

Lecture 12

Soil Compression: Consolidation

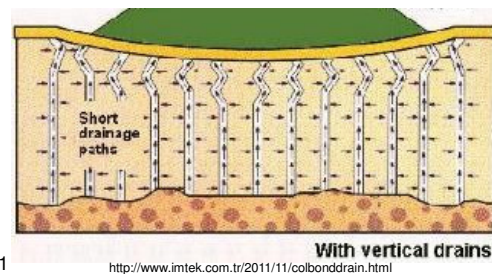
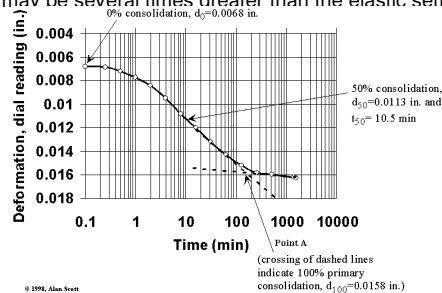
Text: Das, 2010. Chapter 11.3 - 11.12; 11.15-11.18 Settlement

Consolidation is the settlement and compression that occurs when water and air are compressed from the pore spaces of a soil. This can occur either very slowly or almost instantaneously.

When the saturated soil layer is subjected to a sudden stress increase, the pore water pressure is increased suddenly. What happens next depends on the type of soil.

In sandy soils that are highly permeable the drainage caused by the increase in the pore water pressure is completed immediately and the increased stress is directed to the soil particles as an effective stress. Consolidation and elastic settlement occur simultaneously in coarse-grained soils.

Because the hydraulic conductivity of clay soil is significantly smaller than that of sand, the excess pore water pressure generated by loading gradually dissipates over a long period. Settlement caused by consolidation in clay may be several times greater than the elastic settlement.



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Consolidation: Piston-Spring Model

The time-dependent deformation of saturated clay soil can best be understood by considering a simple model consisting of a piston with a spring at its centre (see Fig. 11.5).

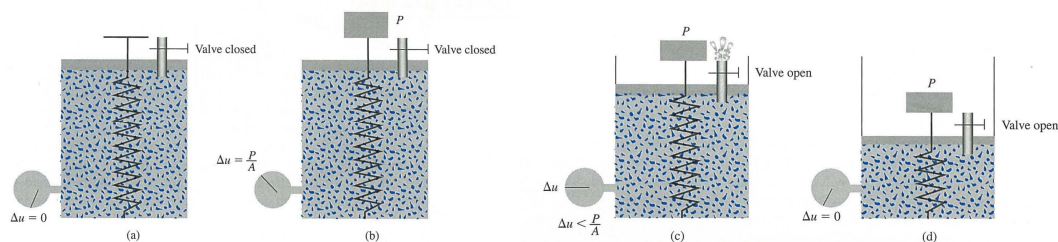


Figure 11-5: Spring-cylinder model: [(a)-(b) valve closed, (c)-(d) valve open] (Das, 2010, p. 305)

Let the inside cross-section area of the cylinder be equal to A . The cylinder is filled with water and has a frictionless watertight piston and valve.

If we place a load P on the piston and keep the valve closed, the entire load will be taken by the water in the cylinder because water is nearly incompressible. The spring will not go through any significant deformation. The excess hydrostatic pressure at this time can be given as: $\Delta u = \frac{P}{A}$

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Consolidation: Piston-Spring Model

Dividing the load P into components supported by the spring and water, we can write:

$$P = P_s + P_w$$

With the valve closed, $P_s = 0$ and $P_w = P$. When the valve is opened, water will flow out under pressure, reducing the volume of the system and placing the spring in compression. At completion of this process we will find $P_s = P$ and $P_w = 0$.

The behaviour of excess effective stress on soil particles is similar to the behaviour of the spring. Similarly, the behaviour of pore water pressure is well modelled by a valve with some degree of hydraulic resistance. From the principle of effective stress we have:

$$\sigma' = \sigma - u$$

$$\Delta\sigma' = \Delta\sigma - \Delta u$$

At $t = 0$ any sudden change in total stress is carried by an equal change in pore water pressure, with no increase in effective stress. As water drains out, the stress is transferred onto soil particles as effective stress.

