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Investigation of Geotextile Yarn Effects on Improvement of Long-Term Deformation of Sandy Soil

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ABSTRACT

Accurate predictions of the amount and the rate of long-term deformation of reinforced soils under an applied load are important key issue in geotechnical engineering, in which the deformation of soil develops with time at a state of constant effective stress. Since geosynthetic is generally considered creep sensitive therefore, evaluation of creep behavior of geosynthetic reinforced soil (GRS) is necessary. In this study, to investigate the effect of reinforcing on the creep behavior of sandy clay soil, experimental tests on soil creep of reinforced sandy clay soil with geotextile yarn in one dimensional consolidation test are conducted and data analysis is explained based on relationship of the change in void ratio (Δe) and coefficient of secondary compression (C_α). Test results indicate that in reinforced water saturated samples with geotextile yarn, with increasing the percent of geotextile yarn creep, deformation decreases and time required for beginning the creep deformation increases.

1. Introduction

The application of GRS in the field of geotechnical engineering, for example bridge abutments and integrated bridge systems (IBS), has increased significantly over the past several years. Geosynthetic-reinforced soil (GRS) systems have been used since the late 1990s to increase the strength of soils in transportation structures. The GRS system offers some advantages over the more conventional rigid foundations (i.e., shallow and deep foundation systems). These advantages include cost effective construction (up to 30% savings), quick and easy

construction and good seismic performance, can be implemented in remote areas, can be used to reconstruct bridges after natural disasters and can be built on top of stiff to soft soils as the unit behave as a whole distributing the load in a uniform way. The creep behavior is concerned in evaluating the long-term performance of GRS retaining structure because geosynthetic, which are manufactured with various polymers, are generally considered creep-sensitive. To investigate the long-term interactive deformation characteristics of soil-geosynthetic composites, Wu and Helwany (2003) devised a long-term soil-geosynthetic performance test. In the test, a

sustained surcharge was applied to the soil. The stress induced in the soil was transferred to the geosynthetic. However, measurement of field performance of GRS retaining walls has indicated that the backfill properties play a very significant role in the long-term performance [1]. Some current design methods (e.g., AASHTO, 1992) for GRS retaining structure evaluate the long-term creep potential of a GRS retaining structure by performing "element" creep tests on the geosynthetic reinforcement alone (in either a confined or unconfined mode) [2]. Other design methods simply apply a safety factor or a creep reduction factor to the ultimate strength of the geosynthetic reinforcement to account for creep. Long term settlements of the soils are mainly due to the creep behavior of soil. Therefore, it is of great importance to accurately calculate and improve this settlement especially in the long lasting structures. In this study, experimental measurements of soil creep are performed on reinforced sandy clay soil with geotextile yarn in one-dimensional consolidation test to investigate the effect of reinforcing on the creep behavior of sandy clay soil.

2. Time Dependent Strain Behavior

Several researchers have reported that the time plays an important role in soil behavior. The investigations about the secondary compression have been start after Terzaghi consolidation theory (1925) which states that the compression of clay occurs after the depreciation of pore water pressure [3]. Laboratory tests and field observations reported by Buisman (1936) and Taylor (1942) clearly indicated the effect of time on the compressibility of clays. Buisman (1936) found a linear behavior of settlements versus logarithm of time under constant effective stress for clay and peat [4]. Taylor was the first one to introduce a time dependent model (Taylor 1942) to describe the creep behavior of soils, in which primary consolidation and secondary compression are considered as two separate processes [5]. In 1967, Bjerrum presented a conceptual model and it was intended to explain the apparent preconsolidation pressure or overconsolidation ratio of virgin Norwegian marine clays, resulting from creep effects.

