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ORIGINAL ARTICLE

Estimation of soil permeability

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Abstract Soils are permeable materials because of the existence of interconnected voids that allow the flow of fluids when a difference in energy head exists. A good knowledge of soil permeability is needed for estimating the quantity of seepage under dams and dewatering to facilitate underground construction. Soil permeability, also termed hydraulic conductivity, is measured using several methods that include constant and falling head laboratory tests on intact or reconstituted specimens. Alternatively, permeability may be measured in the field using insitu borehole permeability testing (e.g. [2]), and field pumping tests. A less attractive method is to empirically deduce the coefficient of permeability from the results of simple laboratory tests such as the grain size distribution. Otherwise, soil permeability has been assessed from the cone/piezocone penetration tests (e.g. [13,14]). In this paper, the coefficient of permeability was measured using field falling head at different depths. Furthermore, the field coefficient of permeability was measured using pumping tests at the same site. The measured permeability values are compared to the values empirically deduced from the cone penetration test for the same location. Likewise, the coefficients of permeability are empirically obtained using correlations based on the index soil properties of the tested sand for comparison with the measured values.
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1. Introduction

Soils are permeable materials because of the presence of interconnected voids that permit the flow of fluids from locations of high energy to locations of low energy. Proper measurement/evaluation of soil permeability is required for calculating the seepage under hydraulic structures and water quantities during dewatering activities. Soil permeability is affected by several factors including voids ratio, distribution of inter-granular pores, and degree of saturation. The discussion presented herein is limited to evaluating the coefficient of permeability of saturated soils. The coefficient of permeability exhibits a wide range of values up to 10 orders of magnitude from coarse to very fine grained soils [16]. Furthermore, previous studies on the coefficient of permeability show that the coefficient of permeability is highly variable within the same deposit with a coefficient of variation as high as 240% [17]. Laboratory constant and falling head permeability tests (e.g. [1]) are easy to perform. However, it is very difficult and expensive to obtain undisturbed samples from granular soil deposits. Accordingly, these tests are typically performed on specimens reconstituted to relative densities “close” to those from the field. Thus, the measured permeability may not be representative of the field permeability because the soil fabric is destroyed due to sampling techniques. Field permeability tests offer another technique for measuring permeability without sample disturbance

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making it more suitable for granular soils. However, it is difficult to evaluate the hydraulic gradient acting on the soil during field permeability tests. Furthermore, most methods of permeability calculation from field tests are theoretically based on several assumptions regarding the test including the water head, flow path. The reliability of the measured values of permeability using field testing depends to what degree the assumptions represent the actual site conditions.

Field permeability may be measured using pumping tests which provide a good measurement of the permeability of an aquifer (e.g. [3,6]). Pumping tests provide an average value of the coefficient of permeability at the test site. Alternatively, permeability could be measured using either falling or constant head tests performed in boreholes (e.g. [2–5]). Tests performed in boreholes provide a detailed permeability profile of the measured permeability values versus depth compared to the average permeability from pumping tests. The test equipment and procedures used meet the guidelines and conditions of BS 5930 [3] and BS 6316 [7]. The measured permeabilities are compared to the values obtained from the results of the Cone Penetration Test (CPT) which was performed in the top soil layer. Moreover, the measured values are matched to permeability estimates obtained from grain size distribution tests as outlined below.

2. Subsurface ground conditions

Rotary drilling and coring were used to execute the boreholes. Water was used as a drilling fluid to eliminate the influence of other drilling fluids (e.g. bentonite) on the soil permeability. Disturbed soil specimens were obtained using split spoon samplers in cohesionless soil layers. Undisturbed samples were extracted using a double tube core barrel with a 76 mm internal diameter in rock formations. The extracted soil specimens were examined, visually classified and then sent to the laboratory for testing.

The subsurface ground consists of a top silty sand layer which extends from the natural ground surface to a maximum depth of 5-m. This layer is underlain by weak sandstone and very dense sand with cemented bands and lumps which extend to the end of drilling, located 40 m below the natural ground surface. The Standard Penetration Test (SPT) was performed in the sand layers at 1-m intervals. Cores were extracted from the rock layers and Rock Quality Designation (RQD) values were calculated at different depths. Fig. 1a and b shows the variation of the SPT-N and RQD with depth, respectively. The recorded SPT-N values of the top silty sand layer exhibit large variability ranging between 2 and 44 indicative of very loose to dense sand. The measured RQD values vary between 10% and 78% with an average value of approximately 30%. Thus the rock quality is described as very poor (RQD less than 25%) and good (RQD between 75% and 90%). The groundwater table at the site is located approximately 1-m below the natural ground level. Representative soil specimens were extracted from the various layers for laboratory testing which included natural moisture content, gradation, Atterberg limits on fines, specific gravity, and natural unit weight for core samples. Grain size distribution curves of representative soil specimens at different depths from a number of boreholes are shown in Fig. 2. According to the Unified Soil Classification System, the tested soil specimens are mainly composed of sand which constitutes 51.4–73.5% of the samples. The percentage

