Jour. of the KGS, Vol. 21, No. 7. September 2005, pp. 31~42

Shear Strength and One-dimensional Compression Characteristics of Granitic Gneiss Rockfill Dam Material

화강편마암 댐 축조재료의 전단강도 및 일차원 압축특성

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요 지

본 연구에서는 락필댐 축조재에 대하여 대형 삼축압축시험 및 대형 오이도미터시험을 실시하고 재료의 전단강도 및 압축 특성에 관하여 조사하였다. 삼축압축시험시 재료의 다짐도 및 구속압을 변화시켜 이에 따른 재료의 전단거동 및 강도특성을 조사하였고 오이도미터 시험을 통하여 일정 단위중량의 입도분포 및 함수상태가 다른 재료의 파쇄 및 압축성을 관찰하였다. 삼축압축시험결과 다짐도 및 구속압에 따라 재료의 전단팽창성에 차이가 나타나고 이들 조건들이 전반적인 거동 및 전단강도에 영향을 미치는 것으로 나타났다. 오이도미터시험결과 재료의 평균입자크기의 증가와 함께 파쇄율이 증가하고 입도분포가 상대적으로 양호한 시료의 압축성이 적은 것으로 나타나 일반적인 조립재의 압축특성을 잘 반영하는 것으로 나타났다. 그리고 포화시료가 건조시료에 비하여 다소간 높은 압축성을 나타내었으며 접선구속계수 산정결과 포화시료의 계수값이 건조시료에 비하여 평균적으로 10~20% 큰 것으로 나타났다.

Abstract

In this study, a rockfill-dam material was investigated on its shear strength and compressibility by performing large-scaled triaxial and oedometer tests. The rockfill material was compacted at two different compaction levels and sheared in triaxial compression at three different confining stresses. Also, rockfill samples were prepared to have three different grain size distributions but the same dry density. Each sample with a given grain size distribution was then compressed one-dimensionally in a large-scaled oedometer cell with and without soaking. The rockfill samples exhibited slightly different shear behaviors with the varying compaction and confining stress levels. The increase in the compaction level changed the behavior from contractive to dilative. Dilation decreased gradually with increasing confining stress, resulting in reduction in the peak shear strength. The large-scaled oedometer test results showed that particle breakages increased with increasing average particle sizes of the samples. Comparing the samples with different gradations, a relatively well-graded sample exhibited lower compressibility. For saturated samples, slightly higher deformations were observed, compared to dry samples. The values of tangent constrained modulis for the dry samples were larger by about 10 to 20%, on the average, than those for the saturated samples.

Keywords : Compressibility, Large-scaled oedometer test, Particle breakage, Rockfill dam, Shear behavior

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1. Introduction

Among large multi-purpose dams in Korea, fill dam is the most common type, which accounts for about 90% of the existing dams (KNCOLD, 2004). Most of them are earth-core rockfill dams (ECRD) in which rockfill material is used as a main fill material. In recent years, concrete-face rockfill dam (CFRD) has gained popularity as a type of new dams, mainly due to its economic advantage. For the ECRD or CFRD, its overall stability is typically governed by that of rockfill zone, since it comprises most of the dam section. Understanding mechanical behavior of the rockfill material, therefore, is crucial to more accurate evaluation of the dam stability.

Although there have been many investigations into the behavior of soil, such as sand and clay, the studies of rockfill material are currently very limited in Korea, probably due to limitations on the test equipments available for the material (Shin et al., 1999). The data resources from foreign dams have been often used directly in designing, building, and evaluating the stability of domestic dams (KOWACO, 2004). To be more reasonable in the applications to domestic dams, accordingly, it is necessary to examine mechanical properties of the rockfills used actually for construction of domestic dams.

In this study, a granitic gneiss rockfill material, obtained from a CFRD construction site in Korea, was investigated on its mechanical characteristics, including compaction, shear strength, and compressibility, by performing large-scaled triaxial and oedometer tests. It is believed that the test results would provide the users of the material with useful information on assessing its mechanical properties and contribute to the data resources for domestic rockfill dam applications.

2. Materials and Methods

2.1 Testing Materials

Testing materials used in this study were rockfill materials of which parent rock type was granitic gneiss. They were sampled from the quarry located downstream of P dam in Kangwondo, Korea. Fig. 1 shows the rockfill materials classified by their sizes, varying from 0.074 to 100 mm. In general, they were light gray with angular shapes.

To examine basic properties of the material, specific gravity test, water absorption test, and point load test were conducted, as described by ASTM C 127 (ASTM, 1984) and ASTM D 5731 (ASTM, 2002), respectively. Table 1 displays the values of measured average specific gravity, water absorption ratio, and the uniaxial compressive strength inferred from the point load test results.

2.2 Compaction Test

Compaction tests were performed to examine compaction characteristics of the rockfill material and to determine its dry density for the large-scaled triaxial test.

Since rockfill material is a granular material, parameters to define physical states of granular soils can be used. Relative density is a measure of the relative compactness with respect to the densest and loosest possible states for the granular material. The relative density can be used

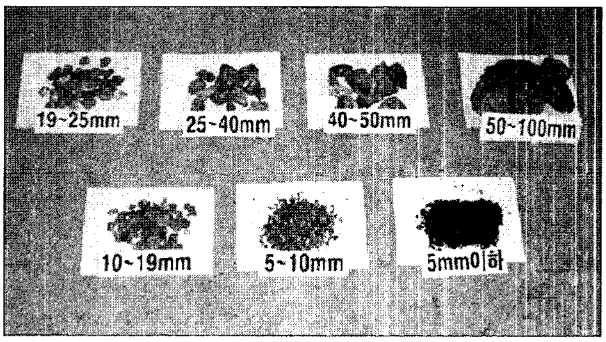


Fig. 1. Tested materials

Table 1. Basic properties of rockfill material

Parent rock type	Specific gravity	Water absorption ratio	Unconfined compressive strength (MPa)
Granitic Gneiss	2.72	0.59	152

to control field compaction. Similarly, relative compaction, defined as the ratio of a compacted dry unit weight to the maximum dry unit weight obtained from the laboratory compaction test, can also be used as an alternative to relative density for controlling compaction (Selig and Ladd, 1973).

