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Centrifuge Technique for Modelling Permeability of Compacted Soils

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Introduction

The main objective of a modelling exercise is to simulate and predict a phenomenon or a mechanism. In a complex situation like subsurface transport process, modelling can provide valuable information about the movement of ground water, the spread and growth of pollutant plumes, and the effectiveness of various containment and remedial action strategies to be adopted or prescribed. This requires determination of permeability (also called coefficient of permeability or hydraulic conductivity) of a geotechnical material either in the laboratory or in-situ. The movement of contaminants in underground flow systems is often predicted by using mathematical modelling techniques. During mathematical modelling, the processes under simulation are modelled by a set of governing equations, that are solved using either analytical or numerical methods. The results of a modelling exercise are wholly dependent upon complete understanding of the fundamental processes involved and accurate modelling of all relevant mechanisms. In the recent years, it has been felt that there is a need to understand and simulate transport mechanisms in soils in accelerated gravity environments. Such simulations are required to validate existing mathematical models, and to develop improved conceptual thinking for fundamental processes, which are being simulated.

In the past, either controlled field experiments or laboratory tests have provided most of the experimental data for transport mechanism in soils. Controlled field experiments facilitate incorporation of complexity of the real life situation. However, these tests are costly, time consuming, difficult to

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perform and generally offer little direct control over the boundary conditions. On the contrary, laboratory tests are generally inexpensive, not much time consuming and relatively easy to perform; at the same time, these tests are of limited value due to their inability to model realistic prototype conditions.

In such a scenario, researchers have recognised the potential of a geotechnical centrifuge to model the transport mechanisms in soils. In a geotechnical centrifuge, complex two and three-dimensional problems, can be modelled under reproducible and controlled boundary conditions. It has been shown that the prototype effective stress regime can be simulated in a centrifuge model with no loss of generality (Arulanandan et al., 1988). However, due to the extreme heterogeneity of the natural soil deposits, their structure and stress conditions, the real field conditions can not be simulated in the laboratory for a soil, and hence the obtained results can not corroborate to the field conditions.

The conventional permeameters used in laboratory are mainly, consolidation cell permeameters, compaction mould permeameters and triaxial cell permeameters. Centrifuge tests, as mentioned above, are also being conducted to estimate the permeability of a geotechnical material under accelerated gravity conditions (Alemi et al., 1976; Nimmo et al., 1987; Mitchell, 1994 a and b). The main advantage of these tests being recreation of prototype vertical effective stresses in the soil samples.

This paper deals with the details of centrifuge modelling of permeability of a compacted soil using a small geotechnical centrifuge. The effect of accelerated environment has been studied on the soil samples with different compaction states. Conventional 1-g falling-head and consolidation tests have also been conducted to evaluate permeability of the soil in the same initial compaction states. The permeability modelling has been attempted by evaluating scaling relationship between 1-g and centrifuge test results.

State-of-the-art on Permeability Studies

1-g permeability tests

Initially permeability studies on soils were considered of importance in connection with the problems associated with seepage, and its effects on settlement and stability of the hydraulic systems. Significant contributions, in this direction, have been made by Lambe (1954), Bjerrum and Huder (1957), Lambe (1958), Seed and Chan (1959), Olsen (1962), Miller and Low (1963), and Mitchell et al. (1965). Lambe (1954) devised a simple apparatus for measuring permeability of soil samples using a constant head method.

Prior to 1970, geotechnical engineering had little input into waste management practices but with the changing times and recent legislative requirements to examine and assess all potential waste sites, the design of containment systems has attracted attention of the Civil Engineering profession. One general solution to avoid groundwater contamination by waste disposal is to design hydraulic barriers (Horizontal or sloping) better named as 'liners'. These liners may be geomembranes or natural liners viz., compacted clays, silty soils, soil bentonite mixes, and mine tailings. Some authors have described the hydraulic behaviour of compacted clays (Daniel, 1984; Harrop-Williams, 1985; Day and Daniel, 1985), while others have described silty soils (Holtz, 1985), mine tailings (Jessberger and Beine, 1981), and soil-bentonite mixes (Lundgren, 1981; Chapuis, 1990). In the past few years, guidelines have been compiled for selecting appropriate soil properties and compaction methods that are likely to result in low permeability (Gordon et al., 1984; Daniel, 1990). These guidelines are based on experience and generally include minimum values or acceptable ranges for properties that describe composition of soil such as Atterbergs' limits and particle size distribution, and recommendations for selecting compaction criteria (i.e. moisture content and dry unit weight) and compaction efforts used. A geotechnical engineer also faces the challenge of extrapolation of soil properties measured at the laboratory scale to the field scale. The investigations of in-situ permeability of compacted soils have been performed by Daniel (1984), Day and Daniel (1985), Daniel and Trautwein (1986), Elsbury and Sraders (1989), and Benson and Daniel (1990). These investigations confirm the disparity between the 'in-situ' and the 'laboratory' simulations of permeability values. This is mainly due to the structure of the soil compacted in the laboratory when compared to the structure of soil as compacted in the field. The soils in the field and laboratory should be in a similar state of compaction otherwise the permeability may be significantly different. To create similar soil conditions in the laboratory and field, the variables that influence the structure of compacted soil must be carefully controlled. An important observation from these investigations is that laboratory measurements of permeability sometimes underestimate the in-situ permeability by at least an order of magnitude.

