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Full-scale failure experiments of geotextile-reinforced soil walls with different facings

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ABSTRACT: The design manuals of geotextile-reinforced soil walls proposed to date vary in the evaluation method of retaining effect of facings. Facings have five kinds of retaining effect; i.e., local rigidity, overall axial rigidity, overall shear rigidity, overall bending rigidity, and gravity resistance. If a design method which can derive these kinds of wall effects is made available, more rational and economical design will become possible. To grasp the influence of wall effect for reinforced soil walls, three geotextile-reinforced walls (6-meter high, vertical walls) with different facings (EPS blocks, concrete panels and concrete blocks) were constructed in a laboratory and a series of experiments was conducted until their failure due to cutting of their reinforcements. This paper deals with new findings from the results of the experiments through comparison.

1 INTRODUCTION

Geotextile-reinforced embankments and soil walls have more and more been constructed with the availability of design manuals (Geogrid Research Board, 1990; Public Works Research Institute (PWRI), 1992; Railway Technical Research Institute, 1992). Besides, various wall-facing materials have been developed and come into use, which will further the construction of such soil structures in future. The design manuals of geotextile-reinforced soil walls proposed to date vary in the evaluation method of retaining effect of facings. Facings have five kinds of wall effect; i.e., local rigidity, overall axial rigidity, overall shear rigidity, overall bending rigidity, and gravity resistance (Tatsuoka, 1992).

To grasp the influence of wall effect for reinforced soil walls, full-scale three geotextile reinforced walls of 6-meter high and vertical with different facings (expanded polystyrol (EPS) blocks, concrete panels and concrete blocks) were constructed in a laboratory and a series of experiments was conducted until their failure due to cutting of their reinforcements. This paper deals with the findings from the results of the experiments through comparison.

2 OUTLINE OF EXPERIMENTS

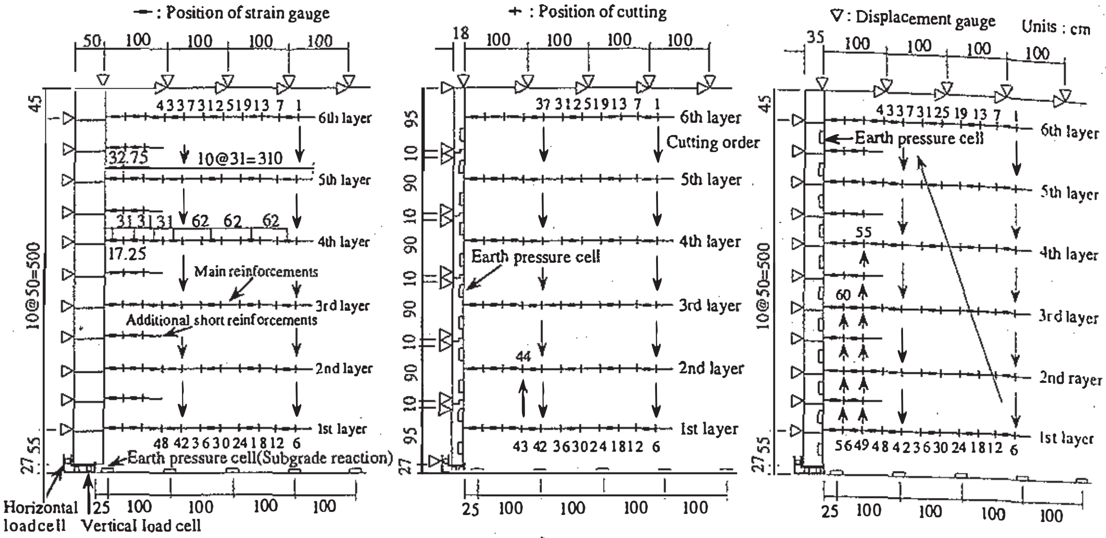
2.1 *Design section*

The laying specifications of main reinforcements in the walls were determined according to the current design method (Onodera et al., 1992; PWRI). Based on a stability analysis where the safety factor against sliding failure, Fs, for internal stability under ordinary condition was assumed to 1.0. The safety factor against sliding failure for general stability was 1.32. The laying specifications of the main reinforcements (Geogrid of type SR-55, Peak strength T_f = 56.4 kN/m, Design strength T_A = 29.4 kN/m) were: length L = 3.5 m × 6 layers (at 1.0 m intervals). The soil parameters of the filling material were: γ_t = 16.0 kN/m^3, c' = 0.0, and ϕ' = 39.4°.

