Evaluation of Field and Laboratory Test Results

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ABSTRACT: Data presented in soil investigation factual reports is generally being regarded as “factual” and is often accepted and used without or with little scrutiny. Due to inevitable uncertainties, including heterogeneous ground condition, difficulties in sampling and testing, sample disturbance etc., field and laboratory test data require engineering interpretation. This paper examines the interpretation behind the derivation of the result of some commonly performed field and laboratory tests. It highlights aspects to pay attention to when reviewing a factual report and present approaches that can be adopted to assess the reliability of test results. The paper concludes that the face-value of test results should not be assumed, a priori, as facts. Instead test data should be interpreted and used after a data reduction process.

1 INTRODUCTION

Site investigations (SI) are important in obtaining data for subsoil to derive values for geotechnical engineering properties. It is not uncommon in practice to indiscriminately use the field tests and laboratory tests results in factual reports to derive the characteristic values for engineering properties. This approach sometimes results in a large scatter in the data or arrives at statistical values that deviate from “past experience”. There is generally insufficient appreciation that all field and laboratory test results are interpreted values and should therefore be reviewed by the design engineer prior to their use.

This paper attempts to highlight some important issues in the data reduction and interpretation process. The laboratory tests covered include consolidation, unconsolidated undrained (UU), consolidated undrained (CU) and consolidated drained (CD) test. The field tests include standard penetration test (SPT), vane shear test (VST), cone penetration test (CPT) and pressuremeter test (PMT).

Broadly, there are two levels of reviews in the data interpretation process. The first level reviews the results of individual tests. The second level reviews all data of the same type of test in totality. The data reduction aims to remove erroneous and unreliable data from the database and thus improves the reliability of subsequent statistical approaches to derive the characteristic values.

2 LABORATORY TESTS

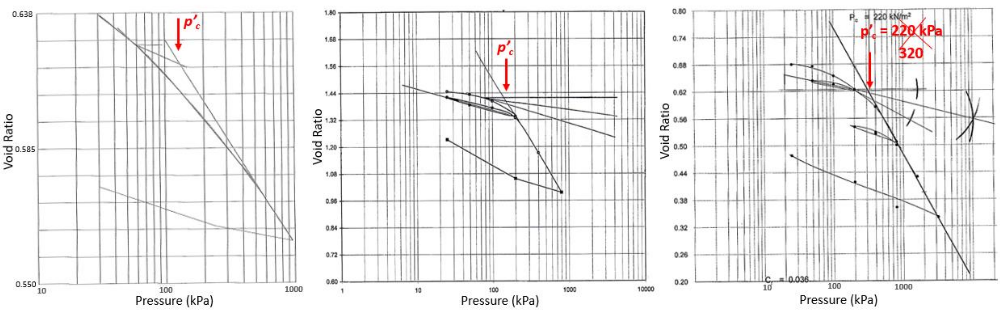
2.1 Consolidation Test

Consolidation test is often conducted on fine-grained soils of the Kallang Formation, viz, marine clay, estuarine and fluvial clay. The properties determined from a consolidation test are the preconsolidation pressure (p'_c), the compression index (C_c), the recompression index (C_r), the coefficient of consolidation during virgin compression (c_v) and during recompression (c_{vr}) and secondary compression (C_α). The last item is not covered in this paper.

In consolidation test, results can be adversely affected if the maximum pressure and the number of load increments are insufficient to load the soil well into the virgin compression range. Often in practice, the number of load increments is limited to six, which appears to follow the suggestion in BS 1377-5:1990 and ISO/TS 17892-5 that “typically four to six load increments are sufficient”. Actually, there are other suggestions in these documents that are sometimes overlooked such as “a sequence of ten load increments from 6 kPa to 3200 kPa” and “loading into the normal consolidation region for over-consolidation soils”. It is a good practice to inform the laboratory on the maximum pressure and the corresponding load increments to be carried out on each test depending on soil type, sample depth, state of consolidation and engineering application.

2.1.1 Preconsolidation Pressure p'_c

The Casagrande’s construction method (1936) is commonly used in practice to determine p'_c. This method pivots on identifying the point of maximum curvature on the plot of void ratio (or compression) versus logarithmic pressure (e-log p′). The choice of this point can be subjective. A poor choice will produce an erroneous p'_c value as illustrated in Figure 1a and 1b. The authors also encountered cases where the p'_c values were incorrectly read from the graph (Figure 1c). These mistakes exemplify why the e-log p’ curve of every test should be reviewed.



(a) No p'_c for this e-log p’ curve (b) Poor choice of maximum curvature location (c) Wrong pressure reading from log scale

Fig. 1 Examples of questionable p'_c values from factual report.

For soft clays, a telltale sign of the quality of the test lies in the shape of c_v curve, i.e. the variation of c_v value with pressure. Figure 2a shows the results of a good quality test where the c_v curve has a "Z" shape. The p'_c is located approximately mid-way along the transition line which has the shape of a back-slash "\". In Figure 2b, the c_v curve does not exhibit a "Z" shape. This could be due to sample disturbance or mis-interpretation of c_v from the settlement-square root time plot as discussed later. In such cases, the p'_c value may be questionable.

Verified values of p'_c values from individual e-log p' curves should be plotted together with depth and checked against the effective overburden (p'_o) line as illustrated in Figure 3a. In this plot, most of the data points fall along the p'_o line which indicates that the soil is normally consolidated. Data points A and B are outliners and should be verified and omitted if they are erroneous or unreliable.

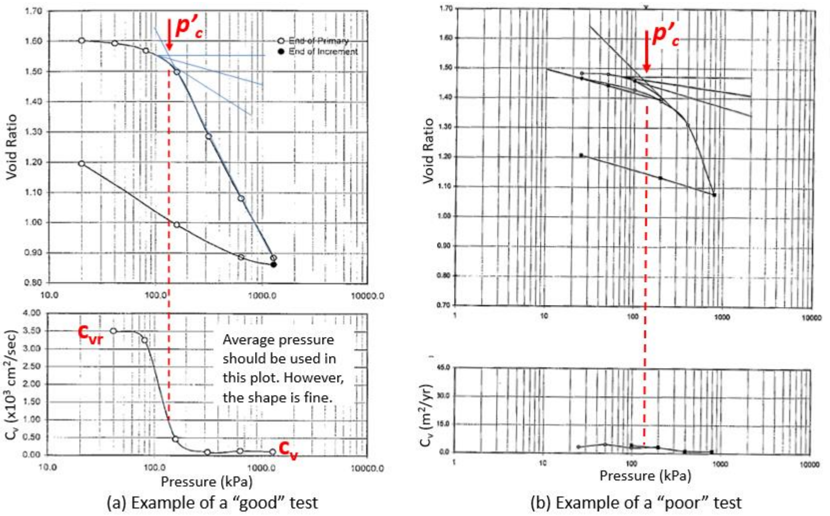


Fig. 2 Typical examples of results of consolidation test on soft clay.

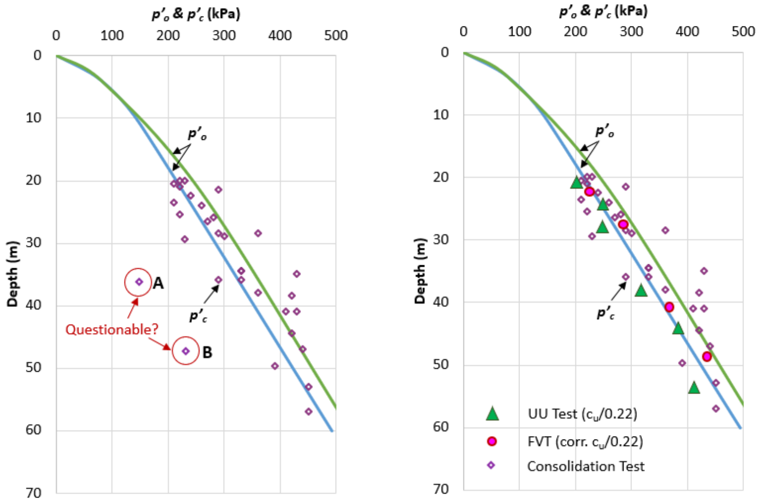
Another check for the reliability of the p'_c value is to compare the values with results from other types of test. Figure 3b shows a plot of the p'_o line, the p'_c and the current effective stress computed from $c_u/0.22$ (from UU) and from VST. If most of the data points fall along the p'_o line, it indicates that the soil is normally consolidated and the p_c' values are consistent with the results from UU and VST.

In certain cases, unreliable p'_c values can grossly affect the assessment on the state of consolidation of the clay. In Figure 4a, the test results that are affected by sample disturbance and poor data interpretation imply that the clay was still undergoing consolidation. Results from well-conducted tests for the same clay shown in Figure 4b indicate that the clay was actually normally consolidated.

2.1.2 Compression Index C_c

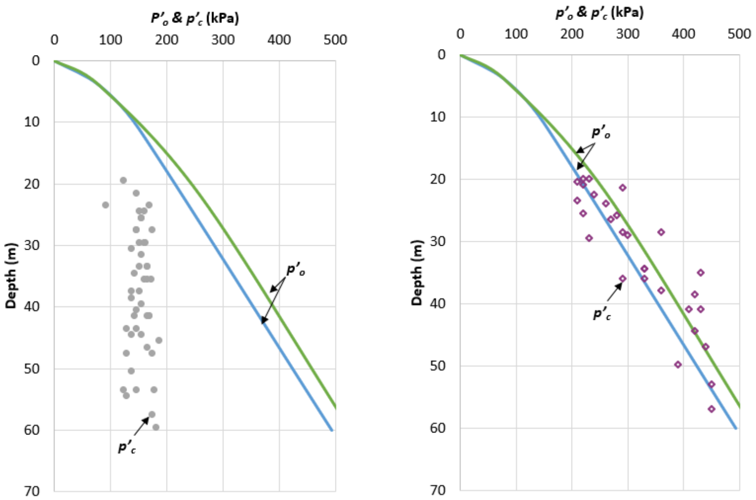
The C_c value is determined from the virgin compression part of the e-log p′ curve. In some soft clays, the C_c value changes with pressures. Sometimes an average C_c value is reported as illustrated in Figure 5a. Some reports present two or more C_c values that correspond to different ranges of pressure as shown in Figure 5b.

Depending on the engineering application, the engineer must decide on the appropriate value to be used.



(a) Checking of p'$_c$ values against p'o line (b) Check of p'$_c$ values against other tests

Fig. 3 Checking of p'$_c$ values against (a) p'$_o$; and (b) other types of test.



(a) Measured p'$_c$ from Company A (b) Measured p'$_c$ from Company B

Fig. 4 Results of consolidation test from (a) Company A and (b) Company B.

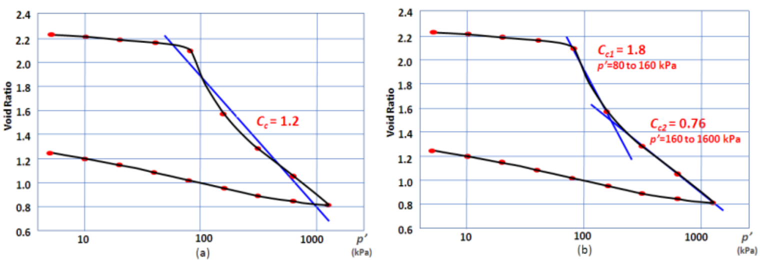
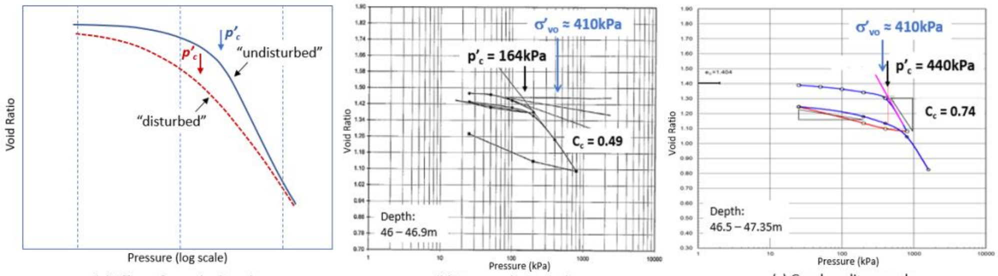


Fig. 5 Determination of Cc value: (a) average; and (b) based on stress range.

