PRE-RECLAMATION IN SITU TESTING OF SOFT SOIL

A. Arulrajah1, M.W. Bo2, H. Nikraz3 and R. Hashim4
1Faculty of Engineering and Industrial Sciences, Swinburne University of Technology, Victoria, Australia
2Faber Maunsell, Bradford, United Kingdom
3Department of Civil Engineering, Curtin University of Technology, Perth, W. Australia
4Department of Civil Engineering, University of Malaya, Malaysia

ABSTRACT

The Changi East Reclamation Project in the Republic of Singapore comprises the ground improvement of marine clay with the installation of prefabricated vertical drains and subsequent preloading. Prior to the commencement of land reclamation works, a series of in situ tests were conducted under marine conditions with the help of various in situ testing equipment. The In Situ Testing Site was located in the northern part of the project where the thickest compressible marine clay existed. The in situ tests carried out were with the field vane, piezocone, flat dilatometer, self-boring pressuremeter and BAT permeameter. In situ tests were conducted to determine the undrained shear strength, overconsolidation ratio, soil stiffness and coefficient of consolidation and permeability of the marine clay. In situ dissipation tests provide a means of evaluating the in situ coefficient of consolidation and hydraulic conductivity due to horizontal flow of soft soil and were used to estimate these properties of Singapore marine clay at Changi.

1 INTRODUCTION

In order to allow for the future expansion of Changi International Airport in Singapore, an additional 2000 hectares of land was reclaimed next to the existing airport. The shear strength, overconsolidation ratio, permeability and consolidation properties of the soil in the horizontal flow direction are important design parameters which need to be determined prior to reclamation. The determination of these design parameters are traditionally based on laboratory consolidation tests with the use of horizontally cut samples. Results of these laboratory tests however are usually subject to uncertainties primarily due to inevitable disturbance of the samples. In situ testing is an alternative to these traditional laboratory testing methods and furthermore the effect of disturbance to marine clays is minimal. In situ testing enables the undrained shear strength and overconsolidation ratio of the marine clay to be determined at various levels. In situ dissipation tests can also be conducted at various levels in the marine clay to estimate the variation of the coefficient of consolidation and hydraulic conductivity due to horizontal flow with depth. The last two decades have seen an emergence of in situ testing methods as an alternative to laboratory testing methods. Accordingly, the objectives of this paper are: 1) to describe the testing and analysis procedure for the various in situ tests; 2) to determine the undrained shear strength and overconsolidation ratio of Singapore marine clay at Changi prior to reclamation; 3) to determine the coefficient of consolidation due to horizontal flow (C<sub>h</sub>) of Singapore marine clay prior to reclamation; 4) to determine the horizontal hydraulic conductivity (k<sub>h</sub>) of Singapore marine clay prior to reclamation; 5) To compare and discuss the results of the various in situ tests.

2 SITE DESCRIPTION

The In Situ Testing Site comprises two distinct layers of marine clay, which are the “Upper Marine Clay layer” and the “Lower Marine Clay layer”. The “Intermediate Stiff Clay layer” separates these two distinct marine clay layers. The upper marine clay is soft with undrained shear strength values ranging from 10 Pa to 30 Pa. Marine or organic matter is found in the upper marine clay. The intermediate layer is a silty clay layer and its formation is believed to have occurred during the lowering of sea level, which was then followed by a rise in sea level and further deposition of the upper marine clay layer. The lower marine clay is lightly overconsolidated with an undrained shear strength varying from 30 Pa to 50 kPa. It is not homogeneous but occasionally interbedded with sandy clay, peaty clay and sand layers. Below the lower marine clay is a stiff sandy clay layer locally known as Old Alluvium. The original seabed level in the site was 3.29 metres below Admiralty Chart Datum (~3.29 mCD).

The In Situ Testing Site was located just adjacent to the proposed future airport runway. Figure 1 indicates the soil profile of the pre-reclamation borehole at the In Situ Testing Site. In situ tests carried out at the In Situ Testing Site were the field vane shear test (FVT), cone penetration test (CPT), dilatometer test (DMT), self-boring pressuremeter test (SBPT) and BAT permeameter test (BAT).
3 FIELD VANE SHEAR TEST (FVT)

The type of field vane instrument used in the In-Situ Testing Site was a Geonor vane (Norwegian Geotechnical Society, 1979). The vane blade dimensions were 65 mm by 130 mm and with a blade thickness of 2 mm. The Vane testing consists of pushing a vane into clay and measuring the maximum torque required to rotate the vane at a given rate of rotation. It follows that the failure surface is cylindrical around the vane (Cadling and Odenstad, 1950). The use of the field vane shear tests as well as the determination of the undrained shear strength has been described by the Norwegian Geotechnical Society (1979). Mayne and Mitchell (1988) have provided an interpretation method of the overconsolidation ratio of clays by using the field vane shear test results. The way in which the test is carried out, including any delay between penetration and vane rotation and time to failure, also influence the results (Flaate, 1966; Aas, 1967).

