Case study on deformation of reinforced soil wall using geo-textiles

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ABSTRACT: When an unsoundness reinforced soil wall was built by some kind of factors, it is necessary to elucidate the mechanism by a detailed field work how deformation occurred, and we must take the measures that a similar phenomenon is not generated in future. This paper is findings to grasp factor of deformation of the geo-textiles reinforced soil wall which did be large deformation. The data which used of the subsidence of a history at the time of the construction, a physical characteristic, the bowling investigation into used laying earth on the ground materials, the rain and ground water level after the construction, the reinforcement domain pass, and is a change at time. We considered factor of deformation of the geo-textiles reinforced soil wall to a base by these data. The point concerned was the topography condition that the rain and melting snow water concentrated on a reinforcement soil wall, and, as a result of investigation, there was it in the environment where the ground water level in the reinforcement domain was easy to rise to by the rain because a substrate was weathering shale, and permeability was low. In addition, internal and external stability were impaired due to the declining interlocking effect of geo-grid and backfill soil. It was presumed that progression of the settlement of the upper part of the reinforced area by increasing the horizontal displacement of the skin wall.

Keywords: Site investigation, Drainage, Friction resistance,

1 INTRODUCTION

Reinforced soil wall construction is superior to concrete retaining wall construction in terms of economy and construction properties. However, cases of deformation in existing reinforced soil walls have been reported recently. The major factors in the reported cases of deformation were external factors such as localized torrential rainfall and large-scale earthquakes and the timing of the construction period, banking materials and construction method. For construction done during winter, when snow and frozen blocks of embankment material are included in the embankment, the frozen parts melt in the melting season and cause the embankment to subside, which results in substantial deformation of the embankment and walls.

When a reinforced soil wall is found to be unsound, the mechanism of deformation should be clarified in detail based on an understanding of the primary and provoking causes, which should be determined from construction records and the results of follow-up surveys. These investigation results shall be applied towards preventing construction results that are similar to those of the deformed walls.

In this report, a geo-textile reinforced soil wall (hereinafter, a reinforced soil wall) that deformed to a considerable extent was investigated and the factors causing the deformation are discussed. The authors investigated the construction records, records on the physical properties of the embankment materials, boring survey results, records of rainfall and groundwater level after construction, and records on subsidence in the reinforced region.

2 OUTLINE OF THE REINFORCED SOIL WALL

2.1 *Basic data of the reinforced soil walls*

Figure 1 a), b), and c) are the top, vertical and cross-sectional views of the reinforced soil wall. The reinforced soil wall was attached to an abutment that straddles a branch road (cut section) constructed halfway up a hillside in a hilly area. We were constructed using a reinforcing material of 3.8m in length with vertical intervals of 0.9m, the middle 2 layers were constructed using a reinforcing material of 6.3m in length with vertical intervals of 1.2m, and the top 3 layers were constructed using a reinforcing material of 8.8m in length with vertical intervals of 1.2m. Reinforcing materials with three different strengths were used. The tensile strengths of the reinforcing material for the bottom, middle, and top layers were 90kN/m, 72kN/m, and 49kN/m, respectively. For Type B, the bottom 3 layers were constructed using a reinforcing material of 3.0m in length with vertical intervals of 0.9m. The middle 2 layers were constructed using a reinforcing material of 5.0m in length with vertical intervals of 1.2m, and the top 2 layers were constructed using a reinforcing material of 7.0m in length with vertical intervals of 1.2m. Reinforcing materials with three different strengths were used for Type B, too. The tensile strengths of the reinforcing material for the bottom, middle, and top layers were 90kN/m, 60kN/m, and 37kN/m, respectively.

The reinforcing materials were installed in compliance with the Design and Construction Manual for reinforced soil walls of this type (hereinafter, the manual). The cohesion was c=0kN/m^2, the angle of shear resistance was φ=30°, and the unit weight of the soil was γ=19kN/m^3. All of the verification items, including the internal stability (circular slip and pull-out strength), the external stability (sliding, overturning, and bearing capacity), and the total stability (slip resistance including that of the foundation ground), were satisfied. The length of each reinforcing material is determined based on the result of the total stability analysis.

