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Estimating In Situ Maximum Past (Preconsolidation) Pressure of Saturated Clays From Results of Laboratory Consolidometer Tests

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A proper evaluation of the in situ stress history of a clay deposit is a necessary prerequisite to estimating compressibility and strength characteristics of the clay. The term stress history refers to the existing in situ vertical effective stress p'_o in relation to the maximum past pressure P_c of the clay deposit. (Other names that have been used in addition to maximum past pressure include preconsolidation pressure, amount of precompression, critical pressure, and maximum past vertical effective stress.) When the maximum past pressure equals the net overburden stress, i.e., $P_c = p'_o$, the clay is called normally consolidated, and volume change caused by loading of a normally consolidated clay deposit is referred to as virgin compression. If the existing in situ stress is less than the maximum past pressure, i.e., $p'_o < P_c$, the clay is called overconsolidated, preconsolidated, or precompressed. The degree of overconsolidation is expressed by the overconsolidation ratio OCR.

$$\mathrm{OCR} = P_c/p'_o \qquad (1)$$

Various mechanisms can cause a clay deposit to be overconsolidated. In particular, removal of overburden stress during the geological history of the clay deposit is only one of several possible causes of P_c. Consequently, the term P_c denotes the stress at which a characteristic change in compressibility occurs rather than a certain value of maximum past vertical effective stress per se.

The concept of a maximum past pressure P_c based on changes in the compressibility of the clay is shown in Figure 1. Vertical strain ϵ_v is plotted versus consolidation stress σ' for 1-dimensional compression of a hypothetical clay with an in situ stress p'_o equal to 500 lb/ft^2 (23.9 kPa). On the natural scale, when σ' approaches a value of 2,000 lb/ft^2 (95.8 kPa), the compressibility increases and then decreases at larger stresses. On the log scale, the curvature of the compression curve in the vicinity of P_c changes markedly and, at stresses greater than P_c, ϵ_v versus σ' is often closely approximated by a straight line with many natural clays.

In this chapter, the Casagrande method of estimating the in situ P_c from the results of laboratory consolidometer tests is presented first and then the various mechanisms that can cause a P_c are discussed. The principal factors influencing laboratory compression curves, recommended consolidometer test procedures, and specific recommendations for determining P_c are then presented. The discussion is restricted to the estimation of P_c from incremental consolidometer tests on undisturbed samples of medium to soft saturated clays obtained from below the water table.

CASAGRANDE METHOD FOR DETERMINATION OF P_c

Casagrande (5) presented the empirical method shown in Figure 2 for estimation of the value of P_c from the results of laboratory consolidometer tests on undisturbed samples of saturated clay. The estimated value of P_c is obtained as follows:

1. Determine the point T of minimum radius on the laboratory compression curve (ϵ_v or e versus log σ').
2. Draw 2 lines from point T, one horizontal and one tangent to the compression curve. The angle θ between the 2 lines is then bisected.
3. Extend the straight-line portion of the compression curve. The point of intersection C of the extension with the bisector line is the estimated value of P_c.
4. Round off the estimated value of P_c to 2 significant figures and report together with possible range of P_c. The lower limit of P_c is usually assumed to be the point of intersection of a horizontal line through point E and the extension of the straight-line portion of the compression curve. The maximum value is assumed to be the point (M in Figure 2) beyond which the compression curve can be approximated by a straight line.

The Casagrande method is widely used in practice and is generally considered to yield satisfactory results if samples are of high quality and consolidometer tests are properly conducted. Other methods for estimating P_c are given by Burmister (4), Janbu (7), and Schmertmann (24), who also presents a procedure for correcting the laboratory compression curve for the effects of sample disturbance.

POSSIBLE MECHANISMS CAUSING P_c

The concept of maximum past pressure is most easily visualized in terms of the maximum stress that has acted on the clay. At stresses less than this maximum value, the compressibility is low compared to the compressibility that is exhibited by the sharp change in slope of the compression curve as the stresses approach and then exceed P_c. This same type of behavior (increased compressibility and characteristic shape) can result from mechanisms other than a prior stress increase.

Table 1 (10) gives mechanisms by which a clay might acquire a P_c greater than the existing vertical effective stress. Change in total stress is the most obvious cause, and change in the pore pressure condition is the next most obvious. Desiccation is commonly recognized if associated with a clay crust. However, desiccation may also have been an important factor during deposition of many alluvial, backswamp, and tidal mud-flat deposits.

Clays may also exhibit an increased P_c without having had an increased effective stress. In Table 1, these mechanisms are referred to as change in soil structure (12), which denotes soil fabric (orientation and distribution of particles) and interparticle forces (types and relative magnitude). The relation between secondary compression and P_c is now well documented (1, 2, 15, 18), but the mechanism postulated by the authors for explaining the relation is different. Environmental changes and chemical alterations can also influence the in situ value of P_c.

When interpreting the results of consolidometer tests and when selecting values of P_c

Figure 1. Definition of P_c in relation to in situ compressibility.

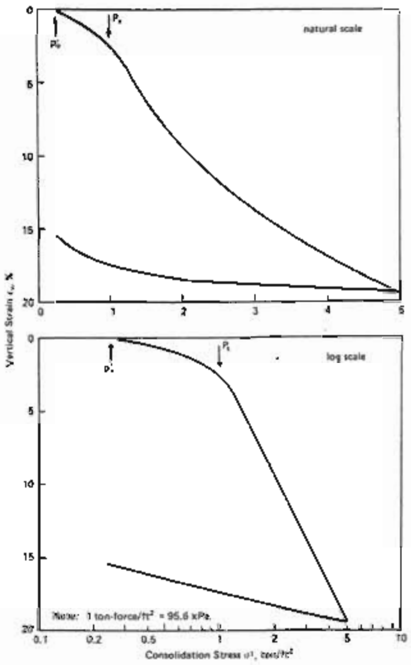


Figure 2. Casagrande method for estimating P_c.

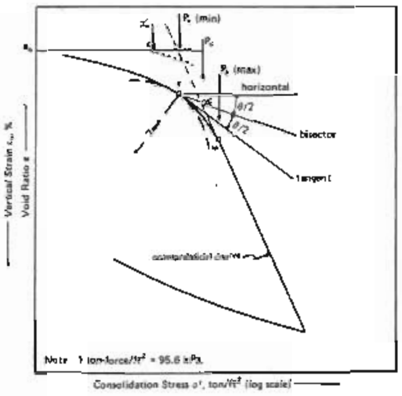


Table 1. Mechanisms causing P_c.

