Compressibility and consolidation

Structures are built on soils. They transfer loads to the subsoil through the foundations. The effect of the loads is felt by the soil normally up to a depth of about two to three times the width of the foundation. The soil within this depth gets compressed due to the imposed stresses. The compression of the soil mass leads to the decrease in the volume of the mass which results in the settlement of the structure.

The compressibility characteristics of a soil mass might be due to any or a combination of the following factors:

1. Compression of the solid matter
2. Compression of water and air within the voids, and
3. Escape of water and air from the voids.

Consolidation

When a saturated clay-water system is subjected to an external pressure, the pressure applied is initially taken by the water in the pores resulting there by in an excess pore water pressure. If drainage is permitted, the resulting hydraulic gradients initiate a flow of water out of the clay mass and the mass begins to compress. This process, involving a gradual compression occurring simultaneously with a flow of water out of the mass and with a gradual transfer of the applied pressure from the pore water to the mineral skeleton is called **consolidation**. The process opposite to consolidation is called **swelling**, which involves an increase in the water content due to an increase in the volume of the voids.

Consolidation may be due to one or more of the following factors:

1. External static loads from structures,
2. Self-weight of the soil such as recently placed fills,
3. Lowering of the ground water table, and
4. Desiccation.

The total compression of a saturated clay strata under excess effective pressure may be considered as the sum of

1. Immediate compression,
2. Primary consolidation, and
3. Secondary compression.

Immediate settlement is due to the immediate compression of the soil layer under undrained condition and is calculated by assuming the soil mass to behave as an elastic soil.

If the rate of compression of the soil layer is controlled solely by the resistance of the flow of water under the induced hydraulic gradients, the process is referred to as **primary consolidation**. The portion of the settlement that is due to the primary consolidation is called primary consolidation settlement or compression. At the present time the only theory of practical value for estimating time-dependent settlement due to volume changes, that is under primary consolidation is the one-dimensional theory.

The **third part** of the settlement is due to secondary consolidation or compression of the clay layer. This compression is supposed to start after the primary consolidation ceases, that is after the excess pore water pressure approaches zero. It is often assumed that secondary compression proceeds linearly with the logarithm of time. However, a satisfactory treatment of this phenomenon has not been formulated for computing settlement under this category.

The process of consolidation

The process of consolidation of a clay-soil-water system may be explained with the help of a mechanical model as described by **Terzaghi** (1936).

The model consists of a cylinder with a frictionless piston as shown in **Fig. 1**. The space underneath the piston is completely filled with water. The springs represent the mineral skeleton in the actual soil mass and the water below the piston is the pore water under saturated conditions in the soil mass. When a load of **P** is placed on the piston, this stress is fully transferred to the water, the water pressure increases. The pressure in the water is

$$u = p$$

This is analogous to pore water pressure, **u**, that would be developed in a clay-water system under external pressures. If the whole model is leakproof without any holes in the piston, there is no chance for the water to escape. If a few holes are made in the piston, the water will immediately escape through the holes. With the escape of water through the holes a part of the load carried by the water is transferred to the springs. This process of transference of load from water to spring goes on until the flow stops when all the load will be carried by the spring and none by the water. The time required to attain this condition depends upon the number and size of the holes made in the piston. A few small holes represent a clay soil with poor drainage characteristics. When the spring-water system attains equilibrium condition under the imposed load, the settlement of the piston is analogous to the compression of the clay-water system under external pressures.

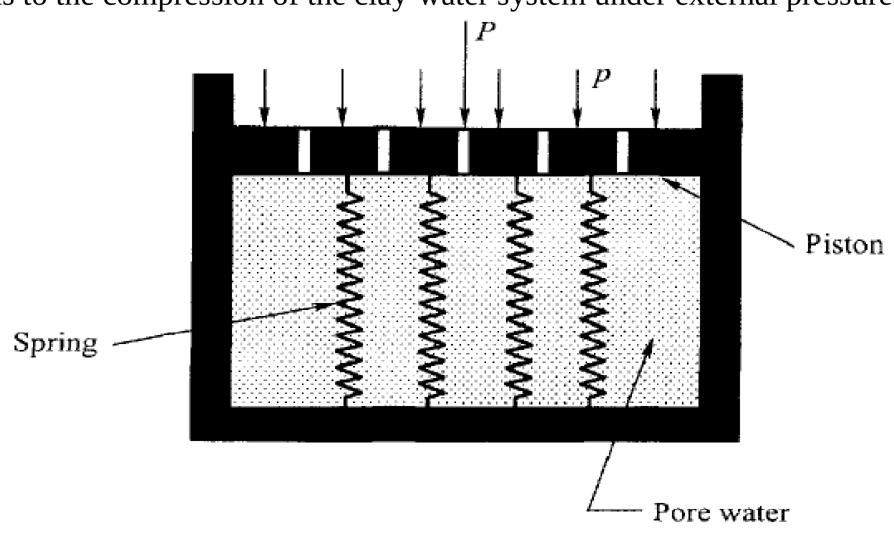


Fig. 1: Mechanical model to explain the process of consolidation

One-dimensional consolidation

In many instances the settlement of a structure is due to the presence of one or more layers of soft clay located between layers of sand or stiffer clay as shown in **Fig. 2**. The adhesion between the soft and stiff layers almost completely prevents the lateral movement of the soft layers. The theory that was developed by Terzaghi (1925) on the basis of this assumption is called the one-dimensional consolidation theory. In the laboratory this condition is simulated most closely by the confined compression or consolidation test.

The process of consolidation as explained with reference to a mechanical model may now be applied to a saturated clay layer in the field. If the clay strata is subjected to an excess pressure **Δp** due to a uniformly distributed load **p** on the surface, the clay layer is compressed over time and excess pore water drains out of it to the sandy layer. This constitutes the process of consolidation. At the instant of application of the excess pressure **Δp**, the load is carried entirely by water in the voids of the soil. As time goes on the excess pore water pressure decreases, and the effective vertical pressure in the layer correspondingly increases. At any point within the consolidating layer, the value **u** of the excess pore water pressure at a given time may be determined from

$$u = u_i - \Delta p_z$$

Where:

u is excess pore water pressure at depth **z** at any time **t**,

$\mathbf{u_i}$ is initial total pore water pressure at time **t** is **0**, and

$\mathbf{\Delta p_z}$ is effective pressure transferred to the soil grains at depth **i** and time **t**.

