On the Non-Linearity and Anisotropy of Seepage Flow in Compacted Soils

Watcharin GASALUCK*, Masaki ATO*, Ken OHNO* and Takeshi KONDO*

* Faculty of Bioresources, Mie University

Abstract 10 soil samples included coarse sand, fine sand, sandy silt, and silty clay were recompacted in cylindrical and cubic molds for 1 and 2-directions falling-head permeability test respectively. The results represent that even though unsaturated condition in compacted soils caused the seepage flow to be non-linear, Darcy’s law could be applied approximately. Deviation from Darcy’s law becomes enormous only if hydraulic gradient is less than 1. Ratio of horizontal discharge flux to vertical discharge flux is roughly constant when hydraulic gradient is greater than 1. The smaller the hydraulic gradient is, the more the ratio varies. Method of compaction hardly affected flow behavior in coarse sand. According to 2-directions test results, anisotropy affected the soils which is finer than coarse sand.

Key Words: Non-Darcy’s flow, Anisotropy, Non-Linearity

I. INTRODUCTION

Darcy’s law is widely used as a powerful tool for the analysis of laminar flow through porous media. Several studies have been made to investigate the range over which Darcy’s law is valid. Scheidegger (1974) discussed several reasons that flow through very small openings may not follow Darcy’s law in soft clay. Some investigators (Swartzendruber, 1962; Miller and Low, 1963) have claimed that in clayey soils, low hydraulic gradient may cause no flow or only low flow rate that is less than proportional to the gradient. Ping (1963) and Gill (1977) also reported about threshold gradient.

In contrast, Miller, Overman, and Peverly (1969) concluded that there was not threshold gradient in their experiments. Seepage consolidation may cause apparent deviation from Darcy’s law (Pane, Croce et al. 1983). For the falling-head permeability test, a long time required to measure low flow rates, care must be taken to avoid evaporation, and the temperature should be kept constant (Remy, 1973).

There are suggestions that most of the researches on Darcy’s law were done on special apparatus and moisture condition was controlled to be saturated. But nevertheless in field problem, water flows through soils naturally as well as the simplified permeability test. Here-in the simplified permeability test means the conventional falling-head permeability test without the process of eliminating the air in soils by vacuum pump. Even if the air eliminating process or otherwise back pressure is applied to saturate soil sample in laboratory, back pressure is needed for maintaining saturated state through out the test (Dun and Mitchell, 1984). Consequently, saturated state in real earth structure is so rare and unsaturated state is quite common for the simplified permeability test. Furthermore, the simplified permeability test is generally run, especially, in developing country because it is cheap and uncomplicated.

The conventional permeability test has the weak point that it is run on soils compacted in cylindrical mold, therefore only 1 direction of flow can be tested. But the coefficient of permeability obtained from the test is used for analysing seepage problem in any flow direction by the assumption that soils is isotropic. In actual fact, during construction of any earth structure, soil stabilization creates anisotropic characteristic in soils. In addition, most of aquifers have anisotropic characteristic of...
seepage flow (Kovacs, 1981).

The objective of this paper is to study the behavior of non-linear and anisotropic flow in compacted fine-grained soils which is unsaturated and has low hydraulic gradient. This condition is actual in irrigation work such as earth dam. The study is based on the results of 1 and 2-directions simplified permeability tests. The coefficient of permeability equation based on Darcy's law, namely saturated flow condition, is also employed for studying the possibility of using this equation on unsaturated flow condition because it is usually used in the computation of data obtained from the simplified permeability test.

II. MATERIALS AND METHOD

1. Materials

10 samples of natural soils with characteristics as listed in Table 1 were used in this research. At first soil samples were tested by JIS compaction method for finding water content-dry density relationships. Optimum water content ($w_{opt}$) and maximum dry density (Max. $\rho_{dry}$) are shown in Table 1. After compacting for permeability test, water content and dry density of soil sample in mold ($w_0$ and $\rho_{dry}$) were found and listed in Table 1. Later on permeability test, water content ($w$), degree of saturation ($S_r$), and void ratio ($e$) of sample were determined and listed in Table 1 as well.

