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Effect of Non-Woven Geotextile Reinforcement on Mechanical Behavior of Sand

모래의 역학적 거동에 미치는 부직포 보강재의 효과

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ABSTRACT

The effects of non-woven geotextiles on mechanical behavior of sand were investigated. A comprehensive series of triaxial compression tests were performed for these investigation on unreinforced and reinforced sand with geotextiles. The Joomunjin standard sand was used and non-woven geotextiles were included into sand specimen with three layers. The inclusion of non-woven geotextile reinforcement into sand increased the peak strength of sand significantly and the reinforced samples exhibited a greater axial strain at failure. Also the effect on number of reinforcement layers was studied and found as increasing the number of reinforcement layers resulting in more ductility by clogging developed in the shear band within the specimens. It was also found that the tendency of samples to dilate is restricted by non-woven geotextile inclusion. The effect of nunber of reinforcement layer increasing is just same to the effect of decreasing void ratio of sand in this case.

요 지

부직포에 의한 보강이 모래의 역학적 거동에 미치는 영향에 대해 검토하였다. 이 검토를 위해 보강모래와 무보강모래에 대한 일련의 삼축압축시험이 수행되었다. 표준사인 주문진사가 사용되었고, 부직포 보강재가 모래시료 속에 최대 3층 배치되었다. 분석결과 모래속의 부직포 배치는 모래의 전단강도를 크게 증가시켰고, 보강모래는 파괴시의 변형률도 증가하였다. 또한, 보강층수가 증가할수록 보강재에 의한 시료내 전단층의 확산 방지효과의 증가로 보다 연성화하고, 모래시료의 체적팽창 경향은 부직포 배치에 의해 억제됨이 밝혀졌다. 보강층수 증가에 의한 효과는 간극비 감소로 인한 효과와 같다.

Keywords **:** Non-woven geotextile, Triaxial test, dilatancy, reinforcement effect

1. Introduction

In the history of construction material technology, it has been known that soil resists well against compression and shear, but very weak against tension. Several attempts have been made to overcome the tension weakness of soil. Utilizing a tensile element within the soil mass, in order to improve the strength characteristics, has been employed long-time ago; more than 3000 years ago, Babylonians used reinforced soil (Jha and Mandal, 1988). It has also been a common practice in Iran to use a mixture of soil and straw as a construction material. In 1966, Vidal(1978) invented a new technique in which galvanized steel strips were used in cohesionless soil to improve its

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engineering properties. Since 1970s, the use of geotextile as reinforcement has become more popular due to a more satisfactory performance compared with metal reinforcement, which has been reported in several instances (Gray and Ohashi, 1983). The reason is that geotextiles or synthetic fabrics have relatively low stiffness compared to that of metals. Thus, they are more compatible with soil in view of deformability. Geotextiles, as reinforcing materials, not only increase shear strength but also improve ductility and provide smaller loss of post-peak strength in reinforced sand in comparison with unreinforced sand. Today, geotextiles have practically invaded Civil Engineering industry the world over, as viable and economical construction materials with multifarious uses. Although the experimental results could not be used directly in the design of reinforced earth structures, they provide an efficient, fast and economical method for investigating and understanding the behavior of reinforced earth. Since the beginning of 1970s, several investigators have studied stress-strain and strength characteristics of reinforced soil using triaxial, direct shear, and plane strain tests(Bishop, 1969; BS 9606, 1991). From 1977, extensive experimental work has been performed on geotextile reinforced sand. Following is the review of some of these investigations. Broms(1977) illustrated reduction in lateral earth pressure of geotextile-reinforced sand using triaxial tests where disks of geotextile were placed horizontally in the soil samples. An increase in the peak strength was observed along with a decrease in distance between the geotextile disks. However, the results showed that the disks, which were placed at the two ends of samples, did not influence the peak strength. Gray, D.H and Al-Refeai(1986) conducted triaxial compression tests on dry reinforced sand using five different types of geotextile. Test results demonstrated that reinforcement increased peak strength, axial strain at failure, and, in most cases, reduced post-peak loss of strength. At very low strain (<1%), reinforcement resulted in a loss of compressive stiffness. Failure envelope of the reinforced sand showed a clear break with respect to the confining pressure. After the point of break, failure envelope for the reinforced sand paralleled the unreinforced sand envelope.

