공학석사 학위논문

Experimental study of long-term creep behavior of sandy soil in embankment

사질토로 구성된 성토체의 장기 침하거동에 대한 실험적 연구

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임철민 석사 학위논문을 인준함

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Abstract

Experimental study of long-term creep behavior of sandy soil in embankment

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Creep is a time-dependent deformation phenomenon at constant effective stress. Long-term severe creep deformation of a structure can provoke serviceability issues. Most embankments in South Korea consist of residual soils and are constructed to sustain the allowable settlements for target infrastructures such as railways and roads.

Settlement management is one of the critical issues of the embankment, and long-term creep settlement is more critical to the structure, which strictly allows residual settlement. Such long-term deformation problems in structure can be prevented through an investigation on creep deformation in advance.

Therefore, early researchers have conducted creep experiments and measured on the embankment to evaluate the susceptibility of creep deformation of the soils.

According to studies on creep so far, it is known that the amount of creep deformation over time depends on various variables, such as stress condition, relative compaction, fine contents, etc.

The long-term creep behavior of residual soils in embankments has not been thoroughly investigated clearly in South Korea. In this study, the influence factors of the creep deformation in residual soils will be identified, and the extent of each factor and their co-relationship will be analyzed based on the experiment. To achieve these goals, a series of triaxial creep tests were conducted to evaluate the effects of the stress ratio, relative compaction and fine contents. The stress ratio of the simulation condition was determined by embankment simulation results. And relative compaction and fine contents of the specimen or soils were determined according to the domestic and international design guidelines for high-speed rail code.

The result of creep deformation under various factors are discussed in detail. Creep strains were analyzed by performing a series of triaxial creep tests. In addition, based on the experimental results, the creep parameters for numerical modeling are evaluated to predict creep deformation in the construction site. Each creep coefficient relationship was evaluated based on the influence factors.

As a result, the creep strains decreased relative compaction and increased fine contents and stress ratio. And it is essential to use high-quality embankment materials such as low fine contents and high relative compaction to prevent severe long-term creep deformation.

Keywords : creep behavior, embankment, fine contents, relative compaction, stress ratio

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Chapter 1. Introduction

1.1 General

Creep is a time-dependent deformation phenomenon at constant effective stress, and Mitchell and Soga(2005) demonstrate the time-dependent strains in shear and volumetric controlled by the resistance of soil structure. Long-term severe creep deformation of a structure can invoke serviceability issues like decrease running speed in railways and roads.

Most embankments in South Korea consist of residual soils and present in most large infrastructure. Embankments were constructed to sustain the allowable settlements for target infrastructures. Settlement management is one of the important issues of the embankment, and long-term creep settlement is more critical to the structure, which strictly allows residual settlement such as 100mm in 10years. Such long-term deformation problems in structure can be prevented through an investigation on creep deformation in advance.

Recent studies have revealed that mechanism of creep deformation. Charles(2008) and Sowers(1965) measured the settlement of the embankment rock-fill dam and revealed a linear relationship between creep settlement and time. Park(2008), Murayama(1984), Mejia et al.(1988), and Yamamuro et al.(1993) conducted creep experiments on various variables and revealed each effect on creep phenomenon.

According to studies on creep so far, it is known that the amount of creep deformation over time depends on various variables, such as stress condition, relative compaction, fine contents, etc.

1.2 Purpose and scope of the study

In this study, the influence factors of the creep deformation in residual soils will be identified, and the extent of each factor and their co-relationship will be analyzed based on the experiment.

To achieve these goals, a series of triaxial creep tests were conducted to evaluate the effects of the stress ratio, relative compaction, and fine contents. Firstly, the stress ratio is determined based on the stress condition in the embankments. In order to obtain the in-situ stress condition, numerical analysis results of the embankment for the high-speed railway was incorporated in the study. Secondly, the compaction and the fine contents of the specimen or soils were determined according to the domestic and international design guidelines for high-speed rail code.

During the triaxial creep test, the vertical displacement of specimen was measured, and the axial strain was calculated. Creep deformation was evaluated under various stress ratio, relative compaction, and fine contents, and each variable was compared.

Based on the experimental results of the relation of axial strain and time, parameters were defined by creep models. And each parameter influence of creep deformation was evaluated. Using the creep parameters obtained in the experiment, the actual settlement of the site was predicted and compared with the measurement data on the site.

Chapter 2. Literature Review

2.1 Previous research

2.1.1 Creep in sand

Creep is a time-dependent deformation phenomenon at constant effective stress And Mitchell and Soga(2005) demonstrate the time-dependent strains in shear and volumetric that controlled by the resist of soil structure.

Park(2015) conducted triaxial test to investigate the creep behavior of granular materials and demonstrated the time-dependent behavior of granular materials depends on the material properties, stress ratio and relative density.

Charles(2008) measured on the embankment of the rockfill dam. There is a linear relationship between creep settlement and time. Equation as follows :

$$S = \alpha H log_{10} \frac{t}{t_0} + S_0 \qquad (1)$$

In this equation, S is the settlement, H is the height, t is the time, α is rate of settlement that occurs over 1 to 10 years. Sowers(1965) calculate α as 0.2 ~ 1.1 % , Charles(2008) calculate α as 0.17 ~ 1.0 %. In common case, α is under 0.2 %. If α over 1.0 %, it is rarely large value.

According to the Japan Railroad Comprehensive Technology Research Institute (2007), compression settling occurs as much as 0.1 to 0.3% of the height of the embankment in the embankment that has been sufficiently compacted according to the compaction standard. When calculating the compression settling of the actual embankment, the settlement – time(log) relationship graph was linearly displayed after 600 days, so the creep settlement was predicted using data after 600 days. In addition, it stipulates that the settlement of the embankment body should be managed by setting the amount of settlement according to the period of the embankment to 10mm/10years.

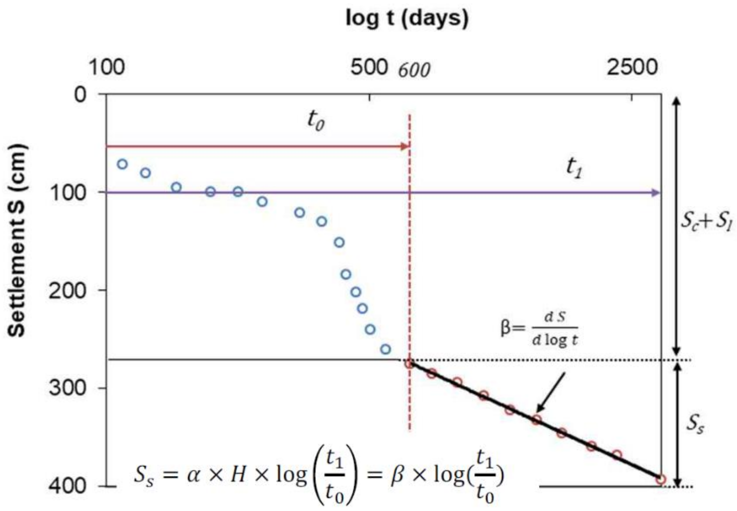

Figure. 2.1 Creep settlement prediction method(JICA, 2015)

Murayama(1984) showed that the creep test results for loose sand and predicted by the equations based upon the rheological model.

Mejia et al. (1988) demonstrated the creep strains increase as the stress ratio, confining pressure, stress increment size and grain angularity increase in the drained triaxial creep tests.

Yamamuro et al. (1993) demonstrated creep deformation increases as the confining pressure increases, and creep settlement resulting from particle breakage is more dominant element at high stress level.

Kuwano et al. (2002) also conducted triaxial tests to investigate the creep behavior of granular materials. Experiment designed different relative density. The tested granular materials showed significant time-dependent behavior.

2.1.2 Mechanism of creep

In high-stress conditions, particle crushing is considered as the primary mechanism of creep in granular materials.

Leung et al. (1997) conducted compression tests with sands. This research was conducted under high confining pressure; the particle breakage played the important role in the creep. Gradual particle breakage increases the contact between the particles and causes settlement, soil resistance, and stiffness gains with time.

Karimpour et al. (2013) detected the particle crushing during creep under high confining pressure.

