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10th Asian Young Geotechnical Engineers Conference and 28th Annual International Conference on Geotechnical Engineering

"Geotechnics Revolution : Redefining the Future of Infrastructure"

Jakarta, 11-13 November 2024
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Theme :

"Geotechnics Revolution: Redefining the Future of Infrastructure"

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Why does Settlement Calculation often Miss-out by a Wide Margin?

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ABSTRACT In practice, it is often found that geotechnical settlement calculations are quite far off from reality. This error, whether in the calculation of embankment, shallow or deep foundation settlement, can be off by tens of percent, sometimes even up to tenfold. This paper discusses the factors that cause these errors, starting from the loading magnitude and stress distribution, errors in the testing and interpretation of settlement parameters, three-dimensional effects of pore water pressure dissipations, limitation of secondary compression parameter, constitutive models applied in carrying out calculations, the volume of soil tested in the laboratory compared to the volume of soil compressed by the load of the structure above it, soil samples disturbance, to the effect of structural rigidity. It cannot be denied that of all these factors, some can be avoided and some are unavoidable, but it is hoped that by knowing the causal factors where errors can arise, the weaknesses can be reduced so that the resulting settlement calculation can be more reliable.

KEYWORDS Settlement calculations; 3D Pore pressure dissipation; Data errors; Constitutive models

1 INTRODUCTION

When designing a geotechnical structure, a geotechnical engineer not only has to consider the stability or the safety factor of the structure. It is very often that he/she also must provide his/her best estimate on the final settlement magnitude or the settlement after a certain time period. While the geotechnical formulas to calculate settlement magnitude and duration seems easy and straight forward to apply and to provide output for the engineers, the calculated values often quite far off from the actual settlement taken place in the field.

In the author's 40 years experiences, the calculated settlement could be way off by three to tenfold compared to the reality. For example, in 1993, when designing a 33-story building, the author estimated that upon completion the building would settle by about 93 mm, a world prominent company estimated the settlement to be in the order of 330 mm. It was found that the wide discrepancies were due to the adopted settlement parameters. After a few rounds of discussion, both sides agreed on the parameters and finally gave a best estimate of around 90 mm for the settlement. Three years later when the building was completed, the actual settlement was less than 33 mm. So, the final estimate was off by about threefold.

For the serviceability of the structures under consideration, when the actual settlement is lower than calculated, it generally does not pose a problem. A much larger actual settlement than the estimated value will certainly have a negative effect on serviceability and in the worst-case scenario it can also have an impact on the safety of the structures itself and result in costly remedial work. This kind of case happened in North Jakarta where a building was undergoing more than 800 mm settlement, much larger than the 100 mm allowed by local standards.

This error can happen in any kind of geotechnical design, whether in the calculations of embankment, shallow or deep foundation settlement. Now, as asked by many, the question is why this settlement estimate can be way off from reality? This paper aims to elaborate on the factors that may cause this problem.

2 IMMEDIATE AND CONSOLIDATION SETTLEMENT FORMULAS

The theory of settlement calculation was first derived by Terzaghi (1943). The formulas are easily found in many geotechnical textbooks, e.g.: Taylor (1948), Terzaghi et al., (1996); Simon & Menzies (2001); Das, (2016) and many others. The formulas are:

To calculate immediate or elastic settlement, S_i :

$$S_i = qB\frac{1-\mu^2}{E}I_p \tag{1}$$

where q is the contact pressure at soil-foundation interface, B is the width of foundation, μ is the Poisson's ratio, E is the soil stiffness modulus, I_p is the influence factor.

To calculate consolidation settlement for normally consolidated soils, S_{cnc} :

$$S_{cnc} = \frac{c_c}{1+e_o} H_o \log\frac{\sigma'_{vo}+\Delta\sigma}{\sigma'_{vo}} \tag{2}$$

where c_c is the compression index, e_o is the initial void ratio, H_o is the thickness of consolidating soil layer, σ'_{vo} is the effective overburden pressure, $\Delta\sigma$ is the loading pressure.

To calculate consolidation settlement for over consolidated soils, S_{coc} (Figure 1.):

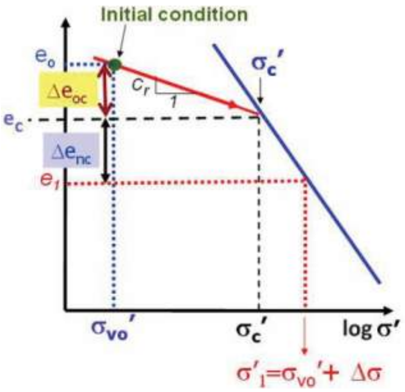

Figure 1. e – log σ′ for consolidation settlement calculation.

$$\text{for } \sigma'_{vo} + \Delta\sigma \leq \sigma'_c \rightarrow S_{coc} = \frac{c_r}{1+e_o} H_o \log\frac{\sigma'_{vo}+\Delta\sigma}{\sigma'_{vo}} \tag{3}$$

$$\text{for } \sigma'_{vo} + \Delta\sigma > \sigma'_c \rightarrow S_c = S_{coc} + S_{cnc} \tag{4}$$

$$S_{coc} = \frac{c_r}{1+e_o} H_o \log\frac{\sigma'_c}{\sigma'_{vo}} \tag{5}$$

$$S_{cnc} = \frac{c_c}{1+e_c} (H_o - s_{coc}) \log\frac{\sigma'_c+\Delta\sigma}{\sigma'_c} \tag{6}$$

where σ'_c is the preconsolidation pressure, c_r is the re-compression index, e_c is the void ratio at σ'_c.

To calculate time rate of consolidation settlement:

$$t = \frac{T_v H^2}{c_v} \tag{7}$$

where t is the consolidation time, T_v is the time factor, H is the thickness of drainage layer = H_o for single drainage and 0.5 H_o for double drainage, c_v is the coefficient of consolidation.