At the end of primary consolidation, the excess pore water pressure **u** becomes equal to zero. This happens when **u** is **0** at all depths.

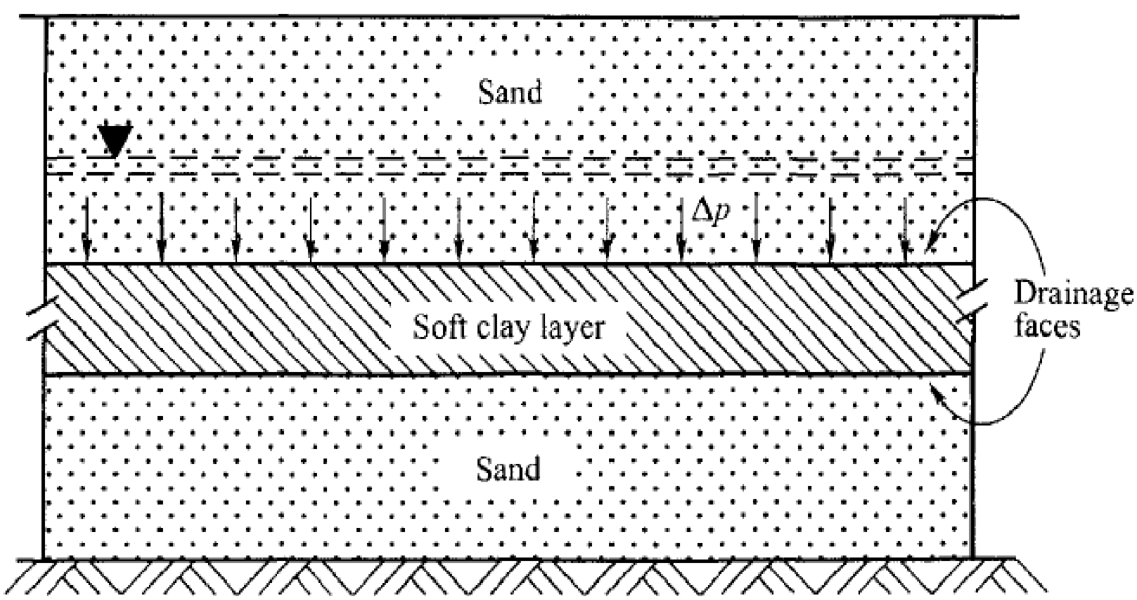


Fig. 2: Clay layer sandwiched between sand layers

Consolidometer

The compressibility of a saturated, clay-water system is determined by means of the apparatus shown diagrammatically in **Fig. 3**. This apparatus is also known as an **Oedometer**.

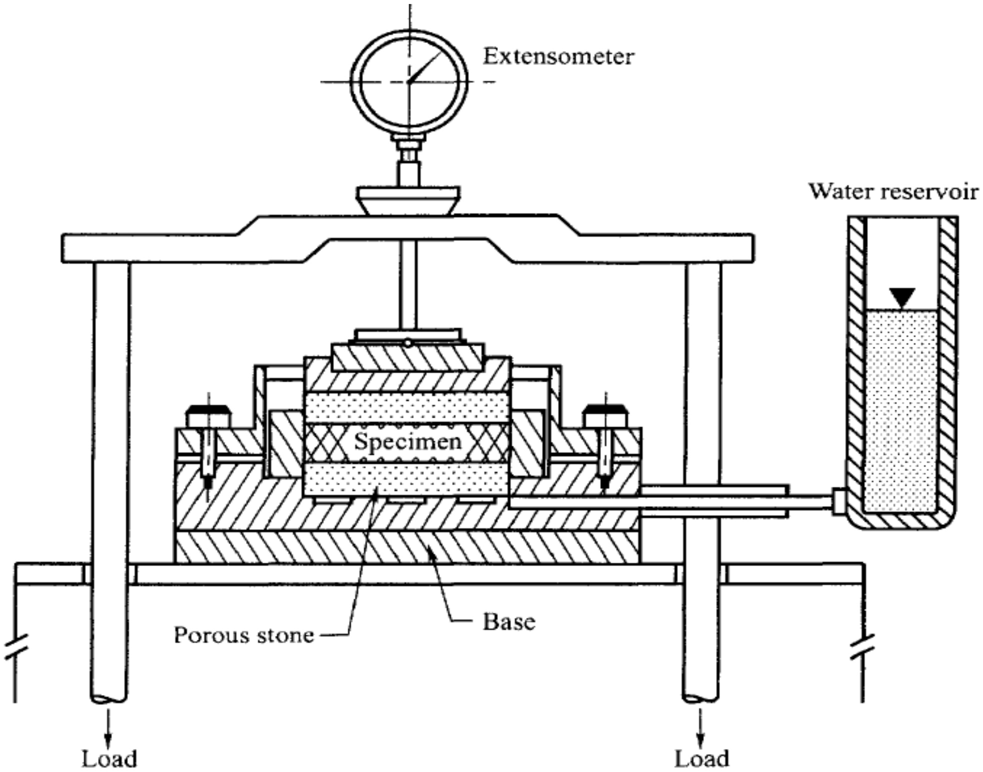


Fig. 3: A schematic diagram of a consolidometer

The standard one-dimensional consolidation test

The main purpose of the consolidation test on soil samples is to obtain the necessary information about the compressibility properties of a saturated soil for use in determining the magnitude and rate of settlement of structures. The following test procedure is applied to any type of soil in the standard consolidation test.

Loads are applied in steps in such a way that the successive load intensity, **p**, is twice the preceding one. The load intensities commonly used to be 25, 50, 100, 200, 400, 800 and 1600kN/m^2. Each load is allowed to stand until compression has practically ceased (no longer than 24 hours). The dial readings are taken at elapsed times of 1/4, 1/2, 1,2, 4, 8, 15, 30, 60, 120, 240, 480 and 1440 minutes from the time the new increment of load is put on the sample. Sandy samples are compressed in a relatively short time as compared to clay samples and the use of one-day duration is common for the latter.

After the greatest load required for the test has been applied to the soil sample, the load is removed in decrements to provide data for plotting the expansion curve of the soil in order to learn its elastic properties and magnitudes of plastic or permanent deformations.

