Bui Dinh Nhuan

Classification and laboratory testing of soft clay

Linköping 1981.
## CONTENTS

ACKNOWLEDGEMENTS  

1. INTRODUCTION  

2. CLASSIFICATION AND IDENTIFICATION  
   2.1 Normally consolidated clays  
   2.2 Overconsolidated clays  

3. LABORATORY TESTS  
   3.1 Determination of the liquid limit  
      3.1.1 Casagrande method  
      3.1.2 Fall-cone method  
   3.2 Oedometer tests  
      3.2.1 Incremental loading method  
         a Standard procedure (STD test)  
         b Loading procedure suggested by Bjerrum  
         c The LIN test  
      3.2.2 Interpretation of oedometer test results  
         with the incremental loading method  
         a Relative compression $\varepsilon$  
         b Determination of the preconsolidation  
            pressure $p_O$  
         c Determination of compression index  
         d Determination of the tangent modulus $M$  
         e Determination of the coefficient of  
            consolidation  
      3.2.3 Oedometer tests with continuous loading  
         a Constant rate of strain test (CRS-test)  
         b Constant gradient test (CGI-test)  
         c Continuous consolidation test (CC-test)  
   3.3 Determination of strength characteristics  
      3.3.1 Determination of undrained shear strength  
         by fall-cone tests  
      3.3.2 Influence of incorrect height adjustment  
         a Correct height adjustment (standard test)  
         b Initial penetration  
         c Initial height of fall  
      3.3.3 Shear strength in direct shear tests  
         a Generalized model for shear strength of  
            soft clay in direct shear tests  
         b Shearing rate  
         c Normal stresses
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Linköping, November 1981
Bui Dinh Nhuan
1. INTRODUCTION

In many parts of the world, large areas are covered by soft clay deposits. Civil engineers, engineering geologists and others who are concerned with the design and construction of structures have been interested in the problems of construction on deposits of soft clay.

Laboratory investigations on soft clays have been intensively developed, especially in Sweden as well as in Scandinavian countries with their extensive deposits of soft clays. Whereas in Vietnam laboratory as well as field investigations on soft clays still have limitations as to methods and equipments.

The purpose of this report is to collect and summarize some Swedish methods and experiences in the laboratory investigation on soft clays. The classification and identification and problem of design parameters of soft clays are also collected.
2. CLASSIFICATION AND IDENTIFICATION

Recent research has shown that soft clays have their particular characteristics and the classification and identification of these clays should be based on their engineering properties. The following information on which the classification and identification on soft clay may be based is:

- the geological history (stress history) of the deposit
- the water content and the Atterberg limits
- the strength properties: vane shear strength
- the deformation properties: the compressibility characteristics determined from oedometer tests.

Based on the above information, Bjerrum (1973) proposed that soft clay can be classified into the following main groups:

1. Normally consolidated clays
   - normally consolidated young clays
   - normally consolidated aged clays

2. Overconsolidated clays

3. Weathered clays

4. Quick clays

5. Cemented clays.

Two groups (normally and overconsolidated clays) are briefly presented below.

2.1 Normally consolidated clays

The normally consolidated clays can be "young" or "aged". The difference between these clays is
shown in Table 1.

Table 1. Characteristics of "young" and "aged" normally consolidated clays.

<table>
<thead>
<tr>
<th>Normally consolidated young clays</th>
<th>Normally consolidated aged clays</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay which has recently been deposited.</td>
<td>Young clay under constant effective stress for long time (hundreds or thousands of years). Without significant settlement under $\sigma'_0 + \Delta \sigma$ ($\Delta \sigma$ is definite value).</td>
</tr>
<tr>
<td>Large settlement under $\sigma'_0 + \Delta \sigma$</td>
<td>Greater strength and small compressibility</td>
</tr>
<tr>
<td>Small strength and greater compressibility</td>
<td>Greater strength and small compressibility</td>
</tr>
<tr>
<td>$\sigma'_0 \leq \sigma'_0$ (from $e$-$\log \sigma'$ curve)</td>
<td>$\sigma'_0 &lt; \sigma'_0$ (from $e$-$\log \sigma'$ curve)</td>
</tr>
<tr>
<td>Overconsolidation ratio $\sigma'_0/\sigma'_0 = 1$</td>
<td>Overconsolidation ratio $\sigma'_0/\sigma'_0 &gt; 1$ and increases with the plasticity index $I_p$.</td>
</tr>
<tr>
<td>$\tau_V$ increases linearly with $\sigma'_0$ smaller ratio $\tau_V/\sigma'_0$</td>
<td>$\tau_V$ increases linearly with $\sigma'_0$ greater ratio $\tau_V/\sigma'_0$</td>
</tr>
</tbody>
</table>

Fig. 1 shows the difference in the geological history and compressibility of a "young" and an "aged" normally consolidated clay according to Bjerrum (1973) based on the $e$-$\log \sigma'$ curve from consolidation test.

![Fig. 1. Geological history and compressibility of a "young" and an "aged" normally consolidated clay.](image-url)
The ratio of the vane shear strength \( \tau_v \) to the effective overburden pressure \( \sigma'_o \) as well as the ratio of the preconsolidation pressure to the effective overburden pressure of both "young" and "aged" normally consolidated clays depend on their plasticity index \( I_p \). Fig. 2 shows the correlation between the \( \tau_v/\sigma'_o \)- and \( \sigma'_o/\sigma'_o \)-values (in figure \( s_u/p_o \) and \( p_c/p_o \)) and the plasticity index \( I_p \).

From Table 1 and Figs. 1 and 2 it is clear that by using some engineering properties we can easily distinguish the "young" normally consolidated clay from the "aged" one.
2.2 Overconsolidated clays

The overconsolidated clays are clays whose present-effective overburden pressure is less than a maximum previous effective pressure under which the clays once were consolidated. Overconsolidation is the result of one of the following causes:

- surface erosion
- decrease in pore water pressure during a certain time in the history of clays
- excavation
- variation in groundwater level.

For these clays the ratio of the maximum previous effective pressure (often called the preconsolidation pressure) to the present effective overburden pressure is used to determine the degree of overconsolidation. This ratio is called the overconsolidation ratio and is expressed by the formula:

\[
\text{overconsolidation ratio} = \frac{\text{preconsolidation pressure}}{\text{present overburden pressure}} = \frac{\sigma'_o}{\sigma'_o}
\]

If clays are only considered normally consolidated and overconsolidated, it is clear that for normally consolidated clays the overconsolidation ratio is unity and for overconsolidated clays it is greater than unity. Depending on this ratio the clays of this group may be lightly or heavily overconsolidated.

A difference between the two groups of clays is that, under the same additional load to the present overburden pressure, the normally consolidated clays will settle more than the overconsolidated clays.
3. LABORATORY TESTS

3.1 Determination of the liquid limit

In Sweden the liquid limit is determined by the fall-cone method or the percussion method. The fall-cone method is the most common method.

3.1.1 Casagrande method

The percussion method (Casagrande method) is based on the specifications of the American Society of Testing Materials (ASTM). The one-point method proposed by the Waterways Experiment Station et al (1949) is normally used. However, this method cannot be used on soils with a liquid limit larger than 150 (Broms, 1981). The liquid limit \( w_L \) in this method is calculated by the equation

\[ w_L = \omega \left( \frac{n}{25} \right)^{tg\beta} \]

where
\( \omega = \) water content
\( n = \) number of blows required to close a groove made by a special tool for a length of 13 m
\( \beta = \) inclination of the flow curve

3.1.2 Fall-cone method

In this method the liquid limit is defined as the water content at which a 60 g/60°-cone gives a penetration of 10 mm for a completely remoulded sample (Geotechnical Commission of the Swedish State Railway, 1914-1922).

As defined, the test for determination of the liquid limit should be repeated several times at different water contents. The determined liquid limit is then the water content of soil when the penetration of the cone is 10 mm. However, this test procedure is time-consuming.
To make the test less time-consuming different one-point methods have been developed. Nowadays a one-point method proposed by the Swedish Geotechnical Institute (Karlsson, 1961) is normally used in Sweden. This method is based on investigations of different Swedish soils and also certain soils from abroad.

The relation between the strength parameter $m/i^2$ (where $m =$ mass of cone, $i =$ cone penetration) according to Hansbo (1957) ($f_u = \kappa \cdot g \cdot m/i^2$) and the water content was plotted in a semi-logarithmic graph. The relation was called the consistency curve and corresponds to Casagrande's flow curve (Fig. 3).

The inclination at $w_L$ can be expressed by

$$tg = \frac{w_L - w_0}{\lg 6 - \lg 0.6} = w_L - w_0$$

The inclination of $w_L$ can be expressed by:

$$tg \alpha = \frac{W_L - W_0}{\lg 6 - \lg 0.6} = W_L - W_0$$

Fig. 3. Consistency curve. Definition of the inclination at $w_L$. 
Within a limited region around the liquid limit the curve can be approximated to a straight line with the following equation:

\[ \omega_L = \omega_i + tga \cdot \log \left( \frac{10}{i} \right)^2 \]

where

\( \omega_i = \) water content at cone penetration \( i \)
\( tga = \) inclination of the consistency curve at the liquid limit

The investigations showed that the value of \( tga \) is dependent on \( \omega_L \) and generally increases linearly with \( \omega_i \).

\[ tga = \frac{\omega_L - 17}{1.8} \]

The following formula can thus be derived

\[ \omega_L = M \cdot \omega_i + N \]

where

\[ M = \frac{1.8}{1.8 + 2 \log \frac{i}{10}} \]
\[ N = \frac{34 \log 10}{1.8 + 2 \log \frac{i}{10}} \]

where

\( \omega_L = \) liquid limit
\( \omega_i = \) water content of remoulded sample at the cone penetration \( i \)
\( M, N = \) correction factors

The evaluation of \( \omega_L \) by the one-point method is illustrated in Fig. 4.
Fig. 4. Evaluation of $\omega_L$ by the one-point method.

Compared with the Casagrande method the cone method is preferred because:

- the test is simple and fast
- the results are more consistent and less liable to experimental and personal errors
- the results depend more directly on the shear strength of the soil.

SGI has determined the shear strength at the Casagrande liquid limit and at the cone liquid limit for different soils by means of a laboratory vane apparatus. The results showed that the strength at the Casagrande liquid limit varied considerably between different soils (0.5-4 kPa) whereas the strength at the cone liquid limit was about the same for all samples.
The cone method is fundamentally more satisfactory because the mechanics of the test depend more directly on the shear strength of the soil. The Casagrande procedure introduces a dynamic component which is not related to shear strength in the same way for all soils. The number of precussions required to make the halves of the sample to flow together is besides the shear strength also dependent on the density of the sample.

Table 2. The shear strength of soil at $\omega_l$ and $\omega_p$ (Karlsson, 1962).

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Liquid limit</th>
<th>$T_{fu}$ (lab. vane test at liquid limit)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cone</td>
<td>Casagrande</td>
</tr>
<tr>
<td>Postglacial clay</td>
<td>62</td>
<td>70</td>
</tr>
<tr>
<td>Mud</td>
<td>215</td>
<td>275</td>
</tr>
<tr>
<td>Bentonite</td>
<td>170</td>
<td>320</td>
</tr>
<tr>
<td>Kaoline I</td>
<td>56</td>
<td>53</td>
</tr>
<tr>
<td>Kaoline II</td>
<td>43</td>
<td>45</td>
</tr>
<tr>
<td>Coarse silt with some organic matter</td>
<td>34</td>
<td>30</td>
</tr>
</tbody>
</table>

SGI also has made an investigation in order to find out the reliability of routine determinations of the cone liquid limit and the Casagrande liquid limit (Karlsson et al, 1974).

The investigation comprised two different soils, a high-plastic clay and a low-plastic, somewhat silty, clay. The determinations were performed by 21 laboratories at different institutions and consulting firms. The results showed that the scatter was considerably smaller for the cone liquid limit, particularly for the high-plastic clay.
An other investigation by Sherwood and Ryley et al (1968) has also shown that results obtained by the cone method are more consistent and less liable to experimental and personal errors than those obtained by the Casagrande method.

The comparison between liquid limit determined by the cone method and by the Casagrande method for Swedish soils was worked out by Karlsson et al (1974). The results showed that for clays the Casagrande and cone liquid limit coincide when $w_L = 40\%$. At higher values the Casagrande liquid limit is generally higher than the cone liquid limit and at lower values the opposite is valid. For silt the Casagrande liquid limit is generally considerably lower than the cone liquid limit and for organic soils considerably higher.

For soft clay the liquid limit as well as the plastic limit is in Sweden normally determined on natural samples (samples which have not been dried in advance). Soils that are dried and sieved before determination are normally used internationally. According to Broms (1981) the drying of a sample in an oven can reduce the liquid and plastic limits especially if the soil is organic as illustrated in Fig. 5.
Fig. 5. Comparison of the cone liquid limit for dried and wet samples.

In routine tests the liquid limit is usually determined on samples which previously have been used to determine the shear strength. If the water content of the soil is too low, water should be added to the samples. In order to reduce the water content when it is too high, the sample is spread or rolled out on a gypsum plate. It is necessary to note that the time for cone penetration in clay and in silty soils is different. In clay soil the cone stops to penetrate into the soil a few seconds after the cone is released and that is enough for reading. In silty soil the cone often does not stop but continues to penetrate into the soil. In this case the penetration is taken about 10 seconds after the cone is released (Karlsson (1977), Broms (1981)).
Nowadays a one-point method for determination of the fall-cone liquid limit is generally made. The relation between cone penetration $i$ (60 g/60°) and factors $M$ and $N$ in the formula $\omega_L = M \cdot \omega_i + N$ is shown in Table 3.

Table 3. Relation between cone penetration $i$ (60 g/60°) and factors $M$ and $N$ in the formula $\omega_L = M \cdot \omega_i + N$.

<table>
<thead>
<tr>
<th>$i$ (mm)</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>$N$</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

3.2 **Oedometer tests**

Compression characteristics of soft clays are generally determined by oedometer tests. There are some different methods for oedometer tests:

- oedometer test with incremental loading
- constant rate of strain tests (CRS-tests)
- constant gradient tests (CGT-tests)
- continuous consolidation tests (CC-tests)
3.2.1 Incremental loading test

This method was suggested by Terzaghi in 1925 and has been widely used since then. In this method the test procedure is performed by incremental loading, each increment equal to the previous load and new increment loaded every 24 hours. During the test the sample is drained from the ends and readings of the compression are taken in a time sequence enabling a plot of the time-settlement curve for each increment. The oedometer test with incremental loading with a duration of 24 hours is considered standard.

However, this test procedure has its disadvantage because it takes a long time, at least a week for one sample. Therefore different variants of test procedure have been suggested.

The apparatus used for incremental loading test is shown in Fig. 6. Fig. 7 shows the cutting device used for mounting clay samples.

Fig. 6. Apparatus used for incremental oedometer tests.
Fig. 7. Cutting device used for mounting clay samples.

The oedometer ring is 40 mm in diameter. This size of oedometer ring is suitable to the 50 mm diameter sampling tube. The tested sample is 20 mm in height.

In oedometer test with incremental loading three test procedures have been used:

a) Standard procedure (STD test): daily load increments, each increment is equal to the previous load and a new increment is loaded every 24 hours. The following increments have often been used for the STD tests: 10, 20, 40, 80, 160 and 320 kPa. The time required for a STD test is at least 6 days.

b) Loading procedure suggested by Bjerrum (1973): for vertical pressures below the preconsolidation pressure the load increments are reduced and new increments are loaded at the end of primary consolidation (100% consolidation). Above the preconsolidation pressure the test is continued with doubled load increments with 24 hours' duration.
The time required for a test will be 3 to 4 days because the first small increments can usually be completed during one working day. According to Sällfors (1975) this method is called the NGI-test.

c) Tests with daily load increments; equal increments usually 10 or 20 kPa each with a duration of 24 hours. This test is a LIN test (Sällfors, 1975). The LIN test takes 8 to 12 days depending on the preconsolidation pressure.

3.2.2 Interpretation of oedometer test results with the incremental loading method

a) Relative compression $\epsilon$.

The results from incremental oedometer tests performed by the STD or NGI procedure are presented in a diagram as a stress-strain curve. In this plot the vertical effective pressure is in log-scale (Fig. 8). From this diagram the relative compression $\epsilon$ between the vertical in situ pressure in ground $\sigma'_0$ and the calculated final pressure $\sigma'$ can be determined and therefore the settlement is calculated by the following formula:

$$\delta H = \epsilon \cdot H$$

where $H =$ thickness of the soil layer.

The stress-strain curves from LIN-tests are presented in linear scales.

b) Determination of the preconsolidation pressure $\sigma'_0$.