The formulas are simple, but the derivation of the parameters is not that easy, and those are the major contributing factors that result to the error in the prediction of settlement. Let's discuss those parameters.

3 SETTLEMENT PARAMETERS

3.1 Loading pressures, q and $\Delta\sigma$

When there is only one type of loading and one loaded area are considered, the contact pressure, q, and the loading pressure, $\Delta\sigma$, is relatively easy to determine. However, when many loading areas and types need to be considered, the interaction of these loaded area makes it difficult to determine the loading pressure, $\Delta\sigma$, at a particular layer of soil under consideration.

Boussinesq's or Westergaard's stress distribution formulas are often used to estimate the stress acting beneath a loaded area. Those formulas do not consider the properties of soil layers, in other words, the stiffnesses of layered soils are assumed to be the same. However, when the stiffnesses of the layered soils are different, the stress distribution is no longer the same as what is derived by using Boussinesq's formula. In such a case, Burmister (1945) formula must be used. Figure 2 shows an example of stress distribution of a two-layers soil system with different stiffness values (Note that Boussinesq's assumption is $E_1 = E_2$).

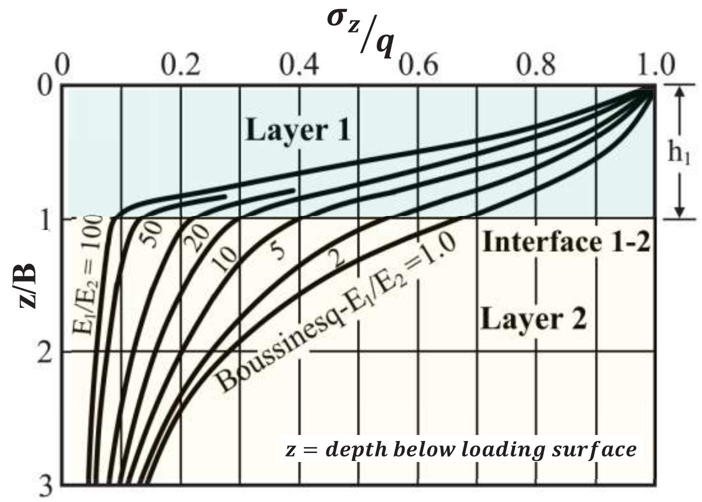

Figure 2. Stress distribution in layered soils (Burmister, 1945 – redrawn).

Manual calculation of stress distribution under numerous loadings, types, and layered soils, is almost impossible. The solution is to use geotechnical software capable of analyzing 3-dimensional stress distribution, and multi-layered soils, such as Settle 3D and other 3D geotechnical finite element software.

3.2 Poisson's Ratio, μ

In practice, Poisson's ratio of soil is never tested. The values are often adopted between 0.30 to 0.35 for drained condition and 0.50 for undrained condition. While the value of 0.50 for undrained condition is suitable, the value of drained condition may differ from reality. Values given by Poulos & Small (2000) shown in Table 1 may be adopted.

Table 1. Poisson's ratio of various soil (Poulos & Small)

Soil Type	Fast Loading	Slow Loading
Gravel	0.30	0.30
Sand	0.35	0.30
Silt and Silty clay	0.45	0.35
Stiff clay	0.45	0.25
Plastic clay	0.50	0.40
Compacted clay	0.45	0.30

3.3 Soil Stiffness, E

Soil stiffness is often not available in soil investigation reports. Most of the time, engineers estimated the stiffness values through correlation with SPT or CPT test results. For examples:

$$E(MPa) = (0.2 \sim 2.0)N \tag{8}$$

$$E(kPa) = (2.0 \sim 8.0)\ q_c(kPa) \tag{9}$$

where N is the SPT blow count and q_c is the CPT cone tip resistance

Such correlation can lead to wide variation of the derived E values, and in turn it gives large variation of the estimated settlement. Based on the author's experience, use of pressuremeter test yields a much better E value of the soil as this is the only in-situ test that directly can give the stress-strain curve (Gouw, 2017a).

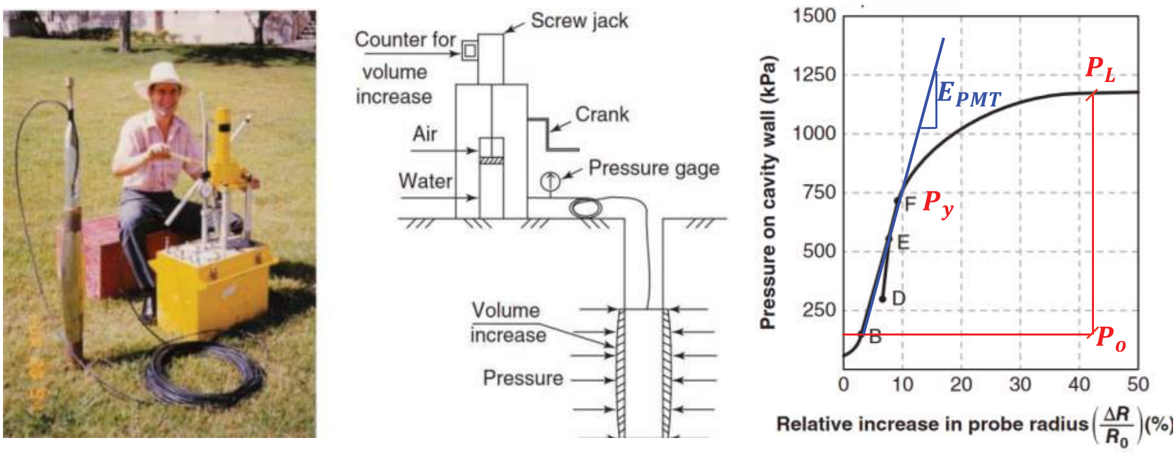

Figure 3. Derivation of soil stiffness, E, from pressuremeter test (modified after Briaud, 2013).