The value of C_c can be affected by sample disturbance. As illustrated in Figure 6a, sample disturbance tends to produce a flatter and more rounded e-log p' curve and give smaller values of p'_c and C_c as compared to those from "undisturbed" samples. Figures 6b and 6c show the test results from samples at about the same depth at nearby boreholes. The poor quality sample (Figure 6b) produces much smaller p'_c and C_c values as compared to a better quality sample (Figure 6c). The difference caused by sample disturbance in this example is evident because there is a comparison of a good quality and a poor quality test. Where there is no comparison, it can be difficult to assess the quality of the test result without prior experience.



(a) Effect of sample disturbance (b) Poor quality sample (c) Good quality sample

Fig. 6 Effect of sample disturbance on p'_c and C_c

Values of C_c can be verified with index properties. For instance, the upper marine clay located within a project site is likely to correlate with the water content as illustrated in Figure 7. Any data points that deviate from the general trend of correlation should be carefully reviewed. There are also many published empirical correlations that correlate C_c with different index properties. Some of them are shown in Figure 8.

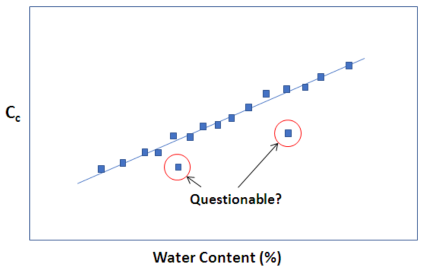


Fig. 7 Variation of C_c with water content

Wroth and Wood (1978)
$C_c = PI/74$ *for* $G_s=2.7$

Terzaghi and Peck (1967)
$C_c = 0.009\ (LL - 10)$

Azzous et al. (1976)
$C_c = 0.4\ (e_o - 0.25)$

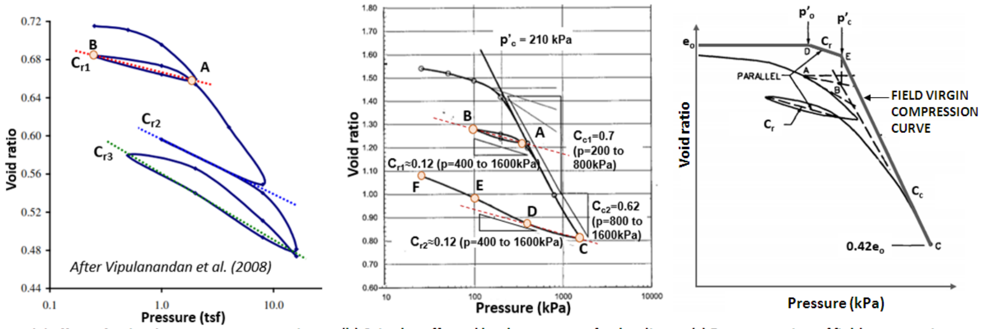
Rendon-Herrero (1983)
$C_c = (1+e_o)^{2.38}/23$ *for* $G_s=2.7$

Fig. 8 Some empirical correlations on C_c

2.1.3 Re-compression Index C_r

C_r is determined from the unloading or unloading-reloading part of the e-log p′ curve (Figure 9). This value is influenced by (i) the pressure at which unloading begins; (ii) the pressure at the end of unloading; and (iii) the curve fitting process.

Figure 9a illustrates that the value of C_r is affected by the pressure at which unloading begins. According to Schmertmann's procedure (1953) for reconstruction of the field compression curve (Figure 9c), it is preferable to start the unloading-reloading cycle at a pressure that is slightly higher than the preconsolidation pressure such as Point A in Figure 9a. Since p'_c is not known during testing, one can use the point of maximum curvature as a reference. For engineering applications where there is a known unloading pressure, an additional unloading-reloading cycle should be conducted at the anticipated unloading pressure.



(a) Effect of unloading pressure on C_r value (b) C_r is also affected by the amount of unloading (c) Reconstruction of field compression curve

Fig. 9 Unloading-reloading cycles and C_r value

To derive a reasonable C_r value for the reconstruction of the field compression curve, the pressure at the end of the unloading can be taken as a quarter to one-tenth of the pressure at the beginning of unloading. The C_r value is the slope of the line that connects the point of intersection (A) of the unloading and reloading curve and the point (B) at the end of unloading.

In some tests, there is only one final unloading without any additional unloading-reloading cycle. Unloading-reloading is considered as optional in BS 1377-5:1990 and ISO/TS 17892-5. In this case, the unloading-reloading curve (A-B) is absent and only one unloading curve (C to F) is present. The slope of a line that joins points C and D will produce a reasonable C_r which is comparable to that of line AB. This is provided that the pressure at D is approximately 1/4 to 1/10 of the unloading pressure at C.

In the absence of reliable test results on C_r, a reasonable approximation is to adopt $C_r \approx 0.2C_c$.

2.1.4 Coefficient of Consolidation c_v

c_v is commonly determined using the Taylor's square root of time fitting method (Taylor, 1948) which hinges on the determination of the time to achieve 90% degree of consolidation, t_{90}. Unfortunately, the time-settlement data at each load step is usually not presented in the factual reports.

The authors have observed that the determination of t_{90} and consequently the c_v value can be subjective. The value of t_{90} is sensitive to the curve fitting as illustrated in Figure 10. At initial load steps, settlement can occur very rapidly (Figure 10c). If the settlement is not measured accurately and at appropriate time intervals, the reliability of the c_v value will be affected.

Various trends of the c_v-log p' plot are presented in Figure 11. Figure 11a shows the results of a good test on soft marine clay. The c_{vr} and c_v values are distinctively associated with recompression and virgin compression, respectively. Figures 11b and 11c show the results of poor quality tests where the c_{vr} value cannot be distinguished from the c_v value. In the absence of reliable results on c_{vr}, this value can be estimated as 5 to 10 times of c_v.

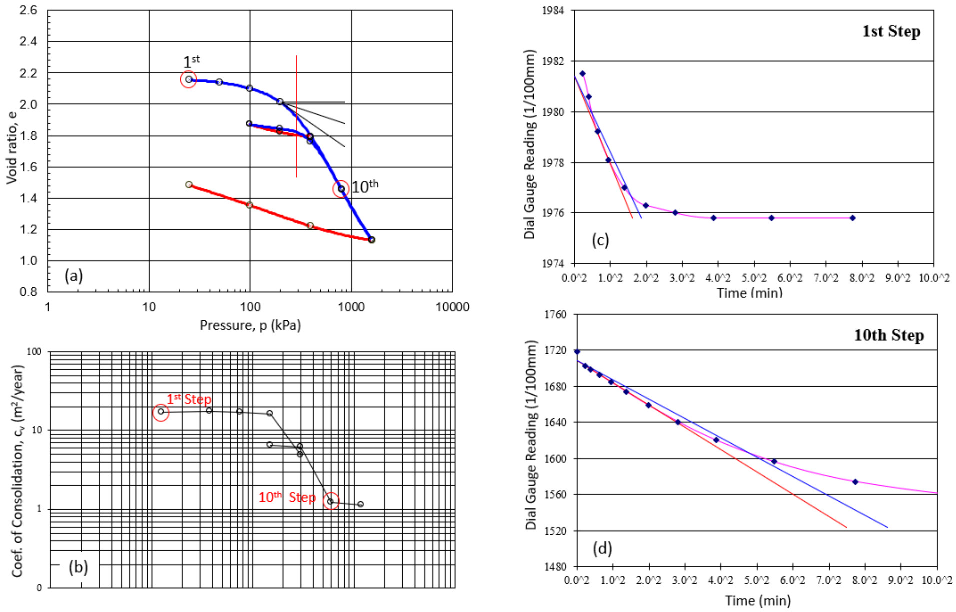


Fig. 10 Determination of c_v during unloading-reloading and primary loading.

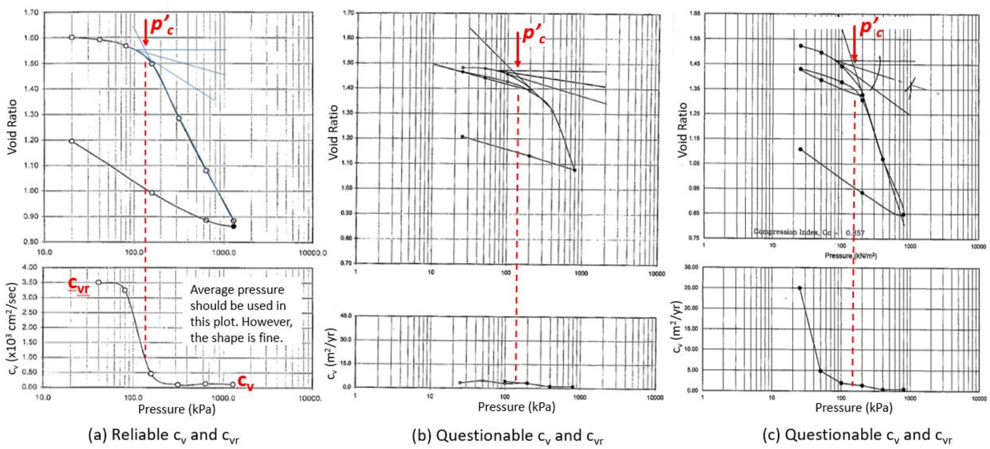


Fig. 11 Examples of c_v curves from consolidation test on soft clay.

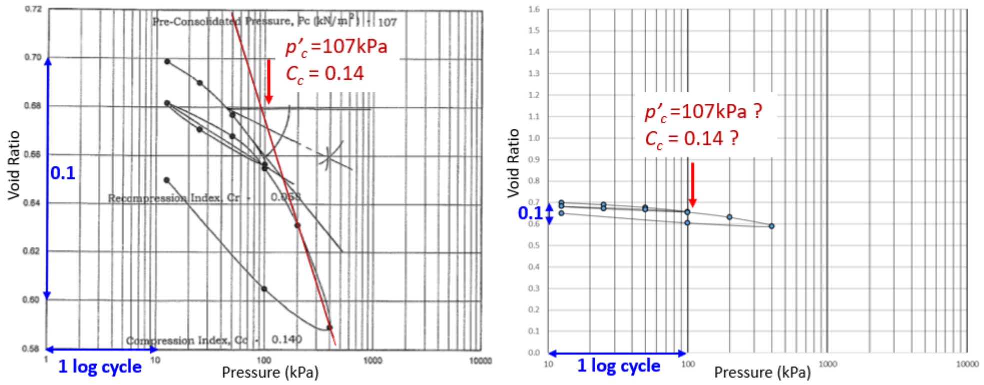
2.1.5 Compressibility of Residual Soils

This discussion has so far been limited to the consolidation of soft clays. Consolidation tests conducted on the residual soils of Bukit Timah Granite and Old

Alluvium reveal a distinctively different behaviour. These materials behave like a heavily over-consolidated clay.

Figure 12a shows the e-log p′ curve of a granitic residual soil presented in a report. The vertical scale is exaggerated for clarity. Unfortunately, it also gives a distorted view of the test results. The parameters (p'_c, C_c and C_r) determined from this plot are questionable.

Figure 12b shows the same curve plotted using the scale recommended by the Japanese Geotechnical Society Standard (JGS 0411-2009). This plot reveals that the compressibility of the soil is very low. The p'_c and C_c values cannot be determined.



(a) Exaggerated vertical scale

(b) Vertical scale according to Japanese Standard

Fig. 12 Influence of the vertical scale in e-log p' curve.

Results of a well-conducted test on a good quality sample are shown in Figure 13. It is difficult to determine a reliable p'_c even though the sample was tested to 6400 kPa. The c_v curve also does not have the "Z" shape which indicates that the soil may not have a p'_c or the maximum applied pressure falls short of the p'_c value.

In calculating the settlement of residual soils, the authors propose the use of the coefficient of volume compressibility, m_v. An approach to derive the m_v value is illustrated in Figure 13c. The first step is to determine the m_{vo} which is the m_v value at the existing effective overburden pressure p'_o. The second step is to determine the correlation between m_v and p' (Figure 14). The value of m_v for calculating settlement can then be determined based on the equation shown in Figure 14.