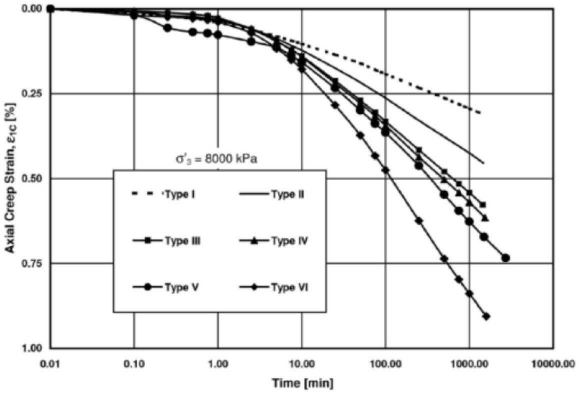

Figure. 2.2 Effect of grain distribution curve on variation of axial creep strains with time(Karimpour and Lade, 2013)

Lade et al. (1998) conducted a series of isotropic compression triaxial tests. The viscous characteristics of granular media result from slippage between particles.

2.1.3 Effects of fine-contents on the time-dependent behavior

Wang et al. (2009) conducted the experiment by using kaolinite powder. Kaolinite powder was added to the sand in order to study the influence of fines. The stiffness of the specimen increases, and contact property between sand particles also changed. As shown in the figure, kaolinite powder has a lager small-strain damping ratio than sand. The sand samples can raise the aging rate because of the more significant creep and aging rate made by the kaolinite.

Karimpour et al. (2013) investigated a series of triaxial creep tests for specimens with different gradation curves. The figure shows the effect of the grain distribution curve on the axial strains. Type 1 with higher uniformly graded soil shows lower axial creep strains. The creep deformation increases with the fine contents because adding fines results in more contact points between the particles. The effect of creep on weathered residual soil containing fine contents is greater than that on uniformly graded sand.

2.2 General creep behavior

Mitchell and Soga(2005) demonstrate creep is the time-dependent strains in shear and volumetric that controlled by the resistence of soil structure. Soils fail under a creep stress less than the peak stress in a shear test wherein a loaded to failure in a few minutes or hours. This phenomenon demonstrated creep rupture. And it is important to development of slope failures in soil.

The creep response is divided into three stages. First is a period of transient creep during which the strain rate decreases with time. The second period creeps at constant rate. And tertiary period creep rate then accelerates, leading to failure in susceptible rupture.

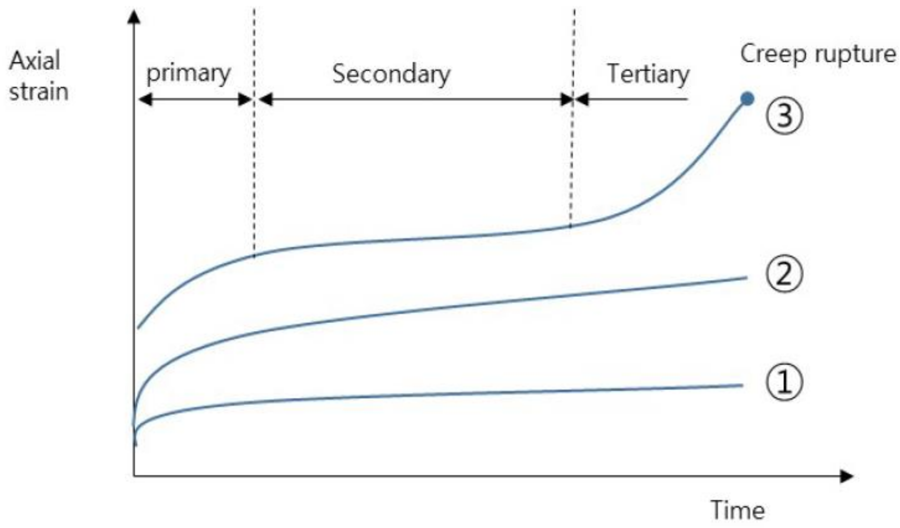

Figure. 2.3 Creep under constant stress(Mitchell and Soga, 2005)

2.3 Application and limitations of literature review in this experiment

Various experiments, such as the triaxial test, were conducted to analyze the long-term behavior of sandy soil. Referring to the research results of Park (2015), the experiment was planned by selecting the influencing factors that most affect the creep phenomenon, such as stress ratio, relative compaction, and fine contents. Other researchers mainly applied the variable to increase the confining pressure from the variable for the stress ratio. However, in this study, to simulate the stress ratio of the embankment, each experiment`s confining pressure was conducted with state values according to the stress ratio of the experiment.

The limitation of this study is that in the case of an actual embankment, the stress ratio changes from the top to the bottom, but there is a limit to reproducing this with a triaxial equipment, so the experiment is conducted with a state stress ratio. In addition, in the case of an embankment that is actually constructed on the upper part of the embankment after about 3 months of leaving period, so to reproduce the exact amount of creep settlement in an triaxial experiment, it is necessary to leave a period of 3 months after the implementation of the experiment. There is a limitation in leaving the sample for a period of time. Even in the case of compaction, the actual embankment is compacted using heavy equipment, such as water compaction, whereas in an indoor experiment, it is compacted by manpower, so there is a limit in simulating the actual site.

Chapter 3. Test Materials and Apparatus

For research creep behavior in sand material, triaxial tests were performed. A testing series was designed to illustrate the stress conditions of the fill materials of an embankment. Different relative densities of the specimens crept under different stress ratios and fine contents. With local LVDT, deformations under constant loading were continuously obtained.

3.1 Testing Materials

Weathered soil was sampled near the Seoul area, Korea. The grain size distribution curve and properties of sampled soil are shown in the figure.

Soil is classified as well-graded silty sand (SM) and contains 15% fines. In the case of 25% and 35% samples, fine particles were collected by wet sieving the original 15% weathered soil, and then fine particles were added to the existing 15% weathered soil to form fine particles of 25% and 35% samples. The 25% and 35% samples made by adding fine particles are also classified as silty sand(SM) in the soil classification. Grain distribution data for each fine powder was organized, as shown in the following graph.

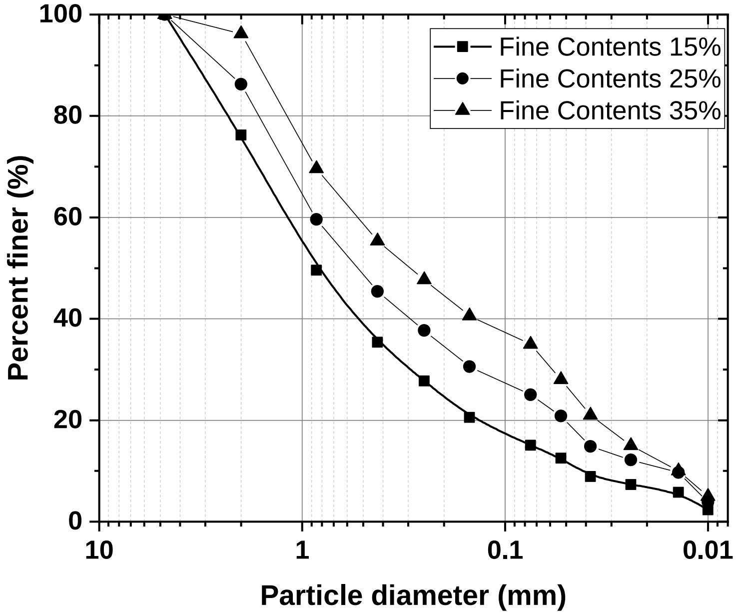

Figure. 3.1 Particle size distribution of tested material

For the triaxial tests, particles larger than the opening size of No.4 sieve, i.e., 4.75 mm, were removed from the original samples. 71.85mm-diameter and 135mm-high triaxial specimens were prepared by compacting sieved soils mixed with water to a moisture content of 11%. To consider the variation in the field density, three different groups of the specimens 83, 93, and 98% were prepared.

The dry unit weight and water content were obtained using the modified compaction test D method using samples for each fine content. When setting the relative compaction, a sample was prepared with the relative compaction of the test conditions using the dry unit weight obtained in the above test. The detailed material properties of the sample are shown in the following table.