To minimize the error in the laboratory, it requires high quality undrained consolidation triaxial test with pore pressure measurement as shown in Figure 3. To obtain a representative E value, The CU triaxial tests must be carried out with the cell pressures arranged within the in-situ and the anticipated load-induced lateral soil effective pressure.

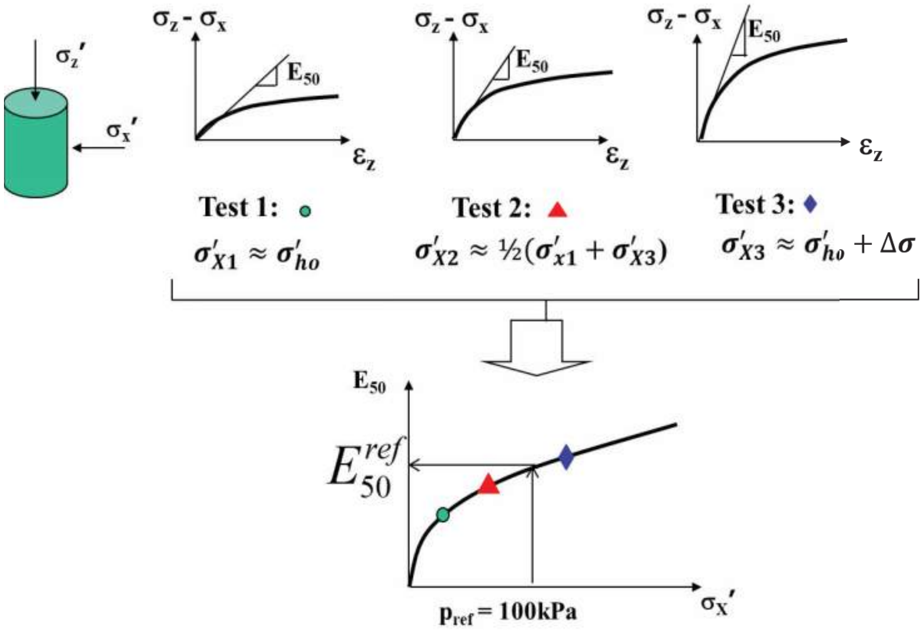

Figure 4. Derivation of soil stiffness, E, from CU triaxial test.

3.4 Preconsolidation Pressure, σ_c', and Compression Index, c_c

The pre-consolidation pressure parameter, σ_c', and compression index, c_c, are commonly derived by employing Casagrande's method (Casagrande, 1936). This method, created 88 years ago, is based on graphical procedures. The interpreter must determine the point of maximum curvature of an e-log σ graph, then draw a horizontal, a tangent, and a bisector lines through that point of maximum curvature. A linear line is then drawn at the normally consolidated (NC) part of the e-log σ' graph. The intersection point of the bisector with the NC linear line is the pre-consolidation pressure (Figure 5). Even when presented with the same data points, different people give different values of pre-consolidation pressure. Sometimes the difference between the lowest value and the highest values can be doubled or even quadrupled! Figure 6 shows the variation of σ_c' from the same e-log σ′ curve interpreted by different laboratory technicians.

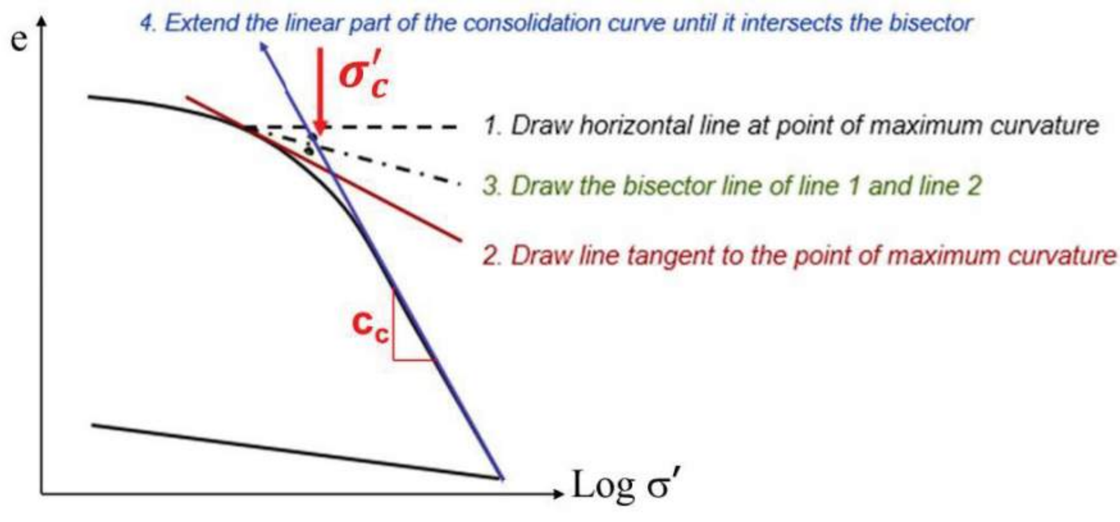

Figure 5. Determination of σ_c' and c_c by Casagrande method (Casagrande, 1936).

