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Hydraulic Conductivity of Loose Coarse Sand

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INTRODUCTION

Hydraulic conductivity of soils is an important parameter in many offshore geotechnical studies, such as wave-soil interaction. In 1856, Darcy published a simple relationship between the discharge velocity (v) and hydraulic gradient (i) for flow through soil, which can be expressed as:

$$v = ki \tag{1}$$

where k = coefficient of permeability of soil. The preceding relationship holds good for the laminar flow of water through the void spaces in soil (sand and clay) and has been subjected to extensive verification during the last 136 years. Based on these studies, it has been concluded that, for flow of water through fine and medium sand, silt, and clay, the flow is laminar and Darcy's law is valid (Muskat, 1937; Mitchell, 1976). Several theoretical and empirical correlations for estimation of the coefficient of permeability in fine and medium sand are now available in the literature (e.g., Hazen, 1930; Kozeny-Carman relationship — Kozeny, 1927 and Carman, 1956; Amer and Awad, 1974; Shahabi, Das and Tarquin, 1984; Kenny, Lau and Ofoegbu, 1984). On the other hand, it is well-known that turbulent flow condition does exist when water flows through clean gravel and rocks. Empirical equations are available for estimation of the velocity of flow in those cases (Leps, 1973; Cedergren, 1977).

In the United States and most other countries, two soil-classification systems are used for engineering purposes—the AASHTO classification system and the Unified classification system. In the AASHTO system, the upper limit of sand is 2 mm (No. 10 U.S. sieve), and in the Unified system it is 4.75 mm. In a sand, however, if the particle sizes are in the range of 1 mm to 4.75 mm and the hydraulic gradient is rather high (i.e., about 1 or greater), then the laminar flow condition may not exist; and, thus, Darcy's law will not be valid. This can be shown by the following approximate calculations. For the laminar flow condition (Muskat, 1937; Taylor, 1948):

$$R_n = \frac{vD\rho}{\mu} \leq 1 \tag{2}$$

where R_n = Reynold's number, v = discharge velocity (cm/sec), D = average diameter of soil particle (cm), ρ = density of water (g/cm^3), μ = coefficient of viscosity g/(cm-s).

Thus, if D = 1 mm, ρ = 1 g/cm^3, $\mu = 981 \times 10^{-5}$ g/cm-sec, then the maximum discharge velocity for laminar flow can be calculated as:

$$v = \frac{\mu}{D\rho} = \frac{981\times10^{-5}}{(0.1)(1)} = 0.0981 \text{cm/s} \tag{3}$$

According to Hazen (1930), for medium to fine sand:

$$k \approx 100(D_{10})^2 \tag{4}$$

where D_{10} = effective diameter in cm. Assuming $D_{10} \approx D$ and combining Eqs. 1 and 4:

$$v = 100(D_{10})^2 i \tag{5}$$

Again, combining Eqs. 3 and 5:

$$i = \frac{0.0981}{(100)(0.1)^2} = 0.0981 \tag{6}$$

Hence, if $i > 0.0981$, the nonlaminar flow condition will exist.

For the above-stated reasons, a number of laboratory *constant head permeability* tests were conducted on 12 artificially mixed, *loose, coarse granular soils* in which the upper and lower limits of the grain sizes were kept between 4.75 mm (No. 4 U.S. sieve) and 0.85 mm (No. 20 U.S. sieve) in order to determine a relationship between v and i for the case in which the laminar flow condition of water will not exist. For turbulent flow conditions in coarse sands, gravels and boulders, Forchheimer (1902) suggested that:

$$i = av + bv^2 \tag{7}$$

where a and b are constants.

SUMMARY OF RESULTS

The grain-size distributions of the 12 artificially mixed coarse sands are shown in Fig. 1. The average grain size (D_{50}) and the corresponding void ratios during the tests are given in Table 1. It is important to note that, for coarse sands of this type, the general void ratio varies from about 0.50 to about 0.75. For all tests in this program, the void ratio e varied from about 0.64 to about 0.79 with an average of about 0.67. This means that the relative density of compaction was less than about 20% for all cases.

Constant head permeability tests were conducted on soils. For all tests, the length of the specimen was kept at 254 mm and each specimen had a diameter of 63.5 mm. During the tests, flow was maintained from the top to the bottom of the specimen. In conducting a test, the hydraulic gradient i was increased step by step from 0.8 to 3 and after that it was decreased step by step back to 0.8.

Fig. 2 shows plots of the discharge velocity versus hydraulic gradient for samples Nos. 1 ~ 5 as obtained from the laboratory

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