Table. 3.1 Physical properties of tested materials

Tested material	G_s	Fine Contents (%)	Dry unit weight (kN/m^3)	Water content (%)
Korean Weathered Residual Soil	2.63	15	19.04	8.74
		25	19.18	9.92
		35	19.61	9.67

3.2 Testing apparatus

For triaxial creep test, using GDS triaxial testing equipment consists of the loading frame, 8-channel data logger for LVDT on cell, and controller to control cell and back pressures. The figure shows the triaxial testing system and specimen.

The creep compression test was performed in the steps of specimen composition, embankment stress ratio simulation, and vertical displacement measurement. According to the testing program shown in 3.3, a specimen (Diameter : Height = 2: 1) was formed by compacting residual soil samples in multiple layers, and the triaxial compression test of GDS, which can simulate anisotropic stress conditions and stress paths, and measure micro-strain equipment was used. The vertical and horizontal stresses were gradually loaded to reach the target stress ratio for the initial 30 minutes so that the specimens were not destroyed. Then, the vertical displacement of the specimen was measured using a LVDT(Linear Variable Differential Transformer) while maintaining the stress state.

Figure. 3.2 Triaxial testing system and specimen

3.3 Testing program

For the anisotropic condition, considering the embankment shape, three stress ratios were chosen, q/p’, of 0.6, 0.8, 0.98, where p’ = ($\sigma'_a + 2\sigma'_r$) / 3 is mean normal effective stress, q = $\sigma'_a - \sigma'_r$ is the deviator stress. σ'_a and σ'_r are axial and radial effective stresses, respectively.

In this research, three variables were selected for the selection of influencing factors: relative compaction, fine contents, and stress ratio. The detailed value selection background for that is as follows.

Table. 3.2 Test program of triaxial creep test

<table>
<tr><th></th><th>Value</th><th>Index</th></tr>
<tr><td rowspan="3">Relative Compaction (%)</td><td>98</td><td>The upper roadbed embankment compaction criteria 95%</td></tr>
<tr><td>93</td><td>The lower roadbed embankment compaction criteria 90%</td></tr>
<tr><td>86</td><td>Below than compaction criteria</td></tr>
<tr><td rowspan="3">Fine Contents (%)</td><td>15</td><td>Upper limit of JPN filling materials</td></tr>
<tr><td>25</td><td>High speed railway filling materials criteria</td></tr>
<tr><td>35</td><td>Below than filling materials criteria</td></tr>
<tr><td rowspan="3">Stress Ratio(q/p’)</td><td>0.6</td><td rowspan="3">Stress ratio of roadbed of embankment</td></tr>
<tr><td>0.8</td></tr>
<tr><td>0.98</td></tr>
</table>

The test conditions were determined by fine contents, relative compaction, and stress ratio (q/p') of the embankment material as factors influencing the creep settlement of the embankment in the Table. The fine contents were determined by referring to domestic and international design standards (Ministry of Construction and Transportation, 2004, Ministry of Land, Infrastructure and Transport, 2007).

In the case of relative compaction, 98% satisfying the upper roadbed compaction standard of 95%; 93% satisfying the lower roadbed compaction standard 90%; and 86% dissatisfying the embankment compaction standard were selected as variables.

In the case of fine contents, 15%, which is the upper limit of the fine content of Japanese embankment material, 25% that satisfies the fine content standard of high-speed rail embankment, and 35% that does not meet the fine content standard of embankment material, were selected as a variable.

Based on the results of numerical analysis, the two conditions representing the internal stress state of the embankment was determined. Assuming that the unit weight density of soil is 2 t/m^3, the stress conditions of the elements at a depth of 10m from the ground surface were illustrated. It was kept at σ'_v = 100kPa. Due to the nature of the triaxial test, the confining pressure also affects the vertical stress. The confining pressure was reduced to increase the stress ratio, and the vertical stress applied from the rod was increased to increase the q/p' value. The q/p' value was made smaller by reducing the normal stress applied.

Table. 3.3 Detailed experimental conditions for stress ratio

No.	Stress ratio(q/p’)	$\sigma_1(\sigma_v)$(kPa)	$\sigma_3(\sigma_r)$(kPa)	q (kPa)	p' (kPa)	K($\frac{\sigma_3}{\sigma_1}$)
1	0.6	100	57.1	42.9	71.4	0.57
2	0.8	100	48	52	65.3	0.48
3	0.98	100	41	59	60.7	0.41

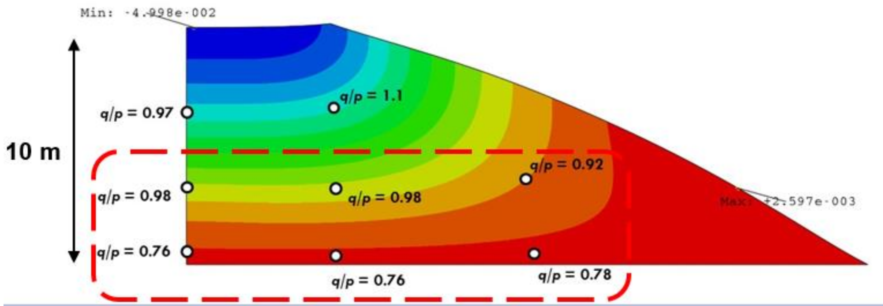

Figure. 3.3 Stress ratio(q/p’) in embankment

Experimental cases were selected based on the variables selected as above. In order to apply three different values for a total of three variables, the lower roadbed compaction standard of the embankment body is satisfied, and the fine powder content was determined based on the fine content of 15% and the relative compaction of 93%, which satisfies the minimum standard of the Japanese standard. In this case, the standard was set as q/p’ = 0.8, which represents the median value in the distribution of q/p' values at the center of the embankment, and experiments were conducted for a total of 7 cases.

Table. 3.4 Test program of triaxial creep test

Test Number	Name	Fine Contents (%)	Relative Compaction (%)	Stress Ratio(%) q/p'(σ'_h/σ'_v)
1	SM0.8_F15_C98	15	98	0.8(0.48)
2	SM0.8_F15_C93		93	
3	SM0.8_F15_C86		86	
4	SM0.8_F25_C93	25	93	
5	SM0.8_F35_C93	35		
6	SM0.6_F15_C93	15		0.6(0.57)
7	SM0.98_F15_C93			0.98(0.41)

Chapter 4. Experimental results and Analysis

4.1 Creep strains of triaxial compression tests

The vertical strain evaluated through the creep compression test is shown in Figures 2.7 and 2.8. The experiment was performed until a graph appeared in a log-linear shape. Each experimental case's measurement period was measured from a minimum of 5 days to a maximum of 50 days.

Most of the experimental results showed a linear relationship between axial strain and time(log) after one day, and between 1 and 10 days in the case of samples with fine contents of 25% and 35%, and stress ratio of 0.98. There was a more linear relationship between axial strain and time(log) past ten days than before.

There was a slight difference in the sample with 15% fine contents depending on the degree of compaction, but the vertical strain rate that occurred after about ten days decreased sharply. Even when the fine contents of 25%, the vertical strain did not converge during the measurement period and increased continuously. In the case of the sample with 35%, a relatively very large vertical strain occurred compared to other samples.

When the fine contents were 15%, and relative compaction was 93%, the axial strain occurred significantly small the q/p’=0.98 condition compared to the q/p’=0.8 condition.

And axial strain increased without convergence even after ten days. As the stress ratio decreased to 0.6, the axial strain rate also decreased.

Although 35% of the fine-grain is an extreme case that does not meet the standards for the composition of the embankment, it can be seen that the actual application of the embankment is limited by confirming the occurrence of a large strain.