To these reinforced soil walls, facing materials of EPS blocks (width 1.0 m × height 0.5 m × thickness 0.5 m), concrete panels (1.5 m × 1.0 m × 0.18 m), and concrete blocks (1.0 m × 0.5 m × 0.35 m) were applied. Besides, for the EPS and concrete blocks, additional short reinforcements (L = 1.0 m) were laid between the main reinforcements, the safety during construction taken into account. The short reinforcements were used of the same material with the main reinforcements.

2.2 *Instrumentations and observation*

In the series of experiments, conducted was the measurement of 9 items; i.e., horizontal displacement of the wall, vertical displacement at wall-top, earth pressure against the wall, strain on reinforcements, subgrade reaction, wall-bottom horizontal and vertical reaction and displacements at embankment-surface. Fig. 1 shows the instrumentations. The measurement was performed by using automatic measuring system of personal computer at every step of the filling and until the failure of the walls due to the cuttings of their reinforcements. After the failure, the embankment was excavated and determined the slipping surfaces. The numbers and arrow marks in Fig. 1 indicate the cutting process. The reinforcements were melted and cut by passing electricity through the nichrome wires which are prewound at cutting points. After cutting took



(a) EPS block facing (b) Concrete panel facing (c) Concrete block facing

Fig. 1 Insturumentations of geogrid-reinforced soil walls.

place, the strains on the reinforcements were monitored. When the strains settled constant, cutting progressed to the next point and monitored in same way.

2.3 *Construction procedure of walls*

The walls were constructed in a concrete pit (5.0 m wide and 4.0 m high) in a laboratory of the PWRI. Because the pit was 2.0 m short of the walls in height, H-beams were erected on the both sides and acrylic boards 15 mm thick were attached on the H-beams. Besides, two vinyl sheets between which grease was applied were attached on the acrylic boards to remove the friction on the side walls. As shown in Fig. 1, the bottoms of the facings were fixed against horizontal displacement with a supporting member for load cell at bottom of the wall which was fixed to the pit's bottom.

In case of the concrete-panel facing, a H-beam grid frame supported by steel-timbering and hydraulic jacks were set up in front of the concrete panels to prevent the deformation during construction. After the completion of the filling, the hydraulic jacks were eased back to have the wall with the H-beam grid frame stand independently as a full-height-panel-wall. After that, the H-beam grid frame was removed to as a discrete-panel-wall.

3 COMPARISON OF EXPERIMENTAL RESULTS

In this paper, mainly dealt with through comparison are the values measured when the embankments became stable after being left for 4 - 5 days after the completion of construction and the values measured immediately before the failure of the walls due to the cutting of the reinforcements. The failure of EPS-block-wall took place creepingly. On the other hand, the failure of other two walls took place instantaneously (refer to Photo 1). No.48, No. 44 and No.60 in Fig. 1 (a) - (c) indicate the last cutting point in the wall with the EPS-block facing, the concrete-panel and the concrete-block, respectively.

