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A series of trial embankments of reinforced earth

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ABSTRACT: Described is a series of full-scale embankments of reinforced earth conducted by the authors in the past decade. The aim of conducting the serial field experiments is to demonstrate that the mechanical interaction between compacted earth and reinforcing materials contributes in stabilizing and strengthening the reinforced earth far more effectively than estimated based on the summation of independent contribution of earth and reinforcing materials. This unexpected effect of earth-reinforcement interaction seems to have existed not only in the earth structures reinforced by geosynthetics, but also in rock-bolted tunnels and in heavy structures supported by a group of friction piles as well as in natural slopes reinforced by nailing. The authors intend to visualize the unexpectedly comprehensive effect of earth-reinforcement interaction by introducing a series of full-scaled embankments of reinforced earth.

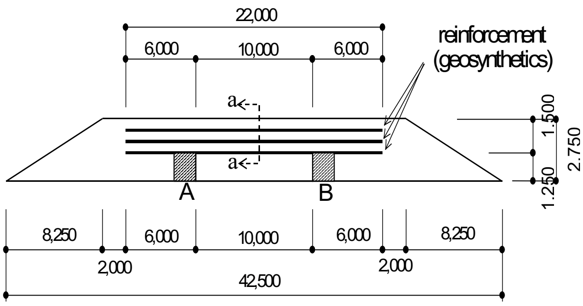
1. INTRODUCTION

During a period from 1992 through 1996, the authors conducted a series of field experiments by constructing 4 trial embankments of reinforced earth aiming at demonstrating how the compacted fine sand can form an unbelievable configuration if it is cleverly reinforced. The first author has long been puzzled by his impression that he got at many of the construction sites where reinforced earth, rock bolting, group friction piles, group mini piles, ground anchoring and slope nailing were under construction. Any kind of these reinforcing techniques gave a consistent impression to him that they work far better than they are expected to. An orthodox way to find the answer to decipher this impression may have been to construct usual type of reinforced earth fills and carefully observe the performance both of the compacted sand and reinforcing materials as have been already tried by many of the geotechnical engineers. However the authors did not take an orthodox approach because they intended to find something unknown that commonly observable in all kind of reinforced earth. They tried to construct the fills of something unusual shapes with an expectation of encountering the situation where unexpectedly surprising performance of reinforced earth could be observed. Hoping such unforeseen findings, the authors had constructed a bridge made of soil in 1992, an overhanging cliff made of soil in 1993, an overhanging cliff weak enough to reach the failure state in 1994 and an overhanging cliff together with a bridge and cantilever made of soil in 1996.

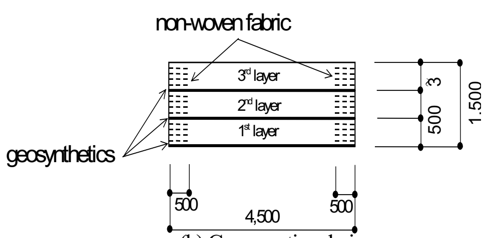
2. THE 1992 PROJECT

After carrying a series of preliminary model tests in the laboratory, the authors constructed a bridge of reinforced earth shown in Photo 1 and Figs. 1(a) and (b) during a period from 20th July through 8th August, 1992. On top of the three layers of H-section steel beams, three layers of a geogrid were placed at a vertical interval of 500 mm with the compacted fine sand sandwiched between geogrid. The reinforced earth bridge was 1500 mm thick, 4500 mm wide. The span of the bridge was widen by removing the H-section steel beams one by one. To avoid the direct contact between the lowermost (third from the top) geogrid and H-section steel beams, sheets of plywood were placed under the lowermost geogrid. The side surfaces of the bridge were exposed to the air so that people could see and feel that the bridge was really made of compacted sand. To keep the side surfaces of the bridge intact, 500 mm wide sheets of

Photo 1 Overall view of the bridge made of soil, 1992



(a) Side view



(b) Cross sectional view

Fig.1 Side view and cross sectional view of the fill, 1992
(in mm)

non-woven fabric were placed along the side surfaces at a vertical interval of 100 mm, see Fig. 1(b). A typical result of tensile test on geogrid is shown in Fig. 2. The cross sectional area and Young's modulus of the geogrid were $5.25\text{x}10^{-4}$ m^2 /m and $2.55\text{x}10^6$ $kN/m^2/m$. Table 1 shows the physical properties of the sand. The compaction tests on the sand are summarized in Fig. 3 while the performance of the field compaction is summarized in Table 2. The construction sequence is seen in Photos 2 and 3.