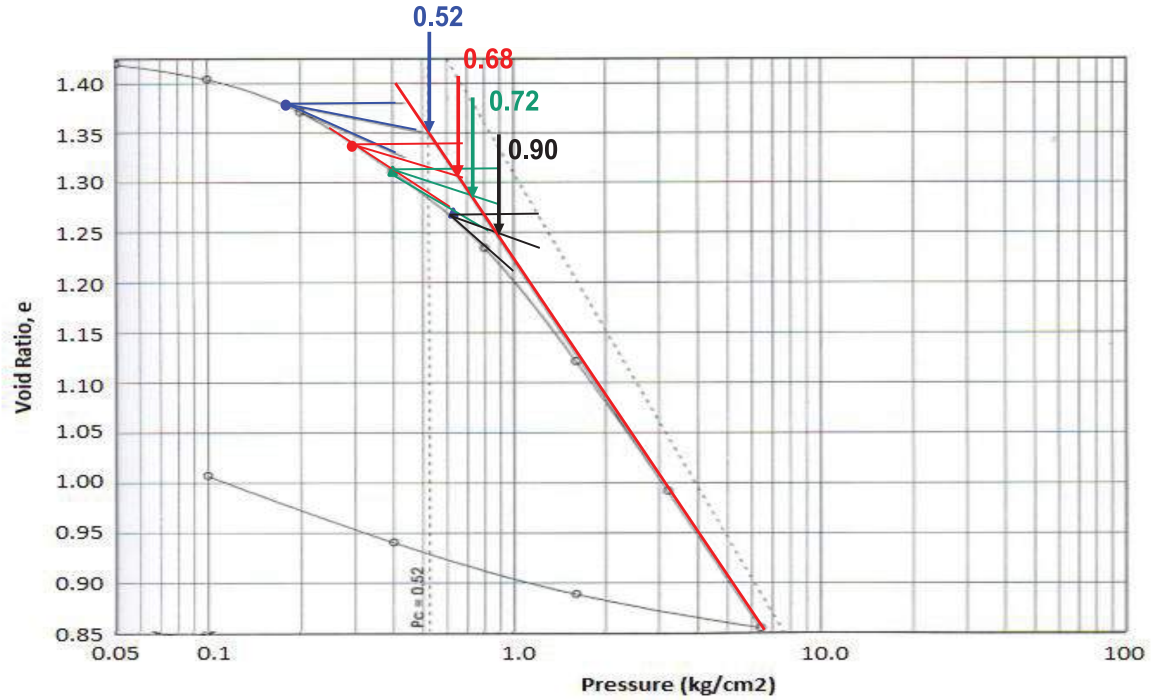

Figure 6. Variation of preconsolidation pressure σ_c' interpreted by different laboratory technicians.

It should be noted that the values of σ_c' and c_c produced by the Casagrande method also depend on the maximum load applied during the consolidation test. Figure 7 shows the variation in these values.

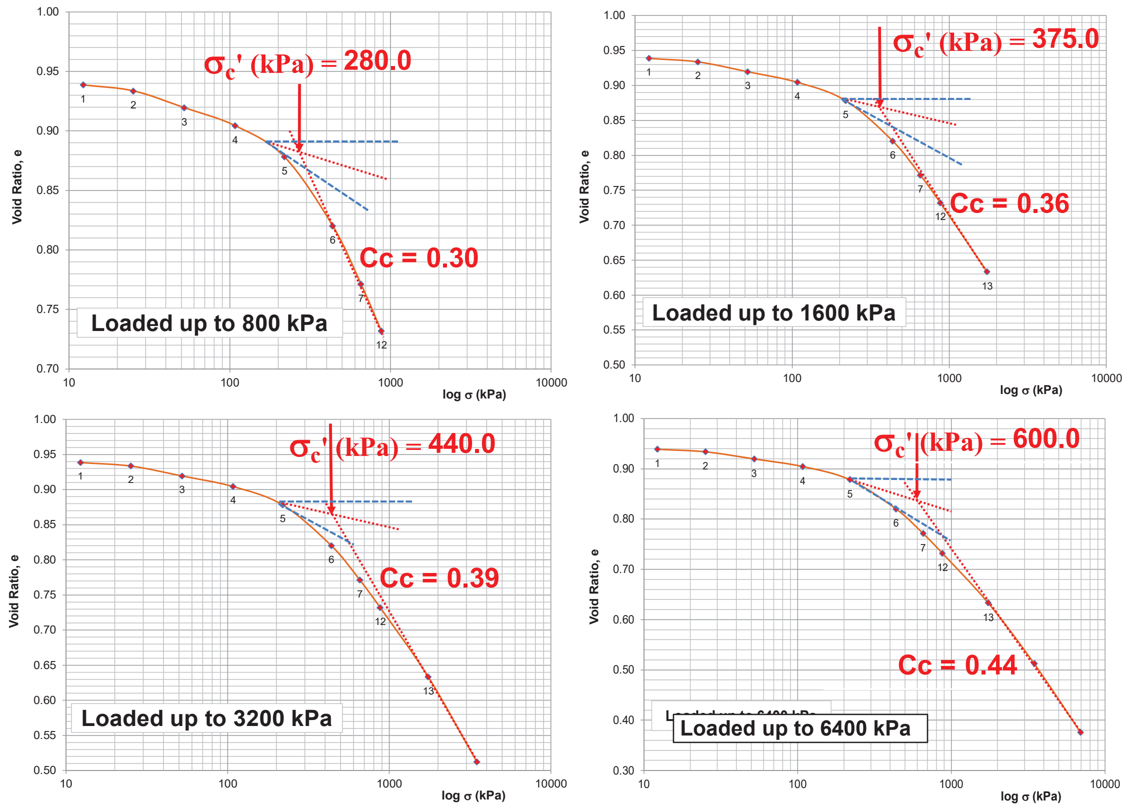

Figure 7. Variation of σ_c' and cc due to the maximum load applied.

From the above data, Casagrande's graphical method, apart from interpreter-dependent, also relies on the scale of the plotted graphs and the maximum load applied. This method greatly depends on the user experiences and judgment. To minimize such user dependency, Gouw (2017b) has proposed a so called parallel rebound method and as suggested by Schmertmann (1953), and the consolidation test be carried out up to 0.42 times the initial void ratio (0.42 e_o) of the soil sample.