Mechanism	Remarks and References
Change in total stress due to	
Removal of overburden	
Past structures	
Glaciation	
Change in pore water pressure due to	
Change in water table elevation	Kenney (9) gives sea level changes
Artesian pressures	Common in glaciated areas
Deep pumping	Common in many cities
Desiccation due to drying	May have occurred during deposition
Desiccation due to plant life	May have occurred during deposition
Change in soil structure due to	
Secondary compression (aging)[a]	Raju (20) Leonards and Ramiah (13) Leonards and Altschaeffl (15) Bjerrum (1, 3)
Environmental changes such as pH, temperature, and salt concentration	Lambe (12)
Chemical alterations due to "weathering," precipitation cementing agents, ion exchange	Bjerrum (1)
Change of strain rate on loading	Lowe (18)[b]

[a] The magnitude of P_c/p_o related to secondary compression for mature natural deposits of highly plastic clays may reach values as high as 1.9 or higher (16, 1, 2, 22).

[b] Further research is needed to determine whether this mechanism should take the place of secondary compression.

to use in design, one should always try to determine the mechanisms involved. This will help in deciding how the maximum past pressure should vary with depth and whether erratic values should be ascribed to sample disturbance, testing procedures, or natural occurrences.

PRINCIPAL FACTORS INFLUENCING THE DETERMINATION OF P_c FROM LABORATORY CONSOLIDOMETER TESTS

The 3 most important factors influencing the determination of P_c from laboratory consolidometer tests are sample disturbance, load increment ratio, and load increment duration. (A change in the electrolyte of the pore water during laboratory testing would also affect the compressibility of certain type clays, but its influence is not discussed here.)

Sample Disturbance

Figure 3 shows compression curves from consolidometer tests on high-quality and poor-quality samples of an overconsolidated clay in relation to the in situ compression curve. Increasing sample disturbance

1. Decreases the void ratio (or increases the strain) at any given value of consolidation stress;
2. Obscures and/or lowers the estimated value of P_c from the Casagrande construction if the disturbance is excessive, especially with sensitive clays and clays that have developed a P_c as a result of secondary compression;
3. Increases the compressibility at stresses less than P_c; and
4. Decreases the compressibility at stresses greater than P_c.

Load Increment Ratio

The load increment ratio LIR denotes the change in consolidation stress divided by the initial consolidation stress.

$$\text{LIR} = \frac{\sigma'\,\text{final} - \sigma'\,\text{initial}}{\sigma'\,\text{initial}} = \frac{\Delta\sigma}{\sigma'\,\text{initial}} \qquad (2)$$

Conventional consolidometer tests usually employ an LIR of unity; that is, the load (consolidation stress) is doubled for each successive increment. The influence of varying the LIR for consolidometer tests on a sensitive clay is shown in Figure 4. The use of the conventional procedure with LIR = 1 does not properly define the in situ compressibility and hence the P_c of clays, especially if the clay is strain sensitive.

Experience with sensitive soft clays that exhibit in situ compression characteristics typical of those shown in Figure 4 also shows that the clay is likely to undergo sudden collapse and may squeeze out of the consolidometer (between the ring and the porous stones) if the load is doubled in the vicinity of P_c. Consequently, the LIR must be reduced in the vicinity of P_c in order to measure, even approximately, the in situ compressibility and P_c.

Load Increment Duration

The load increment duration denotes the total time t_f allowed for consolidation prior to application of the next load increment. Standard consolidometer test procedures often use a duration of 1 day for each increment. Since the time for primary consolidation t_p for typical sample heights and values of the coefficient of consolidation for saturated medium to soft clays is only 5 to 100 min, appreciable secondary compression occurs during a 1-day duration. Consequently, the 1-day compression curve will fall below the compression curve corresponding to the end of primary consolidation. This, in turn, influences the value of P_c determined from the test.

The effect of load increment duration on the estimated value of P_c is shown in Figure 5. The dashed curve corresponds to the strain or void ratio measured at the end of each 1-day increment. The solid curve corresponds to the strain or void ratio measured (the curve is plotted on the assumption that 100 percent of the initial excess pore pressure is dissipated at $t_f = t_p$)

1. From a test in which the next load is applied as soon as primary consolidation is completed; or
2. From the 1-day test, but where the e or ϵ_v at the end of primary consolidation is plotted rather than the value at the end of the increment. [It has been assumed in the second procedure that the location of the compression curve with $t_f = t_p$ is unaffected by the secondary compression that occurred during the previous increment. This is a reasonable assumption if the amount of primary consolidation is a significant fraction (65 to 75 percent) of the total compression for the increment.]

The amount of the reduction in the estimated value of P_c from 1-day tests as opposed to that determined at the end of primary consolidation will obviously be most significant in clays that exhibit a high rate of secondary compression in the vicinity of P_c. For typical soft CL and CH clays, the amount of the reduction in P_c is likely to be on the order of 10 to 20 percent.

RECOMMENDED PROCEDURES FOR DETERMINATION OF P_c

Test Equipment

The equipment employed for incremental consolidometer tests should generally conform to the recommendations of ASTM D2435-70 and Lambe (11). The following are particularly important.

1. A diameter-to-height ratio of at least 2.5 but no more than 6 should be used; a ratio of 3 to 4 is preferable. Side friction is likely to affect the results if a ratio smaller than 2.5 is used, and bending stresses during handling are likely to disturb the soil structure when larger ratios are used.
2. The ring must be noncorrosive, be smooth to the touch, and have sufficient wall thickness to prevent ring distortion. In general, the harder the ring material is, the less the wall friction will be. A coating of silicone grease or molybdenum disulfide lubricant on the ring wall is recommended. Very thin teflon linings may be used, but they are subject to abrasion and increased wall friction with continued use.
3. Minimum specimen dimensions should generally be 2 in. (5 cm) in diameter and 0.75 in. (1.9 cm) in height. The larger the specimen size is, the better.
4. Porous stones, such as Norton P2120 mixture, should generally have a clearance of about 0.01 in. (0.025 cm), that is, have a diameter 0.02 in. (0.05 cm) smaller than that of the ring, and be replaced or cleaned periodically. Truncated, cone-shaped porous stones, if used, reduce the possibility of drag resulting from uneven compression.
5. The applied loads must be constant and accurately known, and vertical deformations must be measured with a sensitivity of 0.0001 in. (0.00025 cm).