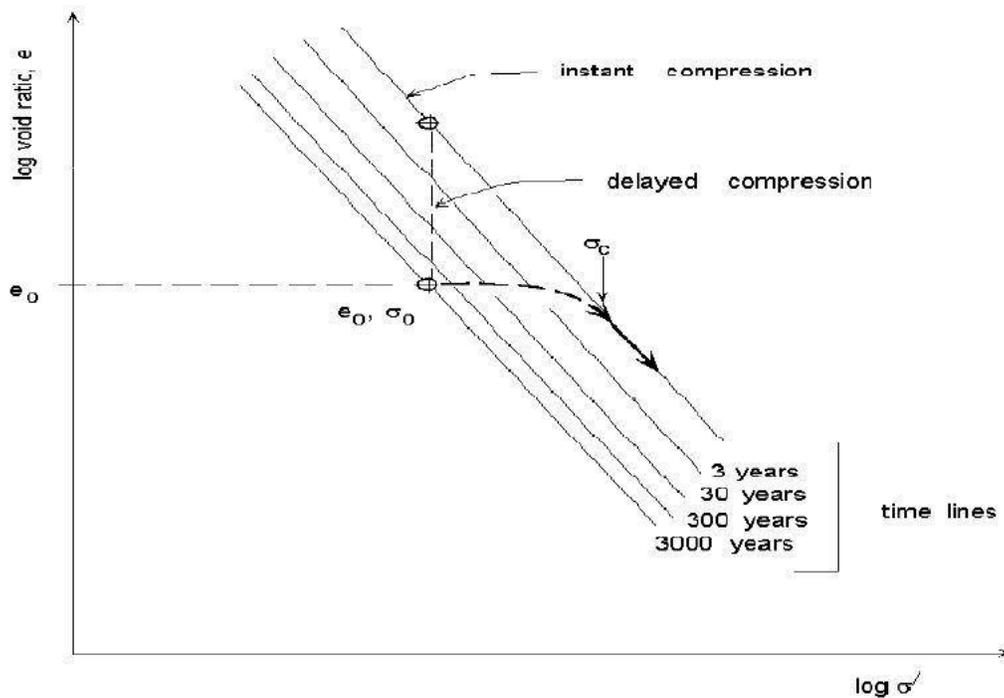


Fig.1 Bjerrum's time lines.

Bjerrum separated strains into “instants” and “delayed” compression and used “time lines” to model the reduced creep rates resulting from the increased duration of loading.. See Fig 1 for an illustration of this approach [6].

McDowell (2013) proposed a new equation for the 1D normal compression line, which contains a parameter controlling the size effect on average strength. They showed that the equation held for a wide range of discrete-element modelling (DEM) simulations is of crushable aggregates. The simulations examine the influence of the size effect hardening law, the time dependence on strength, and stress level. It is proved that the new equation holds true for each case [7]. Yin et al. (2013) studied the evolution of mechanical properties of soft soils during creep, and presented a variable-order fractional creep model, in which the fractional order was expected to represent the mechanical property of soils. The simulated results show that the evolution of mechanical characteristics can be divided into three stages. A comparative analysis illustrates that the evolution of mechanical properties exactly corresponds to the motions of pore water and the solid skeleton. This demonstrates that the proposed variable-order fractional model can be employed to characterize the evolution of the mechanical properties of soft soils during creep [8].

The consolidation sequence can be divided into two sub-categories; primary consolidation and secondary consolidation. Primary consolidation is defined as the deformation that occurs during the pore water pressure dissipation and secondary consolidation when the load is exposed to the structure of the soil and thereby deforming it. Both consolidations will be explained further in next section.

2.1 Primary consolidation

Consolidation is the process of volume decrease of a soil due to a hydrodynamic delayed dewatering from the pores of the soil [9]. A soil with a high permeability has an instantaneous deformation and a soil with a low permeability has a delayed deformation due to that it takes some time for the water to leave the pores of the soil. Soils with low permeability, such as clay and silt, are therefore characterized of time-

dependency [10]. If soil with a low permeability is exposed to a load, the increased stress will create an excess pore water pressure in the soil. Initially, the entire load is held up by the pore water, but as the water is leaving the system, the load is transferred to the structure of the soil which will then be compressed and creates settlements in the soil. At the same time, the excessive pore water pressure is decreasing and the effective stress is increasing.