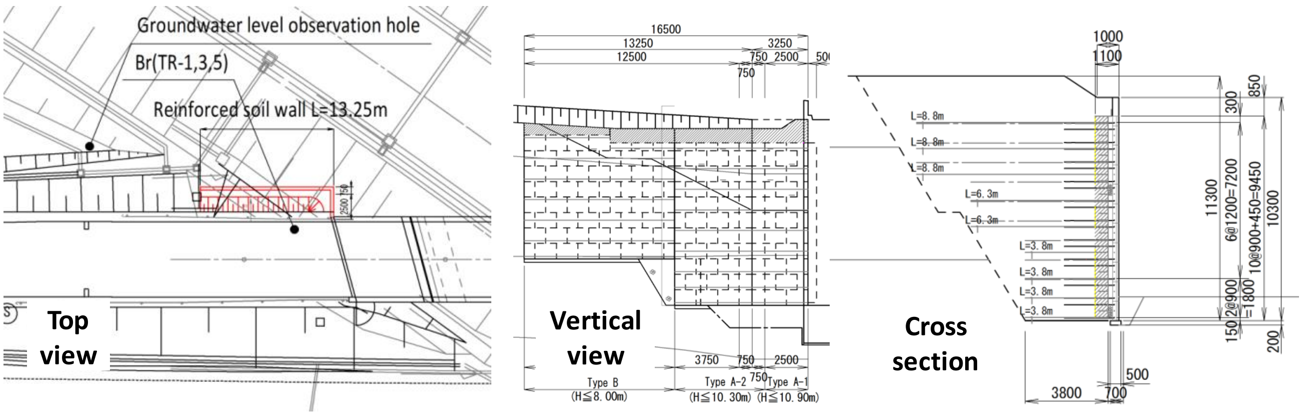

Figure 1. Top, vertical, and cross-sectional views of the reinforced soil wall.

2.2 *Construction and investigation records for the embankment materials*

2.2.1 *Embankment materials*

Table 1 shows the results of soil quality tests on the embankment materials. The sampling after deformation of the wall was done by using a triple tube sampler at the locations of TR1, TR3, and TR5 and the depths from the embankment crown exposed by removing the base course (GH=434.17m) were 0.60 - 1.54m, 5.30 - 6.14m, and 8.40 - 8.83m, respectively (Fig. 1). For the embankment materials, two types of soil generated near the construction site (onsite soils A and B) were used, and the fine fraction content F_c for the two types of soil was 20% or lower. The natural water content w_n for the onsite soils A and B was close to the optimum moisture content w_{opt}. The strength parameters determined in the consolidated drained (CD) tri-axial compression test were c=20kN/m^2 for cohesion and around φ=30° for angle of shear resistance. When conducted after the wall deformed, the test revealed that the fine fraction content at TR3 was about F_c≒50%, which was relatively high among the fine fraction contents of the samples. The soil particle density, natural water content and consistency did not greatly differ among the samples from these three locations. The embankment materials used in the construction were judged as those conforming to the specifications of the manual. The tests revealed that the materials had high shear resistance and low compressibility, making them easy to compact. The materials were determined to be suitable for constructing a reinforced soil wall, even though a considerable amount of fine fraction was mixed in the layer at the mid height of the wall.

Table 1. Results of soul quality tests for the embankment materials.