Although, for rockfill materials, no standard method for determination of relative density (i.e., maximum and minimum density) currently exists, following methods are generally used: 1) rammer compaction, 2) vibratory compaction, and 3) static compression for the maximum density determination, and 1) manual placement by hand for the minimum density determination (Shin et al., 1999).

In this study, the rammer compaction method was used to determine the maximum density of the material. A large-scaled automatic compaction apparatus, designed to compact a large sample with 300 mm in diameter and 620 mm in height, was used for the purpose.

A rockfill material was placed dry in a split mold with 300 mm in inside diameter and 620 mm in inside height and compacted in layers using the rammer with 130 mm in diameter and 157 N in weight. The compaction tests were conducted on the samples by varying compaction energy (i.e., varying the rammer falling height and number). The maximum density value was determined from a hyperbolic curve fitting for the relationship between the compaction energy and the corresponding dry density, as shown in Fig. 2.

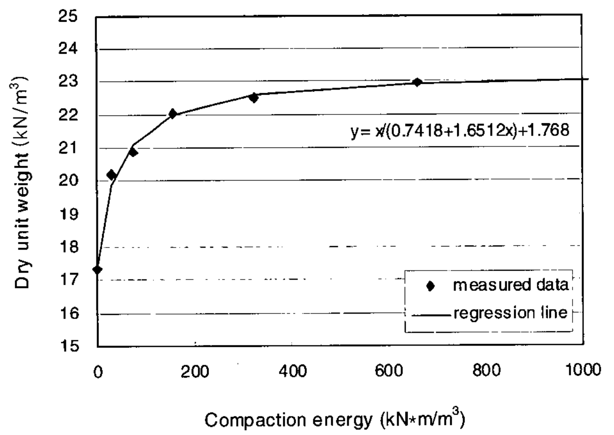


Fig. 2. Compaction energy versus dry unit weight of rockfill material

Table 2 shows the values of the maximum and minimum density obtained.

Table 2. Maximum and minimum dry densities of rockfill material

$\gamma_{d(max)}$ (kN/m^3)	$\gamma_{d(min)}$ (kN/m^3)
23.30	17.22

2.3 Large-Scaled Triaxial Test

In order to investigate stress-strain and volumetric behaviors of rockfill materials under shearing and to determine their shear strength, isotropically consolidated-drained (CD) triaxial tests were performed on the rockfill samples.

The equipment used in the test was the large-scaled triaxial testing system, owned by Korea Institute of Water and Environment of Korea Water Resources Corporation (Fig. 3). It was designed to test a sample as large as 300 mm in diameter and 620 mm in height. The system consists of a loading frame, a triaxial chamber, a vertical loading unit, a hydraulic (oil pump) unit, an air pressure and water control unit, and an electrical measuring and control unit.

The testing apparatus uses an electro-hydraulic servo control system. The axial loading is applied through an oil piston with the diameter of 80 mm, by a static servo controller. The axial load is measured with a load cell. The axial deformation and volume change are measured using linear variable differential transformers (LVDT) placed on top of the chamber and double tube burret, respectively. Pore pressure and confining pressure are

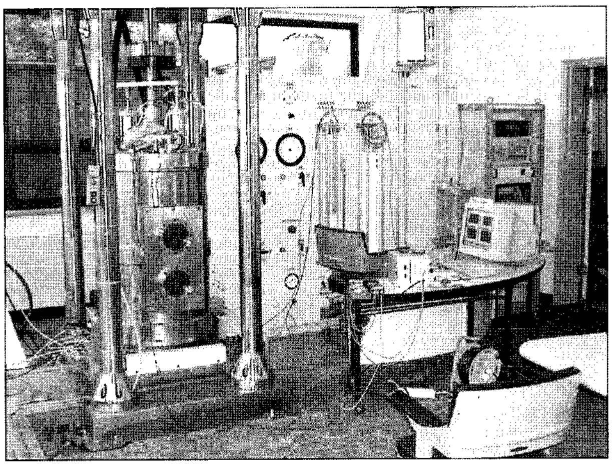


Fig. 3. Large-scaled triaxial testing system

measured with another transducer on bottom of the chamber. The test is computer-controlled, and digital data of stress-strain and volume change are recorded automatically by a digital data acquisition system.

2.3.1 Test Condition

Fig. 4 shows the grain size distribution of the rockfill material used for the large-scaled triaxial test. The grain size varied from about 0.075 to 50.8 mm.

The test was intended to investigate the effects of compaction levels and confining stresses on the behavior and shear strength of the rockfill materials. Hence, two levels of relative compaction (RC) (85 and 90% of $\gamma_{d(max)}$, respectively) were used and, at each compaction level, three tests were performed, each at a specific confining stress level (196, 392, and 588 kPa, respectively).

A summary of the test condition is given in Table 3.

2.3.2 Procedure

A latex membrane was first applied to the same split mold as used in the compaction test. A rockfill sample was then compacted in five layers in the split mold using the compaction rammer of the automatic compaction apparatus. To achieve a known density of the specimen, blows per layer were determined by several trials. Following compaction of the last layer and trimming the top of the compacted sample, the split mold was moved to the triaxial chamber using a fork-lift and carefully removed. A suction pressure of about 20 kPa was applied to the specimen through a vacuum line to prevent the sample from collapsing when removing the split mold.

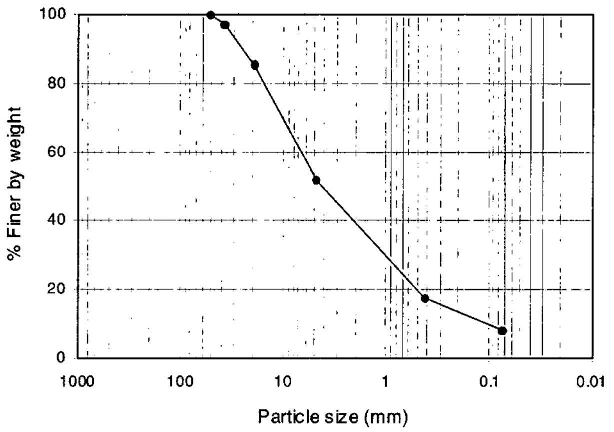


Fig. 4. Particle size distribution of rockfill material used in triaxial test

After the specimen setting, de-aired water was allowed to percolate under a pressure of about 20 kPa plus an elevation head of 1.6 m for the specimen saturation. The saturation was continued for more than 24 hours, until a B-value higher than 0.95 was achieved.