2.2 Secondary consolidation

The phenomenon of creep settlements continuing after complete pore water equalization at a constant effective stress is called secondary consolidation, and is the time-dependency of the effective stress-strain relation. It is also known under the names secondary compression, plastic resistance to compression, time resistance and strain rate effects. But the common name is creep deformations (Larsson, 1986). Creep deformations occur when all the excess pore water pressure is zero and the load put on the soil is exposed to the structure of the soil. Creep is a slow process and does not create any hydraulic gradient [11]. Secondary consolidation in soil is the result a time dependent re-orientation of the particles in the aggregates due to a re-storage and increase of stress that occurs (Hansbo, 1975). Secondary compression is represented by an index called the coefficient of secondary compression (C_α).

The coefficient of secondary compression (C_α) is one of the most useful parameters to describe the behavior and the magnitude of the secondary compression and it is less affected by testing conditions. Coefficient of secondary compression can be expressed in several ways, but the following equation is commonly used:

$$C_\alpha = \Delta e / \Delta \log t \quad (1)$$

Where Δe is change in void ratio during the secondary compression stage and t is time.

3. Experimental procedure

Geosynthetics have a very high tensile strength which the soil lacks. Geosynthetics reduce the differential settlement, increase the

bearing capacity, and the slope stability when used in soils. Mineral composition (i.e., mineral content of the clayey particles), stress level, pore fluid chemistry, drainage condition and fabric structure have been recognized as the important parameters influencing the creep behavior of soil [12]. However the present study mainly focuses to identify and study the effect of reinforcing the sandy clay soil with geotextile yarn on creep deformation in one dimensional consolidation apparatus. In this study, casagrande curve fitting method is used to determine the time (t_{100}) taken to completely dissipate the excess pore water pressure at the particular stress level, and the void ratio (e_{EOP}) at the end of primary consolidation [13]. In casagrande curve fitting method, two linear portions, initial portion of the primary consolidation and secondary compression stages, are plotted and the intersection of both lines are taken as the end of primary consolidation point [14]. This Fig. 2 clearly shows a typical void ratio-log time relation of saturated soil, primary consolidation and secondary compression regions in the one dimensional compression test at a sustained total stress.

3.1 Material and methods

Two types of soils have been used as materials in this study. Ottawa sand, Kaolinite organic clay soil and geotextile yarn have been used to evaluate the creep behavior. The Ottawa sand which is used in the one dimensional consolidation creep tests is ungrounded ASTM-

C778 silica sand. In the grain solid, the content of silicon dioxide is around 99.8%. The sand has a specific gravity of 2.65 gr/cm^3 . The grain sizes of raw sand are from 0.595 mm to 0.841 mm. Kaolinite is the most common clay type found in highly-weathered environments. Kaolinite is composed from pseudo hexagonal triclinic crystals with diameter of 0.2–10 μm . It is composed of silicate sheets (Si_2O_5) bonded to aluminum oxide/hydroxide layers ($\text{Al}_2(\text{OH})_4$). Its chemical structure is $\text{Al}_2\text{Si}_2\text{O}_5(\text{OH})_4$. The molecular weight of kaolinite is 258.071 g/mol. Kaolinite is non-expanding when water is added, because each clay particle has a specific area which is small: ranges between 10 and 20 m^2/g [12]. Details of kaolinite organic clay are given in table 1. The geotextile yarn which is used in one dimensional creep tests is ungrounded ASTM standard which its characteristics are given in table 2.

3.2 Test equipment

The purpose of this experimental study is to investigate the one-dimensional creep behavior of sandy clay soil. All one-dimensional consolidation tests are carried out according to standard ASTM D 2435-90 under single drainage condition. The consolidation apparatus consists of ring with a diameter of 6cm and the height of 2cm, consolidation cell, two porous stone disks and a loading guide resin piston.

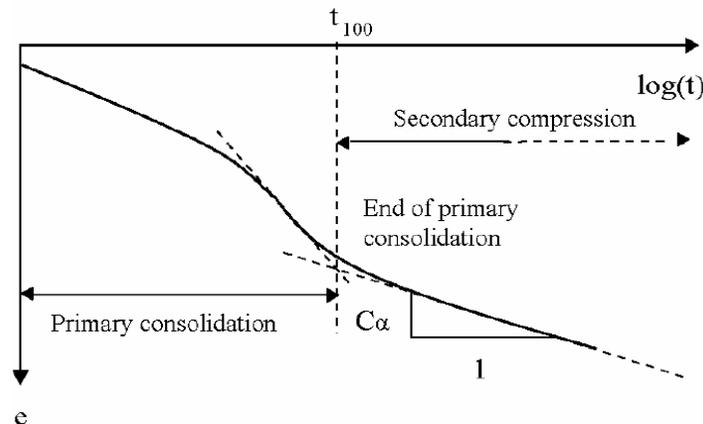


Fig. 2 Typical void ratio-log time relation of saturated soil in one dimensional compression.