Embankment samples		Survey before construction		Survey after deformation		
		Onsite soil A	Onsite soil B	TR1	TR3	TR5
Classification of the ground materials		Sandy gravel with clay		Sandy gravel with clay		
Soil particle density ρ_s(g/cm^3)		2.6	2.7	2.8	2.9	2.8
Natural water content w_n(%)		16.3	21.0	23.3	28.0	7.2
Grain size distribution	Grain fraction(%)	62.7	48.6	76.4	31.1	76.2
	Sand fraction(%)	24.5	25.9	13.0	20.5	14.2
	Fine fraction(%)	5.7	13.6	10.6	48.4	9.6
	Max. grain size(mm)	37.5	37.5	53.0	26.5	53.0
Consistency	Liquid limit W_L(%)	47.2	47.5	63.3	47.4	NP
	Plastic limit Wp(%)	24.4	24.4	26.5	24.9	NP
	Plasticity index Ip(%)	22.8	23.1	36.8	22.5	NP
Compaction test	Testing Methods	B-a	B-a	-	-	-
	Max. dry density(%)	1.7	1.6	-	-	-
	Optimum moisture content(%)	18.6	20.2	-	-	-
Triaxial compression test	Testing Methods	CD	CD	-	CD	-
	Cohesion c(kN/m2)	20.7	22.6	-	13.1	-
	Angle of shear resistance ϕ(°)	32.2	29.1	-	38.1	-

2.2.2 *The construction period and the quality control of the embankment.*

The embankment in question was constructed in two stages in FY 2012. Figure 2 shows the embankment being constructed and the base course being constructed. The first stage of construction, in which the embankment was constructed up to the wall height of 3m, was done in 10 days from mid to late July 2012. The second stage of construction, in which the rest of the wall height up to 10m was constructed, was done in 9 days from late December 2012 to early January 2013. The machine for rolling compaction was a 200kN-class vibrating roller. The completion thickness for each layer was 30cm. The construction work was not done during the period from December 29, 2012 to January 6, 2013.

For quality control of the embankment, a soil density test using a sand replacement method was done for the twelfth compaction layer (equivalent to the wall height of 3.6m). The embankment material in the layer for the quality-control test was onsite soil A. The dry density of that soil was ρ_d=1.538g/cm^2, which satisfied the specified quality control standard value (≧90%) for degree of compaction D_c. The calculation for degree of compaction D_c was done by using the maximum dry densities for onsite soils A and B, which was determined from the dry densities obtained from the samples from TR1, TR3, and TR5. The degree of compaction for both of the onsite soils A and B satisfied the value of D_c≧90%.

Figure 2. The embankment and the base course under construction.

3 RECORDS FOR THE DEFORMED REINFORCED SOIL WALL

3.1 *Deformation of the wall panels*

After the embankment was constructed to the design height of the reinforced soil wall, construction of the base course and pavement were sequentially done. The road entered (opened) service in late January 2013. In late June 2013, which was about 5 months after the road section entered service, deformation of the wall constructed with the wall panels and subsidence in the embankment at the back of the abutment were found. Measurement and visual observation of the deformation of the wall and measurement of subsidence in the reinforced region were started after the wall was found to have deformed and the embankment was found to have subsided.

Figure 3 shows the reinforced soil wall, whose joints between the wall panels had become irregular in height and whose horizontal displacement of wall panels had become conspicuous. The wall was supported by large sandbags. On November 29th, which was about 5 months after the start of measurement, it was judged that the horizontal and forward displacement of the wall panels would not stop, and the deformation of the wall was controlled by banking up large sandbags in front of the wall panels.

Figure 3. The deformed reinforced soil wall and the deformation control work.

3.2 *Settlement of the reinforced region, and weather data*

Figure 4 shows the daily rainfall (mm/day) and air temperature of the area around the site of the reinforced soil wall obtained from the Automated Meteorological Data Acquisition System (AMeDAS). It also shows the subsidence measured 1) near the wall (indicated by red marks), 2) at the center of the road (blue marks), and at the embankment toe (green marks). The measurements for each of 1) to 3) were done at three locations distributed longitudinally to the road axis, i.e., measurements were done at 9 locations in total.