2.2 Undrained Shear Strength from UU Test

UU test is commonly carried out to determine c_u with ϕ_u set to zero. Theoretically, c_u is independent of the confining pressure. In reality, the occurrence of three Mohr circles of different sizes (Figure 15) is common.

The following aspects should be reviewed to assess the quality and reliability of the test results:

- Material composition
- Degree of saturation
- Stress-strain curve
- Multi-stage testing

The quality of UU test results are probably more affected by sampling and sample preparation than CU and CD tests. Observation of the condition and the consistency of the samples during sample extrusion and sample preparation in the laboratory can help in better understanding the test results.

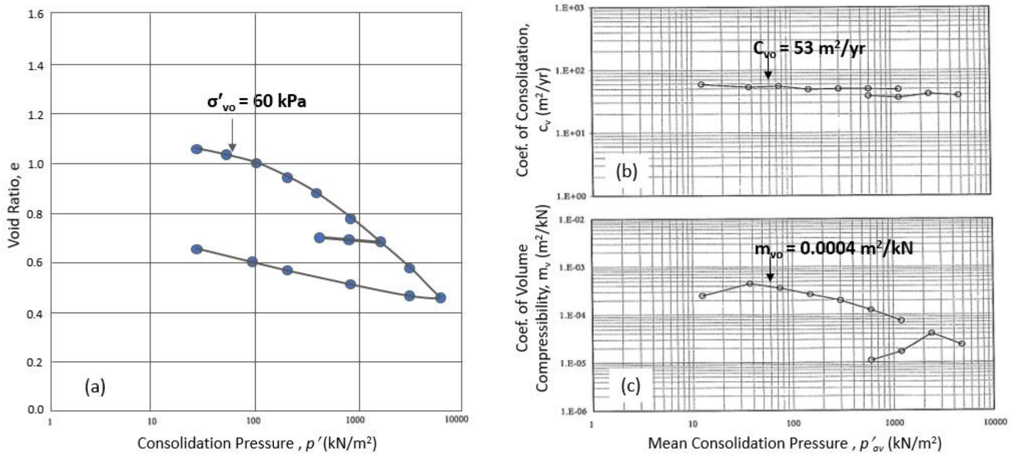


Fig. 13 Typical example of reliable results from consolidation test on granitic residual soil.

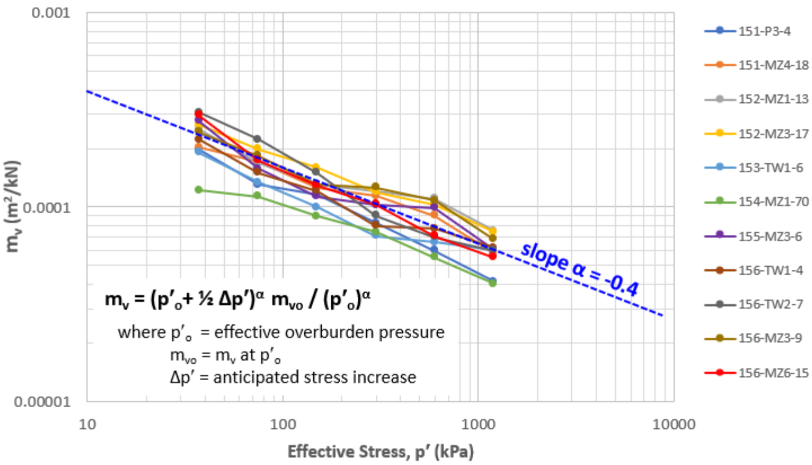


Fig. 14 Variation of m_v with pressure for residual soils of Bukit Timah formation

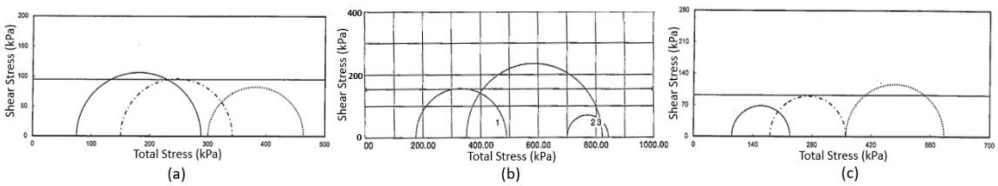


Fig. 15 Examples UU test results with Mohr circles of different sizes.

2.2.1 *Material composition*

First and foremost, the material composition in the three specimens should be checked whether they are similar. The differences can be identified from:

- Difference in soil density and water content among the three samples, e.g. water content = 67%, 54% & 55%.
- Difference in degree of saturation (S_r), e.g. S_r=83%, 87% and 98%.
- Variation of material texture and colour (Figure 16a)
- Presence of relic structure (Figure 16b) or fissures in specimens (Figure 16c)

Variation in material composite is a common reason for difference in the c_u value of the three specimens.

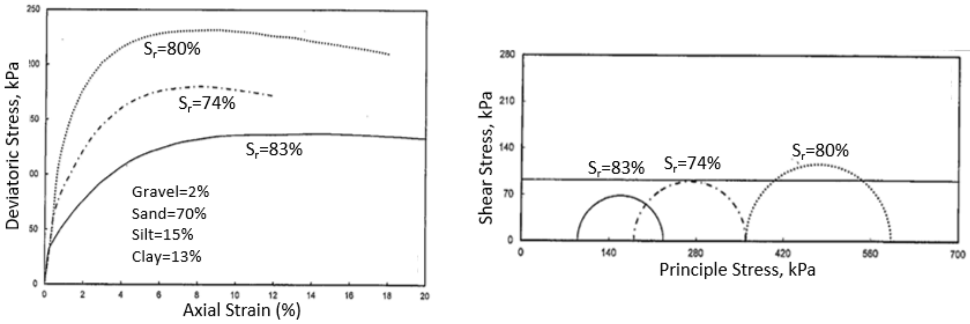
(a) Different materials in 3 specimens (b) Relic structures in G(V) soil (c) Fissure in O(A)

Fig. 16 Possible causes for variation of strength among the three samples.

2.2.2 *Degree of saturation*

UU test specimens are usually tested without back-pressure saturation. If the initial S_r is low, the test results do not reflect the true undrained strength of the soil. The presence of air voids can cause the soil sample to consolidate when subject to the chamber pressure and the test is no longer "unconsolidated undrained". The shear strength tends to increase with increasing confining pressure as shown in Figure 17. When S_r drops below 95%, results may become questionable. If S_r is lower than 90%, the results may be no longer relevant.

UU tests are meant for fine-grained soils and are not suitable for soils of high permeability, such as sands and gravels with less than 35% fines.



(a) Stress-strain curves from UU test (b) Mohr diagram

Fig. 17 Effect of degree of saturation on UU test results.

2.2.3 Stress-strain curve

Failure strain is often used as a guide to assess the quality of the test results. For normally consolidated clay or lightly over-consolidated clay like the Singapore marine clay, good quality tests give stress-strain cures that are bundled together with failure strain less than 5% (Figure 18). For over-consolidated residual soils, the failure strain of good quality sample can vary from 2% to over 10%. Questionable results are sometimes easily identified from the stress-strain curves as illustrated in Figure 19.

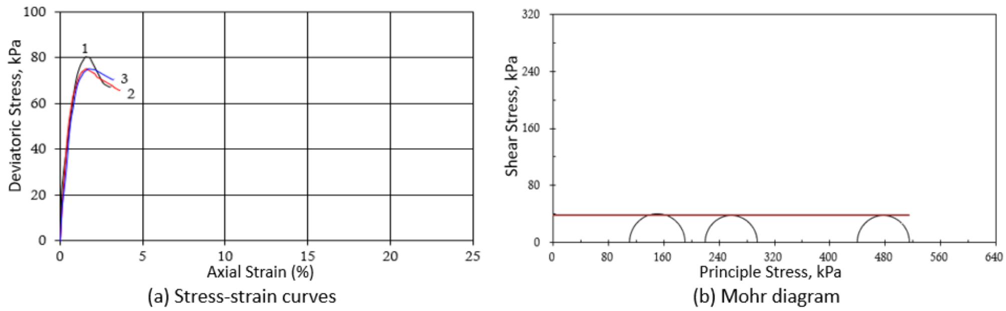


Fig. 18 Results of a reliable UU test on Singapore marine clay.

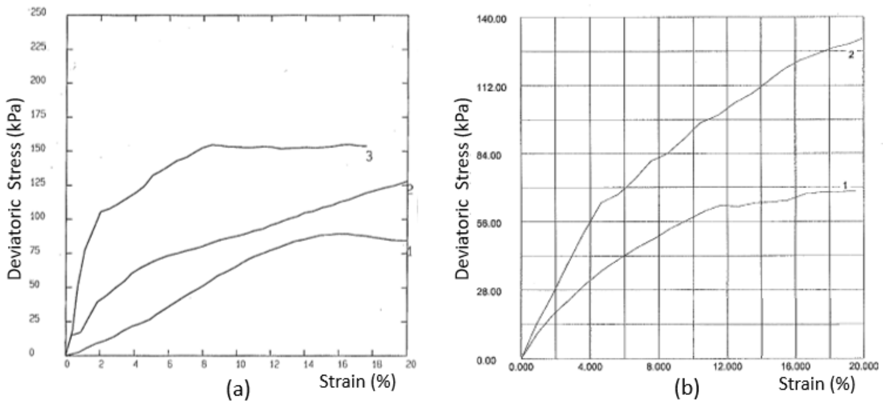


Fig. 19 Samples of questionable results based on appearance of stress-strain curves

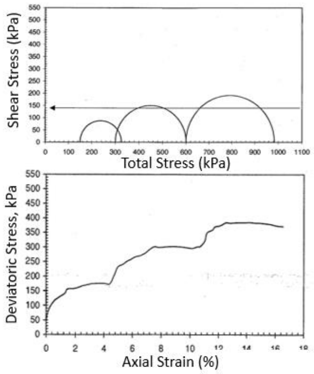
2.2.4 Multi-Stage Testing

Sometimes when there is insufficient sample, a multi-stage test with 3 confining pressures is conducted on a single sample. This is an approved procedure in BS 1377-7:1990. However, multi-stage testing has been found to produce Mohr circles that increase in size with increasing confining pressure as illustrated in Figure 20.

Theoretically, only one Mohr circle is required to determine c_u. Instead of conducting a multi-stage test, a good test on a single sample subject to one confining pressure is sufficient to obtain a reliable c_u. Multi-stage UU test should be avoided.

2.2.5 *Method of sample trimming*

As a good practice, samples should be hand-trimmed using a piano wire (Figure 21a) or a steel blade. To expedite the sample preparation process, trimming is sometimes done by pushing or jacking a 38mm tube into the thin-walled or Mazier sample (Figure 21b). This process can disturb the sample and reduce the shear strength. Although this procedure is described in BS 1377-1:1990, it should be avoided.




(a) Trimming using piano wire


(b) Trimming using a 38mm tube

Fig. 20 Results from multi-stage test

Fig. 21 Trimming of soil sample

2.2.6 *Water Softening*

The Mazier sampler is commonly used to obtain “undisturbed” samples in very stiff and hard soils. The coring process inevitably exposes the soil to the circulation water. If the soil is susceptible to water softening, the shear strength of the soil sample will be greatly affected. Figure 22 shows some affected samples after extrusion. Even at N>40, the sample can be easily indented by fingers with moderate pressure.