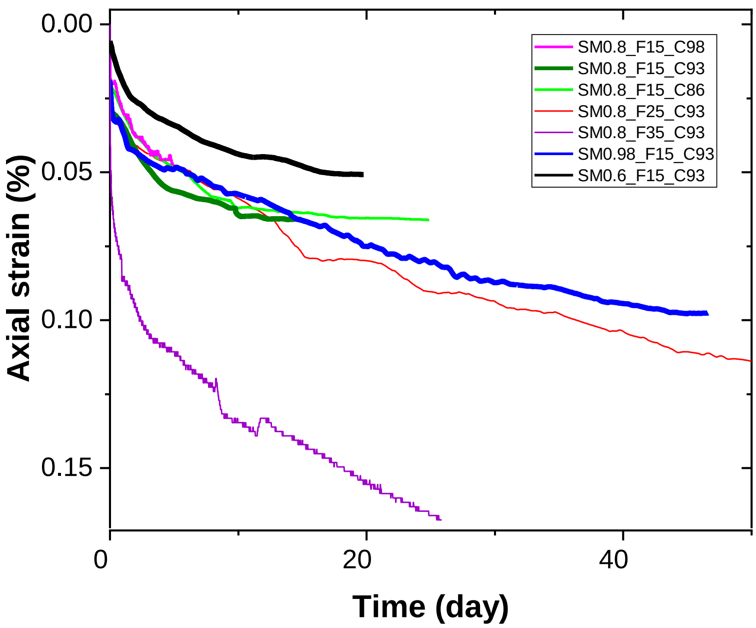

Figure. 4.1 Axial strain curves with linear time(total)

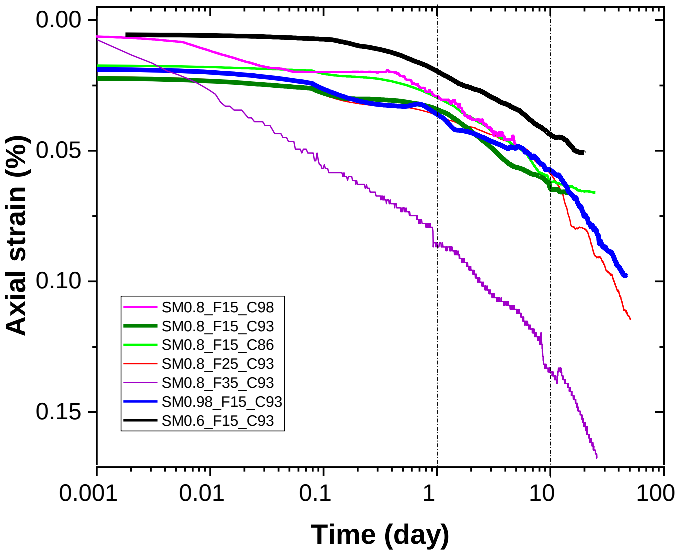

Figure. 4.2 Axial strain curves with log time(total)

4.2 Creep parameters

The creep coefficient α proposed by Charles(2008) was determined by regression analysis of the linear section.

$$S = \alpha H log_{10}\frac{t}{t_0} + S_0 \qquad (2)$$

In this equation, S is the settlement, H is the height, t is the time, α is the creep coefficient.

In this experiment, the assumptions about the creeping section are as follows. The factors that cause the settlement of soil can be divided into three in this experiment: 1) The instant settlement that occurs at the moment of loading; 2) the consolidation settlement that occurs as the excess pore water pressure is dissipated; 3) and the creep settlement that occurs over time under certain stress.

In this experiment, to minimize the effect on the immediate settlement phenomenon, constraining pressure and axial weight were applied over 30 minutes when setting the initial test conditions. Since this experiment is conducted from an unsaturated and drainage state, it is assumed that there is no consolidation phenomenon caused by the dissipation of excess pore water pressure. That is, all settlements occurring in the experiment were assumed to be creep settlement, and the measured data were applied to the logarithmic model to obtain the creep parameters.

As the degree of compaction increased, the vertical strain decreased, and the creep coefficient decreased. In the sample with 25% fine contents, a larger vertical strain occurred compared to the 15% fine contents, and the creep coefficient increased about 2.6 times. In the 35% fine content, the creep coefficient increased almost three times than the 15% fine content. In the shear stress ratio, the vertical strain occurred larger in the q/p’=0.98 condition than in the q/p’=0.8 condition, and the creep coefficient increased by about 2.3 times. And q/p’=0.6 condition decreased 0.89 times greater than q/p’=0.8. In q/p’=0.73, the axial creep strain rate was smaller than q/p’=0.8, but the parameter value was greater than q/p’=0.8. This was considered to be an error that could occur because the difference in stress condition between the two cases was small. Case q/p’=0.73 was omitted from the procedure for calculating the formula that follows.

Using the creep coefficient, α, the settlement of the 10m height embankment was predicted, as shown in Table 4.1. Through this, when evaluating the occurrence amount of the embankment settlement over time, the creep settlement during the initial three months occurred over 50% of the creep settlement after ten years, which is judged that most of the creep settlement occurs within the initial three months.

Table. 4.1 Creep parameters for log model

Index	α	R^2	3 months		10 years		3months ~ 10 years	
			Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)
SM0.8_F15_C98	0.02674	0.965	0.0523	5.23	0.0953	9.53	0.0430	4.30
SM0.8_F15_C93	0.02868	0.980	0.0560	5.60	0.1022	10.22	0.0461	4.61
SM0.8_F15_C86	0.02897	0.956	0.0566	5.66	0.1032	10.32	0.0466	4.66
SM0.8_F23_C93	0.07516	0.950	0.1469	14.69	0.2677	26.77	0.1209	12.09
SM0.98_F15_C93	0.06527	0.920	0.1276	12.76	0.2325	23.25	0.1050	10.50
SM0.8_F35_C93	0.08677	0.948	0.1696	16.96	0.3091	30.91	0.1395	13.95
SM0.6_F15_C93	0.02555	0.990	0.0499	4.99	0.0910	9.10	0.0411	4.11

4.2.1 Series for stress ratio

Detailed experimental results for each variable were summarized. First, the experimental conditions using the stress ratio as a variable are shown in the following table. After preparing a sample with 93% relative compaction and 15% fine contents, the experiment was conducted by setting a variable for the stress ratio (q/p’) when setting the triaxial tester test conditions. The values summarized for the test conditions are shown in the following table.

Experimental cases as stress ratio variables are shown in figure. From q/p’ = 0.6, to 0.98, three cases conducted. Plot without presuming the initial constant time interval as the stabilization period. When the fine contents were 15%, and relative compaction was 93%, the axial strain occurred significantly under the q/p’=0.98 condition compared to the q/p’=0.8 condition and increased without convergence even after ten days. As the stress ratio decreased to 0.6, the axial strain rate also decreased.