The following data should also be obtained:

1. Moisture content and weight of the soil sample before the commencement of the test,
2. Moisture content and weight of the sample after completion of the test,
3. The specific gravity of the solids, and
4. The temperature of the room where the test is conducted.

Pressure-void ratio curve

The pressure-void ratio curve can be obtained if the void ratio of the sample at the end of each increment of load is determined. Accurate determinations of void ratio are essential and may be computed from the following data

1. The cross-sectional area of the sample **A**, which is the same as that of the brass ring,
2. The specific gravity, $\mathbf{G_S}$, of the solids,
3. The dry weight, $\mathbf{W_S}$, of the soil sample, and
4. The sample thickness, **H**, at any stage of the test.

If the thickness **H** of the sample is known at any stage of the test, the void ratio, **e**, at all the stages of the test may be determined by,

$$e = \frac{AH - AH_S}{AH_S} = \frac{H - H_S}{H_S}$$

Change of void-ratio method

In one-dimensional compression the change in height **ΔH** per unit of original height **H** equals the change in volume **ΔV** per unit of original volume **V**.

$$\frac{\Delta H}{H} = \frac{\Delta V}{V}$$

V may now be expressed in terms of void ratio **e**.

$$\frac{\Delta V}{V} = \frac{e_o - e}{1 + e_o} = \frac{\Delta e}{1 + e_o}$$

Therefore,

$$\boldsymbol{\frac{\Delta H}{H} = \frac{\Delta V}{V} = \frac{\Delta e}{1 + e_o}} \qquad (\mathbf{1})$$

$$\Delta e = \frac{\Delta H}{H}(1 + e_o)$$

Where:

Δe is change in void ratio under a load, **H,** is initial height of sample, **e** is initial void ratio of sample, $\mathbf{e_1}$ is void ratio after compression under a load, **ΔH** is compression of sample under the load which may be obtained from dial gauge readings.

Typical pressure-void ratio curves for an undisturbed clay sample are shown in **Fig. 4**, plotted both on arithmetic and on semi log scales. The curve on the log scale indicates clearly two branches, a fairly horizontal initial portion and a nearly straight inclined portion. The coordinates of point **A** in the figure represent the void ratio $\mathbf{e_o}$ and effective overburden pressure $\mathbf{p_o}$ corresponding to a state of the clay in the field as shown in the inset of the figure. When a sample is extracted by means of the best of techniques, the water content of the clay does not change significantly. Hence, the void ratio $\mathbf{e_o}$ at the start of the test is practically identical with that of the clay in the ground. When the pressure on the sample in the consolidometer reaches $\mathbf{p_o}$, the **e-logp** curve should pass through the point A unless the test conditions differ in some manner from those in the field. In reality the curve always passes below point **A**, because even the best sample is at least slightly disturbed.

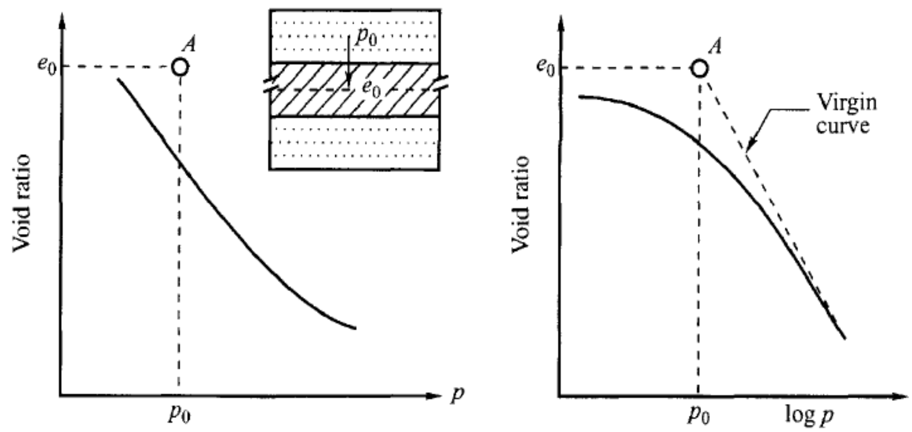


Fig. 4: Pressure-void ratio curves

Normally consolidated and over consolidated clay

A clay is said to be **normally consolidated** if the present effective overburden pressure, $\mathbf{p_o}$ is the maximum pressure to which the layer has ever been subjected at any time in its history, whereas a clay layer is said to be **over consolidated** if the layer was subjected at one time in its history to a greater effective overburden pressure, $\mathbf{p_c}$, than the present pressure, $\mathbf{p_o}$. The ratio $\mathbf{p_c/p_o}$ is called the over consolidation ratio (**OCR**).

Over consolidation of a clay stratum may have been caused due to some of the following factors,

1. Weight of an overburden of soil which has eroded,
2. Weight of a continental ice sheet that melted, and
3. Desiccation of layers close to the surface.

Experience indicates that the natural **moisture content** is commonly close to the **liquid limit**, for normally consolidated clay soil whereas for the over consolidated clay **moisture content** is close to **plastic limit**.

Computation of consolidation settlement

Settlement equations for normally consolidated clay

For computing the ultimate settlement of a structure founded on clay the following data are Required,

1. The thickness of the clay stratum, **H**,
2. The initial void ratio, $\mathbf{e_o}$,
3. The consolidation pressure $\mathbf{p_o}$ or $\mathbf{p_c}$, and
4. The field consolidation curve $\mathbf{K_f}$.

The slope of the field curve **Kf.** on a semi logarithmic diagram is designated as the compression index **Cc** in **Fig. 5**. The equation for **Cc** may be written as,

$$C_c = \frac{e_o - e}{\log p - \log p_o} = \frac{\Delta e}{\frac{\log p}{\log p_o}}$$