The objective of this study is to present the results of undrained triaxial compression tests on reinforced saturated sand with non-woven geotextiles. In this study, in addition to describing the influence of confining pressure, the number of geotextile layers on the test results are also illustrated which have not been reported yet.

2. Experimental Program

2.1 Reinforcement and Specimen Preparation

To investigate the effects of test parameters on the mechanical behavior of unreinforced sand and geotextiles-reinforced sand, the non-woven geotextile is used with three layers as shown in Fig. 1(G.Madhavi Latha, 2006). The mechanical properties of Non-woven geotextile as Table 1. Nominal thickness of non-woven geotextile used is 2.1 mm. The reinforcement was placed horizontally in their position marked in the membrane in the specimen. The diameter of the reinforcement was slightly less than that of sample. The number of layer was selected 1, 2 and 3 and positioned as shown in Fig. 1. In the triaxial apparatus, the specimen has a dimension of 7.5 cm in diameter and 15 cm high.

The samples were prepared by the air-pluviation method to maintain the relative density. Air-dried Joomunjin sand was weighed and sand particles were allowed to be plu-

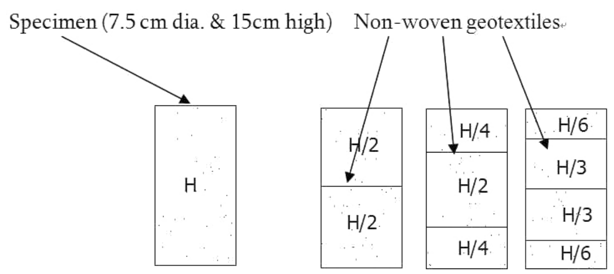

Fig. 1. Non-woven layer arrangements in triaxial tests

Table 1. Mechanical properties of non-woven geotextiles (Lee, 2002)

Geotextiles	Tensile strength Tmax (tf/m)	Axial strain at peak	Young's modulus (tf/m)	
			Initial	Secant at 5%
Non-woven	3.060	0.470	14.1	8.3

viated into the mould from funnel through air. Its gradation curve and material properties are shown in Fig. 2 and Table 2. The height of the pluviation above the sand surface in the mould was always maintained constant by using a string hanged from the funnel. The height of pluviation was variable (5 to 100 cm) in these experiments, which gave a relative density of 35 to 80 % for the tested material(Figs. 3 and 4). During the pluviation, the funnel was moved slowly to cover the entire area so that the sand did not heap on one place. Practice is needed to prepare the specimen within close margin of density or void ratio because the density of the specimen can vary to some extent depending on with the speed of funnel movement. It was also needed to keep the opening of funnel constant throughout the pluviation to get the same density throughout the specimen. Pluviation process was continued till the sand surface became approximately 0.5 mm higher than the top edge of the mould. Extra sand was trimmed with the help of one metal rod, very carefully not to damage the sand surface.