Table. 4.2 Test program for stress ratio

Cr(%) (Relative compaction)	Fc(%)	q/p’
93	15	0.6
		0.8
		0.98

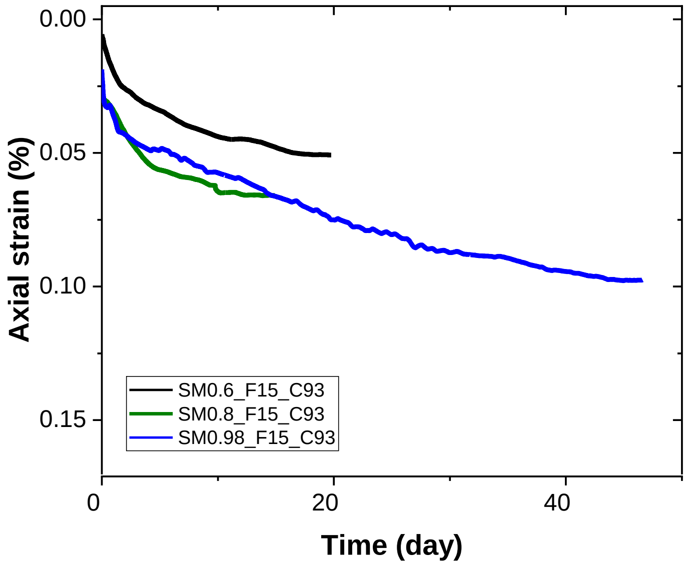

Figure. 4.3 Axial strain curves with linear time

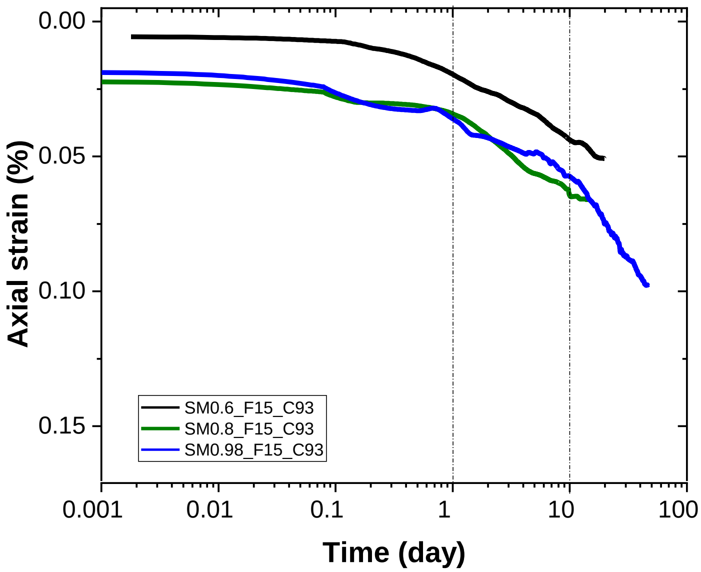

Figure. 4.4 Axial strain curves with log time

As a result of the experiment, q/p’ = 0.6 and 0.8 showed a linear relationship with time(log) after one day converged to a constant value after 10 and 15 days, and q/p’ = 0.98. A strain lower than q/p’ = 0.8 occurred in the initial behavior, but continuous strain occurred over time. The relationship was more linear after ten days than on the 1st to 10th, and the transformation continued to occur after a relatively long period of 20 days or more. When calculating the creep parameter, to remove the error in the occurrence of settlement immediately after loading and use the section and use the section in which a linear relationship appears for each experiment. In the case of q/p’ = 0.6 and 0.8, the data after one day of loading are used. The parameter was calculated, and in the case of q/p’ = 0.98, the parameter was calculated using data after ten days. Each parameter's values and the predicted value of settlement by period based on 10m of the embankment are as follows.

Table. 4.3 Creep parameters for log model(stress ratio)

Index	α	R^2	3 months		10 years		3months ~ 10 years	
			Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)
SM0.6_F15_C93	0.02555	0.990	0.0499	4.99	0.0910	9.10	0.0411	4.11
SM0.8_F15_C93	0.02868	0.980	0.0560	5.60	0.1022	10.22	0.0461	4.61
SM0.98_F15_C93	0.06527	0.920	0.1276	12.76	0.2325	23.25	0.1050	10.50

Under constant conditions of 93% relative compaction and 15% contents, the creep parameter increases 1.33 times and 1.63 times based = 0.6 to 0.8, 0.98. For the stress ratio of q/p' = 0.6, the creep parameter increased 1.12 times and 2.55 times. When analyzing the predicted using the creep parameter, settlement occurs similarly to the creep settlement management standard of 10mm at the stress ratio 0.6 and 0.8 at the elapse of 10 years. In contrast, the settlement of 23.25mm is predicted at the stress ratio of 0.98, and thus, it can be noticed that it far exceeds the settlement management standard.

4.2.2 Series for relative compaction

The experimental conditions using relative compaction as a variable are shown in the following table. A sample of 15% fine contents was divided into several layers with relative compaction of 86%, 93%, and 98% in a test mold to form a composition. When the test conditions were set in the triaxial tester, the experiment was performed by constantly setting q/p' = 0.8. The values summarized for the test conditions are shown in the following table.

Table. 4.4 Test program for relative compaction

Cr(%)	Fc(%)	q/p’
98	15	0.8
93		
86		

Experimental cases as relative compaction variables are shown in the figure. From Cr = 86% until 98%, three cases were conducted. In the sample with 15% fine contents, there was a slight difference depending on the degree of compaction, but the vertical strain rate that occurred after about 10 days decreased sharply.

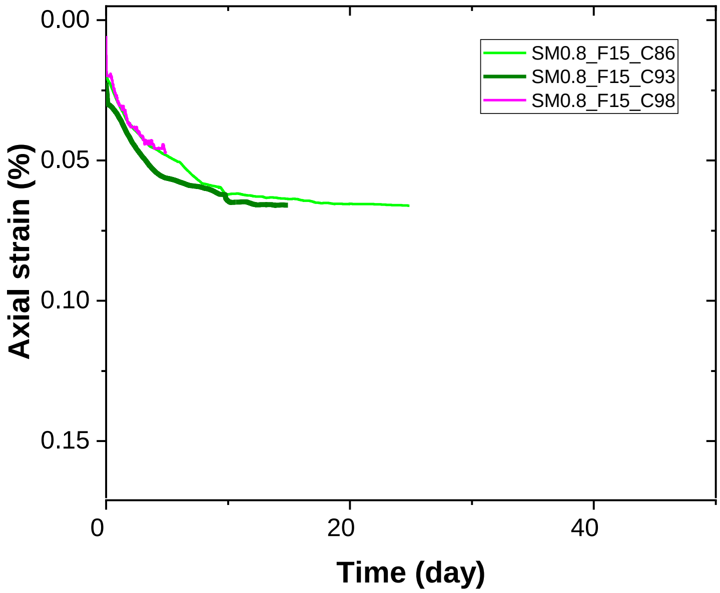

Figure. 4.5 Axial strain curves with linear time

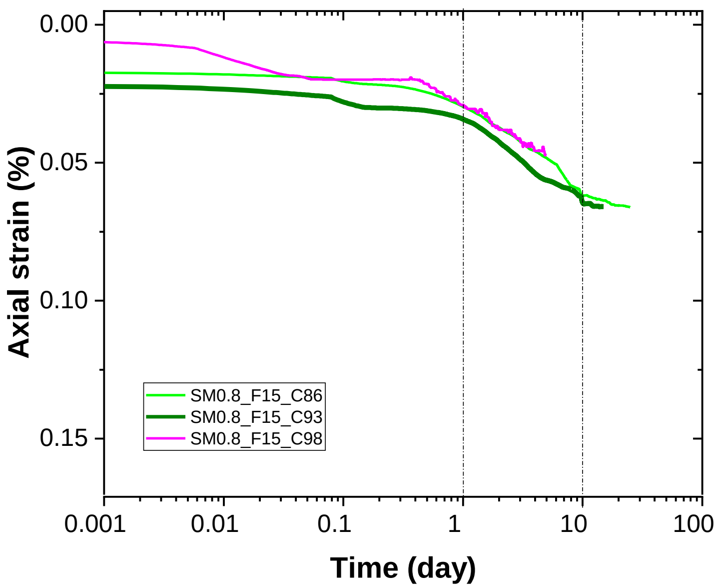

Figure. 4.6 Axial strain curves with log time

As a result of the experiment, Cr = 86, 93, and 98% all showed a linear relationship with time (log) after 1 day and converged to a constant value after 10 days. In the Cr = 86%, the strain initially occurred lower than Cr = 93%, but as time elapsed, the strain converged to a strain greater than Cr = 93%, and the creep parameter showed a larger value. When calculating the creep parameters, the parameters were calculated using the data from the 1st day to the end of each test to remove the error in the occurrence of settlements immediately after loading and to evaluate the creep settlement. Each parameter's values and the predicted value of settlement by period based on 10m of embankment are as follows.

Table. 4.5 Creep parameters for log model(relative compaction)

Index	α	R^2	3 months		10 years		3months ~ 10 years	
			Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)
SM0.8_F15_C98	0.02674	0.965	0.0523	5.23	0.0953	9.53	0.0430	4.30
SM0.8_F15_C93	0.02868	0.980	0.0560	5.60	0.1022	10.22	0.0461	4.61
SM0.8_F15_C86	0.02897	0.956	0.0566	5.66	0.1032	10.32	0.0466	4.66

Under constant conditions of fine contents of 15% and q/p'=0.8, when the relative compaction increases from Cr = 86% to Cr = 93%, 98% (1.08 times and 1.14 times based on Cr = 86%) creep parameter α decreased to 0.98 times and 0.92 times. Also, when analyzing the predicted settlement using the creep parameter, after 10 years, settlement occurs below 10mm in the criterion for creep settlement management in Cr = 98%. In Cr = 86% and 93% approximately 10mm of settlement occur.

4.2.3 Series for fine contents

The experimental conditions using fine contents as a variable are shown in the following table. After preparing a sample of 15% fine powder, which is a raw material. 25% and 35% of fine powder were formed by adding fine particles to the sample. The samples were prepared with relative compaction of 93% and then set to test conditions of q/p' = 0.8 in a triaxial tester. The experiment was conducted by setting it to constant. The values summarized for the test conditions are shown in the following table.