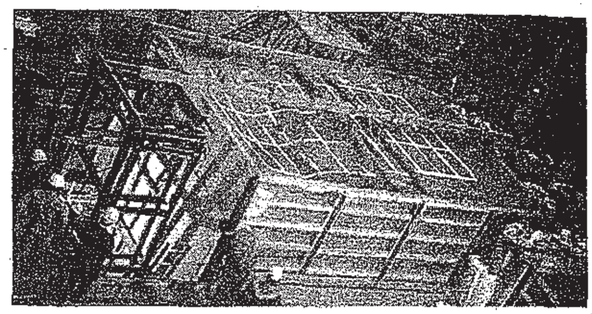
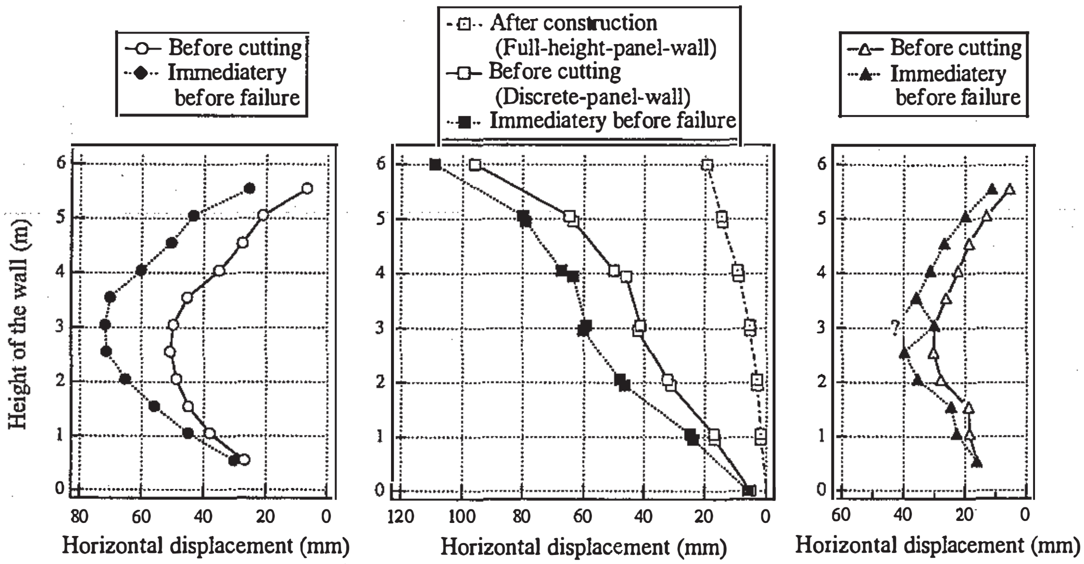


Photo 1 Situation after failure (Concrete-panel wall).

3.1 *Horizontal displacement of the wall*

Fig. 2 shows in contrast the horizontal displacements of the walls after the completion of construction and those immediately before the failure of the walls. In case of the block-type facings (Fig. 2 (a) & Fig. 2(c)), either wall showed large displacements in its middle-height portion, unlike walls with low-rigidity facings of sandbags, etc. which show their largest displacements in their 1/3-height portion. Besides, the displacements of the wall with the EPS-block facing were larger than those of the concrete-block facing, which demonstrates that facings of some overall axial rigidity but of little gravity resistance have small effect of restraining the displacement of walls.

The wall with the concrete-panel facing showed the deformation of a forward-tilting tendency when the jacks were eased back. It is due to the restraining of



(a) EPS block facing (b) Concrete panel facing (c) Concrete block facing

Fig. 2 Horizontal displacement of the wall-surface.

deformation by the timbering during construction. When the H-beam grid frame was removed, leaving the wall as a discrete-type and reducing its overall bending rigidity, the displacement increased rapidly. Besides, alignment at the joints between panels was disturbed. These results suggest the importance of joints between panels. In actual construction, since displacement is corrected step by step during construction, displacement as large as that observed in this experiment would not take place.

3.2 *Earth pressure against the wall*

Fig. 3 shows the observed earth pressures against the wall of the discrete-concrete panels and the concrete blocks. Earth pressures against the wall with EPS-block facing were not measured. In both the cases, displacement occurred before the cutting process and, hence, both the walls were in active state.