3.1 FIELD VANE TEST METHOD

The test procedure was carried out in accordance with the method described by the Norwegian Geotechnical Society (1979) and Chandler (1988) for which a waiting time of five minutes after penetration was allowed for the equalization of pore water pressure generated during penetration of the vane blade. Following the advancement of the borehole, the vane was pushed steadily for a distance of about five times the diameter of the borehole to the proposed test level. Following this, a torque was applied at the surface to the vane blade with a rod rotation rate of 12 degrees per minute. This would ensure that the rotation would not introduce significant viscous and drainage effects on the soil. The maximum torque required for mobilization of the vane was recorded.

Field vane shear tests were carried out to determine the undrained shear strength of the marine clay at the In Situ Testing Site. The vane shear strength is found to be increasing with depth as is expected. The interpretation of undrained shear strength, $C_u$, assumes full and uniform mobilization of shear stress over the entire failure surface and is determined from the following relationship:

$$C_u = \frac{6}{\pi} \left( \frac{T}{\pi D^2} \right)$$

where $C_u$ is in units of kN/m$^2$; $T =$ maximum measured torque and $D =$ diameter of field vane.

Mayne and Mitchell (1988) suggested that the overconsolidation ratio, OCR, can be estimated from undrained shear strength and plasticity index. As such, the OCR of the natural and improved soils can be assessed:

$$OCR = 22 \, PI^{0.48} \left( \frac{C_u}{\sigma_{vo}'} \right)$$

where $PI =$ plasticity index and $\sigma_{vo}'$ = effective vertical stress.

Figure 1: Typical soil profile and engineering parameters at the In Situ Testing Site.
4 PIEZOCONE TEST (CPT)

The type of cone used in the piezocone tests (De Beer et al., 1988) was a Gouda cone, capable of registering a cone resistance of up to 50 MPa, sleeve friction of up to 500 kN/m² and a maximum pore pressure of 2000 kN/m². The cone had a 60 degree cone tip, projected cross-section area of 10 cm², friction sleeve area of 150 mm² and an unequal area ratio "a" of 0.8035. The pore pressure filter was located at the base immediately behind the cone tip. The cone was advanced into the soil with a 20 ton Dutch cone rig.

4.1 INTERPRETATION OF GEOTECHNICAL PARAMETERS USING PIEZOCONE TEST DATA

The piezocone has seen a surge in its use in soft clays in this region. Campannella and Robertson (1988) have described the standard guidelines for the use of the piezocone test equipment. Campannella and Robertson (1988) have also provided various interpretation charts to be used in conjunction with the cone penetration test results. Sugawara (1988) has provided a method of estimating in situ overconsolidation ratio of clays by using the piezocone test. The piezocone is economical, easy to carry out and is widely available in the region. The test can be done relatively quickly over the whole soil profile.

The testing procedure was carried out by the recommended international practice (De Beer et al., 1988) with a continuous penetration at a prescribed rate of 20 mm per second. The recorded parameters of the penetration test are cone resistance, q₀, sleeve friction, fₛ, penetration pore pressure, uₚ, and inclination.

From the measured cone resistance reading, the corrected cone resistance reading, qₚ, was calculated using the following equation to account for the unequal bearing area effect:

$$ qₚ = q₀ + (1 - a) uₚ $$  \hspace{1cm} (3)

where q₀ = cone resistance; uₚ = penetration pore pressure and a = unequal bearing area effect of 0.8035.

Campannella and Robertson (1988) has described that the undrained shear strength, Cᵤ, can be calculated as follows:

$$ Cᵤ = \frac{(qₚ - σₒ)}{N KT} $$  \hspace{1cm} (4)

where Cᵤ is in units of kN/m²; qₚ = corrected cone resistance in kN/m²; σₒ = total over-burden pressure in kN/m² and N KT = the cone factor.