The permeability behaviour of soil bentonite mixes that takes into account several parameters such as particle size distribution, compaction efforts, bentonite content, porosity, degree of saturation, and laboratory and field performance has been investigated by Chapuis (1990), Haug and Wong (1992), and Van Ree et al. (1992). Aubertin et al. (1996) have carried out laboratory investigations on permeability of homogenised tailings from hard rock mines. Suthaker and Scott (1996) have established a relationship between the void ratio and permeability for fine tails resulting from oil sand processing. Kraus et al. (1997) have studied permeability of compacted paper mill sludge in order to assess their suitability as a barrier layer for landfill final covers.

Centrifuge modelling

One of the earliest efforts to study permeability of saturated soils, using geotechnical centrifuge, was made by Alemi et al. (1976). They have proposed two centrifugation techniques, one based on the redistribution of moisture within a soil core after centrifugation, known as the "transient method" and then solving a moisture flow equation to ascertain the value of permeability, and the other based on "measurement of volumetric outflow" of water from the soil core, when the speed of centrifugation is suddenly increased, followed by derivation and solution of the moisture flow equation to yield the permeability.

Since 1976, many researchers have worked on the centrifuges, all over the world, to simulate centrifuge permeability tests. Cargill and Ko (1983) examined the phenomena of transient water flow, in earth embankments, through centrifugal modelling, and proved the feasibility of modelling the transient flow phenomena in the centrifuge. Goforth et al. (1991) evaluated the feasibility of using centrifuge techniques to predict the movement of fluid through saturated and unsaturated soils by deriving and using equations for fluid motion through saturated soils within a centrifuge; an excellent agreement has been noticed for permeability values, for a sand and a sand-clay in saturated conditions, for both centrifuge and 1-g bench tests. Hensley and Schofield (1991) studied the long-term transport of conservative contaminants in soil, surrounding an engineered landfill site, using a balanced arm Cambridge University Centrifuge simulating 30 years of prototype time. The authors further illustrated the enormous potential of centrifuge in providing good quality experimental data for the verification of existing mathematical transport models.

Critical appraisal

A critical review of the literature presented in the above indicates that mostly clayey soils have been used for modelling purposes using large centrifuges. Large centrifuges incorporate high cost of running and difficulty associated in preparing a homogeneous soil sample. To overcome this, inexpensive small centrifuges can be utilised advantageously to study long-term permeability behaviour. At the same time, a large number of samples can be tested economically. In the present study a small centrifuge is used to model permeability of a natural silty soil.

Centrifuge Modelling Technique

Centrifuge modelling enables the study and analysis of various geotechnical engineering problems. If the same soil is used in the model as in the prototype, and if a careful model preparation procedure is adopted ensuring that the packing of the soil particles is replicated, then for the centrifuge model subjected to an inertial acceleration field of N times earth's gravity the vertical stress at depth h_m will be identical to that in the corresponding prototype at depth h_p, i.e.

$$h_p = N h_m \tag{1}$$

where, suffixes m and p refer to the model the prototype respectively.

Equation (1) is the basic scaling relationship for centrifuge modelling, which states that stress similarity is achieved at homologous points in the model and its prototype by accelerating a model of scale $1/N$ to N times earth's gravity. Further, if an acceleration of N times earth's gravity (g) is applied to a soil mass of density ρ, then the vertical stress σ_m at depth h_m in the model is given by:

$$\sigma_m = \rho N g h_m \tag{2}$$

and in the prototype

$$\sigma_p = \rho g h_p \tag{3}$$

Thus if $\sigma_m = \sigma_p$, then $h_m = h_p N^{-1}$, hence the scale factor; model to prototype for linear dimensions is $1/N$. Since the model is a linear scale representation of the prototype, then displacements will also have a scale factor of $1/N$. It therefore follows that strains have a scale factor of 1 and so the part of the soil stress-strain curve mobilised in the model will be identical to the prototype. Again, it is possible to accelerate the flow or transport processes when a fluid passes through the soils and aquifers. Large models can also be tested on large-capacity centrifuge and hence enabling inclusion of large number of heterogeneities in the model. The transport mechanism occurring in the centrifuge models under controlled boundary and initial conditions may be used as 'field data' to verify and improve the capabilities and efficiency of various mathematical models. To predict the prototype behaviour correctly from the observation of model behaviour, similarity condition must be established for the model and the prototype.

While acceleration due to earth's gravity is uniform for the practical range of soil depths encountered in civil engineering practice, the situation in a spinning centrifuge is different where acceleration field varies throughout the model. This is because the inertial acceleration field is given by where $\omega^2 r$ is the angular rotational speed of the centrifuge and r is the distance to any element in the soil model from the axis of rotation. Taylor (1995) has determined the effective centrifuge radius to be selected which overcomes the above problem to a great extent. The effective centrifuge radius is given by the following equation

$$R = R_t + \frac{h_m}{3} \tag{4}$$

where R = effective centrifuge radius measured from the central axis, and

R_t = radius to the top of the model.