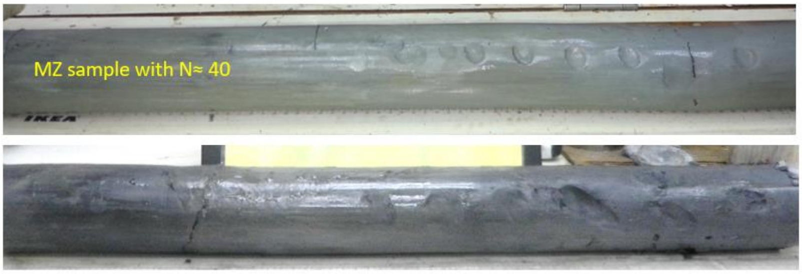
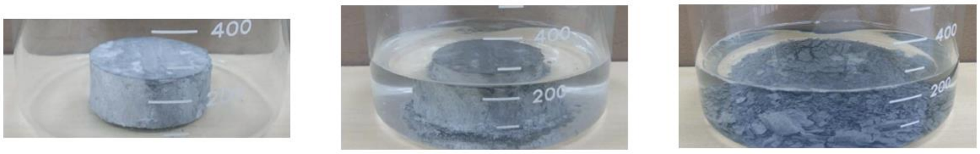


Fig. 22 Mazier samples from Jurong Formation after extrusion

The residual soils of the Jurong formation and the Bukit Timah formation are especially prone to water softening due to high silt content. Figure 23 shows the rapid disintegration of a specimen from the Jurong Formation in the Japanese slaking test. The Old Alluvium soils fare much better due to cementation and high sand content.

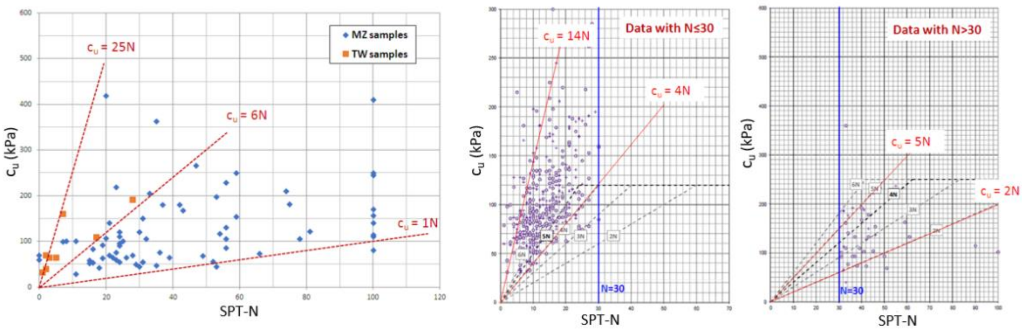
For soils with N<30, water softening may not be critical. The thin-walled sampler is generally used in soils with SPT blow counts up to about N=30. This method does not expose the sample to circulation water. For Mazier sampler, the cutting shoe can also penetrate into the soil of low blowcounts (N<30). Hence it is able to shield the sample from the circulation water.



(a) Before immersion (b) 1 minute after immersion (c) 30 minutes after immersion

Fig. 23 Results of Japanese slaking test on a residual soil specimen of the Jurong Formation.

The effect of water softening in Mazier samples provides an explanation for observed lower correlation factor between c_u and N. Figure 24 shows the results of UU test on residual soils of the Jurong formation and the Bukit Timah formation. At low SPT blow counts, the correlation between c_u and N is much more favourable than at high blow counts.

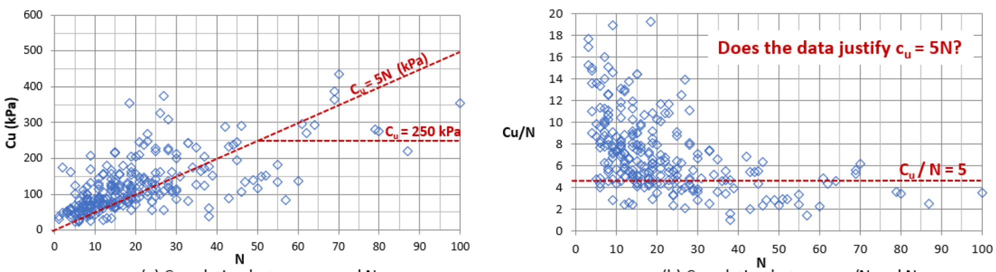


(a) Results of UU tests -- Jurong Formation (b) N≤30 -- Bukit Timah Formation (c) N>30 -- Bukit Timah Formation

Fig. 24 Effect of water softening on UU test results.

2.2.7 Overall review of test results

For residual soils, the plot of c_u versus N is commonly used to justify a correlation of c_u=5N up to a limiting c_u of 200 kPa to 400 kPa (Figure 25a). The data can also be plotted as c_u/N versus N. This plot provides a clearer picture on the variation of c_u with N as illustrated in Figure 25b. The correlation of c_u=5N under-estimates the undrained shear strength at low N values and over-estimates the strength at high N values.



(a) Correlation between c_u and N (b) Correlation between c_u/N and N

Fig. 25 Correlation between (a) c_u and N and (b) c_u/N and N.

The c_u values from all UU tests should be crossed-checked and also reviewed with results from other tests. For soft marine clay, a plot of c_u against depth with the reference line ($0.22p'_o$), such as the one shown in Figure 26a, reveals the state of consolidation of the clay and any outliner data. Data from other types of test such as field vane test (Figure 26b) and consolidation test (Figure 26c) can be used for verification.

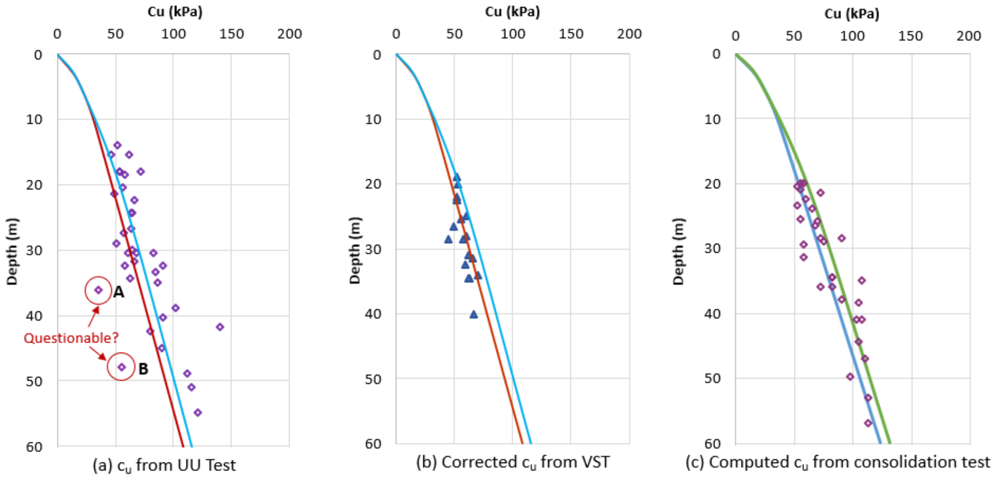


Fig. 26 Checking measured c_u against other data and tests

2.3 CU Test

The purpose of the CU test is to derive the effective stress parameters c′ and ϕ′. Most factual reports present the seven plots below for each CU test.

- B-value plot during back-pressure saturation phase
- Volume change during consolidation phase
- Pore pressure changes during shearing phase
- Deviatoric stress-strain plot during shearing phase
- Principal stress ratio stress-strain plot during shearing phase
- Stress path plot in s'-t' space.
- Mohr diagram

Typical plots from a CU test on marine clay and an Old Alluvium soil O(C) are shown in Figures 27 and 28, respectively.

2.3.1 B-value plot during back-pressure saturation phase

This plot shows the change in the B-value with the cell pressure during the sample saturation. It gives an indication of the uniformity of the three specimens (Figures 27a and 28a). If all three curves bundle closely together, this is an indication that the specimens are relatively uniform. If one of the curves deviates widely from the other two, it may imply that the material composition of one specimen is different from the other two.

2.3.2 Volume change during consolidation phase

This plot shows the rate of consolidation and the relative permeability of the soil (Figures 27b and 28b). The t_{100} value can be used to check whether the shearing rate is appropriate. According to Blight (1963), the time to failure during shearing can be estimated as 2.3 times of t_{100} for a specimen of aspect ratio of 2 with both ends and radial drainage during consolidation.

This plot can also provide a check on the uniformity of the three soil specimens. A notably different t_{100} value is an indication that a specimen has a different material composition as compared to the other two.

2.3.3 Pore pressure changes during shearing phase

Figures 27d and 28d are illustrations of typical pore pressures generated during the shearing phase of a normally-consolidated clay and a heavily over-consolidated soil, respectively. If the expected curves deviate from these trends, it implies the results of this test may be questionable. For example, if the clay is normally consolidated but the curves resemble that of Figure 28d, something is wrong with the results.

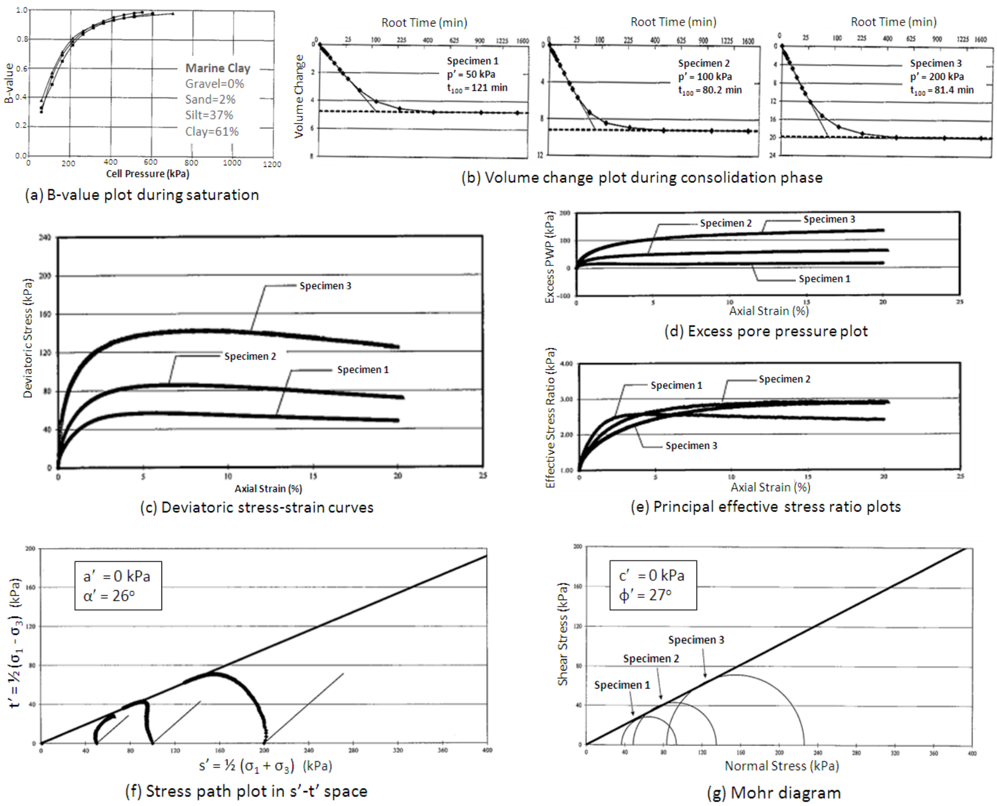


Fig. 27 Plots from CU test on Marine clay

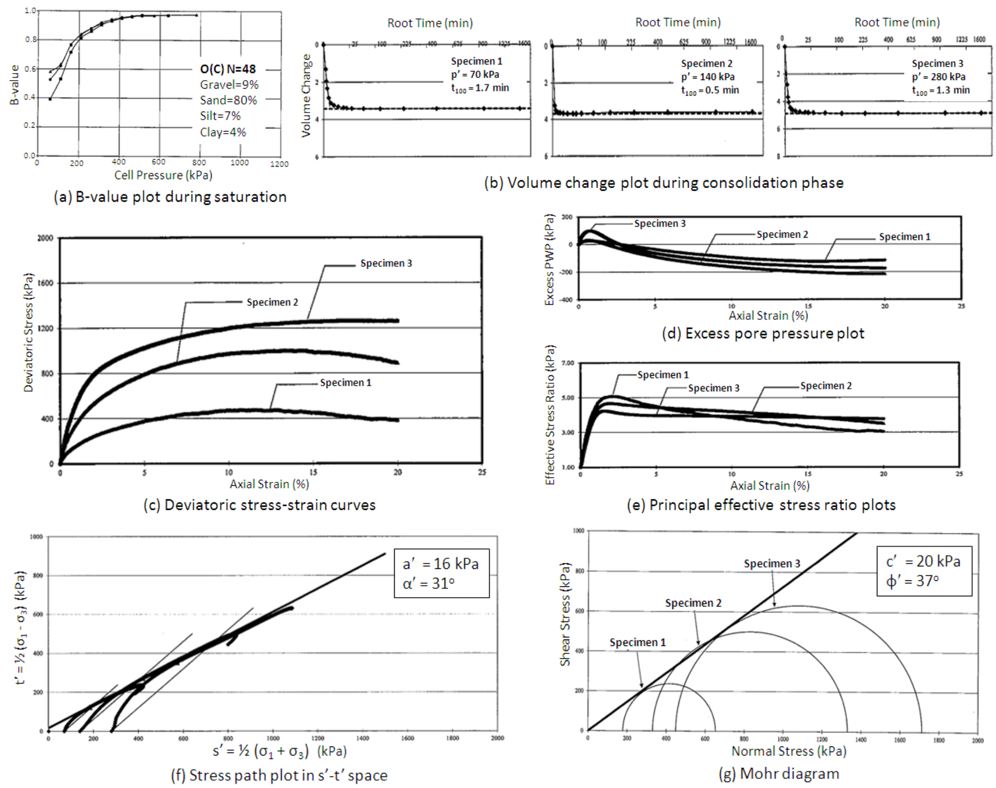


Fig. 28 Plots from CU test on an Old Alluvium soil O(C)

2.3.4 Stress-strain plot based on deviatoric stress and principal stress ratio

For normally consolidated and lightly overconsolidated soils, the deviatoric σ-ε curves are usually well-defined as illustrated in Figure 27c. For heavily overconsolidated soil, such as the Singapore residual soils, the shape of the deviatoric σ-ε curves can vary over a wide range. The curves may not achieve a peak stress value even at 20% strain because the soil continues to dilate and gain strength during shearing (Figure 28c).