Upon completion of the saturation, the specimen was isotropically consolidated by applying a desired effective confining stress and a period of time ranging from 30 to 60 minutes was allowed for the specimen to have enough time to fully dissipate the generated pore pressure and reach primary consolidation.

Triaxial compression (shearing) was performed on the saturated specimens previously consolidated to a given effective confining stress under a strain-controlled condition in the strain rate of 3 mm/min.

2.4 Large-scaled Oedometer Test

Large-scaled oedometer tests were conducted to investigate the compressibility of the rockfill materials. For a rockfill dam of large lateral extent, the compression behavior of a rockfill layer may be considered one-dimensional. Since the standard one-dimensional compression test apparatus (e.g., as described in ASTM 2435

Table 3. Consolidated-drained (CD) triaxial test conditions

Dry unit weight γ_d (kN/m^3)	Relative density (RD) (%)	Relative compaction (RC) (%)	Confining stress σ'_3 (kPa)
19.80	0.50	85.0	196
19.77	0.49	84.9	392
19.86	0.51	85.2	588
21.11	0.71	90.6	196
21.11	0.71	90.6	392
21.12	0.71	90.6	588

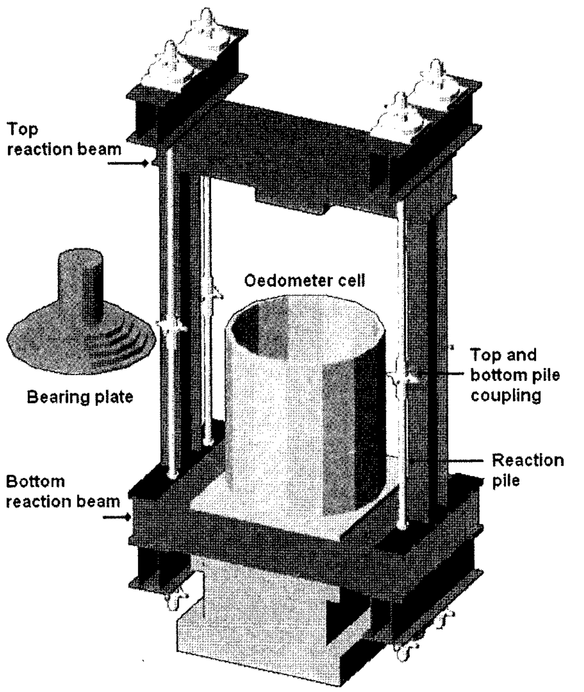


Fig. 5. Schematic diagram of large-scaled oedometer testing apparatus

(ASTM, 1996)) is not applicable to rockfill material due to its large grain size, a large-scaled oedometer test apparatus was used in this study.

The large-scaled oedometer apparatus, developed by Geotechnical lab. of Aju university, consists mainly of a large oedometer cell with 980 mm in diameter and 1000 mm in height, a bearing plate, perforated plates, an oil pressure cylinder, a hydraulic pump, and reaction beams (Fig. 5). With the apparatus, a sample can be loaded up to about 2.6 MPa. To measure the load applied and deformation of samples, four ±50 mm LVDTs, a 2 MN axial load cell, and a data logger (TDS-302) were used.

2.4.1 Test Condition

Fig. 6 shows the grain size distributions of the rockfill materials used for the large-scaled oedometer tests. The

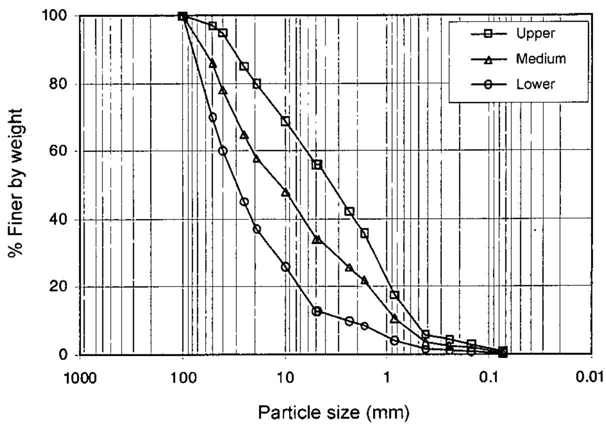


Fig. 6. Particle size distributions of rockfill material used in 1-D compression (oedometer) test

materials with particle sizes varying from 0.074 to 100 mm were tested. To examine the effects of different grain sizes and gradations on the compressibility, the samples with three different gradations (i.e., upper, medium, and lower grain size distributions, respectively) were prepared (see Fig. 6 and Table 4). The samples for the medium and lower grain size distributions were better-distributed, compared to the upper grain size distribution sample. An average grain size was the largest for the lower grain size distribution and the smallest for the upper grain size distribution. For each grain size distribution, two samples were made and tested in different conditions, dry and soaked. The differences of compressibility between each case were investigated.

Table 5 shows a summary of the test condition.

2.4.2 Procedure

For a dry sample, a pre-determined amount of materials (in dry) was put into the oedomter cell and compacted using a vibratory rammer. The compaction was continued in layers until a final height of 500 mm was reached. Upon completion of the compaction, loadings were

Table 4. Grain sizes of rockfill material used in oedometer test

Grain size range	Maximum grain size (mm)	< 50 mm (%)	< 10 mm (%)	< 4.76 mm (%)	< 0.074 mm (%)	C_u (coefficient of uniformity)	C_c (coefficient of curvature)
Upper	100	97	69	56	0.83	11.5	0.7
Medium	100	86	48	34	0.51	26.9	1.0
Lower	100	70	26	13	0.19	16.3	1.9

Table 5. One-dimentional compression (oedometer) test conditions

Sample state	Grain size range	γ_d (kN/m^3)	Test procedure
Dry	Upper	22.2	loading and unloading at 245, 491, 736, 982, and 1,472 kN, respectively
	Medium	22.2	
	Lower	22.2	
Saturated	Upper	23.2	
	Medium	22.9	
	Lower	22.7	

applied incrementally following the placement of the bearing plate on top of the sample.