2. Method

(1) 1-Direction Permeability Test Soil sample was mixed with water to have the water content which was approximately the same as the water content at 95% Max. $\rho_{dry}$ on the wet side of compaction curve. Then the sample was scattered into the cylindrical mold of 100 mm diameter and 63 mm height to be several thin layers, each layer was compacted by a circular hammer of 3 cm diameter. The compaction was done by hand and therefore the compaction energy was non-standard. This compaction method was followed for causing

<table>
<thead>
<tr>
<th>Sample</th>
<th>$G_s$ (%)</th>
<th>$LL$ (%)</th>
<th>$PI$ (%)</th>
<th>Group symbol</th>
<th>Coarse sand (2.0–0.42 mm) (%)</th>
<th>Find sand (0.42–0.0074 mm) (%)</th>
<th>Silt-clay (≤0.0074 mm) (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>2.633</td>
<td>NP</td>
<td>NP</td>
<td>SM–SP</td>
<td>64</td>
<td>27</td>
<td>9</td>
</tr>
<tr>
<td>B</td>
<td>2.692</td>
<td>NP</td>
<td>NP</td>
<td>SM</td>
<td>4</td>
<td>56</td>
<td>40</td>
</tr>
<tr>
<td>C</td>
<td>2.694</td>
<td>50.3</td>
<td>15.3</td>
<td>MH</td>
<td>12</td>
<td>38</td>
<td>50</td>
</tr>
<tr>
<td>D</td>
<td>2.710</td>
<td>44.0</td>
<td>21.3</td>
<td>CL</td>
<td>1</td>
<td>19</td>
<td>80</td>
</tr>
<tr>
<td>E</td>
<td>2.710</td>
<td>48.9</td>
<td>16.7</td>
<td>ML</td>
<td>10</td>
<td>30</td>
<td>60</td>
</tr>
<tr>
<td>F</td>
<td>2.693</td>
<td>41.0</td>
<td>14.3</td>
<td>ML</td>
<td>1</td>
<td>12</td>
<td>87</td>
</tr>
<tr>
<td>G</td>
<td>2.727</td>
<td>53.4</td>
<td>20.2</td>
<td>MH</td>
<td>18</td>
<td>24</td>
<td>58</td>
</tr>
<tr>
<td>H</td>
<td>2.663</td>
<td>52.3</td>
<td>20.5</td>
<td>SM</td>
<td>16</td>
<td>35</td>
<td>49</td>
</tr>
<tr>
<td>I</td>
<td>2.732</td>
<td>38.2</td>
<td>8.4</td>
<td>SM</td>
<td>28</td>
<td>25</td>
<td>47</td>
</tr>
<tr>
<td>J</td>
<td>2.805</td>
<td>39.0</td>
<td>11.4</td>
<td>SM</td>
<td>28</td>
<td>25</td>
<td>47</td>
</tr>
</tbody>
</table>

Remark group symbol was classified in the unified soil classification system.
soil structure to be homogeneous. The size and shape of mold are the same as those inspected by JIS 100% test, then the mold figure is not shown in this paper.

For making constant state in soils, sample with mold was soaked and let water flow through for 3-7 days, and then the falling-head permeability test based on JIS was carried out. The test was started with the hydraulic gradient of about 8. The time for the head drop of every 1 cm was recorded. The recording was taken for the head drop of every 0.3 cm after the hydraulic gradient became less than 1. The variation of testing was recorded by video camera for checking human error. The water used in this test was boiled before test to get rid of the bubbles of air. The surface of water in standpipe with 8 mm diameter was covered by oil film for preventing evaporation. The covered oil film was very thin and the density of oil was small, then the effect of oil film was assumed to be unsignificant. Temperature of the test was controlled by the air conditioner in laboratory and the thermostat put in the reservoir which sample with mold was in. Measuring by thermometer showed that the temperature variation of test was between 19°-21°C.

For calculating, only the data of hydraulic gradient which was less than 5 was used in this paper and coefficient of permeability was obtained from the following equation (Das, 1983).

\[ k = 2.303 \left( \frac{aL}{A(t_2-t_1)} \right) \log(h_1/h_2) \]  

(1)

where

- \(a\) = cross section area of stand pipe (cm²)
- \(L\) = length of sample (cm)
- \(A\) = cross section area of sample (cm²) 
  \= (t_2-t_1) = measured time (s)
- \(h_1\) = water level in stand pipe at \(t_1\) (cm)
- \(h_2\) = water level in stand pipe at \(t_2\) (cm)

(2) 2-Directions Permeability Test Sample preparation was the same as that of 1-direction test. The prepared sample was compacted in a cubic mold as shown in Fig. 1. The internal volume of mold is 1,000 cm³. The soils was placed in the mold in three equal layers. Each layer was compacted by 25 blows of a 2.5 kg metal hammer having a striking square face of 4.5 x 4.5 cm. The hammer was allowed to fall freely through a height of 30.5 cm for each blow. Then mold volume and compaction energy were the same as those of JIS 100% test. Only mold and hammer shape was different from that of JIS 100% test.