The wall with the concrete-block facing before the cutting of the reinforcements, the earth-pressure distribution almost overlapped with the Coulomb's active earth-pressure line. On the other hand, the earth pressures against the wall with the concrete-panel facing were considerably lower than the active earth pressures (the triangular distribution) and in the bottom portion the earth pressure indicated toward the active earth pressure. The earth pressures against the wall immediately before the failure of the walls due to the cutting of the reinforcements were higher than those before the cutting of the reinforcements except the lower portion, indicating the redistribution of the stresses. These results indicate the characteristics of earth pressure against the wall in relation to different facing methods. The larger the deformation-restraining effect of the facing is, the larger effect of resisting the earth pressure. In case of wall with discrete-panel facings which allow the deformation, embankment-reinforcing effect reduces the earth pressure remarkably.

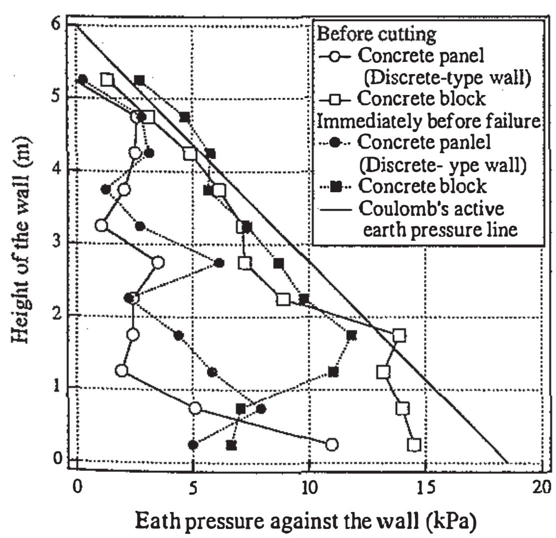


Fig. 3 Distribution of earth pressure against the wall.

3.3 *Subgrade reaction*

Fig. 4 shows the distribution of subgrade reactions at the bottoms of the embankments and at the bottoms of the facings. In the external stability analysis of the current design method, the reinforced zone is assumed to be a rigid body and the distribution pattern of the subgrade reactions to be trapezoidal. According to the measured data, however, the distribution of subgrade reactions was roughly equivalent to that of the overburden pressures by the embankments, which differs from the assumptions. On the other hand, at the bottoms of the facings considerably larger loads than

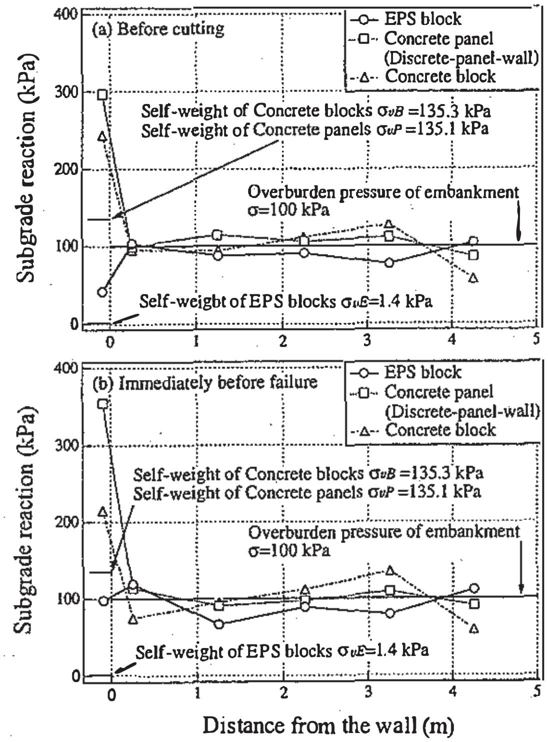


Fig. 4 Distribution of subgrade reaction.

the self-weights of the facings were observed during the period from the completion of the construction to just before the wall failure. This tendency was ascertained through the numerical analysis (Nakane et al., 1996) and the outdoor experimental embankment (Nakajima et al., 1996). Such subgrade reactions at the bottoms of facings are considered a characteristic of reinforced soil walls with high-rigidity facings, and it is necessary to develop a method of estimating such subgrade reactions quantitatively in relation to the compressibility of fill materials, the bearing capacity characteristics of foundation, etc.