The cone factor, N KT for Singapore Marine Clay at Changi can be obtained as follows (Bo et al., 1997a; 1998; 2000; 2001; 2003; 2004):

$$ N KT = 23.8 \times (1/3.8) \ PI $$  \hspace{1cm} (5)

where PI = plasticity index.

Sugawara (1988) proposed that the overconsolidation ratio, OCR, can be estimated from the corrected cone resistance and total effective overburden pressure as follows:

$$ \frac{(qₚ - σₒ)}{σₒ'} = K \cdot OCR $$  \hspace{1cm} (6)

where K is a constant that varies between 2.5 and 5.0. A K value of 3.136 was used for the marine clay (Bo et al., 1997a).

4.2 PIEZOCONE DISSIPATION TEST

The piezocone used in this study had the pore pressure filter located just behind the cone tip. The piezocone dissipation test (CPTU) were carried out at various elevations. Coefficient of consolidation due to horizontal flow was worked out by applying the Baligh and Levadoux (1986) method.

When piezocone is penetrated into soft soil, some excess pore pressure will generate due to penetration. However, if the cone is held in the same elevation for a long time, pore pressures will dissipate until the equilibrium pore pressure is reached. This equilibrium pore pressure will be the same as the pore pressure in the soil at the time of testing.

The first step in the prediction method consists of normalizing dissipation data and plotting the normalized excess pore pressure versus log time. In general, the normalized excess pore pressure decreases monotonically from 1.0 (at t = 0) to 0 (approaching infinity).

$$ \bar{u} = (uₜ - u₀) / (uᵢ - u₀) $$  \hspace{1cm} (7)

where \( \bar{u} \) is the normalized excess pore pressure at time \( t \); \( u₀ \) is the static pore pressure; \( uᵢ \) is the initial or penetration pore pressure (at t=0) and \( uᵢ \) is the pore pressure recorded at time \( t \).
At a given degree of consolidation, the predicted horizontal coefficient of consolidation can be obtained following expression published by Baligh and Levadoux (1986):

\[ C_h (\text{probe}) = \left( \frac{R^2 T_{50}}{t} \right) \]

where \( C_h \) is in units of \( \text{m}^2/\text{yr} \); \( R \) is radius of cone shaft in metres (0.01785 m for the type of cone used); \( T_{50} \) is time factor which is 3.65 for a 60 degree tip at 50% normalised excess pore pressure; \( t \) is time elapsed for degree of consolidation to take place.

For foundation clays consolidated in the normally consolidated range, estimates of the coefficients of consolidation can be obtained from \( C_h (\text{probe}) \) by means of the following expression published by Baligh and Levadoux (1986):

\[ C_h (\text{NC}) = \left( \frac{C_r}{C_c} \right) \left( C_h (\text{probe}) \right) \]

where \( C_h (\text{NC}) \) is in units of \( \text{m}^2/\text{yr} \); \( C_r \) = recompression index and \( C_c \) = compression index.

In order to obtain the hydraulic conductivity in the normally consolidated condition, a correction taking recompression ratio into account needs to be applied. The horizontal hydraulic conductivity, \( k_h \), is given by the following equation:

\[ k_h = \left( \frac{\gamma_w}{2.3 \sigma'_v} \right) (RR) C_h \]

where \( k_h \) is in units of \( \text{m/yr} \); \( \gamma_w \) is unit weight of water in kN/m\(^3\); \( RR \) is recompression ratio and \( \sigma'_v \) is the effective vertical stress of the soil in kPa.

5 FLAT DILATOMETER TEST (DMT)

A Marchetti flat dilatometer (Marchetti and Crapps, 1981) was used for the tests, has a steel membrane on one side of the blade. The dilatometer blade is 96 mm in width and 230 mm in length. The diameter of the membrane is 60 mm. Marchetti (1980) has provided a detailed description of the flat dilatometer and its interpretation methods. The determination of undrained shear strength and overconsolidation ratio from dilatometer tests has been extensively discussed by Marchetti (1980), Chang (1986) and Chang et al. (1997). Chang (1986) has described the methods and interpretation of flat dilatometer dissipation tests. The method of interpretation of coefficient of consolidation due to horizontal flow values from dilatometer holding tests have been described by Marchetti and Totani (1989). The dilatometer requires certain specialised skill and technical knowledge to operate.