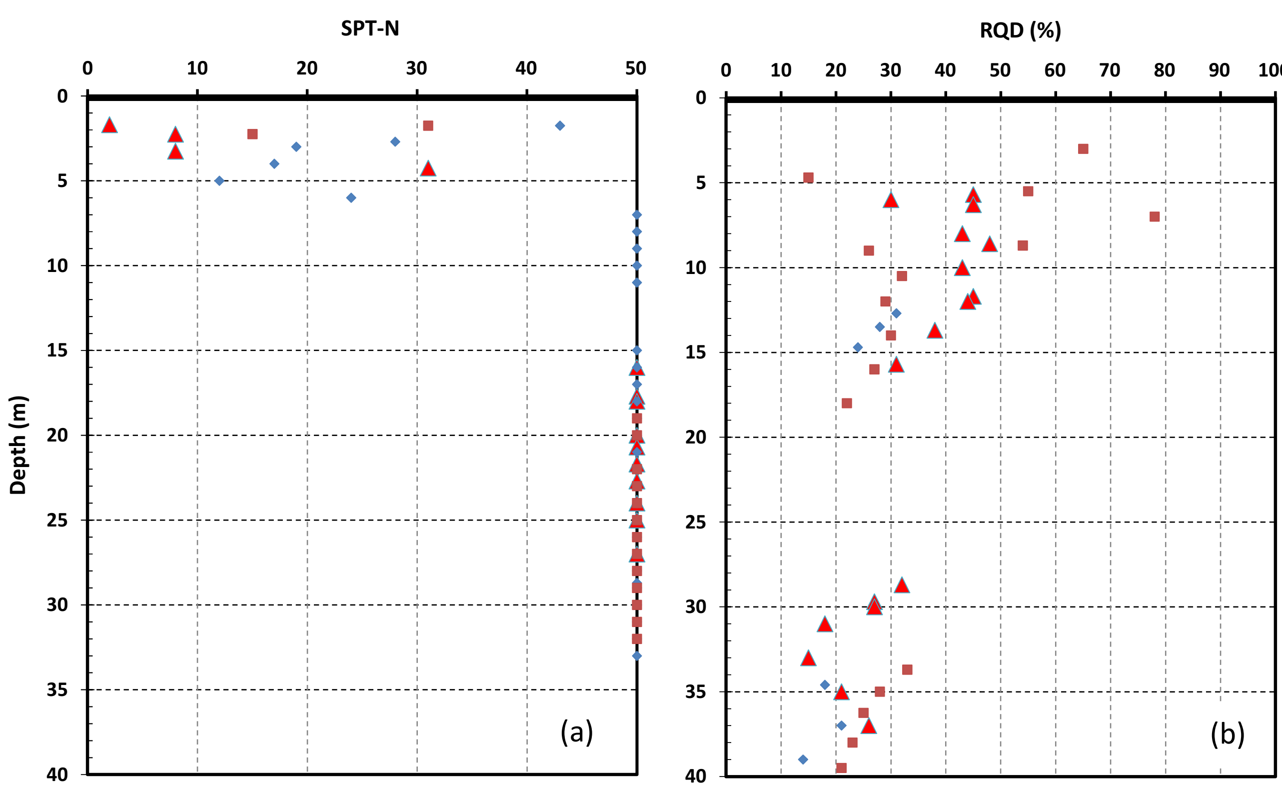

Figure 1 (a) Variation of SPT-N with depth and (b) variation of RQD with depth.

of gravel varies between 3% and 25%, while the percentage of fines (passing sieve # 200) ranges between 9.2% and 29.9%. The fine content is non-plastic classified as silt, according to USCS. The soil is primarily classified as poorly graded silty sand. The average specific gravity of the solid particles was measured to be 2.63. The bulk unit weights of the cemented specimens vary between 14.62 kN/m^3 and 19.52 kN/m^3 with an average value of 16.68 kN/m^3. The natural water contents range from 6% to 27%. Considering the high ground water level, the soil is considered to be fully saturated with equivalent void ratios varying between 0.2 and 0.57 with an average value of 0.37.