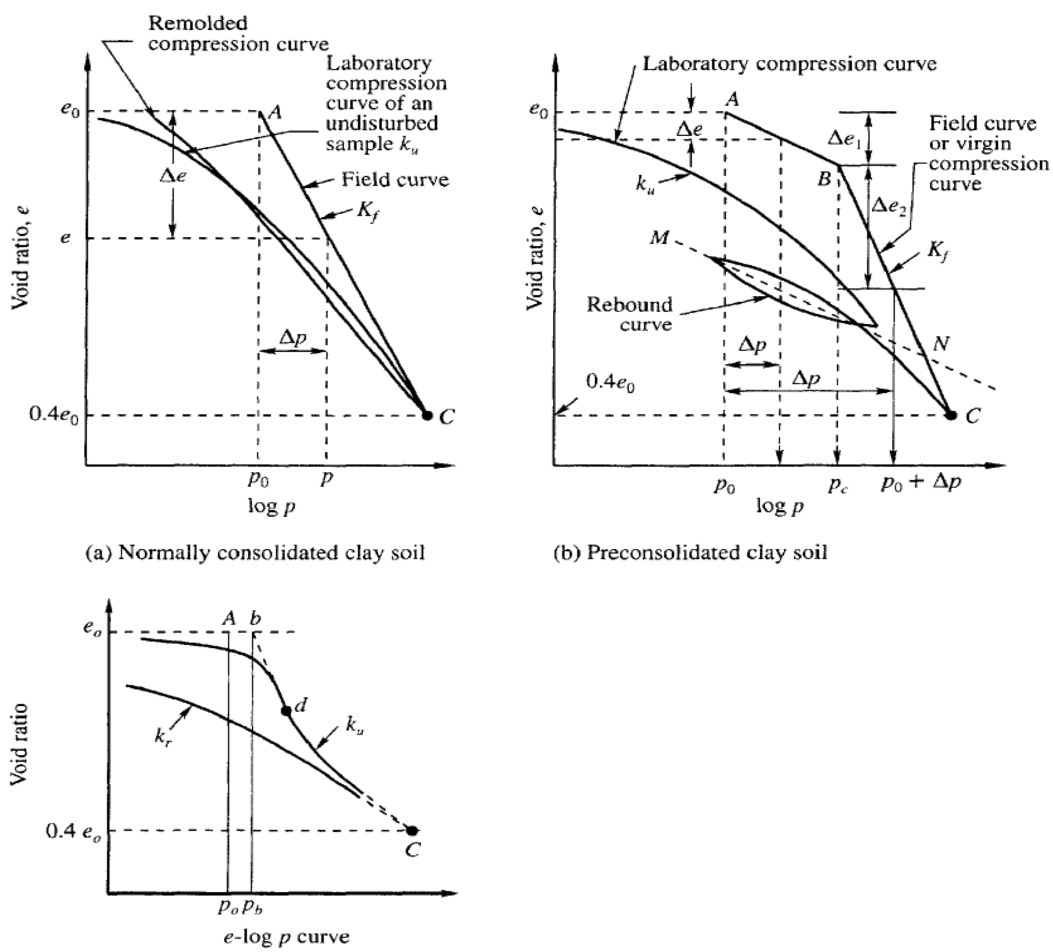
$$C_c = \frac{\Delta e}{\log \frac{p}{p_o}} \qquad (2)$$

In one-dimensional compression, as per **Eqn. 1**, the change in height **ΔH** per unit of original **H** may be written as equal to the change in volume **ΔV** per unit of original volume **V** as

$$\Delta H = H \frac{\Delta e}{1 + e_o}$$

If we designate the compression **ΔH** of the clay layer as the total settlement **S**$_t$ of the structure built on it, we have,

$$\mathbf{S_t = \Delta H = H \frac{\Delta e}{1 + e_o}} \qquad \mathbf{(3)}$$



(a) Normally consolidated clay soil

(b) Preconsolidated clay soil

(c) Typical e-log p curve for an undisturbed sample of clay of high sensitivity (Peck et al., 1974)

Fig. 5: Field e-logp curves

Settlement calculation from e-log p curves

Substituting for **ΔH** in **Eqn. 3** we have,

$$S_t = \frac{Cc}{1 + e_o} H \log \frac{p}{p_o}$$

$$\mathbf{S_t = \frac{Cc}{1 + e_o} H \log \frac{p_o + \Delta p}{p_o}} \qquad (4)$$

The net change in pressure **Δp** produced by the structure at the middle of a clay stratum is calculated from the **Boussinesq** or **Westergaard** theories. If the thickness of the clay stratum is too large, the stratum may be divided into layers of smaller thickness not exceeding **3m**. The net change in pressure **Δp** at the middle of each layer will have to be calculated. Consolidation tests will have to be completed on samples taken from the middle of each of the strata and the corresponding compression indices will have to be determined. The equation for the total consolidation settlement may be written as,

$$\mathbf{S_t = H_i \frac{Cc}{1 + e_o} \log \frac{p_o + \Delta p}{p_o}} \qquad (5)$$

Where the subscript **i**, refers to each layer in the subdivision. If there is a series of clay strata of thickness H_1, H_2, etc., separated by granular materials, the same may be used for calculating the total settlement.

Settlement calculation from e-p curves

We can plot the field **e-p** curves from the laboratory test data and the field **e-logp** curves. The weight of a structure or of a fill increases the pressure on the clay stratum from the overburden pressure $\mathbf{p_o}$ to the value $\mathbf{p_o+\Delta p}$ as shown in **Fig. 6**. The corresponding void ratio decreases from $\mathbf{e_o}$ to **e**.

Hence, for the range in pressure from $\mathbf{p_o}$ to $\mathbf{p_o+\Delta p}$, we may write as,

$$e_o - e = \Delta e = a_v \Delta p$$

Where, $\mathbf{a_v}$ is called the coefficient of compressibility.

For a given difference in pressure, the value of the coefficient of compressibility decreases as the pressure increases. Now substituting for **Δe** in **Eqn. 3**, we have the equation for settlement,

$$\mathbf{S_t = \frac{\Delta e H}{1 + e_o} = \frac{a_v H}{1 + e_o} \Delta p = m_v H \Delta p} \qquad (6)$$

Where, m_v is known as the coefficient of volume compressibility.

$$m_v = \frac{a_v}{1 + e_o}$$

It represents the compression of the clay per unit of original thickness due to a unit increase of the pressure.

Settlement calculation from e-log p curve for overconsolidated clay soil

Fig. 5. b gives the field curve $\mathbf{K_f}$ for pre-consolidated clay soil. The settlement calculation depends upon the excess foundation pressure **Δp** over and above the existing overburden pressure $\mathbf{p_o}$.