Figure 3. Effect of sample disturbance on laboratory compression curves and estimated value of P_c.

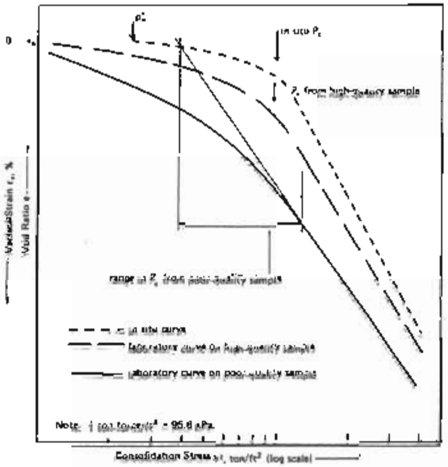


Figure 4. Influence of load increment ratio on laboratory compression curve for sensitive clay.

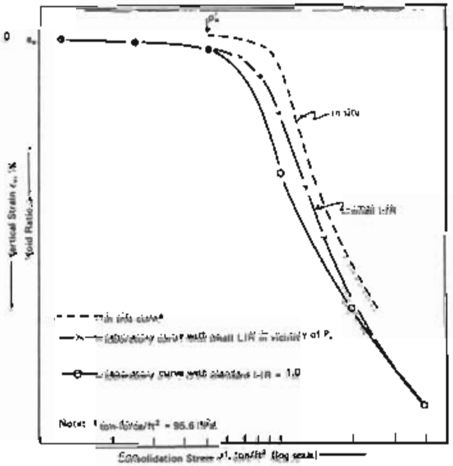
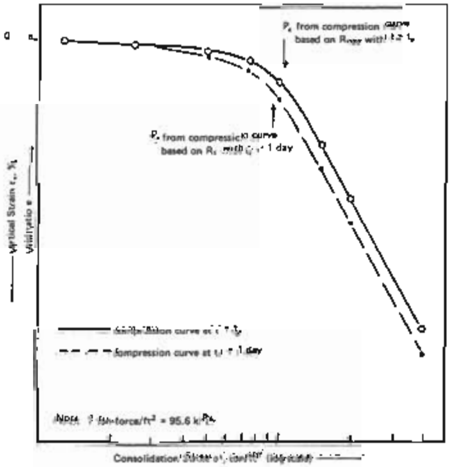


Figure 5. Effect of load increment duration on laboratory compression curve and estimated value of P_c.



Test Procedures

The test procedures employed for incremental consolidometer tests should generally conform to the recommendations of ASTM D 2435-70 and Lambe (11). Modifications and items of importance are noted below.

1. The initial total, the final total, and final dry weights of the entire specimen should be obtained. As an independent measurement of the sample thickness, the dial gauge should be set to 0 by the use of a metal spacer of known thickness and the porous stones to be used in the test.
2. Before water is added, an initial "seating" load of 0.3 to 0.7 lb/in.^{2} (2 to 5 kPa) should generally be applied to the sample while an initial dial reading corresponding to the initial height of the sample is quickly obtained. Water should then be added while the dial reading is watched. If the sample wants to swell, the load should be increased to prevent swelling and to obtain an initial equilibrium consolidation stress.
3. A load increment ratio equal to unity, i.e., doubling of the load, may satisfactorily define the compression curve for many medium to soft saturated clays. However, if the clay has compression characteristics that vary with LIR and the amount of secondary compression under the previous increment (which is typical of very sensitive clays and of soft plastic clays with a high liquidity index), the load increment ratio should be reduced in the vicinity of the estimated P_c. (σ' values of 0.5 P_c, 0.7 P_c, 0.85 P_c, 1.0 P_c, 1.3 P_c, and 1.5 P_c will often be adequate to define P_c and/or to prevent squeezing of clay from the consolidometer ring.)
4. The compression curve in the recompression region is defined by unloading the sample, after it is initially loaded to a stress close to P_c, to the effective overburden pressure and then reloading it. For best definition of the recompression index the specimen should be unloaded when $0.5\ P_c \le \sigma' < 1.0\ P_c$. To define the virgin compression slope requires that loading of the test specimen be continued to a sufficiently high level to establish the straight-line portion of the compression curve beyond P_c. Normally, loading must be continued to a stress level equal to 8 times P_c. For excellent quality samples of soft clays, the maximum applied stress may be less than 8 times P_c; for moderately to highly overconsolidated clays the maximum stress may have to be more than 8 times P_c.
5. The load increment duration t_f should be maintained approximately constant for each increment. However, if the total test time is critical, shorter increment duration times may be used in the overconsolidated region without sacrificing accuracy. It is important that sufficient dial gauge readings be obtained to define both the end of primary consolidation and the slope of the approximately linear secondary compression curve on the plot of dial reading versus log time.
6. Temperature fluctuations of more than a few degrees, approximately ±7 F (±4 C), should be prevented. Since temperature fluctuations significantly influence the rate of secondary compression, accurate estimates of secondary compression necessitate closer temperature control, approximately ±1.8 F (±1 C).

Presentation and Interpretation of Test Data

The objective is to obtain the P_c of a compression curve that corresponds, at least approximately, to the end of primary consolidation (the solid curve t_p in Figure 5). Since it is generally impractical to apply a new load each time the sample reaches the end of primary consolidation, the procedure shown in Figure 6 is recommended. Figure 6 shows a hypothetical set of dial readings versus time from a consolidometer test with a variable LIR and 1-day increments on a sample of clay having a value of P_c between 14.2 and 21.3 lb/in.^{2} (1.0 to 1.5 kg/cm^2). The dial reading R_{100} at the end of primary consolidation ($t = t_p$), not the final dial reading R_f, is used to compute the compression curve. When the LIR is unity, a typical Terzaghi type of consolidation curve is generally obtained, and R_{100} is easily evaluated by the intersection of the straight lines as shown for the top and bottom curves in Figure 6. When the LIR is sufficiently small, such as shown for the middle curve in Figure 6, the clay generally does not follow the Terzaghi consolidation theory, and t_p must be established. [Leonards and Altschaeffl (15) and Lowe (18) give a more complete discussion of this phenomenon.] In this case t_p depends on LIR and on the amount of secondary compression under the previous increment. One can reasonably assume, however, a value of t_p that falls between the values measured in the previous and subsequent increments.