Equation (4) indicates that the effective centrifuge radius should be measured from the central axis to one third the depth of the model. If this effective radius is considered, the maximum error due to the non-linear stress distribution has been shown to be quite small for most of the centrifuges.

Further, the speed of centrifuge arm can be calculated by:

$$\text{Speed of centrifuge (rpm)} = \frac{60}{2\pi}\sqrt{\frac{Ng}{R}} \tag{5}$$

Scale factor for permeability

For fully saturated soils, the Darcy's law for seepage flow is given by:

$$v = ki \tag{6}$$

where v = average seepage flow velocity,

k = Darcy's coefficient of permeability (or simply permeability), and

i = hydraulic gradient.

Further, the intrinsic permeability, K, is related to Darcy's permeability coefficient k, as follows:

$$k = \frac{K\rho g}{\mu} \tag{7}$$

where ρ = mass density of the fluid, and

μ = absolute viscosity of the fluid.

K is a function of the shape, size and packing of the soil grains. If the same pore fluid is used in model and prototype then Eqn. (7) can be rewritten as:

$$\frac{k_m}{k_p} = \frac{\dfrac{K \rho g_m}{\mu}}{\dfrac{K \rho g_p}{\mu}} = \frac{g_m}{g_p} = N \tag{8}$$

or $$k_m = N k_p \tag{9}$$

i.e. the Darcy's coefficient of permeability in a centrifuge model is N times greater than that of the same soil in 1-g environment in the prototype. Hydraulic gradient is defined in the usual way as the ratio of drop in head of the pore fluid, Δh, to the length, Δl, over which the drop occurs. The hydraulic gradient has the same value in the centrifuge model and the prototype because the head and the sample length scale in the same proportion; thus it can be assumed that the hydraulic gradient does not scale, i.e. $i_m = i_p$. As such,

$$v_m = i_m k_m = i_p N k_p = N v_p \tag{10}$$

Equation (10) indicates that the velocity of fluid in the centrifuge model is N times greater than in the prototype; this increase in seepage velocity occurs because of N times increase in permeability in centrifuge models as defined in Eqn. (9). This reasoning helps modelling low permeability soils in a geotechnical centrifuge.

The flow paths, along which pore fluid travels, have a scale factor of 1/N for length. The time for seepage flow is then given by:

$$t_m = \frac{L_m}{v_m} = \left(\frac{L_p}{N}\right)\left(\frac{1}{N v_p}\right) = \frac{1}{N^2} t_p \tag{11}$$

Validity of Darcy's law in a Centrifuge

The validity of Darcy's law for fluid flow in the model is checked by Reynolds number, R_e, given by the following equation:

$$R_e = \frac{\rho v d}{\mu} \tag{12}$$

where d is the characteristic microscopic length, such as particle size, of the soil. R_e represents the dynamic similarity of fluid motion, which can be established by ensuring that the ratio of inertial forces to viscous forces in the fluid remain invariant in the centrifuge model and prototype respectively. But, since flow velocities are scaled in a centrifuge model, it is not possible to maintain R_e constant, if identical soils are used in model and prototype. However, in most of the cases encountered, with respect to seepage flow, the inertial forces are negligible in comparison to the viscous resistance (i.e. $R_e < 1$). In such cases, the condition that the Reynolds number remains invariant can be waived, and Darcy's law can be used to describe the fluid motion. In the present study, the maximum value of Reynolds number is found to be equal to 0.02 that indicates the validity of Darcy's law for fluid flow in centrifuge models.

Experimental Investigations

Soil properties

Locally available silt has been selected to model permeability using a geotechnical centrifuge. This soil is then subjected to a series of tests to identify its physical properties viz., specific gravity, gradational characteristics, Atterberg limits, compaction and consolidation characteristics. The results of such tests are summarised in Table 1.

The gradational characteristics of the soil are determined as per ASTM

TABLE 1 : Properties of the soil

Specific gravity	2.79
Particle size characteristics	
Sand	
Coarse (4.76-2.0 mm.)	3.0%
Medium (2.0-0.420 mm.)	19.0%
Fine (0.420-0.074 mm.)	28.0%
Fines	
Silt size (0.074-0.002mm.)	35.5%
Clay size (<0.002mm.)	14.5%
Consistency limits	
Liquid limit	41%
Plastic limit	28%
Plasticity index	13%

standard D422-63. The percentage of particle sizes presented in Table 1 is the average of a series of four tests. The Atterberg limits are determined by using experimental procedures as outlined in ASTM standard D4318-93. Using the Unified Soil Classification System (USCS), the soil can be identified as ML (silt). The Standard Proctor compaction tests are performed in the laboratory as per ASTM standard D698-91.

The consolidation behaviour of the soil has also been investigated to obtain the permeability of soil samples. As such, several tests have been performed on the soil samples named A, B, C, D, E, F, G, H, I, and J respectively (Table 2) in an oedometer cell, corresponding to different compaction states along the Standard Proctor Compaction curve. Consolidation tests have been conducted according to ASTM standard D2435-90.