Settlement of the embankment tended to increase with time after the start of measurements in late June 2013. The subsidence before the overlay of October 8th had a maximum of about 135mm near the wall (i.e., 3.25m to the reinforced soil wall side from the center of the road), a maximum of about 110mm at the center of the road, and a maximum of 100 mm at the embankment shoulder (i.e., 3.25m to the embankment shoulder from the center of the road). Subsidence at these three locations rapidly progressed after heavy rainfall of about 70mm/day in late August and mid September, all of which characterized the subsidence. The interpretation of the mechanism of this subsidence will be discussed in a later section. On 8th October, overlay was done on the reinforced region at the back of the abutment; however, a second overlay was done on 24th April 2014, because further subsidence with a maximum of 15 mm occurred after the first overlay. After the second overlay, measurements for subsidence were continued until late July 2014. No behaviors that indicated further progress in subsidence were found.

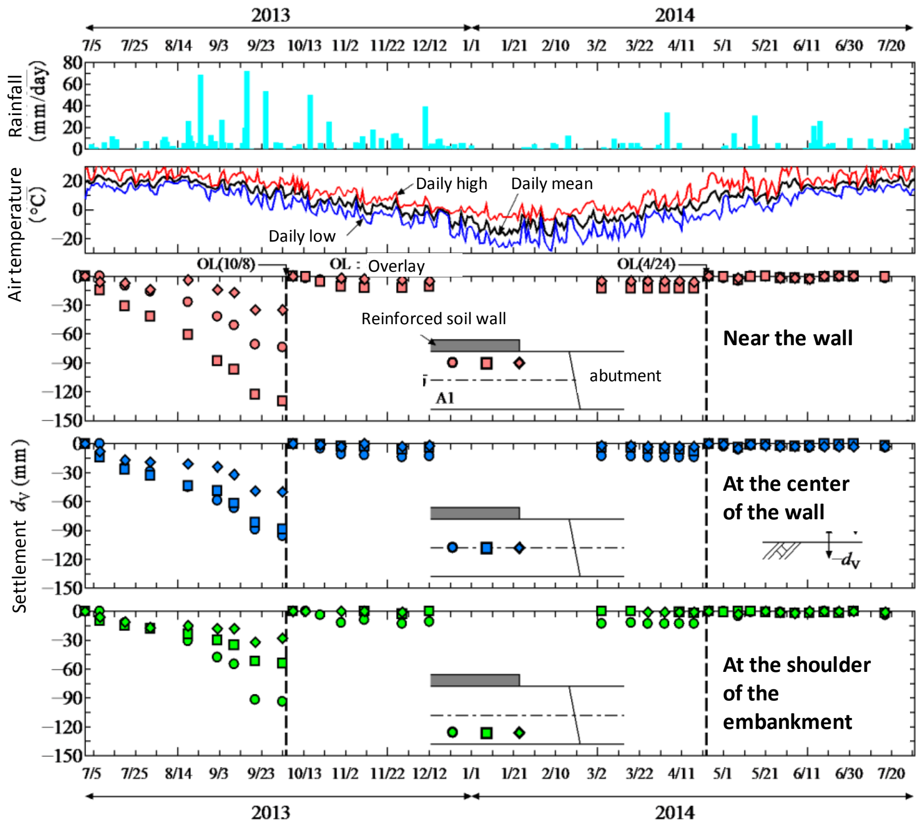

Figure 4. Settlement of the reinforced soil wall with time, and rainfall records.

3.3 *Groundwater level*

To estimate the groundwater level inside the reinforced soil wall, manual measurements were taken regularly by using a tape water level meter from 28th May to 6th August 2014. A polyvinyl chloride pipe (VP 150) was installed in the bore hole near the reinforced soil wall (Figure 1). The polyvinyl chloride pipe was a strainer pipe with holes that secure the smooth inflow of groundwater. The outer surface of the pipe was covered with non-woven fabric to prevent the intrusion of sand into the pipe. Figure 5 a) shows the changes with time in the observed groundwater level and the daily rainfall obtained from AMeDAS.