If a Terzaghi consolidation curve is not obtained for the increments within the recompression region, one can determine an average value of t_p obtained from increments in the normally consolidated range and use this approximate value of time to select the appropriate dial readings for all of the increments.

The next step is plotting the compression curve based on R_{100} dial readings to a suitable scale in order to use the Casagrande method of construction (or perhaps one of the other methods referenced here) to estimate P_c. [The next chapter states that the summation of (R_{100} - R_o) values should be used in the plotting of the compression curve. For saturated clay and for $t_f \approx t_p$, the methods proposed in chapter 2 and in this chapter will result in essentially identical curves. If $t_f > tp$, the curve should be plotted on the basis of R_{100} dial readings.] Figure 7 shows 2 suggested scales of vertical strain versus consolidation stress for use with soft clays of medium to high compressibility [vertical strain = $\Delta h/h_o = \Delta e/(1 + e_o)$]. The use of average strain rather than void ratio is recommended because

1. Strains are easier to compute than void ratios;
2. Differences in initial void ratio may cause samples to exhibit quite different plots of void ratio versus stress but almost identical plots of strain versus stress;
3. Settlements are directly proportional to strain, but use of Δ e data also requires a knowledge of $(1 + e_o)$, which introduces 2 variables, Δ e and $(1 + e_o)$; and
4. Strain plots are easier to standardize than void ratio plots.

CONCLUDING REMARKS

Although this chapter has been restricted to use of incremental consolidometer tests for estimation of P_c, readers should not infer that incremental tests are superior to tests in which the sample is continuously loaded, such as in the controlled gradient test (17) and in the constant rate of strain tests (25, 30). In fact, such tests have several important advantages:

1. A continuous compression curve is obtained;
2. The measured compression curves generally correspond to the end of primary compression, and thus the need to make plots of dial reading versus time in order to obtain R_{100} is eliminated;
3. The problem of loss of soil due to squeezing is minimized or eliminated; and
4. The total time required to perform the primary compression portion of the test is greatly reduced.

However, continuously loaded consolidation tests generally require more sophisticated equipment, including a data acquisition system. Information on secondary compression can be obtained with the equipment used for controlled gradient test, but not with all types of equipment used for constant rate of strain tests. Finally, it should be emphasized that

1. Proper evaluation of the in situ P_c is an art that requires considerable judgment and, above all, an appreciation of the various geological and testing factors that control and influence values of P_c;
2. Evaluation of the in situ P_c from laboratory tests requires that the profile of test values be superimposed on the profile of the effective overburden stress; and
3. It is the exception rather than the rule to encounter normally consolidated cohesive deposits, i.e., with $P_c = p_o'$, except for the case of underwater sediments now being formed or soil deposits that have recently been subject to load.

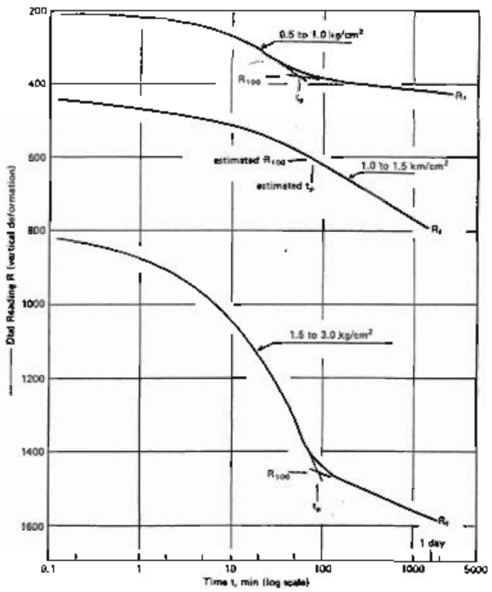


Figure 6. Determination of compression curve corresponding to end of primary consolidation.

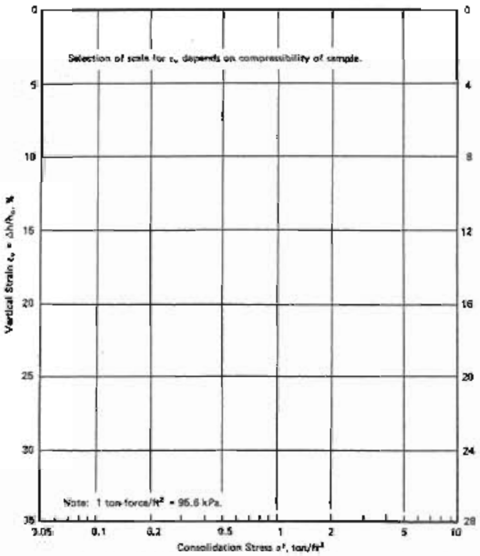


Figure 7. Suggested scales for presentation of compression curves.

2 Estimating Consolidation Settlements of Shallow Foundations on Overconsolidated Clay

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STATEMENT OF PROBLEM

A shallow foundation is designed at a site underlain by a layer of saturated, overconsolidated clay. The maximum stress in the clay stratum due to overburden plus net increase in pressure due to the foundation is everywhere less than the maximum past (preconsolidation) pressure.

The clay layer is of sufficient thickness that the increase in stress due to loading may be attenuated with depth. The magnitudes of these stress increases are calculated from elasticity theory; if necessary (modular ratio > 5), a layered system is assumed.

The factor of safety of the foundation with respect to bearing capacity is 3 or more. Volume changes due to consolidation of the clay layer are assumed to be 1-dimensional. What procedures should be used to estimate the consolidation settlements assuming that "undisturbed" samples can be obtained either from hand-cut blocks or from thin-walled, fixed-piston samplers? If the clay neither swells an extraordinary amount when removed from the sampling tubes nor is especially sensitive to remolding, what magnitude of error in this estimate should be expected?