Sample preparation

Oven dried soil has been mixed with predetermined quantity of water and stored in polythene bags for at least 24 hours to ensure proper mixing and maturing and final water content of matured soil was determined thereafter. Such a matured soil has been used for preparing soil samples to conduct various tests on the soil samples. Soil samples, representing various compaction states as shown in Table 2, have been tested for estimating soil permeability. These tests have been repeated two to three times to ascertain the reproducibility of the results.

TABLE 2 : Initial Moulding States of Soil Samples

Soil Sample	Moisture content (%)	Dry unit weight (kN/m^3)	Void ratio	S (%)
A	10.60	15.54	0.795	37.18
B	12.77	15.88	0.757	47.07
C	14.00	16.12	0.731	53.60
D	16.00	16.45	0.696	64.13
E	18.29	16.78	0.663	77.00
F	20.55	17.01	0.640	89.56
G	21.80	16.94	0.647	94.01
H	23.36	16.56	0.685	95.18
I	24.78	16.38	0.703	98.30
J	27.58	15.38	0.814	94.53

Falling-head tests using a compaction mould permeameter

Traditionally, the permeability of fine-grained soils has been measured either in rigid wall or flexible wall permeameter. Both methods have their own advantages and disadvantages (Daniel et al., 1985; Suthaker and Scott, 1996), however, both methods have shown similar measurements of permeability on the same soil when permeated with water (Daniel et al., 1985; Foreman and Daniel, 1986).

In the present study 'falling-head test' method has been used to estimate the coefficient of permeability of the soil samples. Samples for the rigid wall compaction mould permeameter were prepared by compacting the required amount of soil directly into the mould to get the initial compaction parameters as shown in Table 2. Soil samples have been saturated in the mould by applying a hydraulic gradient of 4 for 24 hours and using a vacuum pump. It is observed that the soil samples are 97% to 100% saturated. Falling-head tests have been performed on soil samples of 7.98 cm. in diameter and 6 cm in height.

Consolidation tests

The consolidation test is an indirect permeability test in which a sample of soil is compressed in a rigid ring at various vertical stress levels, with drainage facility at both top and bottom of the sample. The coefficient of permeability is calculated using Terzaghi's theory of one-dimensional consolidation using the following expression:

$$k = c_v \, m_v \, \gamma_w \tag{13}$$

where c_v = coefficient of consolidation,

m_v = coefficient of volume compressibility, and

γ_w = unit weight of water.

In Eqn. (13) c_v can be obtained by conventional $\sqrt{t}$ and $\log(t)$ relationships for the soil. Olson (1986) has shown superiority of $\sqrt{t}$ method over the $\log(t)$ method, of calculating c_v, which predicts permeability values closer to the experimentally obtained permeability values. As such, $\sqrt{t}$ method is used in this study for estimation of c_v and hence the permeability of the soil.

Kinsky et al. (1971) presented results from falling-head and consolidation tests and have obtained good agreement between the two. Similar results have also been obtained by Mesri and Olson (1971). Olson and Daniel (1981) have shown that for normally consolidated clays, the ratio

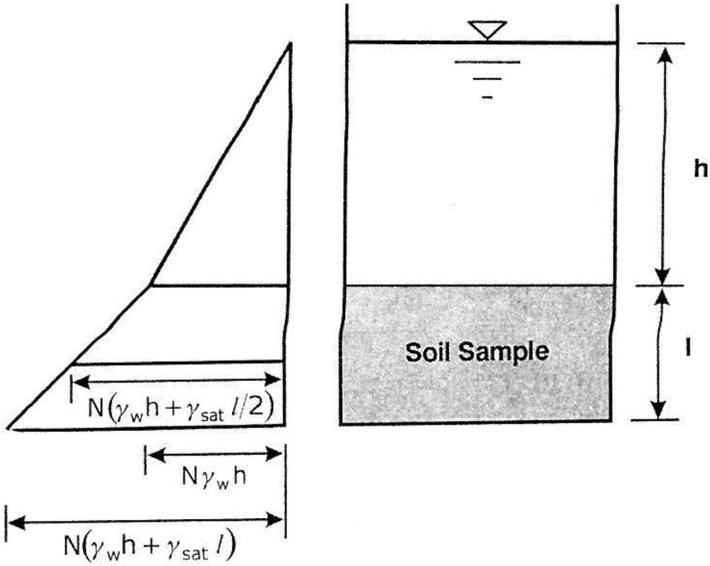

FIGURE 1 : Pressures Acting on a Soil Sample in an Accelerated Environment

of measured permeability to calculated permeability using Eqn. (13) varies from 0.9 to 5, however for over-consolidated soils this ratio varies from 2 to 1000. Tavenas et al. (1983 a and b) reported that the permeability values calculated using Terzaghi's theory of one-dimensional consolidation, underestimate the measured values by a maximum of six times, it has been further concluded in their study that the indirect methods are unacceptable in determining the permeability of highly compressible natural clays.

