^5	Equivalent to geotextile
Poisson's ration ν	—	0.3	
Sectional area A (m²)	0.18	—	
Moment of interia of area I (m⁴)	4.86 x 10^{-4}	—	
Unit weight γ (x 9.8 kN/m³)	2.3	2.3	

Joint element

No.	Joint element	Shearing rigidity Ks1(x 9.8 kN/m²)	Vertical rigidity Kn (x 9.8 kN/m²)
1	Soil - reinforce.	1,000	100,000
2	Soil - facing	100	100,000
3	Between blocks	10,000	100,000

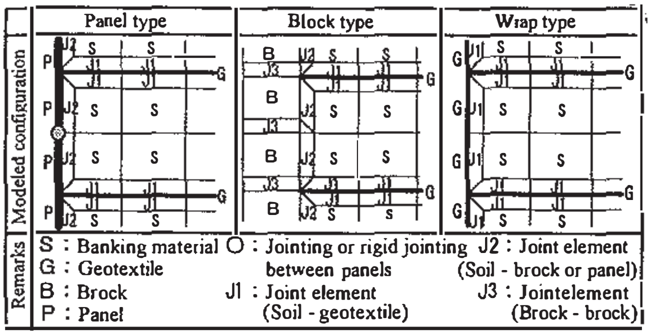

Fig. 4 Models of facings

100 cm (H) x 18 cm (T). Joint elements of linear models were arranged on the upper and lower surfaces of each reinforcement, on the back of each facing, and between blocks. Thus the difference in the frictional characteristics was represented by changing the shearing rigidity (Ks). The values of shearing rigidity were set with reference to the results of past pull-out tests in case of the rigidity on the upper and lower surfaces of the reinforcements (Ks1), with reference to the results of past single-face shearing tests in case of that on the backs of the facings (Ks2), and with consideration of the current block-constraining methods in case of that between blocks (Ks3), the last one being set higher than the first two. Various material properties used in the analyses are shown in Table 1. Analyses were performed in three series as shown in Table 2, under the conditions described in the same table.

4 COMPARATIVE STUDY OF RESULTS OF ANALYSES

4.1 *Series I and II*

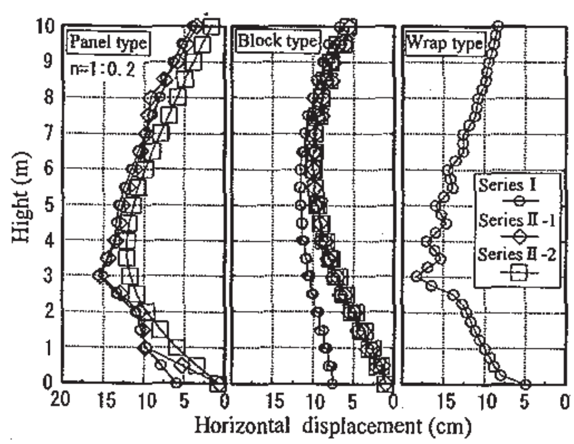

Fig. 5 Distribution of wall-surface horizontal displacement

Table2 Series of analyses

Series No.		Serial No.	Type of facing	Shearing rigidity (x 9.8 kN/m²) Ks2	Ks3	Constraining condition
I	1	1	Block	100	10,000	Nil
		2	Panel		—	
		3	Wrap	—	—	
II	1	4	Block	100	10,000	Facing's bottom
		5	Panel		—	
	2	6	Block		—	Facing's bottom and between facing's units
		7	Panel		—	
III	1	8	Block	10	10,000	Facing's bottom
		9		100		
		10		1,000		
		11		10	100	
		12		100		
		13		1,000		
	2	14	Panel	10	—	Facing's bottom
		15		100		
		16		1,000		

(1) *Deformation behavior*

Fig. 5 shows the distribution of the wall-surface horizontal displacement at n = 1:02 as the representative case. It can be observed from these figures that the concrete-panel facing has little effect of restraining the wall's deformation unless the facing's bottom and the foundation ground are consolidated into one body and the panels are jointed each other. On the other hand, the concrete-block facing slides on the foundation ground and the displacement of the whole wall surface is increased if the condition of constraining the bottom of the facing is not met.

Fig. 6 shows the maximum horizontal displacements of the panel and block facings at every gradient after being normalized by that of the wrap-type facing (U_P max/U_W max and U_B max/U_W max). Although the panel facing, under the Series II-2's condition, exhibited a constant displacement-restraining effect of the order of 0.66 times in the range of n = 1:0.0 - 0.6 (≒ active failure angle of approx. 60°), its displacement at n = 1:0.8 largely exceeded that of the wrap-type facing, indicating that the panel-type wall was turning rapidly into unstable condition. Besides, the panel facing showed no significant displacement-restraining effect under the other constraining conditions.