For a saturated sample, two perforated plates were placed at the bottom and top of the sample, respectively. Prior to loading, water was supplied to the sample so that a constant water level was kept in the above the sample within the cell.

After the sample setting, the specimen was loaded according to the loading procedure as given in Table 5.

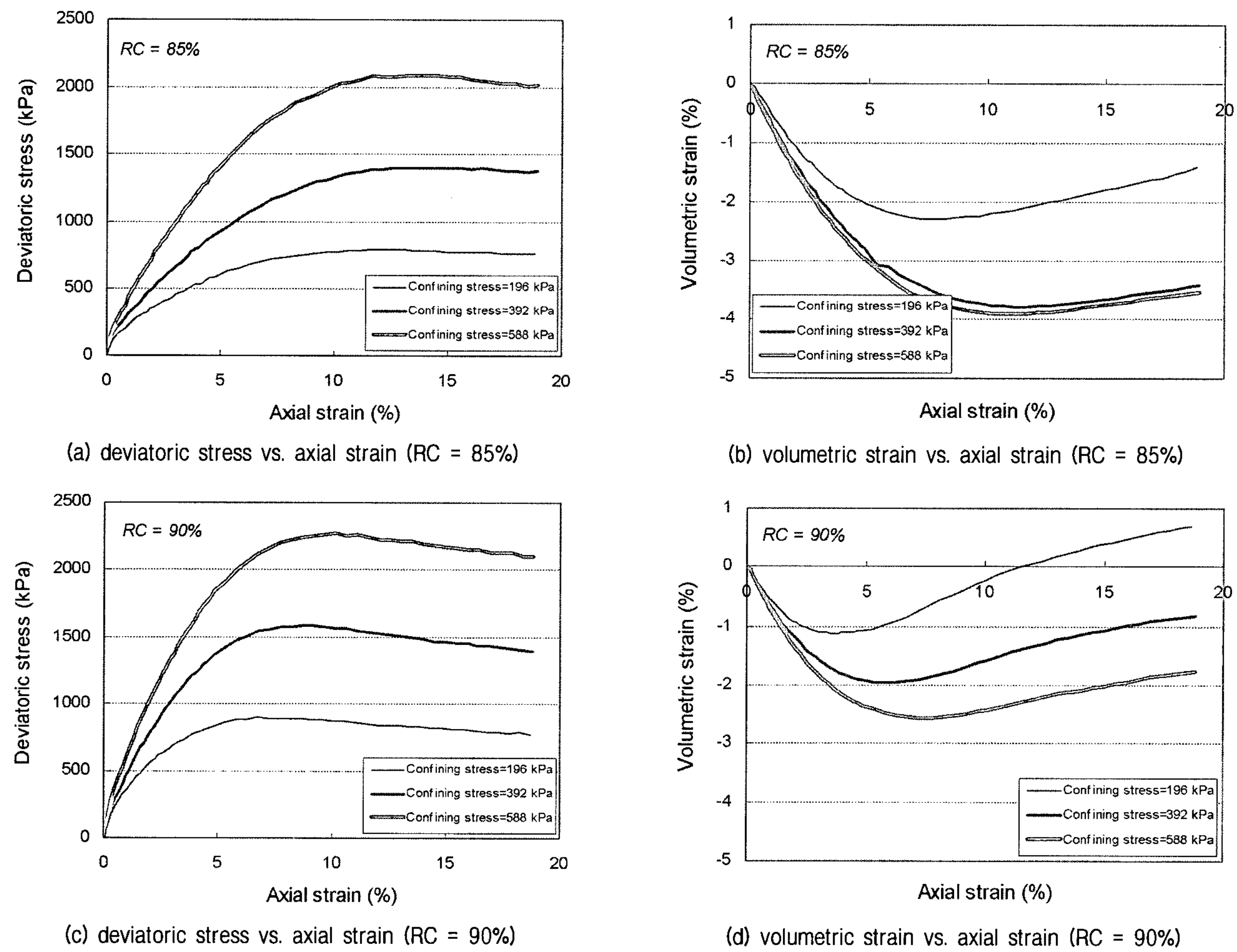
3. Results and Discussion

3.1 Shear Strength Characteristics

3.1.1 Stress-strain and Volumetric Behavior

Fig. 7 shows stress-strain and volume change behaviors of rockfill samples compacted at two different compaction levels (i.e., RC = 85 and 90%) and sheared in triaxial compression at three different confining stresses (i.e., σ'_3 = 196, 392, and 588 kPa) under drained conditions.

For the specimens compacted to RC = 85% (Fig. 7 (a) and (b)), the stress-strain and volumetric behavior typically resembled those of sandy soils in loose to medium conditions. The deviatoric stress (σ'_d) increased gradually up to a peak level and then stayed practically unchanged with increasing axial strain (ε_a). The volumetric strains (ε_v) were mostly contractive throughout shearing. Only mild volume expansion was observed in higher ε_a



(a) deviatoric stress vs. axial strain (RC = 85%)

(b) volumetric strain vs. axial strain (RC = 85%)

(c) deviatoric stress vs. axial strain (RC = 90%)

(d) volumetric strain vs. axial strain (RC = 90%)

Fig. 7. CD triaxial test results

(i.e., after about ε_a = 10%). The contraction increased as σ'_3 increased from 196 to 588 kPa confining stress.