The parallel rebound method is based on the fact that, when a reconstituted saturated clay layer undergoes consolidation, the virgin compression line plotted on an e – log σ′ curve forms a straight line, and when unloaded, the unloading curve will also form a straight line. When unloaded at different pre-consolidation pressures, the unloading or rebound curves are parallel to each other, and the point of unloading is the pre-consolidation pressure, σ_c', that the soil sample has been subjected to (see Figure 8a). When we retrieved a soil sample from a borehole, the sample will be unloaded from its own overburden pressure, therefore, the above principle of parallelism can be applied. The procedure to carry out the proposed parallel rebound method is (Figure 8b):

a) First, draw a linear line through the unloading or rebound part of e-log σ′ plot, named as rebound line (dark blue line).
b) Second, draw a parallel line (light blue) tangent through initial part of e-log σ′ curve, named as initial tangent line.
c) Third, draw the virgin compression line (red line). The intersection of the virgin compression line with the initial tangent line is the pre-consolidation pressure, σ_c'.

The proposed method can be easily mathematically formulated in excel spreadsheet, therefore, eliminating the graphical error and will result in unique values for σ_c', c_c and c_r.

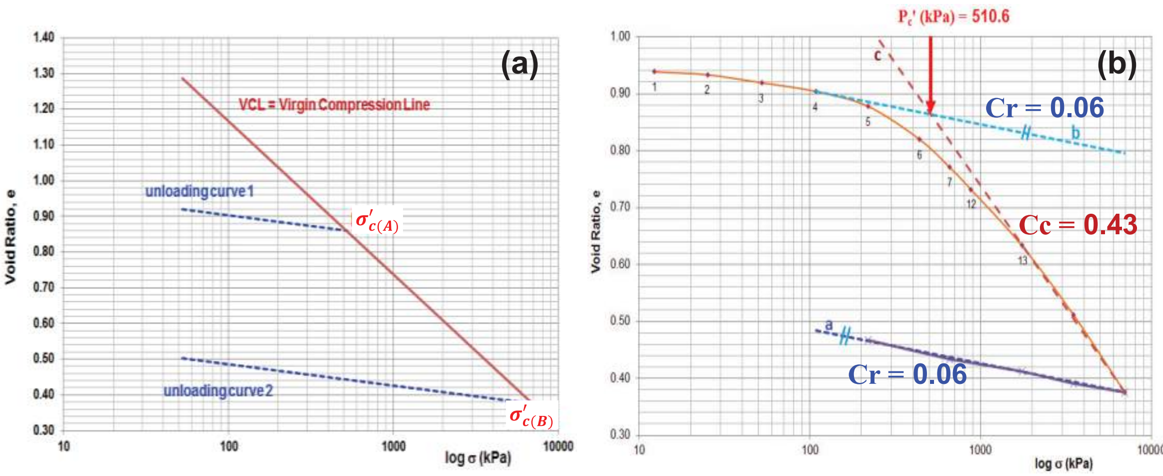

Figure 8. (a) Reconstituted clay consolidation curve, (b) parallel rebound method to determine σ_c', c_c and c_r (Gouw, 2017b).

3.5 Coefficient of Consolidation, c_v

The value of consolidation is another contributing factor in the error of settlement calculation. This c_v value is used to estimate the time of consolidation. If the consolidation time is wrongly calculated, the settlement magnitude at a certain time will also be wrong. As in the derivation of pre-consolidation pressure, the coefficient of consolidation is also derived by a graphical method using the deformation vs square roots of time plot as proposed by Taylor (1942; 1948). Figure 9 shows the procedure and the variability of the results due to its graphical nature.

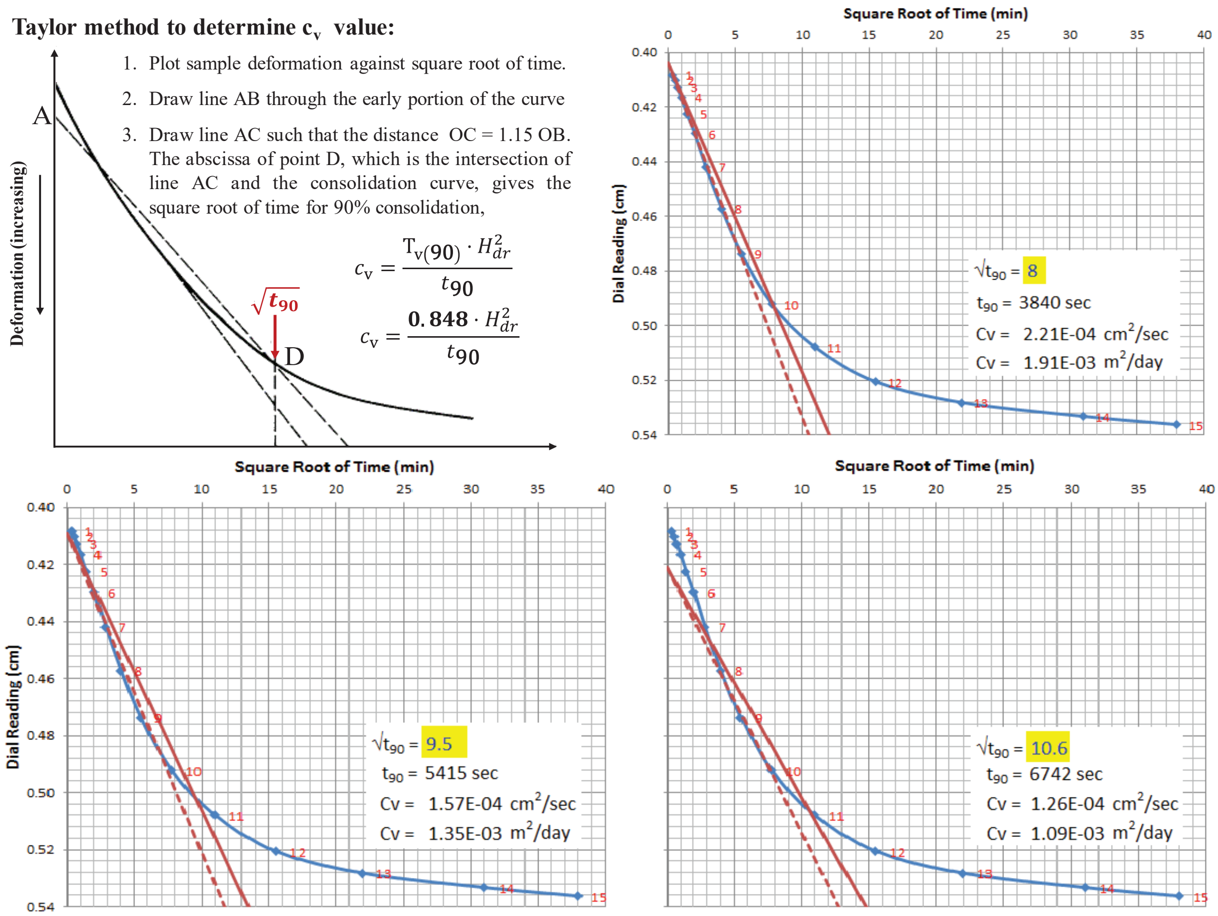

Figure 9. Taylor's graphical method to derive c_v and the variation of results.