GENERAL CONSIDERATIONS

Inasmuch as volume changes are considered to be 1-dimensional, the apparent consolidation settlement S_i of each segment of the clay stratum, within which the initial void ratio e_o, the recompression index C_r, the initial effective vertical overburden pressure p_o', and the net change in vertical stress due to loading Δp do not vary greatly, may be computed from

$$S_i = \frac{C_r}{1 + e_o} H_i \log \frac{p_o' + \Delta p_i}{p_o'} \qquad (3)$$

The apparent total consolidation settlement S is

$$S = \sum_{i=1}^{i=n} S_i \tag{4}$$

The first task is to identify the number and thickness of layers to be considered. This can be done with the help of a plot such as that shown in Figure 8. At least 2 consolidation tests should be performed on samples from each layer; if the layer is thicker than 20 ft (6 m), 1 test for each 10 ft (3 m) of thickness should be conducted. Data on water content and Atterberg limits should be available for every 5 ft (1.5 m) of depth.

In situ values of C_r are generally smaller than those measured in the laboratory principally because of

1. Disturbance during sampling, storage, and preparation of specimens;
2. Recompression of gas bubbles in the soil voids; and
3. Errors in test procedures and methods for interpreting test results, including difficulties in reapplying the initial state of stress that existed in situ.

The main objective of this chapter is to recommend procedures for estimating the consolidation settlements with a minimum of error, perhaps less than 50 percent: Without careful attention to all details the estimates can be too large by more than an order of magnitude. Values of C_r that lie outside the range 0.005 to 0.05 should be questioned (the lower values are for clays of low plasticity and low overconsolidation ratio); values between 0.015 and 0.035 are more typical.

It is assumed that "undisturbed" samples of high quality have been obtained and that the net change in pressure due to construction activities and structural loading and subsequent changes in loading have been appropriately considered.

RECOMMENDED PROCEDURE FOR DETERMINATION OF RECOMPRESSION INDEX C_r

The test equipment is described in chapter 1 . The recommended test procedure and presentation of test data in this chapter differ somewhat from those described in chapter 1 because the primary objectives treated are different.

It is good practice first to reapply p_o' and then to surround the sample with water and allow the dial reading to "equilibrate" (for at least 24 hours) before consolidation testing is commenced. The change in height should be recorded; if there is a swelling tendency, an additional stress increment should be applied.

The stress increments $\Delta\sigma$ in the consolidation test should be large enough so that the consolidation process can be identified from the settlement versus time readings (otherwise, pore water pressures must be measured to establish when the consolidation process has essentially ceased). In most soils this can be accomplished if the load increment ratio (LIR = $\Delta\sigma/\sigma'$) exceeds about 0.65. For lightly overconsolidated clays the minimum possible stress increment should be used; in any case, it is usually undesirable for the LIR to exceed 1. The measured data should be plotted to see whether a Terzaghi type of settlement versus time curve has been obtained (Figure 9). Only the increments in (R_{100} - R_o) should be used in plotting the curve of void ratio e versus effective vertical stress σ' to avoid cumulative additions of initial and secondary compressions. The duration of each load increment should be kept approximately constant, and long periods of secondary compression should be avoided. Ideally, the settlement beyond R_{100} should be less than 10 percent of the (R_{100} - R_o) value for the increment.

Figure 10 shows a typical curve of e versus σ' for 3 cycles of loading and unloading. The recompression index C_r depends on

Figure 8. Typical plot of soil properties versus depth.

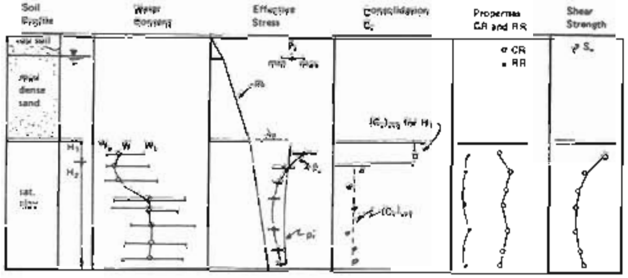


Figure 9. Typical curve of time versus settlement.

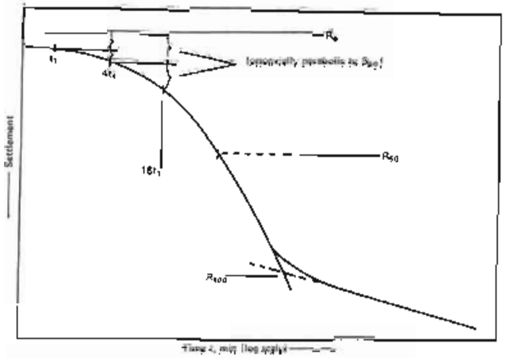


Figure 10. Typical curve of void ratio versus effective vertical stress.

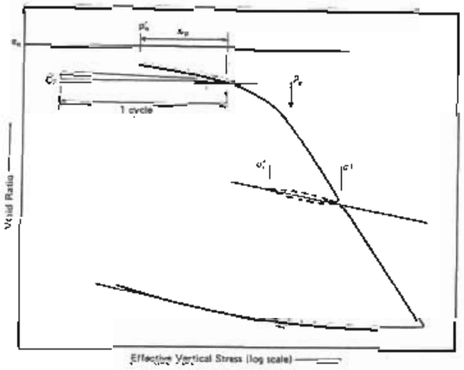
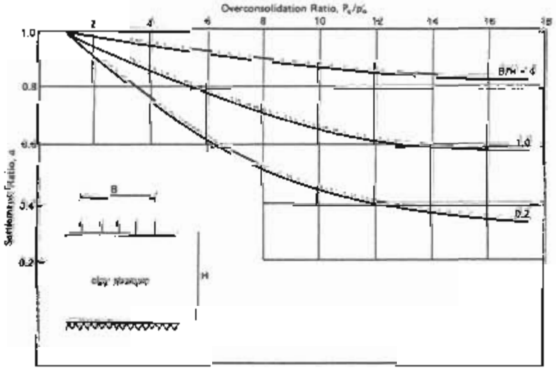


Figure 11. Relation between settlement ratio and over-consolidation ratio.



1. The magnitude of σ' at which unloading is begun, especially whether σ' is less than or exceeds the preconsolidation pressure P_c;
2. The overconsolidation ratio OCR to which rebounding (and reloading) is allowed (e.g., σ'/σ_r' in Figure 10); and
3. The degree to which gas bubbles are present in the sample because of the reduction in pore water pressure that resulted from sampling.