Table 1, Characteristics of organic clay soil.

Density (GS) g/cm ³	2.47
Liquid Limit (LL)	57
Plastic Limit (PL)	33.66
Plastic Index (PI)	23.34

Table 2, Index properties of geotextile yarn used in one dimensional consolidation creep test

specification	Examination result	Examination method
Unit Weight	300gr/m	ASTM D5261
Thickness	3.7mm	ASTM D5199
Punctured strength	720N	ASTM D4833
Grab tensile strength& expansion	950N	ASTM D4632
Width & Length tearing strength	380N- 340N	ASTM D4533
Melting point	>240C	ASTM D276
Permeability	0.22Cm/Se	ASTM D4491

In all tests, the axial load is applied by an consolidation apparatus and the axial deformations during the primary consolidation and the creep stages are recorded using dial gage with an accuracy of 0.01mm. To preserve the saturated condition, pore water is added into the consolidation cell during the test. At the beginning of each test, initial void ratios of sandy clay slurry are determined using the values of moisture content. The initial values of moisture content are determined by drying the samples in an oven at 100°C for 24 hours.

3.3 Experimental program

The objectives of this study are investigation of the effect of reinforcing sandy clay soil with geotextile yarn on the creep deformation, and explanation of the mechanisms involved in creep phenomenon. Creep tests are performed on samples in state of single stage in that soil sample is loaded to specified stress level and is allowed to creep at this stress level. In all tests, the samples with a specified mass (73g) are poured into the confining ring. In reinforced samples, are mixed and are occurred under the test condition. In water-saturated sample with 0.1%, 0.5% and 1% of geotextile, the loadings are applied incrementally for duration of 35, 45 and 60 minutes respectively between two consecutive loads to complete the dissipation of excess pore water pressure, these time scales are selected based on the one-dimensional consolidation tests on water-saturated sandy clay samples, but in dry sample, the loading is applied in a single increment. Fig. 3 shows the initial grain size distribution curve of the sandy clay sample on the basis of ASTM D422-63 standard.

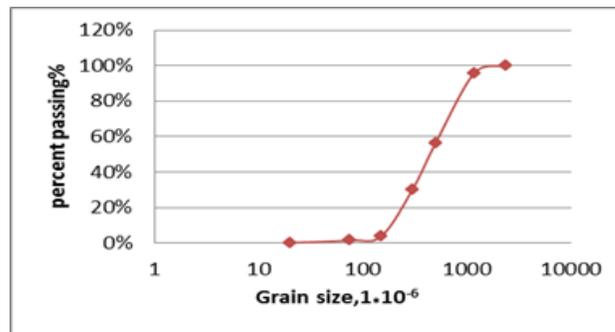


Fig. 3 The initial grain size distribution curve of sandy clay sample.

Single stage compression creep tests are carried out on the water-saturated and dried in air sandy clay samples at the stresses of 300, 600 and 1200 kPa. Notice that the sample under the aforementioned stresses is loaded for about 170 hours.

In this study, it is observed that the axial strain rate-time relations do not change significantly when the period of time exceeds 170 hours for sandy clay samples. Therefore, the creep rates for the sandy clay samples are calculated in a period of time of 170 hours.

A series of one dimensional consolidation creep tests are performed to obtain the creep deformation characteristics of reinforced sandy clay soil with geotextile yarn. Void ratio variations versus time for the single stage compression creep tests on the reinforced water saturated samples in 0.1, 0.5 and 1 weight percent of geotextile at the stresses of 1200, 600 and 300 kPa are shown in Fig. 4, Fig. 5 and Fig. 6, respectively. The figures clearly show incremental loading steps, primary consolidation stage and secondary compression stages.

