CONCLUSIONS

Based on the study of the four wheel gear configuration under quasi-static loading, there are several conclusions that can be drawn. Deflection and upheaval in the asphalt is greater without wander than with wander. In the subgrade, wander causes more stress to be transmitted which leads to a greater amount of vertical plastic strain. Wander also allows for greater vertical stress and plastic strain to be imparted to the subgrade. Rutting and upheaval show decaying attenuation rate. Each run has a diminishing increase in rutting and upheaval. These results indicate that pavement is showing signs of consolidation. If rutting and upheaval rates were accelerating, then the pavement would be showing signs of structural failure. As permanent deformation increases, the amount of increase of elastic strain becomes smaller. This is why a decay function, such as a log function, describes the rate of elastic strain increase. There is a clear difference with plastic strain; wander allows for higher magnitude of plastic strain to be imparted to the subgrade but the rate of increase of plastic strain with each pass of the wheel is reduced. This is important when determining the life of the pavement with wander because even though surface rutting might be reduced due to wander for initial cycles of loading, the greater plastic strains induced in the underlying layers might induce shear failure and reduces the life of the pavement.

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REFERENCES

Federal Aviation Administration, (2004) “Airport Pavement Design and Evaluation,” FAA Advisory Circular 150/5320-6D.

Federal Aviation Administration, (2006) “Material Properties Database Spreadsheets” http://www.airtech.tc.faa.gov/NAPTF/

Hayhoe, G., McQueen, R and Guo, E. (1993) “Airport Pavement Test Machine Design and Cost Study,” FAA Air Traffic Activity, 30 Sep 93, 286 pages.

Hayhoe, G. and Garg, N. (2002) “Subgrade Strains Measured in Full-scale Traffic Tests with Four- and Six-wheel Landing Gears.” FAA Airport Technology Transfer Conference.

HKS, (2006), ABAQUS Theory and Users Manual – Version 6.6, *Hibbit, Karlsson & Sorenson, Inc.*, Pawtucket, Rhode Island.

Johnson, D (2008), Investigation of the Performance of Flexible Pavement Systems under Moving Loads using Finite Element Analyses, Master’s thesis, Rowan University.

Characteristics of Pore Pressure and Volume Change during Undrained Loading of Unsaturated Compacted Granite Soil

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ABSTRACT: A series of triaxial compression tests were performed on samples of compacted granite soils in a modified triaxial cell that can separately control pore air pressure (u_a) and pore water pressure (u_w) in order to examine the characteristics of pore pressure, volume change and stress-strain behavior under undrained loading condition. Unsaturated granite soil samples were prepared by compaction in a mould. These samples were tested at different suction and different confining stresses. The volume change of an unsaturated soil during shearing undrained is much sensitive to the confining pressure compared to the initial water content, and the matric suction. The volume expands during shearing, and the volumetric strain is much larger at the smaller confining pressure and at the higher matric suction. The variation of the internal frictional angle according to the initial water content and the matric suction is negligible, but the effective cohesion increases according to matric suction.

INTRODUCTION

In general, an unsaturated soil generates force that is necessary to absorb water by capillary effect or osmotic suction. Due to this force, the unsaturated soil behaves differently from a saturated soil. This force is defined as total suction. The total suction is divided into the matric suction that is difference between pore air pressure and pore water pressure, and osmotic suction. The osmotic suction occurs usually at special area and special soil, and its value is usually smaller than the matric suction. Meanwhile, the matrix suction is developed by an attractive force between water molecules at contact area between water and air and plays an important role in controlling the shear strength and consequently the stability of many steep slopes.

Thus, the theory of effective stress for saturated soils may not be applicable to such an unsaturated soil.

The state of effective stress of an unsaturated soil can be defined using two sets of independent stress state variables that are the net stress state term, $(\sigma_n - u_a)$ and suction term, $(u_a - u_w)$. This study aimed at investigating the characteristics of pore pressure, volume change and stress-strain behavior of an unsaturated weathered granite soil. A series of triaxial tests with undrained loading condition were conducted to achieve these objects. The tests were performed with different degree of saturation, confining pressure and suction values.

LITERATURE REVIEW

Since the theory of effective stress for an unsaturated soil proposed by Bishop (1959), many researchers have studied on an unsaturated soil. For example, Bishop and Donald (1961) investigated the shear strength of an unsaturated clayey soil. Jenning and Burland (1962) presented that the stress state variables can be divided into the net average stress and suction. Fredlund et al. (1978) proposed the equation of shear strength of an unsaturated soil in terms of two sets of stress variables. Later, he carried out several researches with his coworker, Morgenstern (1976, 1977). Rahardjo et al. (1990, 1995) investigated the characteristics of pore water pressure and volume change of an unsaturated soil in both drained and undrained conditions. Miller and Nelson (1993) published the results of the study on the characteristics of shear strength and relationship between suction and stress state.