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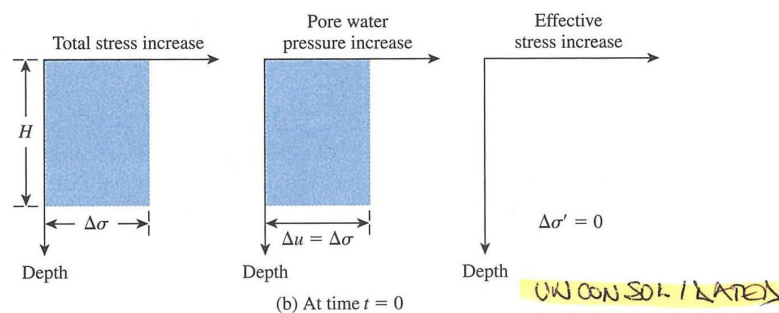
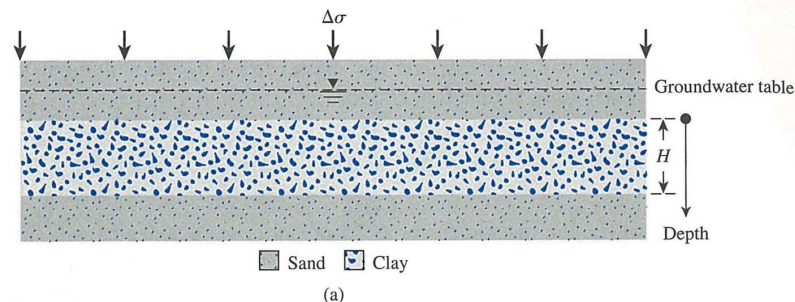


Figure. 11-6 (a)-(b): Variation of total stress, pore pressure, and effective stress in a clay layer drained at top and bottom as the result of an added stress, $\Delta\sigma$ (Das, 2010, p. 307)

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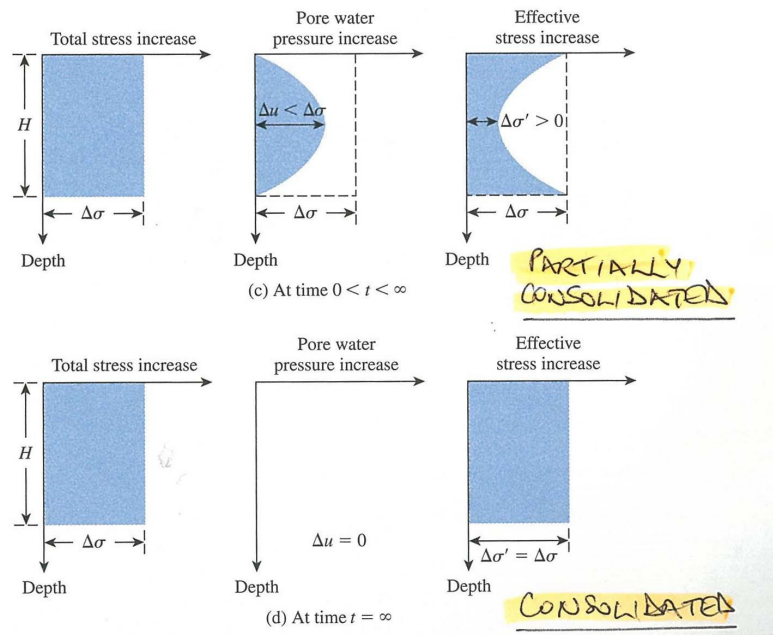
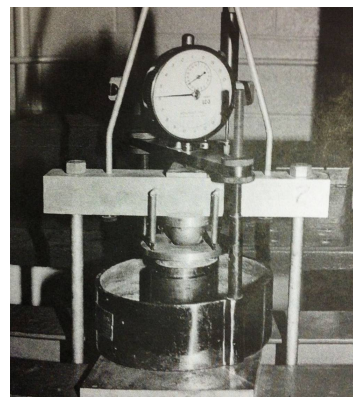
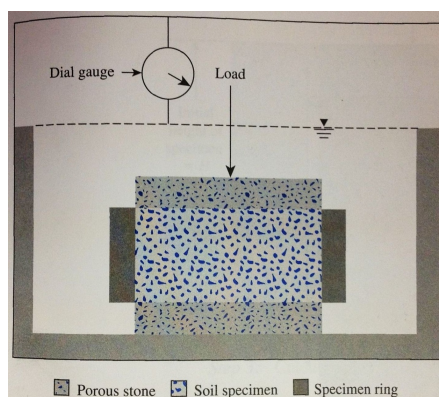


Figure. 11-6 (c)-(d): Variation of total stress, pore pressure, and effective stress in a clay layer drained at top and bottom as the result of an added stress, $\Delta\sigma$ (Das, 2010, p. 307)

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One-Dimensional Consolidation Test

The one-dimensional consolidation testing procedure was first suggested by Terzaghi. This test is performed in consolidometer, sometimes referred to as an oedometer:



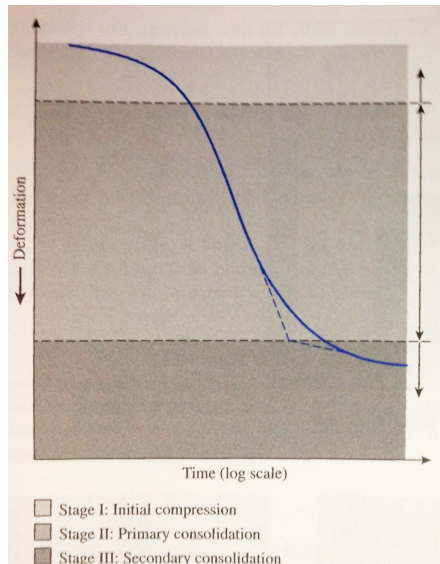
The soil specimen is placed inside a metal ring with two porous stone diaphragms, one above and one below. A load is applied and compression is measured by a micrometer dial gauge.

