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Preparation and Characterization of Model Clay Ground for Centrifuge Tests

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Introduction

The shear strength of normally consolidated soils in the field generally increases with depth as a consequence of the effective stress increase due to the self-weight of the soil. The importance of considering this shear strength increase with depth on the bearing capacity of foundations, stability of slopes and most other geotechnical structures has been well recognized in the literature (Hunter and Schuster 1968, Davis and Brooker 1973). These studies show that patterns of deformations and mechanisms of failure for clays having strength increasing with depth are markedly different from those with uniform strength.

Centrifuge modeling is extensively used in geotechnical engineering research to understand the soil behavior and model an equivalent prototype structure (eg., Schofield 1980, Cheney and Fragaszy 1984, Mitchell 1991). The stress condition and the strength variation with depth expected in the field can be properly simulated in a reduced scale model in a geotechnical centrifuge. Very often reduced scale normally consolidated clay beds are required for centrifuge tests. Preparation of normally consolidated clay beds in the centrifuge by self-weight consolidation of clay slurry often takes very long time. Therefore, alternate methods were developed for the preparation of normally consolidated clay beds.

This paper reviews the methods available to prepare normally consolidated (NC) clays, whose strength increases with depth for centrifuge tests. The limitations of the existing methods are brought out. The effectiveness of preparing NC clays by suction induced seepage consolidation (also called as hydraulic consolidation) is evaluated. In order to interpret the test results, an accurate evaluation of the strength profile in the model clay bed is essential. The advantages of using the T-bar penetrometer is also discussed in this paper.

Methods for the Preparation of NC Clay Beds

A normally consolidated clay stratum, whose undrained shear strength increases with depth, can readily be prepared in the centrifuge (Kimura *et al*. 1984). However, centrifuge time is expensive and soils having low permeability can take a long time to consolidate (Davies and Parry 1982, Schofield 1995, Robinson et al. 2003). To reduce the centrifuge time during the consolidation

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run, highly permeable clays like kaolin are often used in place of soils with low permeability such as marine clays (Davies and Parry 1982). However, the behavior of a model made from kaolin may differ substantially from the one made from prototype soil. In many situations, prototype soils may need to be used for the quantitative estimation of various parameters (Pedersen and Broers 1994, Onitsuka and Yamamoto 1994). Hence 1-g consolidation methods were developed to prepare normally consolidated clays with similar stress gradient and stress state as that prepared in the centrifuge.

Kitazume and Miyajima (1994) prepared a 40 cm thick clay bed with the strength increasing with depth at 1g by consolidating the clay bed in layers starting from the bottom. The bottom layer was first consolidated under a surcharge pressure equivalent to that expected in the centrifuge at the corresponding depth. After the consolidation of the first layer, the second layer was placed and consolidated under the estimated pressure expected in the centrifuge at this depth. This procedure was repeated till the required thickness is achieved. The limitation of this method is that the clay ground prepared by adopting this procedure will have a strength profile increasing with depth in steps instead of a uniform increase. The other method, which is more popular, is the downward hydraulic gradient consolidation method, which uses seepage stresses to generate the required effective stress gradient in the model (Zelikson 1969, Schofield 1995).

In the conventional form of the hydraulic consolidation method, the seepage stresses are generated by applying a high water pressure on to the top of the clay bed and the bottom is maintained at a pressure close to the atmospheric (Zelikson 1969, Imai 1979). This creates the required downward hydraulic gradient, which leads to downward seepage. Imai (1979) and Fox (1996) successfully conducted hydraulic consolidation tests using cylindrical containers. However, the method often fails because of hydraulic fracturing of the clay along the corners, when rectangular containers are used (Takemura 1998). As many centrifuge tests are conducted for plane strain problems, most of the centrifuge strong boxes are rectangular in shape. Hence it is important that an alternative method of inducing downward seepage forces on clay bed contained in rectangular boxes be developed. This paper discusses the effectiveness of achieving hydraulic consolidation by inducing seepage forces through the application of vacuum at the bottom of the clay bed.

Strength Characterization

In order to interpret the test results, an accurate evaluation of the strength profile in the model bed is essential. When relatively permeable soils are used, it is vital to measure the correct strengths of the clay beds during centrifuge flight, as the strength measured after stopping the centrifuge is reported to be lower than in flight because of swelling of the clay model associated with possible cavitation (Davies and Parry 1982, Tani and Craig 1995).