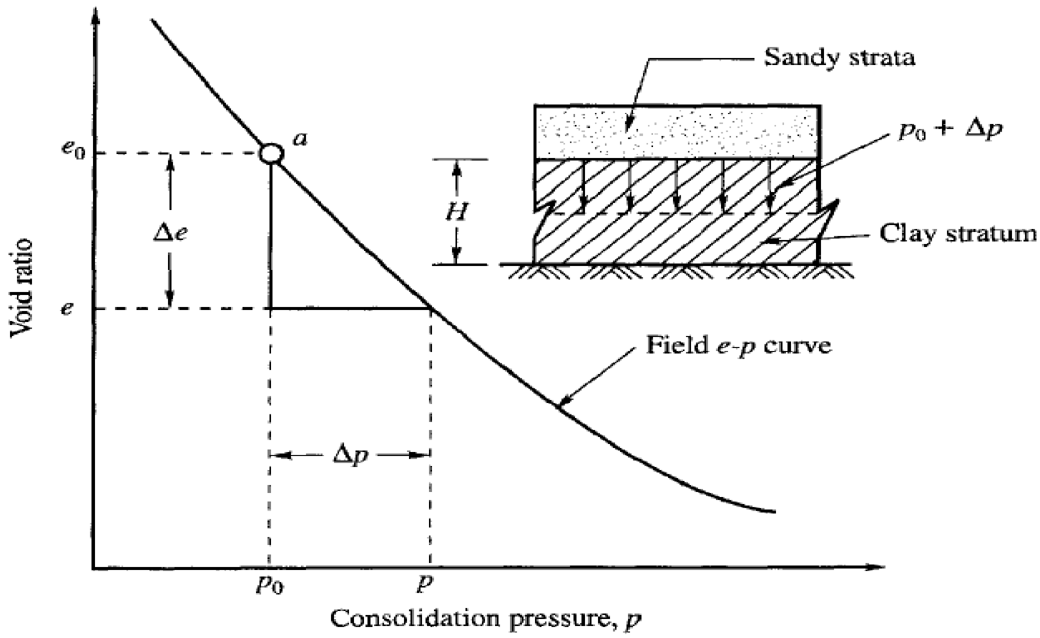


Fig. 6: Settlement calculation from e-p curve

Settlement computation, if $p_o + \Delta p < p_c$

In such a case, use the sloping line **AB**. If **C_S** is slope of this line (also called the swell index), we have,

$$C_S = \frac{\Delta e}{\log \frac{p_o + \Delta p}{p_o}}$$

Then,

$$\Delta e = C_S \log \frac{p_o + \Delta p}{p_o}$$

By substituting for **Δe** in **Eqn. 3**, we have,

$$S_t = \frac{C_S H}{1 + e_o} \log \frac{p_o + \Delta p}{p_o} \qquad (7)$$

Settlement computation, if $p_o < p_c < p_o + \Delta p$

We may write from Fig. **Fig. 5. b**

$$\Delta e = \Delta e_1 + \Delta e_2 = C_S \log \frac{p_c}{p_o} + C_c \log \frac{p_o + \Delta p}{p_c}$$

In this case the slope of both the lines **AB** and **BC** in **Fig. 5. B**, are required to be considered. Now the equation for **S_t** may be written as,

$$S_t = \frac{C_S H}{1 + e_o} \log \frac{p_c}{p_o} + \frac{C_c H}{1 + e_o} \log \frac{p_o + \Delta p}{p_c} \qquad (8)$$

The swell index **Cs** is approximately (0.2 to 0.1) **C_c** can be used as a check.

Nagaraj and Murthy (1985) have proposed the following equation for **Cs** as,

$$C_S = 0.0463 G_S L.L$$

Where, **L.L** is liquid limit, **G_S** is specific gravity of solids.

Compression index C_c - empirical relationships

Research workers in different parts of the world have established empirical relationships between the compression index $\mathbf{C_c}$ and other soil parameters. A few of the important relationships are given below.

Skempton's formula

Skempton (1944) established a relationship between $\mathbf{C_c}$, and liquid limits for remolded clays as,

$$C_c = 0.007(L.L - 10)$$

Where, **L.L** is in percent.

Terzaghi and Peck formula

Based on the work of Skempton and others, Terzaghi and Peck (1948) modified Skempton formula applicable to normally consolidated clays of low to moderate sensitivity as,

$$C_c = 0.009(L.L - 10)$$

Settlement due to secondary compression

In certain types of clays, the secondary time effects are very pronounced to the extent that in some cases the entire time-compression curve has the shape of an almost straight sloping line when plotted on a semi logarithmic scale, instead of the typical inverted **S**-shape with pronounced primary consolidation effects. These so called secondary time effects are a phenomenon somewhat analogous to the creep of other overstressed material in a plastic state. The factors which affect the rate of the secondary compression of soils are not yet fully understood, and no satisfactory method has yet been developed for a rigorous and reliable analysis and forecast of the magnitude of these effects. Highly organic soils are normally subjected to considerable secondary consolidation.

The rate of secondary consolidation may be expressed by the coefficient of secondary compression as,

$$\bar{C}_\alpha = \frac{\Delta e}{1 + e_o} * \frac{1}{\log \frac{t_2}{t_1}} = \frac{C_\alpha}{1 + e_o}$$

$$\Delta e = C_\alpha \log \frac{t_2}{t_1} = C_\alpha \log \frac{t}{t_p}$$

Where, C_α, the slope of the straight-line portion of the e-log t curve, is known as the secondary compression index. Numerically C_α is equal to the value of $\mathbf{\Delta e}$ for a single cycle of time on the curve **Fig. 7**. Compression is expressed in terms of decrease in void ratio and time has been normalized with respect to the duration $\mathbf{t_p}$ of the primary consolidation stage. A general expression for settlement due to secondary compression under the final stage of pressure $\mathbf{p_f}$ may be expressed as,

$$S_S = \frac{C_\alpha}{1 + e_o} H \log \frac{t}{t_p} \qquad (9)$$

Value of $\mathbf{\Delta e}$ from $\mathbf{t/t_p = 1}$ to any time **t** may be determined from **e** versus $\mathbf{t/t_p}$ curve corresponding to the final pressure $\mathbf{p_f}$, $\mathbf{e_o}$ is initial void ratio, **H** is thickness of the clay .