It can be difficult to conclude on the quality of the test results solely based on the deviatoric σ-ε curves. Figure 29 shows the σ-ε curves of three tests A, B and C on Bukit Timah residual soil. From the σ-ε curves of Test C (Figure 29c), it may be mistaken that the specimens are disturbed.

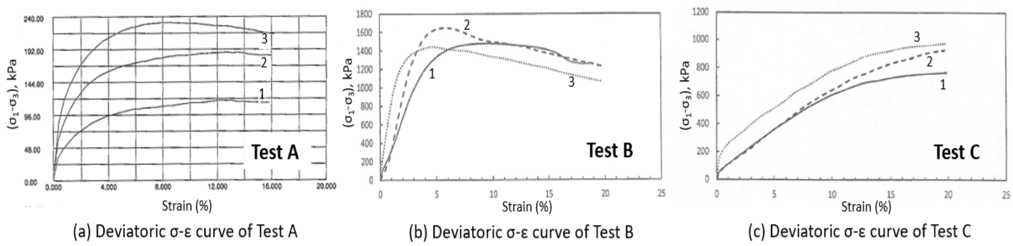
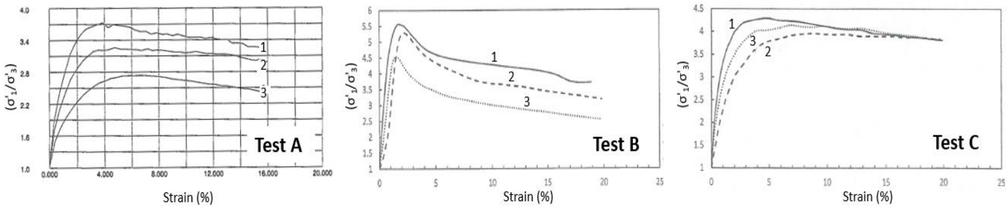


Fig. 29 Deviatoric σ-ε curves of Test A, B and C in Bukit Timah residual soil.

The quality of test results is better reflected in principle stress ratio-strain curves. The failure strain at the peak stress ratio is usually less than 10%. Figure 30 shows the stress ratio plots for the same three tests A, B and C. The curves in Test C are in fact acceptable. When the stress ratio curves are spaced apart, such as in Test A (Figure 30a), the c’ value is large. When the curves bundle closely together, such as in Tests B and C (Figures 30b and 30c), the c′ value is small.

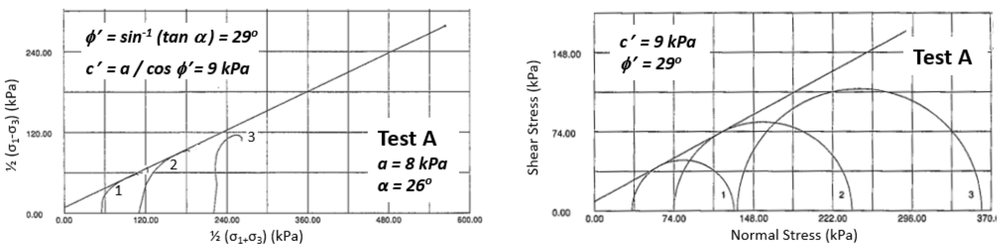


(a) Stress Ratio σ-ε curve of Test A (b) Stress Ratio σ-ε curve of Test B (c) Stress Ratio σ-ε curve of Test C

Fig. 30 Principal stress ratio σ-ε curves of Test A, B and C.

2.3.5 Stress path plot in s’-t’ space

This is an important plot. It is useful in assessing the quality of the test results, the material uniformity of the test specimens and the interpretation for the failure envelope. The plots for a normally consolidated clay and an over-consolidated soil are shown in Figure 27f and 28f, respectively. Figure 31a shows the stress path plot of Test A. The plot indicates that the soil is lightly over-consolidated. The failure envelope fits nicely with the stress paths of Specimens 1 and 2. The one from Specimen 3 falls short of the anticipated failure strength.



(a) Stress path plot in s’-t’ space (a) Mohr diagram

Fig. 31 Results of Test A: (a) stress path plot and (b) Mohr diagram

2.3.6 Mohr diagram

To construct the Mohr diagram, it is necessary to determine the principal stresses at failure. The following failure criteria are commonly adopted in local practice:

- Peak deviatoric stress
- Peak principal stress ratio
- Maximum stress at 20% strain

The choice among these three criteria depends on the failure strain associated with each criterion. The one criterion with the smallest strain at failure is the governing criterion.

For the residual soils, the strain at the peak principal stress ratio (Figure 28e) is usually much smaller than the strain at the peak deviatoric stress (Figures 28c) for all the three specimens. Hence the peak stress ratio failure criterion normally governs. For other soils, it is possible that the failure criterion varies for the three specimens. For example, in Figure 27c and 27e, the peak stress ratio criterion applies to Specimen 1 whereas the peak deviatoric stress criterion applies to Specimens 2 and 3.

It is often the case that the constructed Mohr diagram shows three Mohr circles that do not fit on one straight-line failure envelope. As an example, Figure 32 shows the results of a CU test. It is assumed that the test results are satisfactory and the material is uniform. Therefore, all three Mohr circles are equally valid and reliable. There are four options to fit a straight-line to these Mohr circles. Each option gives a different set of c′ and ϕ′. The c′ value can vary vastly from 0 kPa to 51 kPa whereas ϕ′ varies between 38° and 41.3°. It is worth noting that the c′ value can be very sensitive to the fitting of a straight-line failure envelope.

It should be reminded that the failure envelope of soils is not a straight line. The decision on how to fit a straight-line failure envelope should be based on the stress range of interest. For embankments and dams where there is stress increase over the existing stresses, an appropriate choice will be (c), i.e. fitting the 2nd and 3rd circles. For excavations where there is stress reduction below the existing stresses, an appropriate choice will be (b), i.e. fitting the 1st and 2nd circles.

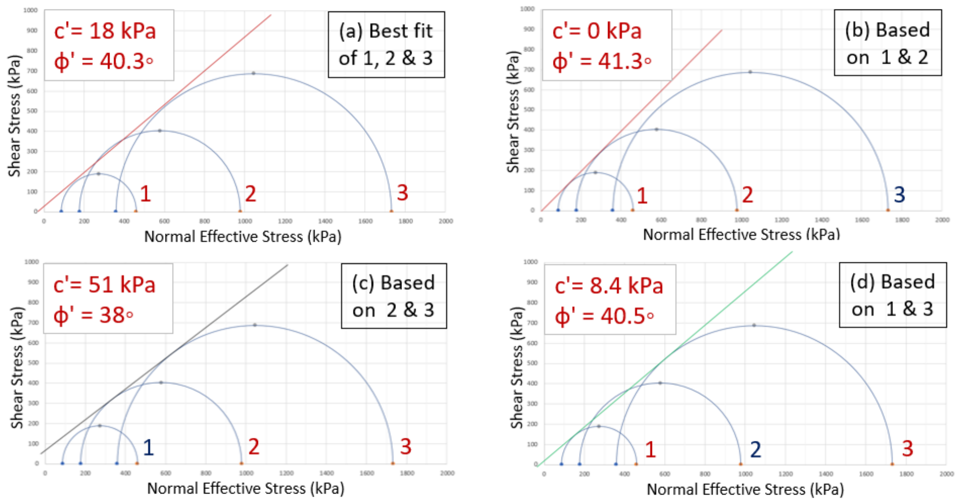


Fig. 32 Effect of curve fitting on c′ and ϕ′ values.

2.4 CD Test

The purpose of the CD test is to produce the drained strength parameters (c′ and ϕ′) and soil modulus (E′). Most factual reports present 6 plots for each test. They are:

- B-value plot during back-pressure saturation phase
- Volume change during consolidation phase

- Volume change during shearing phase
- Deviatoric stress-strain plot during shearing phase
- Principal stress ratio stress-strain plot during shearing phase
- Mohr diagram

Typical plots from a CD test on a residual soil are shown in Figure 33. As in the case of the CU test, the B-value plot during saturation phase and the volume change plot during consolidation phase can provide insight into the material uniformity of the three specimens in each test. The volume change plot during the shearing phase also reflects on the compressive or dilative behavior of the soil during shearing.

The CD test requires a very slow shearing rate to ensure that all excess pore pressure is dissipated. If the shearing rate is too fast, it will give a increasingly high cohesion value. The volume change during consolidation phase provides information (t_{100}) for checking the shearing rate. BS1377-8:1990 estimates that the time to shear failure is about 16 times t_{100} for a specimen of aspect ratio 2 with both ends and radial drainage during consolidation.

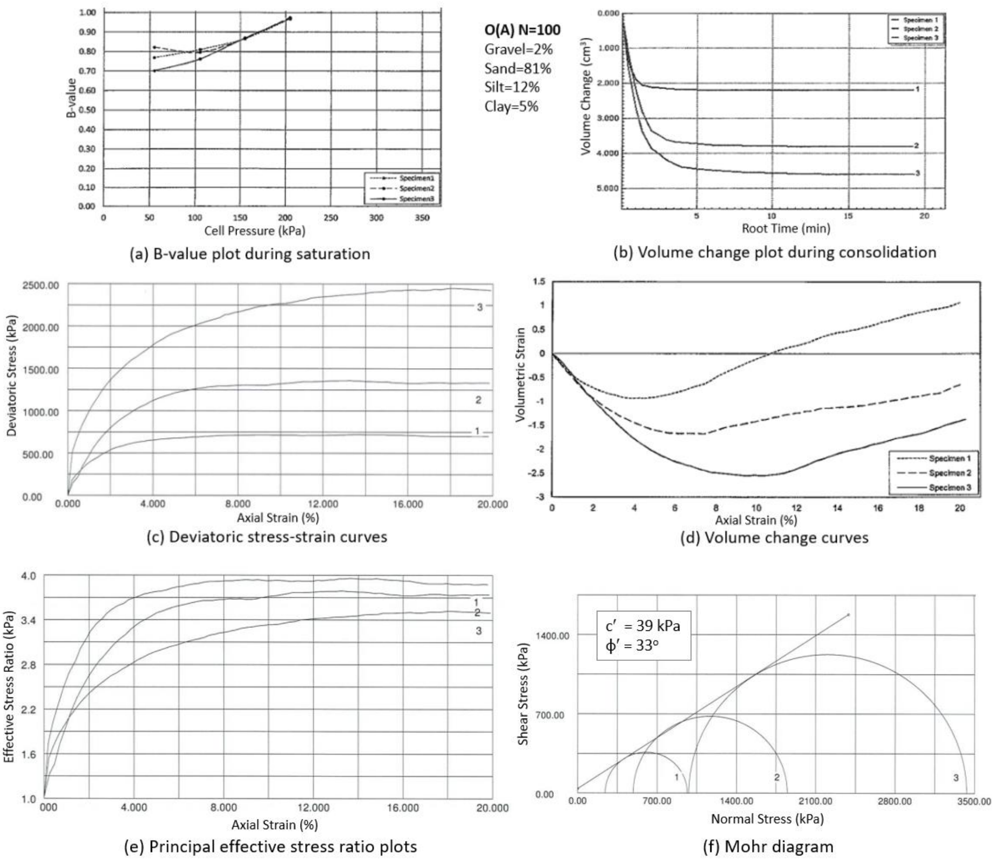
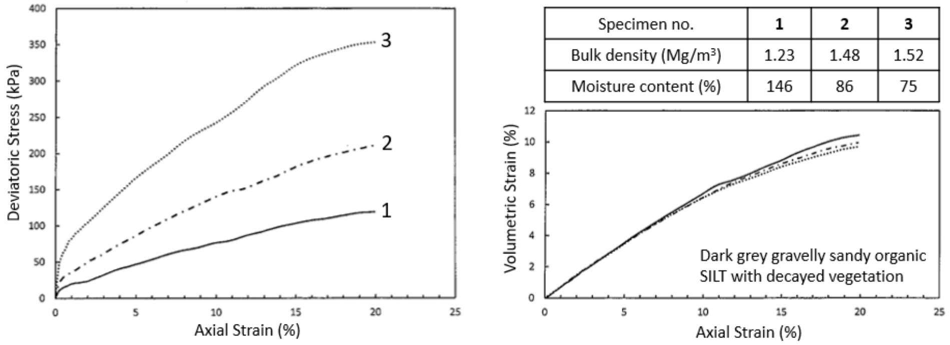