3.4 *Strains and tensions of reinforcements*

Figs. 5 and 6 show the strains on the reinforcements and the maximum tensions measured before the cutting, respectively. Here, 1% strain on the reinforcements is equivalent to the tension of 8.6 kN/m.

The tendencies of the strain distribution on the main reinforcements varied depending upon the types of facings. In case of the EPS-block wall, the point of the maximum strain show the tendency of moving away from the facing, deeper into the embankment compared with those of the other walls. The strain distribution pattern changed from the triangular distribution (in the

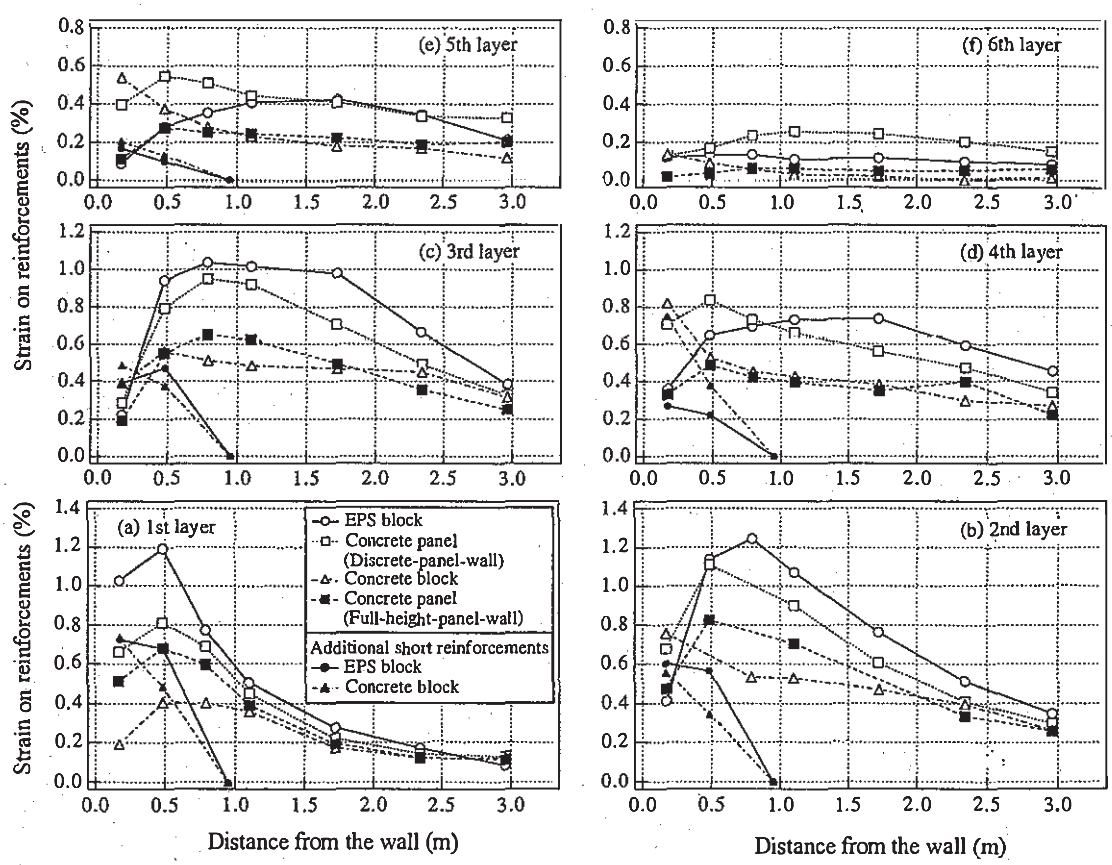


Fig. 5 Distribution of strains on reinforcements.