Sample with mold was soaked and let water flow through for 3-7 days, and the falling-head permeability test was carried out by the same method as 1-direction test.

Later on the test, both top and bottom caps with inlet-outlet valves were detached. Surfaces of sample were then coated with vaseline. Top and bottom caps without valve were attached to the mold. Side caps were replaced with rubber plates, filter papers, porous stones, and caps with inlet-outlet valves. Method of 1-direction test explained above was followed again.

Vertical and horizontal directions of flow mean the direction that is perpendicular and parallel to the layers of compaction respectively.

Both 1 and 2-directions tests were run 5-10 times for each sample. Each test gave the results which were similar to each other. The average data of the tests was used in this
paper.

For checking the effect of porous stone on test results, permeability test was also done on porous stone and coefficient of permeability of porous stone of $8.310 \times 10^{-3}$ cm/s was found.

### III. RESULTS AND CONSIDERATIONS

According to the results, flow condition in soils can be examined by the use of Reynolds number, $R_e$. Let sample A, the coarsest soils of 10 samples, be taken into consideration. Reynolds number is (Cedergren, 1989; Das, 1983)

\[
R_e = \frac{\rho v d}{\mu} \quad \text{(2)}
\]

where $R_e$ = Reynolds number

\[
\rho = \text{density of water} = 1 \text{ g/cm}^3
\]

\[
v = \text{flux of flow} = 1.761 \times 10^{-3} \text{ cm/s as hydraulic gradient of 5.399 in cylindrical mold}
\]

\[
d = \text{diameter of circular pipe} = D_{10}
\]

\[
\mu = \text{coefficient of viscosity of water} = 9.8 \times 10^{-5} \text{ g/(cm-s)}
\]

In the case of flow in soils which is not circular pipe, the effective pore diameter of soil particles (one fifth of effective size, $D_{10}$) is suggested to be used for $d$ (Cedergren, 1989). $D_{10}$ of sample A is 0.011 cm, then the effective pore diameter is $2.200 \times 10^{-7}$ cm. $R_e$ given by Eq. (2) is $3.949 \times 10^{-4}$, less than 1 and flow remains laminar only as long as $R_e$ is smaller than 1 (Das, 1983), or otherwise, Scheidegger