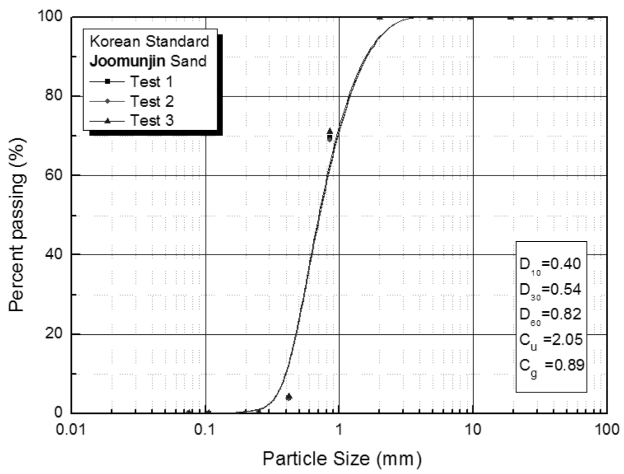

Fig. 2. Particle size distribution curve of Joomunjin sand

Table 2. Material properties of Joomunjin sand

	Joomunjin Sand
e_{max}	0.955
e_{min}	0.608
G_s	2.650

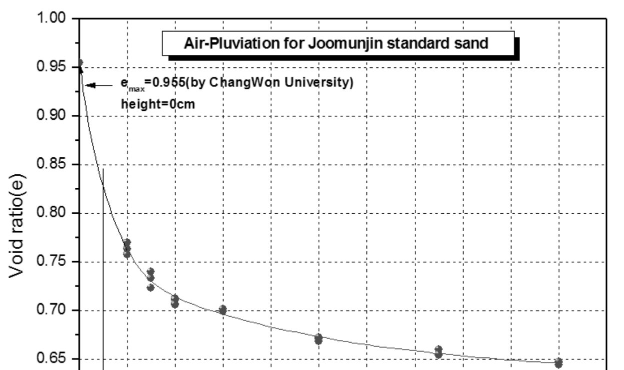

Fig. 3. Void ratio and falling height relationship by pluviation

2.2 Triaxial apparatus

The test was performed on triaxial apparatus. The schematic diagram of triaxial test apparatus is showed Fig. 5. In this study, Consolidated-Undrained (CU) tests were performed with the measurement of pore pressure. The axial load was measured with a load cell placed inside the triaxial cell to eliminate the effects of piston friction. The lateral stress was applied by using air inside the cell and measured by high capacity differential pressure transducer (HCDPT). For the measurement of axial deformation, local displacement transducer (LDT) was placed on the side of the specimen to avoid the bedding error (Tatsuoka, 1988). For all the tests, strain rate of 0.034 mm/min was used. All test methods were followed JGS 0523(2000).