From this graph, the tendencies of increase/decrease in daily rainfall and in groundwater level roughly agree. For example, the groundwater level observed on 18 th June is the highest in the observation period, and a rainfall of about 30 mm/day was observed on the day before 18 th June. Figure 5 b) shows the water level inside the reinforced soil wall estimated from the maximum and minimum groundwater levels measured in the bore hole (Figure 5 a)). If the crown of coping concrete is assumed to be at the design height when the observed groundwater level is the highest (EL=433.15m) during the measurement period, the groundwater level inside the reinforced wall could be estimated at the height of about 1.7m lower than the observed groundwater level. Because of this groundwater level, it was possible that the saturation for most of the soil in the reinforced region was high.

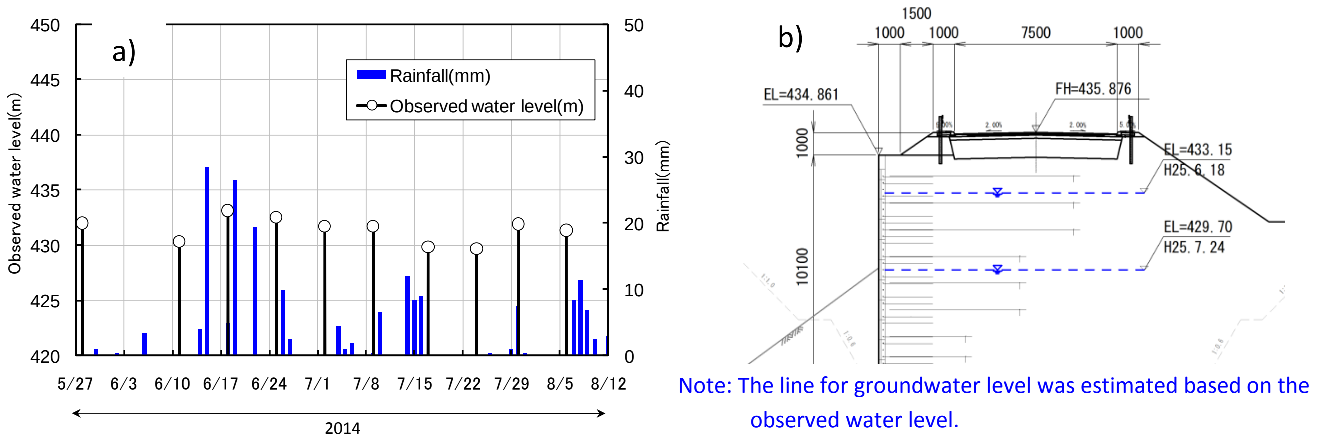

Figure 5. The deformed reinforced soil wall and the deformation control work.

3.4 *The soil condition in the internal part of the embankment*

Figure 6 shows a photo of the sample bore core. In this photo, the depths of laid reinforcing material at the sampling are indicated together with the design installation depths of the reinforcing material. The broken green line shows the location of reinforcing material installed at construction. The broken red line shows the location of the reinforcing material identified in this survey. The difference between the location at construction and that at survey (the depth of the broken green line minus the depth of the broken red line) is the subsidence, which was 25 - 45cm in the layers at the depth of 6m and shallower, and about 5cm at locations deeper than 6m. This showed that the embankment subsided in the short period of one and half years after construction was completed.