Several site investigation devices are used in the centrifuge during flight to assess the undrained shear strength of the model clay deposit. The cone penetration and vane shear tests are the most popular methods of estimating undrained shear strength (Malmeida and Parry, 1988). The vane shear devices developed for the centrifuge suffer from the problem of their physical size compared to the depth of the model foundation. It is only possible to perform a limited number of vane shear tests within the depth of the model and continuous profiling is not possible (Stewart and Randolph 1991). The cone penetration test (CPT) allows a continuous profile of shear strength with depth. When analyzing the CPT data a correction must be made for both the pore water pressure acting at the shoulder of the cone (pore pressure area correction) and the overburden pressure and can be correlated to the undrained shear strength (c_u) using (Robertson and Campanella 1983, Randolph et al. 1998)

$$c_u = \frac{q_c - (\sigma'_v + \alpha u_o)}{N_c(1-(1-\alpha)B_q)} \quad (1)$$

where, q_c is the cone resistance, σ_v' is the effective overburden pressure, u_o is the hydrostatic water pressure, N_c is the bearing capacity factor or the cone factor, α is the area correction factor for the pore pressure acting on the back of the cone and β_q represents the excess pore pressure at the shoulder of the cone (expressed as a ratio of the net bearing pressure). The various factors that contribute to uncertainty in estimating undrained shear strength from cone data can produce errors in shear strength ranging from +35% to –25% (Watson and Randolph 1998). This led to the pursuit of alternate shapes of penetrometers.

Stewart and Randolph (1991) devised a T-bar penetrometer, which overcomes the limitations of CPT. No corrections for overburden and pore pressure are required, since ambient pore pressure and overburden stress act on both the top and bottom surfaces of the bar and are self-equilibrating (Randolph et al. 1998). The net bearing resistance of the T-bar (q_{tnet}) is just the bearing load measured using the load cell divided by the projected area of the bar. The undrained shear strength is obtained by dividing q_{tnet} by the T-bar factor N_t. The factor is reported to vary over a narrow range of 9.1 (smooth bar) and 11.9 (rough bar) and a value of 10.5 has been suggested by Stewart and Randolph (1994) for all types of soils. Owing to its simplicity, T-bar penetrometer was used in the present study to obtain the strength profile. A simple way to obtain the T-bar factor is also suggested.

Theoretical Considerations

The direct way to prepare a reduced scale normally consolidated clay bed is through self-weight consolidation of clay slurry in the centrifuge (Kimura et al. 1984, Robinson et al. 2003). Consider the preparation of a normally consolidated model soil specimen of final thickness L_m, representing a prototype soil of thickness L_p, under a gravity of Ng in a centrifuge, where $N=(L_p/L_m)$ and g is the acceleration due to gravity. This allows homologous points in the model, both in the soil and the pore water, to be subjected to the same stress levels as the prototype, and thereby enable prototype soil behavior to be simulated within the reduced scale model (Tan and Scott 1985).

If the acceleration in the centrifuge is Ng and is assumed to be constant with depth (z), then under steady state conditions, the change of effective stress ($d\sigma_v$') over a small depth (dz) can be expressed as (Robinson *et al*. 2003),

$$d\sigma_v' = \frac{(G_s - 1)}{(1+e)} N\gamma_w dz \tag{2}$$

where, G_s, e and γ_w are the specific gravity of the solid particles, void ratio and unit weight of water, respectively. Eqn. (2) can be integrated by substituting the linear relation between e and $ln(\sigma_v')$ as,

$$e = e_o - \lambda \ln(\sigma_v') \tag{3}$$

where, e_o is the void ratio at σ_v'=1.0 kPa. Substituting Eqn. (3) in Eqn. (2) and integrating with the boundary condition σ_v'=0 at z=0, yields,

$$\sigma_v'\left(1 + e_0 - \lambda \ln \sigma_v' + \lambda\right) = N(G_s - 1)\gamma_w z \tag{4}$$

The effective stress at the bottom of the sample in the centrifuge can be calculated as,

$$\sigma_b' = NL_s(G_s - 1)\gamma_w \tag{5}$$

where, (L_s) is the height of solids in the model sample. At the bottom of the sample, the value of σ_b' calculated using Eqn. (5) should be equal to σ_v' in Eqn (4), at steady state. The final thickness of the clay, and hence the ultimate settlement expected in the centrifuge, can be calculated by solving Eqns. (4) and (5) for z. The effective stress variation with depth at steady state in the centrifuge, at an acceleration of Ng, is schematically shown in Figure 1(a). The purpose of hydraulic consolidation is to induce an effective stress variation in the soil model as close as possible to that shown in Figure 1a. Once this is achieved, the sample will be subjected to further self-weight consolidation in the centrifuge to achieve the stress profile in Figure 1a that is, scaling wise, consistent with that expected in the prototype due to self-weight of the soil.