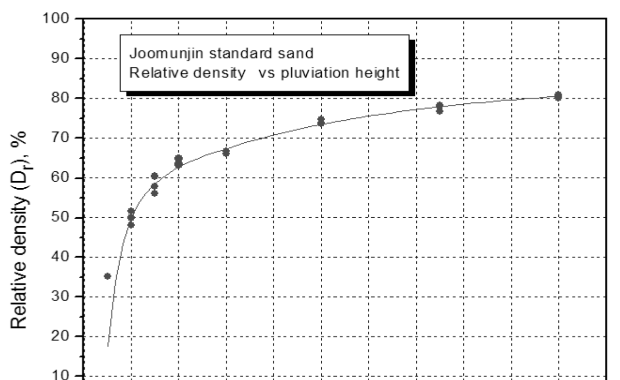

Fig. 4. Relative density and falling height relationship

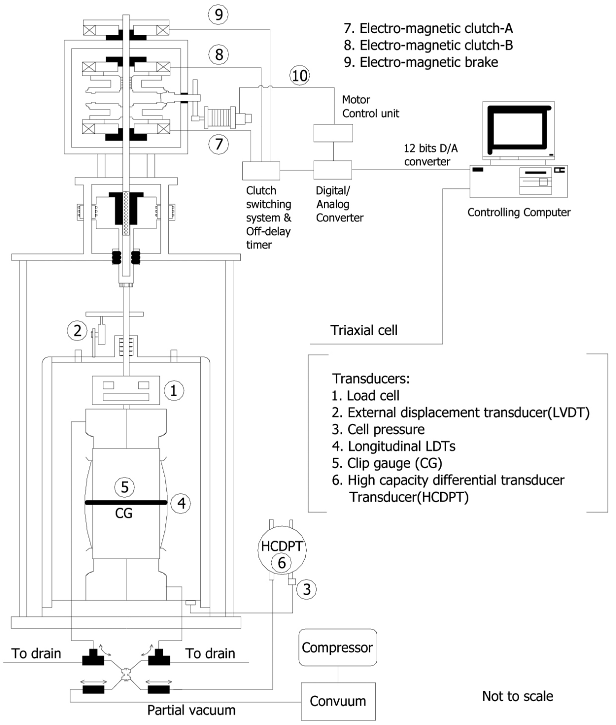

Fig. 5. Schematic diagram of triaxial compression test apparatus

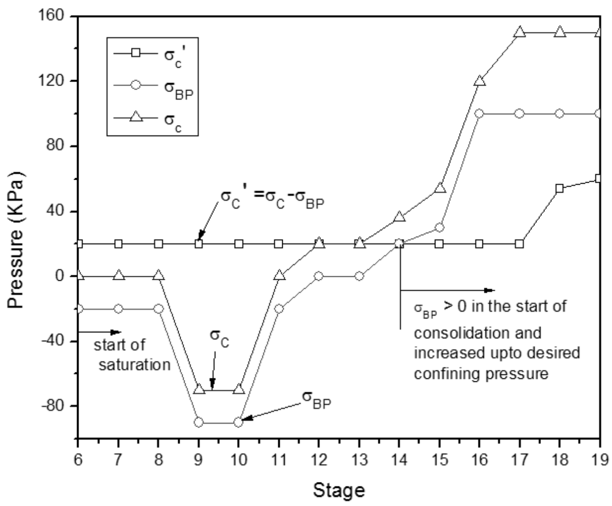

Fig. 6. Saturation and consolidation stages during triaxial test

2.3 Saturation and isotropic consolidation

Fig. 6 is the diagram of saturation and consoilidation stages during triaxial test. After the required backpressure is reached, the pore water pressure value (B) is measured to check saturation degree. If (B) value is less than 0.95, further saturation is required then the designated cell pressure is increased for the complete consolidation and saturation.

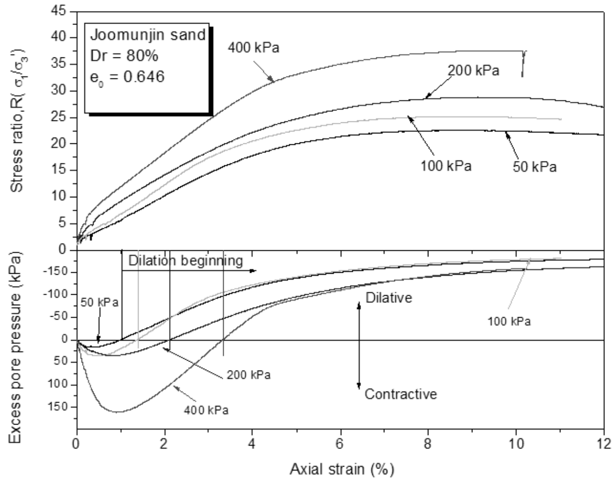

Fig. 7. Stress strain diagram for unreinforced sand

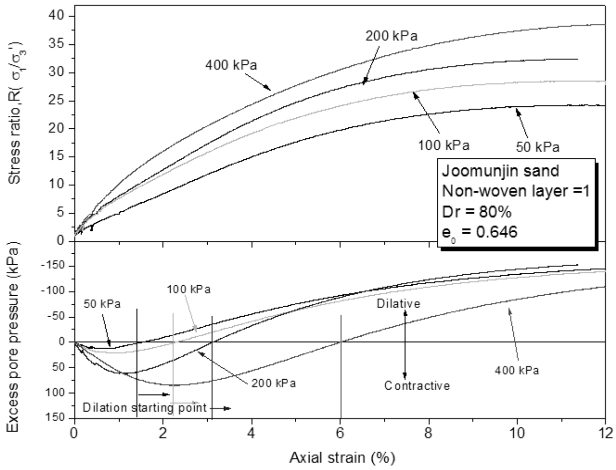

Fig. 8. Stress strain diagram for reinforced sand

3. Test Results and Discussion

Typical stress-strain curves for reinforced and unreinforced sand with different number of geotextile layers are as shown in Figs. 7 and 8. It was shown that the stress-strain, excess pore water pressure and strain relation of unreinforced sand in Fig. 7. In this figure the following points are cleared. The stress ratio is increasing as the confining pressure is increasing. The excess pore water pressure is also increasing in proportion to the confining pressure. Based on the fact that the pore water pressure and strain curve of CU test is similar to volume change and strain curve of CD (Consolidated-Drained) test, it can be said that the strain of the dilatancy starting point increase as the confining pressure is increasing. Also, as