Table. 4.6 Test program for fine contents

Cr(%)	Fc(%)	q/p’
93	15	0.8
	25	
	35	

Experimental cases as fine contents variables are shown in figure. From Fc = 15% to 35%, three cases conducted. The fine contents of 25% satisfies the lower embankment compaction criterion (90% relative compaction degree). The vertical strain did not converge during the measurement period and increased continuously. In the case of the sample with 35%, a considerable vertical strain occurred than other samples. Although 35% of the fine-grain is an extreme case that does not meet the standards for the composition of the embankment, it can be seen that the actual application of the embankment is limited by confirming the occurrence of a large strain.

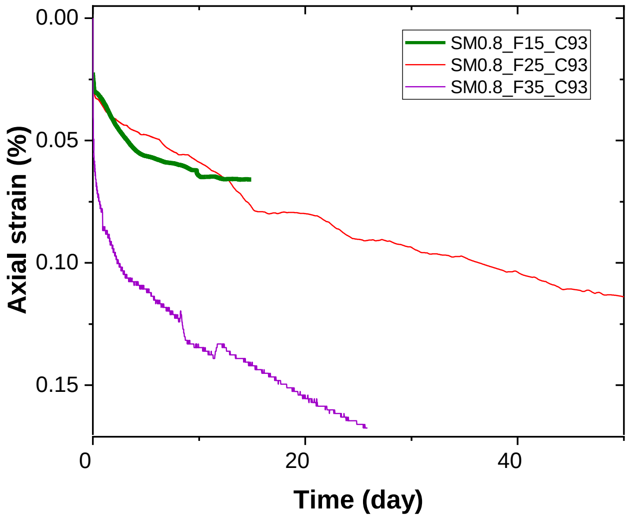

Figure. 4.7 Axial strain curves with linear time

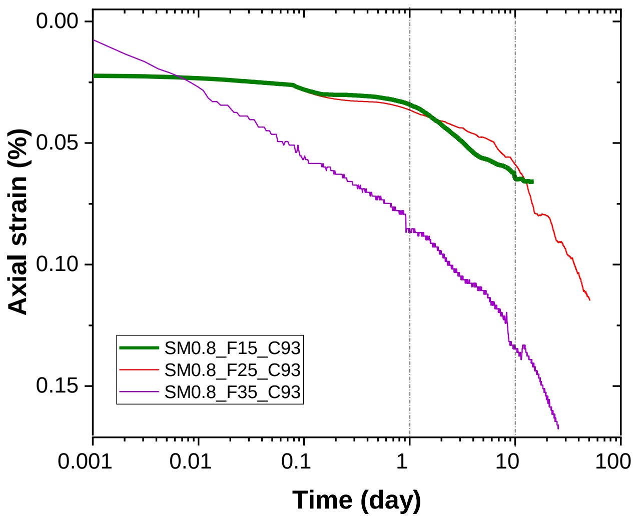

Figure. 4.8 Axial strain curves with log time

As a result of the experiment, Fc = 15% showed a linear relationship with time (log) after 1 day and converged to a constant value after 10 days. In the case of Fc = 25%, the strain rate lower than F=15% initially occurred, but as time passed, the strain continued to occur, resulting in a strain rate greater than F=15%. It showed a linear relationship with time (log) after work. In the case of F=35%, the strain was greater than that of F=15% and 25% from the initial settling amount of the sample, and the strain continued to be large even after 10 days. When calculating the creep parameters, to remove the error about the occurrence of settlement immediately after loading and evaluate the amount of creep settlement, in the case of F = 15%, the parameter was calculated using data from 1 day to the end of each test, and for F = 25% and F = 35%, the parameters were calculated using data after 10 days. Each parameter's values and the predicted value of settlement by period based on 10m of the embankment are as follows.

Table. 4.7 Creep parameters for log model(fine contents)

Index	α	R^2	3 months		10 years		3months ~ 10 years	
			Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)	Strain (%)	Settlement (mm)
SM0.8_F15_C93	0.02868	0.980	0.0560	5.60	0.1022	10.22	0.0461	4.61
SM0.8_F23_C93	0.07516	0.950	0.1469	14.69	0.2677	26.77	0.1209	12.09
SM0.8_F35_C93	0.08677	0.948	0.1696	16.96	0.3091	30.91	0.1395	13.95

Under constant conditions of 93% relative compaction and stress ratio (q/p') = 0.8, when the creep parameter increases 1.67 times and 2.33 times from F=15% to F=25% and 35%, the creep parameter α is increased by 2.62 times to 3.03 times. When analyzing the predicted settlement using the creep parameter, after 10 years, settlement occurs similarly to the creep settlement management standard of 10mm at F=15%, while settlements of 26.77mm and 30.91mm are observed at F=25% and 35%. It can be predicted that the settlement management standard is far exceeded.

4.2.4 Evaluation of the influence of each factor

Synthesized the three influencing factors analyzed so far. As for the relative compaction degree, as the degree of compaction increases, the creep parameter decreases constantly, and as for the stress ratio and the amount of fine particles, the creep parameter value increases as the value increases. Considering the boundaries of the creep parameter change due to the increase of the influencing factors, the factor that affects the creep parameter the most can be the amount of fine contents.

Also, looking at the creep settlement prediction results, it can be seen that more than 50% settlement occurs in the first 3 months based on a total period of 10 years. Considering this, it is considered important to secure a sufficient neglect period of at least 3 months after compaction construction using high-quality embankment materials to prevent the fixed settlement of the embankment.

4.2.5 Creep model analysis

Using the previously evaluated creep parameters, this study tried to find a relational expression that can predict the value of the creep parameter when an arbitrary condition is given. The model used in the relational equation utilized the previously used log model. The relation expression was calculated using the log model equation used as the existing creep analysis model.

$$S = \alpha H log_{10}\frac{t}{t_0} + S_0 \qquad (2)$$

In this equation, S is the settlement, H is the height, t is the time, α is rate of settlement that occurs over 1 to 10 years. Find the parameter according to the variables of stress ratio, relative compaction and find contents. And formula calculation through trend line according to each parameter as (2).

First, we estimated the relational expression using the stress ratio as a variable. The range of the stress ratio, the influence factor set in the previous experiment, and the distribution of the creep parameter α value accordingly are as follows.

Table. 4.8 Creep parameters for log model (stress ratio)

SM	α
0.6	0.02555
0.8	0.02868
0.98	0.06527

The trend line was determined in exponential function using the relationship between the stress ratio and the creep parameter α. As the stress ratio increases, the creep parameter α value increases, and an exponential function was selected as a function that best predicts this.

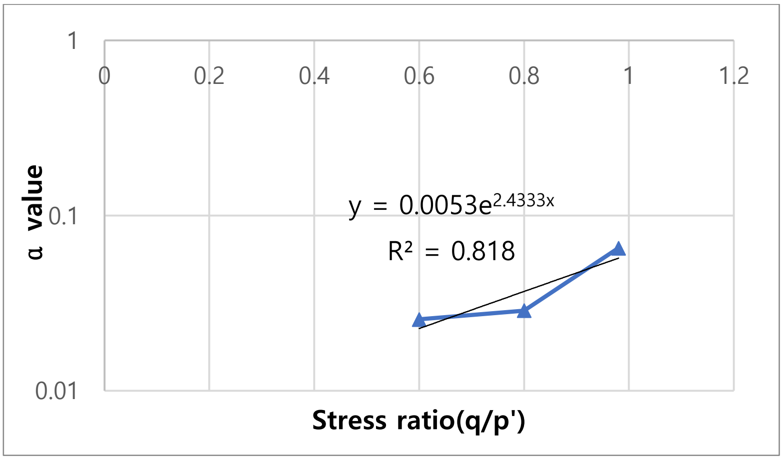

Figure. 4.9 α value graph with stress ratio(q/p')

Using the relational expression in the form of an exponential function, the relation of the creep parameter α with the stress ratio as a variable was determined as follows. In this equation, y is the creep strain, μ is the stress ratio, and t is the time.

$$y = 0.0053 \times \exp(2.4333\mu) \log t \qquad (3)$$

Next, tried to estimate the relational expression using the degree of relative resolution as a variable. The range of the relative compaction degree, which is the influence factor set in the previous experiment, and the distribution of the creep parameter α value according to it are as follows.