When water is made to flow through a soil due to the difference between the hydraulic heads on the top and bottom of the clay bed, the downward seepage force exerted by the flowing water consolidates every element of the soil. The seepage force is applied to the soil skeleton, by the moving water, through viscous drag. The pressure drop related to the loss in total head is transferred from pore pressure to effective stress (Lambe and Whitman 1969). Conventionally, the seepage force is induced on the soil by applying high pressure to the top of the clay bed, thus generating a hydraulic head difference of h as shown in Figure 1b. The soil mass is of uniform cross sectional area and the positive water heads at the top and bottom of the model are h_u and h_2, respectively. The sample consolidates under the head difference (h = h_1 – h_2) to a final thickness of L_m'. At the end of seepage consolidation, under steady flow, the variation of total stress and pore water pressure with depth are shown in Figures 1c and d, respectively. The effective stress at the bottom of the sample is

$$\sigma_b' = L_s(G_s - 1)\gamma_w + \gamma_w h \tag{6}$$

The effective stress variation with depth is shown in Figure 1e.

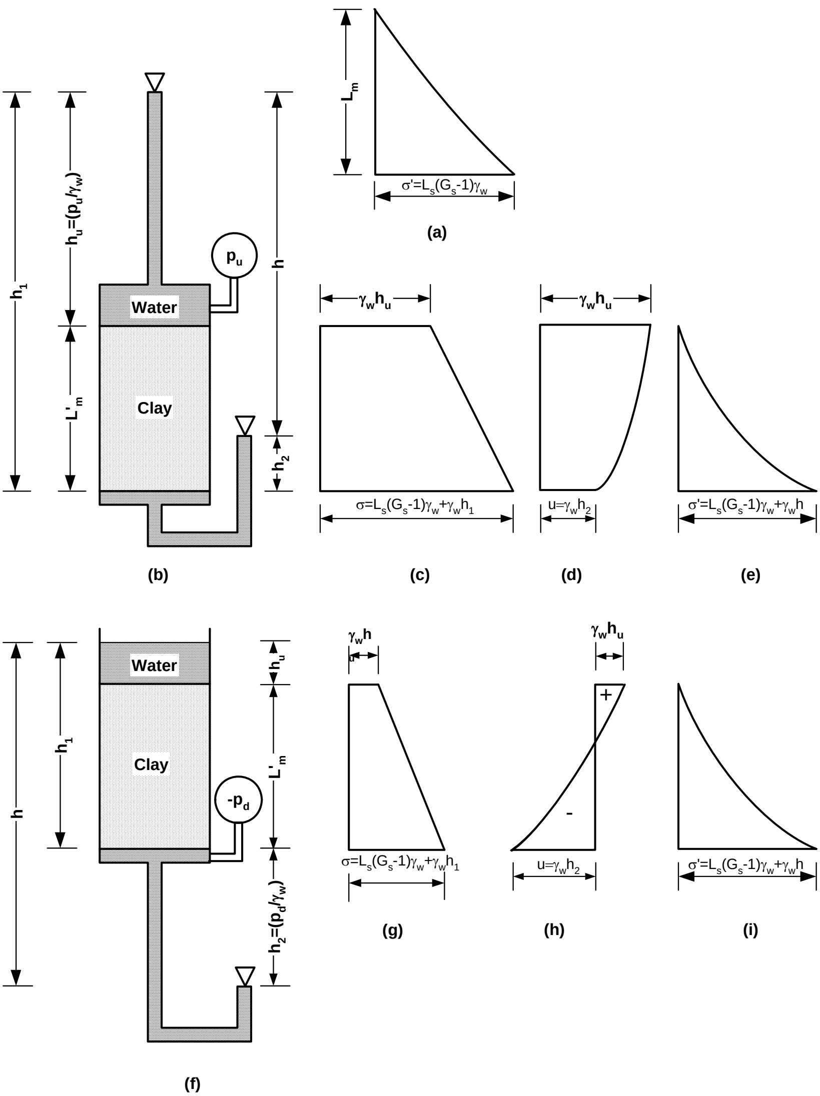


Fig. 1(a) Effective Stress Distribution Expected in the Centrifuge;
(b) Set-Up for the Conventional Method of Seepage Consolidation;
(c), (d) and (e) Variation of Total Stress, Pore Water Pressure and the Effective Stress, Respectively, for the Conventional Method;
(f) Set-Up for the Present Method of Seepage Consolidation;
(g), (h) and (i) Variation of Total Stress, Pore Water Pressure and the Effective Stress, Respectively, Under Suction Induced Hydraulic Consolidation.