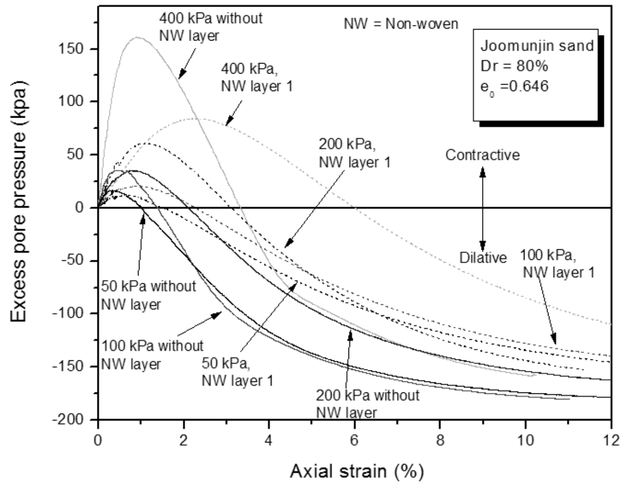

Fig. 9. Pore pressure diagram for reinforced and unreinforced sand

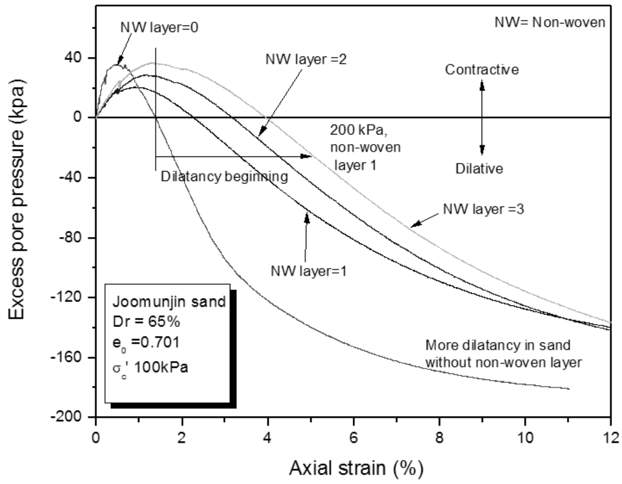

Fig. 10. Pore pressure diagram for reinforced layers

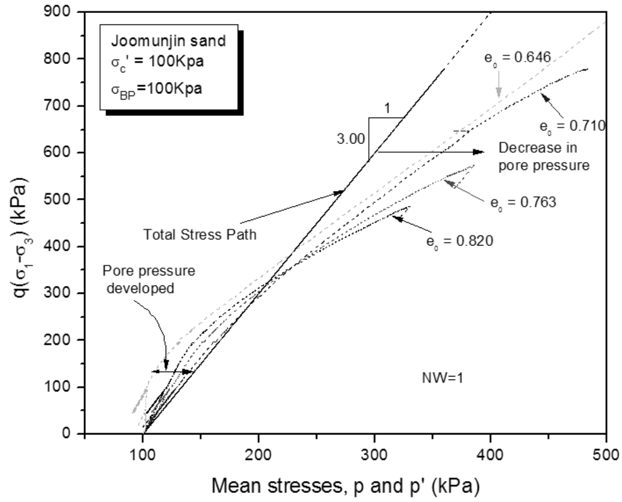

Fig. 11. Stress paths for different initial void ratio of sand

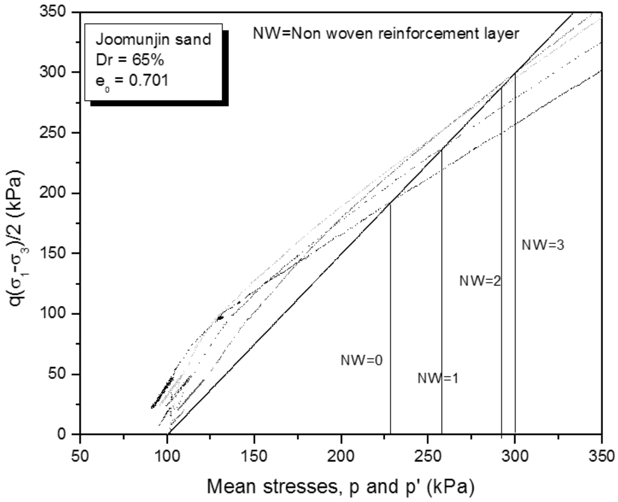

Fig. 12. Stress path diagram for reinforced layers

expected during the initial shear, the volume of unreinforced and reinforced sand decreases slightly. With further shearing, the behavior is reversed and the specimens show an increase in volume, while, increase in confining pressure limits the expansion of both unreinforced and reinforced sand. Fig. 8 is the same curve on the 1-layer reinforced sand as shown in Fig. 7. Compared with Fig. 7, the following facts can be pointed out from Fig. 8. The stress ratio is increasing with increasing the confining pressure, but the peak stress ratio is unclear. The dilatancy starting point increase almost 2 times and the negative excess pore water pressure tends to decrease with only 1 reinforcing layer. Non-woven geotextile inclusion reduces post-peak loss of strength. In fact, increasing the number of reinforcement layers resulted in more ductility to the samples as clogging developed in the shear band within the specimens.

Figs. 9 and 10 are showing excess pore water pressure and axial strain relation. From Fig. 9, it can be pointed out that the change of positive and negative excess pore water pressure is bigger in the case of unreinforced sand than in the case of reinforced sand as the confining pressure increase. It was found from Fig. 10 that the change of pore water pressure as reinforcing layer increase is relatively small comparing with the case of unreinforced sand.

Figs. 11 and 12, shows the total and effective stresses paths for different void ratios and number of non-woven layers at confining pressure 100 kPa. In Figs. 11 and 12, each stress path is showed with the relation of deviator