Five static Cone Penetration Test (CPT) soundings were performed in the top soil layer down to refusal. The cone tip resistance q_c and sleeve friction f_s were recorded with depth and are presented in Fig. 3a and b, respectively. The friction ratio, computed as the ratio of the sleeve friction to the cone tip resistance, is plotted versus depth in Fig. 3c. Excluding the top 1-m, the soil is mainly classified as sand based on the CPT measurements and empirical soil classification charts (e.g. [18,19,13,20]). Generally, there is good agreement between the CPT and grain size distribution classifications which is expected for sand/silt mixtures [20]. Based on the

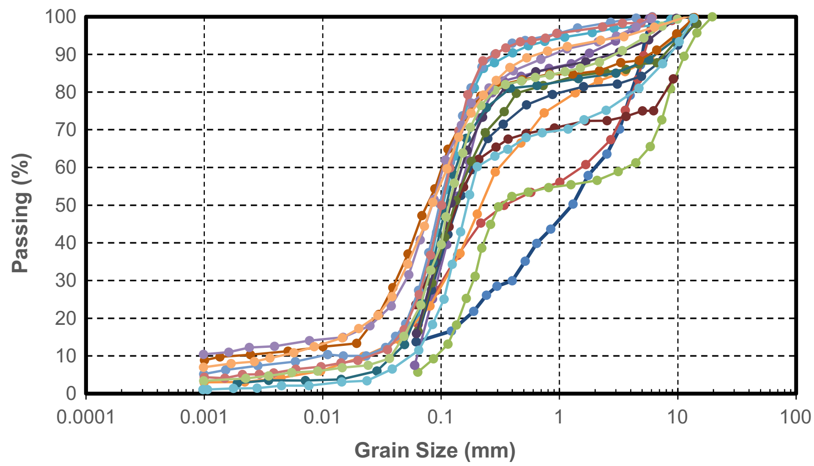

Figure 2 Grain size distribution curves of representative soil specimens.

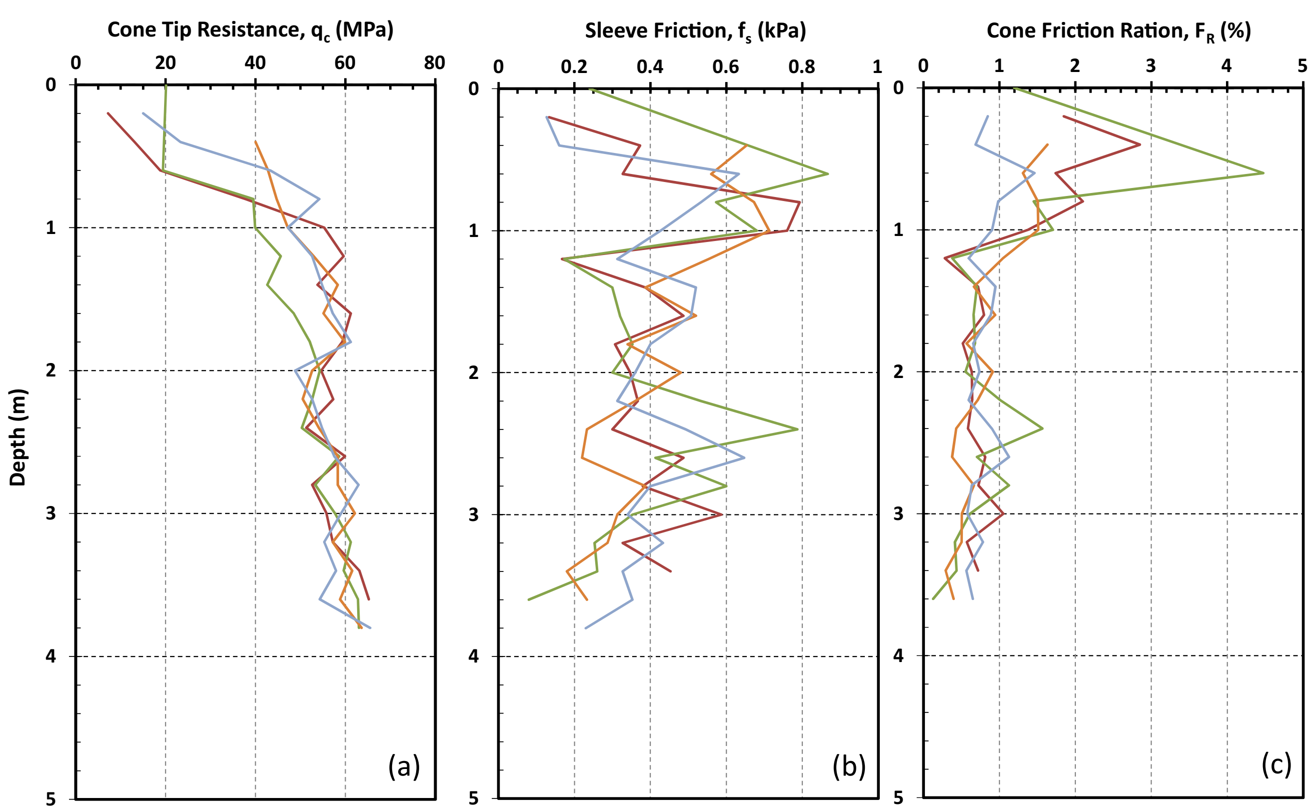

Figure 3 (a) Variation of cone tip resistance with depth, (b) variation of cone side friction with depth, and (c) Variation of the friction ratio with depth.

available data, the top 1-m exhibits a relatively high degree of variability and is considered as heterogeneous fill.