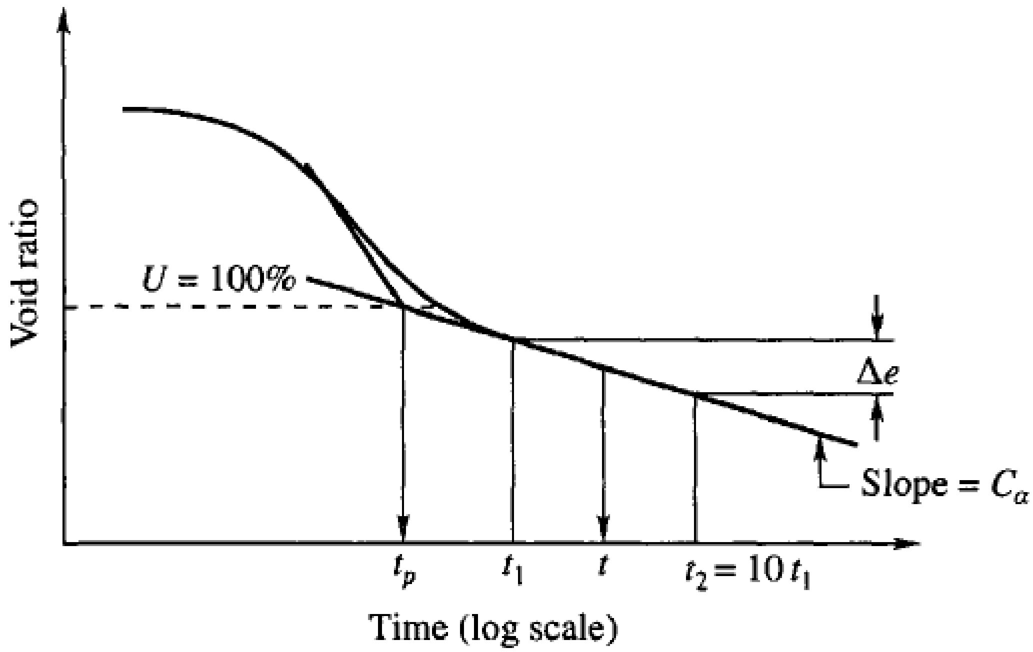


Fig. 7: e-log p time curve representing secondary compression

Example 1

A recently completed fill was 10.0m thick and its initial average void ratio was 1.0. The fill was loaded on the surface by constructing an embankment covering a large area of the fill. Some months after the embankment was constructed, measurements of the fill indicated an average void ratio of 0.8. Estimate the compression of the fill.

Solution

Change in void ratio,

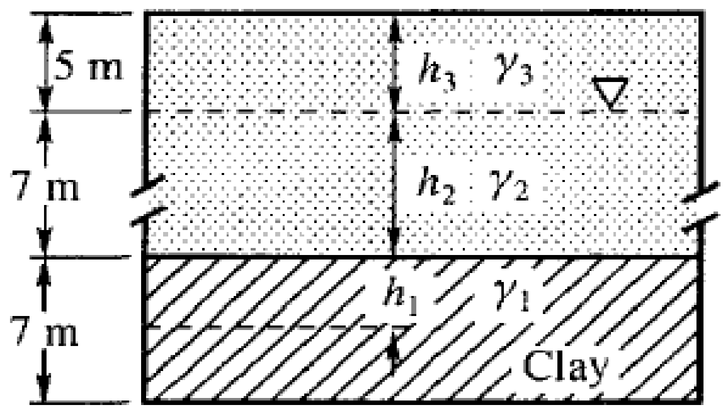
$\Delta e = e_o - e = 1.0 - 0.8 = 0.2$

The compression of the fill may be calculated as,

$$\Delta H = \frac{\Delta e}{1 + e_o} H_o = \frac{0.2}{1 + 1.0} * 10.0 = 1.0m$$

Example 2

A stratum of normally consolidated clay 7m thick is located at a depth 12m below ground level, initial void ratio is 1.118, the saturated unit weight of clay is 17.95kN/m^3 and the liquid limit is 48%. The water table is located at a depth 5m below ground surface. The soil is sand above the clay stratum. The saturated unit weight of the sand is 20.81kN/m^3 and the unit weight 18kN/m^3 above the water table. The average increase in pressure at the center of the clay stratum is 120kN/m^2 due to the weight of a building that will be constructed on the sand above the clay stratum. Estimate the expected settlement of the structure.



Solution,

Determination of overburden pressure,

$p_o = \gamma_1 h_1 + \gamma_2 h_2 + \gamma_3 h_3 = (\gamma_{sat} - \gamma_w)h_1 + (\gamma_{sat} - \gamma_w)h_2 + \gamma h_3$

$p_o = (17.95 - 9.81) * 3.5 + (20.81 - 9.81) * 7.0 + 18 * 5 = 195.5\,\mathrm{kN/m^3}$

Compression index,

$C_c = 0.009(L.L - 10) = 0.009 * (48 - 10) = 0.34$

Excess pressure,

$\Delta p = 120\,\mathrm{kN/m^3}$

Total Settlement,

$$S_t = \frac{Cc}{1 + e_o} H\log\frac{p_o + \Delta p}{p_o} = \frac{0.34}{1 + 1.118} * 7.0 * \log\frac{195.5 + 120}{195.5} = 0.233\mathrm{m} = 23.3\mathrm{cm}$$

Problem 1

A column of a building carries a load from building. The load is transferred to sub soil through a square footing founded at a depth of 2.0m below ground level. The soil below the footing is fine sand up to a depth of 5.0m and below this is a soft compressible clay of thickness 5m. Initial void ratio is 1.09, the saturated unit weight of clay is 17.9kN/m^3 and the liquid limit is 70%. The water table is found at a depth of 2.0 below the base of the footing. The saturated unit weight of the sand is 19.5kN/m^3. The average increase in pressure at the center of the clay stratum is 33kN/m^2 due to the weight of building. The sand above the water table may be assumed to remain saturated. Estimate the probable settlement of the footing. (Ans. 12.5cm)

Problem 2

Soil investigation at a site gave the following information. Fine sand exists to a depth of 10.6m and below this lies a soft clay layer 7.60m thick. The water table is at 4.60m below the ground surface. The saturated unit weight of sand is 20.21kN/m^3, and the wet unit weight above the water table is 17.6kN/m^3. The liquid limit of the normally consolidated clay is 45%, the initial void ratio is 1.11, and saturate unit weight 18.1kN/m^3. The proposed construction will transmit a net stress of 120kN/m^2 at the center of the clay layer. Find the average settlement of the clay layer. (Ans. 26.0cm)

Problem 3

A bed of sand 10m thick is underlain by a compressible of clay 3m thick under which lies sand. The water table is at a depth of 4m below the ground surface. The total unit weights of sand below and above the water table are 20.5 and 17.7kN/m^3 respectively. The clay has a natural water content of 42%, liquid limit 46% and specific gravity 2.76. Assuming the clay to be normally consolidated, estimate the probable final settlement under an average excess pressure of 100 kN/m^2.