For samples prepared using the centrifuge and by the hydraulic consolidation to have the same effective stress at the bottom, Eqns. (5) and (6) must be equal. Equating Eqns. (5) and (6), the hydraulic head (h) difference required to create nearly the same effective stress gradient as the centrifuge sample is,

$$h = (N-1)(G_s - 1)L_s \tag{7}$$

In order to overcome the difficulties involved with this method, another method, schematically shown in Figure 1f, was developed (Robinson et al. 2003). It may be noted that for the method shown in Figure 1b, a very high water pressure p_u (or a hydraulic head $h_u=(p_u/\gamma_w)$) acts on the upstream (top) and a small water pressure acts on the downstream (bottom). Both the pressures acting on the top and bottom are positive. A high value of p_u is required to create the required head difference h. Instead of a large positive head on the top of the clay bed, a large negative head at the bottom and a small positive head on the top of the clay bed is applied thereby resulting in the same head difference (Figure 1f). The variation of total pressure, pore water pressure and the effective stress are shown in Figures 1g, h and i, respectively. The negative water head at the bottom of the sample is created by applying negative water pressure $-p_{d.}$ Guided by the above considerations, experiments were conducted to examine the validity and the effectiveness of the proposed method of preparing normally consolidated clay samples, by seepage consolidation, for testing in the centrifuge, through strength assessment by T-bar penetrometer.

Experimental Program

The experimental set-up requires a regulated vacuum supply, a vacuum chamber and the centrifuge strong box. The vacuum chamber acts as an interface to transfer the negative pressure to the water and is connected to the bottom of the centrifuge strong box, as shown in Figure 2.

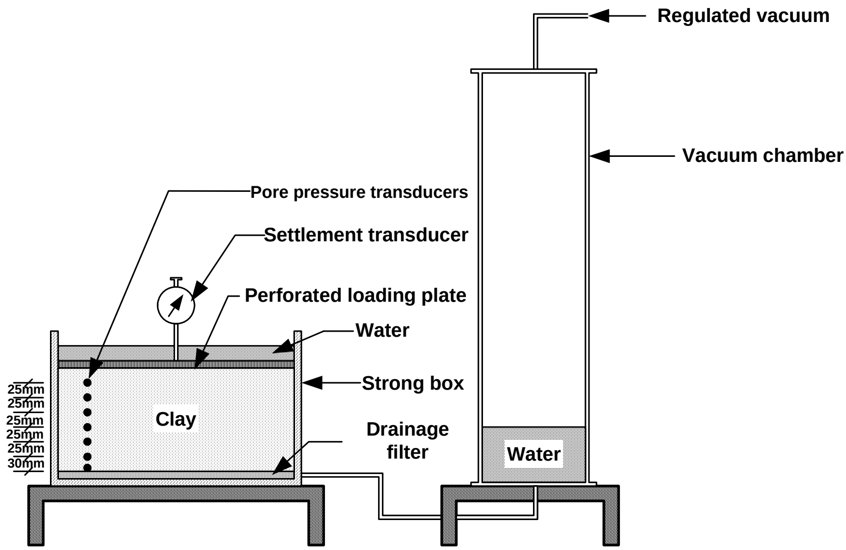


Fig. 2 Schematic of the Set-Up for Suction Induced Seepage Consolidation

The length, breadth and height of the strong box used are 550mm, 205mm and 520 mm, respectively. A perforated flexible plastic pipe of 8 mm diameter was placed at the bottom of the container in order to distribute the negative pressure uniformly throughout the bottom area of the sample. The perforated pipe was covered with a drainage sand layer of 25 mm thickness. A geo-textile filter was then placed over the sand layer in order to separate the clay from the sand. The inner sides of the strong box were lubricated with high vacuum silicone grease so as to reduce the side friction between the soil and the wall.

The Singapore marine clay was used for the seepage consolidation study. The liquid limit, plastic limit and specific gravity of the clay are 83%, 32% and 2.65, respectively. Standard oedometer consolidation test, conducted as per BS: 1377 (1990) on the reconstituted clay having an initial water content of 1.5 times the liquid limit yielded the following void ratio (*e*)-effective consolidation pressure (σ_v', in kPa) relationship:

$$e = 2.76 - 0.69\log_{10}\sigma_v' \tag{8}$$

Falling head permeability test resulted a void ratio (*e*)-permeability (*k*, in m/s) relationship as:

$$e = 12.52 + 1.14\log_{10} k \tag{9}$$

The soil was mixed with water using a mechanical mixer. The resulting clay slurry had a water content of 123%, which is about 1.5 times the liquid limit of the clay. Water content of 1.5 times the liquid limit water content is needed to obtain normally consolidated soils, which lie along the intrinsic compression line (Burland 1990). The utmost care was taken to expel air from the bottom drainage blankets, around the drainage pipes and within the slurry mass. At the surface of the slurry, another piece of geotextile was then placed, followed by a perforated rigid plate having the same length and breadth as those of the strong box. The settlement of this plate was monitored using a potentiometer. The plate imposes a small surcharge pressure of 0.6 kPa to the sample and its influence is negligible. During the centrifuge tests the plate and geotextile filter were removed. Pore water pressure transducers were installed at locations marked in Figure 2.