4. Results and discussion

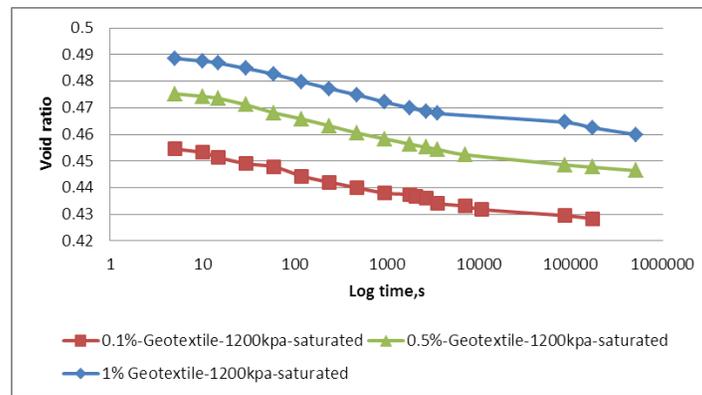


Fig 4 Relationships of void ratio versus time on the reinforced water saturated samples in 0.1, 0.5 and 1 weight percent of geotextile at the stress of 1200 kPa.

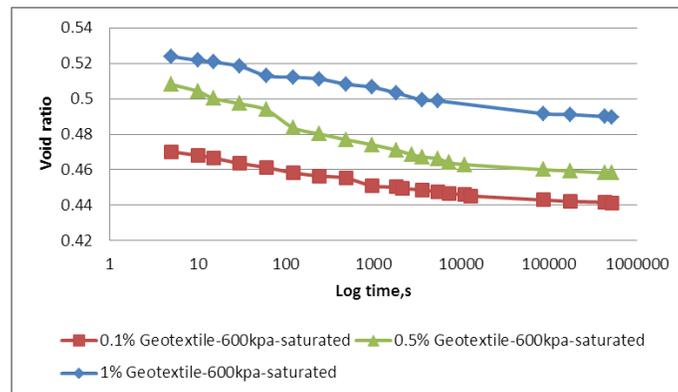


Fig 5 Relationships of void ratio versus time on the reinforced water saturated samples in 0.1, 0.5 and 1 weight percent of geotextile at the stress of 600 kPa.

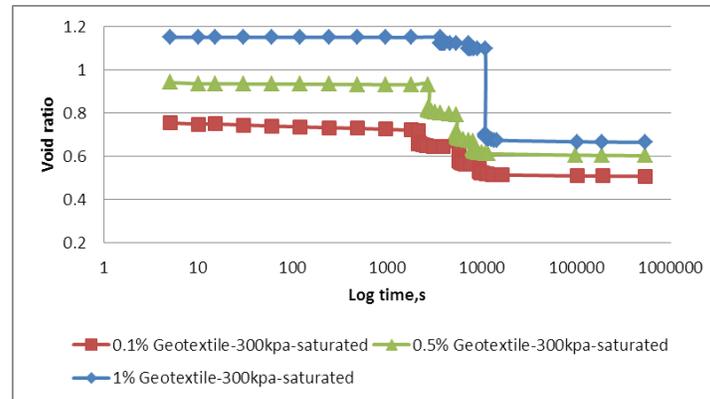


Fig 6 Relationships of void ratio versus time on the reinforced water saturated samples in 0.1, 0.5 and 1 weight percent of geotextile at the stress of 300 kPa.

Void ratio variations versus time for the single stage compression creep tests on the reinforced dried in air samples in 0.1, 0.5 and 1 weight percent of geotextile at the stresses of 1200, 600 and 300 kPa are shown in Fig. 7, Fig. 8 and Fig. 9, respectively. It can be said that at low stress level, the value of C_{α} in reinforced water-saturated sandy clay sample is higher than the dried sample, but with increasing stress level it decreases, and in dry samples, with increment of the stress level, it increases. Loading steps, stress levels, duration of tests and initial void ratios of samples are illustrated in Table. 3. Fig. 10 and Fig. 11 show the relationship of coefficient of secondary compression (C_{α}) with stress level (σ_{creep}) of reinforced dry and saturated sandy clay samples in single stage tests respectively. There is an approximately nonlinear relationship between C_{α} and σ_{creep} . In reinforced saturated sample with increasing stress level creep rate decreases, and in dry samples it increases.