These studies highlight basic difference between the falling-head tests and the consolidation tests, viz. gravity induced flow of fluid, flow due to expulsion of pore fluid due to the external loading and presence of shear stress in consolidation tests (Budhu et al., 1991). Since the soil used in the present study is a normally consolidated silt, the consolidation tests have also been conducted along with the falling-head tests.

To simulate the effect of pressure generated by centrifugation on permeability of the soil samples, an oedometer has been used. Pressures acting on a soil sample in an accelerated environment (N-g) are presented in Fig 1. The state of stress acting on a soil sample of height l and an average water head of h, the pressure at the middle of the sample at Ng is equal to $(h \cdot N \cdot \gamma_w + \gamma_{sat} \cdot 1/2 \cdot N)$. Hence, pressure on the centrifuge model, at various g-levels, is equal to 1.01 times the N value (pressure at 50 and 100g corresponds to 50.5 and 101.0 kN/m^2 respectively).

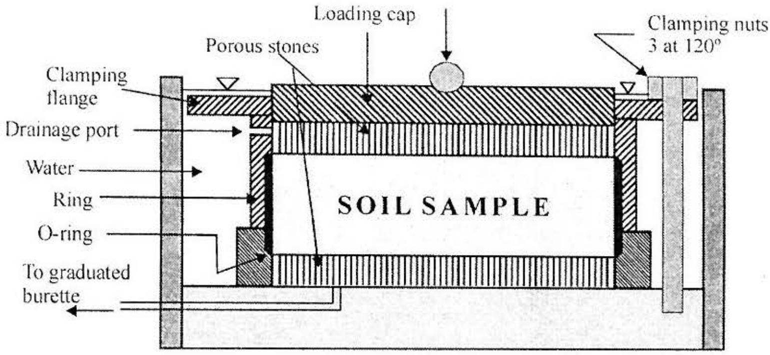

FIGURE 2 : Oedometer Falling-head Test Setup

Soil samples, 7.5cm in diameter and 2.5cm in thickness, have been used to obtain the permeability of the soil in the normal pressure range of 50 kPa to 200 kPa. At the end of the test, the degree of saturation of samples has been noticed to vary from 98% to 100%.

Oedometer falling-head tests

Soil samples (Table 2) were prepared in an oedometer ring of 7.5cm diameter and 2.5cm thickness for various compaction states of the soils. Falling-head permeability tests were conducted, using a calibrated burette connected to the base of the oedometer. Porous stones were placed both on the top and bottom of the sample to provide uniform flow of water. The setup of oedometer falling-head test is shown in Fig.2. This set-up behaves like a fixed wall permeameter, since the soil sample is laterally constrained by the fixed walls of the oedometer ring. This setup has been used to obtain falling-head permeability results corresponding to zero normal stresses.

Centrifuge tests

Details of the geotechnical centrifuge used for the present study are shown in Table 3. Centrifuge tests have been performed in a cylindrical Perspex container set up (inner diameter being 6.61cm) as shown in Fig.3. Samples of 3.0cm thickness were prepared at water contents and dry densities corresponding to the compaction characteristics of the soil as presented in Table 2. Soil samples have been prepared by compacting the soil directly into the cylindrical container The samples have been saturated (by applying a positive hydraulic gradient of 4 from bottom of the sample for 6 to 16 hours and then subjecting the sample to vacuum saturation for about 1 to 2 hours) before starting the centrifuge tests. This set-up, when spun in a centrifuge, simulates a falling-head test in an accelerated

TABLE 3: Details of the Centrifuge

Type	Swinging buckets on both sides of the arm
Arm radius	20 cm
Max. outer radius	31.5 cm
Centrifuge range	250-1000 rpm
Maximum acceleration	300g
Capacity	0.72g tons

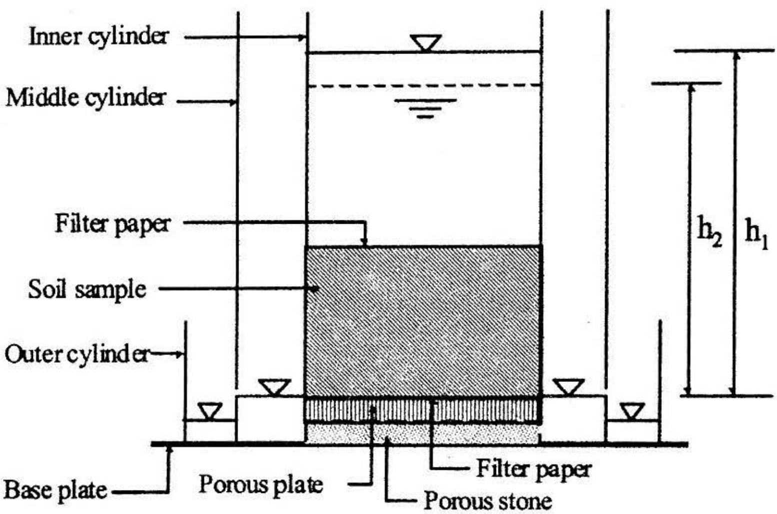

FIGURE 3 : Sample for Centrifuge Tests

environment. The permeability of the model, k_m, can be determined by the following expression which is similar to the general falling-head test setup:

$$k_m = \frac{l}{t} \ln\left(\frac{h_1}{h_2}\right) \tag{14}$$

where l = height of the soil sample,

t = time of centrifugation,

h_1 and h_2 = correspond to the initial and final heads respectively.