The preconsolidation pressure can be determined from the oedometer curve obtained in STD-tests according to the Casagrande method. This method has been widely
used. Fig. 9 shows the Casagrande method for determining the preconsolidation pressure. In this method, the vertical pressure is in log-scale and the relative compression is in linear scale. At the point with the smallest radius of curvature, a tangent to the oedometer curve and a horizontal line are drawn. The angle between these two lines is bisected. Then the straight portion of the oedometer curve is drawn and extended so that it intersects the bisectrix. The pressure at this intersection is the preconsolidation pressure $\sigma_0'$.

Due to disturbance of samples, the evaluated preconsolidation pressure is often too low. Therefore the disturbance should be taken into account when determining the preconsolidation pressure. The determined preconsolidation pressure according to many authors is sensitive to the loading sequence and the duration of each load step.

In LIN-tests the preconsolidation pressure is determined as the intersection of the extended straight portions (before and after $\sigma_0'$) of the curve.

The preconsolidation pressure is often determined from a stress-strain curve with stress in log-scale and strain in linear scale. This strain-log stress curve is suitable for determination of the preconsolidation pressure of normal soft clays. In this case the oedometer curve makes a sharp break and makes the determination of the preconsolidation pressure rather easy, curve 1 in Fig. 10 (B). For some soft clays though, for example clays with a high swelling capacity and relatively high compression
modulus below the preconsolidation pressure, this strain-log stress curve is disadvantageous. On one hand due to swelling characteristics (clays more or less overconsolidated have swollen in the ground) and on the other hand due to disturbance during sampling, most clays brought into the laboratory have undergone some swelling. In this case the strain-log stress curve will give a shape in a regular bend for stresses below and just after the preconsolidation pressure, curve 2 in Fig. 10 (B). This shape of the oedometer curve makes the determination of the preconsolidation pressure difficult because it is difficult to find the smallest radius of curvature. In this case the oedometer curve for soft clays should be plotted in linear scales (both for stress and strain), Fig. 10 (A).

Fig. 10. Oedometer curves in (A) - linear scale and (B) - semilog scale for soft clays.
1 - normal soft clay
2 - clay with a high swelling capacity

c) Determination of compression index

The compression indices $C_c$ and $\varepsilon_2$ are also evaluated from the oedometer curve. There is a difference in
the determination of \( C_c \)- and \( \varepsilon_2 \)-value.

The compression index \( C_c \) is evaluated from an oedometer curve plotted in a void ratio-vertical pressure relationship (Fig. 11).

To avoid the determination of the void ratio, the compression index \( \varepsilon_2 \) is used. The compression index \( \varepsilon_2 \) is evaluated from an oedometer curve plotted in a relative compression-vertical pressure relationship (Fig. 11). In both cases, the vertical pressure is plotted in log-scale.

The straight line of the oedometer curve after the preconsolidation pressure is chosen for evaluation of the compression indices \( C_c \) and \( \varepsilon_2 \) (see Fig. 11).

The compression index \( C_c \) is determined by the following equations:

![Fig. 11. a - Evaluation of compression index \( C_c \). b - Evaluation of compression index \( \varepsilon_2 \).](image-url)
The compression index $c_2$ is used in Sweden, where $c_2$ is the relative compression of a sample at a doubling of the vertical pressure (Fig. 11). The relation between these compression indices is:

$$\frac{c_2}{\log 2} = C_C (1 + e_o)$$

d) Determination of the tangent modulus $M$

The determination of the compression indices $C_C$ and $c_2$ is performed with the assumption that the oedometer curve should be a straight line for stresses higher than the preconsolidation pressure. For some clays, for example for Swedish clays, this assumption is not valid and this method for determining the compressibility ($C_C$ and $c_2$) is not suitable since the method is only valid within a small stress range. Therefore another method for determining the compressibility has been suggested.

Soil compressibility is often expressed by a tangent modulus $M$ (Odhe (1951), Janbu (1967), Brinch-Hanssen (1966) and others). The tangent modulus $M$ is expressed by the following equation:

$$M = m_j \sigma_j^\beta (\frac{\sigma'}{\sigma_j})^{1-\beta}$$

where

- $m_j = \text{modulus number}$
- $\beta = \text{stress exponent}$
- $\sigma'$ = effective vertical stress
- $\sigma_j$ = reference stress (usually 100 kPa)

In this case for calculation of the tangent modulus $M$, it is necessary to determine the modulus number $m_j$ and the stress exponent $\beta$. These two parameters
can be evaluated from the oedometer curve. The oedometer curve is plotted in a diagram with the strain in linear scale and the vertical pressure in log-scale (Fig. 12).

**Fig. 12. Evaluation of** $m_j$ **and** $\beta$.

The $m_j$- and $\beta$-values are evaluated by drawing a tangent to the stress-strain curve at $\sigma_j$ and extending it to $2.7 \sigma_j$ where $\sigma_j'$ is a reference stress ($\sigma_j' > \sigma_j$).

If the stress exponent $\beta$ is equal to 0 ($\beta = 0$) the oedometer curve is a straight line overlapping the tangent to the curve at $\sigma_j'$. The relative compression $\Delta \varepsilon_1$ is evaluated from the intersections of the vertical lines through $\sigma_j'$ and $2.7 \sigma_j'$ with the tangent to the oedometer curve. Thus the modulus number $m_j$ is calculated from the equation

$$\Delta \varepsilon_1 = \frac{1}{m_j}$$
If the oedometer curve after the preconsolidation pressure $\sigma'_d$ is not really straight but inflected as seen in Fig. 12 the real compression $\Delta\varepsilon_{z,\gamma}$ between vertical pressures $\sigma'_d$ and $2.7\sigma'_d$ is evaluated. This occurs in the case with $\beta \neq 0$ and the stress exponent $\beta$ is calculated from the following equation:

$$\Delta\varepsilon_{z,\gamma} = \frac{1}{m_j \beta} (2.7^\beta - 1)$$

This method of describing compressibility of soft clays is not correct either but the approximation can be used for a larger stress interval than the compression index.

e) Determination of the coefficient of consolidation

The coefficient of consolidation $c_v$ is commonly used to predict the rates at which settlement will occur. The $c_v$-value can be determined from the oedometer curve by the Casagrande or the Taylor method. Both methods are derived from the Terzaghi theory:

$$\frac{\delta u}{\delta t} = c_v \frac{\delta^2 u}{\delta z^2}$$

where

- $u$ = excess pore water pressure
- $t$ = time elapsed since loading
- $c_v$ = coefficient of consolidation

and the $c_v$-value can be calculated by the equation:

$$c_v = \frac{K}{\gamma_w m_v} \text{ m}^2/\text{year}$$

$\gamma_w$ = unit weight of water (kN/m$^2$)
$K$ = vertical coefficient of permeability of the soil (m/yr)
\( m_v = \text{coefficient of volume compressibility (volume change, volume decrease)} \ (\text{m}^2/\text{kN}) \)

The determination of the coefficient of consolidation from the oedometer curve is based on the two following quantities:

Time factor \( T_v \) is calculated by the formula:

\[ T_v = \text{Tersaghi time factor} = \frac{c_v t}{d^2} \]

where \( d \) is the drainage path length. In the laboratory \( d = \) sample thickness with "one-way" drainage and \( d = \) half of sample thickness with "two-way" drainage. In the calculation, \( T_v \)-value is dimensionless.

Average degree of consolidation \( U_v \):

The average degree of consolidation is the ratio of the settlement at a definite time, \( t \), to the ultimate settlement and is expressed by the equation:

\[ U_v = \frac{S_t}{S_{ult}} = \frac{\text{settlement at } t}{\text{ultimate settlement}} \]

The \( U_v-T_v \) relationship is founded as follows:

<table>
<thead>
<tr>
<th>( U_v ) (%)</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_v )</td>
<td>0.008</td>
<td>0.031</td>
<td>0.071</td>
<td>0.126</td>
<td>0.197</td>
<td>0.287</td>
<td>0.403</td>
<td>0.567</td>
<td>0.848</td>
</tr>
</tbody>
</table>

The coefficient of consolidation \( c_v \) can be evaluated by one of the two following methods:
\[ c_v = \text{coefficient of volume compressibility (volume change, volume decrease)} \ (m^2/kN) \]

The determination of the coefficient of consolidation from the oedometer curve is based on the two following quantities:

**Time factor** \( T_v \) is calculated by the formula:

\[ T_v = \text{Terzaghi time factor} = \frac{c_v \cdot t}{d^2} \]

where \( d \) is the drainage path length. In the laboratory \( d = \text{sample thickness with "one-way" drainage} \) and \( d = \text{half of sample thickness with "two-way" drainage} \). In the calculation, \( T_v \)-value is dimensionless.

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The average degree of consolidation is the ratio of the settlement at a definite time, \( t \), to the ultimate settlement and is expressed by the equation:

\[ U_v = \frac{S_t}{S_{ult}} = \frac{\text{settlement at } t}{\text{ultimate settlement}} \]

The \( U_v-T_v \) relationship is founded as follows:

<table>
<thead>
<tr>
<th>( U_v(%) )</th>
<th>10</th>
<th>20</th>
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<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_v )</td>
<td>0.008</td>
<td>0.031</td>
<td>0.071</td>
<td>0.126</td>
<td>0.197</td>
<td>0.287</td>
<td>0.403</td>
<td>0.567</td>
<td>0.848</td>
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</table>

The coefficient of consolidation \( c_v \) can be evaluated by one of the two following methods:
In the Casagrande method, the coefficient of consolidation $c_v$ is determined from the time-strain curve with strain in a linear scale and time in log scale (Fig. 13). In Fig. 13 is seen how $\bar{U}_0$ and $\bar{U}_{100}$ are constructed. $\bar{U}_{100} = 100\%$ is constructed as the intersection between the tangent to the curve at its point of inflexion and the extension of the straight end part of the curve. From $\bar{U}_0$ and $\bar{U}_{100}$, $\epsilon_{50}$ at $\bar{U}_{50} = 50\%$ is calculated and $t_{50}$ is constructed. Finally the coefficient of consolidation $c_v$ is calculated by the following formula:

$$c_v = T_{50} \frac{d^2}{t_{50}}$$

where

- $d$ = length of drainage path
- $T_{50}$ = Terzaghi time factor

For oedometers with samples drained at both ends and $d = H_0(1-\epsilon_{50})/2$ where $H_0$ is initial sample height, the time factor $T_{50} = 0.197$. Thus the coefficient of consolidation $c_v$ is calculated as:
Taylor method

As in the Casagrande method, the coefficient of consolidation $c_v$ in the Taylor method is also determined from a time-strain curve with strain in linear scale but with time in square root scale (Fig. 14). As seen in Fig. 14 $U_0 (\varepsilon_0)$ is determined as the beginning point of the curve. At $U_0 (\varepsilon_0)$ a tangent to the curve is drawn. A free horizontal line $z$ is drawn that intersects the tangent of the curve at a certain point. The distance 0.15 $z$ is calculated and the line A is constructed and $U_{90} (\varepsilon_{90})$ is taken from the intersection between the line A and the curve. Now the parameter $U_{50} (\varepsilon_{50})$, $U_{100} (\varepsilon_{100})$ and $t_{90}$ can be constructed and calculated.

The coefficient of consolidation $c_v$ is determined by the following formula:

$$c_v = rac{d^2}{t_{90}}$$
For oedometer with samples drained from both ends
\[ d = \frac{H_0(1-\epsilon_0)}{2} \]
where \( H_0 \) is the initial sample height, and the time factor \( t_{90} = 0.848 \). In this case the coefficient of consolidation is calculated as:

\[ c_v = 0.848 \frac{d^2}{t_{90}} \]

The coefficient of consolidation \( c_v \) is determined for every load step. According to Bjerrum, for small load increments up to \( \sigma_0 \) the \( c_v \)-value can be determined by the Taylor method and for a load exceeding \( \sigma_0 \) both the Casagrande and the Taylor method can be used. The \( c_v \)-values calculated by the above methods should then be plotted against the effective vertical stress, see Fig. 15.

![Fig. 15. The coefficient of consolidation observed in a consolidation test plotted against the vertical load (after Janbu, 1969).]

As seen in Fig. 15, the range of the \( c_v \)-variation is considerable. Therefore, the \( c_v \)-value to be applied on a practical problem has to be chosen in the appropriate stress range (Bjerrum, 1973).
It has been found that the value of the coefficient of consolidation is affected by temperature (Bjerrum (1973), Larsson (1981), and others). Therefore, in order to determine an accurate $c_v$-value the test should be performed at constant temperature in a temperature-controlled room, if possible at the same temperature as that of the in situ soil.

As seen above, both methods are based on types of settlement. There are three types of settlement that are usually termed (Fig. 16):

- immediate settlement (compression)
- primary settlement (compression)
- secondary settlement (compression).

![Fig. 16. The three parts of the time-settlement curve.](image)

As time-settlement curves are plotted in different scales, the identification of these three characteristic settlements in both methods is different.

In the Taylor method, the point where the primary compression is thought to begin is obtained by extending the tangent to the curve back to the compression axis ($U_0$, $\epsilon_0$) assuming that the immediate settlement occurs fairly rapidly and is usually
neglected. After the point of $U_{90}$ is determined, the point where the primary settlement is assumed to finish ($U_{100}$) is determined on the compression axis by the relative compression at $U_{100}$ ($\varepsilon_{100}$), see Fig. 17.

$$\varepsilon_{100} = \frac{\varepsilon_{20}}{0.9}$$

Any settlement below this $\varepsilon_{100}$ is considered as secondary settlement.

In the Casagrande method, the determination of the point where the primary settlement begins is based on the assumption that, in the early stages of consolidation, the time is proportional to the square of the average degree of consolidation ($t_v = f(\bar{\varepsilon})^2$) and therefore in the early stages we have

$$\frac{t_1}{t_2} = \left(\frac{\varepsilon_1}{\varepsilon_2}\right)^2$$

If $\varepsilon_2 = 2\varepsilon_1$

then $$\frac{t_1}{t_2} = \left(\frac{1}{2}\right)^2$$

or $$t_2 = 4t_1$$

The settlement between $t_1$ and $t_2$ ($t_2 = 4t_1$) = $\varepsilon_2 - \varepsilon_1 = d$ (in Fig. 18), because $\varepsilon_2 = 2\varepsilon_1$ so $\varepsilon_1 = d$ and with this assumption $U_0(\varepsilon_0)$ is determined. The point where the primary settlement is assumed to finish is obtained by the help of the coefficient of secondary compression $\alpha_8$ (see Fig. 18).
Fig. 17. Identification of types of settlement: Taylor method.

Fig. 18. Identification of types of settlement: Casagrande method.
3.2.3 Oedometer tests with continuous loading

Oedometer tests with continuous loading have been developed during the last fifteen years. Compared with the traditional incremental loading test, the oedometer tests with continuous loading have three advantages:

- they give continuous stress-strain relations
- they give continuous $c_y$-stress relations
- they can be run automatically.

For this method, the following tests have been performed:

- constant rate of strain tests (CRS-tests)
- constant gradient tests (CGI-tests)
- continuous consolidation tests (CC-tests).

a) Constant rate of strain test (CRS-test)

In the CRS-test the sample is compressed at a constant rate. The sample is drained at the upper end and sealed at the bottom where the pore pressure is measured. During the test, the compressive force, the deformation, the pore pressure at the bottom and time are automatically recorded continuously.

Besides parameters of compressibility obtained from oedometer tests with incremental loading, CRS-tests give the following continuous relations:

- effective stress and strain
- modulus and effective stress
- coefficient of consolidation and effective stress
- permeability and strain.

During 1971-1975 a large investigation was carried out on comparisons between different oedometer tests and between oedometer tests and field observations. This investigation led to the recommendation
of the CRS-test as a routine test for soft clays and this method became a standard test at the Swedish Geotechnical Institute in 1975 and has also been used in many Swedish consulting firms.

b) Constant gradient test (CGI-test)

The constant gradient test is performed with constant pore pressure. In the CGI-test the strain rate should be regulated so that the pore pressure in bottom of the sample is kept constant. Due to test condition, the CGI-test is more complicated and slower than the CRS-test.

c) Continuous consolidation test (CC-test)

This test has mainly been developed at the Norwegian Institute of Technology. The CC-test is performed with a constant relation between the applied load and the pore pressure in the bottom of the sample. It requires the most complicated equipment. According to the Swedish point of view, if compared to the CRS-test, the main advantage of the CC-test is that it can automatically adjust the rate of strain to the tested sample. Norwegian experience found that for low plastic Norwegian clay the CC-test could be performed much faster than the CRS-test.