Fig. 33 Results of CD test on an Old Alluvium soil O(A)

The deviatoric stress-strain plots are usually better defined than those from CU test. Questionable results are easily detected as illustrated in Figure 34.

Contrary to the case for the CU test, the principal stress ratio stress plot is less important in the CD test as the failure strains in the peak stress ratio plot (Figure 33c) are the same as those in the deviatoric stress-strain plot (Figure 33e). Hence, both the maximum deviatoric stress failure criterion and the maximum principle stress ratio failure criterion give the same failure stresses and identical Mohr circles.

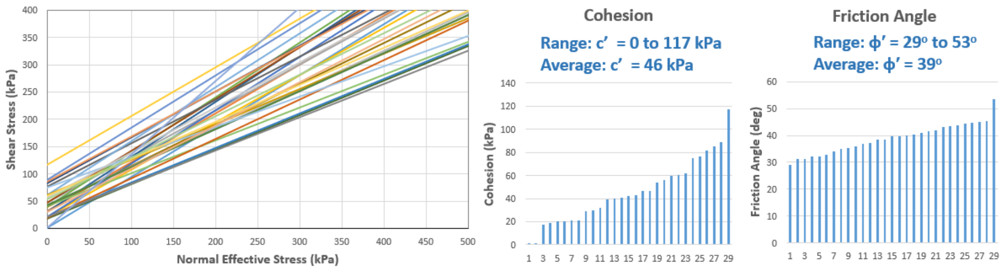


Specimen no.	1	2	3
Bulk density (Mg/m³)	1.23	1.48	1.52
Moisture content (%)	146	86	75

Fig. 34 Results of a questionable CD test.

2.5 Determination of c′ and ϕ′ from a large volume of test data

In determining the characteristic value for c′ and ϕ′ from CU and CD tests, a ubiquitous approach is to replace each Mohr circle with a point in the s’-t’ space which is also known as the modified Mohr diagram. When there is a large volume of data, determining the characteristic c′ and the ϕ′ values from the s’-t’ plot has its pros and cons. One noteworthy shortcoming is that this approach tends to significantly underestimate the value of c′. This is simply because a straight line is fitted among a scatter of data. For instance, Figure 35a shows the Mohr envelopes of 29 CU tests on O(A). The c′ value varies from 0 kPa to 117 kPa and the ϕ′ from 29^{o} to 53^{o} as shown in Figure 35b and 35c respectively. When the same test data is plotted on the s’-t’ plot (Figure 36), the line of best-fit gives c′=0 and ϕ′=42^{o} which evidently misrepresents and undervalues the actual c′ values derived from the tests.



(a) Mohr envelopes of 29 CU tests (b) Variation of c′ values (c) Variation of ϕ′ values

Fig. 35 Mohr envelopes of 29 CU tests and ranges of c′ and ϕ′ values

There are other approaches that can be used to derive the characteristic values. The authors have observed that for the G(V) and G(VI) soils of the Bukit Timah Granite in the Ang Mo Kio area in central Singapore, reasonable correlations can be established between c′ and ϕ′. Figure 37a shows a strong correlation for G(VI) soil. For G(V) soil (Figure 37b), the correlation is more scattered due to: (i) the variable nature of this soil which covers N-values from 10 to 100; (ii) different soil composition varying from clay to gravel and (iii) the presence of relic structures in the test specimens. Another useful correlation which enables one to estimate the friction angle based on SPT-N is shown in Figure 38. Once ϕ′ is known, c′ can be readily determined from Figure 37. It should be noted that the N-values are measured field values not corrected for hammer energy ratio.

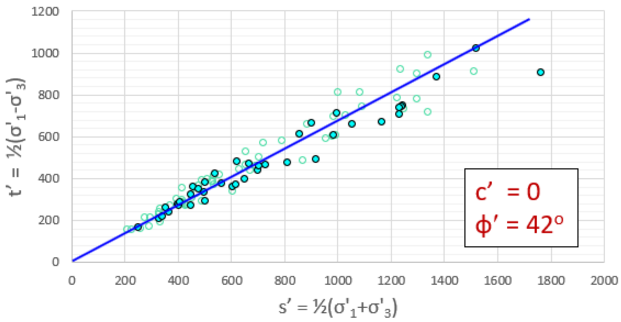


Fig. 36 Best fit line in s’-t’ plot

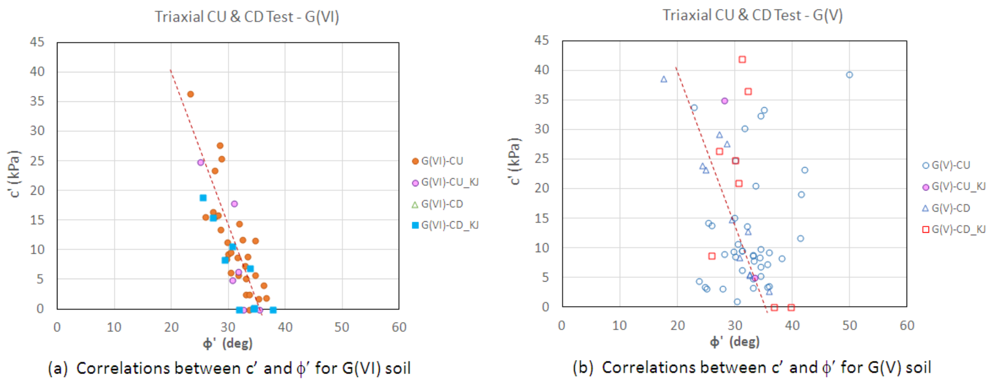


Fig. 37 Correlations of c′ and ϕ′ for residual soils of Bukit Timah formation in Ang Mo Kio

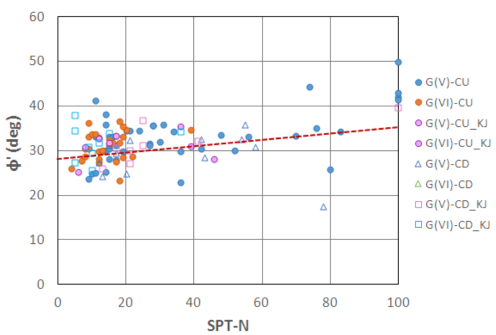


Fig. 38 Correlations between ϕ′ and (a) SPT-N and (b) fines content for G(V) & G(VI) soils

For Old Alluvium soils, the materials are very variable. Soil characterization is further complicated by the presence of cementation in these soils. No clear correlation between c′ and ϕ′ can be found. Three methods have been considered to determine the moderately conservative values of c′ and ϕ′.

Method A considers each pair of c’ and ϕ’ to be inter-related and inseparable. For the data shown in Figure 35, the shear strength is computed for each set of c′ and ϕ′ for confining pressures σ'_3 of 100 kPa, 300 kPa and 500 kPa. The data is shown in the s’-t’ plot in Figure 39. The mean values and standard deviation of s’ and t’ are then computed for each confining pressure. A line that fits through these mean values give c′=45 kPa and ϕ′=40^{o}. Following the proposal by Schneider (1999), the computed characteristic or moderately conservative values are c′=42 kPa and ϕ′=37^{o}.

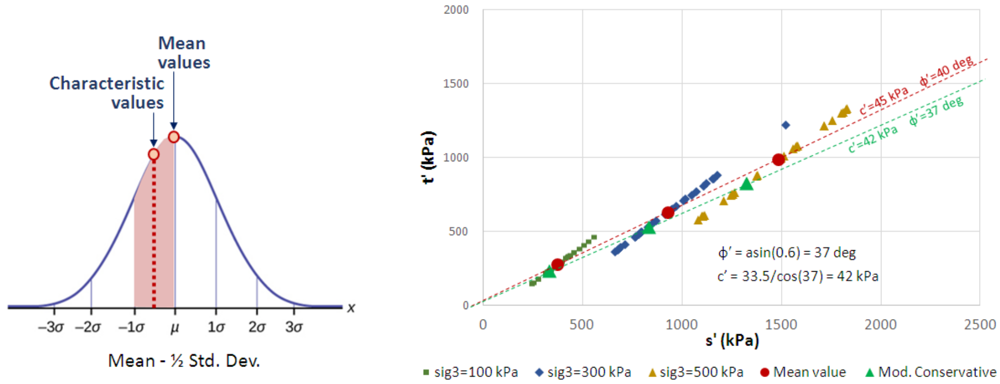
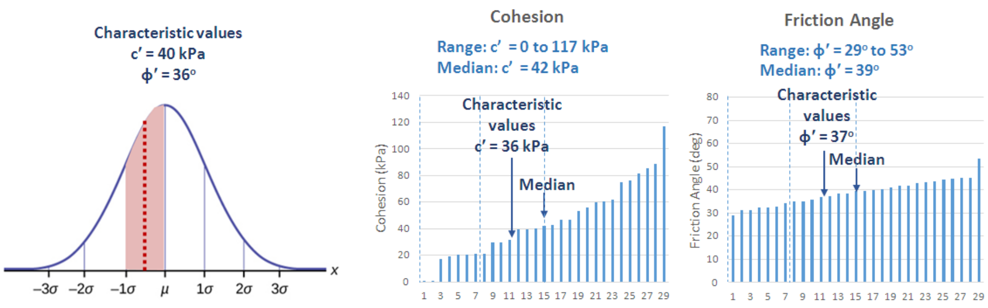


Fig. 39 Method A for determination of mean and moderately conservative values of c′ & ϕ′.

Method B evaluates c′ and ϕ′ separately. It is based on the mean and the standard deviation values as illustrated in Figure 40a. The characteristic value is taken as the value at half a standard deviation below the mean value (Schneider, 1999). For the example shown in Figure 35, the mean values of c′ and ϕ′ are 46 kPa and 39^{o} respectively. The characteristic values derived using this approach are c′=40 kPa and ϕ′=36^{o}.