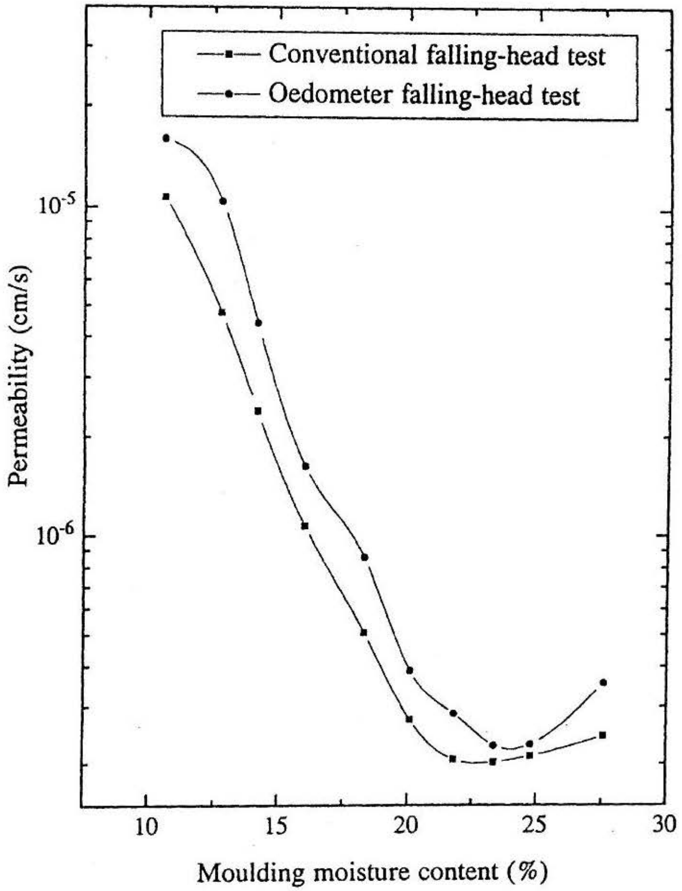

FIGURE 4 : Variation of Permeability with Moulding Moisture Content

Results and Discussions

The variation of permeability with moulding moisture content, for the state of soil samples as per Table 2, for conventional falling-head and oedometer falling-head tests is shown in Fig.4. Permeability values for the soil as obtained by various 1-g and centrifuge model tests are presented in Table 4.

It is evident from 1-g and centrifuge test results that the permeability shows a decreasing trend with moulding moisture content. These observations are well in agreement with the previous reported studies on permeability of compacted soils (e.g. Mitchell et al., 1965; Kennedy and Mitchell, 1995).

Table 4 : Test results

Soil sample	k_f (cm/sec)	k_o (cm/sec)	k_c (cm/sec)	N	k_{cen} (cm/sec)	x_f	x_o	x_c
A	1.61×10^{-5}	6.51×10^{-6}	7.57×10^{-6}	50	5.61×10^{-4}	0.908	1.139	1.101
		3.62×10^{-6}	5.25×10^{-6}	100	8.45×10^{-4}	0.860	1.184	1.103
		2.74×10^{-6}	4.34×10^{-6}	150	1.04×10^{-3}	0.832	1.185	1.093
		1.86×10^{-6}	3.42×10^{-6}	200	1.31×10^{-3}	0.830	1.237	1.123
B	1.05×10^{-5}	5.86×10^{-6}	3.91×10^{-6}	50	2.38×10^{-4}	0.798	0.947	1.050
		3.02×10^{-6}	2.06×10^{-6}	100	3.63×10^{-4}	0.769	1.040	1.123
		2.04×10^{-6}	1.50×10^{-6}	150	4.85×10^{-4}	0.765	1.092	1.153
		1.07×10^{-6}	9.46×10^{-7}	200	5.72×10^{-4}	0.755	1.185	1.209
C	4.42×10^{-6}	2.61×10^{-6}	1.98×10^{-6}	50	1.19×10^{-4}	0.842	0.976	1.047
		1.81×10^{-6}	9.27×10^{-7}	100	1.84×10^{-4}	0.810	1.004	1.149
		1.33×10^{-6}	8.22×10^{-7}	150	2.56×10^{-4}	0.810	1.050	1.146
		8.49×10^{-7}	7.16×10^{-7}	200	3.15×10^{-4}	0.805	1.117	1.149
D	1.63×10^{-6}	9.78×10^{-7}	9.08×10^{-7}	50	5.42×10^{-5}	0.896	1.026	1.045
		8.14×10^{-7}	6.53×10^{-7}	100	8.92×10^{-5}	0.869	1.020	1.068
		6.68×10^{-7}	5.41×10^{-7}	150	1.24×10^{-4}	0.865	1.043	1.085
		5.22×10^{-7}	4.29×10^{-7}	200	1.48×10^{-4}	0.851	1.066	1.103
E	8.59×10^{-7}	6.44×10^{-7}	4.06×10^{-7}	50	2.41×10^{-5}	0.852	0.926	1.044
		5.28×10^{-7}	2.84×10^{-7}	100	4.44×10^{-5}	0.857	0.962	1.097
		4.42×10^{-7}	1.97×10^{-7}	150	6.03×10^{-5}	0.848	0.981	1.142
		3.57×10^{-7}	1.09×10^{-7}	200	6.62×10^{-5}	0.820	0.986	1.210
F	3.88×10^{-7}	2.15×10^{-7}	2.38×10^{-7}	50	1.40×10^{-5}	0.917	1.068	1.042
		1.44×10^{-7}	1.70×10^{-7}	100	2.67×10^{-5}	0.919	1.134	1.098
		1.20×10^{-7}	1.37×10^{-7}	150	3.72×10^{-5}	0.911	1.145	1.118
		9.65×10^{-8}	1.05×10^{-7}	200	4.51×10^{-5}	0.898	1.160	1.144
G	2.85×10^{-7}	1.69×10^{-7}	1.97×10^{-7}	50	1.25×10^{-5}	0.967	1.100	1.061
		1.08×10^{-7}	1.47×10^{-7}	100	2.29×10^{-5}	0.952	1.163	1.096
		9.66×10^{-8}	1.23×10^{-7}	150	3.16×10^{-5}	0.940	1.156	1.107
		8.52×10^{-8}	9.88×10^{-8}	200	3.84×10^{-5}	0.925	1.153	1.125