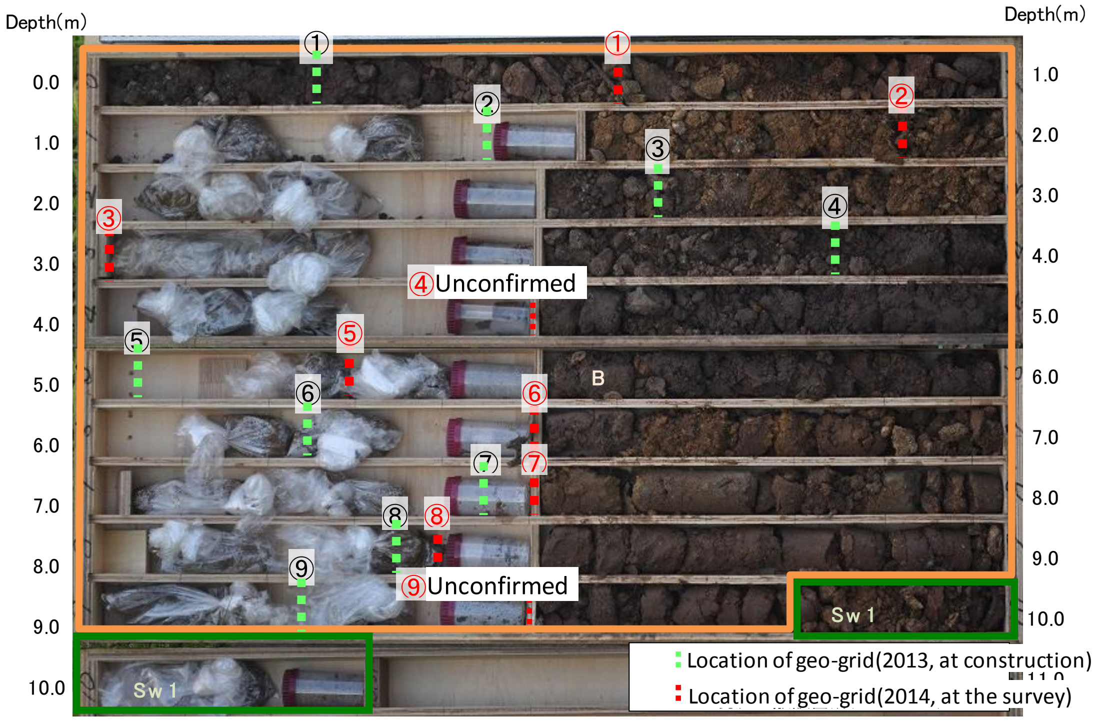

Figure 6. Settlement of the reinforced soil wall with time, and rainfall records.

4 ESTIMATIONS OF THE MECANISM OF DEFORMATION OF THE REINFORCED SOIL WALL

Figure 7 shows a topographic map and the catchment of the area that includes the reinforced soil wall. The area at the back of the reinforced soil wall is land with gentle hills (a meadow), and the shape of the site where the reinforced soil wall was constructed is a concavity that collects rainwater and snowmelt down toward the abutment (reinforced soil wall). The reinforced soil wall in question was constructed in a shallow valley whose shape makes water tend to collect there. It was found in the boring survey that the bearing stratum for the reinforced soil wall and the abutment was weathered shale with low permeability; therefore, the rainfall and snowmelt water from the hilly area, which was higher in elevation than these two structures, was estimated to flow down toward the abutment (and the reinforced soil wall) through the ground over the weathered shale with low permeability (Figure 8). The factors described above are thought to have contributed to the increase in the degree of saturation inside the reinforced soil wall at times of rainfall and in the snowmelt season.

Figure 9 shows the grain size distribution curves of the embankment materials as clarified in the pre-construction survey. The fine fraction content of the embankment material was about F_c=10%, which should raise no problem for the embankment material in a reinforced soil wall. However, material with a fine fraction content of nearly 50% was used at the depths of 5.30 - 6.14m (TR3) from the embankment surface from which the base course was removed.