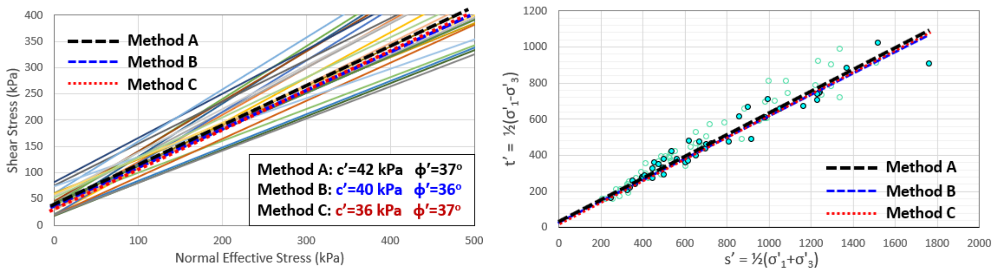
Method C also evaluates c′ and ϕ′ separately. The characteristic value is taken as the value at a quarter point below the median as illustrated in Figure 40b. For the data shown in Figure 35, the median values of c′ and ϕ′ are 42 kPa and 39^{o} respectively. The characteristic values are c′=36 kPa and ϕ′=37^{o}. An advantage of this method is that the characteristic values are not affected by the very large or very small outliner data in the data set.

Figure 41 shows a close comparison of the failure envelopes derived using these three methods with the measured values.



(a) Method B: Mean - ½ Std. Dev. (b) Method C: Median - ½ Quarter

Fig. 40 Schematics illustrating methods to determine the characteristic values.



(a) Comparison of design envelopes in Mohr diagram (b) Comparison of design envelopes in s′-t′ plot

Fig. 41 Comparison of measured data with design envelopes from Methods A, B and C.

3 FIELD TESTS

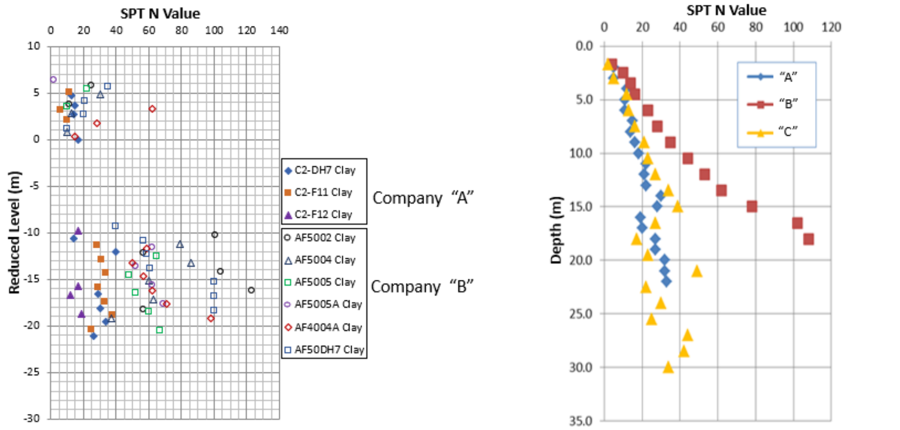
Field tests that are commonly conducted in Singapore include the Standard Penetration Test (SPT), the Vane Shear Test (VST), the Cone Penetration Test (CPT) and the Pressuremeter Test (PMT).

3.1 The Standard Penetration Test (SPT)

Contrary to being a “standard” test, the SPT can be influenced by field procedures, such as borehole drilling and cleaning, execution of the test and the condition of the testing equipment.

The effect of different field practices is sometimes manifested as inconsistency in the field-recorded SPT N-values. Figure 42 shows two projects whereby different testing companies give different range of SPT N-values in similar ground condition. From the field-recorded SPT N-values alone, it is difficult to assess the reliability of the tests.

When assessing SPT results, correction factors should be considered. These include correction for the hammer energy ratio, borehole diameter, rod length, sample liner and effective overburden.



(a) Site I – SPT results from two different companies (b) Site II – SPT results from three different companies

Fig. 42 SPT results from different companies at two different sites

Traditionally in local practice, SPT N-values are not corrected for hammer energy ratio because of the absence of hammer energy measurement. In a recent study, the energy ratios of 70 numbers of SPT hammers used in Singapore are compared. Figure 43 shows that the energy ratio of these hammers varies from 56% to 85%. This implies a difference in the SPT N-value of up to 1.5 times.

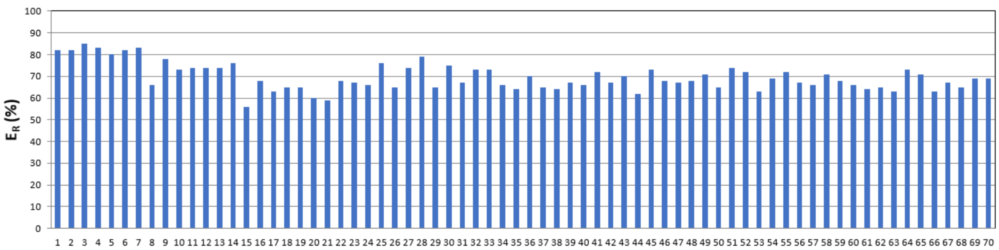


Fig. 43 Hammer energy efficiency of 70 hammers in Singapore

The effect of correcting for the energy ratio is illustrated in Figure 44. The figure compares the SPT N-values before and after correcting the energy ratio from 85% to 60%. This correction can be expected to influence the design of pile foundation using SPT N-values.

SPT correction is also covered in the BS EN ISO 22476-3:2005. The Standard includes correction for the energy ratio, rod length and sample liner for sands. For design applications, it may be necessary to correct the N values in sand to take into account the effect of overburden pressure.

It should be reminded that the SPT has limitations. For example, SPT N-values are not reliable for gravelly soils. In Singapore, the use of SPT N-value for correlation with the undrained shear strength for cohesive soils (such as c_u=5N kPa) is common. This correlation, however, tends to underestimate the c_u at shallow depths and is not appropriate for soft clays.

3.2 The Vane Shear Test (VST)

The VST is conducted to determine the undrained shear strength in soft to firm clays. Figure 45a shows the measured c_u values from three boreholes. The strength profile is well defined as the c_u values are all bundled closely together. This is a good example of VST result in a uniform soil deposit. Figure 45b shows the results from another site. The data are very scattered. This is due to the presence of shells, wood chips and sandy zones within the clay deposit. This is also a good example showing questionable results from VST results. This test is not suitable for certain types of soil.

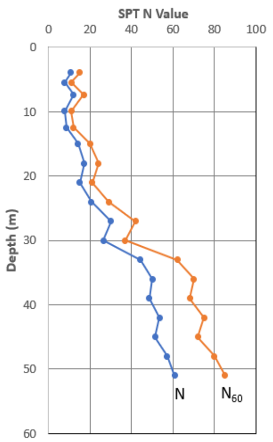
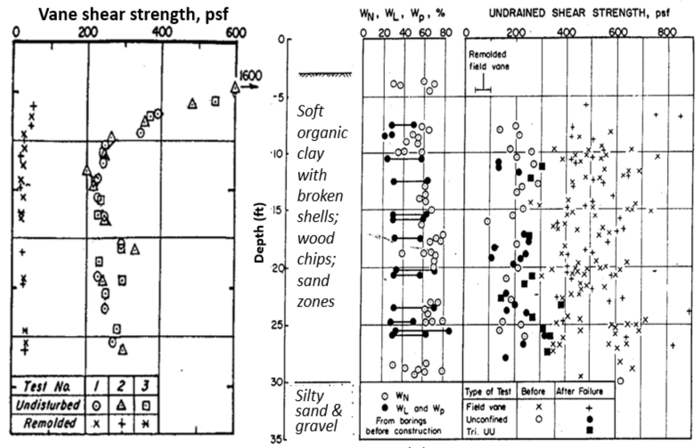


Fig. 44 N and N_{60} comparison

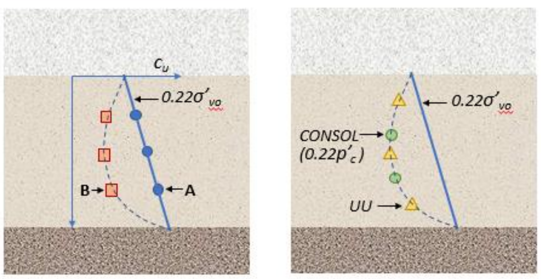


(a) Site A – reliable c_u from VST (b) Site B – unreliable c_u from VST

Fig. 45 VST results from Site A and B

Figure 46 illustrates examples of good versus questionable results. VST results from Company A fall along the c_u=0.22σ'$_{vo}$ line (Figure 46a) suggesting that the clay is normally consolidated. Results from Company B fall below the reference line indicating that the clay is still undergoing consolidation (Figure 46b). By comparing with the results of UU tests and consolidation tests, the VST results from Company B are validated as being more reliable.

The shear strength from VST should be corrected in practical application. The correction factor proposed by Bjerrum (1972) is presented in Figure 47.



(a) VST results from Company A & B (b) Results from other tests

Fig. 46 VST results from Company A and B

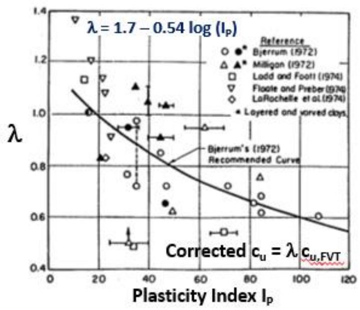
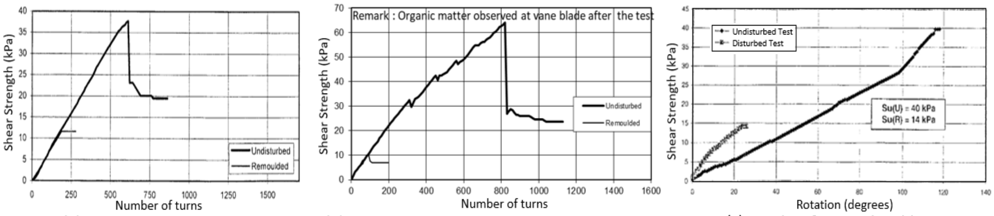


Fig. 47 Bjerrum's VST correction factor

It is a good practice to review the plot of the shear strength versus rotation or the number of turns for individual VST, such as the ones shown in Figure 48. Figure 48a shows the results of a reliable test. Results in Figure 48b may be affected by the presence of organic matter. Results in Figure 48c are questionable. The soil has not reached failure even after a rotation of 118 degrees.

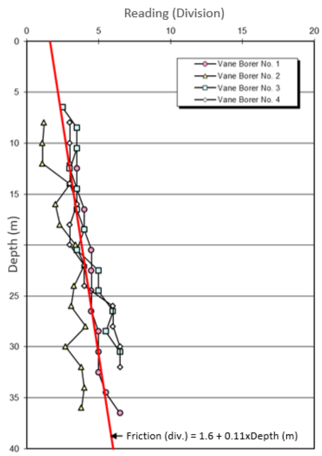


(a) Results of a reliable test (b) Results of may be affected by organic matter (c) Results of a questionable test

Fig. 48 Typical examples of VST results

The VST shear strength can also be affected by friction between the connecting inner rods and the casing. Frictional resistance increases the torque required to induce failure of the clay and may appear as an apparent additional shear strength of the clay. Figure 49 shows the measurement of the rod friction by performing the test in a cased hole without the vane blade. The measurement reveals that the rod friction induces an apparent shear strength up to 6 kPa for depths down to 40 m. The rods used in this measurement were well-maintained. The effect of rod friction in poorly maintained rods could be higher. The data presented in this figure is relevant only to a particular set of equipment.

There are other potential sources of errors that are related to the test equipment and procedures. These include poor condition of the equipment such as worn-out blades, incorrect rate of rotation, insufficient penetration of the vane into the undisturbed soil, outdated calibration and use of incorrect vane factor, etc. While most of these errors are not easily identified from factual reports, the last two factors may be traced by including records of calibration of the vane and the vane factor in factual reports.



Strength Correction Δc_u (kPa)

Depth	Vane Size 65x130mm	Vane Size 55x110mm
5	1.3	2.2
10	1.6	2.7
15	2.0	3.3
20	2.3	3.8
25	2.6	4.4
30	2.9	4.9
35	3.3	5.5
40	3.6	6.0

corrected c_u = measured c_u + Δc_u

Fig. 49 Rod friction in VST increases the measured shear strength

3.3 The Cone Penetration Test with Pore Pressure Measurement (CPTU)

The CPTU is a powerful tool to investigate the subsoil conditions. It can provide a continuous soil profile, cone resistance, frictional resistance and pore pressure response with depth. There are many published empirical or semi-empirical correlations with engineering parameters.