Table 4 Contd.....

Soil sample	k_f (cm/sec)	k_o (cm/sec)	k_c (cm/sec)	N	k_{ccn} (cm/sec)	x_f	x_o	x_c
H	2.27×10^{-7}	1.41×10^{-7}	1.95×10^{-7}	50	1.13×10^{-5}	0.999	1.121	1.038
		1.07×10^{-7}	1.44×10^{-7}	100	2.11×10^{-5}	0.984	1.147	1.083
		9.52×10^{-8}	1.19×10^{-7}	150	2.79×10^{-5}	0.960	1.134	1.089
		8.34×10^{-8}	9.48×10^{-8}	200	3.42×10^{-5}	0.947	1.136	1.111
I	2.29×10^{-7}	1.40×10^{-7}	2.04×10^{-7}	50	1.17×10^{-5}	1.006	1.131	1.035
		1.12×10^{-7}	1.48×10^{-7}	100	2.11×10^{-5}	0.982	1.138	1.077
		1.01×10^{-7}	1.22×10^{-7}	150	2.84×10^{-5}	0.962	1.125	1.088
		9.04×10^{-8}	9.66×10^{-8}	200	3.44×10^{-5}	0.946	1.121	1.109
J	3.56×10^{-7}	2.27×10^{-7}	2.35×10^{-7}	50	1.29×10^{-5}	0.918	1.033	1.024
		1.63×10^{-7}	1.68×10^{-7}	100	2.27×10^{-5}	0.902	1.072	1.065
		1.33×10^{-7}	1.33×10^{-7}	150	3.12×10^{-5}	0.893	1.089	1.089
		1.04×10^{-7}	9.88×10^{-8}	200	3.81×10^{-5}	0.882	1.114	1.124

Note : k_f, k_o, and k_c, are permeabilities with respect to falling-head, oedometer falling-head, and consolidation tests respectively; x_f, x_o, and x_c are scale factors with respect to falling-head, oedometer falling-head, and consolidation tests respectively. k_{ccn} is the permeability in centrifuge tests.

It can be noticed from Fig.4 that the permeability values for conventional falling-head tests are slightly lower than the oedometer falling-head tests at zero pressure. This difference can be attributed to the pressure caused by the water column on the soil sample in case of the conventional falling-head tests. As such, zero pressure oedometer test results have been used as 1-g falling-head test results. Permeability results obtained from conventional consolidation tests and results of centrifuge tests are also presented in Table 4.

The scaling relationship for permeability between 1-g tests (denoted by suffix p representing prototype) and centrifuge tests (denoted by suffix m representing model) can be written as:

$$\frac{k_m}{k_p} = N^x \tag{15}$$

where x is a scale factor whose value has to be ascertained with the help of various 1-g and centrifuge tests. To evaluate the values of x, Eqn. (15) can be written in the following form:

$$x = \frac{\ln\left(\frac{k_m}{k_p}\right)}{\ln(N)} \tag{16}$$

Using Eqn. 16, the scaling factors for permeability, for different g values, with respect to 1-g laboratory tests have been evaluated and are presented in Table 4; *as shown in this table*, x_f, x_o, *and* x_c *are the scale* factors evaluated with respect to conventional falling-head tests, oedometer falling-head tests, and the consolidation tests respectively. It can be noticed that these scaling factors are close to unity and are consistent with the scaling factors obtained by Eqn. 9.