<table>
<thead>
<tr>
<th>Sample</th>
<th>Least square (cm/s)</th>
<th>Linear regression (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>$(-0.899 i^{-0.8} -3.45 \times 10^{-2} +2.268 i +1.155 i^2) \times 10^{-4}$</td>
<td>$2.822 \times 10^{-4}$</td>
</tr>
<tr>
<td>B</td>
<td>$(1.34 \times 10^{-17} i^{-9} -9.43 \times 10^{-2} +2.206 i +0.101 i^2) \times 10^{-4}$</td>
<td>$2.537 \times 10^{-4}$</td>
</tr>
<tr>
<td>C</td>
<td>$(2.64 \times 10^{-3} i^{-1} -1.319 +8.534 i +0.320 i^2) \times 10^{-6}$</td>
<td>$9.293 \times 10^{-4}$</td>
</tr>
<tr>
<td>D</td>
<td>$(-2.96 \times 10^{-2} i^{-0.3} -0.152 +2.066 i +7.22 \times 10^{-2} i^2) \times 10^{-6}$</td>
<td>$2.538 \times 10^{-4}$</td>
</tr>
<tr>
<td>E</td>
<td>$(4.69 \times 10^{-11} i^{-11} +0.111 +2.576 i +9.53 \times 10^{-2} i^2) \times 10^{-6}$</td>
<td>$2.858 \times 10^{-7}$</td>
</tr>
<tr>
<td>F</td>
<td>$(5.81 \times 10^{-3} i^{-0.6} -0.154 +5.122 i -0.740 i^2) \times 10^{-7}$</td>
<td>$3.637 \times 10^{-7}$</td>
</tr>
<tr>
<td>G</td>
<td>$(-4.02 \times 10^{-3} i^{-0.3} -2.15 +3.69 i +0.199 i^2) \times 10^{4}$</td>
<td>$4.755 \times 10^{-8}$</td>
</tr>
<tr>
<td>H</td>
<td>$(-3.186 i^{-0.3} +5.368 i +1.462 i +1.214 i^2) \times 10^{-8}$</td>
<td>$6.738 \times 10^{-8}$</td>
</tr>
<tr>
<td>I</td>
<td>$(-2.702 i^{-0.3} -0.474 +7.923 i +0.493 i^2) \times 10^{7}$</td>
<td>$5.474 \times 10^{-7}$</td>
</tr>
<tr>
<td>J</td>
<td>$(1.98 \times 10^{-4} i^{-0.38} +0.566 +8.266 i +0.365 i^2) \times 10^{-7}$</td>
<td>$9.421 \times 10^{-11}$</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Sample</th>
<th>Flow direction</th>
<th>Least square (cm/s)</th>
<th>Linear regression (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Vertical</td>
<td>$-0.899 i^{-0.8} -3.45 \times 10^{-2} +2.268 i +1.155 i^2 \times 10^{-4}$</td>
<td>$2.822 \times 10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$1.34 \times 10^{-17} i^{-9} -9.43 \times 10^{-2} +2.206 i +0.101 i^2 \times 10^{-4}$</td>
<td>$2.537 \times 10^{-4}$</td>
</tr>
<tr>
<td>B</td>
<td>Vertical</td>
<td>$2.64 \times 10^{-3} i^{-1} -1.319 +8.534 i +0.320 i^2 \times 10^{-6}$</td>
<td>$9.293 \times 10^{-4}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$4.69 \times 10^{-11} i^{-11} +0.111 +2.576 i +9.53 \times 10^{-2} i^2 \times 10^{-6}$</td>
<td>$2.538 \times 10^{-4}$</td>
</tr>
<tr>
<td>C</td>
<td>Vertical</td>
<td>$5.81 \times 10^{-3} i^{-0.6} -0.154 +5.122 i -0.740 i^2 \times 10^{-7}$</td>
<td>$3.637 \times 10^{-7}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$4.02 \times 10^{-3} i^{-0.3} -2.15 +3.69 i +0.199 i^2 \times 10^{4}$</td>
<td>$4.755 \times 10^{-8}$</td>
</tr>
<tr>
<td>D</td>
<td>Vertical</td>
<td>$3.186 i^{-0.3} +5.368 i +1.462 i +1.214 i^2 \times 10^{-8}$</td>
<td>$6.738 \times 10^{-8}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$2.702 i^{-0.3} -0.474 +7.923 i +0.493 i^2 \times 10^{7}$</td>
<td>$5.474 \times 10^{-7}$</td>
</tr>
<tr>
<td>E</td>
<td>Vertical</td>
<td>$1.98 \times 10^{-4} i^{-0.38} +0.566 +8.266 i +0.365 i^2 \times 10^{-7}$</td>
<td>$9.421 \times 10^{-11}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$-3.63 \times 10^{-38} i^{-24} +8.79 \times 10^{-2} +1.36 i +3.82 \times 10^{-2} i^2 \times 10^{-7}$</td>
<td>$1.530 \times 10^{-1}$</td>
</tr>
<tr>
<td>F</td>
<td>Vertical</td>
<td>$-6.50 \times 10^{-37} i^{-23} -2.41 \times 10^{-2} +3.066 i +0.221 i^2 \times 10^{-7}$</td>
<td>$3.902 \times 10^{-11}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$-2.03 \times 10^{-35} i^{-28} -3.13 \times 10^{-2} +3.590 i +0.236 i^2 \times 10^{-7}$</td>
<td>$4.389 \times 10^{-1}$</td>
</tr>
<tr>
<td>G</td>
<td>Vertical</td>
<td>$1.73 \times 10^{-4} i^{-2.40} -0.740 +8.994 i +0.237 i^2 \times 10^{-7}$</td>
<td>$9.489 \times 10^{-1}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$-4.52 \times 10^{-25} i^{-15} +3.37 \times 10^{-7} +2.167 i +0.141 i^2 \times 10^{-7}$</td>
<td>$2.640 \times 10^{-7}$</td>
</tr>
<tr>
<td>H</td>
<td>Vertical</td>
<td>$1.81 \times 10^{-4} i^{-0.8} -0.109 +1.212 i +0.331 i^2 \times 10^{-8}$</td>
<td>$2.445 \times 10^{-8}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$-1.40 \times 10^{-48} i^{-52} -0.176 +2.425 i +4.53 \times 10^{-2} i^2 \times 10^{-7}$</td>
<td>$2.533 \times 10^{-1}$</td>
</tr>
<tr>
<td>I</td>
<td>Vertical</td>
<td>$0.441 i^{-0.14} -0.726 +1.198 i +6.05 \times 10^{-2} i^2 \times 10^{-5}$</td>
<td>$1.265 \times 10^{-5}$</td>
</tr>
<tr>
<td></td>
<td>Horizontal</td>
<td>$-1.03 \times 10^{-27} i^{-23} -6.00 \times 10^{-2} +1.815 i +0.107 i^2 \times 10^{-7}$</td>
<td>$2.170 \times 10^{-7}$</td>
</tr>
<tr>
<td>J</td>
<td>Vertical</td>
<td>$4.06 \times 10^{-4} i^{-0.12} -0.254 +3.081 i +0.243 i^2 \times 10^{-7}$</td>
<td>$4.026 \times 10^{-7}$</td>
</tr>
</tbody>
</table>
(1974) concluded that the values of Re for which the flow in porous media become turbulent have been measured as low as 0.1 and as high as 75. Consequently, the flows taking place in this experiment were certain to be laminar.

There are 2 suggestions of the results:

1. Flow behavior can be roughly separated to be 2 parts, that of hydraulic gradient which is less than 1 and which is greater than 1. Furthermore, when hydraulic gradient is more than 5, coefficient of permeability is nearly constant. Graphs represented in this paper therefore show the results of hydraulic gradient which is less than 2.

2. Sample C, E, F, G, H, I, and J those have the prominent characteristic of silt, indicated the similar results. So, only the result of sample C will be discussed.

Let equation's form of \( f_O = a_i^v + f_A + f_Ei + f_Oi^2 \) be used for the relationship between discharge flux, \( f_O \), and hydraulic gradient, \( i \), due to the precise nonlinear relationship. The term of \( a_i^v \) is for emphasizing the flow characteristic of small hydraulic gradient. The value of \( i \) is head divided by sample length. But the head varied while water level in stand pipe changed from \( h_1 \) to \( h_2 \). The average value of water level, \( (h_1 + h_2)/2 \), was used for computing \( i \) value. The equations were defined by the least square method and shown in Tables 2 and 3 for 1-direction test results and 2-directions test results respectively.

In case the validity of Darcy's law was assumed, flux should have linear relation with hydraulic gradient and linear regression method was employed for finding the linear equations which are shown in Tables 2 and 3. The coefficients of permeability obtained by linear regression method, namely the regression coefficients were plotted as horizontal straight lines in Figs. 3, 5, 7, 9, 11, 13, 15, and 17 for comparing to the coefficients calculated from laboratory data by the use of Eq. (1) directly.

Rectangular marks and cross marks plotted in Figs. 2 to 17 are the experimental data of flow in vertical direction and horizontal direction respectively. Broken line of vertical flux's equation and thick line of horizontal flux's equation were plotted in those figures as well. All of data were converted into those of 20°C.

Note that sample A, B, C, and D are coarse sand, fine sand, sandy silt, and silty clay respectively. In the other word, the sequence of letter A to D represents the coarsest soils to finest soils.

1. 1-Direction Test

Result of sample A shows that even if the degree of saturation of soils was nearly 100%, the deviation from Darcy’s law is clearly seen. Therefore, it may be concluded in the same way as the conclusion of Lambe and Whitman (1979) that for the soils more pervious than a medium sand, Darcy’s law is not valid and the relationship between hydraulic gradient and flux should be obtained from the experiment run on the soils with characteristics under study. Flux-gradient curve of sample A as shown in Fig. 2 represents concave curve at the small value of hydraulic gradient. The coefficient of permeability, \( k \), calculated by the use of Eq. (1) are represented in Fig. 3. From the figure, the value of \( k \) varies with hydraulic gradient, especially, small hydraulic gradient creates a big variation.