As a result, it is often that the plot of c_v vs log σ' show wavy and irregular curve, while in fact the c_v values should decrease with increasing effective stress (Figure 10). Theoretically, as the soil compresses, the void ratio reduces, and so does the permeability, hence, a higher consolidation pressure should correspond to a lower coefficient of consolidation.

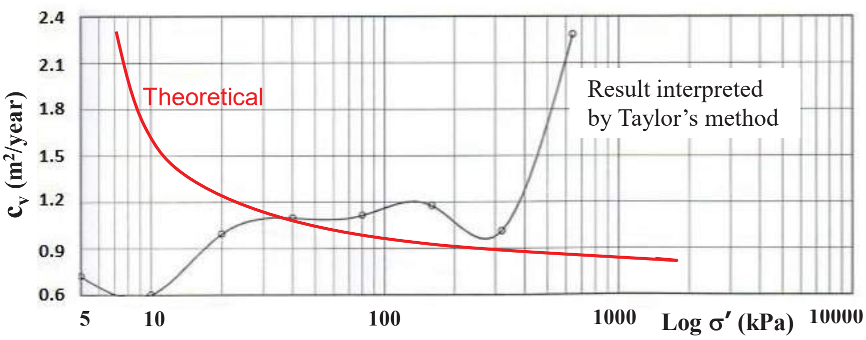

Figure 10. Theoretical vs irregular c_v values derived by Taylor's method.

To eliminate the error of Taylor's graphical procedure, Asaoka's method (Asaoka, 1978) can be mathematically applied in determining the end of primary consolidation of a consolidation test and hence deriving the corresponding c_v value. To apply this method, the settlement reading interval of the oedometer test during a loading stage must be read at a regular interval, say at Δt = 60 minutes time intervals. Then the settlement at time t_i, $\Delta H(t_i)$, is plotted in x axis, against settlement at time $t_i+\Delta t$, $\Delta H(t_i+\Delta t)$ in y axis. Final consolidation is achieved when $\Delta H(t_i) = \Delta H(t_i+\Delta t)$. By knowing the final consolidation (settlement), the 90% degree of consolidation time can be determined, and the c_v value can be calculated. Figure 11a shows the settlement curve of a consolidation test from a certain load stage, figure 11b shows the Asaoka's graph and the resulting c_v value. By employing this Asaoka's method, it was found that the coefficient of consolidation can be derived in a more consistent way, in accordance with the theory, i.e., the c_v values reduce with increasing pressures (Figure 11c).

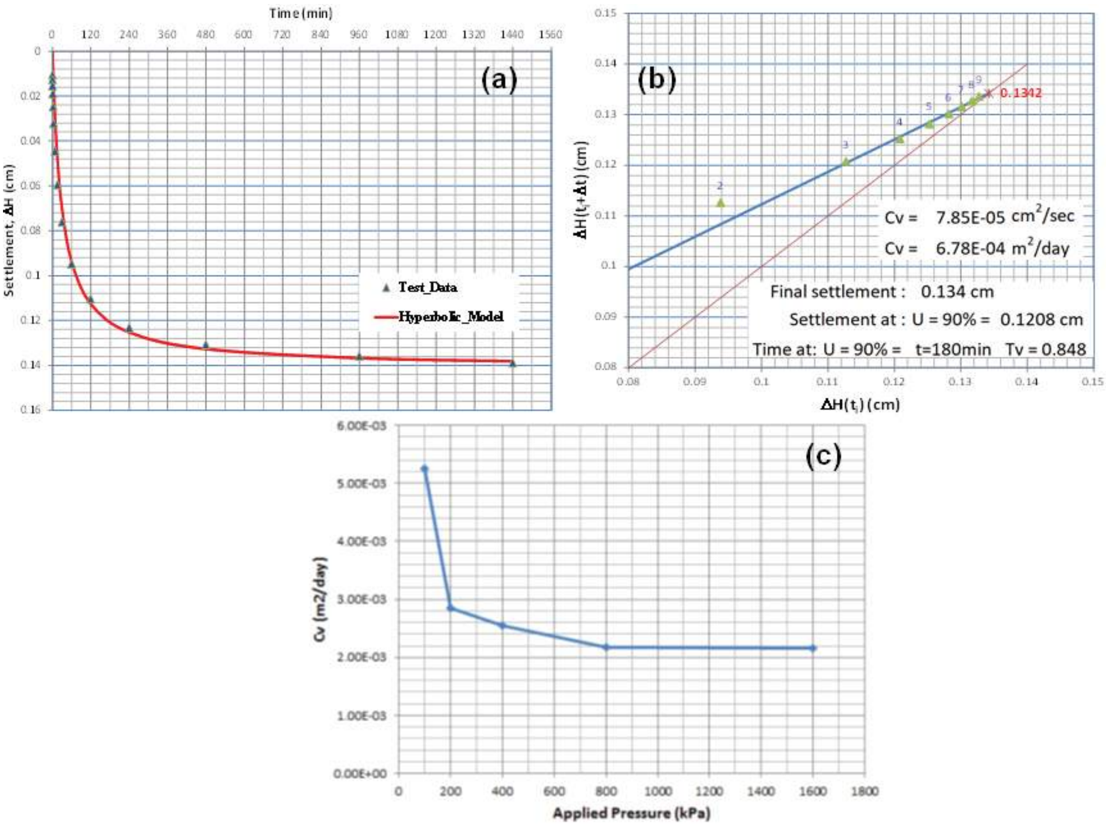

Figure 11. Use of Asaoka's method to derive c_v values.