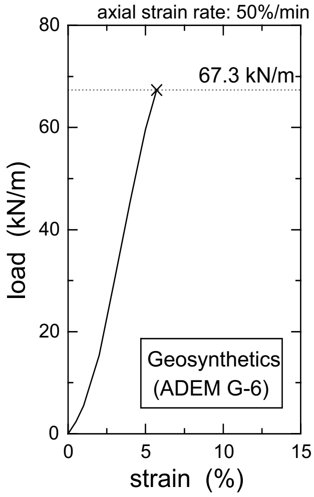


Fig.2 Tensile test on geogrid, 1992

Table 1 Physical properties of Onma sand (1992)

specific gravity of soil particle ρ_s (t/m^3)	2.71
grain size distribution	
gravel fraction 2mm～75mm (%)	0.2
sand fraction 75μm～2mm (%)	89.3
silt fraction 5μm～75μm (%)	10.5
clay fraction less than 5μm (%)	
uniformity coefficient U_c	1.60
coefficient of curvature U_c'	1.23
maximum grain size (mm)	4.75

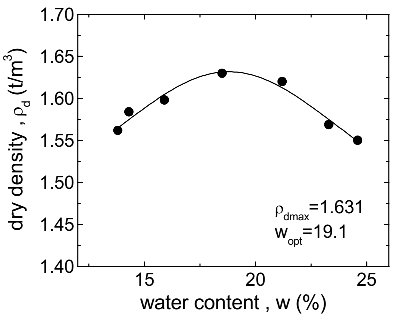


Fig.3 Compaction tests on the sand taken at Maki, Kanazawa, 1992

Photo2 Placement of H-section steel beams

The bridge was loaded by removing the supporting H-section steel beams. The maximum deflection of the earth bridge observed in the process of stepwise removal of the support was 590 mm when the span was broaden up to about 3 330 mm. The deformation of the earth bridge is shown in Photo 4. At the 6th step of loading, the lowermost geogrid was cut by the edge of the H-section steel beam being pulled out. The strain gauges on the geogrid indicated the concentration of the strain near the center of the bridge span.

3. THE 1993 PROJECT

Learning a lesson from the 1992 project, the authors constructed an overhanging cliff made of reinforced earth shown in Fig. 4 during a period from 4th September through 4th October, 1993. A support fill drawn at the left end of Fig. 4 was placed during construction and then excavated to load the overhanging cliff during the test. The reinforced fill was 5 000 mm high, 7 000 mm wide and 12 000 mm long. The vertical side surfaces of the fill were reinforced by horizontally placing non-woven fabric (500 mm wide) along the surfaces at a vertical interval of 100 mm in exactly the same way adapted in the 1992 project. The fill body was reinforced by geogrid (Photo 5) placed at a vertical interval of 500 mm. The overhanging cliff was wrapped layer by layer in geogrid with an overlapping length of 1 500 mm. Displacement markers were placed on the side surface to monitor stepwise progress of the deformation of the fill caused by stepwise removal of the support fill from the top to the bottom. The tensile test of the geogrid (cross-sectional area: $3.76\text{x}10^{-4}$ m^2/m, Young's modulus E: $3.72\text{x}10^6$ $kN/m^2/m$) is summarized in Fig. 5. The physical properties of the fine sand are summarized in Table 3. Compaction test results are shown in Fig. 6. The sand was compacted in the field by a vibratory roller with the thickness of each layer of 100 mm at the edges of the fill and 250 mm at the remaining part. Table 4 summarizes the performance of the field compaction. The full removal of the support fill resulted in unexpectedly small amount of deformation (about 200 mm of the settlement at the top of the overhanging cliff). The strength of reinforced earth was demonstrated by placing additional sand on top of the overhanging part of the fill together with a backhoe as seen in Photo 6.