The block facing, under the condition of con-

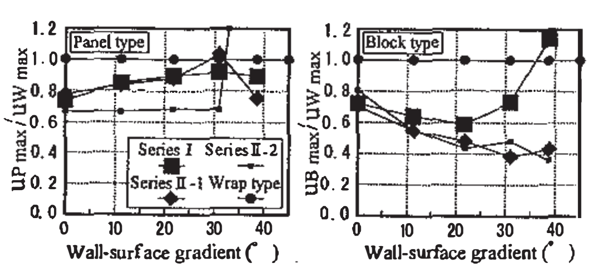

Fig. 6 Relation between wall-surface gradient and displacement

straining the facing's bottom only, presented a linear displacement-restraining effect of 0.7 - 0.4 times in the range of n = 1:0.0 - 0.6 and a constant effect even when its slope became less steep. Under the Series I's condition, however, its effect was unstable and its displacement at n = 1:0.8 exceeded that of the wrap-type facing. These results suggest the importance of measures for constraining the lowest blocks of block facings.

(2) *Tensile force of reinforcements*

Fig. 7 shows the distribution of tensile forces mobilized in the reinforcements of each wall at n = 1:0.2. The tensile force of every reinforcement represents the value when it takes charge of one-meter depth of the embankment at its position. Under the Series I's condition, both the panel- and block-type walls showed significant increase of the tensile force in lower layers, suggesting the importance of consolidating the facing's bottom and the foundation ground into one body. Under the conditions which allow the facings to function effectively, the wrap-type wall showed a triangular distribution of tensile force, the panel-type wall an approximately triangular distribution though the tensile force decreased in the lower layers, and the block-type wall a trapezoidal distribution with fairly constant values through the layers. It is important to the design of effective arrangement of reinforcements to estimate the distribution of the tensile force correctly.

Fig. 8 shows the total tensile force of the reinforcements of each wall at every gradient after being normalized by that of the wrap-type wall ($\Sigma Ti_P/\Sigma Ti_W$ and $\Sigma Ti_B/\Sigma Ti_W$). In case of the panel facing, although a tensile-force reducing effect of 0.7 times or so was observed in the range of n = 1:0.0 - 0.6 under the Series II-2's condition, under the other constraining conditions no significant effect could be expected from the panel facing.

In case of the block facing, a linear effect of 0.9 -

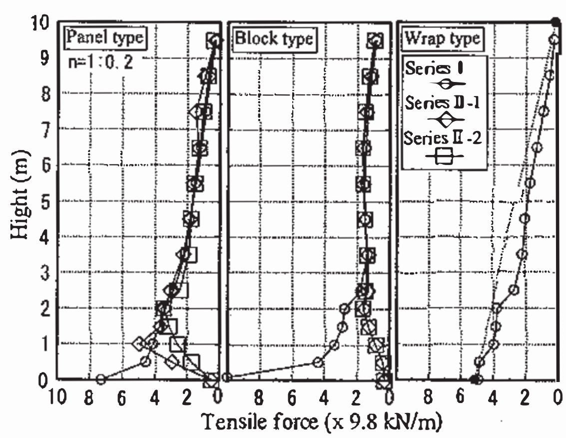

Fig. 7 Distribution of tensile force

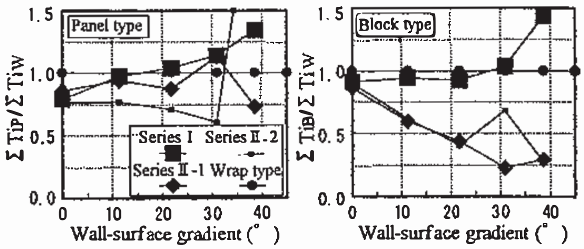

Fig. 8 Relation between wall-surface gradient and required tensile force

0.2 times was observed in the range of n = 1:0.0 - 0.6 under the Series II's condition. Even at n = 1:0.8, the effect was maintained.

Fig. 9 shows the distribution of the tensile force in every reinforcement of each wall at n = 1:0.2 and 1:0.8. At n = 1:0.2, the wrap-type wall showed convex distribution lines and the maximum tensile force of each reinforcement appeared in the vicinity of the Coulomb's active failure line, whereas the block- and panel-type walls exhibited distribution lines where the maximum tensile forces were located in the direct vicinities of the facings, except in some lower layers.