It has come to the author's attention that some commercial laboratories did not comply with the requirement of consolidation tests, that is, to maintain each loading stage for 24 hours. Instead, they often immediately increase to the next loading stage, once the test data curve gives 90% degree of consolidation time, t_{90}, as derived by Taylor's method. This practice is not acceptable as the residual pore pressure at the previous stage will accumulate with the excess pore pressure induced by the subsequent loading stage. The error will get larger if at every loading stage the same "short cut" procedure is applied. The e – log σ′ curve will not be representative. Hence, all the consolidation parameters will also be erroneous.

3.6 Three-Dimensional Effect

The above consolidation settlement formula is derived based on one-dimensional dissipation of pore pressure, i.e., the excess pore pressure dissipated in a vertical way upward or/and downward, and the settlement occurs only in vertical direction. However, in reality, pore pressure can dissipate in every direction within the soil body, and soil underneath a foundation also undergoes lateral deformation as a result of applied loading, meaning that the settlement is not a mere 1D consolidation. Due to this, in general the induced pore water pressure is less than the increment of the vertical stress on the element. Therefore, to consider this 3D effect, the calculated consolidation settlement, S_c, should be corrected with Skempton & Bjerrum (1957) correction factor, m, as follows:

$$S_{c3D} = m \cdot S_c \tag{10}$$

where S_{c3D} is the corrected consolidation settlement in consideration to 3D effect, m is the settlement coefficient (Skempton-Bjerrum correction factor), S_c is the consolidation settlement as calculated by Terzaghi 1D consolidation formula.

Figure 12 shows the Skempton-Bjerrum correction factor, and Table 2 can be used to estimate the pore pressure coefficient. Unless it can be ensured that only 1D consolidation shall take place, this correction factor should be applied

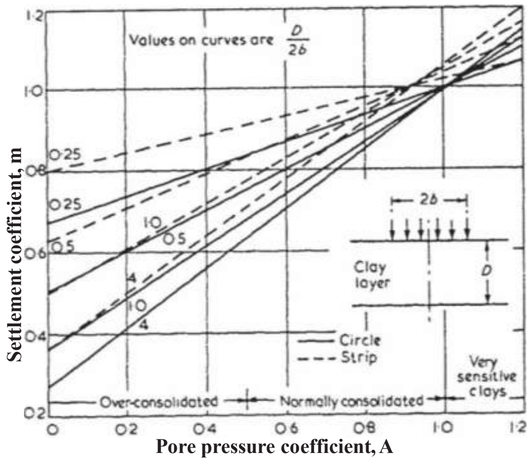

Figure 12. Skempton-Bjerrum's correction factor.

Table 2. Pore pressure coefficient A

Soil Type	Volume changes due to shear	A_f *(Head & Epps, 2014)*	A_f *(Skempton, 1954)*
Sensitive clay	Large contraction		1.50 ~ 2.50
Highly sensitive clay	Large contraction	0.75 ~ 1.50	
Lightly overconsolidated clay	None	0.00 ~ 0.50	0.30 ~ 0.70
Normally consolidated clay	Contraction	0.50 ~ 1.00	0.70 ~ 1.30
Heavily overconsolidated clay	Dilate	-0.50 ~ 0.00	-0.50 ~ 0.00
Loose, fine sand	Contraction		2.00 ~ 3.00
Compacted sand clay	Slight contraction	0.25 ~ 0.75	0.25 ~ 0.75
Compacted clay gravel	Dilate	-0.25 ~ 0.25	

4 SECONDARY SETTLEMENT

The secondary settlement formula is:

$$S_s = \frac{c_\alpha}{1+e_p} H \log \frac{t_s}{t_p} \tag{11}$$

$$c_\alpha = \frac{\Delta e}{\log t_s - \log t_p} = \frac{\Delta e}{\log \frac{t_s}{t_p}} \tag{12}$$

where c_α is the secondary compression index, Δe is the change in void ratio from time t_p to t_s, t_p is the time at end of primary consolidation, t_s is the time of secondary settlement, S_s is the secondary settlement, H = thickness of the soil layer. Figure 13 shows the necessary notation for the above formula.

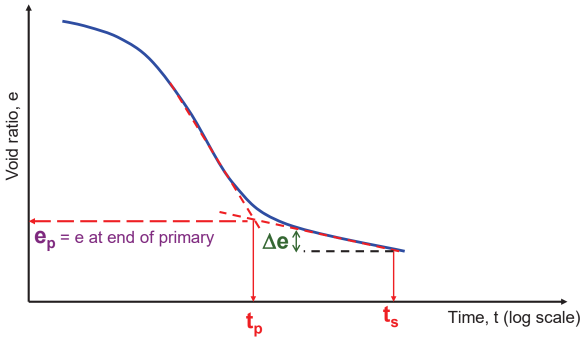

Figure 13. Secondary settlement.

In certain soil, such as: organic clays and peat, secondary settlement is one of the important deformation criteria. However, the accuracy of the parameter c_α becomes a problem, as it is impractical to conduct such test due to the very long testing duration. Therefore, most of the time, one can only use a chart (as shown in Figure 14) to estimate the secondary compression index. The range of the secondary compression index provided in the chart is very wide, which means the estimation of the secondary settlement can be far off from the reality. In this case, one can only rely on the local experiences and his/her best engineering judgment.