Photo 3. Placement of the geogrid

Table 2 Summary of field compaction, 1992

layer	water content w (%)	dry density ρ_d (t/m^3)
1	15.8	1.46
2	18.1	1.40
3	16.8	1.52

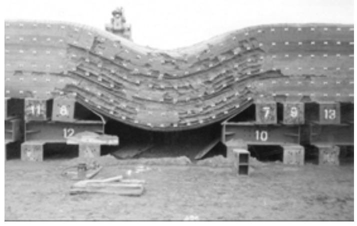


Photo 4 Deflection of the bridge (span: about 3 330 mm)

4. THE 1994 PROJECT

Since the reinforced earth in the 1993 project was found to have been too strong, the authors decided to construct another fill that is weak enough to reach the failure state. Chosen was the geogrid (cross-sectional area: $5.25\text{x}10^{-4}$ m^2/m, Young's modulus E: $1.06\text{x}10^6$ kN/m^2/m) much weaker than that in the 1993 project as seen in Fig. 7. The 1994 project was carried out during a period from 8^{th} August through 30^{th} September, 1994. Fig. 8 shows the dimensions of the fill placed by compacting the sand such as shown in Table 5. 18 sets of cutters were horizontally embedded in the fill body at a level of each geogrid. These cutters were located along two lines directing towards right upward at angles of 60 and 70 degrees (Fig. 8) standing from the toe of the reinforced overhanging cliff. These cutters were placed aiming at bringing the fill to failure at the final stage of the field experiment. The geogrid were to be cut by these cutters one by one starting from the one embedded at the highest level ending at the one embedded at the lowest level. The geogrid were to be cut firstly along the line standing at an angle of 60 degrees (Fig. 8) and then along the steeper line. The overhanging part was expected to slide down along either the gentler slope or the steeper slope at some stage during the process of cutting geogrid from the top to the bottom. The cutters were fixed to a steel chain similar to a bicycle chain and placed in the fill body during the construction stage. To the best of the authors' knowledge, the first trial of inducing the failure of reinforced earth by cutting the reinforcing material was made by Miki, Kudo, Taki, Fukuda, Iwasaki & Nishimura (1992). Vertical side surfaces of the fill were reinforced by horizontally placing 500 mm wide sheets of non-woven fabric in the same fashion as the previous projects. The vertical interval of placing the non-woven fabric was increased from 100 mm to 167 mm judging from the satisfactory performance of the wall reinforcement in the previous projects. The sand was compacted by a vibratory roller in each layer of 167 mm thick. The compaction tests are summarized in Fig. 9 while the field performance of the compaction work is summarized in Table 6.

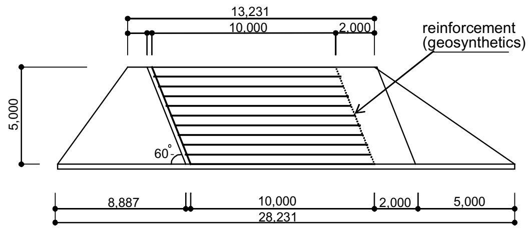


Fig.4 Side view of an overhanging cliff in the 1993 project (in mm)

Photo 5 Start of the construction, 1993

Photo 6 Overall view of the fill in 1993

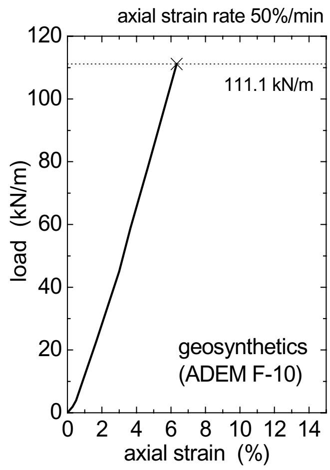


Fig. 5 Tensile test of the geogrid, 1993

Table 3 Physical properties of Onma sand
(sampled from Yuhidera, Kanazawa, in 1993)

specific gravity of soil particle	ρ_s	(t/m^3)	2.69
grain size distribution			
gravel fraction	2mm～75mm	(%)	0
sand fraction	75μm～2mm	(%)	82
silt fraction	5μm～75μm	(%)	9
clay fraction	less than 5μm	(%)	9
uniformity coefficient	U_c,		23.4
coefficient of curvature	U_c		12.3
maximum grain size	(mm)		2.0