3.3 Determination of strength characteristics

In Sweden the undrained shear strength of soft clay has commonly been determined by laboratory fall-cone test (Fig. 19). Besides this test, the shear strength of soft clay can be determined by the direct shear test or the triaxial test.
The fall-cone test was developed by the Geotechnical Commission of the Swedish State Railway between 1914 and 1922 and has been widely used in Sweden since then. It is a simple and rapid method for determining the undrained shear strength of both undisturbed and remoulded clays.

In a test the cone is usually placed in the stand of the apparatus in such a way that the tip of the cone just touches the surface of the soil sample. The cone is then dropped freely into the soil and the depth of penetration measured.

Different cones have been used and nowadays the following cones are standard for different range of the shear strength:

- 400 g 30°
- 100 g 30°
- 60 g 60°
- 10 g 60°
3.3.1 Determination of undrained shear strength by fall-cone tests

The cones 60° and 100° are often used today. The 60°-cone was chosen as unit cone and the relative strength number for 10 mm penetration with this cone was set = 10. The strength number for a completely remoulded sample was indicated by \( H_1 \), and for a partly disturbed by \( H_2 \) and for an undisturbed by \( H_3 \). Comparisons with direct shear tests and landslides have resulted in the following relation between the undrained shear strength \( \tau_{fu} \) in \( \text{kN/m}^2 \) and the strength value \( H_3 \).

\[
\tau_{fu} = 10H_3(38+0.064H_3) \quad \text{(SGI)}
\]

The SGI relation is a mean value of the two relations mentioned above (Skaven-Haug's and Hultin's).

The evaluation of undrained shear strength is nowadays often made according to Hansbo (1957).
\[ \tau_{fu} = k \cdot g \cdot \frac{m}{a} \left(1 + \frac{g}{a} \right) \]  
(Hansbo, 1957)

where

\( \tau_{fu} \) = undrained shear strength, kPa
\( k \) = constant (primarily depending on the cone angle)
\( g \) = 9.81 m/s\(^2\)
\( m \) = mass of cone, g
\( i \) = cone penetration, mm
\( a \) = free height of fall, mm

In Hansbo's formula the value of \( k \) depends primarily on the cone angle. The evaluation of \( k \) has been made by calibration against results from field vane tests for undisturbed clays and from laboratory vane tests for remoulded clays. Fig. 20 present the \( k \)-value for Swedish clays taken with standardized piston sampler (Hansbo, 1957).

![Fig. 20. \( k \)-value for Swedish clays taken with standardized piston sampler (Hansbo, 1957).](image-url)

According to Karlsson (1962), \( k \)-values for remoulded soils by calibration from vane tests in laboratory are evaluated as follows.
The recommended values for $k$ in Sweden are $k = 0.25$ for cone angle $30^\circ$ and $k = 1.0$ for cone angle $60^\circ$.

### 3.3.2 Influence of incorrect height adjustment

In a test the cone should be placed in the stand of the apparatus in such a way that the tip of the cone just touches the surface of the soil sample. The tests are easy and simple but it is very important to make the correct height adjustment, because it is a main source of error of the fall-cone method.

Any incorrect height adjustment can be corrected in the tests. There are three cases of height adjustment.

**a) Correct height adjustment (standard test)**

The undrained shear strength in standard test is calculated according to the formula:

$$\tau_f u = k \cdot g \cdot \frac{m}{i_i}$$

where $i = \text{correct cone penetration}$.

**b) Initial penetration**

$$\tau_f u = k \cdot g \cdot \frac{m}{i_0^2 + 3i_0i_2 + 3i_2^2}$$

$i_0 = \text{initial penetration}$  
$i_2 = \text{determined cone penetration}$  
$i_3 = i + \Delta i$
c) Initial height of fall

\[ \tau_{fu} = k \cdot g \frac{m}{\Delta i} (1 + \frac{a}{i_1}) \]

where

- \( a \) = initial height of fall
- \( a + i_1 \) = determined cone penetration
- \( a + \Delta i \) = \( i + \Delta i \)

As seen above, the most important part of the fall-cone method is the height adjustment (height of fall of the cone). According to Broms (1982) a deviation of only 0.3 mm can lead to an error of about 2 to 3% with respect to the shear strength of the soil when the water content is 100% and the penetration is 7 mm.

In order to obtain the \( \tau_{fu} \)-value quickly, a table has been prepared (Table 4). The prepared table is applicable for different cones with different penetration (different range of shear strength). By choosing a suitable cone, the fall-cone test can be used to determine the undrained shear strength in a range of 0.060 to 95 kPa.
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Table 4.
3.3.3 Shear strength in direct shear tests

In Sweden the direct shear tests are often used to determine the shear strength by a modified SGI oedometer (Fig. 21). In this apparatus the sample in the tests is first consolidated and then sheared to failure. The drained or undrained shear strength can be determined in direct shear tests. The test sample has a diameter of 50 mm and a height of 10 mm and 20 mm in drained and undrained tests respectively. During the tests horizontal stress, horizontal deformation and vertical deformation are measured.

Fig. 21. The SGI shear apparatus.

From the test results the shear stress versus angular distortion is plotted for every vertical stress as seen in Fig. 22.
Fig. 22. Shear stress, angular distortion and vertical deformation in consolidated drained direct shear tests (a) and in consolidated undrained direct shear tests (b).

The shear stress at failure is evaluated as a peak shear stress. In the case no peak the shear stress at failure is evaluated according to Swedish practice as the shear stress at an angular distortion of 0.15 radians.

The shear strength obtained from direct shear tests is plotted as in Fig. 23.
Research and experiments have shown that for soft clays, the relation between shear strength and vertical stress changes at the preconsolidation pressure in both undrained and drained tests. It has also been found that there is a second breaking point at half of the preconsolidation pressure in the relation between drained shear strength and vertical stress ($T_d - c'$ curve) as seen in Fig. 23.

a) Generalized model for shear strength of soft clay in direct shear tests

The shear strength from direct shear tests can generally be expressed by the equation:

$$\tau = c' \tan \phi'$$

where

- $c'$ = vertical effective stress
- $\phi'$ = effective angle of friction

According to Larsson (1977), the effective angle of friction can be evaluated by the formula:

$$\phi' = \phi_p - \alpha$$

where

- $\phi'$ = effective angle of friction
- $\phi_p$ = angle of interparticle friction
- $\alpha$ = angle between direction of particle displacement and horizontal plane ($\alpha = \arctan \frac{d\epsilon_y}{d\epsilon_H}$)

The effective angle of friction $\phi'$ depends on $\phi_p$ and $\alpha$ and also on stress level.

**Generalized drained shear strength**

According to Larsson (1977), a failure line in the relation between drained shear strength in direct
shear and effective vertical pressure has the shape as seen in Fig. 25 for three types of clay:

- normally compressible soft clay has \(\omega_n \approx \omega_L \approx 70\%\) and \(s_t = 15\) and a clay content of about 60% 
- clay with low compressibility, sensitivity and rapidity (low rapidity means that the structure of the clay is insensitive to deformations and vibrations and a lot of work is required to break it down, Söderblom et al, 1974)
- highly sensitive clay with high rapidity often has \(\omega_n > \omega_L\).

![Three types of clay](image)

Fig. 24. Generalized drained shear strength in direct shear (after Larsson, 1977).

As seen in this figure the shear strength directly depends on the compressibility of the clay and the stress level. For three types of clay there are two breaking points on a failure line, one at 0.5 \(\sigma'_C\) and another at \(\sigma'_o\) (\(\sigma'_o\) is the preconsolidation pressure). Research and practice have found that all failure lines for vertical stress below 0.5 \(\sigma'_o\) are drawn as straight lines through origo with an angle of about 30° without great errors.
For the effective stresses higher than the pre-consolidation pressure, the drained shear strength can be calculated by the formula:

\[ t_{fd} = c' \tan \phi \]

where

- \( t_{fd} \) = drained shear strength, kPa
- \( c' \) = effective stress, kPa
- \( \phi \) = effective angle of friction, degree

**Generalized undrained shear strength**

The undrained shear strength that can be mobilized in soft clay is shown in Fig. 25. In undrained shear the change in pore pressure during the test affects the effective stresses. According to Larsson (1977), the pore pressure will decrease in the sample consolidated under vertical stress lower than about 0.45 \( c' \) and the pore pressure will increase in the sample consolidated under higher vertical stress.

![Fig. 25. Undrained strength in direct shear tests](after Larsson, 1977).
b) Shearing rate

The direct shear tests are usually performed with the shearing as a horizontal movement of the upper part of the sample. In Sweden the following shearing rate is normally used for undrained and drained tests with stepwise loading in direct shear tests, Table 5.

Table 5. The shearing rate in direct shear tests for every normal stress, $\sigma'$.  

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<th>Test</th>
<th>Horizontal deformation of sample</th>
<th>Shearing rate</th>
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<tr>
<td>Undrained</td>
<td>0-0.5 mm</td>
<td>$\Delta t = \sigma'/20$ each 30'</td>
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<td></td>
<td>$&gt;0.5$ mm</td>
<td>$\Delta t = \sigma'/40$ each 15'</td>
</tr>
<tr>
<td>Drained</td>
<td>0-0.25 mm</td>
<td>$\Delta t = \sigma'/20$ each 30'</td>
</tr>
<tr>
<td></td>
<td>$&gt;0.25$ mm</td>
<td>$\Delta t = \sigma'/40$ each 15'</td>
</tr>
</tbody>
</table>

c) Normal stresses

It is known that the direct shear strength for soft clay directly depends on the compressibility of the clay and the stress level. The preconsolidation pressure $\sigma'_c$ of the tested soil is required in the tests and in the interpretation of the test results.

That the relation between shear strength and normal stress changes at a stress of 0.5 $\sigma'_c$ and $\sigma'_c$ is generally accepted for soft clay. Before shear tests the $\sigma'_c$-value for the tested soil should be known. The $\sigma'_c$-value is necessary to decide suitable consolidation stresses in the tests (also the stresses at which the samples are sheared).

To enable the plotting of the failure line as a relation between shear strength and normal stress samples of the tested clay should be sheared to
failure under at least four normal stresses: $0.3 \sigma'_C$, $0.6 \sigma'_C$, $0.85 \sigma'_C$ and $1.5 \sigma'_C$ (where $\sigma'_C$ is the preconsolidation pressure). The failure line that can be plotted from drained direct shear tests is shown in Fig. 26.

![Graph showing the failure line drawn from drained direct shear tests.](image)

Fig. 26. The failure line drawn from drained direct shear tests.

In Fig. 26 the empirical finding that the failure line for normal stresses between 0 and $0.5 \sigma'_C$ is fairly straight and passes through the origin is used together with the knowledge that the failure line for normal stresses above $\sigma'_C$ is straight and its extension passes through the origin.
4. PRACTICAL SIGNIFICANCE OF THE PRECONSOLIDATION PRESSURE

The preconsolidation pressure has great significance in the practice of geotechnical problems. The most important practical application of the preconsolidation pressure is in connection with settlement analyses and geological investigation. The role of $\sigma'_c$ in the geological investigation has been presented above (part: classification and identification of soft clays).

The preconsolidation pressure $\sigma'_c$ plays its important role in settlement analyses. The following example presented by Casagrande (1936) will illustrate the role of $\sigma'_c$ in settlement analysis (Fig. 27).

A clay layer has been compressed at one time in the history by an overburden pressure of 3 kg/cm$^2$ which later was reduced by for instance erosion to the present overburden pressure of 1 kg/cm$^2$. If a building with a load of 1 kg/cm$^2$ now is set up on this clay, the compression under the building load ($\Delta_1$) will take place along the re-compression curve from point B to C. If the preconsolidation pressure (in this example $\sigma'_c = 3$ kg/cm$^2$) is not taken into account and the settlement is calculated based on the present overburden pressure ($\sigma'_c = 1$ kg/cm$^2$) the compression from the building load would then follow the virgin compression curve from D to E equal to $\Delta_2$. As seen in Fig. 27 $\Delta_2 > \Delta_1$ and according to the author $\Delta_2$ may be five to ten times greater than $\Delta_1$.

From the above example it is clear that in connection with settlement analysis it is necessary to carefully study the preconsolidation pressure.
5. SETTLEMENT CALCULATION

The classical method for calculation of settlement on soft clay is based on the assumption that the clay is normally consolidated \((\sigma'_c = \sigma'_0)\). Therefore the settlement is calculated by the classical equation:

\[
\delta = z \frac{C_c}{l + e} \log \frac{\sigma'_0 + \Delta \sigma}{\sigma'_0} \Delta z
\]

where

- \(\Delta z\) = thickness of the individual clay layer
- \(\sigma'_0\) = effective overburden pressure
- \(\Delta \sigma\) = vertical load increment
- \(C_c\) = compression index
- \(e'_0\) = initial void ratio of clay.

According to Bjerrum (1972, 1973) the settlement of soft clay calculated by the classical equation is too large, because the clay in fact is lightly or heavily overconsolidated.

In this method, the compression index \(C_c\) is calculated from \(e - \log \sigma'\) curve (see Fig. 11a):

\[
C_c = \frac{\Delta e}{\log \frac{\sigma'_0 + \Delta \sigma'}{\sigma'}} = \frac{\Delta e}{\Delta \log \sigma'}
\]

In order to avoid the determination of the void ratio, the compression index \(\epsilon_2\) is used in Sweden. The compression index \(\epsilon_2\) is the relative compression of the sample and it is evaluated from compression-log pressure curve (see Fig. 11b).

The relation of these compression indices is:

\[
\epsilon_2 / \log 2 = C_c / (1 + e'_0)
\]

The settlement based on the compression index \(\epsilon_2\) is therefore calculated by the formula:
The above calculation of settlement is based on the assumption that the oedometer curve is linear in a $e-log\sigma'$ curve as well as in a $e-log\sigma''$ curve for stresses higher than the preconsolidation pressure. However, for some clays this assumption is not valid (Larsson, 1977, 1981). In this case the settlement calculated by the above formula is not correct, as the method for evaluating the compression indices ($C_o$ and $\varepsilon_2$) is not suitable.

In the cases, when the oedometer curve is not linear for stresses higher than the preconsolidation pressure, the tangent modulus $M$ is often used for expression of the soil compressibility (Larsson, 1981, Janbu et al, 1967). The method for evaluating the tangent modulus $M$ is described in this report, see part 4.

The settlement with the tangent modulus theory is calculated by the formula:

$$\delta = \sum_{o}^{z} \frac{\Delta \sigma'}{M} \Delta \sigma$$

$$= \sum_{o}^{z} \left[ \frac{1}{mB} \left( \frac{\sigma'+\Delta \sigma}{\sigma'} \right)^{\beta} - \left( \frac{\sigma'}{\sigma'} \right)^{\beta} \right] \Delta \sigma$$

in which the tangent modulus $M$ is calculated by the formula:

$$M = m \sigma' \left( \frac{\sigma'}{\sigma'} \right)^{1-\beta}$$

where

$m$ = modulus number

$\beta$ = pressure exponent

$\sigma'$ = effective vertical pressure

$\sigma'$ = reference pressure, usually 100 kPa (Andréasson et al, 1973)
The above calculation of settlement is based on the assumption that the oedometer curve is linear in a \(e - \log a\) curve as well as in a \(e - \log a'\) curve for stresses higher than the preconsolidation pressure. However, for some clays this assumption is not valid (Larsson, 1977, 1981). In this case the settlement calculated by the above formula is not correct, as the method for evaluating the compression indices \((C_c\) and \(\varepsilon_r\)) is not suitable.

In the cases, when the oedometer curve is not linear for stresses higher than the preconsolidation pressure, the tangent modulus \(M\) is often used for expression of the soil compressibility (Larsson, 1981, Janbu et al, 1967). The method for evaluating the tangent modulus \(M\) is described in this report, see part 4.

The settlement with the tangent modulus theory is calculated by the formula:

\[
\delta = \sum_{o} \frac{\Delta \sigma'}{M} \Delta z = \sum_{o} \left[ \frac{1}{m^\beta} \left( \frac{\sigma' + \Delta \sigma}{\sigma'} \right)^\beta - \frac{\sigma'}{\sigma'}^\beta \right] \Delta z
\]

in which the tangent modulus \(M\) is calculated by the formula:

\[
M = m \frac{\sigma'}{\sigma'}^{1 - \beta}
\]

where

\(m\) = modulus number

\(\beta\) = pressure exponent

\(\sigma'\) = effective vertical pressure

\(\sigma'\) = reference pressure, usually 100 kPa (Andréasson et al, 1973)
In the case $\beta = 0$ the settlement is then calculated by the following formula:

$$
\delta = \sum_{0}^{a} \frac{1}{m} \ln \frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \Delta z
$$

and the modulus number $m = \frac{\sigma'_0 \theta}{\varepsilon_2}$

The above formulas in the calculation of settlements are applicable to normally consolidated soft clay with $\sigma'_0 = \sigma'_c$ (where $\sigma'_0$ is the effective overburden pressure and $\sigma'_c$ is the preconsolidation pressure). In the calculation of settlement for overconsolidated clay ($\sigma'_c > \sigma'_0$) the $\sigma'_c$ is used instead of $\sigma'_0$ in the formulas.
6. DESIGN PARAMETERS OF SOFT CLAYS

The shear strength of a soil is a basic parameter used in stability calculation for many structures, such as:

- embankments on soft clay,
- slopes,
- foundation, etc.