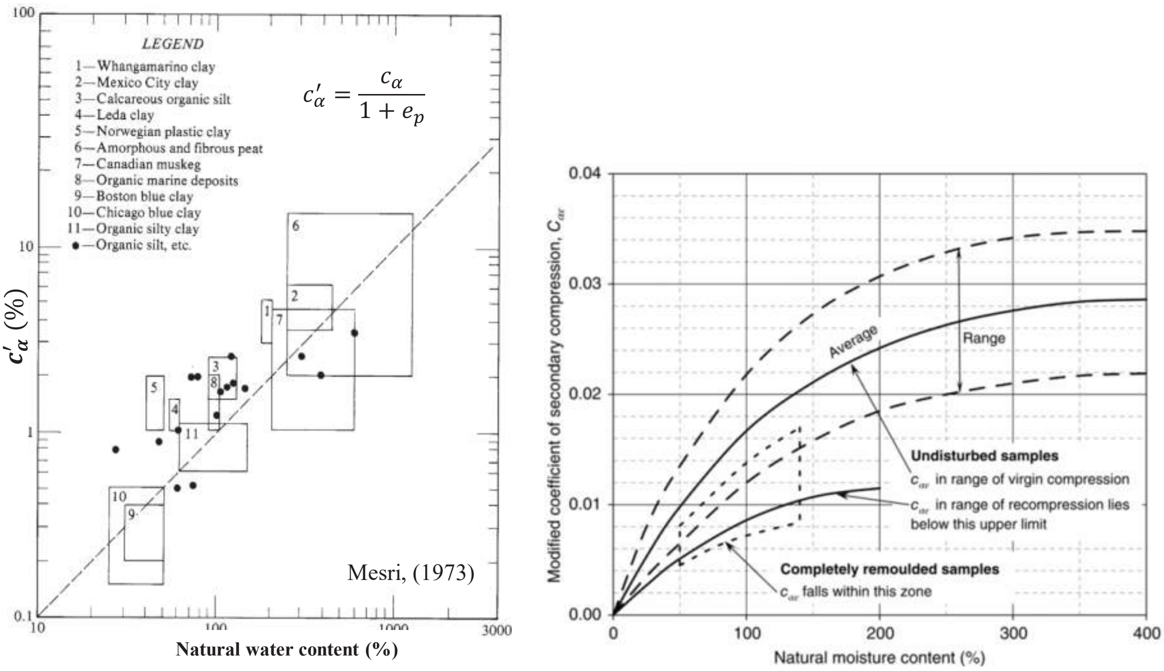

Figure 14. Charts to estimate secondary compression index (Mesri, 1973; US Navy 1982).

5 CONSTITUTIVE MODELS

Nowadays, with the advancement of computer technology, 3D geotechnical finite element (FEM) software, such as Plaxis 3D, RS3, Midas GTS NX, etc. are readily available for engineers to carry out 3D analysis for estimating soil deformation in a more thorough manner. To use this FEM software, one must have a good understanding of soil mechanics theory, and the limitation of the constitutive model adopted. Many engineers adopted Mohr-Coulomb model due to its simplicity without knowing the limitations of this model.

Mohr-Coulomb model is a bilinear approach, while in reality, soil behavior is non-linear. It only adopts a single value of soil stiffness, E. Assuming all the parameters adopted are representative, for a factor of safety larger than 2, the deformation prediction is larger than reality. However, for a factor of safety less than 2, the deformation prediction will be smaller than reality. Deformation calculated for unloading condition is theoretically not accurate, as no unloading E value is considered.

To achieve reliable results, more advanced constitutive models must be adopted, e.g., soft soil model, soft soil creep model, hardening soil model, hardening soil with small strain, modified cam-clay model, etc. Each of these constitutive models has its own limitations and may not be suitable to be adopted with the problem at hand. This issue will not be elaborated further as this is out of the scope of this paper.

6 OTHER FACTORS

Other factors that affect the result of the settlement prediction are:

a) Percentage wise, the volume of the soil tested in the field and in the laboratory are very small compared to the loaded volume of the soil. For example: A building footprint of 40 m x 40 m and with an assumed depth of compressed soil reaching 80 m, the total volume of the soil involved is 128,000 m^3. If 5 boreholes of 10 cm diameter were executed to 80 m depth, the volume of the soil tested is only a mere 3.14 m^3, which is only 0.0025% of the soil volume affected underneath the building, and that is assuming all soil in the boreholes are fully tested, which in reality, they are not. Of course, budget wise, we cannot do the soil test let's say at 10 m grid intervals, Therefore, it is of utmost important to carry out the soil investigation in a systematic manner so that a representative stratigraphy of the soil underneath the building can be drawn and the relevant soil parameters can be obtained.
b) Careful handling of the soil samples, from the field to the laboratory and during the preparation of the relevant testing, is very important. Sample disturbances as much as possible must be minimized, although some kinds of disturbances cannot be avoided. Rough handling and pressure in the preparation of consolidation test will adversely affect the parameters obtained.
c) Structural rigidity of a building is also one the important factors that affect the settlement prediction. A simple approach that is often adopted for building with high rigidity compared to the soil is to multiply the calculated values through the above formulas by 0.7 to 0.8, i.e., reduces the settlement magnitude by 20 to 30%. However, since settlement will affect the building performance, it is suggested to carry out soil structure interaction analysis. For this, a cooperation of geotechnical and structural engineers is of utmost importance. Structural and geotechnical finite element software is certainly required to carry out iterative analysis until the best output is established. Nowadays, interactive analysis between structural and geotechnical software is available, e.g., StaadPro with Plaxis, Midas Structure and Midas GTX NX, etc.

7 CONCLUSION

The calculation of settlement is not an easy task as many factors can affect the calculated output, hence, it will never be accurate. If the calculated value falls within, say ± 30% compared to the reality, it can be considered as a very good prediction. To get a best estimate of the settlement prediction, all the parameters must be as representative as possible. Upper bound and lower bound analysis is a good practice to perform. A good geotechnical knowledge and familiarity of the local soil characteristics is also required.

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