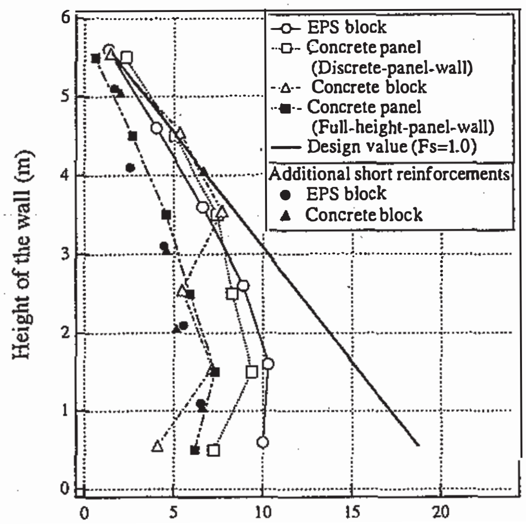


Fig. 6 Distribution of maximum tension.

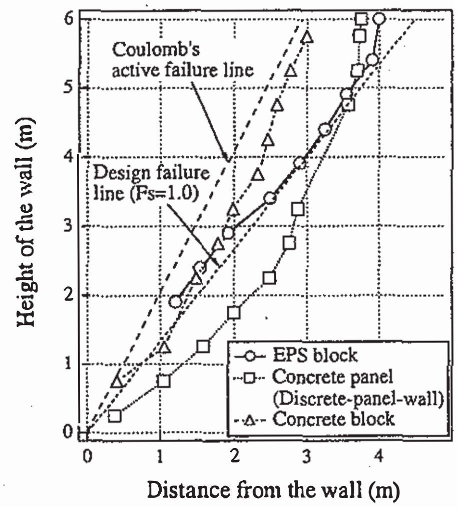


Fig. 7 Shapes of failure lines.

1st and 2nd layers), where the maximum strains were observed near the facing, to the trapezoidal distribution (in the 3rd to 5th layers). In case of the wall with the concrete-block facing of which the wall-rigidity is high, its strains were wholly smaller than those of the other walls. In the vicinity of the facing, the strains in all the layers except the first layer were more or less larger than those of the other walls. In the 2nd, 4th, and 5th layers, the wall showed its maximum strains in the vicinity of the facing. In case of the wall with the concrete-panel facing, the shapes of the strain distribution of the full-height-panel-wall and the discrete-panel-wall were similar. Larger strains were observed in the discrete-panel-wall which had larger horizontal displacement of wall. On the other hand, as is shown in Fig. 6, the measured maximum tensions were almost the same with the design values in the upper 3-meter zone and lower than those in the lower 3-meter zone.

In case of the walls with the block-type facings, the additional short reinforcements (L = 1.0 m) were laid between the main reinforcements. The observed strain distribution on the additional short reinforcements ware shown in Fig. 5. The strain distribution in the walls with EPS-block facing and the concrete-block's one differed from each other, but both the walls showed the maximum strains near the facings and the magnitude of tensions were the same order of those working on the main reinforcements. It was made clear that additional short reinforcements contribute considerably toward the stability of reinforced soil walls.

3.5 *Shapes of failure lines*

The deformation of the thin lime layers was measured after the failure of each wall to determine its failure line. Fig. 7 shows the failure lines thus determined. Also indicated in this figure are the design failure line (Fs = 1.0) and the Coulomb's active failure line. The observed failure line corresponds to the cutting of a reinforcement at a point of time when the safety factor against sliding becomes about 1.0 ($Fs \fallingdotseq 1.0$), i.e., the cutting point No. 30 (the length of reinforcement, L = 1.88 m) of the EPS-block wall, No. 33 (L = 1.67 m) of the concrete-panel wall, and No. 36 (L = 1.57 m) of the concrete-block wall (Tajiri et al., 1995).