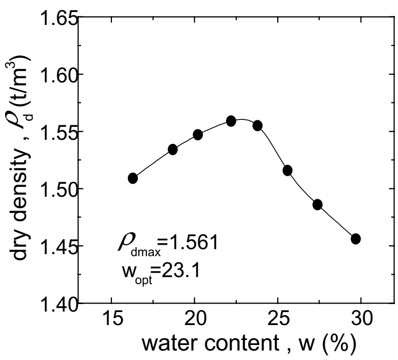


Fig.6 Compaction tests on Onma sand
(sampled from Yuhidera, Kanazawa, 1993)

Table 4 Summary of field compaction, 1993

layer	water content w (%)	dry density ρ_d (t/m^3)
1	25.2	-
2	26.2	1.41
3	26.9	-
4	24.1	1.45
5	26.4	-
6	25.7	1.41
7	26.5	-
8	26.3	1.39
9	26.2	-
10	16.9	1.37

The test started by removing the supporting fill shown in the left end of Fig. 8. Loading of the overhanging cliff was performed by stepwise removal of the supporting fill to depths of 0.7 m, 1.5 m and 2.4 m from the top. Large deformation of the overhanging cliff was observed during this process of loading. After leaving the fill for 5 days at this stage of loading, further loading went on by removing the supporting fill to depths of 3.3 m, 4.1 m and 5.0 m. Excessive deformation such as seen in Photo 7 eventually resulted in falling down of the overhanging part of the reinforced fill (Photo 8). After having the unexpected partial collapse, the intended collapse was initiated by cutting the geogrid. Firstly the cutters began to cut the geogrid at their embedded positions along the line inclined 60 degrees from the original ground surface (see Fig. 8). Cutting of all the geogrid at 9 levels resulted in the cracks along the line of 60 degrees. However the overall collapse did not take place at this stage. After spending 1 hour to make sure that the fill did not show any sign of progressive overall collapse, second series of cutting the geogrid along the line inclined at an angle of 70 degrees to the ground surface. Cutting went on from the 10th layer of geogrid at the top of the reinforced fill gradually downward, 9th, 8th and 7th. At the stage of cutting the 7th layer of geogrid, the overall collapse took place as seen in Photo 9.

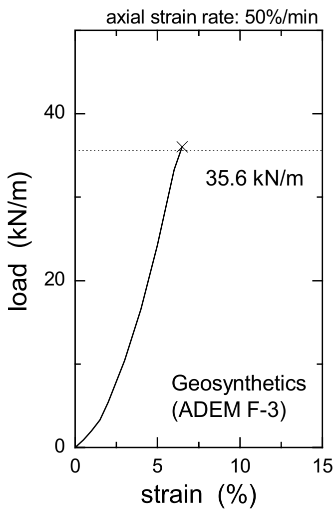


Fig. 7 Tensile test on the geogrid

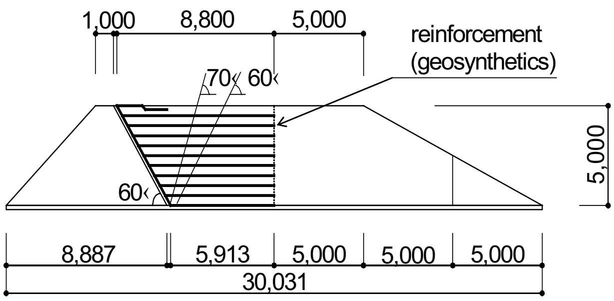


Fig.8 Side view of an overhanging cliff in the 1994 project (in mm)