In the present study it was aimed to prepare a clay stratum, in the centrifuge at 100 g, by consolidating a clay slurry with an initial thickness of 260 mm. The height of solids is 61 mm. The final thickness obtained using Eqns (5) and (4) is 163.8 mm. Therefore, the ultimate settlement expected in the centrifuge is 96.2 mm. The hydraulic head required to induce an effective stress level equal to that at the bottom of the sample in the centrifuge was calculated using Eqn. (7) as 9.97 m. The top of the sample is open to the atmosphere and a water head $h_{1,}$ measured from the bottom of the sample of 400$\pm$20 mm, is always maintained. The negative water pressure head required at the bottom of the sample is therefore 9.57 m. In other words, the negative pressure to be applied at the base of the sample is 94 kPa.

The negative water head required to induce seepage forces to the sample is established by applying suction at the bottom of the sample.

The suction was increased in steps and the full suction of 94 kPa was achieved in three steps. While the first two increments (0 to -30 and –30 to -50 kPa) were maintained only for 24 hours, the last increment (-50 to -94 kPa) was maintained till the end of consolidation. During the suction application, surface settlement and pore water pressure data were monitored using a data acquisition system.

A total of three tests were conducted in the present study. Sample `*Cent*' is the control sample and was prepared by self-weight consolidation entirely in the centrifuge. The slurry sample was consolidated using the NUS centrifuge (Lee *et al*. 1991) at 100 g. Sample `*Seep*' was prepared by seepage consolidation to examine the effectiveness of this method alone. Sample `*Seep-cent*' was first prepared identically to the procedure for sample *Seep*. After that it was further consolidated in the centrifuge, in order to be directly comparable to sample *Cent*.

To study the effect of soil type on the T-bar factor N_t, another soil kaolinite (LL=80% and PI=45%) is also used for the study. As the permeability of kaolinite is very high the samples were prepared directly in the centrifuge without performing seepage consolidation.

T-bar Penetration Test

The T-bar penetrometer used in the present study was very similar to the one developed by Stewart and Randolph (1991) and is illustrated in Figures 3a and 3b. It comprises of a 5 mm diameter cross bar, 25 mm long, attached at right angles (to form a T) at the end of a vertical shaft of 4 mm diameter. A sensitive load cell is situated immediately behind the bar as shown in Figure 3a. The cylindrical surface of the T-bar is sand blasted to create a relatively rough surface, while the ends of the bar were machined smooth to reduce friction.

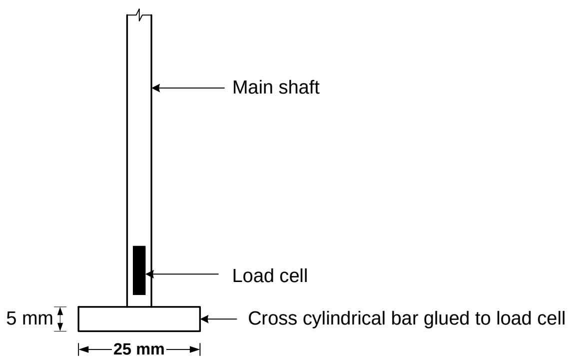


Fig. 3a Schematic of T-Bar Penetrometer

The tests were conducted at a penetration rate of 5 mm/s both during the centrifuge flight and also immediately after spinning down the centrifuge. Immediately after the centrifuge tests, vane shear tests were conducted at various depths of the sample to provide another measurement of the undrained shear strength. Stewart and Randolph (1994) recommended a T-bar factor of 10.5. However, the following procedure is suggested, in the present study, to independently obtain the T-bar factor N_t. Using the vane shear and the T-bar test results, conducted at 1g immediately after spinning down the centrifuge, the T-bar factor N_t can be determined using the following expression.

$$N_t = \frac{q_{tnet}}{c_u} \tag{10}$$

where, q_{tnet} is the bearing pressure under the T-bar and c_u is the vane shear strength, both measured at the same depth at 1g.

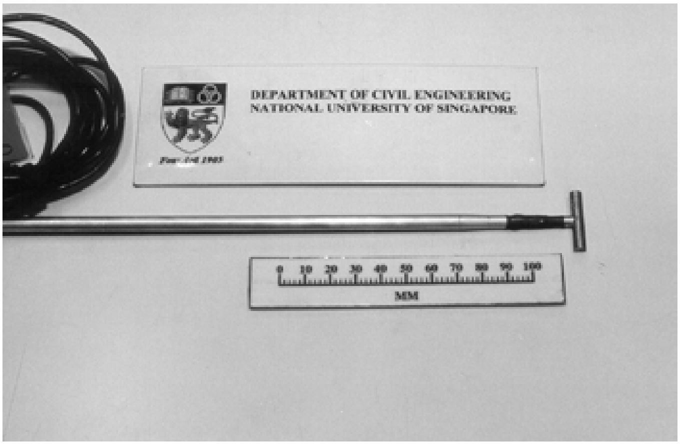


Fig. 3b Photographic View of T-Bar Penetrometer

Results and Discussions

The time-compression curves obtained during the centrifuge consolidation of Samples *Cent* and *Seep-cent* and that for Sample *Seep* during the seepage consolidation are shown in Figure 4. The expected ultimate settlement predicted by the finite strain consolidation theory (Mikasa 1965, Tan and Scott 1988) if the sample is subjected to self-weight consolidation in the centrifuge is 96.3mm. It is seen that sample *Cent* achieved a degree of consolidation (U) of only 87% after being subjected to self-weight consolidation in the centrifuge for 119 hours.