The measurement of pore water pressure in CPTU affects the computation of the cone resistance. The reliability of pore water pressure measurement relies on full saturation of the porous stone. Figure 50 presents the difference in the measurements due to poor saturation. In CPT-39, the porous stone was submerged in water for 24 hours. In CPT-39A, the porous stone was submerged in a water-glycerin mixture and boiled for 10 minutes. The difference in the measurements is evident.

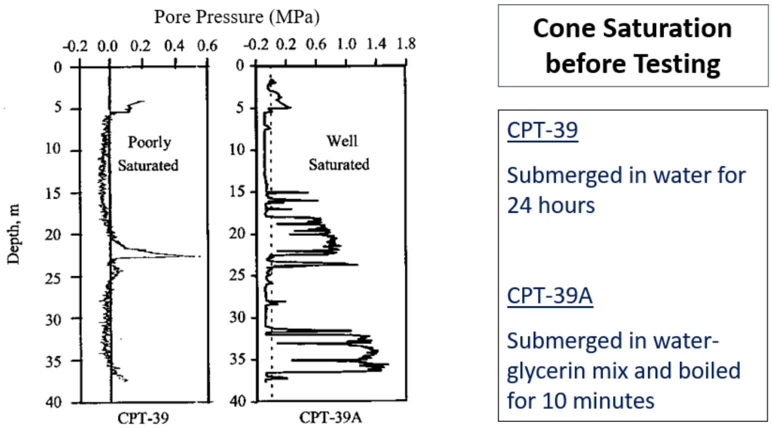


Fig. 50 Importance of pore stone saturation in pore pressure measurement.

The CPTU is also commonly used to determine the profile of the undrained shear strength of cohesive soils with depth. The calculation of the undrained shear strength depends on the cone factor N_{kt}. Figure 51 illustrates that the choice of N_{kt} value affects the assessment of the state of consolidation of the cohesive soil. In this example, the soil appears to be (i) over-consolidated if N_{kt} is 12; (ii) normally consolidated N_{kt} if 14; and (iii) still undergoing consolidation if N_{kt} is 20.

Accordingly, the N_{kt} value should be calibrated with results of other tests, including the VST (using corrected shear strength), the UU test and the consolidation test. Figure 52 presents an example of a CPTU test that is calibrated with other tests and gives the calibrated N_{kt} value as 20. It is a good practice to calibrate the N_{kt} value.

The CPTU is also liable to other sources of errors. Some of these include the presence of gravel, shells and wood chips, inappropriate rate of cone penetration, excessive idling time during rod connection and absence of proper calibration of the cone.

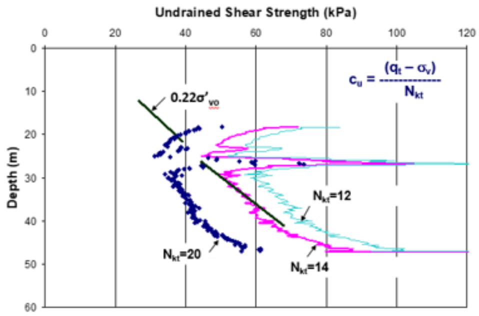


Fig. 51 Effect of N_{kt} on c_u profile

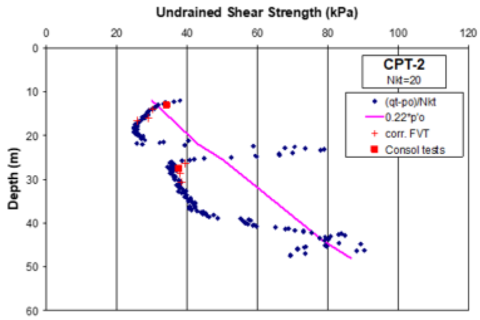


Fig. 52 Calibration of N_{kt} against other test data

3.4 The Pressuremeter Test (PMT)

The pressuremeter is commonly used in Singapore to determine the modulus of soils and rocks. Various types of pressuremeters have been used, including borehole type, push-in type and self-boring type, to determine the modulus in soft clay to stiff soils. This discussion is limited to two borehole type pressuremeters that are used in local practice. They are the OYO Elastmeter-2 and the Menard type mono-cell Pressuremeter.

The OYO Elastmeter-2 probe is pressurized by water to expand a rubber membrane. The radial displacement is measured by two diametrically opposed arms. A schematic of the OYO Elastmeter-2 probe and a typical plot of test results is shown in Figure 53. According to the manufacturer, the OYO Elasmeter-2 is for testing in stiff soil to soft rock. It should be noted that this system has only one pair of measuring arms, i.e. one pair of calipers at the middle of the measuring cell whereas BS EN ISO 22476-5:2012 specifies the use of three measuring transducers to measure the diametrical displacement.

The original Ménard system consists of three cells - a measuring cell in the middle and guard cells above and below the measuring cell. Water is injected into the middle cell causing a radial expansion. The air-filled guard cells ensure radial expansion of the measuring cell.

There is also a mono-cell system which comprises only the measuring cell without any guard cells. Two makes of the mono-cell system are available in Singapore. One is the Pressiometer manufactured by Kiso-Jiban and the other is the LLT manufactured by OYO. Figure 54 shows a schematic of the mono-cell system and a typical graphical presentation of test results.

In local practice, the most useful information obtained from PMT in soil is the unloading-reload modulus (E_{ur}) and the reloading modulus (E_r) at different strains following the recommendation by Goh et al. (2012).

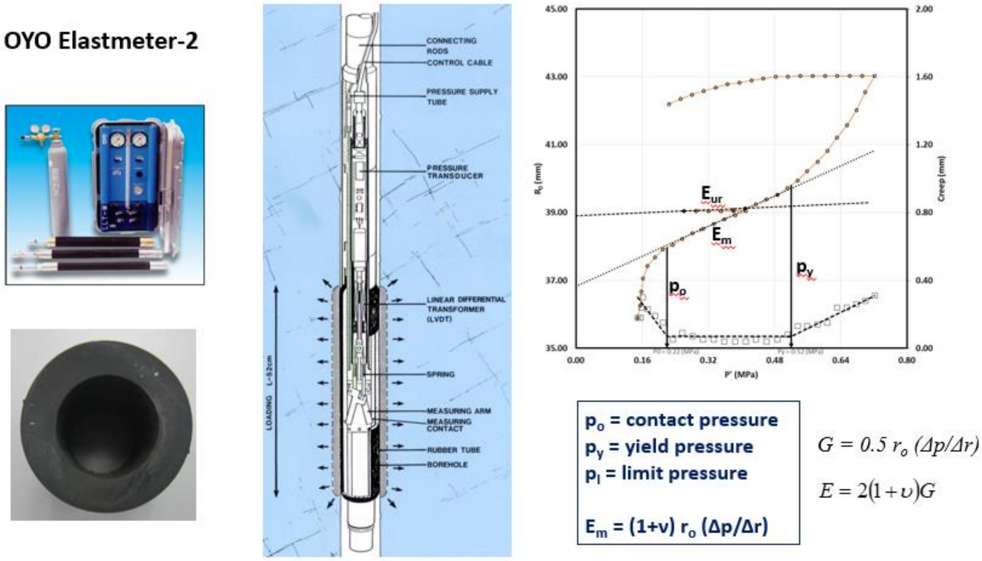


Fig. 53 OYO Elastmeter-2 and graphical presentation of typical results

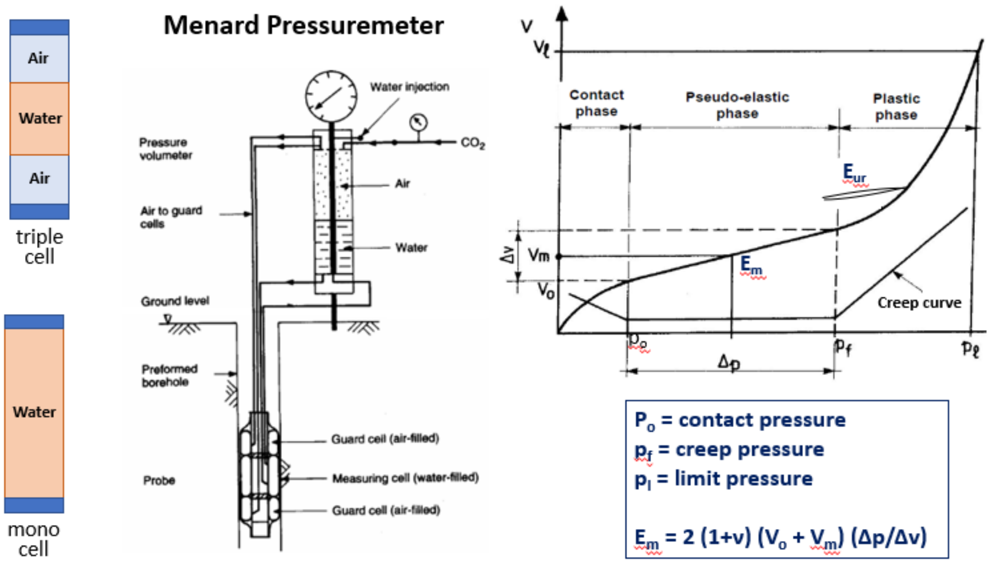
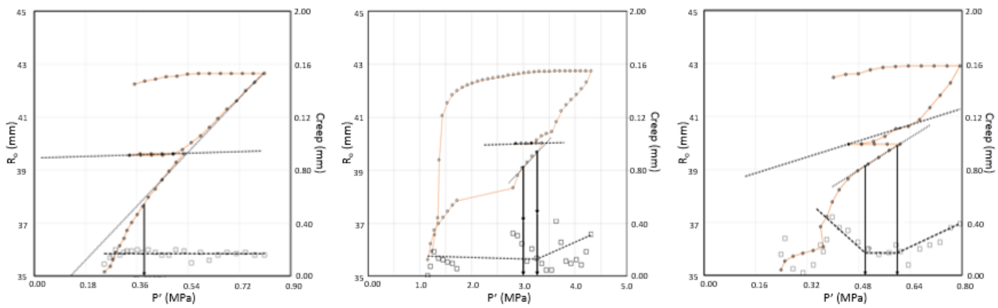


Fig. 54 Menard Type Pressuremeter and graphical presentation of typical results

For testing in soil, the Menard system (triple-cell) or its variation (mono-cell system) is preferred. This system measures the change in volume instead of the change in diameter as in the caliper system. It appears to be more sensitive for measuring the unloading-reloading modulus.

Knowledge and experience of the personnel conducting the PMT and processing the field data is important as issues in testing may not be easily detected from factual reports. Such issues could be due to the borehole being over-sized or under-sized, the in-situ soils being disturbed or softened by water during boring, partial borehole collapse, the equipment not being properly or adequately calibrated, and leakage in the hydraulic system, etc.

In some cases, test results can reflect the quality of the PMT. Figure 55a shows the results where the borehole was likely to be under-sized. In Figure 55b, the abrupt "jumps" in the pressure are due to errors in the measuring unit. For Figure 55c, the erroneous results are due to incorrect pressure measurements.



(a) Borehole too small (b) Error in measuring unit (c) Error in pressure measurements

Fig. 55 Samples of questionable results from PMT

4 CONCLUSIONS

It is important not to accept the face values of the test results presented in the laboratory report. Every test involving strength and compressibility should be checked and scrutinised.

Some suggestions for assessing the reliability of the results of various tests are discussed in this paper. Laboratory tests include consolidation, UU, CU and CD tests. Field tests include SPT, VST, CPT and PMT.

The importance of reviewing the details of test results is repeatedly emphasized. Individual tests should be checked for correctness in the interpretation, reliability in the data and applicability for the intended engineering purpose. As a whole, all test results should also be checked for outliners and compared against known trends or correlations. Cross-checking of results from different tests further verifies the consistency in the overall engineering behavior.

By the time the design engineer reviews the factual reports, little can be done to change the data presented. So, perhaps it is even more important for engineering input to be given before and during the site investigation. Such input should include allowing adequate provisions in the specifications and providing clear instructions to the driller and laboratory on what to test and how to test. If the design engineer can witness the extrusion of undisturbed samples and see and feel the soils extruded, it will provide invaluable insight in understanding the nature of the soils and carrying out the engineering design.

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