3. Determination of soil permeability using pumping tests

Field permeability may be measured using pumping tests which provide a good measurement for the permeability of an aquifer. When water is pumped from a well, the water level is lowered within the well thus creating a hydraulic gradient which causes the water to flow to the well lowering the water level in the aquifer which results in a cone of depression forming around the well. The hydraulic properties of the aquifer affect the drop in water depth and its lateral extent. Thus, the soil permeability of the aquifer is calculated using the field measurements. The test comprises of pumping water from a well or borehole and measuring the drop in water levels at the well location as well as at the locations of observation wells placed in an array around the pumping well. The drawdown can be measured using a minimum of 2 wells to monitor the drop in the water level.

The pumping test was conducted in accordance with BS 5930 [3] over a duration of two days. On the first day, a pump was installed and operated for 2–4 h to remove any sand and define a suitable flow rate. On the second day, water levels were monitored in the observation wells till a constant level was reached (steady state). An electrical submersible pump was lowered into each well such that the pump inlet was located 8–10-m below the natural ground water level. Readings were taken every 5 s for the first 10 min and then every 20 s up to 100 min. Beyond 100 min, readings were made every 1 min. The coefficient of permeability was calculated assuming confined conditions. The thickness of the aquifer was taken as the height of water from the bottom of the well to the natural ground water level.

Tests were conducted at water heights – measured from the bottom of the well – of 10-m, 20-m, and 30-m, allowing the evaluation of permeability with depth. The water was measured in two observation wells located 12.2-m and 42.7-m radially from the well. Fig. 4 shows the variation of the water level with time till stabilization (steady state) is reached for the 3 pumping tests. The steady state was reached within approximately 12 min and 45 min from the start of pumping. The corresponding coefficients of permeability were calculated to be 3.0×10^{-3} m/s, 3.4×10^{-5} m/s, and 4.4×10^{-6} m/s, for pumping tests 1, 2, and 3, respectively. The results are representative of the average permeability of the top 10-m, 20-m, and 30-m for the tests, respectively, indicative of that the permeability decreases with depth. These results agree with the ground stratigraphy which shows the sand as the deeper strata exhibits a higher degree of compactness as exhibited with SPT-N values greater than 50 or cementation for the sandstone layers. Notwithstanding, the measured permeability values lie within the typical range for silty sands and sands which range between 10^{-3} m/s and 10^{-6} m/s as reported by Carter and Bentley [21].

4. Measurement of permeability from falling head permeability tests in boreholes

In situ falling head permeability tests were conducted at regular intervals in three boreholes down to a depth of 40-m, in accordance with the procedures provided in BS 5930 [3]. The

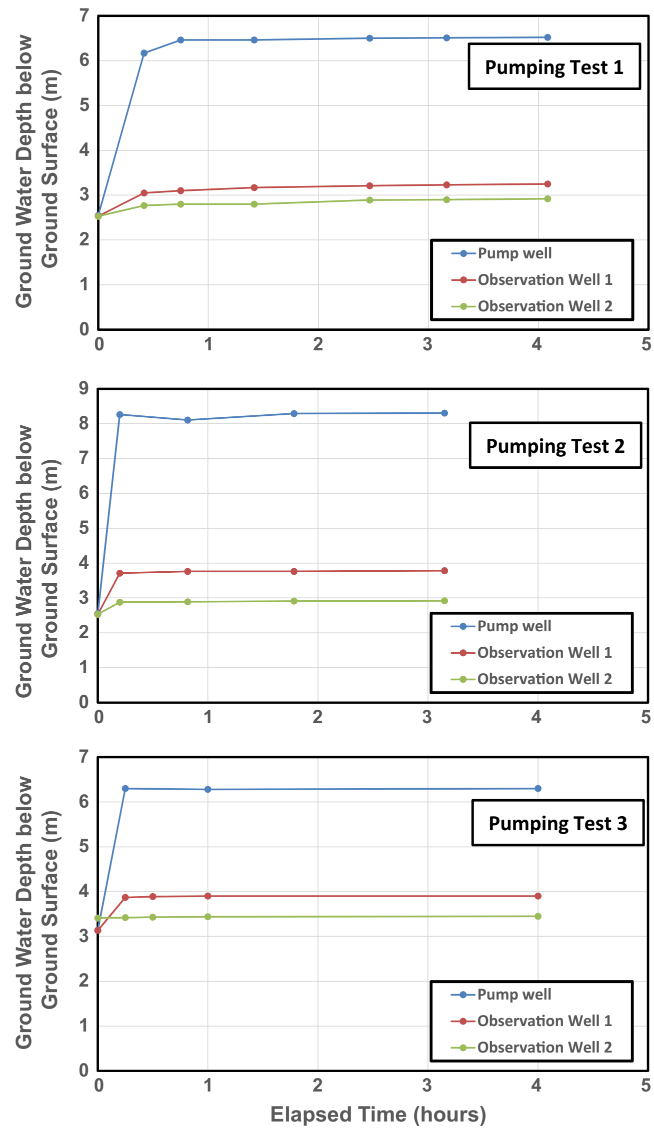

Figure 4 Variation of water level with time at the locations of the pump and observation well.

test is performed by filling the casing with water which is allowed to seep into the soil. The water depth inside the casing is measured at specific time intervals from the start of the test. These measurements are made until the rate of drop is very small or a sufficient number of readings are obtained to accurately determine the permeability. Field falling head tests provide more reliable data compared to laboratory tests as they avoid soil disturbance and test a relatively larger volume of soil. In comparison with pumping tests, the falling head test measures the permeability of a limited volume of soil but provides a more detailed profile of the permeability versus depth. Fig. 5 shows the measured values of permeability versus depth at three boreholes. The measured permeabilities varied between 4.1×10^{-4} m/s and 8.2×10^{-8} m/s with an average value of approximately 2×10^{-5} m/s. Most of the values lie between 5×10^{-7} m/s and 1×10^{-4} m/s, which are within the typical range of permeabilities of very fine sands and silty sands [22]. Fig. 5 also compares the falling head permeability

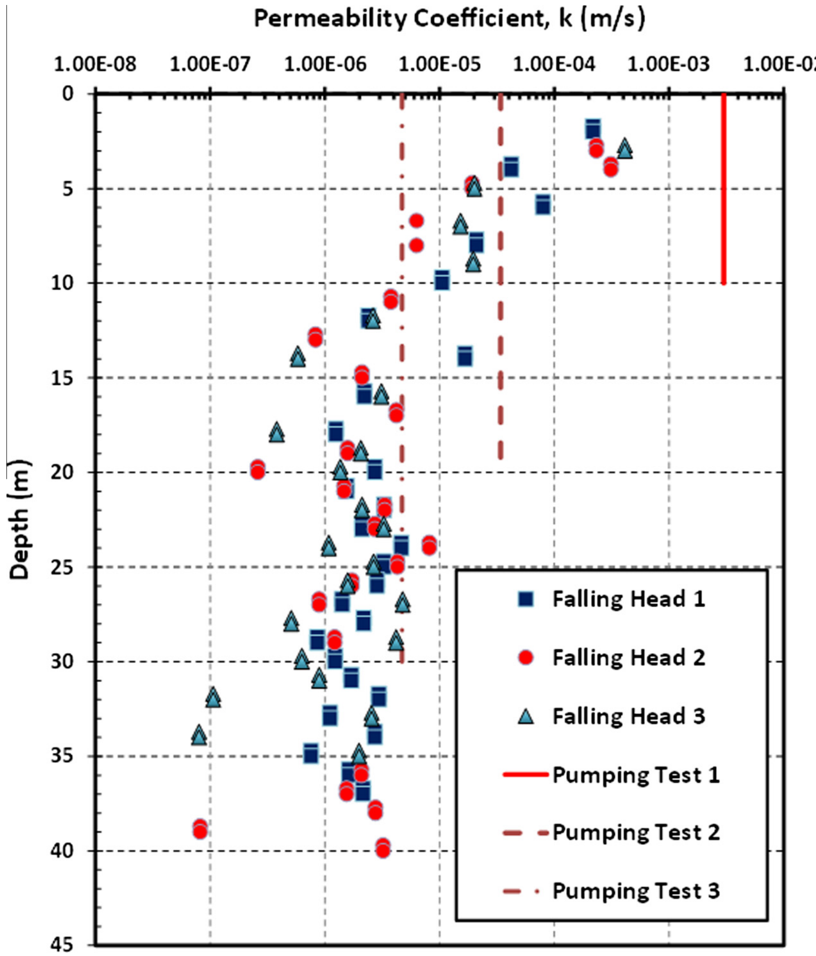

Figure 5 Field permeability from falling head test versus depths compared to pumping test results.

test results with the values obtained from pumping tests. Both tests show similar trends with the measured permeabilities decreasing with depth. Pumping test 1 provides a coefficient of permeability slightly higher than the values obtained from the falling head test. On the other hand the permeability values obtained from pumping tests 2 and 3 compare well with the values measured from the falling head test.