Table. 4.9 Creep parameters for log model (relative compaction)

Cr(%)	α
86	0.02674
93	0.02868
98	0.02897

The trend line was determined in the form of an exponential function using the relationship between the relative compaction degree and the creep parameter α. As the relative compaction increases, the creep parameter α decreases, and an exponential function was selected as a predictable function.

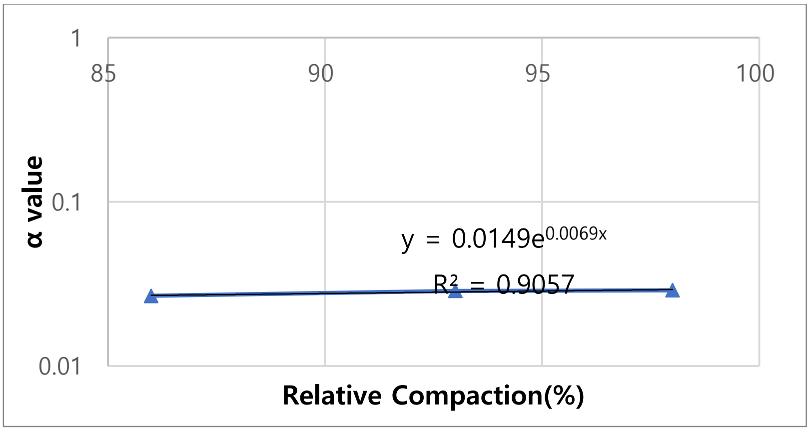

Figure. 4.10 α value graph with relative compaction

Using the relational expression in the form of an exponential function, the relational expression of the creep parameter α with relative compaction as a variable was determined as follows. In this equation, y is the creep strain, C is the relative compaction, and t is the time.

$$y = 0.0149 \times \exp(0.0069C) \log t \qquad (4)$$

Finally, we estimated the relational expression using the amount of fine powder as a variable. The distribution of the influencing factor, fine powder content, and the creep parameter α value set in the previous experiment is as follows.

Table. 4.10 Creep parameters for log model (fine contents)

Fc(%)	α
15	0.02868
23	0.07516
35	0.08677

The trend line was determined in exponential function using the relationship between the amount of fine particles and the relative compaction degree, and the creep parameter α. As the amount of fine particles increases, the creep parameter α value also increases, and an exponential function was selected as a predictable function.

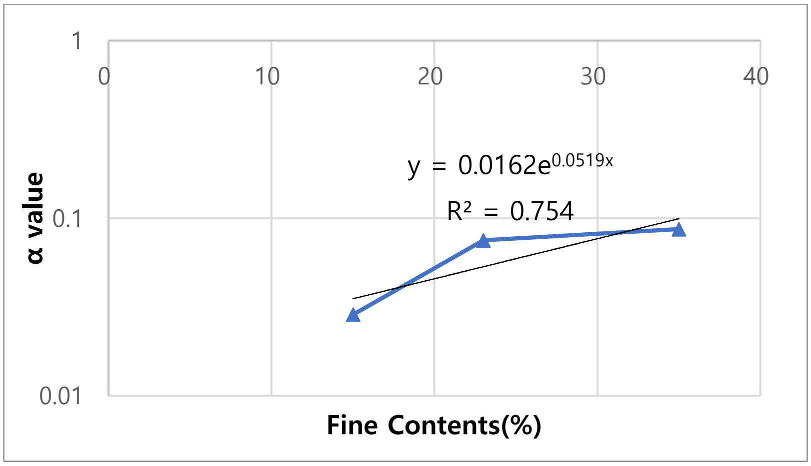

Figure. 4.11 α value graph with fine contents

The creep parameter α relation with the amount of fine particles as a variable was determined using the relational expression in exponential function as follows. In this equation, y is the creep strain, F is the amount of fine particles contained, and t is the time.

$$y = 0.0162 \times \exp(0.0519F) \log t \qquad (5)$$

Using the relationship between the creep parameter and the creep influencing factors such as the stress ratio, the relative compaction, and the amount of fine particles, the material properties of the sandy soil and the stress ratio distribution value according to the shape of the embankment are calculated for the embankment composed of the actual sandy soil. It can be used to estimate the creep strain over time.

4.2.6 Comparison with field data

Using the relationship between creep parameters and influencing factors obtained in the previous section, we applied it to actual field data to check the applicability of this relationship. The target site for application is the embankment for the formation of the roadbed of the high-speed railroad, and the primary embankment material for the embankment is composed of sandy soil. Among the site locations, two sites were selected that have data of settlement measurement for three years from 600 days after the opening of the high-speed railroad, and know the characteristics of the ground obtained through the site survey. These areas are the points where settlements have occurred well above 30mm, which is the standard for high-speed rail residual settlements during the measurement period. At these points, the settlement amount is calculated using the actual field measurement data, and the settlement of each floor is calculated using the distribution of the stress ratio considering the shape of the embankment and the amount of fine particles, and the relative compaction obtained through the site survey. I compared the values.

4.2.6.1 Site A

The first site A was an 8m-high embankment, and a total of 78mm of settlement occurred during the measurement period. From this settlement, it was judged that 70% of settlement occurred in the embankment, and the actual settlement value of the embankment was set to 55mm. Through the on-site geotechnical survey, in the schematic diagram as shown in the following figure, the amount of fine-grain content and the relative compaction value from the 2nd to 6th floors in the survey target embankment were applied. In the case of, it was calculated by applying the value of the fine-grain content of the embankment material used in the design and the 95% minimum compaction standard of the upper roadbed. In the case of the stress ratio of the embankment, the stress ratio value was determined for each location by referring to the embankment simulation data used to determine the experimental variables in the indoor experiment. The stress ratio values, relative compaction, and granular content of each floor are substituted into the relational formula. The formula was obtained through an indoor experiment to determine the average of the parameter values, the height of each floor, and the actual embankment for calculating the settlement amount through the log model. By applying the measurement period value, the amount of settlement for each floor, and the total amount of possible settlement were calculated, and the values are as follows.

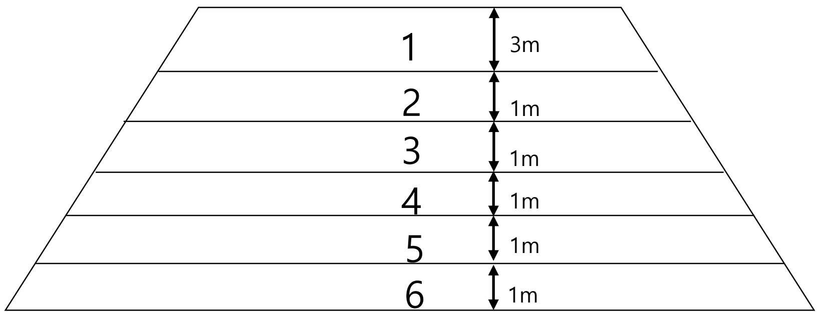

Figure. 4.12 Schematic diagram of embankment A

Table. 4.11 Creep settlement for embankment A

No.	Length	Relative compaction(%)	Stress ratio (q/p')	Fine contents(%)	α	Settlement (mm)
1	3	95	0.97	24	0.0470	4.73
2	1	90	0.97	28	0.0510	1.71
3	1	90	0.98	32	0.0568	1.90
4	1	90	0.98	26	0.0492	1.65
5	1	90	0.98	42	0.0762	2.55
6	1	80	0.76	48	0.0850	2.85
Total settlement						15.38

4.2.6.2. Site B

The second site B is a 9.4m-high embankment, and a total of 106mm of settlement occurred during the measurement period. It was determined that 70% of the settlement occurred in the embankment, and the actual settlement value of the embankment was set to 74mm. Through the on-site geotechnical survey, , the amount of fine content and the relative compaction value from the 2nd to 7th floors were applied as shown in the following schematic diagram. It was calculated by applying the value of the fine-grain content of the embankment material used in the design and the 95% minimum compaction standard of the upper roadbed. In the case of the embankment stress ratio, the stress ratio value was determined for each location by referring to the embankment simulation data used to determine the experimental variables in the indoor experiment. The values of the stress ratio, relative compaction, and granular content of each floor are substituted into the relational formula obtained through an indoor experiment to determine the average parameter values. The parameter values, height of each floor, and actual embankment were considered for calculating the settlement amount through log model. By applying the measurement period value, the amount of settlement for each floor and the total amount of possible settlement were calculated, and the values are as follows.