Modelling of models

Figure 5, depicts modelling of models for Sample J. It is noticed that modelling of models yields prototype (1-g) permeability of 6.0×10^{-7} cm/sec

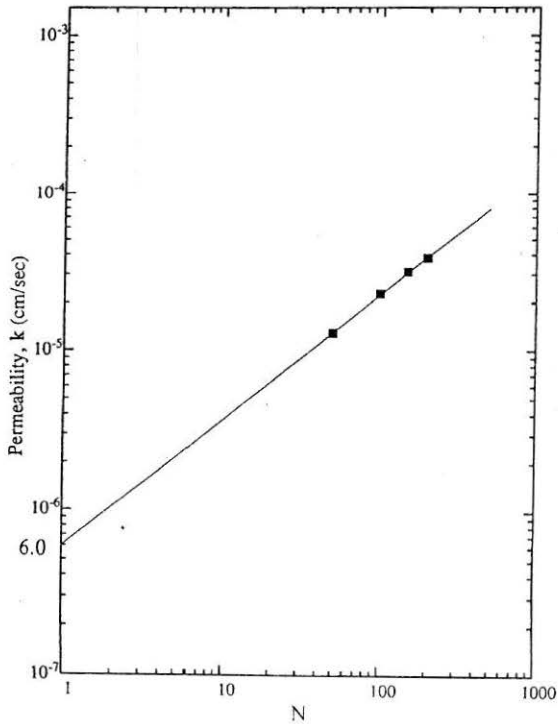

FIGURE 5 : Modelling of Models for Sample J

TABLE 5 : Comparison of 1-g Permeability Values

Soil sample	Falling head (cm/sec)	Modelling of models (cm/sec)	Y
A	1.61×10^{-5}	5.00×10^{-5}	3.10
B	1.05×10^{-5}	2.00×10^{-5}	1.90
C	4.42×10^{-6}	7.40×10^{-6}	1.67
D	1.63×10^{-5}	3.10×10^{-6}	1.90
E	8.59×10^{-7}	1.30×10^{-6}	1.51
F	3.88×10^{-7}	5.10×10^{-7}	1.31
G	2.85×10^{-7}	5.20×10^{-7}	1.82
H	2.27×10^{-7}	5.10×10^{-7}	2.25
I	2.29×10^{-7}	5.60×10^{-7}	2.45
J	3.56×10^{-7}	6.00×10^{-7}	1.68

which is in very close agreement with the falling-head permeability value of 3.56×10^{-7} cm/sec. Such a study indicates usefulness of centrifuge modelling. Similar studies have also been done for other states of the soil, as shown in Table 5. A factor Y, indicating the ratio of modelling of models results to the falling-head results, has been introduced which is noticed to vary from 1.3 to 2.5, in general. ,Keeping limitations of testing in view these values indicate reasonably good modelling of models. Such a study highlights usefulness of a geotechnical centrifuge for predicting/modelling permeability of compacted fine- grained soils.

Conclusions

On the basis of results and discussions presented in this paper, following generalised conclusions can be drawn:

1. It has been demonstrated that the coefficient of permeability gets modelled to N times in an accelerated environment. This is very much consistent with the mathematical models presented by various investigators.

2. The centrifuge test results, when extrapolated back to the prototype level, show a good agreement with the results of conventional 1-g tests.

3. Time taken to test a soil sample in a centrifuge is less as compared to 1-g tests. As such, centrifuge modelling is very useful for determining permeability of fine-grained soils.

4. Since the prototype stress conditions can be created (simulated) in a geotechnical centrifuge, the obtained permeability values are much more near to in-situ values.

5. There is a good agreement between permeability values obtained from falling-head tests and consolidation tests although permeability values obtained from consolidation tests are on lower side due to the applied pressure.

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Notation

c_v	=	coefficient of consolidation,
d	=	characteristic microscopic length (e.g. particle size)
h_1, h_2	=	initial and final water heads respectively,
h_p, h_m	=	height of prototype and model respectively,
g	=	acceleration due to gravity,

i	=	the hydraulic gradient,
k	=	coefficient of permeability,
k_{rep}	=	representative value of permeability,
k_m, k_p	=	permeability of the centrifuge model and prototype (1g) respectively,
K	=	intrinsic permeability,
L	=	length of flow path,
l	=	sample height,
$_{m}$, $_{p}$	=	suffixes indicating the model and prototype respectively,
m_v	=	volume compressibility of the soil,
N	=	Centrifuge acceleration level,
r	=	distance to any element in the soil model from the axis of rotation,
R	=	Reynolds number,
R_e	=	effective centrifuge radius measured from the central axis,
R_t	=	centrifuge radius to the top of the model,
S	=	degree of saturation,
t	=	time,
v	=	average seepage flow velocity,
w	=	moisture content,
x	=	the scale factor,
Y	=	ratio of modelling of models permeability to the falling-head permeability,
γ_w	=	unit weight of the water,
γ_{sat}	=	saturated unit weight of the soil,
γ_d	=	dry unit weight of the soil,
ρ	=	mass density,
σ_m, σ_p	=	vertical stresses in model and prototype respectively,
ω	=	angular rotational speed of the centrifuge,
μ	=	absolute viscosity.