The time taken to consolidate Sample *Seep* by seepage consolidation alone at 1g is 553 hours (23 days). Pore pressure readings indicate that the consolidation is virtually 100% at the end of this time. A time of 23 days in 1g for the seepage consolidation looks quite long but it may be noted that during this time several batches of samples can be prepared simultaneously on the laboratory floor. The ultimate settlement of Sample *Seep* by seepage consolidation alone is 85.3mm. This is only 89% of the ultimate settlement expected at the end of consolidation in the centrifuge. Even though the effective stress induced at the bottom of the sample by seepage forces is the same as that in the centrifuge, the settlement obtained is less than the ultimate settlement expected in the centrifuge. This suggests that the effective stress induced to the sample by seepage consolidation may not be totally equivalent to that expected in the centrifuge over the entire depth of the sample. This is examined from the pore water pressure measurement over the depth of the sample during seepage consolidation.

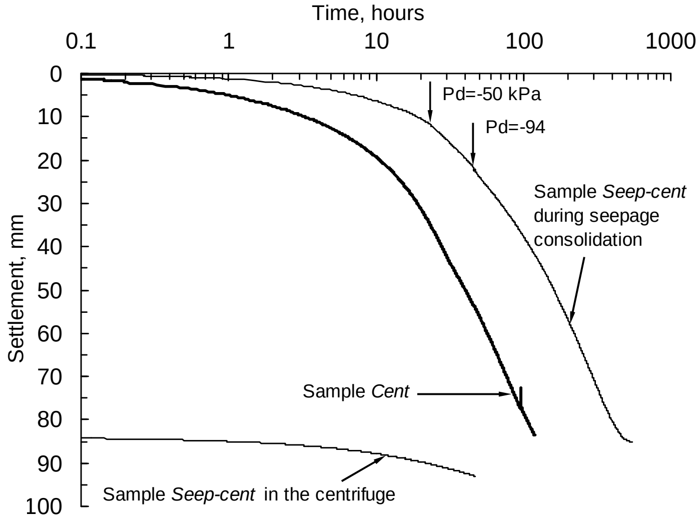


Fig. 4 Time-Settlement Curves Obtained During Self-Weight Consolidation and Seepage Consolidation

The effective stress distribution of Sample *Seep* with depth, calculated from pore pressure measurements, at steady state is shown in Figure 5. The effective stress variation with depth expected in the centrifuge is also depicted in the figure. The plot clearly shows that the effective stress induced by seepage consolidation is not equivalent to that in the centrifuge but is less than that at all locations other than the top and bottom of the sample. The settlement observed during seepage consolidation suggests that the degree of consolidation obtained by seepage consolidation is only 89% of that expected in the centrifuge. The effective stress variation with depth, predicted from the isochrones obtained using the finite strain consolidation theory (Tan and Scott 1988) for degree of consolidation of 89% is plotted in Figure 5. Eqns. (8) and (9) were used in the finite strain consolidation theory. It is interesting to see that the effective stress induced by seepage consolidation lies very close to the effective stress corresponding to a degree of consolidation of 89% by self-weight consolidation. This suggests that seepage consolidation at 1g is equivalent to about 89% of the consolidation in the centrifuge.

Sample *Seep-cent*, which had undergone seepage consolidation first, settled a further 8.9 mm after 48 hours of in-flight consolidation (Figure 4). The time taken after removal of suction to the starting of centrifuge is 3 hours. During this time a swelling of 1.2 mm was recorded. Considering this swelling, the total settlement experienced by Sample *Seep-cent* during seepage consolidation and self-weight consolidation in the centrifuge is 93 mm (85.3+8.9-1.2). This corresponds to a degree of consolidation of 97%, well beyond that usually imposed for centrifuge tests. To achieve 90% consolidation, the additional time required for consolidation in the centrifuge is only 5 hours.