5. Determination of the permeability from CPT and CPTu

Soil permeability can be estimated from CPT based on the soil type (e.g. [14,13]). Alternatively, it can be evaluated using the piezocone (CPTu) based on the results of pore water pressure dissipation tests performed at specific depths (e.g. [23–25]). Soil permeability was correlated with soil type using the Soil Behavioral Type (SBT) charts as proposed by Lunne et al. [14]. Soil permeability was estimated using the Soil Behavior Type Index (I_c) using Eq. (1) proposed by Robertson [15].

$$I_c = [(3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2]^{0.5} \tag{1}$$

where

$Q_{tn} = [(q_t - \sigma_{vo})/p_a]\,(p_a/\sigma'_{vo})^n$,
$F_r = [f_s/(q_t - \sigma_{vo})]$ 100%,
q_t = CPT corrected cone resistance,
f_s = CPT sleeve friction,
σ_{vo} = in situ total vertical stress,
σ'_{vo} = in situ effective vertical stress,
$n = 0.381\,(I_c) + 0.05\,(\sigma'_{vo}/p_a) - 0.15$, where $n \leqslant 1.0$,
p_a = atmospheric pressure in same units as q_t, σ_{vo} and σ'_{vo}.

The Soil Behavior Type Index (I_c) is determined iteratively by assuming a value of n to compute Q_{tn} that is used to calculate the corresponding I_c. Iterations are performed till the value of n reaches convergence. It has been shown that I_c increases as the soil becomes finer (e.g. [15,26]). Accordingly, the soil permeability decreases as I_c increases. Robertson [15] proposed the following equations for evaluating soil permeability based on I_c.

$$\text{For } 1.0 < I_c \leqslant 3.27, \quad k\ (\text{m/s}) = 10^{(0.952 - 3.04\, I_c)} \tag{2}$$

$$\text{For } 3.27 < I_c < 4.0, \quad k\ (\text{m/s}) = 10^{(-4.52 - 1.37\, I_c)} \tag{3}$$

Eqs. (2) and (3) provide approximate values of the soil permeability. This technique is useful in providing a detailed permeability profile with depth using CPT results. Fig. 6 shows the variation of the coefficient of permeability with depth estimated from the CPT soundings compared to the falling head test results. The values of the coefficient of permeability measured from the falling head permeability test varied between 4.21×10^{-5} m/s and 4.11×10^{-4} m/s, with an average value

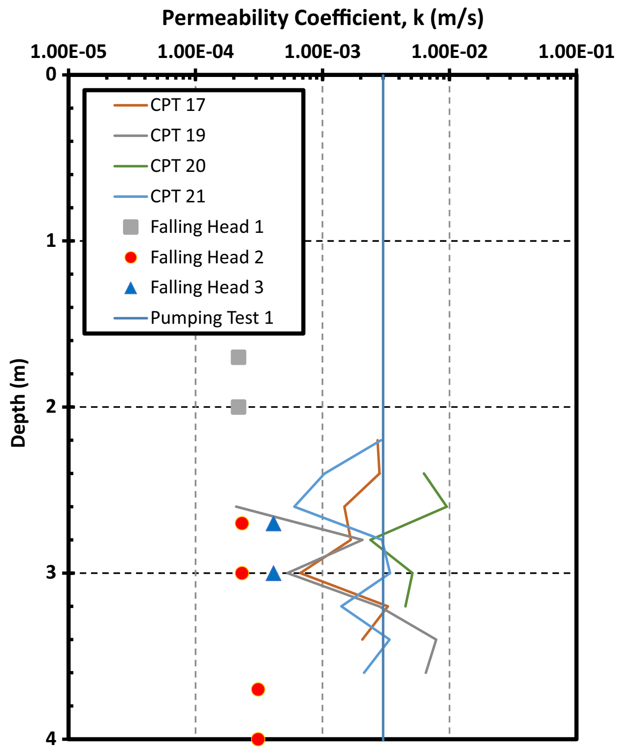

Figure 6 Variation of the coefficient of permeability estimates from CPT with depth compared with the falling head and pumping test results.

of 2.43×10^{-4} m/s. The permeability values estimated from the CPT ranged between 7.98×10^{-2} m/s and 1.92×10^{-6} m/s, with a mean coefficient of 3.86×10^{-3} m/s. These values are within the typical range of the coefficient of permeability for sands (e.g. [27]). The permeability values estimated from the CPT readings are approximately half to one order of magnitude higher than the measured permeability using the falling head field test. Better agreement is found between the permeabilities measured from the pumping tests compared to those estimated from CPT results. Noting the high variability in soil permeability [17], the CPT estimates of permeability provide reasonable estimates of permeability.