Hayashi et al. examined the tendencies in the decrease and recovery of friction resistance between a geo-grid and an embankment in a laboratory pull-out test. The test was done by generating vertical confining pressure on a geo-grid placed in the soil chamber, repeatedly submerging the embankment soil in water and then pulling out the geo-grid. The test found that, regardless of the size of the fine fraction, when

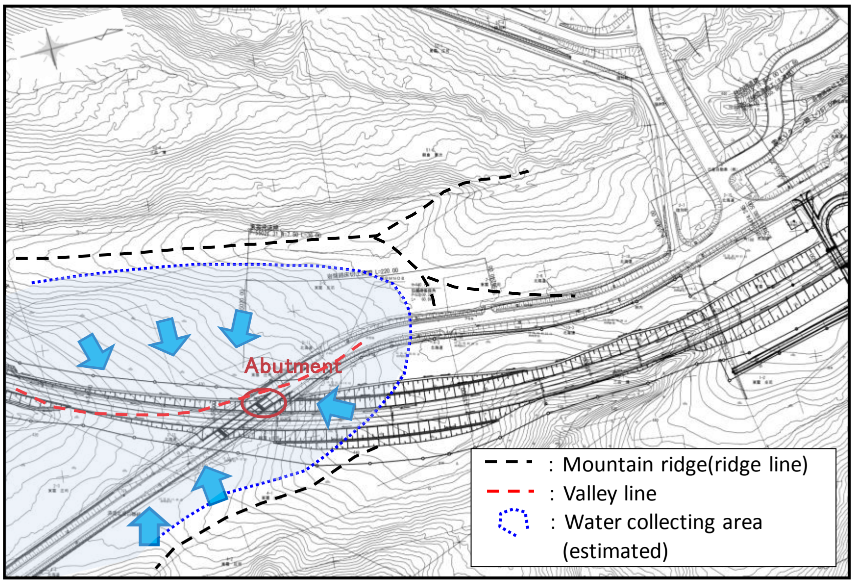

Figure 7. Topography of the land behind the reinforced soil wall.

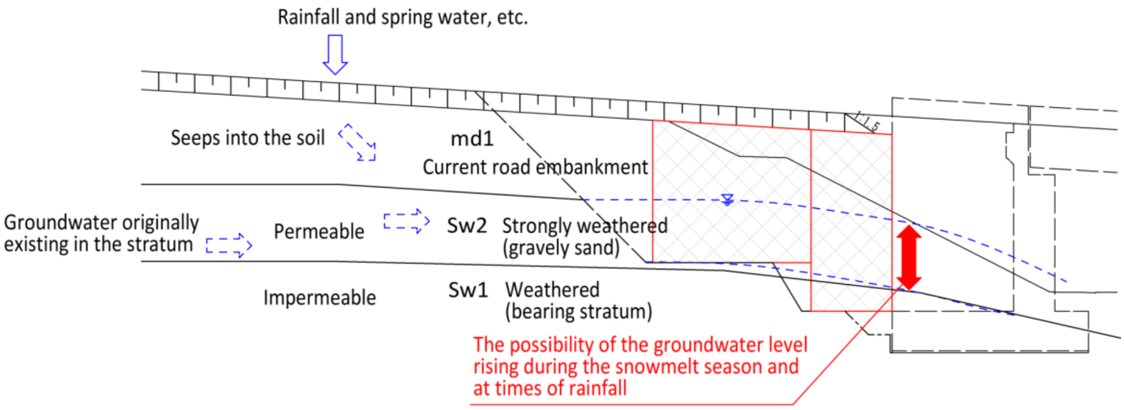

Figure 8. Concept of movement of seepage water toward the reinforced soil wall.