Table 5 Physical properties of Onma sand
（sampled from Yuhidera, Kanazawa，in 1994）

specific gravity of soil particle ρ_s (t/m^3)	2.714
grain size distribution	
gravel fraction 2mm～75mm (%)	0
sand fraction 75μm～2mm (%)	69.8
silt fraction 5μm～75μm (%)	30.2
clay fraction less than 5μm (%)	
uniformity coefficient U_c	-
coefficient of curvature U_c’	-
maximum grain size (mm)	2.0

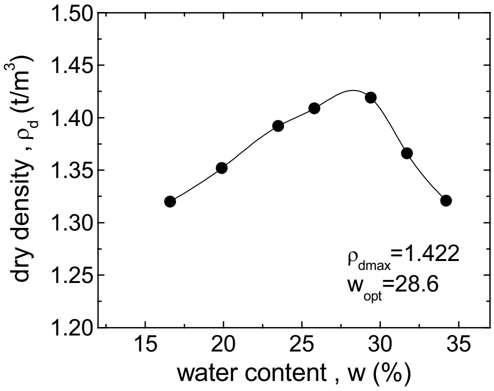


Fig.9 Compaction tests on Onma sand
（sampled from Yuhidera, Kanazawa, 1994）

Table 6 Summary of field compaction, 1994

layer	water content w (%)	dry density ρ_d (g/cm^3)
1	22.5	-
2	25.9	1.32
3	26.4	-
4	26.0	1.20
5	25.2	-
6	24.1	1.24
7	24.0	-
8	22.9	1.26
9	23.1	-
10	20.5	1.22

Photo 7 Excessive deformation of the third embankment in 1994

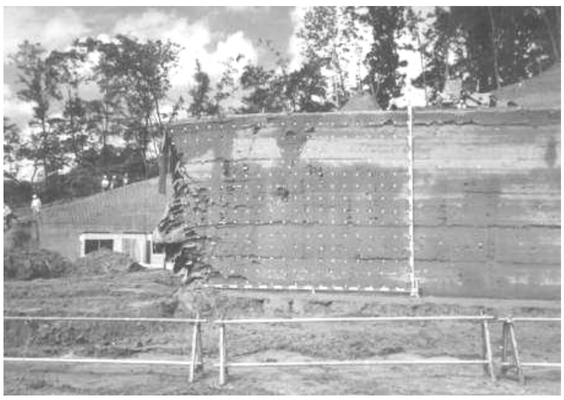


Photo 8 Falling down of the overhanging part of the reinforced fill

5. THE 1996 PROJECT

The 1996 project was planned as the final of a series of field tests on reinforced earth and performed during a period from 8^{th} July through 5^{th} October, 1996. The physical properties and the compaction test results of the sand used in the experiment are summarized in Table 7 and in Fig. 10. The mechanical properties of the geogrid (cross sectional area: 3.20×10^{-4} m^2/m, Young's modulus E: 4.86×10^{6} $kN/m^2/m$) are summarized in Fig. 11. The dimensions of the fill are shown in Figs. 12. The left hand side of the fill is an overhanging cliff as in the previous field tests while the right hand side of the fill is designed in such a way that the performances of a soil bridge and a soil cantilever are to be tested. The soil bridge and the soil cantilever are initially supported by blocks of expanded polystyrene and steel beams and then loaded by eliminating the supporting members during the experiment. To avoid the ill success that the authors had experienced in the first 1992 project in which a soil bridge collapsed, the authors decided to apply pre-stressing to the reinforced earth. The use of pre-stressing was suggested to the first author by Prof. F. Tatsuoka who had successfully established a new technique of pre-stressed reinforced earth (Uchimura, Tatsuoka, Sato & Tateyama, 1995, Uchimura, Tatsuoka, Koseki, Sato, Kodaka & Tateyama, 1995, Muramoto, Tateyama, Uchimura & Tatsuoka, 1996). Pre-stressing was applied in two ways as shown in Fig. 12 (b) where a half of the fill is densely pre-stressed while the remaining half is sparsely pre-stressed. Pre-stress was applied vertically through a steel bar sleeved by a chloroethylene pipe eliminating the friction between the soil and the steel bar.