For this reason, many methods and apparatus have been developed for determining the strength properties. The choice of suitable methods and apparatus in both laboratory and field tests is very important. It has been found though, that a more important problem is that of how to use the geotechnical properties in design. This problem is generally concerned with soft clays and has become an interest for many researchers.

Generally the soft clays are very sensitive to structural disturbance. The disturbances of samples occur in the process of boring, sampling, transportation, storage and testing. The samples brought into the laboratory are therefore seldom undisturbed. The results of laboratory soil tests are affected by sample disturbance. For laboratory technicians who test soil samples and for engineers who analyze geotechnical problems, it is important to know how design parameters obtained from laboratory tests are influenced by sample disturbance and which parameters are sensitive to sample disturbance. Many researchers have found that for soils, especially for soft cohesive soils, the deformation and strength properties are sensitive to sample disturbance.

In order to limit the sample disturbance (that means to increase the sample quality), many different methods and samplers for sampling in soft cohesive soils have been developed. A description of different methods for
taking samples in soft cohesive soils was presented in International Manual for Sampling of Soft Cohesive Soils recommended by the Sub-committee on Soil Sampling, International Society for Soil Mechanics and Foundation Engineering, Tokyo, 1981. On the other hand, a great number of research and tests on the quality in soil sampling and the determination of effects of sample disturbance on geotechnical properties have been carried out. All of this work has been carried out to obtain real geotechnical properties of soft cohesive soils. For this purpose, numerous observations from failures of slopes and embankments and comparison of the test results in stability calculation of embankments and foundations have been performed. From this work different methods for correlation of geotechnical properties have been introduced.

Some methods for correction of undrained shear strength of soft clays are presented below. The effects of sample disturbance on geotechnical properties of soft clays are presented in report No 3.

6.1 Correction of undrained shear strength of soft clays

The undrained shear strength of soft clays is usually determined in field vane tests or in laboratory tests. The corrections of undrained shear strength of soft clays (normally consolidated or slightly overconsolidated) according to field vane tests and fall-cone tests is summarized below.

Many observations from failures of slopes, embankments and foundations on soft clay have shown that the undrained shear strength measured by field tests or laboratory tests on soft clays are often too high (sometimes too low). The theoretical calculations of factors of safety have also showed this. In stability calculation the undrained shear strength of soft clays
measured by field vane tests and laboratory fall-cone or vane tests therefore should be corrected in order to correspond to the real shear strength of these clays. The correction (or reduction) of undrained shear strength is made as follows.

If the undrained shear strength measured by field vane test and fall-cone test is expressed in $T_v$ and $T_k$ respectively, the undrained shear strength to be used in stability calculations $T_{fu}$ should be reduced by the reduction factor $\mu$:

$$T_{fu} = \mu T_v \quad \text{or} \quad T_{fu} = \mu T_k$$

The reduction factor $\mu$ is less than unity (1) and depending on soil type, state of soil etc.

There exists different methods for correcting the undrained shear strength of soft clays. The methods for correcting the undrained shear strength have developed in two directions. In the first direction, one attempts to correct the undrained shear strength by modelling field behaviour via consolidated-undrained tests performed with different stress histories and modes of failures. The second direction develops empirical correlation among soil type, the in situ test methods and the type of stability calculation. The methods based on model require complicated equipments. Therefore methods based on empirical correlation seem to be preferred by many engineers.

Empirical methods for correction of undrained shear strength are based on some physico-mechanical properties of clays. Almost every method uses soil properties such as liquid limit, plasticity index, ratio of vane shear strength to the effective overburden pressure, overconsolidation ratio as parameters in empirical relation. Representative for these methods are
Bjerrum, Hansbo, SGI and others. Other authors such as Pilot, Aas use the relationship between the theoretical factor of safety and the empirical factor of safety at failure. In this method, physico mechanical properties are used as a basis for the correction and should then be calculated as a mean value for the critical slip surface. It should be noted that the plasticity index $I_p$ internationally has been commonly used as a parameter in empirical relations, but in Swedish practice the liquid limit $w_L$ is often used. Corrections of undrained shear strength are often made for organic and high plastic clays but according to some authors corrections should be made also for inorganic clays.

6.1.1 Methods for reducing undrained shear strength of soft clays measured by vane tests and fall-cone tests

In 1946 the Swedish Geotechnical Institute (SGI) recommended that the reduction factor $\mu$ for undrained shear strength from fall-cone test should be based on the organic content of the clay:

$$\mu = 0.80 \text{ for organic clay}$$
$$\mu = 0.60 \text{ for gyttja (ooze)}$$

SGI (1969) found that the reduction should be made not only for fall-cone tests but also for vane tests and the same reduction factor $\mu$ should be used for both $\tau_k$ and $\tau_v$ based on the liquid limit $w_F$. Table 6 shows the $\mu$-value for undrained shear strength measured in fall-cone tests or field vane tests recommended by SGI (Broms, 1972).
In 1972, 1973 Bjerrum, through his comprehensive study of reported failures in different parts of the world, found that the undrained shear strength measured by vane test in soft clay should be reduced. The Bjerrum's reduction factor is based on the plasticity index $I_p$. According to Helenelund (1977) this reduction factor can be expressed approximately by the formula

$$u = \frac{1.2}{\frac{l}{I_p}}$$

If this reduction factor is depending on the liquid limit $w_L$, according to Helenelund (1977) the relationship between $u$ and $w_L$ is as showed in Fig. 28. In this figure the reduction factor according to SGI (table 6) is also showed in order to compare these two reduction factors.

![Fig. 28. Reduction factor as a function of the liquid limit.](image)
As seen in Fig. 28, according to SGI, the reduction of undrained shear strength is made for clays with a liquid limit $w_L \geq 80\%$, whereas according to Bjerrum it is made even for clays with a liquid limit $w_L < 50\%$. On the other hand, there are differences between these reduction factors. For clays with a liquid limit less than about 120\% the differences between reduction factors recommended by SGI and Bjerrum are important. For higher plastic clays ($w_L > 120\%$) the SGI's and Bjerrum's reduction factors are of small difference.

Based on analyses of embankment failures, Pilot (et al., 1972) found that the theoretical factor of safety at failure increases with both increasing liquid limit and increasing plasticity index. According to the author the factor of safety at failure can be expressed by the empirical equations

\[
P_{sf} = 0.6 w_L + 0.7
\]

\[
P_{sf} = 0.7 I_p + 0.9
\]

where

- $w_L$ = liquid limit (%/100)
- $I_p$ = plasticity index (%/100)

In this case, the reduction factor can be calculated as $\mu = 1/P_{sf}$. This reduction factor is applicable for undrained shear strength measured by vane tests.

\[
\mu = \frac{1}{P_{sf}} = \frac{1}{0.6 w_L + 0.7}
\]

\[
\mu = \frac{1}{P_{sf}} = \frac{1}{0.7 I_p + 0.9}
\]

From the above equations it is clear that $\mu = 1$ when the equations
if the reduction factors are considered equal for undrained shear strength measured in fall-cone tests and vane tests and the reduction factors $\mu$ are compared, according to different authors the difference in these $\mu$-values can be found. On the other hand we can find the limits of the consistency of clay where the undrained shear strength has to be reduced or not (Table 7). Table 7 shows a lower limit of consistency of clay where the reduction factor $\mu$ should be made according to SGI, Bjerrum, Pilot.

Table 7. The reduction factor $\mu$ should be made for a lower limit of consistency ($\omega_L$, $I_p$).

<table>
<thead>
<tr>
<th>Author</th>
<th>Reduction when</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\omega_L$ (%)</td>
<td>$I_p$ (%)</td>
<td></td>
</tr>
<tr>
<td>SGI (1972)</td>
<td>&gt;80</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Bjerrum (1972, 1973)</td>
<td>&gt;50</td>
<td>&gt;20</td>
<td></td>
</tr>
<tr>
<td>Pilot (1972)</td>
<td>&gt;50</td>
<td>&gt;14</td>
<td></td>
</tr>
</tbody>
</table>

The ratio of undrained shear strength from vane tests to effective overburden pressure ($\tau_v/\sigma_0'$) is also used for the calculation of the reduction factor $\mu$.

In 1976 Aas (et al) found that a linear relationship between the factor of safety at failure and the $\tau_v/\sigma_0'$-ratio can be expressed by the formula:

$$F_{sf} = 2.7 \tau_v/\sigma_0' + 0.38$$

In this case, the reduction factor $\mu$ will be
\[ \mu = \frac{1}{F_{sf}} = \frac{1}{2.7 \frac{\tau_v}{\sigma_o'} + 0.38} = \frac{2.6}{1 + 2 \frac{\tau_v}{\sigma_o'}} \]

where

\[ \tau_v = \text{mean (uncorrected) vane shear strength} \]
\[ \sigma_o' = \text{mean effective overburden pressure} \]

Helenelund (1977) found by his comparison between different methods for reducing the undrained shear strength of soft clay that:

- The ratio \( \tau_v/\sigma_o' \) seems preferably to be used as a basis for reduction, for example according to the formula

\[ \mu = \frac{2.6}{1 + 2 \frac{\tau_v}{\sigma_o'}} \]

- The use of a reduction factor is depending on the ratio \( \tau_v/\sigma_o' \) and the relationship between this ratio and the liquid limit \( w_L \) (Fig. 29).

\[ \mu \leq \frac{2}{1 + 2 \frac{\tau_v}{\sigma_o'}} \]  

\[ \mu \leq \frac{2.5}{1 + 2 \frac{\tau_v}{\sigma_o'}} \]  

\[ \text{USE SMALLER \( \mu \)-VALUE} \]

Fig. 29. Relation between the \( w_L \)-scale and the \( \tau_v/\sigma_o' \)-scale in the reduction factor \( \mu \) (according to Helenelund, 1977).
According to the author, the reduction factor can be found using the \( \omega_L \)-scale in Fig. 29 when

\[
\frac{\tau_v}{\sigma_0'} < 0.10 + 0.25 \omega_L
\]

If the \( \frac{\tau_v}{\sigma_0'} \)-ratio is greater, the \( \mu \)-value should be determined using the \( \frac{\tau_v}{\sigma_0'} \)-scale.

Instead of the liquid limit used in Table 6, the reduction factor \( \mu \) is taken from Table 8 according to the formula

\[
\mu = \frac{0.30}{\frac{\tau_v}{\sigma_0'}}
\]

Table 8. Reduction factors as a function of the \( \frac{\tau_v}{\sigma_0'} \)-ratio.

<table>
<thead>
<tr>
<th>Ratio ( \frac{\tau_v}{\sigma_0'} )</th>
<th>Reduction factor ( \mu )</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.30 - 0.35</td>
<td>0.90</td>
</tr>
<tr>
<td>0.35 - 0.40</td>
<td>0.80</td>
</tr>
<tr>
<td>0.40 - 0.475</td>
<td>0.70</td>
</tr>
<tr>
<td>0.475 - 0.55</td>
<td>0.60</td>
</tr>
<tr>
<td>0.55 - 0.65</td>
<td>0.50</td>
</tr>
<tr>
<td>0.65 - 0.85</td>
<td>0.40</td>
</tr>
<tr>
<td>&gt; 0.85</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Many relationships between the \( \frac{\tau_v}{\sigma_0'} \)-ratio and the liquid limit \( \omega_L \) or the plasticity index \( I_p \) have been proposed by authors, such as:

Skempton (1954) \( \frac{\tau_v}{\sigma_0'} = 0.11 + 0.37 I_p \)
Hansbo (1957) \( \frac{\tau_v}{\sigma_0'} = 0.45 \omega_L \)
Helenelund (1977) \( \frac{\tau_v}{\sigma_0'} = 3.10 + 0.25 \omega_L \)

Fig. 30 shows the undrained shear strength determined by field vane test over preconsolidation pressure versus liquid limit according to different authors.
The same data in Fig. 30 plotted against the plasticity index are shown in Fig. 31.

Fig. 30. Undrained shear strength determined by field vane tests over preconsolidation pressure versus liquid limit (after Larsson, 1980).
Moreover, due to different other factors affecting the results from field tests and laboratory tests, some factors correcting undrained shear strength of soft clays should be considered:

- correction factor for the effect of time
- correction factor for anisotropy
- correction factor for progressive failure, etc.

All reduction factors or factors correcting different effects affecting undrained shear strength are concerned with the calculation of the economic optimum value of the factor of safety ($F_{opt}$). The factor of safety for earthworks has normally been recommended...
as $F_s = 1.30-1.50$. The relationship between the factor of safety and different costs for construction ($B =$ construction costs, $R =$ risk of reconstruction cost, $M =$ risk of machine damages, $P =$ risk of human accidents) by a calculation of the construction costs of the Saima canal is shown in Fig. 32. The optimum factor of safety in this case is found, $F_{opt} = 1.50$. It should be noted that the optimum factor of safety depends on the $R$-, $M$- and $P$-costs that in turn depend on different types of structures, during which period the failure occurs. The optimum factor of safety also depends on an analytic method (short-term stability or long-term stability). The $F_{opt}$-value applied in long-term stability may be much higher than one in short-term stability (Helene Lund, 1977).

Fig. 32. Determination of the optimum factor of safety $F_{opt}$. $R$-, $M$- and $P$-costs are proportional to the probability of failure.
The influence on shear strength reduction on the numerical value of the optimum factor of safety \( F_{opt} \) has been considered. Fig. 33 shows the economic optimum values of the factor of safety according to construction costs and costs of repair of a section of the Saima canal (at failure 2, Slunga et al., 1972).

![Fig. 33. Influence of shear strength reduction on the numerical value of \( F_{opt} \).](image)

As seen from Fig. 33 it is clear that the \( F_{opt} \)-values without reduction of undrained shear strength are higher than the ones with reduction. From this figure it is also seen that the \( F_{opt} \)-values depend on which costs that are taken into account.
7. REFERENCES


CLASSIFICATION AND LABORATORY TESTING OF SOFT CLAY

BUI DINH NHUAN
Swedish Geotechnical Institute
1981
CONTENTS

ACKNOWLEDGEMENTS 3

1. INTRODUCTION 4

2. CLASSIFICATION AND IDENTIFICATION 5
  2.1 Normally consolidated clays 5
  2.2 Overconsolidated clays 8

3. LABORATORY TESTS 9
  3.1 Determination of the liquid limit 9
    3.1.1 Casagrande method 9
    3.1.2 Fall-cone method 9
  3.2 Oedometer tests 16
    3.2.1 Incremental loading method 17
      a Standard procedure (STD test) 18
      b Loading procedure suggested by Bjerrum 18
      c The LIN test 19
    3.2.2 Interpretation of oedometer test results with the incremental loading method 19
      a Relative compression ε 19
      b Determination of the preconsolidation pressure \( \sigma_c \) 19
      c Determination of compression index 21
      d Determination of the tangent modulus \( M \) 23
      e Determination of the coefficient of consolidation 25
  3.2.3 Oedometer tests with continuous loading 33
      a Constant rate of strain test (CRS-test) 33
      b Constant gradient test (CGI-test) 34
      c Continuous consolidation test (CC-test) 34
  3.3 Determination of strength characteristics 34
    3.3.1 Determination of undrained shear strength by fall-cone tests 36
    3.3.2 Influence of incorrect height adjustment 38
      a Correct height adjustment (standard test) 38
      b Initial penetration 38
      c Initial height of fall 39
    3.3.3 Shear strength in direct shear tests 41
      a Generalized model for shear strength of soft clay in direct shear tests 43
      b Shearing rate 46
      c Normal stresses 46
4. PRACTICAL SIGNIFICANCE OF THE PRECONDENSATION PRESSURE 48
5. SETTLEMENT CALCULATION 49
6. DESIGN PARAMETERS OF SOFT CLAYS 52
   6.1 Correction of undrained shear strength of soft clays 53
   6.1.1 Methods for reducing undrained shear strength of soft clays measured by vane tests and fall-cone tests 55
7. REFERENCES 65
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Linköping, November 1981
Bui Dinh Nhuan
1. INTRODUCTION

In many parts of the world, large areas are covered by soft clay deposits. Civil engineers, engineering geologists and others who are concerned with the design and construction of structures have been interested in the problems of construction on deposits of soft clay.