The equation of shear strength for an unsaturated soil proposed by Fredlund et al. (1978) is defined as follows.

$$\tau = c + (\sigma - u_a)\tan\varphi' + (u_a - u_w)\tan\varphi^b \qquad (1)$$

where c = effective cohesion; $\sigma - u_a$ = the net normal stress; ϕ = internal frictional angle; $u_a - u_w$ = matric suction; ϕ^b = angle that indicates the increase of the effective cohesion due to the matric suction. Equation (1) indicates that the shear strength of an unsaturated soil increases as the net normal stress or the matric suction increase. The relationship between σ, τ, and S is shown in figure 1. This figure shows that one failure surface is formed as the matric suction increases. The increase of cohesion due to increases of matric suction is defined as follows.

$$c = c' + (u_a - u_w)\tan\varphi^b \qquad (2)$$

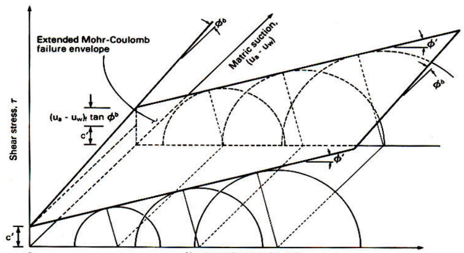

FIG. 1. Extended Mohr-Coulomb failure envelope for unsaturated soils

EXPERIMENT

Material and Specimen

The weathered residual soil used in this study had been sampled at areas around Pocheon Kyungkido, Korea. The soil was air dried in a room temperature, and then filtered through the sieve #4. The basic material properties are shown in Table 1. Three sets of specimens that have the initial moisture content of 11.6 %(OMC), 6.5% (Dry side), 18.5 %(Wet side), and dry density of 1.7(g/cm^3) which is 92% of the maximum dry density were prepared. The specimens have dimensions 100mm height and 50mm diameter.

Table 1 Material properties on the Pocheon granite soil

d_{max}	O.M.C	LL	PL	#200	GS	Li(%)	USCS
1.85	11.6	NP	NP	17.15	2.67	2.80	SM

Matric Suction

Matric suction of a soil can be measured by means of several methods such as Filter Paper method, Thermocouple Psychrometers method, and Thermal Matric Potential Sensors method etc. Of them is Tensiometer method, which is able to measure in-situ using a fine porous ceramic sensing tip. However, since pore water pressure in an unsaturated soil is negative, it is almost impossible to measure the pore water pressure in the condition of high suction. In this case, a high air entry ceramic disk, or a membrane should be used. The other method is to use a pressure plate apparatus, which adopted the axis translation method developed by Hilf (1956).

In this study, a pressure plate apparatus with a high air entry membrane and a porous ceramic stone was used. The high air entry membrane and the porous ceramic stone allow nothing but water to flow through. Experiments were performed based on the reference, ASTM D 2325.

Triaxial Consolidated Undrained Compression Test

Triaxial tests for unsaturated soils were carried out using the modified triaxial compression apparatus that can control suction value. The suction was applied by means of axis-translation technique to avoid cavitations. In this technique, the air pressure (u_a) and backwater pressure (u_w) were applied on the soil sample. The difference between the air pressure and the backwater pressure applied on the sample is taken as the applied suction ($u_a - u_w$). In the study, the air pressure was applied to the top of the sample whereas the backwater pressure was applied to bottom of the soil sample. The suction applied is not to exceed the air entry value of the high air entry ceramic disc at the cell.

A series of undrained triaxial tests with 3 different confining pressure (1, 2, 4 kgf/cm^2), and 3 different suction values (0.5, 1.9, 2.0 kgf/cm^2) were performed. The rate of strain set to be 0.1%/min. The detail of the test procedure is as follows.

A specimen is first consolidated with applying constant suction pressure. After having done the consolidation, the specimen is subjected to deviator stress. If the initial moisture content of the specimen is, during this stage, different from the matric suction, the deviator stress is applied after the specimen has reached the equilibrium state. In order to confirm whether or not the equilibrium state has been reached, the moisture content with respect to time is monitored. The minimum time that requires the specimen to be stable is then evaluated. The specimen is subjected to the deviator stress after time that is three times as long as the minimum time.