To reproduce as closely as possible the initial in situ state of stress, one should consolidate the sample to a stress slightly less than P_c and then allow it to rebound. Early in the testing program, before values of P_c have been established, the sample should initially be consolidated only to ($p_o' + \Delta p$): In either case, C_r should be evaluated over the range $p_o' + \Delta p$), as shown in Figure 10. Limited data indicate that C_r values obtained when a back pressure (to redissolve any gases in the soil voids) has been applied to the sample are less than those obtained when conventional apparatus is used. The use of consolidometers that permit application of a back pressure is recommended; alternately, the measured values of C_r may be appropriately reduced (about 25 to 50 percent).

CALCULATIONS OF CONSOLIDATION SETTLEMENTS

The values of C_r obtained as recommended in the previous section are used to calculate the apparent consolidation settlement S from equation 4. The consolidation settlement S_c is given by

$$S_c = \alpha S \qquad (5)$$

The coefficient α depends on the overconsolidation ratio ($OCR = P_c/p_o'$) in the clay stratum and the effect of small departures from 1-dimensional compression on the initial excess pore pressures produced by Δp. If the width of the loaded area exceeds 4 times the thickness of the clay stratum or if the depth to the top of the clay stratum exceeds twice the width of the loaded area, α may be assumed to be equal to 1. If loads are applied directly on the clay stratum, values of α may be interpreted from Figure 11, depending on the OCR and the ratio of the width of the loaded area B to the thickness H of the clay stratum. (Factors other than previous effective stresses can produce an overconsolidation effect in clays. These factors include long-term secondary compressions, changes in the ionic content of the pore water, weathering of clay minerals, and precipitation of cementations compounds. Such "bonded" clays are generally lightly overconsolidated, but their pore pressure response differs from that of clay deposits that are overconsolidated as a result of past changes in effective stress. Accordingly, Figure 11 is not applicable to bonded clays.) If α is less than about 0.7, the immediate settlements caused by shear strains may be significant compared to the consolidation settlements; accordingly, an estimate of the immediate settlements (based on elastic theory) would also be in order.

3 Estimating 1-Dimensional Consolidation, Including Secondary Compression, of Clay Loaded From Overconsolidated to Normally Consolidated State

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STATEMENT OF PROBLEM

A large shallow foundation is to be placed at a site underlain by saturated clay. The initial in situ stresses are less than the maximum past (preconsolidation) pressure P_c, i.e., in its initial state the soil behaves as an overconsolidated soil. (The past history of the soil that caused P_c is immaterial to the present discussion.) The final imposed stresses, due to the foundation loads and the overburden pressure, will exceed P_c. The geometry is such that a 1-dimensional analysis is valid. The problem is to estimate the rate and magnitude of the settlement that will occur, including recompression, virgin compression, and secondary compression.

PRIMARY CONSOLIDATION AND SECONDARY COMPRESSION

The solution outlined in this chapter assumes that primary and secondary effects can arbitrarily be separated.

1. Primary consolidation is the time-dependent volume change that takes place because and during the time when the excess pore pressure is dissipating and the effective stress is increasing.
2. Secondary compression is the time-dependent volume change that takes place at essentially constant effective stress and thus occurs after the completion of primary consolidation.

The above approach is the most commonly used approach although the above definitions are not universally accepted.

GENERAL CONSIDERATIONS

Inasmuch as volume changes are considered to be 1-dimensional, the settlement of each segment of the clay stratum, within which the soil properties and stresses do not vary greatly, may be computed from primary consolidation settlement S_p and secondary compression S_s by the following equations:

$$S_p = \sum \left\{ \frac{C_r}{1 + e_o} H_o \log \frac{p_c}{p_o'} + \frac{C_c}{1 + e_o} H_o \log \frac{p_f'}{p_c} \right\}_i \qquad (6)$$

$$S_s = \sum \left\{ \frac{C_\alpha}{1 + e_o} H_o \right\}_i \log \frac{t_s}{t_p} \qquad (7)$$

For instantaneous load applications, all times are measured from the instant at which load is applied. When the load is applied gradually over a finite time interval, engineering judgment must be used to select t_o, the initial time from which all time values are measured. For loads that increase approximately linearly with time, t_o may be estimated as the time at which one-half of the total load has been applied (28). This estimate also is satisfactory for irregular load applications when the total loading period is less than 20 percent of t_p. However, if the loading period is large compared to the time required for completion of primary consolidation, more complex methods, e.g., numerical analysis, are required for computing the rate of primary consolidation. Such cases are beyond the scope of this discussion.

The first task is to identify the number and thickness of layers to be considered. This can be done with the help of a plot such as that shown in Figure 8. At least 2 consolidometer tests should be performed on samples from each layer and from each 10 ft (3 m) of thickness of a single layer. Data on water content and Atterberg limits should be available for every 5 ft (1.5 m) of depth.

WORKING HYPOTHESIS

As stated earlier, it is assumed that primary consolidation and secondary compression can be separated into 2 distinct processes. Primary consolidation can be predicted from Terzaghi's 1-dimensional consolidation theory or extensions of his theory (6, 21); therefore, this discussion will be restricted to secondary effects.

As a working hypothesis of secondary compression for engineering practice, Ladd (10) suggested the following assumptions for cases in which the load increment ratio is of sufficient magnitude to cause some primary consolidation:

1. C_α is independent of time, at least during the time span of interest.
2. C_α is independent of the thickness of the soil layer (of course, the thicker the layer is, the longer the time required for primary consolidation will be, but the strain, or Δe/log cycle of time, remains constant);
3. C_α is independent of load increment ratio as long as some primary consolidation occurs; and
4. $C_\alpha/(\Delta e/\Delta \log \sigma')$ at any given stress is constant (at least approximately), and for many normally consolidated clays over the normal range of engineering stresses, $(\Delta e/\Delta \log \sigma')_{max} = C_c$ = constant and thus C_α/C_c is constant for such a clay.

Behavior resulting from the working hypothesis is shown in Figure 12. The effects are shown of varying drainage distance, consolidation stress, and load increment ratio for a normally consolidated clay with a constant compression index.