Notes

Weight of solid particles,

$$W_S = V_S G_S \gamma_w = 1.0 * G_S \gamma_w$$

Water content,

$$w = \frac{W_w}{W_S}$$

Unit weight of water,

$$\gamma_w = \frac{W_w}{V_w}$$

Initial void ratio,

$$e_o = \frac{V_w}{V_S}$$

Total weight of sample,

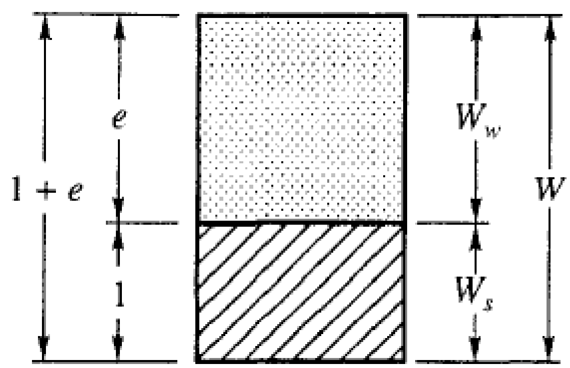
$$W = W_S + W_w$$

Saturated unit weight,

$$\gamma_{sat} = \frac{W}{1 + e_o}$$

Submerge unit weight,

$$\gamma_{sub} = \gamma_{sat} - \gamma_w$$



Rate of one-dimensional consolidation theory of Terzaghi

One dimensional consolidation theory as proposed by Terzaghi is generally applicable in all cases that arise in practice where

1. Secondary compression is not very significant,
2. The clay stratum is drained on one or both the surfaces,
3. The clay stratum is deeply buried, and
4. The clay stratum is thin compared with the size of the loaded areas.

The following assumptions are made in the development of the theory,

1. The voids of the soil are completely filled with water,
2. Both water and solid constituents are incompressible,
3. Darcy's law is strictly valid,
4. The coefficient of permeability is a constant,
5. The time lag of consolidation is due entirely to the low permeability of the soil, and
6. The clay is laterally confined.

The curves in **Fig. 8** shows the distribution of the pressure **Δp** between solid and liquid phases at various depths. At a particular depth, say **z/H = 0.5**, the stress in the soil skeleton is represented by **AC** and the stress in water by **CB**. **AB** represents the original excess hydrostatic pressure $\mathbf{u_i=\Delta p}$. The degree of consolidation $\mathbf{U_z}$ percent at this particular depth is then,

$$U_z = \frac{AC}{AB} * 100 = \frac{\Delta p - u}{\Delta p} * 100$$

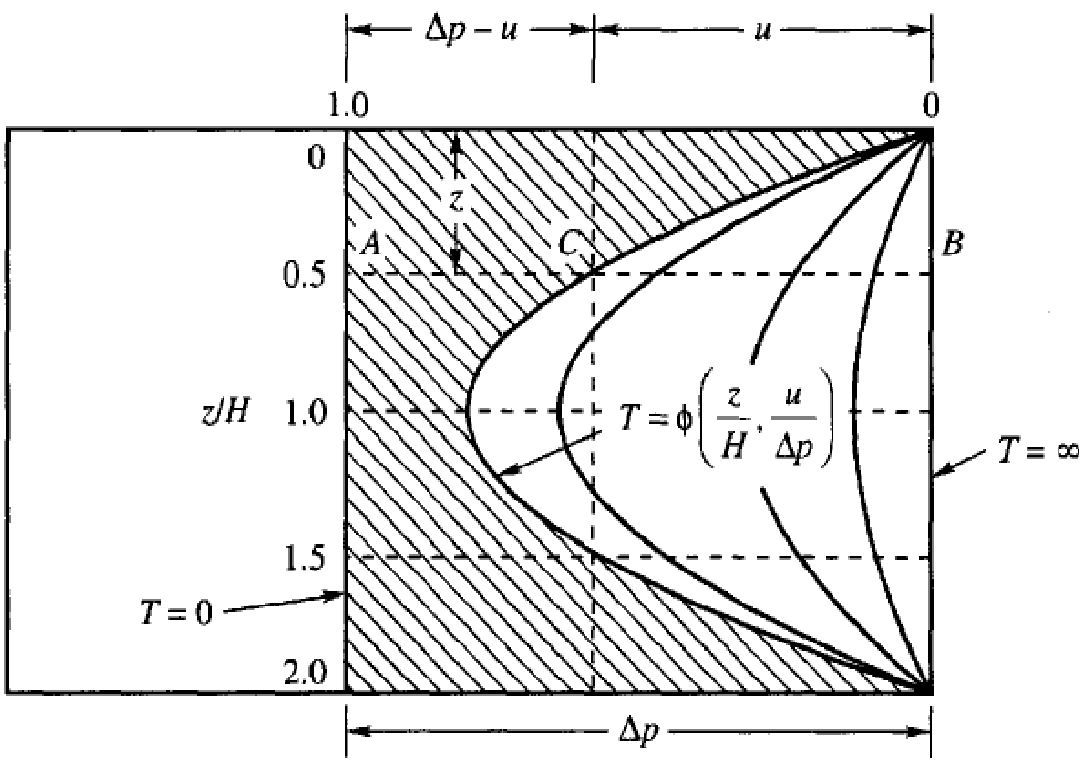


Fig. 8: Consolidation of clay layer as a function T

For values of **U%** between **0** and **60%**, the curve in **Fig. 9** can be represented the time factor almost exactly by the equation,

$$\left(\frac{U\%}{100}\right)^2 = \frac{4}{\pi}T = \frac{4}{\pi}\left(\frac{c_v t}{H^2}\right)$$

$$T = \frac{\pi}{4}\left(\frac{U\%}{100}\right)^2 \qquad (10)$$

Where, T is time factor, non-dimension.