Performance of the compaction work is summarized in Table 8. Photo 10 shows the ongoing compaction work while pre-stressing is seen in Photo 11. Photo 12 shows the appearance of the completed reinforced earth ready to undergo the various kinds of test. A series of test started by eliminating the supporting fill seen at the left end of Fig. 12(a). Photo 13 shows the completion of the removal of the supporting fill. At this stage the top edge of the overhanging cliff moved 320 mm downwards. Then the supporting members under the soil bridge were removed resulting in the maximum deflection of 110 mm as seen in Photo 14. The right hand side of the fill were planned to form a cantilever, but this was not achieved due to an excessive compression of the supporting members during the process of eliminating them one by one. Then the geogrid was cut along a vertical line at the center of the fill (Fig. 12 (a)) aiming at mechanical separation of the fill at the centre. The final stage of the test was to cut the geogrid along a line shown in the left side of Fig. 12 (a). Cutting proceeded from the geogrid placed at the highest level and went down layer by layer. The overall collapse took place at the stage when the geogrid at a height of 3,000 mm from the bottom of the fill (ie the ground surface) was cut as seen in Photos 15.

Table 7 Physical properties of Onma sand
(sampled from Taiyogaoka, Kanazawa，in 1996)

specific gravity of soil particle ρ_s (t/m^3)	2.74
grain size distribution	
gravel fraction 2mm～75mm (%)	2
sand fraction 75μm～2mm (%)	80
silt fraction 5μm～75μm (%)	11
clay fraction less than 5μm (%)	7
uniformity coefficient U_c	21.8
coefficient of curvature U_c'	5.82
maximum grain size (mm)	9.5

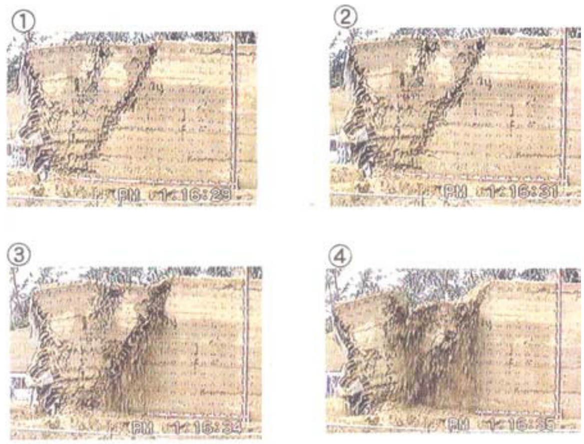


Photo 9. Video pictures at the moment of mechanically induced failure

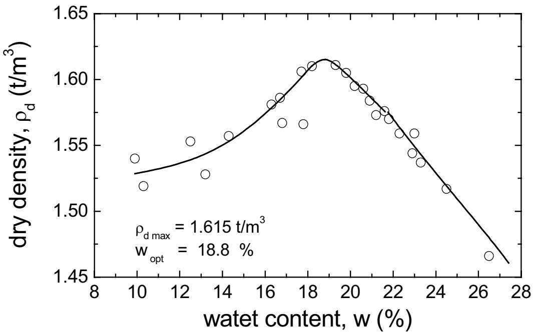


Fig.10 Compaction tests on Onma sand
(sampled form Taiyogaoka, Kanazawa, 1996)