The working hypothesis is admittedly an oversimplification of actual behavior. However, there are data to support, at least as a first approximation, the assumptions. Some of the references are listed below:

C_α Assumption	References
Independent of thickness	14, 19
Independent of load increment ratio	14, 19, 29
Independent of consolidation stress for constant compression index	8, 14, 16, 19, 23

TEST APPARATUS AND PROCEDURES

The test apparatus and procedures that are recommended for this application are the same as those recommended in chapter 1 for evaluation of P_c.

EVALUATION OF INDIVIDUAL TEST RESULTS

The dial gauge reading should be plotted against the logarithm of elapsed time since the addition of a new load for each load increment. This can best be accomplished on semilogarithmic paper. If an S-shaped curve results, the standard logarithmic (Casagrande) curve-fitting method may be used. If not, the methods suggested by Taylor (27) or Su (26) may be used and may be necessary for small load increment ratios. These methods are discussed below.

1. Semilogarithm method by Casagrande. The logarithm of time method is shown in Figure 13. It assumes that for the first 50 percent primary consolidation the graph is parabolic so that the settlement from $t = 0$ to $t = t_1$ equals the settlement from $t = t_1$ to $t = 4t_1$, provided the consolidation at $4t_1$ has not exceeded 50 percent primary compression. Thus, the 0 consolidation and seating correction should be the same as that obtained from the square root of time method. The 100 percent primary consolidation is obtained from drawing a tangent at the point of inflection of the S-shaped graph (i.e., the steepest slope) and an approximate straight line through the tail points of the graph.
2. Semilogarithmic method by Su. Su makes use of the maximum slope of the compression and logarithm of time plot. Using the Fourier series solution for Terzaghi consolidation theory, Su shows that

$$R_u = R_o + \frac{\text{max slope}}{0.688} U \qquad (8)$$

This method is shown in Figure 14. A tangent to the steepest part of the settlement curve intersects a horizontal line through the corrected 0 consolidation point (as determined by the logarithm of time method) at point A. Point B is 1.5 times the distance of 1 log cycle along the horizontal line from A. From point B a vertical line intersects the tangent at C, which is the 100 percent primary consolidation point. The method may be applied to curves that do not exhibit the characteristic S shape; or, when secondary time effects are not of interest, the method may be applied before the straight-line tail is established, thus reducing the experimental test time required.

3. Square root of time method by Taylor. If dial readings are plotted versus the square root of time (Figure 15), a straight line can be fitted through the data points for the first 60 percent primary consolidation. This line can be extrapolated to $t = 0$ to obtain the theoretical 0 dial reading. A straight line then is constructed through the theoretical 0 point with a slope such that abscissas are 1.15 times the abscissas on the experimental curve for corresponding dial readings. This line will intersect the experimental curve at 90 percent consolidation. The 100 percent consolidation can be determined by taking (10/9) times the difference between the dial readings at theoretical

Figure 12. Illustration of 6 hypotheses for secondary compression.

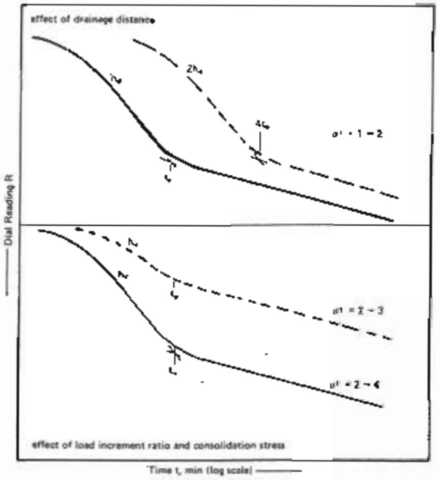


Figure 15. Taylor curve-fitting method.

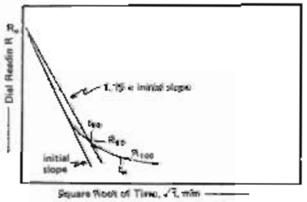


Figure 13. Casagrande curve-fitting method.

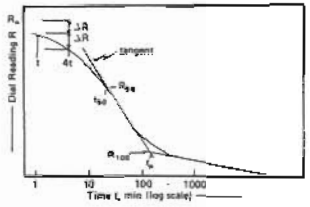


Figure 14. Su curve-fitting method.

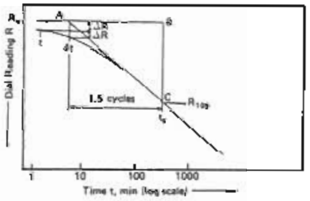
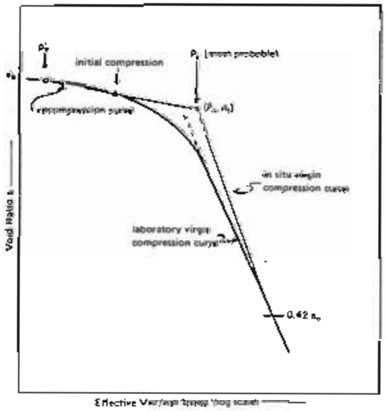


Figure 16. Schmertmann construction of in situ compression curve.



0 and 90 percent consolidation. The coefficient of consolidation C_v is computed as

$$C_v = T_{90}\left(\frac{h_d^2}{t_{90}}\right) \tag{9}$$

T_{90} is the theoretical time factor for 90 percent consolidation, = 0.85, t_{90} is the laboratory time for 90 percent consolidation, and h_d is the drainage distance in the laboratory sample at t_{90}.

These methods will enable the values of the time to reach the end of primary consolidation, the amount of primary compression or strain, and the secondary compression coefficient C_α or secondary compression ratio SCR to be directly obtained for each load increment.

The dial gauge readings corresponding to the end of primary consolidation obtained from the above plots should now be plotted against the logarithm of the applied vertical stress. This again is best accomplished on semilogarithmic paper. This plot enables the laboratory virgin compression index C_c or virgin compression ratio CR and the minimum and probable P_c to be obtained as outlined in chapter 1. If the soil specimen is heavily overconsolidated, then the recompression index C_r or recompression ratio RR may also be obtained from a straight line through the dial gauge readings at the overburden pressure and at half the initially estimated value of P_c. If the soil is lightly overconsolidated, resulting in no initial expansion, the recompression ratio may be disregarded and made equal to 0.