For values of **U%** greater than **60%**, the curve in **Fig. 9** may be represented by the equation,

$$T = 1.781 - 0.933\log(100 - U\%) \qquad (11)$$

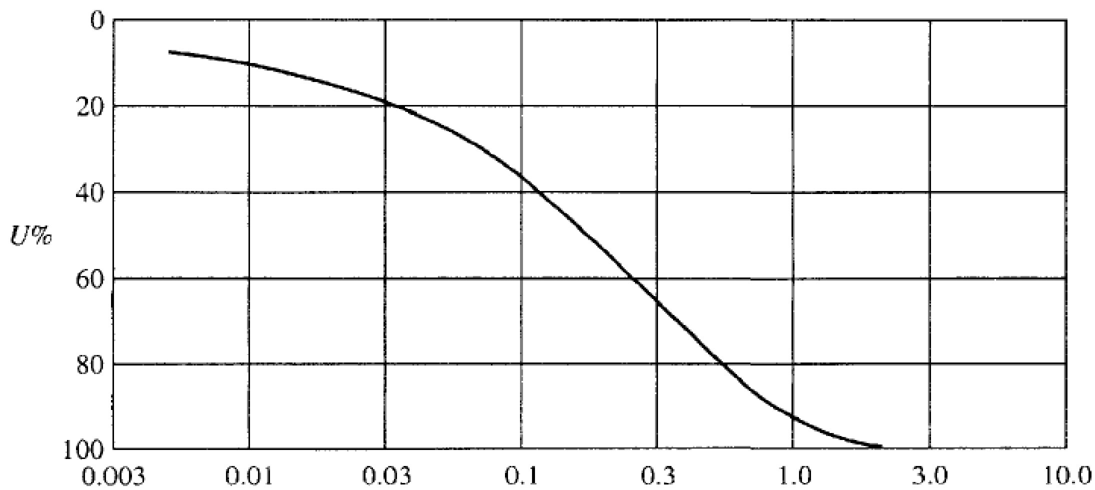


Fig. 9: Time factor T (log scale)

Table. 1: Relationship between U and T

U%	*T*	*U%*	*T*	*U%*	*T*
0	0	40	0.126	75	0.477
10	0.008	45	0.159	80	0.565
15	0.018	50	0.197	85	0.684
20	0.031	55	0.238	90	0.848
25	0.049	60	0.287	95	1.127
30	0.071	65	0.342	100	∞
35	0.096	70	0.405		

Determination of the coefficient of consolidation

The coefficient of consolidation c_v can be evaluated by means of laboratory tests by fitting the experimental curve with the theoretical.

There are two laboratory methods that are in common use for the determination of $\mathbf{c_v}$. They Are,

1. Casagrande Logarithm of Time Fitting Method, and
2. Taylor Square Root of Time Fitting Method.

The value of $\mathbf{c_v}$ is computed by taking the time **t** and time factor **T** at **50%** consolidation. The equation to be used is,

$$c_v = \frac{T_{50} {H_{dr}}^2}{t_{50}} = 0.197 \frac{{H_{dr}}^2}{t_{50}} \qquad (12)$$

For double drainage is,

$$H_{dr} = \frac{H_i - \Delta H}{2}$$

The compression between **0** and **100%** point is called "**primary compression**". At the point of **90%** consolidation, the value of **T** is **0.848**. The equation of $\mathbf{c_v}$ may now be written as,

$$c_v = \frac{T_{90} {H_{dr}}^2}{t_{90}} = 0.848 \frac{{H_{dr}}^2}{t_{90}} \qquad (12)$$

Example 3

A 2.5cm thick sample of clay was taken from the field for predicting the time of settlement for a proposed building which exerts a uniform pressure of $100 kN/m^2$ over the clay stratum. The sample was loaded to $100 kN/m^2$ and proper drainage was allowed from top and bottom. It was seen that 50% of the total settlement occurred in 3 minutes. Find the time required for 50% of the total settlement of the building, if it is to be constructed on a 6m thick layer of clay which extends from the ground surface and is underlain by sand.

Solution

T for 50% consolidation is 0.197

The lab sample is drained on both sides,

$$H_{dr} = \frac{2.5}{2} = 1.25 cm$$

The coefficient of consolidation c_v is found from,

$$c_v = 0.197 \frac{{H_{dr}}^2}{t_{50}} = 0.197 * \frac{1.25^2}{3} = 10.26 * 10^{-2}\ cm^2/min$$

The time t for 50% consolidation in the field will be found as follows,

$$T_{50} = \frac{c_v t}{H^2}$$

$$H = \frac{6.0}{2} = 3.0m = 300cm$$

$$0.197 = \frac{10.26 * 10^{-2} * t}{300^2}$$

$$t = \frac{172807}{60 * 24} = 120\ days$$

Example 4

An oedometer test is performed on a 2cm thick clay sample. After 5 minutes, 50% consolidation is reached. After how long a time would the same degree of consolidation be achieved in the field where the clay layer is 3.7m thick? Assume the sample and the clay layer have the same drainage boundary conditions (double drainage).

Solution

The time factor T is defined as,

$$T = \frac{c_v t}{H^2}$$

H is half the thickness of the clay for double drainage,

$$H_1 = \frac{2.0}{2} = 1.0\text{cm}$$

$$H_2 = \frac{3.7}{2} = 1.85\text{m} = 185\text{cm}$$

The time factor T and coefficient of consolidation are the same for both the sample and the field clay layer. The parameter that changes is the time t. Let t_l and t_2 be the times required to reach 50% consolidation both in the oedometer and field respectively. t_1 = 5 min. Therefore,

$$\frac{c_v t_1}{H^2{}_1} = \frac{c_v t_2}{H^2{}_2}$$

$$\frac{t_1}{H^2{}_1} = \frac{t_2}{H^2{}_2}$$

$$t_2 = \frac{H^2{}_2}{H^2{}_1} t_1 = \frac{185^2}{1.0^2} * 5 = \frac{171125}{60 * 24} = 119 \text{ days}$$

Problem 4

A laboratory sample of clay 2cm thick took 15min to attain 60% consolidation under a double drainage condition. What time will be required to attain the same degree of consolidation for a clay layer 3m thick under the foundation of a building for a similar loading and drainage condition? (Ans. 234days)

Problem 5

A 2.5cm thick sample was tested in a consolidometer under saturated conditions with drainage on both sides. 30% consolidation was reached under a load in 15 minutes. For the same conditions of stress but with only one-way drainage, estimate the time in days it would take for a 2m thick layer of the same soil to consolidate in the field to attain the same degree of consolidation.