5.1 INTERPRETATION OF GEOTECHNICAL PARAMETERS USING FLAT DILATOMETER TEST DATA

The testing procedure followed that described by Marchetti and Crapps (1981). The testing consisted of pushing the flat dilatometer blade gradually into the soil at a prescribed rate of 20 mm per second with the use of a 20 ton Dutch cone rig. The pushing was temporarily stopped at each of the proposed testing levels at which the two pressure readings A and B (corresponding to two prefixed states of expansion of the membrane) were recorded. The first pressure reading (A-reading, \( p_a \)) corresponds to the membrane lift-off pressure while the second pressure reading (B-reading, \( p_b \)) corresponds to the pressure required for the centre of the membrane to deflect by a preset distance of 1 mm into the soil.

From the two pressure readings, three dilatometer indices are obtained being the material index, \( I_D \), horizontal stress index, \( K_D \), and dilatometer modulus, \( E_D \):

\[ I_D = \frac{(p_b - p_a)}{(p_b - u_0)} \]  
\[ K_D = \frac{(p_b - u_0)}{\sigma_{vo} - u_0} \]  
\[ E_D = 34.7 \left( \frac{p_b - p_a}{p_0} \right) \]

where \( p_a \) is the A-reading corresponding to the membrane lift-off pressure in units of bar; \( p_b \) is the B-reading corresponding to the pressure required for the centre of the membrane to deflect by a preset distance of 1 mm into the soil in units of bars.

Marchetti (1980) proposed the following correlation between the undrained shear strength, \( C_u \), with the horizontal stress index, \( K_d \):

\[ C_u = 0.22 \sigma_{vo} (0.5 K_d)^{\alpha} \]

\[ K_d = \frac{(p_0 - U_0)}{\sigma_{vo}} \]
where \( C_0 \) is in units of kN/m\(^2\); \( \sigma_{vo} \) = vertical effective stress; \( K_d \) = material index; \( P_0 \) = the A reading from dilatometer; \( U_0 \) = pre-inserting water pressure and \( \eta \) is a constant depending on the clay type. For Singapore Marine Clay at Changi, \( \eta \) can be taken as 1 for upper marine clay and intermediate clay while \( \eta \) can be taken as 0.7 for lower marine clays (Bo et al., 1997a, 1998a, 2000, 2003, 2004).

Marchetti (1980) proposed the following correlation for the estimation of the overconsolidation ratio, OCR, for soft clay:

\[
\text{OCR} = (0.5 \, K_d)^n
\]

where \( K_d \) = the material index; \( n = 1 \) for upper and lower marine clays and 0.8 for intermediate clays (Bo et al., 1997a, 2000, 2004).

The dissipation test which makes use of the A-reading is called the DMTA dissipation test and can be performed at any depth by the procedure described by Marchetti and Tottani (1989). In this method, the A-reading is taken at different time intervals and plotted against log time. The time corresponding to the point of reverse curvature on the A-decay curve, \( T_{50} \), is used as a basis for the interpretation of the \( C_h \). For the DMTA dissipation test, the following expression was proposed by Marchetti and Tottani (1989):

\[
C_h \times T_{50} = 5 \text{ - 10 cm}^2
\]

where \( C_h \) is in units of cm\(^2\)/min.

For Singapore marine clay, the following expression is valid (Bo et al., 1998a; 2003):

\[
C_h \times T_{50} = 5 \text{ cm}^2
\]

In the dissipation test procedure which makes use of the C-reading, the C-reading is plotted against square root time and the time corresponding to 50\% consolidation, \( T_{50} \), is determined and used in the interpretation of \( C_h \) (Schmertmann, 1988; Gupta (1983) procedure, developed for piezocone dissipation analysis was modified and used in the interpretation of \( C_h \). The dissipation test which makes use of the C-reading is called the DMTC dissipation test and can be performed at any depth. The procedure involves estimating rigidity index, \( E_s/C_u \), and pore pressure at failure, \( A_f \), for the clay and determining the time factor corresponding to 50\% pore pressure dissipation, \( T_{50} \), from the dissipation curves for \( A_f = 0.9 \) (Schmertmann, 1988). An adjustment of the time factor may be required if \( A_f \) is different from 0.9. The \( T_{50} \) can then be used in the following equation which assumes \( R^2 = 600 \text{ mm}^2 \) for a test involving the standard Marchetti dilatometer (Chu et al., 2002), which is based on the radius of cavity expansion:

\[
C_h \times T_{50} = 600 \times \left( T_{50} / t_{so} \right)
\]

where \( C_h \) is in units of mm\(^2\)/min; \( T_{50} \) is the theoretical time factor; \( t \) is time elapsed for 50\% degree of consolidation to take place.