Flow behavior of sample B presented in Fig. 4 resembles that of sample A but constant \( k \) occures roughly for the hydraulic gradients those are greater than 1 as shown in Fig. 5.

Sample C of sandy silt and sample D of silty clay have characteristic of flow as plotted in Figs. 6 and 8 respectively. Convex curves occure at low hydraulic gradient in both samples. Figs. 7 and 9 show that for the hydraulic gradients those are higher than 1, \( k \) is approximately constant. The decrease of \( k \) on sample C is the same as those of sample A and B, along with the decrease of hydraulic gradient, but sample D gave the opposite result.

2. 2-Directions Test

Flux-gradient graphs and \( k \)-gradient graphs of sample A, B, C, and D which shown respec-
Fig. 2 Discharge flux versus hydraulic gradient for 1-direction permeability test of sample A

Fig. 3 $k$ versus hydraulic gradient for 1-direction permeability test of sample A

Fig. 4 Discharge flux versus hydraulic gradient for 1-direction permeability test of sample B

Fig. 5 $k$ versus hydraulic gradient for 1-direction permeability test of sample B

Fig. 6 Discharge flux versus hydraulic gradient for 1-direction permeability test of sample C

Fig. 7 $k$ versus hydraulic gradient for 1-direction permeability test of sample C

Fig. 8 Discharge flux versus hydraulic gradient for 1-direction permeability test of sample D

Fig. 9 $k$ versus hydraulic gradient for 1-direction permeability test of sample D

Soil compaction did not affect the anisotropy of seepage flow in sample A because only small results.
differences between vertical discharge flux and horizontal discharge flux was found. From the result of sample B tested on cubic mold, horizontal flux is higher than vertical flux. Incidentally, vertical flux obtained from cubic mold and cylindrical mold is nearly same. Test
Fig. 18 $k_x/k_y$ versus hydraulic gradient for sample A

Fig. 19 $k_x/k_y$ versus hydraulic gradient for sample B

Fig. 20 $k_x/k_y$ versus hydraulic gradient for sample C

Fig. 21 $k_x/k_y$ versus hydraulic gradient for sample D

results of sample C and D indicate that flow direction influenced flow equations.

Ratio of horizontal coefficient of permeability to vertical coefficient of permeability, $k_x/k_y$ (where $k_x$ and $k_y$ are horizontal and vertical coefficient of permeability respectively), were plotted against hydraulic gradient in Figs. 18 to 21. The alteration of ratio becomes clearly considerable as hydraulic gradient becomes small. The relationship between soil characteristics and alteration of ratio is unpredictable.

There is a hint on flux-gradient curves of low hydraulic gradient that curve of sample A is concave, curve of sample B is similar to but flatter than that of sample A. Sample C imparts small convex curve and convex curve of sample D is clearly seen.

The characteristics of flux-gradient curves of low hydraulic gradient could be affected by the following causations:

1. In clay, variation of electrical potential (the work to bring a water molecule to be attached to a soil particle surface, $\Psi$) with distance from a soil particle surface can be presented as shown in Fig. 22 (Mitchell, 1976). Suppose the hydraulic gradient of soils under study creates the force which is equal to the potential of $\Psi_1$, the layer of water attached to soil surface is $D_1$ thick. The thickness of water layer becomes $D_2$ as the force created by hydraulic gradient is equal to the potential of $\Psi_2$.

It means that the thickness of water layer increases $D_2 - D_1$ as the force decreases $\Psi_1 - \Psi_2$. In the same way, the thickness of water layer increases $D_3 - D_2$ as the force decreases $\Psi_2 - \Psi_3$. If $\Psi_1 - \Psi_2$ is equal to $\Psi_2 - \Psi_3$, it can be seen that the thickness of water layer increases more rapidly at the small value of hydraulic gradient. The rapid increase of thickness causes the discharge flux to rapidly
On the Non-Linearity and Anisotropy of Seepage Flow in Compacted Soils

Fig. 23 Relative permeability relating to degree of saturation

decrease and flux-gradient curve of clay such as sample D becomes convex when hydraulic gradient is small. Moreover, threshold gradient can be explained by Fig. 22 (Yong and Warkentin, 1975; Das, 1983). At the small hydraulic gradient, the water layers covering 2 parallel clay particles combine with each other. The combination point is the middle between the 2 particles and at the potential of \( \Phi_d \). If the potential created by hydraulic gradient is less than \( \Phi_d \), water can not seep through soils. This hydraulic gradient is called threshold gradient.