Laboratory investigations on soft clays have been intensively developed, especially in Sweden as well as in Scandinavian countries with their extensive deposits of soft clays. Whereas in Vietnam laboratory as well as field investigations on soft clays still have limitations as to methods and equipments.

The purpose of this report is to collect and summarize some Swedish methods and experiences in the laboratory investigation on soft clays. The classification and identification and problem of design parameters of soft clays are also collected.
2. CLASSIFICATION AND IDENTIFICATION

Recent research has shown that soft clays have their particular characteristics and the classification and identification of these clays should be based on their engineering properties. The following information on which the classification and identification on soft clay may be based is:

- the geological history (stress history) of the deposit
- the water content and the Atterberg limits
- the strength properties: vane shear strength
- the deformation properties: the compressibility characteristics determined from oedometer tests.

Based on the above information, Bjerrum (1973) proposed that soft clay can be classified into the following main groups:

1. Normally consolidated clays
   - normally consolidated young clays
   - normally consolidated aged clays

2. Overconsolidated clays

3. Weathered clays

4. Quick clays

5. Cemented clays.

Two groups (normally and overconsolidated clays) are briefly presented below.

2.1 **Normally consolidated clays**

The normally consolidated clays can be "young" or "aged". The difference between these clays is
shown in Table 1.

Table 1. Characteristics of "young" and "aged" normally consolidated clays.

<table>
<thead>
<tr>
<th>Normally consolidated young clays</th>
<th>Normally consolidated aged clays</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clay which has recently been deposited. large settlement under $\sigma'_f + \Delta \sigma$</td>
<td>Young clay under constant effective stress for long time (hundreds or thousands of years). Without significant settlement under $\sigma'_f + \Delta \sigma$ ($\Delta \sigma$ is definite value).</td>
</tr>
<tr>
<td>Small strength and greater compressibility</td>
<td>Greater strength and small compressibility</td>
</tr>
<tr>
<td>$\sigma'_f = \sigma'_o$ (from $e$-$\log \sigma'$ curve)</td>
<td>$\sigma'_f &lt; \sigma'_o$ (from $e$-$\log \sigma'$ curve)</td>
</tr>
<tr>
<td>Overconsolidation ratio $\sigma'_f/\sigma'_o = 1$</td>
<td>Overconsolidation ratio $\sigma'_f/\sigma'_o &gt; 1$ and increases with the plasticity index $I_p$.</td>
</tr>
<tr>
<td>$T_v$ increases linearly with $\sigma'_o$ smaller ratio $T_v/\sigma'_o$</td>
<td>$T_v$ increases linearly with $\sigma'_o$ greater ratio $T_v/\sigma'_o$</td>
</tr>
</tbody>
</table>

Fig. 1 shows the difference in the geological history and compressibility of a "young" and an "aged" normally consolidated clay according to Bjerrum (1973) based on the $e$-$\log \sigma'$ curve from consolidation test.

![Fig. 1. Geological history and compressibility of a "young" and an "aged" normally consolidated clay.](image-url)
The ratio of the vane shear strength $\tau_v$ to the effective overburden pressure $\sigma'_o$ as well as the ratio of the preconsolidation pressure to the effective overburden pressure of both "young" and "aged" normally consolidated clays depend on their plasticity index $I_p$. Fig. 2 shows the correlation between the $\tau_v/\sigma'_o$- and $\sigma'_d/\sigma'_o$-values (in figure $s_u/p_o$ and $p_c/p_o$) and the plasticity index $I_p$.

![Graph showing correlation between $s_u/p_o$ and $p_c/p_o$ with plasticity index $I_p$.]

**Fig. 2.** Typical values of $(s_u/p_o)$ vane and $p_c/p_o$ observed in normally consolidated late glacial and post glacial clays.

From Table 1 and Figs. 1 and 2 it is clear that by using some engineering properties we can easily distinguish the "young" normally consolidated clay from the "aged" one.
2.2 Overconsolidated clays

The overconsolidated clays are clays whose present effective overburden pressure is less than a maximum previous effective pressure under which the clays once were consolidated. Overconsolidation is the result of one of the following causes:

- surface erosion
- decrease in pore water pressure during a certain time in the history of clays
- excavation
- variation in groundwater level.

For these clays the ratio of the maximum previous effective pressure (often called the preconsolidation pressure) to the present effective overburden pressure is used to determine the degree of overconsolidation. This ratio is called the overconsolidation ratio and is expressed by the formula:

\[
\text{overconsolidation ratio} = \frac{\text{preconsolidation pressure}}{\text{present overburden pressure}} = \frac{\sigma_0}{\sigma_x}
\]

If clays are only considered normally consolidated and overconsolidated, it is clear that for normally consolidated clays the overconsolidation ratio is unity and for overconsolidated clays it is greater than unity. Depending on this ratio the clays of this group may be lightly or heavily overconsolidated.

A difference between the two groups of clays is that, under the same additional load to the present overburden pressure, the normally consolidated clays will settle more than the overconsolidated clays.
3. LABORATORY TESTS

3.1 Determination of the liquid limit

In Sweden the liquid limit is determined by the fall-cone method or the percussion method. The fall-cone method is the most common method.

3.1.1 Casagrande method

The percussion method (Casagrande method) is based on the specifications of the American Society of Testing Materials (ASTM). The one-point method proposed by the Waterways Experiment Station et al. (1949) is normally used. However, this method cannot be used on soils with a liquid limit larger than 150 (Broms, 1981).

The liquid limit \( w_{L} \) in this method is calculated by the equation

\[
  w_{L} = w \left( \frac{n}{S} \right) g \beta
\]

where
\( w \) = water content
\( n \) = number of blows required to close a groove made by a special tool for a length of 13 m
\( S \) = inclination of the flow curve
\( \beta \) = inclination of the flow curve

3.1.2 Fall-cone method

In this method the liquid limit is defined as the water content at which a 60 g/60°-cone gives a penetration of 10 mm for a completely remoulded sample (Geotechnical Commission of the Swedish State Railway, 1914-1922).

As defined, the test for determination of the liquid limit should be repeated several times at different water contents. The determined liquid limit is then the water content of soil when the penetration of the cone is 10 mm. However, this test procedure is time-consuming.
can be evaluated from the oedometer curve. The oedometer curve is plotted in a diagram with the strain in linear scale and the vertical pressure in log-scale (Fig. 12).

The $m_j$- and $\beta$-values are evaluated by drawing a tangent to the stress-strain curve at $\sigma_j$ and extending it to $2.7 \sigma_j$ where $\sigma_j'$ is a reference stress ($\sigma_j' > \sigma_j$).

If the stress exponent $\beta$ is equal to 0 ($\beta = 0$) the oedometer curve is a straight line overlapping the tangent to the curve at $\sigma_j'$. The relative compression $\Delta \varepsilon_1$ is evaluated from the intersections of the vertical lines through $\sigma_j'$ and $2.7 \sigma_j'$ with the tangent to the oedometer curve. Thus the modulus number $m_j$ is calculated from the equation

$$\Delta \varepsilon_1 = \frac{1}{m_j}$$
If the oedometer curve after the preconsolidation pressure $\sigma'_p$ is not really straight but inflected as seen in Fig. 12 the real compression $\Delta \varepsilon_{2.7}$ between vertical pressures $\sigma'_p$ and $2.7\sigma'_p$ is evaluated. This occurs in the case with $\beta \neq 0$ and the stress exponent $\beta$ is calculated from the following equation:

$$\Delta \varepsilon_{2.7} = \frac{1}{m_2 \beta \cdot 2.7^{\beta-1}}$$

This method of describing compressibility of soft clays is not correct either but the approximation can be used for a larger stress interval than the compression index.

e) Determination of the coefficient of consolidation

The coefficient of consolidation $c_v$ is commonly used to predict the rates at which settlement will occur. The $c_v$-value can be determined from the oedometer curve by the Casagrande or the Taylor method. Both methods are derived from the Terzaghi theory:

$$\frac{\delta u}{\delta t} = c_v \frac{\delta^2 u}{\delta z^2}$$

where

$u$ = excess pore water pressure
$\delta t$ = time elapsed since loading
$c_v$ = coefficient of consolidation

and the $c_v$-value can be calculated by the equation:

$$c_v = \frac{K}{\gamma_w m_v} \text{ m}^2/\text{year}$$

$\gamma_w$ = unit weight of water (kN/m$^2$)
$K$ = vertical coefficient of permeability of the soil (m/yr)
To make the test less time-consuming different one-point methods have been developed. Nowadays a one-point method proposed by the Swedish Geotechnical Institute (Karlsson, 1961) is normally used in Sweden. This method is based on investigations of different Swedish soils and also certain soils from abroad.

The relation between the strength parameter $m/i^2$ (where $m =$ mass of cone, $i =$ cone penetration) according to Hansbo (1957) ($\tau_{fu} = k \cdot g \cdot m/i^2$) and the water content was plotted in a semi-logarithmic graph. The relation was called the consistency curve and corresponds to Casagrande's flow curve (Fig. 3).

The inclination at $w_L$ can be expressed by

$$\tan \theta = \frac{w_L - w_0}{\log 6 - \log 0.6} = \frac{W_L - W_0}{\log 6 - \log 0.6}$$

Fig. 3. Consistency curve. Definition of the inclination at $w_L$. 

Within a limited region around the liquid limit the curve can be approximated to a straight line with the following equation:

\[ w_L = w_i + tga \cdot \log \left( \frac{10}{i} \right)^2 \]

where

- \( w_i \) = water content at cone penetration \( i \)
- \( tga \) = inclination of the consistency curve at the liquid limit

The investigations showed that the value of \( tga \) is dependent on \( w_L \) and generally increases linearly with \( w_L \).

\[ tga = \frac{w_L - 17}{1.8} \]

The following formula can thus be derived

\[ w_L = M \cdot w_i + N \]

where

\[ M = \frac{1.8}{1.8 + 2 \log \frac{i}{10}} \]

\[ N = \frac{34 \log \frac{i}{10}}{1.8 + 2 \log \frac{i}{10}} \]

where

- \( w_L \) = liquid limit
- \( w_i \) = water content of remoulded sample at the cone penetration \( i \)
- \( M, N \) = correction factors

The evaluation of \( w_L \) by the one-point method is illustrated in Fig. 4.
Fig. 4. Evaluation of $w_L$ by the one-point method.

Compared with the Casagrande method the cone method is preferred because:

- the test is simple and fast
- the results are more consistent and less liable to experimental and personal errors
- the results depend more directly on the shear strength of the soil.

SGI has determined the shear strength at the Casagrande liquid limit and at the cone liquid limit for different soils by means of a laboratory vane apparatus. The results showed that the strength at the Casagrande liquid limit varied considerably between different soils (0.5-4 kPa) whereas the strength at the cone liquid limit was about the same for all samples.
The cone method is fundamentally more satisfactory because the mechanics of the test depend more directly on the shear strength of the soil. The Casagrande procedure introduces a dynamic component which is not related to shear strength in the same way for all soils. The number of precussions required to make the halves of the sample to flow together is besides the shear strength also dependent on the density of the sample.

Table 2. The shear strength of soil at \( W_L \) and \( W_F \) (Karlsson, 1962).

<table>
<thead>
<tr>
<th>Type of soil</th>
<th>Liquid limit</th>
<th>( \tau_{fL} ) (lab. vane test at liquid limit)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>cone</td>
<td>Casagrande</td>
</tr>
<tr>
<td>Postglacial clay</td>
<td>62</td>
<td>70</td>
</tr>
<tr>
<td>Mud</td>
<td>215</td>
<td>275</td>
</tr>
<tr>
<td>Bentonite</td>
<td>170</td>
<td>320</td>
</tr>
<tr>
<td>Kaoline I</td>
<td>56</td>
<td>53</td>
</tr>
<tr>
<td>Kaoline II</td>
<td>43</td>
<td>45</td>
</tr>
<tr>
<td>Coarse silt with some organic matter</td>
<td>34</td>
<td>30</td>
</tr>
</tbody>
</table>

SGI also has made an investigation in order to find out the reliability of routine determinations of the cone liquid limit and the Casagrande liquid limit (Karlsson et al, 1974).

The investigation comprised two different soils, a high-plastic clay and a low-plastic, somewhat silty, clay. The determinations were performed by 21 laboratories at different institutions and consulting firms. The results showed that the scatter was considerably smaller for the cone liquid limit, particularly for the high-plastic clay.
An other investigation by Sherwood and Ryley et al (1968) has also shown that results obtained by the cone method are more consistent and less liable to experimental and personal errors than those obtained by the Casagrande method.

The comparison between liquid limit determined by the cone method and by the Casagrande method for Swedish soils was worked out by Karlsson et al (1974). The results showed that for clays the Casagrande and cone liquid limit coincide when $w_L = 40\%$. At higher values the Casagrande liquid limit is generally higher than the cone liquid limit and at lower values the opposite is valid. For silt the Casagrande liquid limit is generally considerably lower than the cone liquid limit and for organic soils considerably higher.

For soft clay the liquid limit as well as the plastic limit is in Sweden normally determined on natural samples (samples which have not been dried in advance). Soils that are dried and sieved before determination are normally used internationally. According to Broms (1981) the drying of a sample in an oven can reduce the liquid and plastic limits especially if the soil is organic as illustrated in Fig. 5.
Fig. 5. Comparison of the cone liquid limit for dried and wet samples.

In routine tests the liquid limit is usually determined on samples which previously have been used to determine the shear strength. If the water content of the soil is too low, water should be added to the samples. In order to reduce the water content when it is too high, the sample is spread or rolled out on a gypsum plate. It is necessary to note that the time for cone penetration in clay and in silty soils is different. In clay soil the cone stops to penetrate into the soil a few seconds after the cone is released and that is enough for reading. In silty soil the cone often does not stop but continues to penetrate into the soil. In this case the penetration is taken about 10 seconds after the cone is released (Karlsson (1977), Broms (1981)).
Nowadays a one-point method for determination of the fall-cone liquid limit is generally made. The relation between cone penetration \( i \) (60 g/60°) and factors \( M \) and \( N \) in the formula \( w_L = M \cdot w_i + N \) is shown in Table 3.

Table 3. Relation between cone penetration \( i \) (60 g/60°) and factors \( M \) and \( N \) in the formula \( w_L = M \cdot w_i + N \).

<table>
<thead>
<tr>
<th>M (mm)</th>
<th>0</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
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</thead>
<tbody>
<tr>
<td>M</td>
<td>1.21</td>
<td>1.20</td>
<td>1.19</td>
<td>1.18</td>
<td>1.17</td>
<td>1.16</td>
<td>1.15</td>
<td>1.14</td>
<td>1.14</td>
<td>1.13</td>
</tr>
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<td>-3.2</td>
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<td>-2.7</td>
<td>-2.6</td>
<td>-2.5</td>
<td>-2.3</td>
<td>-2.2</td>
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<td>1.11</td>
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<td>-0.6</td>
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<td>-0.2</td>
<td>-0.1</td>
</tr>
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<td>0.97</td>
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</tr>
<tr>
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<td>+0.2</td>
<td>+0.2</td>
<td>+0.3</td>
<td>+0.4</td>
<td>+0.5</td>
<td>+0.5</td>
<td>+0.6</td>
<td>+0.7</td>
</tr>
<tr>
<td>M</td>
<td>0.96</td>
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</tr>
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<td>N</td>
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<td>+1.3</td>
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<tr>
<td>M</td>
<td>0.92</td>
<td>0.92</td>
<td>0.91</td>
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<td>0.89</td>
<td>0.89</td>
</tr>
<tr>
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<td>+1.4</td>
<td>+1.5</td>
<td>+1.5</td>
<td>+1.6</td>
<td>+1.7</td>
<td>+1.7</td>
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<td>+1.8</td>
<td>+1.9</td>
</tr>
<tr>
<td>M</td>
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<td>0.87</td>
<td>0.87</td>
<td>0.86</td>
</tr>
<tr>
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<td>+2.0</td>
<td>+2.1</td>
<td>+2.1</td>
<td>+2.2</td>
<td>+2.2</td>
<td>+2.2</td>
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<tr>
<td>M</td>
<td>0.86</td>
<td>0.86</td>
<td>0.86</td>
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</tr>
<tr>
<td>N</td>
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<td>+2.7</td>
<td>+2.7</td>
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<td>+2.7</td>
</tr>
</tbody>
</table>

### 3.2 Oedometer tests

Compression characteristics of soft clays are generally determined by oedometer tests. There are some different methods for oedometer tests:

- oedometer test with incremental loading
- constant rate of strain tests (CRS-tests)
- constant gradient tests (CGT-tests)
- continuous consolidation tests (CC-tests)
3.2.1 Incremental loading test

This method was suggested by Terzaghi in 1925 and has been widely used since then. In this method the test procedure is performed by incremental loading, each increment equal to the previous load and new increment loaded every 24 hours. During the test the sample is drained from the ends and readings of the compression are taken in a time sequence enabling a plot of the time-settlement curve for each increment. The oedometer test with incremental loading with a duration of 24 hours is considered standard.