Similar to CPTU tests, the \( C_h \) values determined from either DMTA or DMTC corresponds to the unloading/reloading range. Corrections have to be made to obtain the in situ \( C_h \) value. When converting the \( C_h \) values into the \( C_h \) value at the normally consolidated state, the conversion using Equation (9) has been found to provide consistent results. Equation (10) can be used to determine the horizontal hydraulic conductivity of the marine clay.

6 SELF-BORING PRESSUREMETER TEST (SBPT)

The Cambridge-type self-boring pressuremeter with 6 strain measuring arms located at the mid-level (Cambridge In-Situ, 1993) was used for the testing purposes. The probe is about 83 mm in diameter and 1.4 metres in length and is made up of stainless steel and brass. Over the critical part of the instrument, the diameter is maintained to an accuracy.
of 0.1 mm. The instrument consists of strain gauge type transducers attached to the central core or pressuremeter body. The pressuremeter body was covered with a rubber membrane for direct recording of the radial displacement and the applied pressure. A rotary bit is present at the base of the equipment. The self-boring pressuremeter also requires certain specialised skill and technical knowledge to operate.

6.1 INTERPRETATION OF GEOTECHNICAL PARAMETERS USING SELF-BORING PRESSUREMETER TEST DATA

Mair and Wood (1987) have described the methods of testing of various pressuremeters including the self-boring pressuremeter. Windle and Wroth (1997) has described the determination of the undrained properties of clay by means of the self-boring pressuremeter. Whittle et al. (1993) have described the lift-off stress and analysis of the initial stress distribution of the six arm self-boring pressuremeter.

Testing involves the advancement and insertion of the pressuremeter to the proposed depth by use of the self-boring technique. After the insertion of the pressuremeter, the rubber membrane was inflated by injection of gas pressure. Both the applied pressure and the corresponding displacement of the borehole (cavity) wall were measured during the test. Raw testing results are produced in plots of applied pressure versus radial cavity strain, which is interpreted by the cavity expansion theory.

Windle and Wroth (1997) suggested undrained shear strength, \( C_u \), can be estimated from the limit pressure from the self-boring pressuremeter as follows:

\[
C_u = \frac{(P_c - \sigma_{ho})}{[1 + \log_e (G / Cu)]} \\
C_u = \frac{(P_c - \sigma_{ho})}{N_p} \\
N_p = 1 + \log_e (G / Cu)
\]

where \( C_u \) is in units of \( kN/m^2 \); \( \sigma_{ho} \) = total horizontal stress; \( G \) = shear modulus and \( N_p \) is the pressuremeter constant defined by Marsland and Randolph (1977). For Singapore marine clays, \( N_p \) values of 6.6, 6.4 and 7.2 can be applied for the upper marine clay, intermediate clay and lower marine clay respectively (Bo et al. 1997a; 1998a; 2000 and 2003). Estimation of shear modulus can be obtained from small unload-reload cycles. The undrained shear strength is obtained from the expansion tests.

The OCR for the pre-reclamation SBPT was calculated from the SBPT shear strength values by using Equation (2) which is also used for the FVT.

6.2 SELF-BORING PRESSUREMETER DISSIPATION TEST

The pore pressure cells are located 43 mm below the centre of the pressuremeter probe. The holding test proceeds as a normal pressuremeter test until the point when the soil is to be unloaded. Instead of unloading at a constant rate of strain, the expanded cavity is held fixed at the current dimensions. The excess pore water pressure generated by the preceding expansion will begin to drain and the decay of pore pressure is recorded. When the level of excess pore pressure has fallen by slightly more than half, the test is terminated.

When the pore water pressures fall, the total pressure in the instrument will be greater than that required to maintain the cavity at a fixed size. Left alone, the cavity would continue to expand. An automatic strain control unit is used to monitor this tendency for the cavity to increase, and the unit vents a little of the pressure in the instrument to compensate. Hence, information about the decay of pore pressures is available directly from the pore water pressure transducers on the outside of the instrument and indirectly from the necessary decline in total pressure.