As shown in Fig. 7, the failure line of the wall with the EPS-block facing is closest to the design failure line. On the other hand, the failure line of the concrete-block wall is close to the Coulomb's active failure line, taking a shape of the failure line by the slip circle method. The failure line of the wall with the concrete panel facing is located deeper than the design failure line, taking a shape close to that of the failure line by the two-part wedge method. These results suggest that the difference in the effects of facings have influence on the shapes of failure lines in the critical condition.

4. CONCLUSIONS

For the purpose to ascertain the effects of facings upon the stability and deformation of reinforced soil walls, failure experiments of full-scale reinforced walls with different facings were conducted. The findings were as follows:

1. During the construction, the horizontal displacement of the reinforced soil wall with the concrete-block facing was smaller than that of the EPS-block facing, which means that displacement of a wall during construction can be reduced by the gravity resistance effect of its facing. The soil wall with the discrete-panel facing showed larger deformation than those of the EPS- and concrete-block facings.

2. In every case, a subgrade reaction larger than the self-weight of facing was observed. And the distribution of subgrade reaction under the reinforced zone was roughly equivalent to that of overburden pressure.

3. In every case, the maximum tensions distributed on the main reinforcements were almost equivalent (in

the upper half of the wall) to, or smaller (in the lower half) than, the required tensions at design without considering the wall effect of the facing. The larger the overall axial rigidity of the facing is, the stronger this tendency is.

4. Tensions of the same level with those working on the main reinforcements were observed on the additional short reinforcements laid in the walls with block-type facings, suggesting that the additional short reinforcements were contributing to the stability of the walls.

5. The failure line of the wall with the EPS-block facing after its failure was closest to the slip-circle failure line by the current design method. The failure line of the wall with the concrete-block facing was close to the Coulomb's active failure line, taking a shape of the sliding line by the slip circle method. The failure line of the wall with the concrete-panel facing took a shape which was close to that of the failure line by the two-part wedge method.

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REFERENCES

Geogrid Research Board. (1990). *Geogrid construction method guideline*, Vols. 1, 2, 244p, 150p, (in Japanese)

Nakajima, T., N. Toriumi, H. Shintani, H. Miyatake and K. Dobashi (1996), Field performance of a geotextile-reinforced oil wall with concrete facing blocks, *Proc. Int. Symp. on Earth Reinforcement Practice (IS Kyushu '96)*, Fukuoka (under contribution)

Nakane A., M. Taki, Y. Yokota and H. Miyatake (1996), FEM comparative analysis of facing rigidity of geotextile-reinforced soil walls, *Proc. Int. Symp. on Earth Reinforcement Practice (IS Kyushu '96)*, Fukuoka (under contribution)

Onodera, S., S. Naemura, A. Nakane and S. Maruo (1992). A design method for steep reinforced embankments with geotextiles, *Proc. Int. Symp. on Earth Reinforcement Practice (IS Kyushu '92)*, Fukuoka, Vol. 1, pp.391-396.

Public Works Research Institute, Ministry of Construction. (1992). *Design and construction manual of geotextile-reinforced embankments, Technical memorandum of Public Works Research Institute*, No.3117, 404p, (in Japanese)

Railway Technical Research Institute. (1992). *Design standard of railway structures and their explanation, earthworks*, 332p, (in Japanese)

Tajiri, N., Y. Tsukada, M. Yamamoto, Y. Ochiai and K. Dobashi (1995), Comparison among full-scale failure experiments of geotextile reinforced soil walls with different facing types, *Proc. 10th Geosynthetics Symposium on JC-IGS*, pp.37-46 (in Japanese)

Tatsuoka, F. (1992). Keynote lecture: Roles of facing rigidity in soil reinforcing, *Proc. Int. Symp. on Earth Reinforcement Practice (IS Kyushu '92)*, Fukuoka, Vol. 2, pp.831-870.