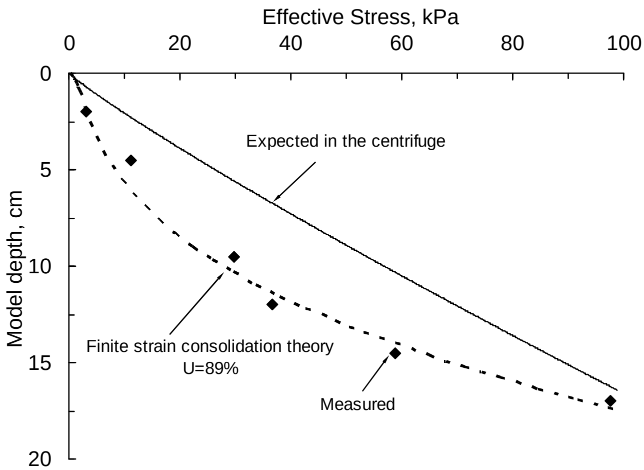


Fig. 5 Comparison of the Effective Stress Induced by Suction Induced Seepage Consolidation with That at U=89% Expected in Self-Weight Consolidation

In practice centrifuge tests are conducted once the clay achieves a degree of consolidation of about 90 to 95%. The time required to achieve a degree of consolidation of 90%, for the Singapore marine clay, exclusively using the centrifuge (sample *Cent*) is as high as 142 hours (Figure 4). However, this degree of consolidation can be achieved by further consolidating the sample, prepared by seepage consolidation, in the centrifuge for only 5 hours, saving a centrifuge time of 137 hours, which is a significant saving both in terms of the demand on centrifuge operation time as well as in the wear and tear due to prolonged non-stop operation.

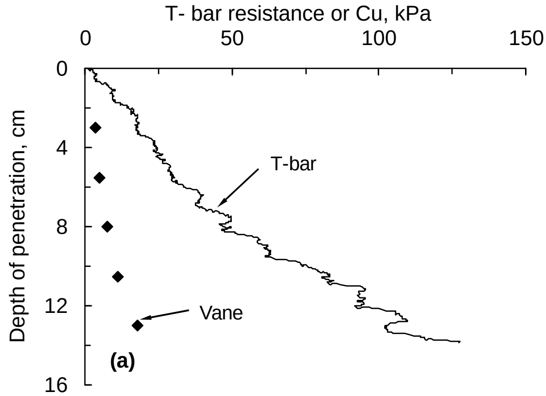
Limitations of the Method

One of the limitations of the method is that a negative pressure of only 100 kPa can be applied to the bottom of the sample. In other words, the method, in its present form, is ideal for preparing clay samples to represent prototype depths of less than about 17 m. In addition, seepage consolidation cannot produce the effective stress distribution equivalent to self-weight consolidation in the centrifuge. However, it can induce an effective stress to achieve a degree of consolidation close to 90%, which saves considerable centrifuge time during the consolidation run. There is a need to develop methods that can induce effective stress profile equivalent to that obtained in the centrifuge.

Evaluation of shear strength

The T-bar resistance (q_{tnet}) and the vane shear strength of sample Seep after seepage consolidation at 1g are shown in Figure 6a. This data is used to obtain the T-bar factor N_t using Eqn. 10. A N_t value of 8.5 fits the data very well.

Using this value the undrained shear strength were obtained from the T-bar resistance of samples Cent and Seep-cent and plotted in Figure 6b.



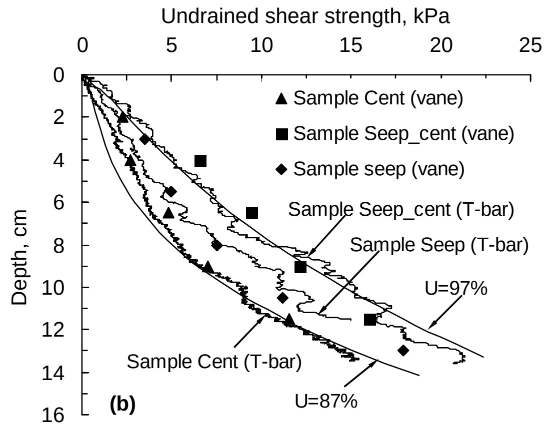


Fig. 6(a) T-Bar Resistance and Vane Shear Strength of Sample *Seep* and (b) Undrained Shear Strength from in-Flight T-Bar Penetration Tests, Vane Shear Tests at 1g and the Predicted Values of Marine Clay

The vane shear strength values obtained immediately after spinning down the centrifuge is also plotted in Figure 6b. The results show that the shear strength values obtained after spinning down the centrifuge (at 1g) is practically the same as that at 100g. Davies and Parry (1982) reported that shear strength values measured posttest tends to be lower than those measured in-flight. However, their results were based on centrifuge tests on kaolin, which typically has a coefficient of permeability about an order of magnitude larger than that of marine clay. Thus when relatively less permeable soils are used, the shear strength measured immediately after spinning down the centrifuge is practically the same as that during flight. It can be also seen from the figure that the undrained shear strength of sample Seep-cent is higher than that of sample Cent. This is expected because the degree of consolidation achieved by sample Seep-cent during seepage consolidation followed by self-weight consolidation is

97%, whereas, the degree of consolidation achieved by sample Cent is only 87%. Also, what is notable is that the shear strength variation in sample Seep, which has not been subjected to centrifuge consolidation, is very close to that in sample Cent which has been subjected to 5 days of spinning in the centrifuge. The effectiveness of seepage consolidation is thus clear.