The analysis used was proposed by Clarke et al. (1979). The analysis assumes that the Gibson and Anderson model of soil deformation applies (Clarke et al., 1979) and hence that the pore water pressures generated by an undrained expansion can be calculated and converted to a time factor. Coefficient of consolidation due to horizontal drainage can thus be worked out as follows:

\[
C_{oh}(\text{probe}) = \frac{T_{50} \gamma_b^2 \gamma_m}{k_h} \\
\gamma_b = \text{radius of cavity} \ (m); \\
T_{50} = \text{theoretical consolidation time} \ (yr); \\
k_h = \frac{(C_{oh} / G) \gamma_m \{(1 - 2 \mu) \sqrt{2 - (1 - \mu)}\}}{1 - \mu}
\]

where \( k_h \) is in units of \( m/yr \); \( G \) is shear modulus in MPa; \( \mu \) is poisson ratio which was assumed to be 0.5 for the current test and \( \gamma_m \) is the unit weight of water.

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The BAT permeameter developed by Torstensson (1983) was used in this study for the in situ testing of horizontal hydraulic conductivity. This involves the functions of sampling of ground water and at the same time the measurement of pore water pressure in the sample container. Diameter of the BAT filter used is 30 mm and the length is 40 mm. The key element in the BAT system is the filter tip. The different test adapters make a tight temporary connection to the filter tip with the aid of a hypodermic needle. When the test adapter is lowered down the extension pipe, it is coupled to the nozzle in the filter tip and gravity draws the hypodermic needle downward, penetrating the rubber disc mounted in the filter tip. The needle provides a hydraulic connection between the interior of the filter tip and the test adapter.

The BAT permeameter results are used as the baseline data for horizontal hydraulic conductivity since this system of measurement is more direct compared to other in situ testing methods in which the measurement method is indirectly evaluated from $C_u$ values.

The test can be carried out either as an "inflow test" or as an "outflow test". In the former case the gas/water container is completely gas-filled at the start of the test. An inflow test can be conducted simultaneously with extraction of pore water sample. In an outflow test, the container is partially filled with compressed gas. The air in the chamber is evacuated (or pressurized) to any desired pressure. As water flows into (or out of) the probe, the air pressure in the chamber changes. A pressure transducer monitors the pressure change.

The test is based on measurement of flow into and out of a sample container. This rate is computed by measuring the change in the gas pressure in the chamber. Analysis of the time-pressure record thus yields the horizontal hydraulic conductivity. The quantity of flow and heads are computed from Boyle's Law and the measured change in the gas pressure in the chamber.

\[
\begin{align*}
  k_h &= \frac{P_0 V_0}{F t} \left[ \frac{1}{P_0 U_0} - \frac{1}{P_1 U_0} - \frac{1}{U_0^2} \ln \left( \frac{P_0 - U_0}{P_0} \times \frac{P_1}{P_1 - U_0} \right) \right] \\
  F &= \frac{2 n L}{\ln \left[ \frac{1}{d_d} + \sqrt{1 + \left( \frac{1}{d_d} \right)^2} \right]}
\end{align*}
\]

where $k_h$ is the horizontal hydraulic conductivity in units of m/s; $P_0$ is absolute initial system pressure; $V_0$ is initial gas volume; $F$ is shape factor and is calculated as 228.76 mm for the current test; $U_0$ is static pore water pressure; $P_1$ is absolute pressure at time $t$; $L$ is length of filter and $d$ is diameter of filter.

8 RESULTS

8.1 UNDRAINED SHEAR STRENGTH

The pre-reclamation in situ test results by the various methods are in close agreement with each other. Figure 2 shows a comparison of the shear strengths from the various in situ tests. Figure 3 compares the OCR obtained from the various in situ tests prior to reclamation. In the shear strength and OCR comparisons, the various tests indicate similar increasing trend profiles for increasing depths. There is a clear distinction of higher shear strength and OCR values obtained by the various tests in the intermediate marine clay layer.

The values of undrained shear strength of the Singapore marine clay obtained by the various methods are in good agreement with each other. The undrained shear strength obtained from the various test methods was analysed to obtain empirical correlation of the undrained shear strength ($C_u$) of the marine clay at the In Situ Test Site. The empirical correlation of shear strength increase with depth obtained from the in situ tests at the In Situ Test Site is as follows:

\[
C_u = 7.06 + 1.7 (\text{Depth below seabed})
\]

where $C_u$ is in units of kN/m$^2$.