However, this test procedure has its disadvantage because it takes a long time, at least a week for one sample. Therefore different variants of test procedure have been suggested.

The apparatus used for incremental loading test is shown in Fig. 6. Fig. 7 shows the cutting device used for mounting clay samples.

Fig. 6. Apparatus used for incremental oedometer tests.
Fig. 7. Cutting device used for mounting clay samples.

The oedometer ring is 40 mm in diameter. This size of oedometer ring is suitable to the 50 mm diameter sampling tube. The tested sample is 20 mm in height.

In oedometer test with incremental loading three test procedures have been used:

a) Standard procedure (STD test): daily load increments, each increment is equal to the previous load and a new increment is loaded every 24 hours. The following increments have often been used for the STD tests: 10, 20, 40, 80, 160 and 320 kPa. The time required for a STD test is at least 6 days.

b) Loading procedure suggested by Bjerrum (1973): for vertical pressures below the preconsolidation pressure the load increments are reduced and new increments are loaded at the end of primary consolidation (100% consolidation). Above the preconsolidation pressure the test is continued with doubled load increments with 24 hours' duration.
The time required for a test will be 3 to 4 days because the first small increments can usually be completed during one working day. According to Sällfors (1975) this method is called the NGI-test.

c) Tests with daily load increments; equal increments usually 10 or 20 kPa each with a duration of 24 hours. This test is a LIN test (Sällfors, 1975). The LIN test takes 8 to 12 days depending on the preconsolidation pressure.

3.2.2 Interpretation of oedometer test results with the incremental loading method

a) Relative compression $\varepsilon$.

The results from incremental oedometer tests performed by the STD or NGI procedure are presented in a diagram as a stress-strain curve. In this plot the vertical effective pressure is in log-scale (Fig. 8). From this diagram the relative compression $\varepsilon$ between the vertical in situ pressure in ground $\sigma_0^v$ and the calculated final pressure $\sigma'$ can be determined and therefore the settlement is calculated by the following formula:

$$\delta h = \varepsilon \cdot H$$

where $H =$ thickness of the soil layer.

The stress-strain curves from LIN-tests are presented in linear scales.

b) Determination of the preconsolidation pressure $\sigma_0^v$.

The preconsolidation pressure can be determined from the oedometer curve obtained in STD-tests according to the Casagrande method. This method has been widely
used. Fig. 9 shows the Casagrande method for determining the preconsolidation pressure. In this method, the vertical pressure is in log-scale and the relative compression is in linear scale. At the point with the smallest radius of curvature, a tangent to the oedometer curve and a horizontal line are drawn. The angle between these two lines is bisected. Then the straight portion of the oedometer curve is drawn and extended so that it intersects the bisectrix. The pressure at this intersection is the preconsolidation pressure $\sigma'_d$.

Due to disturbance of samples, the evaluated preconsolidation pressure is often too low. Therefore the disturbance should be taken into account when determining the preconsolidation pressure. The determined preconsolidation pressure according to many authors is sensitive to the loading sequence and the duration of each load step.

In LIN-tests the preconsolidation pressure is determined as the intersection of the extended straight portions (before and after $\sigma'_d$) of the curve.

The preconsolidation pressure is often determined from a stress-strain curve with stress in log-scale and strain in linear scale. This strain-log stress curve is suitable for determination of the preconsolidation pressure of normal soft clays. In this case the oedometer curve makes a sharp break and makes the determination of the preconsolidation pressure rather easy, curve 1 in Fig. 10 (B). For some soft clays though, for example clays with a high swelling capacity and relatively high compression
modulus below the preconsolidation pressure, this strain-log stress curve is disadvantageous. On one hand due to swelling characteristics (clays more or less overconsolidated have swollen in the ground) and on the other hand due to disturbance during sampling, most clays brought into the laboratory have undergone some swelling. In this case the strain-log stress curve will give a shape in a regular bend for stresses below and just after the preconsolidation pressure, curve 2 in Fig. 10 (B). This shape of the oedometer curve makes the determination of the preconsolidation pressure difficult because it is difficult to find the smallest radius of curvature. In this case the oedometer curve for soft clays should be plotted in linear scales (both for stress and strain), Fig. 10 (A).

![Oedometer curves](image)

**Fig. 10.** Oedometer curves in (A) - linear scale and (B) - semilog scale for soft clays.

1 - normal soft clay
2 - clay with a high swelling capacity

**c) Determination of compression index**

The compression indices $C_o$ and $\epsilon_2$ are also evaluated from the oedometer curve. There is a difference in
the determination of $C_o$- and $\varepsilon_2$-value.

The compression index $C_o$ is evaluated from an oedometer curve plotted in a void ratio-vertical pressure relationship (Fig. 11).

To avoid the determination of the void ratio, the compression index $\varepsilon_2$ is used. The compression index $\varepsilon_2$ is evaluated from an oedometer curve plotted in a relative compression-vertical pressure relationship (Fig. 11). In both cases, the vertical pressure is plotted in log-scale.

![Fig. 11](image.png)

**Fig. 11.** a - Evaluation of compression index $C_o$.  
b - Evaluation of compression index $\varepsilon_2$.

The straight line of the oedometer curve after the preconsolidation pressure is chosen for evaluation of the compression indices $C_o$ and $\varepsilon_2$ (see Fig. 11).

The compression index $C_o$ is determined by the following equations:
The compression index $\varepsilon_2$ is used in Sweden, where $\varepsilon_2$ is the relative compression of a sample at a doubling of the vertical pressure (Fig. 11). The relation between these compression indices is:

$$\varepsilon_2/\log 2 = C_\sigma (1 + \varepsilon_0)$$

d) Determination of the tangent modulus $M$

The determination of the compression indices $C_\sigma$ and $\varepsilon_2$ is performed with the assumption that the oedometer curve should be a straight line for stresses higher than the preconsolidation pressure. For some clays, for example for Swedish clays, this assumption is not valid and this method for determining the compressibility ($C_\sigma$ and $\varepsilon_2$) is not suitable since the method is only valid within a small stress range. Therefore another method for determining the compressibility has been suggested.

Soil compressibility is often expressed by a tangent modulus $M$ (Odhe (1951), Janbu (1967), Brinch-Hanssen (1966) and others). The tangent modulus $M$ is expressed by the following equation:

$$M = m_j \sigma'_j \left(\frac{\sigma'_j}{\sigma'_j}\right)^{1-\beta}$$

where

- $m_j$ = modulus number
- $\beta$ = stress exponent
- $\sigma'_j$ = effective vertical stress
- $\sigma'_j$ = reference stress (usually 100 kPa)

In this case for calculation of the tangent modulus $M$, it is necessary to determine the modulus number $m_j$ and the stress exponent $\beta$. These two parameters
neglected. After the point of \( U_9 \) is determined, the point where the primary settlement is assumed to finish \( (U_{100}) \) is determined on the compression axis by the relative compression at \( U_{100} (\varepsilon_{100}) \), see Fig. 17.

\[
\varepsilon_{100} = \frac{\varepsilon_{91}}{0.9}
\]

Any settlement below this \( \varepsilon_{100} \) is considered as secondary settlement.

In the Casagrande method, the determination of the point where the primary settlement begins is based on the assumption that, in the early stages of consolidation, the time is proportional to the square of the average degree of consolidation \( (t_v = f(U_v)^2) \) and therefore in the early stages we have

\[
\frac{t_1}{t_2} = \left(\frac{\varepsilon_1}{\varepsilon_2}\right)^2
\]

If \( \varepsilon_2 = 2\varepsilon_1 \), then

\[
\frac{t_1}{t_2} = \left(\frac{1}{2}\right)^2
\]

or \( t_2 = 4t_1 \)

The settlement between \( t_1 \) and \( t_2 (t_2 = 4t_1) = \varepsilon_2 - \varepsilon_1 = d \) (in Fig. 18), because \( \varepsilon_2 = 2\varepsilon_1 \) so \( \varepsilon_1 = d \) and with this assumption \( U_0(\varepsilon_0) \) is determined. The point where the primary settlement is assumed to finish is obtained by the help of the coefficient of secondary compression \( \varepsilon_S \) (see Fig. 18).
Fig. 17. Identification of types of settlement: Taylor method.

Fig. 18. Identification of types of settlement: Casagrande method.
3.2.3 Oedometer tests with continuous loading

Oedometer tests with continuous loading have been developed during the last fifteen years. Compared with the traditional incremental loading test, the oedometer tests with continuous loading have three advantages:

- they give continuous stress-strain relations
- they give continuous $c_{v}$-stress relations
- they can be run automatically.

For this method, the following tests have been performed:

- constant rate of strain tests (CRS-tests)
- constant gradient tests (CGI-tests)
- continuous consolidation tests (CC-tests).

a) Constant rate of strain test (CRS-test)

In the CRS-test the sample is compressed at a constant rate. The sample is drained at the upper end and sealed at the bottom where the pore pressure is measured. During the test, the compressive force, the deformation, the pore pressure at the bottom and time are automatically recorded continuously.

Besides parameters of compressibility obtained from oedometer tests with incremental loading, CRS-tests give the following continuous relations:

- effective stress and strain
- modulus and effective stress
- coefficient of consolidation and effective stress
- permeability and strain.

During 1971-1975 a large investigation was carried out on comparisons between different oedometer tests and between oedometer tests and field observations. This investigation led to the recommendation
of the CRS-test as a routine test for soft clays and this method became a standard test at the Swedish Geotechnical Institute in 1975 and has also been used in many Swedish consulting firms.

b) Constant gradient test (CGI-test)

The constant gradient test is performed with constant pore pressure. In the CGI-test the strain rate should be regulated so that the pore pressure in bottom of the sample is kept constant. Due to test condition, the CGI-test is more complicated and slower than the CRS-test.

c) Continuous consolidation test (CC-test)

This test has mainly been developed at the Norwegian Institute of Technology. The CC-test is performed with a constant relation between the applied load and the pore pressure in the bottom of the sample. It requires the most complicated equipment. According to the Swedish point of view, if compared to the CRS-test, the main advantage of the CC-test is that it can automatically adjust the rate of strain to the tested sample. Norwegian experience found that for low plastic Norwegian clay the CC-test could be performed much faster than the CRS-test.

3.3 Determination of strength characteristics

In Sweden the undrained shear strength of soft clay has commonly been determined by laboratory fall-cone test (Fig. 19). Besides this test, the shear strength of soft clay can be determined by the direct shear test or the triaxial test.
Fig. 19. Laboratory fall-cone apparatus.

The fall-cone test was developed by the Geotechnical Commission of the Swedish State Railway between 1914 and 1922 and has been widely used in Sweden since then. It is a simple and rapid method for determining the undrained shear strength of both undisturbed and remoulded clays.

In a test the cone is usually placed in the stand of the apparatus in such a way that the tip of the cone just touches the surface of the soil sample. The cone is then dropped freely into the soil and the depth of penetration measured.

Different cones have been used and nowadays the following cones are standard for different range of the shear strength:

- 400 g $30^\circ$
- 100 g $30^\circ$
- 60 g $60^\circ$
- 10 g $60^\circ$
3.3.1 Determination of undrained shear strength by fall-cone tests

The cones 60°6 and 100°30° are often used today. The 60°6-cone was chosen as unit cone and the relative strength number for 10 mm penetration with this cone was set = 10. The strength number for a completely remoulded sample was indicated by $H_1$, and for a partly disturbed by $H_2$ and for an undisturbed by $H_3$. Comparisons with direct shear tests and landslides have resulted in the following relation between the undrained shear strength $\tau_{fu}$ in kN/m$^2$ and the strength value $H_3$.

$$\tau_{fu} = 10H_3(32+0.073H_3) \quad \text{(Skaven-Haug)}$$
$$\tau_{fu} = 10H_3(40+0.055H_3) \quad \text{(Hultin)}$$
$$\tau_{fu} = 10H_3(36+0.064H_3) \quad \text{(SGI)}$$

The SGI relation is a mean value of the two relations mentioned above (Skaven-Haug's and Hultin's).

The evaluation of undrained shear strength is nowadays often made according to Hansbo (1957).
\[ \tau_{fu} = k \cdot g \cdot \frac{m}{i^2} \left( 1 + \frac{a}{i} \right) \]  

(Hansbo, 1957)

where

\( \tau_{fu} \) = undrained shear strength, kPa
\( k \) = constant (primarily depending on the cone angle)
\( g \) = 9.81 m/s²
\( m \) = mass of cone, g
\( i \) = cone penetration, mm
\( a \) = free height of fall, mm

In Hansbo's formula the value of \( k \) depends primarily on the cone angle. The evaluation of \( k \) has been made by calibration against results from field vane tests for undisturbed clays and from laboratory vane tests for remoulded clays. Fig. 20 present the \( k \)-value for Swedish clays taken with standardized piston sampler (Hansbo, 1957).

![Graph](image)

Fig. 20. \( k \)-value for Swedish clays taken with standardized piston sampler (Hansbo, 1957).

According to Karlsson (1962), \( k \)-values for remoulded soils by calibration from vane tests in laboratory are evaluated as follows.
<table>
<thead>
<tr>
<th>cone angle</th>
<th>k-value</th>
</tr>
</thead>
<tbody>
<tr>
<td>30°</td>
<td>0.27</td>
</tr>
<tr>
<td>60°</td>
<td>0.80</td>
</tr>
</tbody>
</table>

The recommended values for \( k \) in Sweden are \( k = 0.25 \) for cone angle 30° and \( k = 1.0 \) for cone angle 60°.

### 3.3.2 Influence of incorrect height adjustment

In a test the cone should be placed in the stand of the apparatus in such a way that the tip of the cone just touches the surface of the soil sample. The tests are easy and simple but it is very important to make the correct height adjustment, because it is a main source of error of the fall-cone method.

Any incorrect height adjustment can be corrected in the tests. There are three cases of height adjustment.

**a) Correct height adjustment (standard test)**

The undrained shear strength in standard test is calculated according to the formula:

\[
\tau_f \mu = k \cdot g \cdot \frac{m}{\tilde{\nu}}
\]

where \( i = \) correct cone penetration.

**b) Initial penetration**

\[
\tau_f \mu = k \cdot g \cdot \frac{m}{\tilde{\nu} + \Delta i_0 + \Delta i_2 + \Delta \tilde{\nu}}
\]

\( i_0 = \) initial penetration
\( i_2 = \) determined cone penetration
\( i_2 = i + \Delta i \)
c) Initial height of fall

\[ \tau_{fu} = k \cdot g \cdot \frac{m}{\kappa_1} \left( 1 + \frac{a}{\kappa_1} \right) \]

where

\[ a = \text{initial height of fall} \]
\[ a + i_1 = \text{determined cone penetration} \]
\[ a + i_1 = i + \Delta i \]

As seen above, the most important part of the fall-cone method is the height adjustment (height of fall of the cone). According to Broms (1982) a deviation of only 0.3 mm can lead to an error of about 2 to 3% with respect to the shear strength of the soil when the water content is 100% and the penetration is 7 mm.

In order to obtain the \( \tau_{fu} \)-value quickly, a table has been prepared (Table 4). The prepared table is applicable for different cones with different penetration (different range of shear strength). By choosing a suitable cone, the fall-cone test can be used to determine the undrained shear strength in a range of 0.060 to 95 kPa.
Table 4.