At n = 1:0.8, the wrap-type wall showed convex distribution lines of tensile forces of the reinforcements and the maximum forces were located in the vicinity of the slip-circle failure line. Tensile force was mobilized over the whole length of each reinforcement, and thus the whole body of the embankment was effectively reinforced. In the panel-type wall, however, large tensile forces were observed locally and compressive forces in the upper layers. In the block-type wall, compressive force was observed on every reinforcement in the vicinity of the facing. Thus in these cases the reinforcements do not function well as tensional ones. It can be reasoned that the continuity of the units of a panel or block facing less steep than 1:0.6 is disturbed and the dead weight of the facing has the effect of holding down the face of the slope, thus creating compressive force on the reinforcements.

(3) *Subgrade reaction*

Fig. 10 shows the distribution of the subgrade reaction of each wall at n = 1:0.2. The panel-type wall always exhibited large subgrade reaction directly under the facing only, regardless of the different constraining conditions. In case of the block-type wall, the effects of the different constraining conditions upon the subgrade reaction were observed saliently, due to the dead weight of the blocks, the earth frictional force working downward on the back of the facing, and the turn of the lowest blocks. In either case of the panel- and block-type walls,

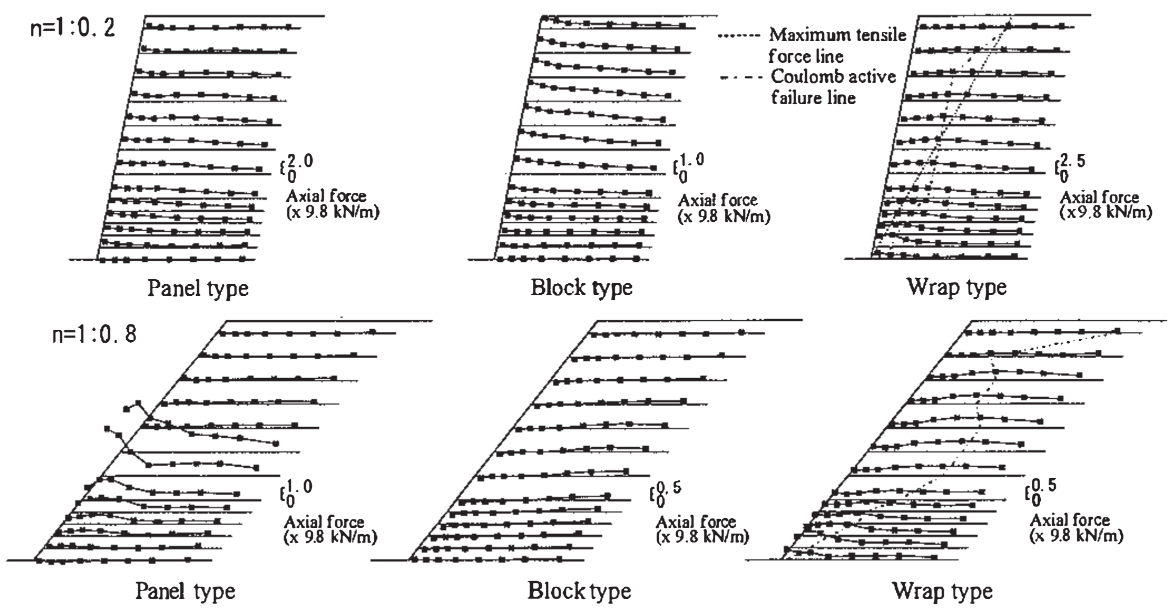

Fig. 9 Distribution of tensile force of reinforcements

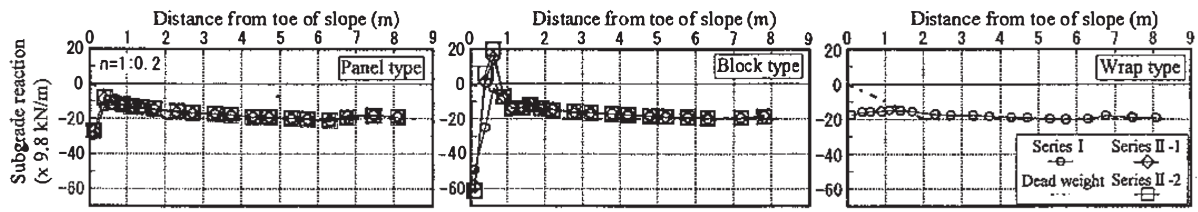

Fig. 10 Distribution of subgrade reaction

however, the subgrade reaction increased in the vicinity of the facing and corresponded to the overburden pressure in the other area. Therefore, required are measures to reduce local, large subgrade reaction under the bottoms of the facings; for example, by providing a rigid footing foundation with suitable width.

4.2 *Series III*

(1) *Effects of frictional characteristics on back of panel facing (Ks2)*

Fig. 11 shows the effects, at n = 1:0.2, of the shearing rigidity on the back of the panel facing upon the wall-surface horizontal displacement, the tensile force of the reinforcements, and the subgrade reaction. These figures shows that the variation of the shearing rigidity has little effects upon them and no effect upon the deformation behavior of the reinforced wall. A similar tendency appeared at every gradient.