RESULTS AND DISCUSSION

Stress-Strain Behavior

Deviator stresses and volume changes with respect to axial strain for each case are shown in figure 2. As indicated, the deviator stresses appear to decrease as the initial water contents increase.

The initial tangent seems to be a little stiff for the case that the initial water content is OMC than the wet side. In addition, the maximum deviator stress occurs at small strain. The deviator stress for the initial water content being dry side is less sensitive to the matric suction than that in wet side. That might be because of the attractive force between water molecules at contact area. The volumetric stain appears to be hardly affected by the initial water content. For the smaller confining pressure, the volume expands during shearing, while the volume dilates for the larger confining pressure at about 10% of axial strain. In case or the same confining pressure, similar phenomenon may occur at the larger matric suction.

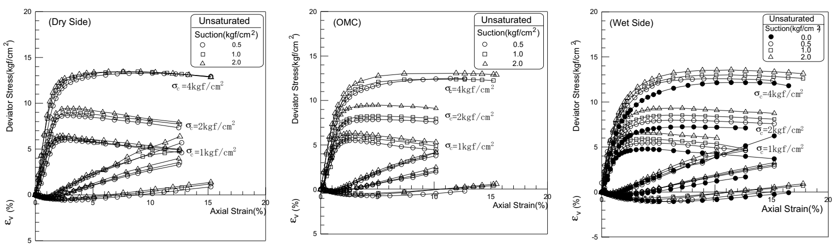

FIG. 2. Consolidated undrained test on unsaturated granite soil

Effective Cohesion

Figure 3 plots the test results in the space of net normal stress, matric suction, and shear stress. According to this figure, as the matric suction increases it forms a failure surface in the space. This is the Mohr-Coulomb failure surface at constant matric suction. There seems to be small changes in internal frictional angle appears to be almost constant according to the matric suction, while effective cohesion seems to increase. The increases of the effective cohesion for each case are ϕ^b=3°(dry side), 8° (OMC), and 9°(wet side).

Figure 4 shows the relationships between the matric suction and the cohesion. In this figure, the solid line represents a linear regression, and the dotted line represents a polynomial regression. As we can see, the cohesion for dry side of the initial water content appears to be 0.98 (linear regression), and 0.96 (polynomial regression). In the other two cases, we can hardly see difference between the linear and the polynomial regression. Thus, we may conclude that it is really not matter which regression analysis is chosen, when evaluates a slope that indicates increase of the effective cohesion due to increase of the matric suction.

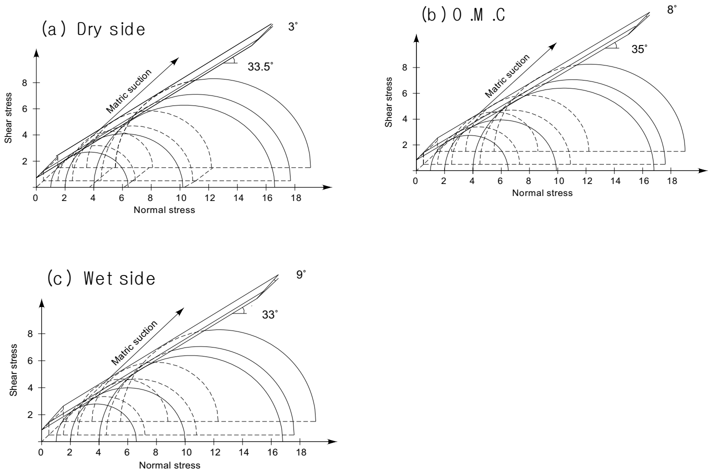

FIG. 3. Extended Mohr-Coulomb failure envelope for consolidated undrained test on the unsaturated Pocheon granite soil

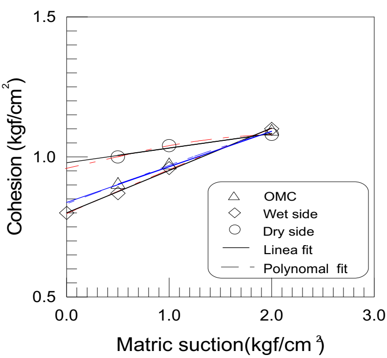

FIG. 4. Relationships between the matric suction and the cohesion for consolidated undrained tests

Pore Air Pressure

Since the volume of an unsaturated soil varies during shearing undrained, and the void is filled with air and water, it may be important to bring attention to the mechanical characteristics of pore air pressure and water pressure. The changes in pore air pressure with respect to axial strain with different initial water contents, confining pressures, and matric suctions are shown in figure 5. When the confining pressure is 1 kgf/cm^2, the pore air pressure appears to be positive values at the only beginning part of the axial strain. When the confining pressure is 4 kgf/cm^2, the values of the pore air pressure become positive during almost entire test. And they appear to be higher as the matric suctions increase, and the initial water content of the wet side is larger.