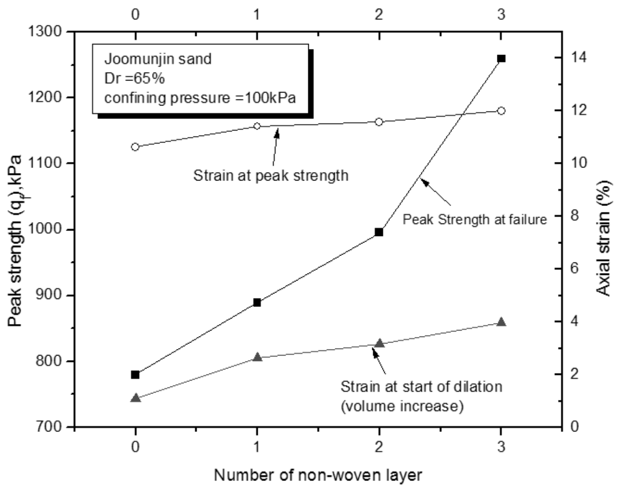

Fig. 13. Peak strength versus number of layers

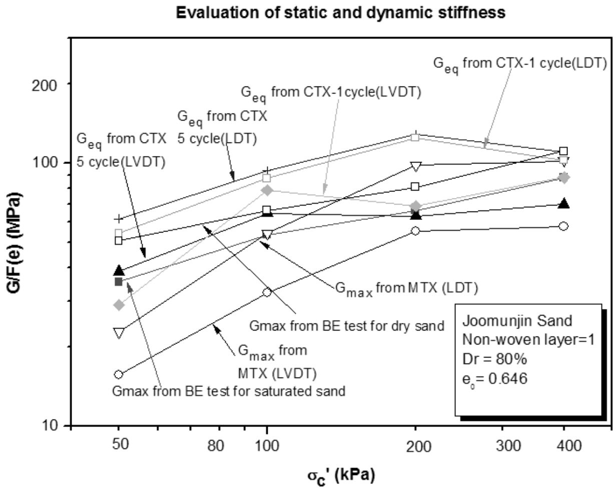

Fig. 14. Shear modulus of reinforced and unreinforced sand

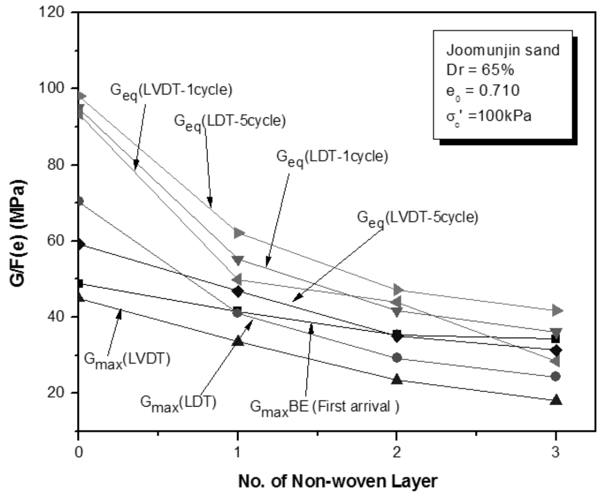

Fig. 15. Shear modulus and number of non-woven layer

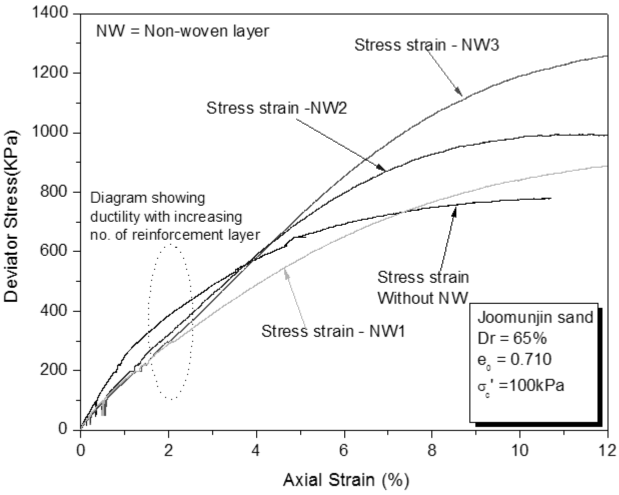

Fig. 16. Deviator stress and axial strain relation

stress(q) and mean effective stress(p'). Two figures show clearly the sample contractive or dilative from the total and effective stress lines and the successive stress states as well. In Fig. 11, it is clear that as the initial void ratio decrease, development of positive and negative pore pressure are decreasing. Also in Fig. 12, as the No. of reinforcement layer increase, development of positive and negative pore pressure are decreasing. It is also clear the effect of number of reinforcement layer increasing is just same to the effect of decreasing void ratio of sand in this case. The tendency of samples to dilate is restricted by non-woven geotextile inclusion as shown in Figs. 7 to 10, which can be due to the following reason. As reported by many researchers (Duncan and Dunlop, 1968), dilation occurs mainly in the center of specimens. At the top or bottom of the sample, the cap and the base restrain lateral deformation and dilatancy. Also, in this study, failure of reinforced sand was observed by bulging between geotextile layers. It is evident that geotextile effectively restricts the dilatancy of the samples. This effect becomes further apparent when the number of geotextile layer increases.

Fig. 13 shows that non-woven geotextile inclusion increases the peak strength significantly. This matter is due to the increase in confinement by reinforcement layer. The results of this Fig. 13 demonstrate that the reinforced samples exhibited a greater axial strain at failure and a greater strain at start of dilatancy to those of the corresponding unreinforced sample. From Figs. 14 and 15, it can be said that the initial young's modulus of reinforced was found to be smaller than the reinforced one. This may be due to the reinforcement layer deformation as it is weak in compression as shown in Fig. 16.

4. Conclusions

The main conclusions of this study were summarized as follows.

(1) Geotextile inclusion enhances peak strength, axial strain at failure. The progress is more effective with a higher number of geotextile layers. However the initial Young's modulus was found lower than the unreinforced one.
(2) Geotextile inclusion reduces the dilatancy of reinforced sand due to confinement enhancement.
(3) The effect of nunber of reinforcement layer increasing is just same to the effect of decreasing void ratio of sand in this case.

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