(2) In unsaturated soils, there is air which its volume increases if pressure decreases, the increase is little when the pressure is high but great when the pressure is low (Choudhury, 1973). Consequently, degree of saturation of soils decreases as hydraulic gradient decreases and in particular in low hydraulic gradient range. The ability of flow in soils can be represented as the relative permeability relating to degree of saturation which is shown in Fig. 23 (Bear, 1979). According to the figure, flow ability decreases along with the decrease of degree of saturation and the smaller degree of saturation is, the smaller flow ability decrease rate becomes, for example, \( S_1 - S_2 \) is equal to \( S_2 - S_3 \) but \( R_1 - R_2 \) is greater than \( R_2 - R_3 \). It means that the decrease rate of discharge flux becomes lower and lower as hydraulic gradient becomes smaller, and then concave flux-gradient curves occur at low hydraulic gradient in sample A and B, the samples which are non-plastic soils.

The causation explained above is only the assumption based on 3 sample (A, B, and D) which can be indicated the characteristic of sand or clay. The other samples are composed of various characteristics, then it is hard to achieve the discussion. The exact causation needs a large number of test results.

Deviation from Darcy's law occures in this research could be no significance if compares to the different value of \( k \) obtained from laboratory and field, because in some reports, the actual \( k \) of clay liners were generally found to be 10 to 1,000 times larger than value obtained from laboratory tests on either undisturbed or recompacted samples of clay liner (Daniel, 1984; Day and Daniel, 1985). Consequently, for making a prediction about seepage problem by the use of laboratory test result, the most essential query is the representative sample.

As mentioned in II. MATERIALS AND METHOD, coefficient of permeability of porous stone in vertical direction of flow \( (k_{stone(v)}) \) is \( 8.310 \times 10^{-3} \text{cm/s} \). Effect of porous stone on coefficient of permeability of soils can be examined. Let 2-directions permeability test results of sample A be in consideration. Vertical coefficient of permeability of soils \( (k_{soils(v)}) \) as shown in Fig. 11 is \( 2.600 \times 10^{-4} \text{cm/s} \) for the hydraulic gradient of 2. This value was calculated by use of 10 cm thickness of soil sample. If the same data is calculated exactly by use of 11.5 cm thickness of soil sample, vertical coefficient of permeability becomes \( 2.990 \times 10^{-4} \text{cm/s} \). The value of \( 2.990 \times 10^{-4} \text{cm/s} \) is effective coefficient of permeability \( (k_{eff}) \) of sample composing of soils and porous stone. In accordance with effective coefficient of permeability for stratified soils (Das, 1983):

\[
k_{eff} = H_{soils} + H_{stone} / \left( \frac{(H_{soils} / k_{soils(v)})}{(H_{stone} / k_{stone(v)})} \right)
\]

where \( H_{soils} \) and \( H_{stone} \) are thickness of soils and porous stone respectively.

By the help of Eq. (3), \( 2.990 \times 10^{-4} \text{cm/s} \), 10 cm, 1.5 cm, and \( 8.310 \times 10^{-3} \text{cm/s} \) are substi-
tuted for \(k_{(v)}\), \(H_{\text{soils}}\), \(H_{\text{stone}}\), and \(k_{\text{stone}(v)}\) respectively; coefficient of permeability of soils \(k_{\text{soils}(v)}\) is \(2.612 \times 10^{-4}\) cm/s which is different only 0.46% from \(k_{\text{soils}(v)}\) of \(2.600 \times 10^{-4}\) cm/s calculated by ignoring porous stone.

The smaller coefficient of permeability is, the smaller difference become. For 1-direction test result of sample C, the difference is only 0.0014%. Therefore effect of porous stone was ignored in this research.

IV. CONCLUSION

1. Except for sample A of coarse sand, the relationship between discharge flux and hydraulic gradient in vertical direction and that in horizontal direction is noticeably different.

2. Coefficient of permeability, \(k\), based on Darcy’s law varies with hydraulic gradient. The alteration of \(k\) is uncertainly, especially, that of low hydraulic gradient. Consequently, the simplified permeability test should be done for various hydraulic gradients and \(k\) should be solved from flux–gradient relationship by the use of linear regression method. But nevertheless the test is generally run for 3–4 value of hydraulic gradient and coefficient of permeability is computed by the use of Eq. (1). Figs. 3, 5, 7, 9, 11, 13, 15, and 17 show that Eq. (1) can be used approximately for the value of hydraulic gradient which is greater than 1.