4.1 Comparison of the paper test results with two similar previous projects

In this section, the comparison between the test results of this paper and two previous works carried out by Zhechao wang and Sivarajan varatharajan is presented. Zhechao wang using one-dimensional consolidation creep test on Ottawa sand and Kaolinite clay sample concluded that in Ottawa sand in high stress creep deformation is along with particles

crushing and this deformation increases during the time. Moreover, in Kaolinite clay with stress increment, creep deformation decreases. Indeed, with stress increment the sample become denser and more stable and less creep deformation occurs [15].

Sivarajan varatharajan carried out an experimental program to investigate the effective parameters on the creep behavior Kaolinite clay in one-dimensional consolidation creep test. The coefficient of secondary compression is used for data analysis. The test results show stress variations and creep deformation rate have inverse relation. However, the creep test results in this work show that reinforcing the soil samples decreases the creep deformation in saturated sample and increases it in dry samples [12]. The abbreviation of comparison the paper test results with the mentioned previous projects are given in Table 4.

5. Conclusions

Creep behavior of sample is analyzed and discussed using the coefficient of secondary compression (C_{α}). Casagrande curve fitting method is used to determine the void ratio at the end of primary consolidation (e_{EOP}), time required to complete the primary consolidation (t_{100}) and the coefficient of secondary compression (C_{α}).

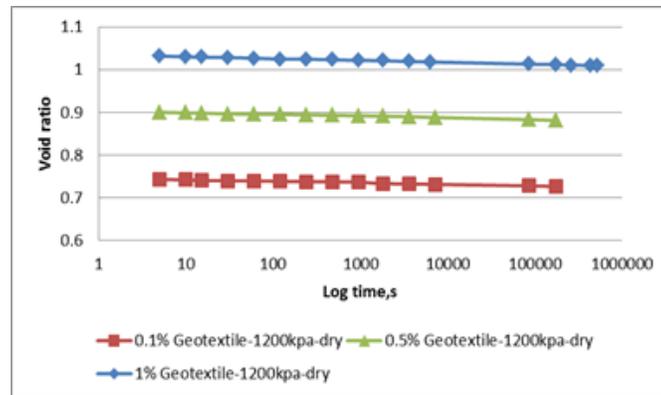


Fig 7 Relationships of void ratio versus time, on the reinforced dried in air samples in 0.1, 0.5 and 1 weight percent of geotextile at the stress of 1200 kPa.

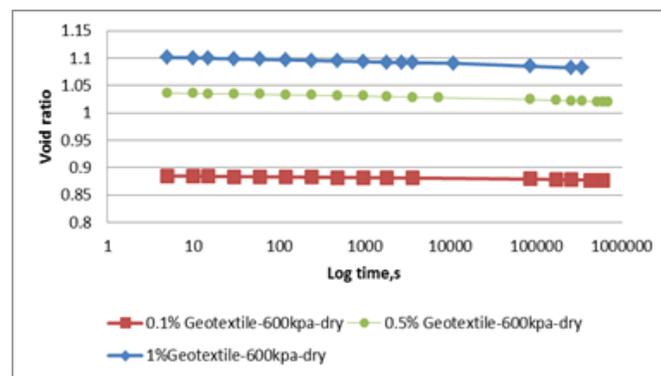


Fig 8 Relationships of void ratio versus time, on the reinforced dried in air samples in 0.1, 0.5 and 1 weight percent of geotextile at the stress of 600 kPa.

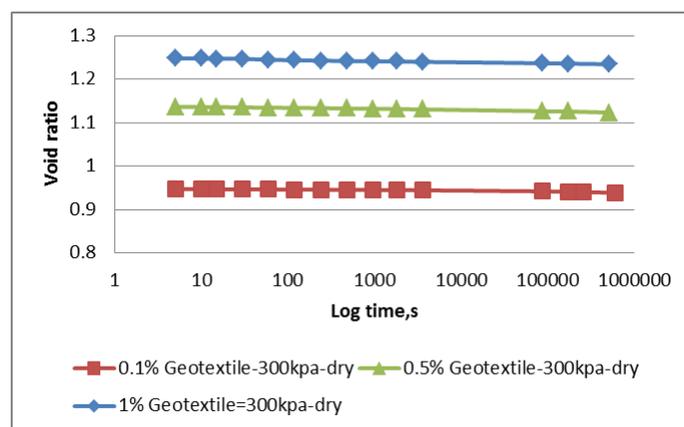


Fig 9 Relationships of void ratio versus time, on the reinforced dried in air samples in 0.1, 0.5 and 1 weight percent of geotextile at the stress of 300 kPa.

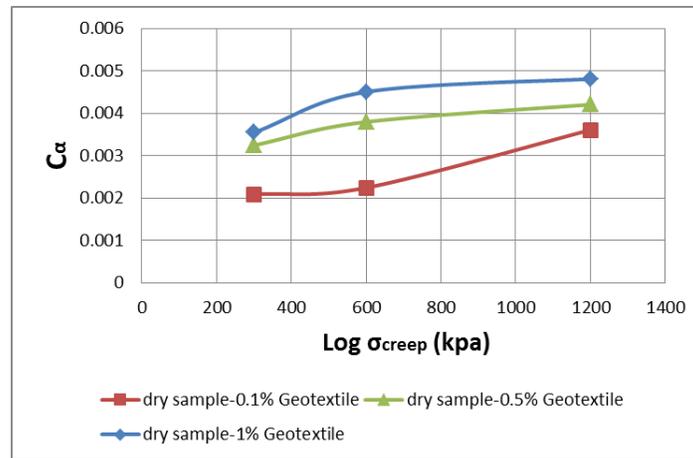


Fig. 10 Relationships of secondary compression and creep stress in the single stage tests on the reinforced dried in air samples.

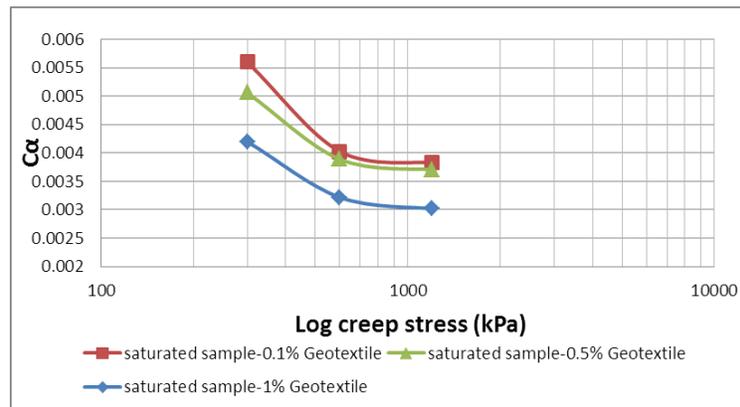


Fig. 11 Relationships of secondary compression and creep stress in the single stage tests on the reinforced water saturated samples.

Single stage creep tests are carried out at different stress levels on dried in air and water-saturated reinforced sandy clay soil with geotextile yarn to illustrate the effect of reinforcing on the creep behavior of samples.

In this paper, it can be result that at low creep stress level, the value of C_a in water-saturated sample is higher than the dried in air sample because in saturated sample, due to the higher sliding ability and lower frictional, particles slide very easily. But increases stress level make the samples denser and smaller, and so the soil structures become stable and creep rate decrease, whereas in dry samples at low

stress levels, the creep rate increase with the stress increment. With increasing the percent of geotextile yarn, the rate of secondary deformation in water-saturated reinforced sandy clay soil decreases and time required for beginning the creep deformation increases, but in dry samples the rate of secondary deformation increases. Pore water significantly influences the fabric structure of soil, ratios of micro and macro pores, compressibility and coefficient of secondary compression.

Table 3, The result of conducted compression creep test on the samples.

Type test	Type of pore fluid	Sample	Stress (kPa)	Incremental loading steps (kPa)	Initial void ratio (e ₀)	Void ratio end	Coefficient of secondary
						primary consolidation (e _{EOP})	compression (C _α)
Saturated sample (water)	SS-WS-300kpa-0/1% Geotextile	300	50-100-200-300	0.98993	0.592183	0.005600	
	SS-WS-300kpa-5/0% Geotextile	300	50-100-200-300	1.2119	0.6310077	0.005060	
	SS-WS-300kpa-1% Geotextile	300	50-100-200-300	1.2923	0.6608207	0.004991	
	SS-WS-600kpa-0/1% Geotextile	600	50-100-200-400-600	0.98993	0.449439	0.004028	
	SS-WS-600kpa-5/0% Geotextile	600	50-100-200-400-600	1.2119	0.468488	0.003891	
	SS-WS-600kpa-1% Geotextile	600	50-100-200-400-600	1.2923	0.499195	0.003219	
	SS-WS-1200kpa-0/1% Geotextile	1200	50-100-200-400-800-1200	0.98993	0.437439	0.003830	
	SS-WS-1200kpa-5/0% Geotextile	1200	50-100-200-400-800-1200	1.2119	0.455177	0.003705	
	SS-WS-1200kpa-1% Geotextile	1200	50-100-200-400-800-1200	1.2923	0.46789	0.003019	
	Single stage	SS-AD-300kpa-0/1% Geotextile	300	300	1.06162	1.945877	0.002086
SS-AD-300kpa-5/0% Geotextile		300	300	1.3279	1.132726	0.003244	
SS-AD-300kpa-1% Geotextile		300	300	1.3983	1.241474	0.003544	
SS-AD-600kpa-0/1% Geotextile		600	600	1.06102	0.88194	0.002243	
SS-AD-600kpa-5/0% Geotextile		600	600	1.3279	1.02144	0.003795	

	SS-AD-600kpa-1% Geotextile	600	600	1.3983	1.09472	0.004502
	SS-AD-1200kpa-0/1% Geotextile	1200	1200	1.06162	0.73684	0.003608
Dry sample	SS-AD-1200kpa-5/0% Geotextile	1200	1200	1.3279	0.892276	0.004205
	SS-AD-1200kpa-1% Geotextile	1200	1200	1.3983	1.021193	0.0048093

DA: Dried in air, WS: Water saturated, SS: Single stage, SW: Stepwise, σ_{creep} : Creep Stress.

Table.4. The abbreviation of comparison between the paper test results and the mentioned two previous projects

Comparison item	Soil Sample	Duration of test (minutes)	Stress	e_0	C_α	
Studied project			50 kPa	1.551	0.0152	
			100 kPa	1.553	0.0144	
	Sivarajan varatharajan (2011)	Saturated Kaolinite	10080	200 kPa	1.491	0.0133
			400 kPa	1.521	0.0121	
			800 kPa	1.551	0.0101	
Zhechao wang (2010)	Ottawa sand	120	10 MPa	0.543	8.85×10^{-5}	
			18 MPa	0.543	1.26×10^{-4}	
			28 MPa	0.543	4.71×10^{-4}	
	Dry Kaolinite	12000	200kPa	1.894	0.0140	
			800kPa	1.894	0.0074	
	Saturated Kaolinite	12000	200kPa	1.590	0.0076	
		500kPa	1.590	0.0053		
		900kPa	1.590	0.0037		
Reinforced dry sample (0.1% geotextile)			300kpa		0.002086	
			600kpa		0.002243	
			1200kpa	1.06162	0.003608	
			300kpa		0.003244	
			600kpa	1.3279	0.003795	

In this study	(0.5% geotextile)	10080	1200kpa	0.004205	
	Reinforced dry sample		300kpa	0.003544	
			600kpa	1.3983	0.004502
			1200kpa		0.004809
	(1% geotextile)		300kpa		0.005600
			600kpa		0.004028
			1200kpa	0.9899	0.003830
	Reinforced saturated sample		300kpa		0.005060
			600kpa		0.003891
			1200kpa	1.2119	0.003705
	(0.5% geotextile)		300kpa		0.004991
			600kpa		0.003219
1200kpa		1.2923	0.003019		

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