The upper marine clay is generally overconsolidated with an OCR of about 1.5 to 3. The lower marine clay is lightly overconsolidated with OCR of 1 to 2. The intermediate stiff clay is moderately overconsolidated due to desiccation, with OCR of 1.5 to 3. The desiccated layer found close to the seabed is also found to register high OCR values. Higher OCR at seabed normally occurs due to hydrodynamic effect caused by wave and current action. It is apparent that the OCR from CPT is the lowest of the in situ testing. This is possibly due to the value of the constant, $K$ used in equation for the OCR computations by the CPT.
8.2 COEFFICIENT OF CONSOLIDATION DUE TO HORIZONTAL FLOW
The pre-reclamation coefficient of consolidation for horizontal flow \((C_v)\) as obtained from various in situ dissipation tests vary between 2 m\(^2\)/yr and 26 m\(^2\)/yr as shown in Figure 4. The pre-reclamation CPTU dissipation tests indicate that
the Ch values of the upper and lower marine clay varies between 2 m$^2$/yr and 6 m$^2$/yr. Ch values of 4 m$^2$/yr to 7 m$^2$/yr were obtained in the intermediate stiff clay, separating the upper and lower marine clay layers. The CPTU results are found to be the closest to the laboratory testing results.

Among the in situ tests the Ch values in the marine clay layers from SBPT are the highest overall, while those from the CPTU dissipation test indicate the least variations with depth. The DMT results are reasonable in the lower marine clay layer. It is observed that all the methods indicate large Ch values in the intermediate stiff clay layer.

The Ch determined by the various in situ testing methods are relatively higher overall as compared to the laboratory testing results, as evident in the prior to reclamation test results. Horizontal laminations and micro lenses present in the marine clay profile will lead to higher Ch values and subsequently higher k_h for the in situ tests. The presence of laminations and lenses is difficult to detect by laboratory tests due to the sampling intervals and the sampling process. Furthermore, laboratory results are subject to various complexities such as borehole quality, sample quality, testing methods and method of interpretation which could lead to lower test values.

It is apparent that Ch varied between the various in situ testing methods due to the differing assumption in cavity radius in the various test methods. The varying Ch values will subsequently lead to differing k_h in the CPTU, DMT and SBPT results as k_h computations are worked out indirectly from Ch values.

The Ch value derived from the CPTU dissipation test is generally lower than those obtained from the other in situ dissipation tests. The Ch value obtained from the DMT dissipation tests is usually smaller than that from the SBPT holding test. The Ch value obtained from the SBPT exhibits a larger variation in comparison with that of other tests. In general, the Ch value measured by the SBPT is much larger than those obtained from the other in situ dissipation tests. The SBPT does not appear to be desirable for the measurement of Ch for soft marine clay at Changi, as the Ch values obtained from SBPT are normally too high to be directly used for the design. The Ch determined by the DMT and SBPT is noted to be an order of magnitude greater than the laboratory data.

The smear effect affects the CPTU and DMT measurements for Ch. In the CPTU and DMT dissipation test, a penetrometer has to be pushed into the clay and a smear effect similar to the insertion of a mandrel could have been introduced prior to the measurements. This could lead to the CPTU and DMT measurements for Ch being lower than that of the SBPT.

![Graph showing pre-reclamation coefficient of consolidation](image)

Figure 4: Prior to reclamation coefficient of consolidation due to horizontal flow from various in situ dissipation tests.

### 8.3 Horizontal Hydraulic Conductivity

The pre-reclamation horizontal hydraulic conductivity (k_h) as obtained from the various in situ dissipation tests are shown in Figure 5. Based on the results obtained, the BAT was found to give the lowest values whereas the dilatometer
and CPTU gave the highest values. In situ dissipation tests using the BAT is recommended as the most suitable method for the determination of the \( k_h \) of marine clay. The horizontal hydraulic conductivity of Singapore marine clay prior to reclamation is in the order of \( 10^{-13} \) to \( 10^{-10} \) m/s based on the BAT readings and is close to that of the laboratory testing results.

The smear effect also affects the BAT, CPTU and DMT measurements for \( k_h \). In the BAT, CPTU and DMT dissipation test, a penetrometer has to be pushed into the clay and a smear effect similar to the insertion of a mandrel could have been introduced prior to the measurements. The smear effect for BAT permeameter could be greater than that for the CPTU, as the BAT permeameter had a filter with a larger surface area. This may explain why \( k_h \) measured by the BAT permeameter is normally lower than that by the CPTU, although the working mechanisms of the two tests are very similar. The SBPT should not be affected by the smear effect due to its self-boring mechanism.