The laboratory virgin compression index may be corrected for sample disturbance effects by the method suggested by Schmertmann (24).

In this method, the correction to the laboratory virgin compression line is obtained as follows (Figure 16):

1. The recompression curve is extended from the overburden stress p_o' to the most probable preconsolidation pressure (P_c, e_o); and
2. The in situ virgin compression line is constructed from (P_o', e_o) to intersect the laboratory virgin compression line at 0.42 e_o.

Experience indicates that for many soft to medium clays this correction increases the virgin compression index by approximately 10 to 20 percent.

For each specimen tested, the following results should be summarized for each pressure increment:

1. Final increment pressure,
2. Strain or void ratio at the end of primary consolidation,
3. Coefficient of consolidation, and
4. Coefficient of secondary compression or secondary compression ratio.

Similarly for each specimen tested, the following data should be recorded:

1. Depth,
2. Initial effective overburden stress,
3. Final effective vertical stress imposed by the overburden and engineering structure, if known,
4. Minimum P_c,
5. Probable P_c,
6. Maximum P_c,
7. Recompression index or ratio, and
8. Virgin compression index or ratio.

Plotting these data versus depth, as shown in Figure 8, is considered to be essential to better selecting the probable P_c with depth and possibly in interpreting the previous geological history of the deposit.

EVALUATION OF OVERALL SOIL PROPERTIES

Amount of Primary Settlement

The depth-stress profile for the whole deposit should be plotted. From these results, one can select a depth-probable preconsolidation profile, bearing in mind any relevant details of known geological history and general soil behavior (i.e., known erosion from studies of other sites, induced preconsolidation due to cementation or secondary compression) and the fact that sample disturbance will lower P_c.

Once this profile has been established, the total primary settlement may be calculated from equation 6. If several tests represent 1 soil layer or increment of depth, these should be averaged, neglecting, of course, any results that are suspiciously incorrect.

Rate of Primary Settlement

The rate of primary consolidation is characterized by the coefficient of consolidation C_v, which is determined in the laboratory as

$$C_v = T_{50}\left(\frac{h_d^2}{t_{50}}\right) \tag{10}$$

T_{50} is the theoretical time factor for 50 percent consolidation, ≃ 0.2, t_{50} is the laboratory time for 50 percent consolidation, and h_d is the drainage distance in the laboratory sample at t_{50}. Alternately, C_v may be computed at 90 percent consolidation by the Taylor method. Values computed by the Taylor method generally are larger than the corresponding values from equation 10.

In general, C_v will vary with stress level during a consolidation test. The variation may be determined by plotting C_v versus the final stress for each load increment. The value of C_v corresponding to the final in situ stress p_f' at the sample depth then should be selected. (Because of sample disturbance and load increment ratio effects, some adjustment may be required when $p_f' < 2P_c$. For such cases, it is suggested that C_v corresponding $\sigma' = 2P_c$ be used.)

Primary consolidation occurs much more rapidly in the overconsolidated range than in the normally consolidated range. For the conventional sizes of consolidation test specimens, sufficient time data for the evaluation of C_v may not be obtained in the overconsolidated ranges. In such circumstances, the value of $(C_v)_{\infty}$ for the overconsolidated range may be estimated approximately as

$$(C_v)_{\infty} = \frac{C_c}{C_r}(C_v)_{vc} \tag{11}$$

Other alternatives are to use larger specimens or dynamic recording instruments or to measure the permeability directly, but these alternatives are beyond the scope of this chapter.

Consider first the case of the entire soil layer loaded into the virgin compression range. The selected value of C_v from each test is plotted against sample depth, and an average value is estimated for the entire layer. The time t_u for any percentage of consolidation may then be estimated from

$$t_u = T_v\frac{h_d^2}{C_v} \tag{12}$$

h_d is the estimated in situ drainage distance for the entire layer and depends on the evaluation of drainage conditions at the layer boundaries. If the layer is assumed to drain at both top and bottom, h_d is one-half of the entire layer thickness. If the layer is assumed to drain at only 1 surface, h_d equals the entire layer thickness.

In the second case, a portion of the layer, either near the surface or deep in the deposit, may remain overconsolidated. In this case, one average C_v should be estimated over the depth that is loaded into the virgin compression range, and another average C_v should be selected over the depth that remains overconsolidated. The problem then is one of the consolidation of 2 contiguous layers with different values of C_v. The rate of consolidation may be approximated by replacing the thickness of overconsolidated soil by an equivalent thickness of soil in the virgin compression range. Let

$$H' = H_{\infty}\sqrt{\frac{(C_v)_{vc}}{(C_v)_{oc}}} \tag{13}$$

The rate of consolidation can then be determined from equation 12 by using $C_v = (C_v)_{vc}$. The equivalent layer thickness H' plus the thickness of soil loaded into the virgin compression range is the effective layer thickness.

Rate of Secondary Compression

The rate of secondary compression of the soil should be characterized by the coefficient of secondary compression or the secondary compression ratio for the load increment whose final stress approximates the final in situ stress at the same depth. Tests at any one depth should first be averaged in a similar fashion to that described for primary settlement. The secondary compression can then be evaluated from equation 7.

t_p in equation 7 must be estimated from the in situ rate of primary consolidation. If the typical S-shaped curve is obtained on a semilogarithmic plot, t_p can be estimated as the time coordinate of the intersection of the tangent constructed at the inflection point and the extension of the linear secondary compression line (Figure 12). If an S-shaped curve is not obtained, t_p may be estimated as the time to complete 100 percent primary consolidation as determined by the Su or Taylor methods.

Combined Primary and Secondary Compression

The total compression of a layer is determined by combining equations 6 and 7. Thus,

$$S(t) = U(t)\,S_p \tag{14}$$

for $t \le tp$, and

$$S(t) = S_p + S_{\alpha}(t) \tag{15}$$

for $t \ge tp$.

CONCLUDING REMARKS

Although this chaper has been restricted to the use of the incremental consolidometer tests, the reader should not infer that incremetal tests are superior to other types of tests such as the constant rate of strain or controlled gradient tests. Nevertheless, the incremental test is the most common test in present-day practice, and the standardization of its interpretation is highly desirable.

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