The specimens at RC = 90% (Fig. 7 (c) and (d)) behaved similar to sandy soils in medium to dense conditions. Increases in the deviatoric stress (σ'_d) were associated with a slight initial volumetric contraction, followed by a gradually increasing rate of volume expansion (dilation). The peak strength occurred when the rate of change of the volumetric strain with respect to the axial strain ($d\varepsilon_v/d\varepsilon_a$) reached its maximum value. The post-peak reduction in σ'_d is associated with a decreased rate of dilation until the stress state reaches the critical state with constant stress and volume. Dilation decreased gradually with increasing σ'_3.

3.1.2 Peak Shear Strength

Rockfills are frictional materials. The peak friction angle (ϕ'_p) can be expressed in terms of the principal effective stresses at peak state based on the Mohr-Coulomb failure criterion with zero cohesion intercept as follows:

$$\sin\phi'_p = \left(\frac{\frac{\sigma'_1}{\sigma'_3} - 1}{\frac{\sigma'_1}{\sigma'_3} + 1} \right) \qquad (1)$$

where σ'_1/σ'_3 = effective principal stress ratio or stress obliquity. For dilative behavior, ϕ'_p is associated with the maximum rate of dilation, which normally develops at relatively small strains (Wood, 1990 ; Salgado et al., 2000). For ideal contractive behavior, ϕ'_p coincides with the critical state friction angle ϕ'_c, occurring at large strains. Thus, ϕ'_p can be considered simply as the summation of two components: one due to dilatancy effects, which varies with shear strain (or axial strain in a triaxial test) and the other due to ϕ'_c.

The peak friction angle of the rockfill materials was found to be a function of the relative compaction (RC) and the confining pressure (σ'_3). As shown in Fig. 8, the reduction of RC from 90 to 85% led to a slight drop in ϕ'_p. Also, ϕ'_p decreased as σ'_3 increased from 196 to 588 kPa. The reduction in ϕ'_p with decreasing RC and increasing σ'_3 is due to reduced dilatancy. The values of ϕ'_p ranged from about 40° to 44° with the varying RC and σ'_3.

3.2 One-dimensional Compression Characteristics

3.2.1 Crushing Rate

To examine the degree of crushing occurring in the rockfill materials during the 1-D compression loadings, particle size analyses were performed on the samples after the oedometer tests. Fig. 9 presents the particle size distribution curves of the materials before and after the oedometer tests (i.e., pre- and post-particle size distribution curves).

For a granular material, the degree of particle crushing due to loadings is estimated typically by measuring 1) the increased amount of fines smaller than the minimum grain size before crushing, 2) the difference between pre-

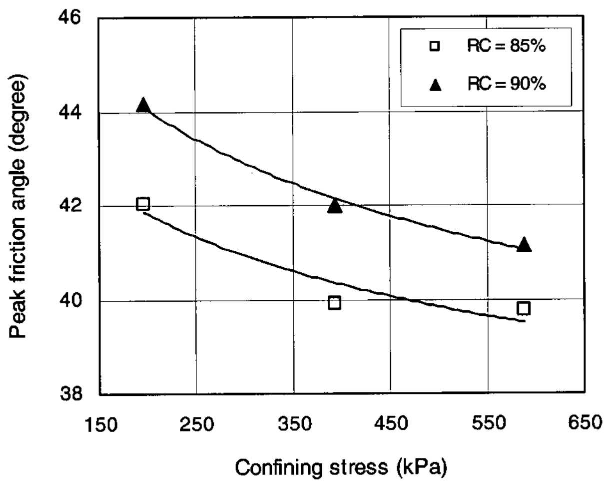


Fig. 8. Peak friction angle versus confining stress of rockfill material

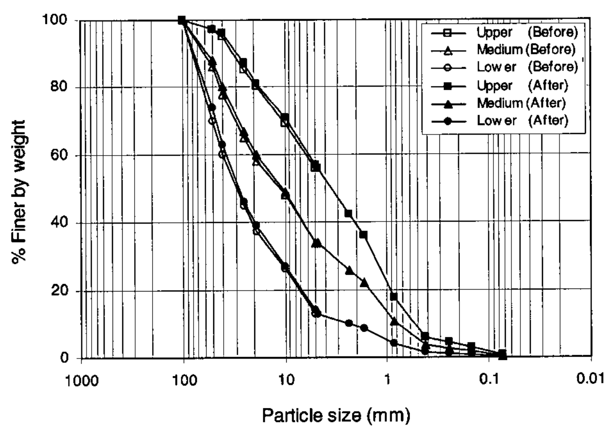


Fig. 9. Particle size distributions of rockfill material before and after 1-D compression test

and post-particle size distributions, and 3) the change in specific surface area between before and after the loadings (Marsal, 1973).

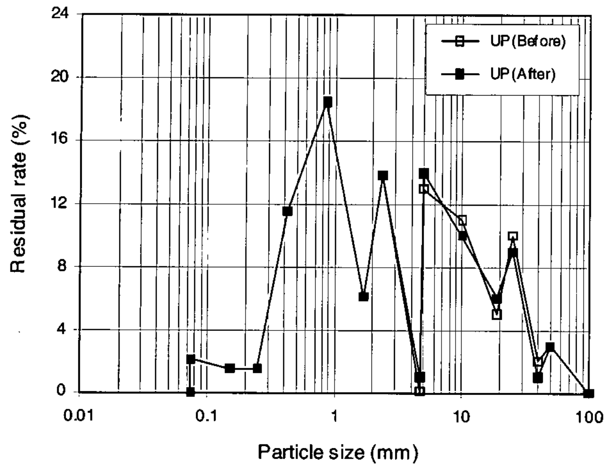
In this study, the differences between the pre- and post-particle size distributions were measured for the rockfill materials with different grain size distributions. For each distribution, the crushing rate was obtained by calculating the ratio of the difference between the areas below the pre- and the post-particle size distribution curves to the area below the pre-particle size distribution curve.

Fig. 10 shows the residual rates of the samples (i.e., percentages retained on each sieve) for the three particle size distributions (upper, medium, and lower particle size distributions) before and after the tests. It was found that the changes in particle size occurred mostly for the particles larger than 4.75 mm. Also, as can be seen in Fig. 11, the crushing rate of the samples was the highest for the lower grain size distribution sample and the lowest for the upper grain size distribution sample, indicating that the crushing rate increased slightly with increasing grain size.

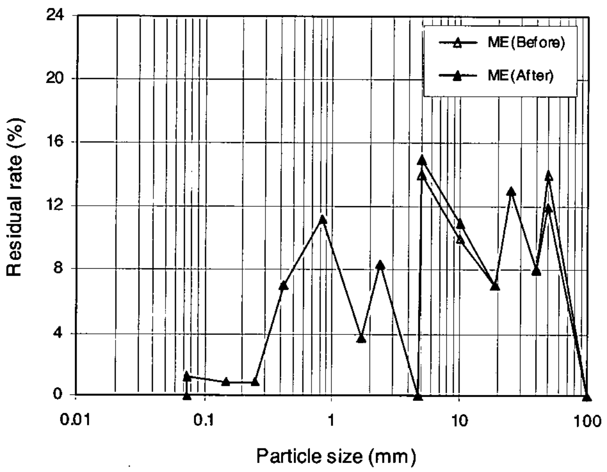
3.2.2 Vertical Stress vs. Vertical Strain

Fig. 12 (a)~(f) shows the compression curves (i.e. vertical stress-strain relationships) of the rockfill materials with three different grain size distributions tested in both dry and saturated conditions. For all the samples, the maximum vertical strains (occurring at vertical stress of about 2 MPa) ranged from about 3 to 5%.

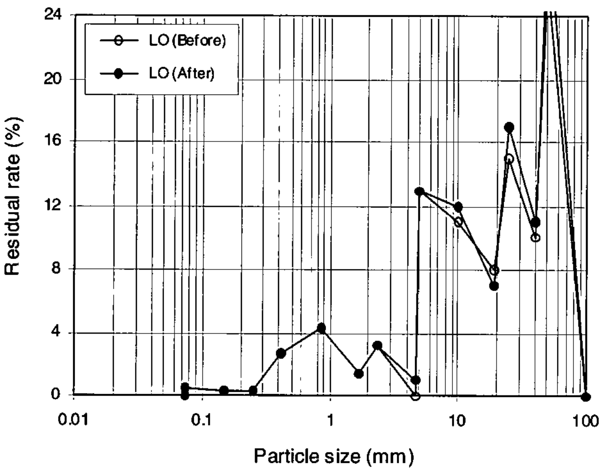
In a granular material, deformation is caused by two mechanisms : 1) distortion and crushing of individual



(a) upper grain size



(b) medium grain size



(c) lower grain size

Fig. 10. Residual rate of rockfill material

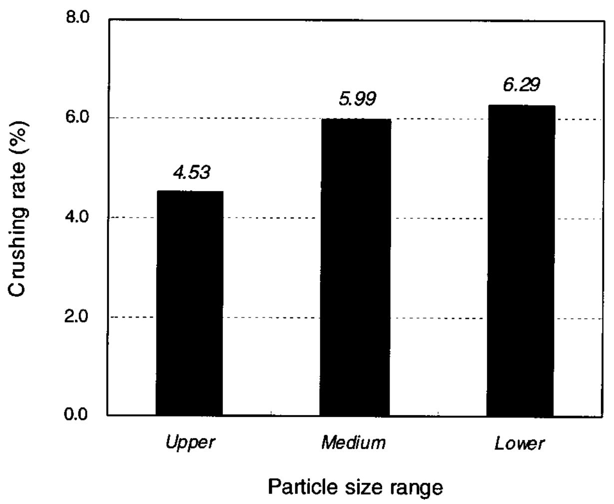
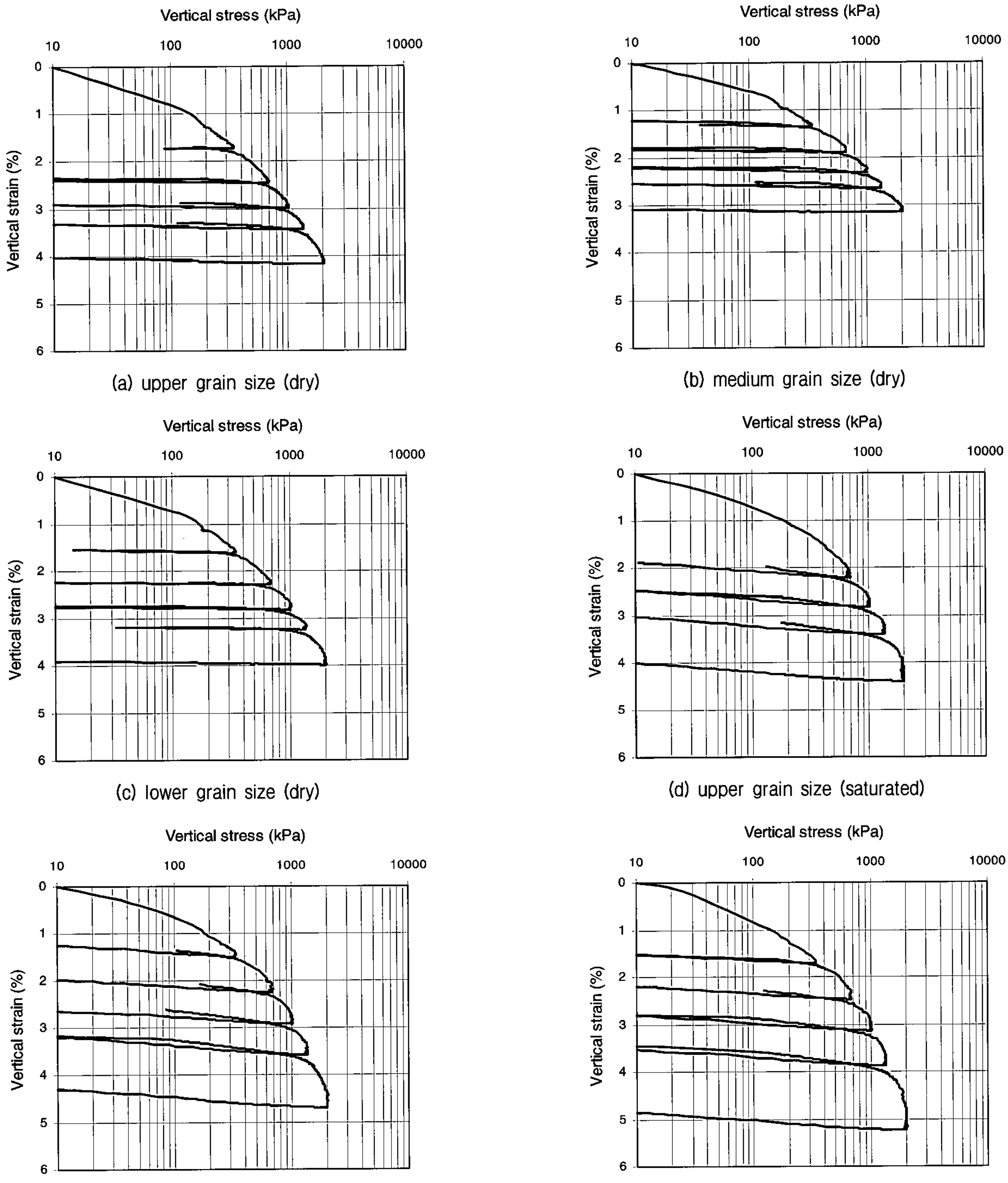


Fig. 11. Crushing rate of rockfill material

particles, and 2) relative motion between particles as the result of sliding or rolling (Lambe and Whitman, 1979). While the sliding between the particles occurs at all stress levels, the crushing and fracturing of particles begins in a minor way at very low stresses, but becomes evident when some critical stress is reached. Beyond the level of critical stress, the behavior of the material is plastic due to the fracturing of the individual particles, which permits large relative motions between particles. It is known that the critical stress is dependent on the particle size, particle size distribution, the angularity of the particles, and the strength of the individual particles (Roberts and DeSouza, 1958 ; Schultze and Moussa, 1961 ; Hendron, 1963 ; Lee and Farhoomand, 1967).



(a) upper grain size (dry)

(b) medium grain size (dry)

(c) lower grain size (dry)

(d) upper grain size (saturated)

(e) medium grain size (saturated)

(f) lower grain size (saturated)

Fig. 12. Vertical stress versus vertical strain curves of rockfill material

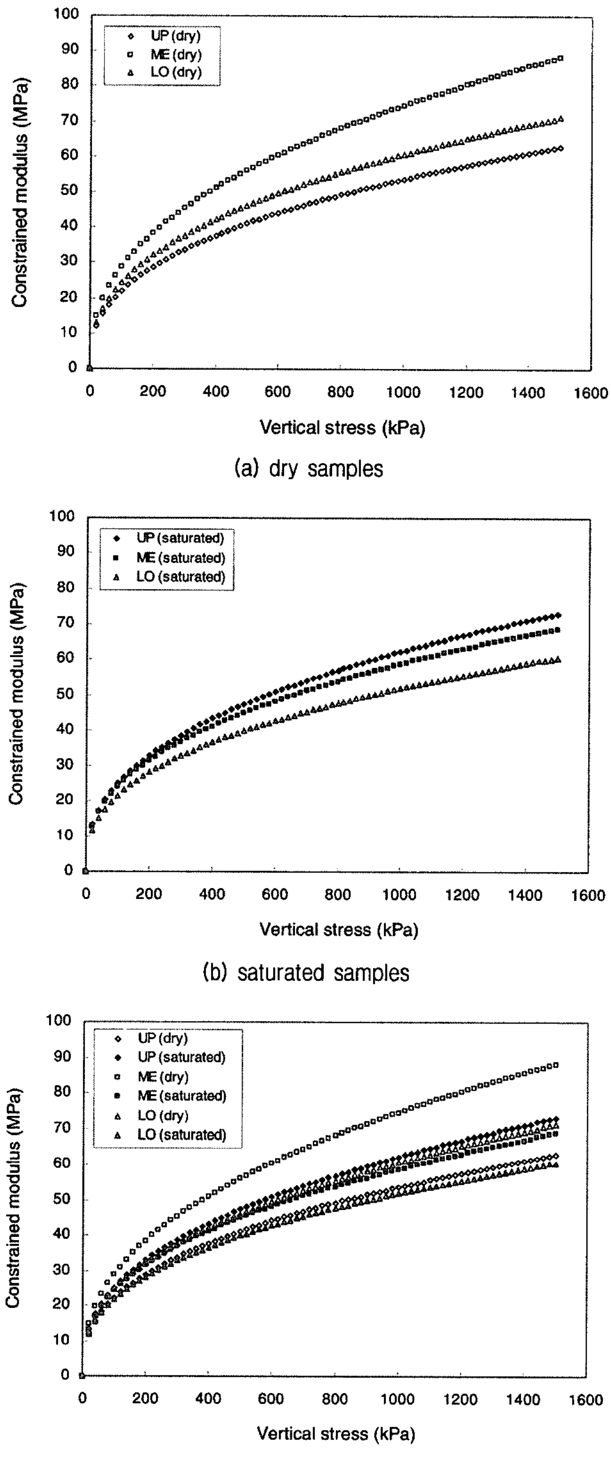
The vertical stress-strain relationships of the rockfill materials exhibited the typical compression characteristics of granular materials described in the above. For dry samples, comparisons between the vertical stress-strain relationships for the three grain size distributions indicated that the sample with the medium grain size distribution exhibited relatively low compressibility, compared to the others (i.e., upper and lower grain size distribution sample). It reflects the fact that a well-graded material is less compressible. For saturated samples, however, relatively high compressibility was observed for the medium grain size distribution sample, compared to the upper grain size distribution sample. It seems to be due to a slightly lower density of the medium sample (see Table 5).

Comparing the dry and saturated samples for a given grain size distribution, slightly higher compressibilities were observed for the saturated samples. The higher deformations in the saturated samples appear to be because water not only acted as a lubricant between particles, facilitating their relative movement, but also reduced the strength of particle itself. This result is consistent with Oldecop and Alonso (2001)'s test results on rockfill compressibility, and also verifies that, in practice, adding water to fill materials and leading to settlements during dam construction is a reasonable approach to reduce post-construction settlements of dam.

The level of critical stress of the rockfill materials, which was defined in this study as the vertical stress corresponding to the point where the vertical stress-strain curve plotted in log-scales for both axes changes sharply, was found to be beyond the maximum vertical stress applied during the tests (i.e., about 2 MPa). For all the samples, the vertical stress and strain exhibited a linear relationship within the applied load range when plotted in both log-scales. Accordingly, it is expected that in critical stress levels, crushing rates of the tested materials would be much higher than those shown in Fig. 11. In a practical point of view, however, high deformations due to material crushing in the high stress levels may not be a concern for typical large dams (with a height less than 100 m) because the stress levels that they experience are commonly lower than 2 MPa.

3.2.3 Constrained Modulus

When rockfill materials are to be used for a dam construction, the settlement of a rockfill layer may be estimated simply using elastic theory. The constrained modulus is the parameter used in estimating settlement under one-dimensional compression (i.e. confined com-



(a) dry samples

(b) saturated samples

(c) total samples

Fig. 13. Tangent constrained moduli of rockfill material

Table 6. Tangent constrained moduli (MPa) of rockfill material

Sample state	Grain size range	Vertical stress (kPa)				
		50	100	300	800	1500
Dry	Upper	16.9	22.1	33.7	49.2	62.7
	Medium	21.6	28.8	45.3	68.0	88.1
	Lower	18.6	24.5	37.7	55.5	71.1
Saturated	Upper	19.1	25.1	38.7	56.9	72.9
	Medium	18.4	24.0	36.8	53.8	68.6
	Lower	16.6	21.7	32.9	47.6	60.5

pression). It is defined as the vertical stress change needed to cause a unit increase in the vertical strain under conditions of zero lateral strain. It can be expressed as:

$$M=\frac{\Delta\sigma_V}{\Delta\varepsilon_V}=\frac{\sigma_{V2}-\sigma_{V1}}{\varepsilon_{V2}-\varepsilon_{V1}} \qquad (2)$$

where M = constrained modulus, ε_{v1} = vertical strain at a stress of σ_{v1}, and ε_{v2} = vertical strain at a stress of σ_{v2}.

In this study, in order to find a tangent constrained modulus at any vertical stress, power functions were curve-fitted to each data series of the measured vertical stress-strain curves ($R^2 \geq 0.99$) and then differentiated. Figs. 13 (a)~(c) and Table 6 present the calculated tangent constrained moduli for the rockfill samples tested for vertical stresses ranging from 0 to 1,500 kPa. They show that the tangent constrained modulus increases gradually with increasing vertical stress.

Comparing the values of the tangent constrained modulus of the dry samples with different gradations, the values for the medium-grain size distribution sample were found to be larger by about 20 to 30%, on the average, than those for the upper and lower grain size distribution samples. Also, the comparisons of the dry and saturated samples showed that overall the values for dry samples were slightly larger (i.e., about 10 to 20%) than those for the saturated samples, as found in the results of the vertical stress and strain relationship.

4. Conclusions

In this study, shear strength and compressibility of a rockfill material, sampled from a dam construction site in Korea, were investigated in the laboratory by performing the large-scaled traixial and oedometer tests. Findings obtained from this study are summarized as follows ;

(1) The rockfill samples exhibited slightly different shear behaviors with varying compaction level (i.e., 85 to 90% relative compaction) and confining stress (i.e., 196 to 588 kPa). The 85% relative compaction samples behaved similar to sandy soils in loose to medium states, whereas the 90% relative compaction samples in medium to dense states. The increase in the compaction level changed the behavior from contractive to dilative. Dilation decreased gradually with increasing confining stress. For the varying conditions (i.e., compaction level and confining stress), the peak friction angle ranged from about 40° to 44°.

(2) The large-scaled oedometer test results showed that under the maximum vertical stress of 2 MPa, particle crushing occurred mainly for the particles larger than 4.75 mm. The crushing rate, calculated for the samples with different gradations (i.e. lower, medium, and upper particle size distributions) ranged from 4.53 to 6.29%, increasing in order of the average particle sizes of the samples. The levels of critical stress of the tested rockfill materials were found to be higher than the maximum vertical stress level applied during the tests (i.e., 2 MPa). The high stresses beyond the critical stress level may lead to sudden and high deformations in a dam due to material crushing. For typical large dams, however, the large deformations in the high stresses do not appear to be a concern since the stress levels in the dams would normally be in a lower range.

(3) For all the tested samples, the maximum vertical

strains (at the vertical stress of about 2 MPa) ranged from about 3 to 5%. A relatively well-graded sample (i.e., medium particle size distribution sample) exhibited lower compressibility, compared to the others (i.e., upper and lower particle size distribution sample). Different compressibilities varying with the different gradations need to be considered in determining the gradation of fill materials for a dam construction. Comparing the dry and saturated samples for a given particle size distribution, slightly higher deformation was observed for the saturated samples. In other words, this verifies that adding water to rockfills during dam construction would lead to initial settlements in the fill and help reduce post-construction settlements (i.e., including settlements during impounding).

(4) Tangent constrained moduli for the medium particle size distribution sample were found to be larger by about 20 to 30%, on the average, than those for the upper and lower particle size distribution samples. Also, overall the values for dry samples were slightly larger (i.e., about 10 to 20%) than those for the saturated samples. The tangent constrained modulus may be used as a basic parameter for simply determining fill amounts or settlements etc. in a dam construction.

(5) Besides the factors considered in this study (i.e., density, gradation, grain size, and water content), the strength and compressibility of rockfill materials are dependent on other various factors, such as the type of parent rock, weathered state of rock, and particle shape etc.. Therefore, different strength and compression characteristics for different rockfill materials will need to be investigated in further studies.

Acknowledgments

The authors would like to acknowledge the Korea Water Resources Corporation for financial support. The authors also would like to thank to the geotechnical group of the department of civil engineering of Aju University for conducting the large-scaled oedometer tests.

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(received on May 12, 2005, accepted on Sep. 26, 2005)