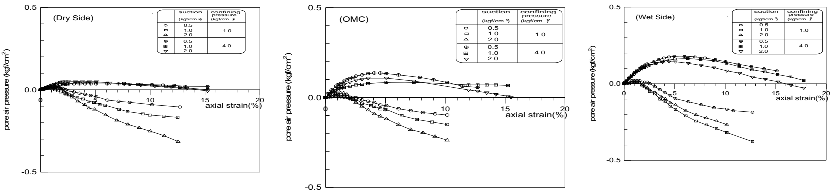

FIG. 5. Variation of pore air pressure with respect to axial strain with different initial water contents, confining pressures, and matric suctions for consolidated undrained tests

Pore Water Pressure

The changes in pore water pressure with respect to axial strain with different initial water contents, confining pressures, and matric suctions are shown in figure 6. The pore water pressure appears to be higher at the confining pressure of 1 kgf/cm^2 than the confining pressure of 4 kgf/cm^2. In case of the same confining pressure, the higher the matric suction the larger the pore water pressure. For instance, for the matric suction of 0.5 kgf/cm^2, the pore water pressure appears to be negative after certain amount of axial strain.

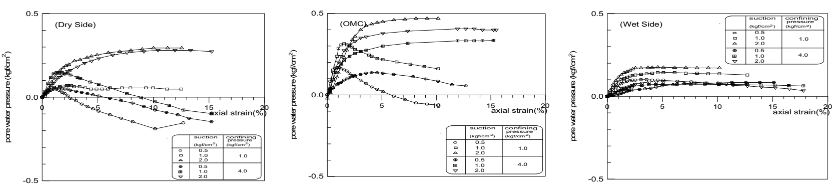

FIG. 6. Variation of pore water pressure with respect to axial strain with different initial water contents, confining pressures, and matric suctions for consolidated undrained tests

CONCLUSIONS

In order to investigate the characteristics of pore pressure, volume change, and stress-strain behavior according to an initial water content of an unsaturated soil, a series of consolidated undrained experiments were conducted. Based on the investigations, we obtained the following results.

The volume change of an unsaturated soil during shearing undrained is much sensitive to the confining pressure compared to the initial water content, and the matric suction.

The matric suction has more influence to the deviator stress, when initial water content is wet side. In most case, the volume expands during shearing. The volumetric strain is much larger at the smaller confining pressure, and at the higher matric suction.

The change of the internal frictional angle according to the initial water content and the matric suction is negligible. However, the effective cohesion appears to increase.

REFERENCES

Bishop, A. W. (1959). “The principle of effective stress.” Technisk Ukeblad, 106(99): 859-863

Bishop, A. W. and Donald, I. B. (1961). “The experimental of partly saturated soil in the triaxial apparatus.” *Proceeding of the 5th int. conference on soil mechanics and foundation engineering,* 1: 13-21

Fredlund, D. G. and Morgenstern, N. R. (1976). “Constitutive Relations for Volume Change in Unsaturated Soils.” *Canadian J. Geotechnical*, 13:261-276

Fredlund, D. G. and Morgenstern, N. R. (1977). “Stress State Variables for Unsaturated Soils.” *Pro. ASCE*, 103(5):447-466

Fredlund, D.G., N.R. Morgenstern, and R.A. Widger (1978) The shear strength of unsaturated soils. Canadian Geotechnical Journal. 15(3):313-321

Hilf, J. W. (1956). “An investigation of pore water pressure in compacted cohesive soils.” US Bureau of Reclamation, Tech. Mem. 654, Denver, US Bureau of reclamation.

Jenning, J. E. and Burland J. B. (1962). “Limitation to the use of effective stress in partly saturated soils.” *Geotechnique*, 12(2):125-144

Ladd, R. S. (1978). “Preparing Test Specimens Using Under compaction.” *Geotechnical Testing Journal, GTJODJ*, 1(1): 16-23

Miller, D. J. and Nelson, J.D. (1993). “Osmotic Suction as a Valid Stress State Variable in Unsaturated Soil Mechanics.” Unsaturated Soils, *Geotechnical Special Publication* No.30: 64-76

Nelson, J. D. and Drumright, E. E. (1995). “The Shear Strength of Unsaturated Tailings Sand.” Unsaturated Soils, 1: 45-50

Rahardjo, H. (1990). “The Study Undrained and Drained Behavior of Unsaturated Soil.” Ph. D. dissertation, univ. Saskatchewan, Canada.