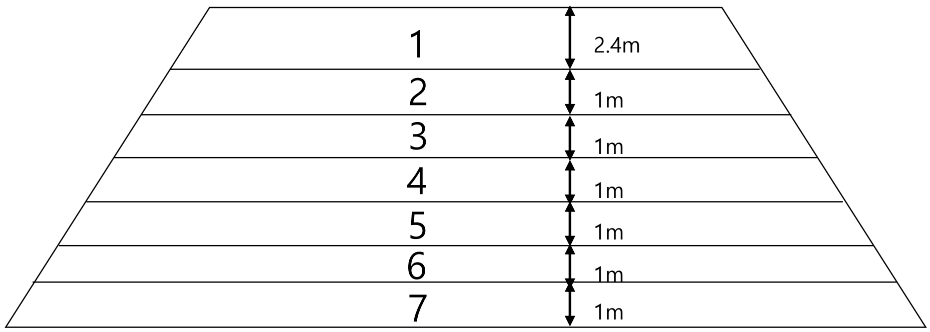

Figure. 4.13 Schematic diagram of embankment B

Table. 4.12 Creep settlement for embankment B

No.	Length	Relative compaction(%)	Stress ratio (q/p′)	Fine contents(%)	α	Settlement (mm)
1	2.4	95	0.97	24	0.0470	3.78
2	1	80	0.97	20	0.0426	1.43
3	1	80	0.97	49	0.0960	3.21
4	1	80	0.98	51	0.1040	3.48
5	1	80	0.98	10	0.0369	1.23
6	1	90	0.98	21	0.0445	1.49
7	1	70	0.98	49	0.0959	3.21
8	1	80	0.76	44	0.0728	2.44
Total settlement						20.27

Table. 4.13 Comparison with field data

	#50 A	#51 B
Height of embankment	8m	9.4m
Settlement of embankment	54.6mm	74.2mm
Settlement from lab test	15.4mm	20.3mm

The predicted values of settlement for each site are summarized in the table above. The amount of fine-grained soil obtained through geotechnical surveys at the site was measured in various ways, from 10% to 51% by floor. The values exceeding 25%, which is the standard for high-speed rail, were detected in many sections. In addition, the height of the embankment is 8m to 9.4m, and it is distributed adjacent to 10m, which is the classification standard of the old embankment. It means that a large amount of settlement can occur due to the height of the embankment itself even at the typical creep deformation rate. The amount of settlement of the embankment was obtained by comparing the measurement data of the upper part of the embankment with a settling pin and the settling plate installed on the original ground. Since the settlement measured at the top of the embankment represents the sum of the settlement of the original ground and the embankment, the measurement data of the settlement plate installed on the original ground was considered together. It was confirmed that the 70% of the measured settlement is due to the embankment.

This finding was adopted for the calculations of the creep parameter of the embankment. The settlement obtained by field measurement was calculated as 3.55 times bigger in the case of A site and 3.66 times bigger in the case of B site, and the settlement using the relational equation obtained through an indoor experiment was applied. The reason is that there is a large difference in the time between the indoor experiment and the measurement data. The experimental period of the indoor experiment is short, from 5 to 50 days, while the actual field measurement data differs in that the data from about 600 days to 2320 days after the opening of the high-speed rail was applied. It is judged that this difference occurred because the indoor experiment was performed under constant conditions under which variables were controlled, and various unpredictable variables were additionally applied to the field measurement data. When referring to these results, to predict the creep settlement of the embankment more effectively, it is necessary to add a factor to derive a relational expression and add an experiment based on that factor. Representative factors include index D10, which indicates pore water pressure and particle size distribution distributed in the ground.

Chapter 5. Conclusion and further study

5.1 Conclusion and further study

The main objectives of this research were to evaluate the behavior of the creep deformation occurring in the residual soil considering various influence factors such as the relative compaction, stress ratio, and fine content. Also, the correlation among those factors was analyzed, and the dominant factor was therefore identified. In order to achieve these goals, a series of triaxial creep tests were performed on a number of constituted specimens under drained conditions. The vertical deformation of the specimen was measured to determine the creep strain.

The results showed that the creep strains decreased moderately by the relative compaction but increased significantly by the fine content and stress ratio. It was found that the fine content had a dominant effect on the creep strain.

The settlement was compared to evaluate the applicability of the real site. The settlement obtain by field measurement was calculated bigger than the settlement using the relational equation obtained through triaxial creep experiment. The difference in the predicted value occurs because the experiment execution period is much smaller than the site measurement period, and during the experiment, other variables are controlled than the site.

It is highly suggested to use high-quality embankment materials such as low fine contents and high relative compaction to prevent severe long-term creep deformation.

Further research is recommended to supplement this research. It is necessary to measure the radial strain for the horizontal deformation of specimens. Another key consideration is to account for the effect of the pore water pressure, which can occur in the actual field. And increase the duration of the experiment and conduct an experiment on additional variables in order to obtain more realistic measured vales can be effective method to further study.

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초 록

크리프 현상은 일정한 유효 응력 하에서 발생하는 시간에 따른 변형 현상이다. 각종 구조물에서 장기간에 발생하는 심각한 크리프 변형은 구조물의 기능 수행 문제를 유발할 수 있다. 국내에서 성토체는 주로 사질토로 구성되어 있으며 철도, 도로 등 주요 기반시설의 설계 지반고를 유지하기 위해 시공된다. 성토체는 침하관리가 중요하며, 특히, 엄격한 허용 잔류 침하량을 기준으로 관리하는 경우 장기 크리프 거동으로 인한 침하량 평가가 중요하다. 크리프 변형에 대한 사전 연구를 통해 통해 장기적인 구조 변형 문제등을 예방할 수 있다. 초기 연구자들은 지반의 크리프 변형에 대한 민감성을 평가하기 위해 댐 등의 제방에서 직접 크리프 변형에 대한 계측을 실시하거나 실내 실험을 수행하였다. 지금까지 수행된 연구에 따르면 시간에 따른 크리프 변형량은 응력 조건, 상대 다짐도, 세립분 포함량 등과 같은 다양한 변수에 영향을 받는다. 국내에서 성토체에 대한 장기 크리프 거동은 아직 명확하게 연구된 바가 없어 본 연구에서는 크리프 거동에 대한 영향인자를 성토재료의 세립분 함유량, 성토체의 상대다짐도 및 전단응력비(q/p′)로 선정하고 각 영향인자에 따른 크리프 거동을 실험적으로 분석하였다. 크리프 실험에서 적용한 시험조건 중 상대다짐도와 세립분 함유량 조건은 국

내, 외 설계기준을 참고하여 결정하였으며, 성토체의 전단응력비는 성토체의 단계성토를 모사한 수치해석 결과를 바탕으로 성토체 내부 응력상태를 대표하는 조건으로 결정하였다. 크리프 압축시험을 통해 평가된 연직 변형률을 이용하여 시간-변형률 관계 그래프에서 선형구간을 회귀분석하여 크리프 계수를 결정하였다. 실험결과 시료의 전단응력비 및 세립분 포함량이 크고 상대 다짐도가 낮을수록 연직 변형률과 크리프 계수가 크게 나타남을 확인하였다. 또한 실험을 통해 구한 크리프 계수를 이용하여 실제 성토체 현장에 적용하여 크리프 침하량을 예측하고 실제 계측된 침하량과 비교하였다. 전단응력비가 큰 성토체의 형상을 고려하였을 때, 효과적인 침하관리를 위해 세립분의 함유량이 낮은 성토 재료 사용과 충분한 다짐 시공을 실시하는 것이 중요할 것으로 판단된다.

주요어 : 크리프 현상, 성토체, 세립분 포함량, 상대 다짐도, 전단응력비

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