(2) *Effects of frictional characteristics on back of block facing (Ks2) and between blocks (Ks3)*

Fig. 12 shows the effects, at n = 1:0.2, of the shearing rigidity on the back of the block facing and the shearing rigidity between blocks upon the wall-surface horizontal displacement and the tensile force of reinforcements. Under the condition of Ks3 = 10,000 x 9.8 kN/m^2, as the shearing rigidity on the back of the block facing increased, the displacement increased slightly in the upper area of the wall surface and the total tensile force of the reinforcements also increased. At any other gradient, however, no significant effects were observed. Under the condition of Ks3 = 100 x 9.8 kN/m^2, no significant effects were observed at any gradient. These results suggest that the frictional characteristics on the back of a block facing have no effects upon the deformation behavior of the reinforced wall. On the other hand, the frictional resistance between blocks has large effects upon the deformation behavior. Under Ks3 = 100 x 9.8 kN/m^2, the wall-surface displacement took, at every gradient, a bulging distribution pattern similar to that of the wrap-type wall. At the gradients less steep than n = 1:0.4, the wall-surface horizontal displacement and the total tensile force of the reinforcements exceeded those of the wrap-type wall.

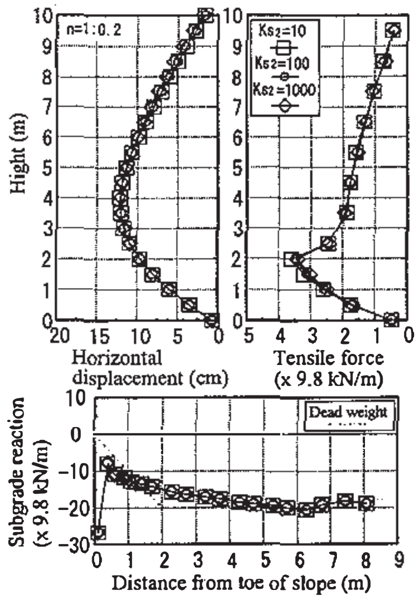

Fig. 11 Effects of shearing rigidity (Panel facing)

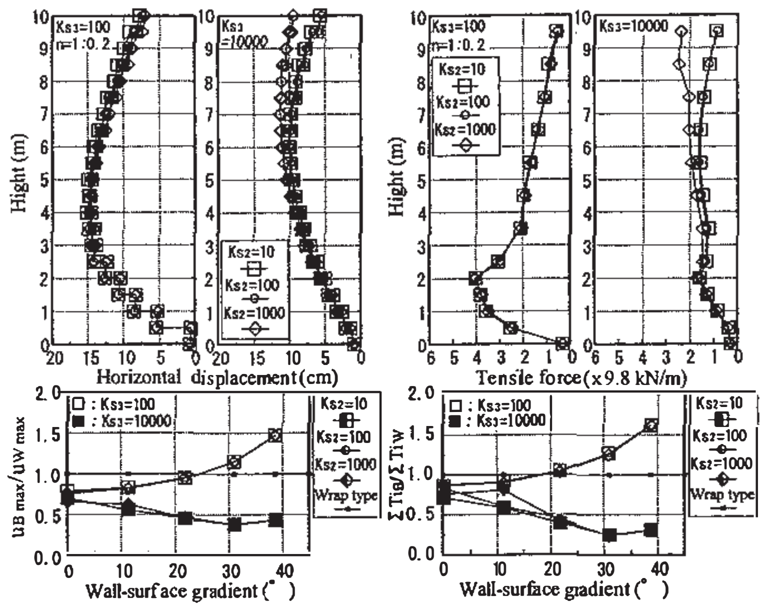

Fig. 12 Effects of shearing rigidity (Block facing)

5 CONCLUSIONS

A series of analyses using a two-dimensional elasto-plastic FEM was performed on geotextile-reinforced walls with different types of facings, and the following findings were obtained:

(1) In case of a comparatively rigid facing made of blocks or panels, it is necessary to provide such a foundation structure as make the lowest blocks or panels and their foundation ground behave as one body.

(2) Necessary for maintaining the rigidity and continuity over the whole wall surface are the resistance between blocks in case of block-type walls and the jointing between panels in case of panel-type walls.

(3) The tensile force of each reinforcement in a wall with a comparatively rigid facing takes a distribution pattern where the maximum force is located in the direct vicinity of the facing, but such forces are not so large as to present problems in particular.

(4) When a comparatively rigid facing is adopted, large subgrade reaction arises locally under the bottom of the facing. This requires some measures to reduce the local, large subgrade reaction such as a rigid footing foundation.

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