<table>
<thead>
<tr>
<th>Kon- typ</th>
<th>Sjunk- mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>100 g</td>
<td></td>
</tr>
<tr>
<td>30°</td>
<td>2, 95,6</td>
</tr>
<tr>
<td></td>
<td>3, 68,0</td>
</tr>
<tr>
<td></td>
<td>4, 47,8</td>
</tr>
<tr>
<td></td>
<td>5, 35,5</td>
</tr>
<tr>
<td></td>
<td>6, 27,9</td>
</tr>
<tr>
<td></td>
<td>7, 21,9</td>
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<tr>
<td></td>
<td>8, 17,1</td>
</tr>
<tr>
<td></td>
<td>9, 13,7</td>
</tr>
<tr>
<td></td>
<td>10, 10,8</td>
</tr>
<tr>
<td></td>
<td>11, 8,80</td>
</tr>
<tr>
<td></td>
<td>12, 7,44</td>
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<td></td>
<td>13, 6,30</td>
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<td></td>
<td>14, 5,30</td>
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<td></td>
<td>15, 4,70</td>
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<td>18, 3,20</td>
</tr>
<tr>
<td></td>
<td>19, 2,60</td>
</tr>
<tr>
<td>60°</td>
<td>4, 6,90</td>
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<tr>
<td></td>
<td>5, 5,95</td>
</tr>
<tr>
<td></td>
<td>6, 4,52</td>
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<td>7, 3,44</td>
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<tr>
<td></td>
<td>8, 2,60</td>
</tr>
<tr>
<td></td>
<td>9, 2,10</td>
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<td></td>
<td>10, 1,70</td>
</tr>
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<td></td>
<td>11, 1,40</td>
</tr>
<tr>
<td></td>
<td>12, 1,19</td>
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<tr>
<td></td>
<td>13, 1,00</td>
</tr>
<tr>
<td></td>
<td>14, 0,850</td>
</tr>
<tr>
<td></td>
<td>15, 0,700</td>
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<tr>
<td></td>
<td>16, 0,600</td>
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<tr>
<td></td>
<td>18, 0,450</td>
</tr>
<tr>
<td></td>
<td>19, 0,400</td>
</tr>
<tr>
<td>10°</td>
<td>5, 1,10</td>
</tr>
<tr>
<td></td>
<td>6, 0,780</td>
</tr>
<tr>
<td></td>
<td>7, 0,583</td>
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<tr>
<td></td>
<td>8, 0,449</td>
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<tr>
<td></td>
<td>9, 0,360</td>
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<tr>
<td></td>
<td>10, 0,290</td>
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<tr>
<td></td>
<td>11, 0,242</td>
</tr>
<tr>
<td></td>
<td>12, 0,203</td>
</tr>
<tr>
<td></td>
<td>13, 0,173</td>
</tr>
<tr>
<td></td>
<td>14, 0,143</td>
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<td></td>
<td>15, 0,120</td>
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<tr>
<td></td>
<td>16, 0,102</td>
</tr>
<tr>
<td></td>
<td>17, 0,087</td>
</tr>
<tr>
<td></td>
<td>18, 0,074</td>
</tr>
<tr>
<td></td>
<td>19, 0,066</td>
</tr>
</tbody>
</table>
3.3.3 Shear strength in direct shear tests

In Sweden the direct shear tests are often used to determine the shear strength by a modified SGI oedometer (Fig. 21). In this apparatus the sample in the tests is first consolidated and then sheared to failure. The drained or undrained shear strength can be determined in direct shear tests. The test sample has a diameter of 50 mm and a height of 10 mm and 20 mm in drained and undrained tests respectively. During the tests horizontal stress, horizontal deformation and vertical deformation are measured.

Fig. 21. The SGI shear apparatus.

From the test results the shear stress versus angular distortion is plotted for every vertical stress as seen in Fig. 22.
Fig. 22. Shear stress, angular distortion and vertical deformation in consolidated drained direct shear tests (a) and in consolidated undrained direct shear tests (b).

The shear stress at failure is evaluated as a peak shear stress. In the case no peak the shear stress at failure is evaluated according to Swedish practice as the shear stress at an angular distortion of 0.15 radians.

The shear strength obtained from direct shear tests is plotted as in Fig. 23.
\( m_v = \text{coefficient of volume compressibility (volume change, volume decrease)} \ (m^2/kN) \)

The determination of the coefficient of consolidation from the oedometer curve is based on the two following quantities:

Time factor \( T_v \) is calculated by the formula:

\[
T_v = \text{Terzaghi time factor} = \frac{c_v \cdot t}{d^2}
\]

where \( d \) is the drainage path length. In the laboratory \( d = \) sample thickness with "one-way" drainage and \( d = \) half of sample thickness with "two-way" drainage. In the calculation, \( T_v \)-value is dimensionless.

Average degree of consolidation \( U_v \):

The average degree of consolidation is the ratio of the settlement at a definite time, \( t \), to the ultimate settlement and is expressed by the equation:

\[
U_v = \frac{s_t}{s_{ult}} = \frac{\text{settlement at } t}{\text{ultimate settlement}}
\]

The \( U_v-T_v \) relationship is founded as follows:

<table>
<thead>
<tr>
<th>( U_v(%) )</th>
<th>10</th>
<th>20</th>
<th>30</th>
<th>40</th>
<th>50</th>
<th>60</th>
<th>70</th>
<th>80</th>
<th>90</th>
</tr>
</thead>
<tbody>
<tr>
<td>( T_v )</td>
<td>0.008</td>
<td>0.031</td>
<td>0.071</td>
<td>0.126</td>
<td>0.197</td>
<td>0.287</td>
<td>0.403</td>
<td>0.567</td>
<td>0.848</td>
</tr>
</tbody>
</table>

The coefficient of consolidation \( c_v \) can be evaluated by one of the two following methods:
Casagrande method

In the Casagrande method, the coefficient of consolidation $c_v$ is determined from the time-strain curve with strain in a linear scale and time in log scale (Fig. 13). In Fig. 13 is seen how $U_0$ and $U_{100}$ are constructed. $U_{100} = 100\%$ is constructed as the intersection between the tangent to the curve at its point of inflexion and the extension of the straight end part of the curve. From $U_0$ and $U_{100}$, $\varepsilon_{50}$ at $U_{50} = 50\%$ is calculated and $t_{50}$ is constructed. Finally the coefficient of consolidation $c_v$ is calculated by the following formula:

$$c_v = T_{50} \frac{d^2}{t_{50}}$$

where

- $d$ = length of drainage path
- $T_{50}$ = Terzaghi time factor

For oedometers with samples drained at both ends and $d = H_0(1-\varepsilon_{50})/2$ where $H_0$ is initial sample height, the time factor $T_{50} = 0.197$. Thus the coefficient of consolidation $c_v$ is calculated as:
\[ d_2 = 0.197 \frac{d^2}{t_{50}} \]

Taylor method

![Taylor construction of \(c_v\).](image)

As in the Casagrande method, the coefficient of consolidation \(c_v\) in the Taylor method is also determined from a time-strain curve with strain in linear scale but with time in square root scale (Fig. 14). As seen in Fig. 14 \(\bar{U}_0 (\varepsilon_0)\) is determined as the beginning point of the curve. At \(\bar{U}_0 (\varepsilon_0)\) a tangent to the curve is drawn. A free horizontal line \(z\) is drawn that intersects the tangent of the curve at a certain point. The distance 0.15 \(z\) is calculated and the line A is constructed and \(\bar{U}_{90} (\varepsilon_{90})\) is taken from the intersection between the line A and the curve. Now the parameter \(\bar{U}_{50} (\varepsilon_{50})\), \(\bar{U}_{100} (\varepsilon_{100})\) and \(t_{90}\) can be constructed and calculated.

The coefficient of consolidation \(c_v\) is determined by the following formula:

\[ c_v = T_{90} \frac{d^2}{t_{90}} \]
For oedometer with samples drained from both ends

\[ d = H_0 (1 - \epsilon_{50}) / 2 \]

where \( H_0 \) is the initial sample height, and the time factor \( t_{90} = 0.848 \). In this case the coefficient of consolidation is calculated as:

\[ c_v \approx 0.848 \frac{d^2}{t_{90}} \]

The coefficient of consolidation \( c_v \) is determined for every load step. According to Bjerrum, for small load increments up to \( \sigma_d \) the \( c_v \)-value can be determined by the Taylor method and for a load exceeding \( \sigma_d \) both the Casagrande and the Taylor method can be used. The \( c_v \)-values calculated by the above methods should then be plotted against the effective vertical stress, see Fig. 15.

![Fig. 15. The coefficient of consolidation observed in a consolidation test plotted against the vertical load (after Janbu, 1969).](image)

As seen in Fig. 15, the range of the \( c_v \)-variation is considerable. Therefore, the \( c_v \)-value to be applied on a practical problem has to be chosen in the appropriate stress range (Bjerrum, 1973).
It has been found that the value of the coefficient of consolidation is affected by temperature (Bjerrum (1973), Larsson (1981), and others). Therefore, in order to determine an accurate $c_v$-value the test should be performed at constant temperature in a temperature-controlled room, if possible at the same temperature as that of the in situ soil.

As seen above, both methods are based on types of settlement. There are three types of settlement that are usually termed (Fig. 16):

- immediate settlement (compression)
- primary settlement (compression)
- secondary settlement (compression).

![Time-settlement curve](image)

**Fig. 16.** The three parts of the time-settlement curve.

As time-settlement curves are plotted in different scales, the identification of these three characteristic settlements in both methods is different.

In the Taylor method, the point where the primary compression is thought to begin is obtained by extending the tangent to the curve back to the compression axis ($U_0$, $\varepsilon_0$) assuming that the immediate settlement occurs fairly rapidly and is usually
Research and experiments have shown that for soft clays, the relation between shear strength and vertical stress changes at the preconsolidation pressure in both undrained and drained tests. It has also been found that there is a second breaking point at half of the preconsolidation pressure in the relation between drained shear strength and vertical stress ($\tau_{fd}-\sigma'$ curve) as seen in Fig. 23.

a) Generalized model for shear strength of soft clay in direct shear tests

The shear strength from direct shear tests can generally be expressed by the equation:

$$\tau = \sigma' \tan \phi'$$

where

$\sigma'$ = vertical effective stress

$\phi'$ = effective angle of friction

According to Larsson (1977), the effective angle of friction can be evaluated by the formula:

$$\phi' = \phi_p - \alpha$$

where

$\phi'$ = effective angle of friction

$\phi_p$ = angle of interparticle friction

$\alpha$ = angle between direction of particle displacement and horizontal plane ($\alpha = \arctan \frac{d\varepsilon_v}{d\varepsilon_H}$)

The effective angle of friction $\phi'$ depends on $\phi_p$ and $\alpha$ and also on stress level.

Generalized drained shear strength

According to Larsson (1977), a failure line in the relation between drained shear strength in direct
shear and effective vertical pressure has the shape as seen in Fig. 25 for three types of clay:

- normally compressible soft clay has $\omega_n \approx \omega_L \approx 70\%$ and $s_t \approx 15$ and a clay content of about 60%
- clay with low compressibility, sensitivity and rapidity (low rapidity means that the structure of the clay is insensitive to deformations and vibrations and a lot of work is required to break it down, Söderblom et al, 1974)
- highly sensitive clay with high rapidity often has a $\omega_n > \omega_L$.

![Fig. 24. Generalized drained shear strength in direct shear (after Larsson, 1977).](image)

As seen in this figure the shear strength directly depends on the compressibility of the clay and the stress level. For three types of clay there are two breaking points on a failure line, one at 0.5 $\sigma_C'$ and another at $\sigma_C'$ ($\sigma_C'$ is the preconsolidation pressure). Research and practice have found that all failure lines for vertical stress below 0.5 $\sigma_C'$ are drawn as straight lines through origo with an angle of about 30° without great errors.
For the effective stresses higher than the pre-consolidation pressure, the drained shear strength can be calculated by the formula:

\[ \tau_{fd} = \sigma' \tan \phi \]

where

- \( \tau_{fd} \) = drained shear strength, kPa
- \( \sigma' \) = effective stress, kPa
- \( \phi \) = effective angle of friction, degree

**Generalized undrained shear strength**

The undrained shear strength that can be mobilized in soft clay is shown in Fig. 25. In undrained shear the change in pore pressure during the test affects the effective stresses. According to Larsson (1977), the pore pressure will decrease in the sample consolidated under vertical stress lower than about 0.45 \( \sigma'_c \) and the pore pressure will increase in the sample consolidated under higher vertical stress.

![Fig. 25. Undrained strength in direct shear tests (after Larsson, 1977).](image-url)
b) Shearing rate

The direct shear tests are usually performed with the shearing as a horizontal movement of the upper part of the sample. In Sweden the following shearing rate is normally used for undrained and drained tests with stepwise loading in direct shear tests, Table 5.

Table 5. The shearing rate in direct shear tests for every normal stress, $\sigma'$.  

<table>
<thead>
<tr>
<th>Test</th>
<th>Horizontal deformation of sample</th>
<th>Shearing rate</th>
</tr>
</thead>
<tbody>
<tr>
<td>Undrained tests</td>
<td>$0-0.5$ mm</td>
<td>$\Delta \tau = \sigma'/20$ each $30'$</td>
</tr>
<tr>
<td></td>
<td>$&gt;0.5$ mm</td>
<td>$\Delta \tau = \sigma'/40$ each $15'$</td>
</tr>
<tr>
<td>Drained tests</td>
<td>$0-0.25$ mm</td>
<td>$\Delta \tau = \sigma'/20$ each $30'$</td>
</tr>
<tr>
<td></td>
<td>$&gt;0.25$ mm</td>
<td>$\Delta \tau = \sigma'/40$ each $15'$</td>
</tr>
</tbody>
</table>

c) Normal stresses

It is known that the direct shear strength for soft clay directly depends on the compressibility of the clay and the stress level. The preconsolidation pressure $\sigma_0'$ of the tested soil is required in the tests and in the interpretation of the test results.

That the relation between shear strength and normal stress changes at a stress of $0.5 \sigma_0'$ and $\sigma_0'$ is generally accepted for soft clay. Before shear tests the $\sigma_0'$-value for the tested soil should be known. The $\sigma_0'$-value is necessary to decide suitable consolidation stresses in the tests (also the stresses at which the samples are sheared).

To enable the plotting of the failure line as a relation between shear strength and normal stress samples of the tested clay should be sheared to
failure under at least four normal stresses: 0.3 σ₀', 0.6 σ₀', 0.85 σ₀' and 1.5 σ₀' (where σ₀' is the preconsolidation pressure). The failure line that can be plotted from drained direct shear tests is shown in Fig. 26.

![Diagram of failure line](image)

Fig. 26. The failure line drawn from drained direct shear tests.

In Fig. 26 the empirical finding that the failure line for normal stresses between 0 and 0.5 σ₀' is fairly straight and passes through the origin is used together with the knowledge that the failure line for normal stresses above σ₀' is straight and its extension passes through the origin.
4. PRACTICAL SIGNIFICANCE OF THE PRECONSOLIDATION PRESSURE

The preconsolidation pressure has great significance in the practice of geotechnical problems. The most important practical application of the preconsolidation pressure is in connection with settlement analyses and geological investigation. The role of $\sigma'_c$ in the geological investigation has been presented above (part: classification and identification of soft clays).

The preconsolidation pressure $\sigma'_c$ plays its important role in settlement analyses. The following example presented by Casagrande (1936) will illustrate the role of $\sigma'_c$ in settlement analysis (Fig. 27).

A clay layer has been compressed at one time in the history by an overburden pressure of 3 kg/cm$^2$ which later was reduced by for instance erosion to the present overburden pressure of 1 kg/cm$^2$. If a building with a load of 1 kg/cm$^2$ now is set up on this clay, the compression under the building load ($\Delta_1$) will take place along the re-compression curve from point B to C. If the preconsolidation pressure (in this example $\sigma'_c = 3$ kg/cm$^2$) is not taken into account and the settlement is calculated based on the present overburden pressure ($\sigma'_c = 1$ kg/cm$^2$) the compression from the building load would then follow the virgin compression curve from D to E equal to $\Delta_2$. As seen in Fig. 27 $\Delta_2 > \Delta_1$ and according to the author $\Delta_2$ may be five to ten times greater than $\Delta_1$.

From the above example it is clear that in connection with settlement analysis it is necessary to carefully study the preconsolidation pressure.

Fig. 27. Settlement with preconsolidation pressure.
5. SETTLEMENT CALCULATION

The classical method for calculation of settlement on soft clay is based on the assumption that the clay is normally consolidated ($\sigma'_o = \sigma'_f$). Therefore the settlement is calculated by the classical equation:

$$
\delta = \sum \frac{C_o}{1+e} \log \frac{\sigma'_o + \Delta \sigma}{\sigma'_o} \Delta z
$$

where

- $\Delta z$ = thickness of the individual clay layer
- $\sigma'_o$ = effective overburden pressure
- $\Delta \sigma$ = vertical load increment
- $C_o$ = compression index
- $e'_o$ = initial void ratio of clay.

According to Bjerrum (1972, 1973) the settlement of soft clay calculated by the classical equation is too large, because the clay in fact is lightly or heavily overconsolidated.

In this method, the compression index $C_o$ is calculated from $e$-$\log \sigma'$ curve (see Fig. 11a):

$$
C_o = \frac{\Delta e}{\log \frac{\sigma'_o + \Delta \sigma}{\sigma'_o}} = \frac{\Delta e}{\log \sigma'}
$$

In order to avoid the determination of the void ratio, the compression index $\varepsilon_2$ is used in Sweden. The compression index $\varepsilon_2$ is the relative compression of the sample and it is evaluated from compression-log pressure curve (see Fig. 11b).