6. Estimation of permeability using index sand properties

The permeability of granular soils is affected by their grain size distribution, which is typically described by the equivalent particle diameters D_{10}, D_{30} and D_{60} corresponding to 10%, 30% and 60% passing by weight. Other commonly used indices are the coefficients of curvature C_C and uniformity C_u. The coefficient of curvature $C_c = \frac{D_{30}^2}{D_{10}D_{60}}$ describes the curvature (or concavity) of the grain size distribution curve. The coefficient of curvature, which is computed using three points on the grain size distribution curve, differentiates between gap-graded and well-graded soils [11,12]. Another descriptive parameter of the grain size distribution is the coefficient of uniformity C_u which is defined as the ratio of D_{60}–D_{10}. Higher values of the coefficient of uniformity are indicative of well graded soils. Onur [11] found that well graded sands have lower permeabilities compared to uniformly graded sands as the smaller grains fill the voids between the larger grains in well graded sands resulting in less interconnectivity. Additionally, soil permeability increases by increasing the voids ratio. Permeability is also affected by the particle shape and arrangement as these parameters influence the interconnectivity between the voids. Soils with angular particles exhibit lower permeability compared to more rounded soil particles [28].

A large number of correlations were developed to, empirically, evaluate soil permeability using the grain size distribution (e.g. [8,9,29,30,10,31–33]). The most commonly used correlation proposed by Hazen [8] is presented below:

$$k\ (\mathrm{m/s}) = C D_{10}^2 \tag{4}$$

where C is a constant which could vary between 0.1 and 100 [10]. However, the typical values of C range between 0.4 and 1.2 and are typically taken as 1 [28].

Alternatively, Kozney proposed another semi-empirical equation for estimating permeability in 1927 which was later modified by Carman [9]:

$$k = \left(\frac{\gamma}{\mu}\right)\left(\frac{1}{C_{\text{K-C}}}\right)\left(\frac{1}{S_0^2}\right)\left[\frac{e^3}{1+e}\right] \tag{5}$$

where γ = the unit weight of the fluid, μ = the viscosity of the fluid, $C_{\text{K-C}}$ = Kozney-Carman empirical coefficient, S_0 = specific surface area divided by the volume of the particles and e = voids ratio. The value of $C_{\text{K-C}}$ was reported to be 4.8 ± 0.3 for spheres with uniform diameter according to Carman [9]. Hence, it is typical to use $C_{\text{K-C}}$ equal to 5 [10]. The Kozney-Carman equation has not been regularly used by geotechnical engineers partly because soil specific area is not classically measured during routine soil tests. Carrier [10] demonstrated that the soil specific surface can be estimated from the grain size distribution assuming soil particles are spherical with non-uniform diameters according to Eq. (6).

$$S_0 = \mathrm{SF}/D_{\mathrm{eff}} \tag{6}$$

where
SF = shape factor that depends on particle angularity,
D_{eff} = effective particle diameter = $100\%/[(\Sigma(f_i/D_{\mathrm{ave\ i}}))]$.

The values of the shape factor, as reported in the literature, vary between 6 and 8.4 for spherical and angular particles, respectively. A shape factor of 6.6 was used for the soil under investigation. Assuming that the grain size distribution is log-linear between consecutive sieve sizes, the equation for estimating permeability would be reduced to Eq. (7) [10]:

$$k = 1.99 \times 10^4\left(100\%/\left\{\Sigma[f_i/(D_{li}^{0.404} \times D_{si}^{0.595})]\right\}\right)^2(1/\mathrm{SF}^2) \times [e^3/(1+e)] \tag{7}$$

where
f_i (%) = fraction of soil particles between two sieve sizes, larger (l) and smaller (s),
$D_{\mathrm{ave\ i}}$ = average particle size retained between sieves = $D_{li}^{0.5} \times D_{si}^{0.5}$.

Both Hazen and Kozney-Carman formulae are appropriate for soils with no electrochemical reactions between the soil particles and water i.e. the formulae are not suitable for clayey soils. The equations are derived assuming laminar flow in accordance with Darcy's law, making them applicable for flow