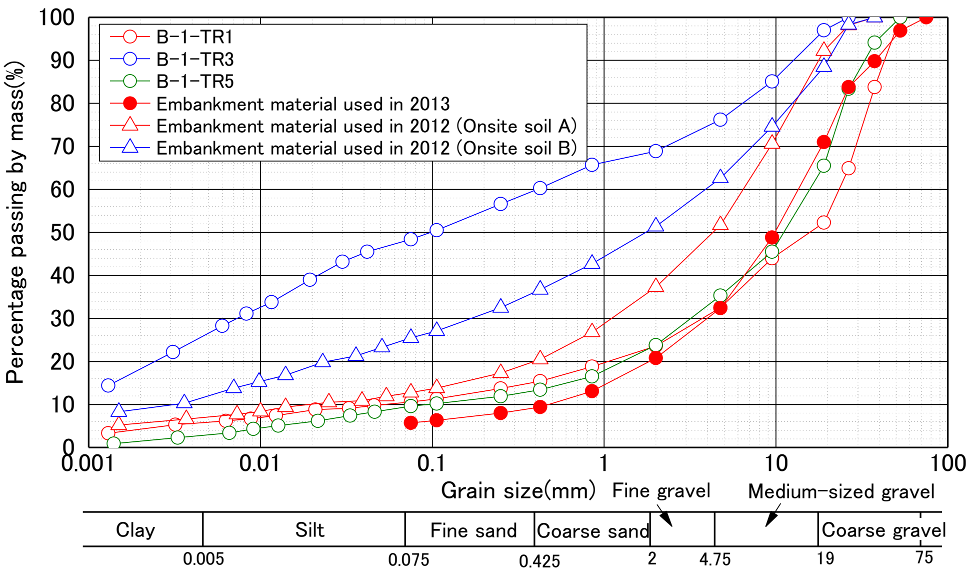

Figure 9. Grain size distribution curves of the embankment materials.

the geo-grid and embankment soil were saturated, the apparent cohesion was lost and the friction resistance between the geo-grid and the embankment material was less than that which had existed at partial saturation(unsaturation). Hayashi et al. also reported that the lower was the confining pressure on the geo-grid and the higher was the fine fraction content, the smaller was the recovery in friction resistance between the geo-grid and the embankment after saturation and drainage. Based on these test results, the major factors in the deformation of the reinforced soil wall were judged to be as follows.

The reinforced soil wall was constructed in a small, shallow valley on a hillside whose shape made the site prone to collect water. The rainfall and snowmelt water from the hilly land flowed into the ground, over the weathered shale with low permeability, and into the ground through the boundary between the cut and fill sections toward the abutment. The embankment subsidence rapidly progressed after rainfall of about 70mm/day in late August 2013, which was immediately after the reinforced soil wall was completed, and in mid September 2013. The rapid subsidence was attributed to large amounts of rainwater having intruded into the reinforced region and raised the saturation of the embankment material in the reinforced region, which lowered the interlocking effect between the reinforcing material and the embankment materials. The soil pressure acting on the wall panels exceeded the design friction resistance when the friction resistance between the reinforcing material and the embankment material decreased from the increase in saturation and the delay in recovery of friction resistance, which were caused by the existence of a high fine fraction content in part of the embankment. It is inferred that, because of the factors described above, the horizontal and forward displacement of the wall panels increased with time and the subsidence of the embankment crown also progressed in the reinforced region.

5 SUMMARY

In this report, a reinforced soil wall in which deformation had occurred was investigated to clarify the factors that caused the deformation and the mechanism of deformation. The investigated items included construction records, quality control records of the embankment, records of physical properties for the embankment materials, bore core test results, records of rainfall and groundwater levels after construction completion, and changes in subsidence for the reinforced region with time. The investigation identified several factors involved in the deformation, including the reinforced soil wall’s construction in a shallow valley where rainfall and snowmelt water flow in, and the water’s flow into the embankment of the reinforced soil wall. The ground at the construction site was in an environment where the saturation within the reinforced soil wall was prone to rises because the load-bearing stratum of the site was weathered shale with low permeability. Water from rainfall and other water from the land of higher elevation than the reinforced soil wall seeped into the ground, flowed over the shale stratum, and increased the saturation within the reinforced soil wall. During a heavy rainfall, saturation increased in the reinforced region, up to the top embankment layers, and this reduced the interlocking effect between the geo-grid and the embankment materials. The decrease in friction resistance between the reinforcing material and the embankment materials resulted in the loss of internal stability. It is inferred that, because of the factors described above, the horizontal and forward displacement of the wall panels increased with time and the subsidence of the embankment also progressed in the reinforced region.

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