Rahardjo, H. and Fredlund, D.G. (1995). “Pore Pressure and Volume Change Behavior During Undrained and Drained Loadings of an Unsaturated Soil.” *Proceedings of the First International Conference on Unsaturated Soils*: 165-170

Sonic Wave Testing Technique in Lamellar Rock Mass

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ABSTRACT: Sonic wave testing technique is the most commonly used one for ascertaining the loosening zone of roadway surrounding rock, but it has a great limitation in lamellar rock mass. For overcoming the limitation, the problems of sonic wave testing boreholes arrangement mode were studied. Firstly, the theoretical bases of the testing boreholes arrangement in inclined lamellar rock mass were analyzed on the basis of the mechanical characteristics and the propagation rule of sonic wave in layered rock mass. Secondly, three principles of sonic wave testing boreholes arrangement were summarized from numerous testing experiences on field. Then, new arrangement modes in four representative lamellar rock mass (dip angle was respectively 0°, 15°, 45° and 90°) were proposed. The testing results from the engineering example with the gently inclined lamellar rock mass showed the improved arrangement mode could determine the scope of the loosening zone quickly.

INTRODUCTION

The scope of the loosening zone in roadway surrounding rock is an important basis for stability evaluation and support design (Huo et al. 1994). There are many mehods to ascertain the scope of the loosening zone, such as sonic wave testing mehod, seismic wave method, GPR method and so on, and the sonic wave testing method is the most commonly used one. The single-borehole testing method in sonic wave testing method has been widely applied in underground rock engineering by its simplicity in the testing process and less work load than the double-borehole testing method. Fig.1 shows the conventional testing boreholes arrangement mode. If the singel-borehole testing method along the conventional mode to ascertain the scope of the loosening zone in lamellar rock mass, the change of the relationship curves (V_p-L) between p-wave velocity (V_p) and borehole depth (L) would present the irregular feature due to weak interlayers. Only by the curves, the scope of the loosening zone could not be ascertained difficultly.

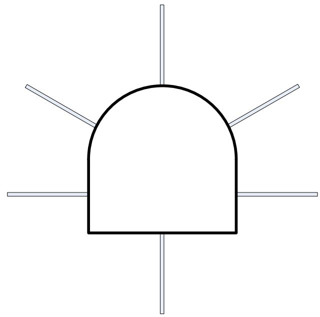

FIG. 1. Conventional testing boreholes arrangement mode.

THEORETICAL BASES OF TESTING BOREHOLE ARRANGEMENT

Theoretical Foundation of Sonic Wave Detection Technique

If rock mass is regarded as the elastic body, unit motion equations can be list as (Chen 1990)

$$\begin{cases} \dfrac{\partial \sigma_{xx}}{\partial x} + \dfrac{\partial \sigma_{yx}}{\partial y} + \dfrac{\partial \sigma_{zx}}{\partial z} = \rho \dfrac{\partial^2 u}{\partial t^2} \\ \dfrac{\partial \sigma_{xy}}{\partial x} + \dfrac{\partial \sigma_{yy}}{\partial y} + \dfrac{\partial \sigma_{zy}}{\partial z} = \rho \dfrac{\partial^2 v}{\partial t^2} \\ \dfrac{\partial \sigma_{zx}}{\partial x} + \dfrac{\partial \sigma_{zy}}{\partial y} + \dfrac{\partial \sigma_{zz}}{\partial z} = \rho \dfrac{\partial^2 w}{\partial t^2} \end{cases} \tag{1}$$

According to the Hooke law, the relationship formula of stress and strain is expressed as

$$\begin{cases} \sigma_{xx} = \lambda\Delta + 2\mu\varepsilon_{xx} \\ \sigma_{yy} = \lambda\Delta + 2\mu\varepsilon_{yy} \\ \sigma_{zz} = \lambda\Delta + 2\mu\varepsilon_{zz} \\ \sigma_{xy} = \mu\varepsilon_{xy} \\ \sigma_{yz} = \mu\varepsilon_{yz} \\ \sigma_{zx} = \mu\varepsilon_{zx} \end{cases} \tag{2}$$

Where ρ is density of medium, u, v and w are the displacement in the direction of x, y and z, $\Delta = \varepsilon_{xx} + \varepsilon_{yy} + \varepsilon_{zz}$ is volumetric strain, λ and μ are Lame coefficient.

From Eq. (1) and Eq. (2), the following equation can be derived as follows: