PERMEABILITY CHARACTERISTICS OF HIGH-QUALITY UNDISTURBED GRAVELLY SOILS MEASURED IN LABORATORY TESTS

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ABSTRACT

A series of permeability tests on both high-quality undisturbed samples and reconstituted samples was performed using a large-scale triaxial cell. Based on the test results and discussion, the following conclusions were noted.

- (1) The coefficient of permeability in the horizontal direction is larger than that in the vertical direction. However, its difference is between 10% and 70% and not so large.
- (2) The coefficient of permeability decreases with increasing confining stress. The effect of the confining stress can be understood as the effect of the void ratio. And the small change of the void ratio due to consolidation only leads to a small change of the coefficient of permeability.
- (3) There is no clear correlation between the physical properties and the coefficient of permeability.
- (4) The coefficient of permeability of gravelly soils is almost the same as that of sandy soils, even though the 50% diameter of gravelly soils is about ten to a hundred times that of sandy soils. This result implies that the large size particles of gravelly soils have no significant effect on permeability characteristics of gravelly soils.
- (5) A new definition for determination of D_{10} , D_{20} and fines content only from those soil particles with a diameter smaller than 2 mm was introduced. A correlation similar to sandy soils can be seen between the coefficient of permeability and D_{10} , D_{20} , and fines content based on the new definitions.
- (6) The effect of the small size particles of the gravelly soils on the coefficient of permeability was found significant based on the test results using samples with a special blend of particle size and density reconstituted from undisturbed samples.
- (7) The coefficient of permeability of multi-layer gravelly soils, in the direction perpendicular to the sedimentation, was significantly affected by the lowest permeability. And the coefficient of permeability of the multi-layer in total agrees well with the theoretical estimation.
- (8) Although the data is limited, there was no significant difference of coefficient of permeability between undisturbed and reconstituted samples. This result accords with that reported for sandy soils by Hatanaka et al. (1997). This result also means that the coefficient of permeability of gravelly soil is not affected by the soil fabric. As a result, the in-situ coefficient of permeability of gravelly soils can be well estimated for practical purposes from the reconstituted samples with the same gradation properties.

Key words: coefficient of permeability, fines content, gravel content, gravelly soils, inherent anisotropy, permeability test (IGC: D4/E7)

INTRODUCTION

The coefficient of permeability of a soil is one of the most important design parameters in estimating the stability of an underground excavation and also the stability of earth fill and rock fill dams. In practice, the coefficient of

permeability of cohesionless soils is usually measured by conducting a field test using a boring hole. This method is not adequate, however, to determine the coefficient of permeability for each individual soil layer, because the boundary conditions of the water flow in situ are not clear. It is also difficult to determine the anisotropy of the

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permeability with this method.

In order to overcome these disadvantages, a triaxial cell type testing apparatus has been used by some researchers for measuring the coefficient of permeability. In this method, the boundary condition of the water flow is clear, and it is basically possible to measure the permeability of gravel in both the vertical and horizontal directions under controlled stress conditions.

Compared with the substantial data on the coefficient of permeability for clay, only limited data for permeability on the undisturbed sand have been indicated in previous studies (e.g. Hatanaka et al., 1997), mainly due to the difficulty of obtaining high-quality undisturbed sand and gravel samples. The first objective of this paper is to study the permeability characteristics of gravelly soil in situ by conducting permeability tests on high-quality undisturbed gravel samples using a large-scale triaxial cell. As pointed out by many previous studies (Yoshimi et al., 1978; Singh et al., 1982; Hatanaka et al., 1995), the in situ freezing method can provide high-quality undisturbed sand and gravel samples, retaining the skeleton structure of the in situ gravelly soils. As a result, by using the high-quality gravel samples recovered by the in situ freezing sampling, the effect of inherent anisotropy on the permeability characteristics can be investigated. The effects of confining stress, void ratio and soil gradation characteristics on the coefficient of permeability were investigated. The second objective of this study was to investigate the possibility of the previously proposed empirical equations to estimate the coefficient of permeability of gravelly soils. In order to study the availability of the theoretical method to estimate the coefficient of permeability of multi-layered gravelly soils, a series of permeability tests was also performed on reconstituted samples made from soil particles of undisturbed samples. The effect of the sample disturbance on the permeability was also checked in the present study.

SOIL PROFILE OF SAMPLING SITE AND PHYSICAL PROPERTIES OF GRAVELS TESTED

High-quality undisturbed gravel samples used in this

study were obtained at six sites (Sites TN, KF, HT, KK, TSB and KP) with different sedimentary characteristics, using the freezing technique. The sampling procedures used for obtaining high-quality gravel samples from the six sites were basically the same. A typical procedure for freezing sampling was reported by Hatanaka et al. (1995).

Samples TN, KF and HT consist of Holocene gravel. Samples of KK and TSB were obtained from Pleistocene gravel layers. Samples of Site KP were recovered from uncompacted gravel fill made from Masa (a weathered granite). The soil profiles and sampling depths of each sampling site are shown in Fig. 1. Figure 2 shows the grain size distributions of undisturbed samples. The physical properties of the samples tested are summarized in Table 1. All the specimens had nearly equal or more than 50% of gravel content (G_c : a percentage of soil particles with a diameter larger than 2 mm to total weight). Apart from the KP1-V and KP2-V samples, all the specimens had a fines content of less than 1%. Also shown in Table 1 are the effective overburden stress at the sampling depth (σ'_v) and the test results of the coefficient of permeability at a confining stress (σ'_c) = σ'_v and hydraulic gradient (i) = 0.3.

SAMPLE PREPARATION

In order to study the effect of anisotropy on the coefficient of permeability, two different types of specimen were prepared from the frozen gravel column with an electrical saw: that with an axis perpendicular to the sedimentary plane of the soil layer is known as a V-specimen, while that with an axis parallel to the sedimentary plane is called an H-specimen, as illustrated in Fig. 3. The V-specimen, which has a cylindrical shape, is 15 or 30 cm in diameter, and 30 to 50 cm in height. All the H-specimen were prepared in a cubic shape (10 × 10 × 10 cm) from the frozen gravel column. The coefficients of permeability of the V-specimen and the H-specimen are designated k_v and k_h , respectively.

Basically, the test specimen is considered as an element representing an in situ soil layer. In the case of the V-specimen, however, if the height of the specimen was

Fig. 1. Soil profiles of sampling sites and depth of undisturbed samples

Fig. 2. Grain size distribution of undisturbed gravel samples

Fig. 3. Method of sample preparation for undisturbed V and H-specimens

large enough to be considered to consist of several soil layers, then the test results should indicate the permeability characteristics of these soil layers, in total. In this study, the maximum height of the test specimen was 50 cm, which is short enough to represent a gravel layer for ordinary design purposes. As a result, the test results were considered to indicate the permeability characteristics of each gravel layer.

TESTING APPARATUS AND TEST METHOD

A schematic diagram of the permeability testing apparatus using a triaxial cell is illustrated in Fig. 4. In this testing system, a constant-head type permeability test can be carried out for a specimen at an isotropic confining stress. A test specimen was placed between two porous plates permitting the free drainage of pore water during consolidation and permeability tests. The coefficient of permeability of the porous plate was measured to be larger than 1×10^{-1} cm/s in the preliminary tests, a value almost one order larger than those of the test specimens. As a result, the effect of the porous plate on the coefficient of permeability of test specimens could be ig-

nored. The specimen was then covered with a waterproof rubber membrane. The rubber membrane was sealed from the top cap to the pedestal using O-rings. The top and bottom of the specimen were connected to burette B and water supply tank A (hereafter: tank A), respectively. The frozen specimen was then allowed to thaw under a small confining stress of 0.3 kgf/cm² (29.4 kPa) in the drained condition. After the specimen had completely thawed, it was saturated with the aid of CO₂ gas, the de-aired water and a back pressure of about 1.0 to 2.0 kgf/cm² (98 ~ 196 kPa). The pore pressure coefficient B -value was 0.90 to 0.95. After saturation, confining pressure was applied by air pressure through a water medium. The pore pressure coefficient B -value was larger than 0.95. The water temperature during the permeability test was about 13°C ~ 18°C. After consolidation, the de-aired water was allowed to flow through the specimen from tank A to burette B. It was possible to produce high hydraulic gradients by applying the air pressure p into tank A if necessary. The hydraulic gradient, i , during the permeability test can be calculated by Eq. (1).

$$i = \Delta h / L = (h_A - h_B) / L \quad (1)$$

Fig. 4. Schematic diagram showing large scale triaxial cell for permeability test

$$q = kiA_s \quad (2)$$

where Δh is the pressure difference between the two burettes, and L is the length of the test specimen. The coefficient of permeability, k , in this test system can be obtained by using Darcy's law, which relates the rate of flow through a porous medium to the hydraulic gradient i across the medium as Eq. (2). Where q is the rate of flow and A is the cross-sectional area of the test specimen. Fifteen minutes of elapsed time for measuring the coefficient of permeability at each level of hydraulic gradient, i , was found to be adequate to obtain the stable test results in the preliminary test. Three measured values of the coefficient of permeability at five minute intervals during the elapsed time, were averaged for a test result.

The water flow through the side boundary surface between the rubber membrane and the specimen for this type of test method might greatly affect the test results. According to the test results reported by Hatanaka et al. (1997), if the seepage force is below about 10% of the initial effective confining stress, the effect of the water flow through the side boundary surface between the rubber membrane and the specimen is negligible. The effect of sample size was found negligible based on the results obtained in preliminary tests by using reconstituted samples with different sample sizes (cylindrical shape specimens 10 cm in diameter and 20 cm in height, or 30 cm in diameter and 35 to 50 cm in height).

TEST RESULTS

Effect of the Anisotropy on the Permeability

Figure 5 indicates the test results of the coefficient of permeability for four pairs of undisturbed samples (TN2, KFU1, KP2 and TSB samples). The coefficient of permeability at a hydraulic gradient $i=0.3$, for each pair of V and H-specimens is summarized in Table 1. The following results can be seen from Fig. 5 and Table 1.

The coefficient of permeability in the horizontal direc-

Fig. 5. Comparison of coefficient of permeability between V and H-specimens

tion, k_h , was about 10 to 70% higher than that in the vertical direction. Although the void ratio of the H-specimen was smaller than V-specimen and the soil grain size distribution of H and V-specimens were almost the same as indicated in Table 1.

According to the test results stated above, in most cases, k_h was nearly equal to or larger than k_v . The difference in the permeability between V and H-specimens is considered due to the characteristics in the particle arrangement affected by the sedimentation, though no data are shown in this paper. The effect of anisotropy on the permeability was within 70% and therefore not considered so large. These results are almost the same as those for sandy soils reported by Hatanaka et al. (1997).

Effect of the Confining Stress on the Permeability

The effect of the stress changes on the coefficient of permeability has been studied by Morgenstern and Guthrie (1972) for excavation problems, Fukushima and Ishii (1986) on an earth dam material and Matsumoto and Yamaguchi (1991) on the test fill for a rock fill dam.

Fukushima and Ishii (1986) reported that the confining stress has a great influence on the permeability of the core

Table 1. Physical properties of undisturbed samples and test results

Sample name	TN2		KFU1		KP2		TSB		KFU3	KFL1	KFL2	KFL3	KP1	HT1	TN1	KFU2	KKL1
Sample type	V	H	V	H	V	H	V	H	V	V	V	V	V	V	H	H	H
ρ_s (g/cm ³)	2.68	2.68	2.70	2.70	2.62	2.62	2.74	2.74	2.70	2.72	2.71	2.69	2.62	2.76	2.67	2.71	2.62
Gravel content, G_c (%)	57.40	73.90	67.00	69.10	49.40	55.80	76.40	61.80	68.10	79.70	80.50	69.60	55.90	54.90	72.00	74.30	58.40
Sand (%)	41.80	25.50	32.40	30.30	47.60	43.80	23.00	37.50	31.30	20.00	19.30	30.00	37.50	44.90	27.80	25.20	41.00
Fines content, F_c (%)	0.20	0.60	0.60	0.60	3.20	0.30	0.60	0.70	0.60	0.30	0.20	0.40	6.60	0.20	0.20	0.50	0.60
U_c	23.90	42.10	63.30	80.00	15.10	20.80	18.20	18.20	34.10	54.70	83.70	12.30	32.30	12.10	75.70	19.40	51.20
D_{10} (mm)	0.44	0.38	0.28	0.40	0.27	0.25	0.45	0.33	0.41	0.70	0.67	0.60	0.15	0.66	0.37	0.62	0.43
D_{20} (mm)	0.61	0.92	0.60	0.80	0.50	0.43	1.20	0.67	0.79	2.00	2.00	1.20	0.46	1.23	0.57	1.40	0.79
D_{50} (mm)	4.50	12.50	10.60	20.00	1.94	2.90	7.00	4.20	8.10	24.30	33.10	5.00	2.77	5.60	21.00	8.00	6.30
ρ_d (g/cm ³)	1.94	1.99	2.02	1.97	1.84	1.93	2.03	2.05	1.99	2.09	2.24	2.00	1.88	2.02	2.00	1.99	1.98
Void ratio, e	0.39	0.38	0.34	0.38	0.42	0.37	0.35	0.33	0.36	0.30	0.21	0.35	0.40	0.37	0.38	0.36	0.32
σ'_v (kPa)	186	186	108	108	157	157	127	127	108	157	157	157	108	54	186	108	167
Permeability coefficient, k at $i=0.3$ and $\sigma'_{co}=\sigma'_v$	1.15×10^{-2}	1.66×10^{-2}	8.10×10^{-3}	1.15×10^{-2}	1.21×10^{-2}	2.00×10^{-2}	2.00×10^{-2}	2.60×10^{-2}	6.20×10^{-3}	5.20×10^{-3}	4.40×10^{-3}	7.80×10^{-4}	2.37×10^{-3}	2.02×10^{-2}	1.25×10^{-2}	1.11×10^{-4}	2.10×10^{-2}

* V: V-specimen, H: H-specimen.

material due to the reduction of the void ratio of the soil caused by an increase in confining stress. In order to study the effect of confining stress on the coefficient of permeability, permeability tests were performed under three different initial effective confining stresses of $0.5\sigma'_v$, σ'_v and $2\sigma'_v$, where σ'_v is the effective overburden stress at the sampling depth. The permeability test was performed by increasing σ'_c from $0.5\sigma'_v$ to $2\sigma'_v$ or from σ'_v and $2\sigma'_v$ step by step. For each step of σ'_c , the hydraulic gradient, i , was applied from 0.1 to 0.5 at an interval of 0.1.

Figures 6 and 7 show the effect of the confining stress on the permeability of the V and the H-specimens at a hydraulic gradient (i) of 0.3, respectively. It can be seen, that in the stress range of $\sigma'_c \geq \sigma'_v$, the coefficient of permeability decreases with increasing confining stress very slightly, for both V and H-specimens for each sample. In the stress range of $\sigma'_c < \sigma'_v$, the void ratio of the gravel samples decreases with increasing confining stress. The change in the void ratio due to the consolidation is, however, very small (of the order of 0.001). It can be concluded that, from Figs. 6 and 7, the effect of the confining stress may be related to the effect of the void ratio. The effect of the confining stress on the coefficient of permeability of in situ gravelly soils is also quite small.

Effect of the Physical Properties on the Permeability

a) Effects of the void ratio on the coefficient of permeability

Figure 8 illustrates the relationship between void ratio and the coefficient of permeability at a hydraulic gradient, $i=0.3$. In the range of the void ratio of 0.2 to 0.5, no correlation can be seen between the void ratio and the coefficient of permeability. This result indicates that the coefficient of permeability of undisturbed gravel-

Fig. 6. Effect of confining stress on K_v . **Fig. 7. Effect of confining stress on K_h .**

ly soils can not be described as a function of void ratio alone.

b) Effect of the uniformity coefficient (U_c) on the coefficient of permeability

Figure 9 shows the correlation between the uniformity coefficient, U_c , and the coefficient of permeability of undisturbed gravelly soils plotted at a logarithmic scale ($i=0.3$). In the case of V-specimens, although the data is scattered, a trend that the coefficient of permeability increases with increasing U_c can be seen. This result is not

Fig. 8. Effect of void ratio on coefficient of permeability

Fig. 9. Effect of U_c on coefficient of permeability

Fig. 10. Effect of F_c on coefficient of permeability

in accordance with common understanding that well graded soils may have lower permeability. In total including H-specimens, however, the effect of U_c on the coefficient of permeability is not clear.

c) Effect of the fines content (F_c) on the coefficient of permeability

Figure 10 describes the effect of fines content on the permeability of undisturbed gravelly soils ($i=0.3$). No clear correlation can be seen between fines content and the coefficient of permeability. Also shown in Fig. 10 is the empirical equation (see Eq. (3)) to relate fines content and coefficient of permeability for sandy soils proposed by Hatanaka et al. (1997). As shown in Fig. 10, because the fines content is very small (less than 7%) for most of the gravelly samples tested, the availability of the

Fig. 11. Effect of D_{50} on coefficient of permeability

Fig. 12. Effect of D_{10} on coefficient of permeability

proposed empirical equation for gravelly soils is not clear.

$$\log k = -1.5F_c - 0.65 \quad (3\% \leq F_c \leq 30\%) \quad (3)$$

where, k is the coefficient of permeability in cm/sec, F_c is fines content of sandy soils in percentage.

d) Effect of the 50% diameter (D_{50}) on the coefficient of permeability

Figure 11 exhibits the effect of the 50% diameter of soil particles, D_{50} , on the coefficient of permeability of undisturbed gravelly samples ($i=0.3$). In total, no clear correlation can be seen between D_{50} and the coefficient of permeability.

e) Effect of the 10% size of the soil particles (D_{10}) on the coefficient of permeability

Figure 12 indicates the effects of the 10% size of soil particles, D_{10} , on the coefficient of permeability ($i=0.3$). No trend can be seen between D_{10} and the coefficient of permeability. In Fig. 12, Hazen's equations, which are widely used in practice for estimating the coefficient of permeability from D_{10} of sands are also shown. In the range of D_{10} for undisturbed gravel samples tested, Hazen's equations greatly overestimate the coefficient of permeability. One of the reasons for the discrepancy in the coefficient of permeability between test results and the empirical equations, is the appropriateness of the definition of D_{10} for gravelly soils. This will be discussed later.

f) Effect of the 20% size of the soil particles (D_{20}) on

Fig. 13. Effect of D_{20} on coefficient of permeability

Fig. 14. Effect of gravel content (G_c) on coefficient of permeability

the coefficient of permeability

Figure 13 displays the effect of D_{20} on the coefficient of permeability of undisturbed gravel samples ($i=0.3$). No correlation between D_{20} and test results was found. In Fig. 13 Creager's proposal is also shown. In the range of D_{20} of samples tested, the coefficient of permeability obtained by using Creager's proposal from D_{20} greatly overestimated the in situ value. The reason for the disagreement of the coefficient of permeability between test results and empirical equation will be discussed later from the point of view of the effect of large size particles.

g) Effect of the gravel content (G_c) on the coefficient of permeability

Figure 14 indicates the effect of the gravel content (G_c) on the coefficient of permeability of undisturbed gravelly soils ($i=0.3$). Although the data has much scattering, there is a slight trend evident that the coefficient of permeability decreases with increasing gravel content. This result is contrary to the common understanding that large size particles will bring higher permeability, that is to say, gravelly soils have higher permeability than sandy soils. This result also suggests that the coefficient of permeability is less affected by the portion of large size soil particles, and this result will be discussed later.

DISCUSSION

Figure 15 shows the effect of D_{50} on the coefficient of permeability for sandy soils as reported by Hatanaka et

Fig. 15. Effect of D_{50} on coefficient of permeability for sandy soils (after Hatanaka et al., 1997)

Fig. 16. Effect of F_c on coefficient of permeability for sandy soils (after Hatanaka et al., 1997)

al. (1997). By comparing Fig. 11 with Fig. 15, it is interesting to note that even though the 50% diameter for gravelly soils is about ten to one hundred times larger than that of sandy soils, the coefficient of permeability of both soils is almost in the same range. Moreover, according to the test results reported by Hatanaka et al. (1997) on undisturbed sandy soils, the most significant factor in the permeability characteristics of sandy soils is fines content (see Fig. 16 and Eq. (3)). In order to study the effect of the soil particle size on gravelly soils precisely, two approaches were adopted. One was to exclude the effect of large size (> 2 mm) soil particles and the other was to perform a series of permeability test on reconstituted samples made from a part of the soil particles of undisturbed samples. Test procedure and results are shown below.

Effect of the Content of Large Size Soil Particle

A simple approach to account for the effect of the large size soil particles was made by introducing a new definition of the D_{50} , D_{10} , D_{20} , F_c and U_c . In the new definition, only those soil particles with a diameter smaller than 2 mm of the undisturbed samples are used. Table 2 summarizes the physical properties of virtual samples with only soil particles under 2 mm. Figures 17 to 20 show the effect of the new D_{50} , D_{10} , D_{20} and F_c on the coefficient of permeability. The same trend can be qualitatively seen between the coefficient of permeability and D_{50} , D_{10} , D_{20}

Table 2. Physical properties of virtual specimens under 2 mm

Sample name	TN2		KFU1		KP2		TSB		KFU3	KFL1	KFL2	KFL3	KP1	HT1	TN1	KFU2	KKL1
Sample type	V	H	V	H	V	H	V	H	V	V	V	V	V	V	H	H	H
ρ_s (g/cm ³)	2.68	2.68	2.70	2.70	2.62	2.62	2.74	2.74	2.70	2.72	2.71	2.69	2.62	2.76	2.67	2.71	2.62
Sand (%)	99.50	97.70	98.20	98.10	93.70	99.20	97.40	98.10	98.10	98.50	99.00	98.70	85.00	99.30	99.00	98.10	98.60
Fines content, F_c (%)	0.50	2.30	1.80	1.90	6.30	0.80	2.60	1.90	1.90	1.50	1.00	1.30	15.00	0.70	0.90	1.90	1.40
U_c	2.40	4.00	4.60	3.70	5.40	3.80	4.10	3.60	4.80	4.40	3.50	3.70		3.70	3.90	3.30	4.60
D_{10} (mm)	0.30	0.16	0.13	0.20	0.14	0.17	0.15	0.21	0.15	0.20	0.21	0.30	0.01	0.30	0.13	0.28	0.20
D_{20} (mm)	0.42	0.24	0.21	0.30	0.27	0.23	0.25	0.37	0.27	0.35	0.43	0.46	0.12	0.45	0.30	0.41	0.37
D_{50} (mm)	0.63	0.51	0.43	0.60	0.62	0.50	0.52	0.65	0.60	0.70	0.63	0.87	0.56	0.87	0.44	0.77	0.80
ρ_d (g/cm ³)	1.94	1.99	2.02	1.97	1.84	1.93	2.03	2.05	1.99	2.09	2.24	2.00	1.88	2.02	2.00	1.99	1.98
Void ratio, e	0.39	0.38	0.34	0.38	0.42	0.37	0.35	0.33	0.36	0.30	0.21	0.35	0.40	0.37	0.38	0.36	0.32
σ'_v (kPa)	186	186	108	108	157	157	127	127	108	157	157	157	108	54	186	108	167

* V: V-specimen, H: H-specimen.

Fig. 17. Effect of new D_{50} on coefficient of permeability Fig. 19. Effect of new D_{20} on coefficient of permeability Fig. 18. Effect of new D_{10} on coefficient of permeability Fig. 20. Effect of new F_c on coefficient of permeability

and F_c based on the new definition as was seen with usual definition. And by using the new definition, the empirical equations used for sandy soils were shown to be also useful for estimating the maximum value of the coefficient of permeability of gravelly soils (by comparing Figs. 12 and 18, Figs. 13 and 19, respectively). In the case of F_c , the empirical equation proposed for sandy soils by Hatanaka et al. was located at the upper boundary of the

data area. In the case of U_c as shown in Fig. 21, no correlation can be seen between k and new U_c . The data is greatly scattered even for V-specimens compared with those indicated in Fig. 9. Based on the above, the effect of the large size soil particles of gravelly soils on the coefficient of permeability was found not to be significant.

Fig. 21. Effect of new U_c on coefficient of permeability

Fig. 22. Illustration of reconstituted samples

Effect of the Content of Small Size Soil Particles

Four different types of samples (sample name: R_1 to R_4 , see Fig. 22) were reconstituted from the soil particles of two undisturbed samples (TSB-V and TSB-H specimen). Both TSB-V and TSB-H specimens are cubes of 10 cm. All reconstituted samples are 10 cm in diameter and 10 cm in height. Sample R_1 was prepared only from the gravel content. Samples R_2 and R_3 were reconstituted only from the sand fraction of undisturbed gravel samples (2 mm under), but with a different state of density ($D_r=15$ and 87%). Sample R_4 consists of two layers in the vertical direction: one is a sand layer (under 2 mm with a relative density of 87%) while the other is only gravel (larger than 2 mm). Sample R_1 was prepared by pouring dry gravel particles into a steel mold in five layers; after each layer was made, it was compacted to the desired density by hammering the mold. Sample R_2 was prepared by a method similar to that for measuring the minimum dry density of sand. Sample R_3 was made by pouring sand particles into a water filled PVC mold in ten layers; after each layer was made, the sample was compacted by one hundred horizontal tamps to the mold from outside. Then the sample was frozen in a refrigerator. After that, the frozen sample was removed from the PVC (Poly vinyl chloride) mold. Both ends of the frozen column were prepared to a length of 10 cm. Sample R_4 consisted of two pieces (one piece was sand and the other gravel), each piece 5 cm in height and 10 cm in diameter, as illustrated in Fig. 22. The method for preparing the sand and gravel pieces was the same as those used for preparing samples R_3 and R_1 , respectively. The apparatus and procedure of the permeability test were same as those used for undisturbed samples. Figure 23 shows the soil gradations of reconstituted samples. The physical proper-

Fig. 23. Grain size distribution TSB and reconstituted samples

ties of reconstituted samples R_1 to R_4 are listed in Table 3 with those of undisturbed samples TSB-V and H.

Figure 24 indicates the test results of sample R_1 and undisturbed samples. The coefficient of permeability of sample R_1 is 2 to 2.5 times larger than that of the undisturbed samples.

According to the test results for sandy soils reported by Hatanaka et al. (1997), the effect of the sample disturbance on the coefficient of permeability is negligible. This is also clearly shown later in the present study. As a result, the difference of the permeability between the sample R_1 and undisturbed samples is mainly affected by the small size particles. Small size particles will significantly reduce the permeability.

Figure 25 indicates the results of samples R_2 and R_3 with undisturbed samples. Samples R_2 and R_3 have the same gradations. The loose sample (R_2) has a larger coefficient of permeability than the dense sample (R_3), as shown in Fig. 25. This clearly suggests the effect of the soil density or void ratio for the same soil. Judging from the test results that the coefficient of permeability of undisturbed samples is in the range covered by that of sample R_2 and R_3 , the coefficient of permeability of gravel samples depends significantly on the sand fractions involved in the gravel samples.

Figure 26 shows the test results of samples R_1 , R_3 and R_4 ; dense sand sample R_3 shows the lowest permeability, and sample R_1 the highest. Sample R_4 consisted of sand and gravel pieces. The sand part has the same relative density as sample R_3 . Sample R_4 shows a value near that of sample R_3 . This indicates that the permeability of sample R_4 is greatly affected by the sand part. Based on the above test results, it can be understood that the coefficient of permeability of gravel samples mainly depends on the sand fractions.

The coefficient of permeability of sample R_4 obtained from laboratory tests is also compared with the theoretical estimation by using Eq. (4) as derived for a multi-layer soil. A good agreement between the test results and the theoretical estimation implies that the coefficient of multi-layer samples can be roughly calculated by using Eq. (4) (see Fig. 27).

Table 3. Physical properties of reconstituted and undisturbed samples

Sample name	TSB-V	TSB-H	R1	R2	R3	R4
Sample condition	Undisturbed	Undisturbed	Reconstituted	Reconstituted	Reconstituted	Reconstituted
	V-specimen	H-specimen	Gravel only	2 mm under	2 mm under	Sand-Gravel
Void ratio, e	0.22	0.24	0.52	0.88	0.63	0.62
Relative density, D_r (%)	—	—	—	15	87	—
50% diameter D_{50} (mm)	7.0	4.2	7.6	0.6	0.6	5.2
Fines content, F_c (%)	0.6	0.7	0.0	2.2	2.2	0.0

Fig. 24. Comparison of coefficient of permeability between undisturbed samples and sample R₁

Fig. 27. Comparison of coefficient of permeability between test results and theoretical estimation for two-layer specimen

Fig. 25. Comparison of coefficient of permeability between undisturbed samples and samples R₂ and R₃ Fig. 28. Comparison of coefficient of permeability between undisturbed samples and sample R₄

Fig. 26. Effect of particle size on coefficient of permeability

$$k_v = \frac{L}{L_1/k_1 + L_2/k_2 + \dots + L_n/k_n}, \quad L = L_1 + L_2 + \dots + L_n \quad (4)$$

where, $L_1, L_2, \dots, L_n$ are thickness of each soil layer, and $k_1, k_2, \dots, k_n$ are the coefficient of permeability of each soil layer.

Figure 28 indicates the comparison of test results obtained from TSB-V and H-specimens and sample R₄ (multi-layer sample reconstituted from the soil particles of TSB-V and H-specimens). As shown in Fig. 28, the coefficient of permeability of the R₄ sample is nearly equal to that of the undisturbed samples. This result

agrees with those reported for sandy soils by Hatanaka et al. (1997). It also implies that the coefficient of permeability of in situ gravelly soils can be well estimated from reconstituted samples having the same gradation characteristics and void ratio.

CONCLUSION

Based on a series of permeability tests performed in this study using a large scale triaxial cell on high-quality undisturbed gravel samples recovered by the in situ freezing sampling method and also the reconstituted samples, the following conclusions can be made.

- (1) The coefficient of permeability in the horizontal direction is larger than that in the vertical direction. However, its difference is below 70% and not so large.
- (2) The coefficient of permeability decreases with increasing confining stress. The effect of the confining stress can be understood as the effect of the void ratio. And the small change of the void ratio due to consolidation only leads to a small change of the coefficient of permeability.
- (3) There is no good correlation between the physical properties and the coefficient of permeability.
- (4) The coefficient of permeability of gravelly soils is almost the same as that of sandy soils, even though the 50% diameter of gravelly soils is about ten to a hundred times that of sandy soils. This result implies that the large size particles of gravelly soils are not significant in the permeability characteristics of gravelly soils.
- (5) A new definition for determination of D_{10} , D_{20} and fines content only from the part of the soil particles with a diameter smaller than 2 mm was introduced. A correlation similar to sandy soils can be seen between the coefficient of permeability and D_{10} , D_{20} , and fines content based on the new definitions.
- (6) The effect of the small size particle of the gravelly soils on the coefficient of permeability was found significant based on the test results using samples with a special blend of particle size and density reconstituted from undisturbed samples.
- (7) The coefficient of permeability of multi-layer gravelly soils, in the direction perpendicular to the sedimentation, was significantly affected by the lowest permeability. And the coefficient of permeability of a multi-layer in total agrees well with the theoretical estimation.

- (8) Although the data is quite limited, there was no significant difference of the coefficient of permeability between undisturbed sample and reconstituted sample. This result coincides with that reported for sandy soils by Hatanaka et al. (1997), and means that the coefficient of permeability for gravelly soils is not affected by soil fabric. As a result, the in situ coefficient of permeability of gravelly soils can be well estimated for practical purposes from the reconstituted samples with the same gradation properties.

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