![Graph](image_url)

Figure 5: Prior to reclamation horizontal hydraulic conductivity from various in situ dissipation tests.

9 CONCLUSIONS

In the shear strength and OCR comparisons, the various tests indicate similar increasing trend profiles for increasing depths. The undrained shear strength of the Singapore marine clay calculated by the various methods is reasonably similar. The undrained shear strength obtained from the various in situ test methods was analysed to obtain an empirical correlation of the undrained shear strength of the marine clay at the In Situ Testing Site.

The upper marine clay is generally overconsolidated with OCR of about 1.5 to 3. The lower marine clay is lightly overconsolidated with OCR of 1 to 2. The intermediate stiff clay is overconsolidated due to desiccation with an OCR of 1.5 to 3. The desiccated layer found close to the seabed is also found to register high OCR values. The higher OCR at seabed normally occurs due to a hydrodynamic effect caused by the wave and current action. It is apparent that the OCR from CPT is the lowest of the in situ tests. This is possibly due to the value of the constant used in the OCR computations by the CPT.

In situ dissipation tests using the CPTU are recommended as the most suitable method for the determination of the \( C_h \) of marine clay. The CPTU results are found to be the closest to the laboratory testing results. The pre-reclamation CPTU dissipation test indicate that the \( C_h \) values of the upper and lower marine clay varies between 2 m\(^2\)/yr and 6 m\(^2\)/yr. \( C_h \) values of 4 m\(^2\)/yr to 7 m\(^2\)/yr were obtained in the intermediate stiff clay separating the upper and lower marine clay layers.

The \( C_h \) determined by the various in situ testing methods are relatively higher overall as compared to the laboratory testing results, as evident in the prior to reclamation test results. Horizontal laminations and micro lenses present in the marine clay profile, will lead to higher \( C_h \) values and subsequently higher \( k_h \) for the in situ tests. The presence of
laminations and lenses is difficult to detect by laboratory tests due to the sampling intervals and the sampling process. Furthermore, laboratory results are subject to various complexities such as borehole quality, sample quality, testing methods and method of interpretation which could lead to lower test values.

It is apparent that \( C_h \) varied between the various \textit{in situ} testing methods due to the differing assumption in cavity radius in the various test methods. The varying \( C_h \) values will subsequently lead to differing \( k_h \) in the CPTU, DMT and SBPT results as \( k_h \) computations are worked out indirectly from \( C_h \) values. The \( C_h \) value derived from the CPTU dissipation test is generally lower than those obtained from the other \textit{in situ} dissipation tests. In general, the \( C_h \) value measured by the SBPT is much larger than those obtained from the other \textit{in situ} dissipation tests. The \( C_h \) determined by the DMT and SBPT is noted to be an order of magnitude greater than the laboratory data.

The smear effect affects the CPTU and DMT measurements for \( C_h \). In the CPTU and DMT dissipation test, a penetrometer has to be pushed into the clay and a smear effect similar to the insertion of a mandrel could have been introduced prior to the measurements. This could lead to the CPTU and DMT measurements for \( C_h \) being lower than that of the SBPT. The smear effect also affects the BAT, CPTU and DMT measurements for \( k_h \). In the BAT, CPTU and DMT dissipation test, a penetrometer has to be pushed into the clay and a smear effect similar to the insertion of a mandrel could have been introduced prior to the measurements. The smear effect for BAT permeameter could be greater than that for the CPTU, as the BAT permeameter had a filter with a larger surface area. This may explain why \( k_h \) measured by the BAT permeameter is normally lower than that by the CPTU, although the working mechanisms of the two tests are very similar. The SBPT should not be affected by the smear effect due to its self-boring mechanism.

\textit{In situ} dissipation tests using the BAT are recommended as the most suitable method for the determination of the \( k_h \) of marine clay, since the system measures horizontal hydraulic conductivity directly whereas the other \textit{in situ} tests require the introduction of additional parameters to evaluate the \( k_h \) indirectly from \( C_h \) values. The horizontal hydraulic conductivity of Singapore marine clay prior to reclamation is in the order of \( 10^{-9} \) to \( 10^{-10} \) m/s based on the BAT readings and is close to that of the laboratory testing results.

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