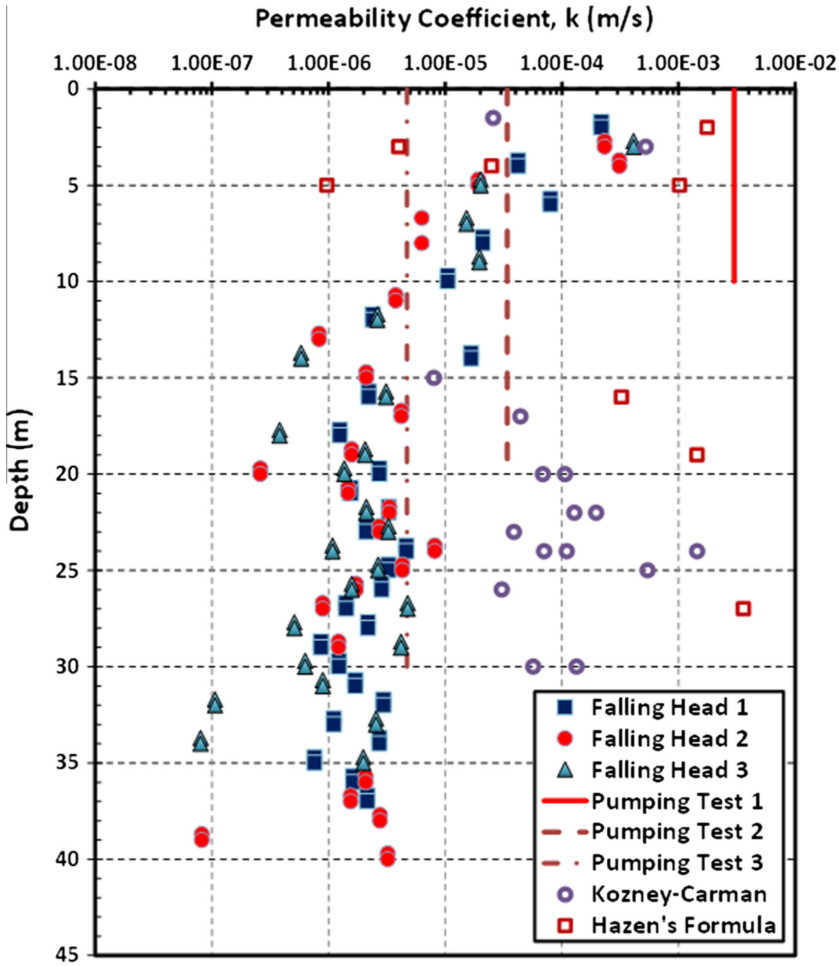

Figure 7 The coefficient of permeability estimates based on index soil properties versus depth compared to falling head permeability and pumping test results.

through silts and sands. These assumptions would not be valid for gravels with large pore sizes as the velocity of flow increases and the effect of turbulence should be considered. These simplified equations cannot be used for extreme particle shape and size distributions [28].

Fig. 7 shows the variation in the estimated coefficient of permeability versus depth using both Kozney-Carman and Hazen's formulae. The estimated values range between 9.7×10^{-7} m/s and 3.6×10^{-3} m/s. The average estimated coefficients of permeability were 2.2×10^{-4} m/s and 1.0×10^{-3} m/s using Kozney-Carman and Hazen's formulae, respectively. It is noted that the permeability estimated based on the grain size distribution provides reasonable values compared to measured permeability in the top soil layers where there is no or weak cementation. On the other hand, the permeabilities estimated from the grain size distribution in the very dense sand—with cemented bands and lumps/sandstone layers—are approximately one order of magnitude higher compared to the measured values. This may be attributed to the influence of cementation on reducing permeability, which is not accounted for in the empirical formulae. This highlights the importance of calibrating empirical correlations using site specific measurements. Kozney-Carman and Hazen's formulae would yield better estimates of permeability in the cemented layers by reducing the estimates obtained from Eqs. (4) and (7) by one order of magnitude.

7. Discussion and summary

Permeability was measured using field falling head permeability and full scale pumping tests. The recorded values of permeability exhibited natural variability within the expected range of 240% [17]. The falling head test measures the permeability at specific depths yielding a detailed permeability profile versus depth. Conversely, the pumping test provides an average permeability for the soil stratum. The measured permeability values lie within the typical ranges for sands and silty sands (e.g. [22,21]). The permeability profiles from both field tests demonstrate that the permeability decreases with depth, which reflects the effects of increasing soil compactness and cementation with depth. The coefficient of permeability was also empirically evaluated using the cone penetration test results based on the mechanical response of the soil during penetration [15]. In general, the permeability values obtained using the CPT were in agreement with the field measured values. The CPT was used to estimate permeability in the top uncemented sand layer as the cone penetrometer could not penetrate the cemented sand layers.

Alternatively, the coefficient of permeability was obtained using the grain size distribution [8]. The coefficient of permeability was also computed using the grain size distribution, void ratios, and particle shape by means of the correlations proposed by Carrier [10]. Onur [11] found that uniformly graded sands have higher permeabilities compared to well graded sands as the smaller soil particles fill the voids between the larger ones. The coefficients of curvature and uniformity may be used to describe the degree of uniformity of coarse grained soils. In this paper, the coefficient of permeability evaluated using the Hazen [8] and Carrier [10] formulae yielded relatively good results in the uncemented sand layer. On the other hand, these formulae overestimated the permeability in the cemented sand/sandstone layers by approximately one order of magnitude, which may be attributed to the higher cementation—not accounted for in the empirical formulae.

This indicates that there is no generalized method for estimating soil permeability for all soil types. It is important to calibrate such empirical method using actual field measurements especially for important projects. Furthermore, it is important to be aware that each method of measurement has its own limitations and shortcomings that should be taken into consideration. Drilling or pushing a probe into the ground may destroy the soil structure, thus affecting the in situ measurements. Borehole permeability tests measure the permeability at a specific depth by testing a small soil mass around the borehole. On the other hand, pumping tests offer a representative average permeability of the aquifer.

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