It is often required to predict the shear strength of samples in the absence of in-flight site investigation tools. Hence, it is attempted here to predict the shear strength of samples prepared in the centrifuge. The undrained shear strength of saturated clays can be estimated from the water content (w) using the following relationship based on the position of the critical state line (Schofield and Wroth 1968):

$$c_u = 0.5M \exp\left(\frac{\Gamma - 1 - wG_s}{\lambda}\right) \tag{11}$$

where, M=slope of the critical state line in the p'-q stress space; p'=mean effective stress; q=deviatoric stress; $\Gamma =$ specific volume of soil at critical state when p' = 1.0 kPa. For the Singapore marine clay used, the values of M, Γ and λ are 1.0, 3.63 and 0.30, respectively. As the samples *Cent* and *Seep-cent* are under consolidated (U<100%), the water content distribution with depth was predicted using the finite strain consolidation theory from the isochrones corresponding to the respective degree of consolidation. The undrained shear strength values obtained using the critical state concept from the water content values predicted using the finite strain consolidation theory, for U=87% and 97% are also shown in Figure 6b. The predicted values matches very well with those obtained from the experiments. Thus the procedure of using the non-linear finite strain consolidation theory coupled with critical state soil model gives reliable strength profiles in the soil model prepared.

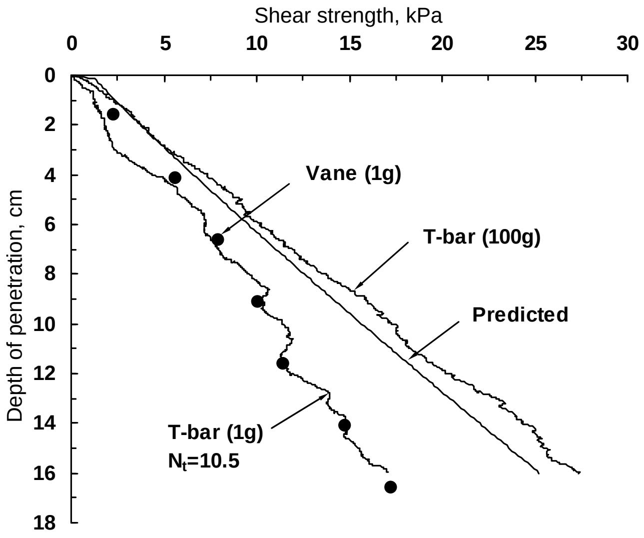


Fig. 7 Undrained Shear Strength of Kaolinite at 100g and Immediately after Swing Down the Centrifuge

The kaolinite sample with a final thickness of 16 cm achieved full consolidation within 8 hours in the centrifuge. The N_t values were obtained from the T-bar test and vane shear test conducted immediately after swing down the centrifuge. An N_t value of 10.5 fits the data very well. The results are shown in Figure 7. The predicted values, by the procedure outlined earlier, for the kaolinite (Γ=3.67, λ=0.244, M=0.95) is also shown in the figure. The predicted values agree reasonably well with the experimental values. Contrary to the marine clay, the shear strength values during in-flight are considerably higher than those measured after swing down the centrifuge. This is because the kaolinite is highly permeable clay and swells very fast, once the stresses are released by reducing the gravity from 100g to 1g. This result further supports the results of Davies and Parry (1982). Thus, when highly permeable soils are used for centrifuge tests, the strength should be measured in-flight.

Comparison of the T-bar factors of marine clay and kaolinite suggest that N_t is not a constant but depends on soil type. The values of N_t can be evaluated by calibrating the T-bar resistance against the vane shear test conducted immediately after swing down the centrifuge.

Conclusions

Normally consolidated clay beds, with the strength increasing with depth, can be prepared by using seepage induced hydraulic consolidation. Suction induced seepage consolidation eliminates the risk of hydraulic fracturing that is reported, when conventional seepage consolidation techniques are adopted. The seepage consolidation is not equivalent to centrifuge consolidation throughout the depth of the clay bed but can produce only 90% of the consolidation in the centrifuge. The T-bar penetrometer is a good tool for obtaining the shear strength of clay beds in the centrifuge. The T-bar factor N_t can be evaluated by calibrating the T-bar resistance against the vane shear test conducted at 1g after swing down the centrifuge.

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