3. Compaction method hardly affectes behavior of flow in nonplastic soils, but behavior’s inconsistency of flow takes place in plastic soils. Therefore, “How to compensate for the variation of compaction attribute”, is very important for the prediction of flow in earth dam which is usually composed of plastic soils.

REFERENCES

農業土木学会論文集 第180号 内容紹介

【研究論文】
On the Non-Linearity and Anisotropy of Seepage Flow in Compacted Soils (本文＝英文)
Watcharin Gasaluck・阿藤 正樹
大野 研・近藤 武

10種類の土（粗砂, 細砂, 砂質シルト, シルト質粘土）を, 回転円筒モデルと立った方体モデルで圧密後, 1 方向と2 方向の変水位透水試験を行った。圧密土の不飽和状態が浸透流の流れに非線形性を引き起こすことがわかった。ダルシーの法則が近似には適用できることが分かった。特に, RB FMモデルに基づく2次元非線形解析（RBSM）と非線形解析（RBSM）を組み合わせ, 觀測変位から周辺地盤の地盤変数を推定するための新たな逆解析法を開発・提案する。今回, 非線形モデルにおいても本逆解析法の検証を行い, 感応として, すべりを生じた地下空間, 特に隣接構造物を有する地下空間のモデルを想定した。周辺地盤の地盤変数を推定した結果について述べる。

キーワード 非線形性, 非均質性

【農業土木学会論文集 180, pp. 1-10】

【研究論文】
CB 法の降雨時における供用度数と広域水田地域における降雨有効化的分析
千家 正照・西出 勤・太田利・小倉 健

反復利用を考慮した広域水田地域の降雨有効化の分析法について提案し, その適用事業について紹介した。従来の浸透水質試験工法による降雨有効化度数については明らかにした。1. NB ブロックの降雨是有効度数に影響しない。2. CB ブロックの有効程度の上限値は深さによる。3. RB ブロックの有効程度の上限値は, 焼却水深 Dm と下流 CB ブロックの取水量 Qm の大小関係によって, Dm ≤ Qm の時は深さ水深 Dm となり, Dm > Qm の時は蒸発散 ET となる。

【農業土木学会論文集 180, pp. 21-28】

【研究論文】
構造システム地盤における土壌水消費特性—山脇切趾法面を対象にした連続融解緑生基盤工的適用化—
横塚 亨・山本 太平・尖比 萬・光弘 杉山・瀬川 進

自然環境の保全・復元が大きな課題として提起されている今日, それを具体化する技術としての緑生土が注目を浴びている。開発や整備を伴い続ける山脇切趾法面などの荒廃環境において, 緑生土工の導入は不可欠である。こうした立脚心から改善された分層の植生基盤の水分貯留について, 有限要素法によるシミュレーション解析を行った。また, 計算結果と実験フィールドにおける測定値との比較により, 水分動態予測と山脇切趾法面などの緑化環境の改善への適用の可能性を検討した。

キーワード 斜面緑化, 植生基盤, 有限要素法, 土壌水分動態, 斜面消費量, 斜面浸透速度

【農業土木学会論文集 180, pp. 39-47】

【研究論文】
ポアスカルバート壁体における温度変化による土壌水分と養生方法の簡易的システム化
緒方 英彦・國武 昌人・近藤 文義
中沢 隆雄・山下 博

コンクリートポアスカルバート壁体に発生する温度ひびわれの防止方法として保養養生法に有効である。今回は, 新たに保養養生方法を実施のコンクリートポアスカルバート壁体に適用した結果, 1 本の温度ひびわれを発生しなかった。この事実を基に, これまで行われたコンクリート壁体内部の温度計測結果を用いて, 保養養生方法についての基礎研究を行った。その結果, 温度ひびわれを発生させないために必要な外環境温度の影響を考慮することが可能であった。これららの総合化することで保養養生方法の簡易的システム化を行った。

キーワード ポアスカルバート, 温度ひびわれ, 保養養生, 緑生, 範囲外環境温度の影響

【農業土木学会論文集 180, pp. 49-57】