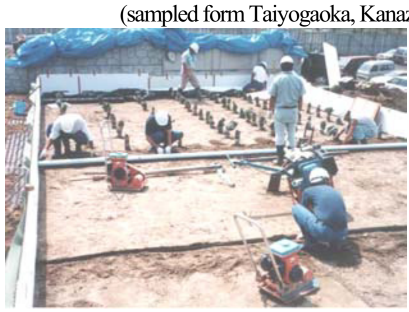


Photo 10. Construction works going on, 1996

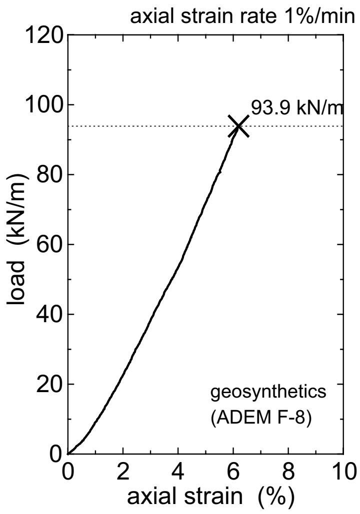
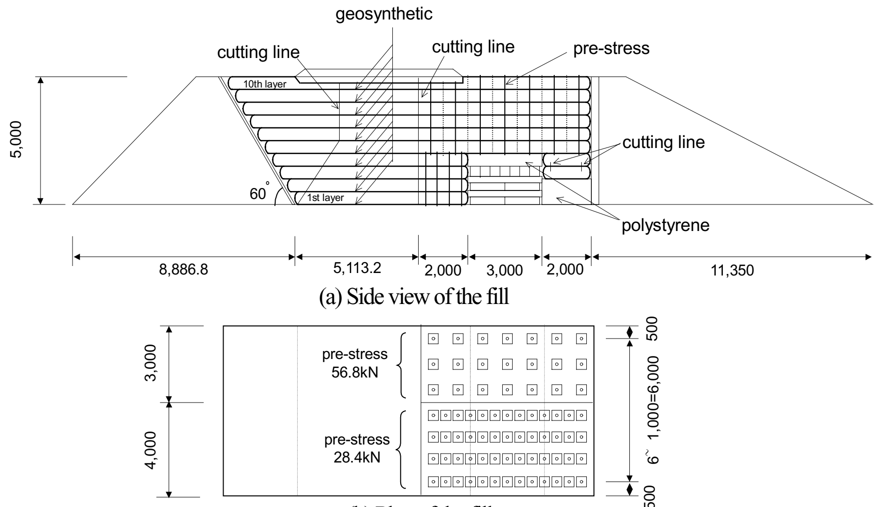


Fig.11 Tensile test on the geogrid, 1996



(a) Side view of the fill

(b) Plan of the fill

Figs.12刈Side view and plan of the fill 1996 (in mm)

5. CONCLUDING REMARKS

After carrying out a series of tests on full-scale fills of reinforced earth in something unusual shape, the authors are currently on the process of analysing the tests results. The unforeseen findings of unexpectedly surprising performance of reinforced earth may not have come true in the process of testing. But there are still many interesting aspects of the serial experiments that can be used as the evidences justifying some possible hypotheses needed in revealing the true mechanism of strength and rigidity mobilization commonly existing, but not apparently seen, in all types of the reinforced earth. A hopefully realistic constitutive model of compacted soils is being tested in a series of analyses of fill type dams by some of the authors of this paper. A soil/water coupled finite element programme for finite strain is being tested by some of the authors of this paper. These tools are currently used by the authors in the analyses of the interaction between compacted soils and reinforcing materials.

Table 8刈 Summary of field compaction, 1996

layer	water content w(%)	dry density ρ_d(g/cm^3)
1	19.7	-
2	14.7	1.29
3	13.7	-
4	16.2	1.30
5	13.0	-
6	12.4	1.28
7	15.1	-
8	12.4	1.31
9	14.6	-
10	17.5	1.37

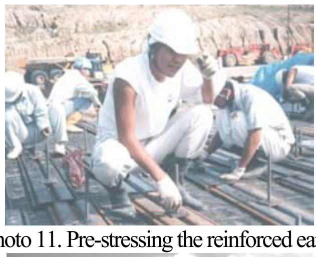


Photo 11. Pre-stressing the reinforced earth


Photo 12刈 The completed reinforced earth, 1996

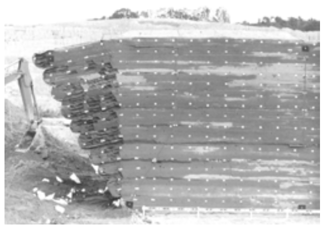


Photo 13 Completion of the removal of the supporting fill

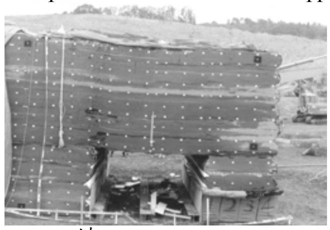


Photo 14刈Completion of the soil bridge

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