The relation of these compression indices is:

$$
\varepsilon_2/\log 2 = C_o/1 + e'_o
$$

The settlement based on the compression index $\varepsilon_2$ is therefore calculated by the formula:
The above calculation of settlement is based on the assumption that the oedometer curve is linear in a $\varepsilon$-$\log\sigma'$ curve as well as in a $\varepsilon$-$\log\sigma'$ curve for stresses higher than the preconsolidation pressure. However, for some clays this assumption is not valid (Larsson, 1977, 1981). In this case the settlement calculated by the above formula is not correct, as the method for evaluating the compression indices ($C_o$ and $\varepsilon_2$) is not suitable.

In the cases, when the oedometer curve is not linear for stresses higher than the preconsolidation pressure, the tangent modulus $M$ is often used for expression of the soil compressibility (Larsson, 1981, Janbu et al, 1967). The method for evaluating the tangent modulus $M$ is described in this report, see part 4.

The settlement with the tangent modulus theory is calculated by the formula:

$$\delta = \sum_o^z \frac{\Delta\sigma'}{M} \Delta z = \sum_o^z \left[ \frac{1}{m^\beta} \frac{(\sigma_o' + \Delta\sigma')^\beta - (\sigma_o')^\beta}{\sigma_o'} \right] \Delta z$$

in which the tangent modulus $M$ is calculated by the formula:

$$M = m \sigma_o' (\frac{\sigma_o'}{\sigma_o'})^{1-\beta}$$

where

$m$ = modulus number

$\beta$ = pressure exponent

$\sigma_o'$ = effective vertical pressure

$\sigma_o'$ = reference pressure, usually 100 kPa (Andréasson et al, 1973)
In the case $\beta = 0$ the settlement is then calculated by the following formula:

$$\delta = \sum \frac{1}{m} \ln \frac{\sigma' + \Delta \sigma'}{\sigma_0'} \Delta z$$

and the modulus number $m = \frac{0.69}{\varepsilon_2}$

The above formulas in the calculation of settlements are applicable to normally consolidated soft clay with $\sigma_0' = \sigma_0$ (where $\sigma_0'$ is the effective overburden pressure and $\sigma_0$ is the preconsolidation pressure). In the calculation of settlement for overconsolidated clay ($\sigma_0' > \sigma_0$) the $\sigma_0'$ is used instead of $\sigma_0$ in the formulas.
6. DESIGN PARAMETERS OF SOFT CLAYS

The shear strength of a soil is a basic parameter used in stability calculation for many structures, such as:

- embankments on soft clay,
- slopes,
- foundation, etc.

For this reason, many methods and apparatus have been developed for determining the strength properties. The choice of suitable methods and apparatus in both laboratory and field tests is very important. It has been found though, that a more important problem is that of how to use the geotechnical properties in design. This problem is generally concerned with soft clays and has become an interest for many researchers.

Generally the soft clays are very sensitive to structural disturbance. The disturbances of samples occur in the process of boring, sampling, transportation, storage and testing. The samples brought into the laboratory are therefore seldom undisturbed. The results of laboratory soil tests are affected by sample disturbance. For laboratory technicians who test soil samples and for engineers who analyze geotechnical problems, it is important to know how design parameters obtained from laboratory tests are influenced by sample disturbance and which parameters are sensitive to sample disturbance. Many researchers have found that for soils, especially for soft cohesive soils, the deformation and strength properties are sensitive to sample disturbance.

In order to limit the sample disturbance (that means to increase the sample quality), many different methods and samplers for sampling in soft cohesive soils have been developed. A description of different methods for
taking samples in soft cohesive soils was presented in International Manual for Sampling of Soft Cohesive Soils recommended by the Sub-committee on Soil Sampling, International Society for Soil Mechanics and Foundation Engineering, Tokyo, 1981. On the other hand, a great number of research and tests on the quality in soil sampling and the determination of effects of sample disturbance on geotechnical properties have been carried out. All of this work has been carried out to obtain real geotechnical properties of soft cohesive soils. For this purpose, numerous observations from failures of slopes and embankments and comparison of the test results in stability calculation of embankments and foundations have been performed. From this work different methods for correlation of geotechnical properties have been introduced.

Some methods for correction of undrained shear strength of soft clays are presented below. The effects of sample disturbance on geotechnical properties of soft clays are presented in report No 3.

6.1 Correction of undrained shear strength of soft clays

The undrained shear strength of soft clays is usually determined in field vane tests or in laboratory tests. The corrections of undrained shear strength of soft clays (normally consolidated or slightly overconsolidated) according to field vane tests and fall-cone tests is summarized below.

Many observations from failures of slopes, embankments and foundations on soft clay have shown that the undrained shear strength measured by field tests or laboratory tests on soft clays are often too high (sometimes too low). The theoretical calculations of factors of safety have also showed this. In stability calculation the undrained shear strength of soft clays
measured by field vane tests and laboratory fall-cone or vane tests therefore should be corrected in order to correspond to the real shear strength of these clays. The correction (or reduction) of undrained shear strength is made as follows.

If the undrained shear strength measured by field vane test and fall-cone test is expressed in \( \tau_v \) and \( \tau_k \) respectively, the undrained shear strength to be used in stability calculations \( \tau_{fu} \) should be reduced by the reduction factor \( \mu \):

\[
\tau_{fu} = \mu \tau_v \quad \text{or} \quad \tau_{fu} = \mu \tau_k
\]

The reduction factor \( \mu \) is less than unity (1) and depending on soil type, state of soil etc.

There exists different methods for correcting the undrained shear strength of soft clays. The methods for correcting the undrained shear strength have developed in two directions. In the first direction, one attempts to correct the undrained shear strength by modelling field behaviour via consolidated-undrained tests performed with different stress histories and modes of failures. The second direction develops empirical correlation among soil type, the in situ test methods and the type of stability calculation. The methods based on model require complicated equipments. Therefore methods based on empirical correlation seem to be preferred by many engineers.

Empirical methods for correction of undrained shear strength are based on some physico-mechanical properties of clays. Almost every method uses soil properties such as liquid limit, plasticity index, ratio of vane shear strength to the effective overburden pressure, overconsolidation ratio as parameters in empirical relation. Representative for these methods are
Bjerrum, Hansbo, SGI and others. Other authors such as Pilot, Aas use the relationship between the theoretical factor of safety and the empirical factor of safety at failure. In this method, physico mechanical properties are used as a basis for the correction and should then be calculated as a mean value for the critical slip surface. It should be noted that the plasticity index $I_p$ internationally has been commonly used as a parameter in empirical relations, but in Swedish practice the liquid limit $w_L$ is often used. Corrections of undrained shear strength are often made for organic and high plastic clays but according to some authors corrections should be made also for inorganic clays.

6.1.1 Methods for reducing undrained shear strength of soft clays measured by vane tests and fall-cone tests

In 1946 the Swedish Geotechnical Institute (SGI) recommended that the reduction factor $\mu$ for undrained shear strength from fall-cone test should be based on the organic content of the clay:

$$\mu = 0.80 \text{ for organic clay}$$
$$\mu = 0.60 \text{ for gyttja (ooze)}$$

SGI (1969) found that the reduction should be made not only for fall-cone tests but also for vane tests and the same reduction factor $\mu$ should be used for both $\tau_k$ and $\tau_v$ based on the liquid limit $w_F$. Table 6 shows the $\mu$-value for undrained shear strength measured in fall-cone tests or field vane tests recommended by SGI (Broms, 1972).
Table 6. Reduction factor for $T_k$ or $T_v$ recommended by SGI (Broms, 1972).

<table>
<thead>
<tr>
<th>Cone liquid limit $w_L$ (%)</th>
<th>Reduction factor $\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>80-100</td>
<td>0.90</td>
</tr>
<tr>
<td>100-120</td>
<td>0.80</td>
</tr>
<tr>
<td>120-150</td>
<td>0.70</td>
</tr>
<tr>
<td>150-180</td>
<td>0.60</td>
</tr>
</tbody>
</table>

In 1972, 1973 Bjerrum, through his comprehensive study of reported failures in different parts of the world, found that the undrained shear strength measured by vane test in soft clay should be reduced. The Bjerrum's reduction factor is based on the plasticity index $I_p$. According to Helenelund (1977) this reduction factor can be expressed approximately by the formula

$$\mu = \frac{1.2}{I + I_p}$$

If this reduction factor is depending on the liquid limit $w_L$, according to Helenelund (1977) the relationship between $\mu$ and $w_L$ is as showed in Fig. 28. In this figure the reduction factor according to SGI (table 6) is also showed in order to compare these two reduction factors.

Fig. 28. Reduction factor as a function of the liquid limit.
As seen in Fig. 28, according to SGI, the reduction of undrained shear strength is made for clays with a liquid limit $w_L \geq 80\%$, whereas according to Bjerrum it is made even for clays with a liquid limit $w_L < 50\%$. On the other hand, there are differences between these reduction factors. For clays with a liquid limit less than about 120\% the differences between reduction factors recommended by SGI and Bjerrum are important. For higher plastic clays ($w_L > 120\%$) the SGI's and Bjerrum's reduction factors are of small difference.

Based on analyses of embankment failures Pilot (et al, 1972) found that the theoretical factor of safety at failure increases with both increasing liquid limit and increasing plasticity index. According to the author the factor of safety at failure can be expressed by the empirical equations

$$F_{sf} = 0.6 \; w_L + 0.7$$

$$F_{sf} = 0.7 \; I_p + 0.9$$

where

- $w_L =$ liquid limit (%/100)
- $I_p =$ plasticity index (%/100)

In this case, the reduction factor can be calculated as $\mu = 1/F_{sf}$. This reduction factor is applicable for undrained shear strength measured by vane tests.

$$\mu = \frac{1}{F_{sf}} = \frac{1}{0.6 \; w_L + 0.7}$$

$$\mu = \frac{1}{F_{sf}} = \frac{1}{0.7 \; I_p + 0.9}$$

From the above equations it is clear that $\mu = 1$ when the equations
\[ 0.6 \omega_L + 0.7 = 1 + \omega_L = 50\% \]

and \[ 0.7 I_p + 0.9 = 1 + I_p = 14.3\% \]

If the reduction factors are considered equal for undrained shear strength measured in fall-cone tests and vane tests and the reduction factors \( \mu \) are compared, according to different authors the difference in these \( \mu \)-values can be found. On the other hand we can find the limits of the consistency of clay where the undrained shear strength has to be reduced or not (Table 7). Table 7 shows a lower limit of consistency of clay where the reduction factor \( \mu \) should be made according to SGI, Bjerrum, Pilot.

Table 7. The reduction factor \( \mu \) should be made for a lower limit of consistency \( (\omega_L, I_p) \).

<table>
<thead>
<tr>
<th>Author</th>
<th>Reduction when</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \omega_L ) (%)</td>
</tr>
<tr>
<td>SGI (1972)</td>
<td>&gt;80</td>
</tr>
<tr>
<td>Bjerrum (1972, 1973)</td>
<td>&gt;50</td>
</tr>
<tr>
<td>Pilot (1972)</td>
<td>&gt;50</td>
</tr>
</tbody>
</table>

The ratio of undrained shear strength from vane tests to effective overburden pressure \( (\tau_v/\sigma_o) \) is also used for the calculation of the reduction factor \( \mu \).

In 1976 Aas (et al) found that a linear relationship between the factor of safety at failure and the \( \tau_v/\sigma_o \)-ratio can be expressed by the formula:

\[ F_{sf} = 2.7 \tau_v/\sigma_o + 0.38 \]

In this case, the reduction factor \( \mu \) will be
\[ \mu = 1/F_{8f} = \frac{1}{2.7 \frac{\tau_v}{\sigma'_o} + 0.38} = \frac{2.6}{1 + 7 \frac{\tau_v}{\sigma'_o}} \]

where
\[ \tau_v = \text{mean (uncorrected) vane shear strength} \]
\[ \sigma'_o = \text{mean effective overburden pressure} \]

Helenelund (1977) found by his comparison between different methods for reducing the undrained shear strength of soft clay that:

- The ratio \( \tau_v/\sigma'_o \) seems preferably to be used as a basis for reduction, for example according to the formula

\[ \mu = \frac{2.6}{1 + 7 \frac{\tau_v}{\sigma'_o}} \]

- The use of a reduction factor is depending on the ratio \( \tau_v/\sigma'_o \) and the relationship between this ratio and the liquid limit \( w_L \) (Fig. 29).

![Figure 29. Relation between the \( \omega_L \)-scale and the \( \tau_v/\sigma'_o \)-scale in the reduction factor \( \mu \) (according to Helenelund, 1977).](image-url)
According to the author, the reduction factor can be found using the $w_L$-scale in Fig. 29 when

$$\frac{\tau_v}{\sigma_d'} < 0.10 + 0.25 \ \omega_L$$

If the $\tau_v/\sigma_d'$-ratio is greater, the $\mu$-value should be determined using the $\tau_v/\sigma_d'$-scale.

Instead of the liquid limit used in Table 6, the reduction factor $\mu$ is taken from Table 8 according to the formula

$$\mu = \frac{0.30}{\tau_v/\sigma_d'}$$

Table 8. Reduction factors as a function of the $\tau_v/\sigma_d'$-ratio.

<table>
<thead>
<tr>
<th>Ratio $\tau_v/\sigma_d'$</th>
<th>Reduction factor $\mu$</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.30 - 0.35</td>
<td>0.90</td>
</tr>
<tr>
<td>0.35 - 0.40</td>
<td>0.80</td>
</tr>
<tr>
<td>0.40 - 0.475</td>
<td>0.70</td>
</tr>
<tr>
<td>0.475 - 0.55</td>
<td>0.60</td>
</tr>
<tr>
<td>0.55 - 0.65</td>
<td>0.50</td>
</tr>
<tr>
<td>0.65 - 0.85</td>
<td>0.40</td>
</tr>
<tr>
<td>&gt; 0.85</td>
<td>0.30</td>
</tr>
</tbody>
</table>

Many relationships between the $\tau_v/\sigma_d'$-ratio and the liquid limit $\omega_L$ or the plasticity index $I_p$ have been proposed by authors, such as:

Skempton (1954) \quad \frac{\tau_v}{\sigma_d'} = 0.11 + 0.37 \ \omega_L

Hansbo (1957) \quad \frac{\tau_v}{\sigma_d'} = 0.48 \ \omega_L

Helenelund (1977) \quad \frac{\tau_v}{\sigma_d'} = 0.10 + 0.25 \ \omega_L

Fig. 30 shows the undrained shear strength determined by field vane test over preconsolidation pressure versus liquid limit according to different authors.
The same data in Fig. 30 plotted against the plasticity index are shown in Fig. 31.

Fig. 30. Undrained shear strength determined by field vane tests over preconsolidation pressure versus liquid limit (after Larsson, 1980).
Fig. 31. Undrained shear strength determined by field vane tests over preconsolidation pressure versus plasticity index (after Larsson, 1980).

Moreover, due to different other factors affecting the results from field tests and laboratory tests, some factors correcting undrained shear strength of soft clays should be considered:

- correction factor for the effect of time
- correction factor for anisotropy
- correction factor for progressive failure, etc.

All reduction factors or factors correcting different effects affecting undrained shear strength are concerned with the calculation of the economic optimum value of the factor of safety ($F_{opt}$). The factor of safety for earthworks has normally been recommended
as $F_s = 1.30-1.50$. The relationship between the factor of safety and different costs for construction ($B =$ construction costs, $R =$ risk of reconstruction cost, $M =$ risk of machine damages, $P =$ risk of human accidents) by a calculation of the construction costs of the Saima canal is shown in Fig. 32. The optimum factor of safety in this case is found, $F_{opt} = 1.50$. It should be noted that the optimum factor of safety depends on the $R$, $M$- and $P$-costs that in turn depend on different types of structures, during which period the failure occurs. The optimum factor of safety also depends on an analytic method (short-term stability or long-term stability). The $F_{opt}$-value applied in long-term stability may be much higher than one in short-term stability (Helenelund, 1977).

![Fig. 32. Determination of the optimum factor of safety $F_{opt}$. $R$, $M$- and $P$-costs are proportional to the probability of failure.](image-url)
The influence on shear strength reduction on the numerical value of the optimum factor of safety \( F_{opt} \) has been considered. Fig. 33 shows the economic optimum values of the factor of safety according to construction costs and costs of repair of a section of the Saima canal (at failure 2, Slunga et al., 1972).

As seen from Fig. 33 it is clear that the \( F_{opt} \)-values without reduction of undrained shear strength are higher than the ones with reduction. From this figure it is also seen that the \( F_{opt} \)-values depend on which costs that are taken into account.
7. REFERENCES