The specimen is kept under water during the test. The load is maintained for 24 hours and then is usually doubled for another 24 hours.

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One-Dimensional Consolidation Test

The general shape of the plot of deformation of the specimen for a given load increment is shown below.



From the plot we can observe three distinct stages:

- Initial elastic compression of water and soil particles (water is *not* in fact perfectly incompressible!)
- Primary consolidation during which excess pore water pressure is gradually transferred to effective stress because of the expulsion of pore water.
- Secondary consolidation which occurs after complete dissipation of excess pore water pressure. Secondary consolidation is a plastic deformation of the soil fabric under constant effective stress. This completes a process which begins during stage II.

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Void ratio and compression

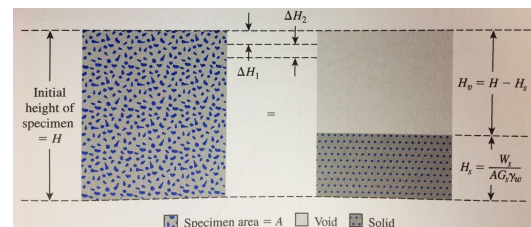
Elastic settlement and secondary consolidation both reflect a change in volume and density of the liquid and solid components of soil without necessarily altering the void ratio. Primary consolidation is the opposite, ie the volume change is entirely due to a change in void volume with no net change in material densities.

The change in void ratio is calculated as follows:

Step 1. Calculate the height of solids in the specimen:

$$H_s = \frac{W_s}{AG_s\gamma_w} = \frac{M_s}{AG_s\rho_w}$$

where W_s = dry weight of the specimen
 M_s = dry mass of the specimen
 A = area of the specimen
 G_s = specific gravity of soil solids
 γ_w = unit weight of water
 ρ_w = density of water



Step 2. Calculate the initial height of voids in the specimen:

$$H_v = H - H_s \quad \text{where } H \text{ is the initial height of the specimen (around 25 mm for the standard apparatus)}$$

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Void ratio and compression

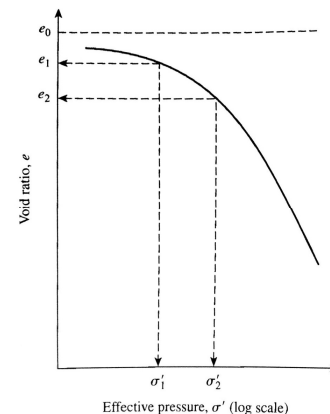
Step 3. Calculate the initial void ratio, e_0 :

$$e_0 = \frac{V_v}{V_s} = \frac{H_v}{H_s} \frac{A}{A} = \frac{H_v}{H_s}$$

Step 4. For each incremental loading σ_n and deformation ΔH_n calculate the change and new value of the void ratio:

$$\Delta e_n = \frac{\Delta H_{vn}}{H_s} \quad e_n = e_{n-1} - \Delta e_n$$

Step 5. Noting that at the end of each increment the additional stress has been consolidated as effective stress, plot effective stress against void ratio on semi-log paper:



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Normally Consolidated and Overconsolidated Clays

A soil in the field has been subjected to a maximum effective stress in its geologic history. This maximum *preconsolidation* stress may be the same or more than the effective stress at the time of sampling.

The change in effective stress may be caused by natural geological processes or human processes including the disturbance of sampling.

When such a specimen is subjected to a consolidation test a relatively small amount of compression will occur when the stress is less than the maximum prior stress.

However when the applied stress exceeds the prior stress, the change in void ratio is much larger and the $e - \log \sigma'$ relationship is practically linear with a steeper slope.

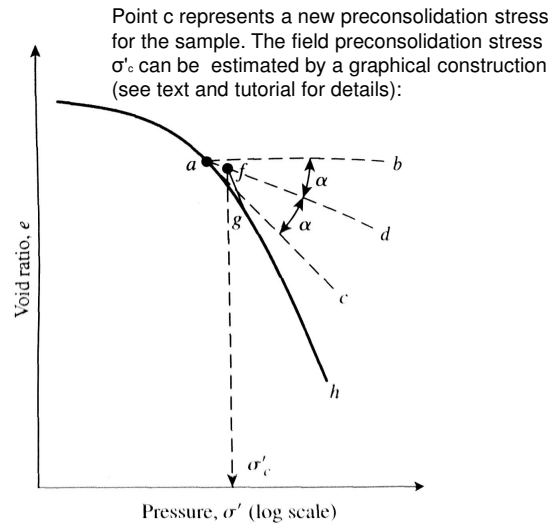
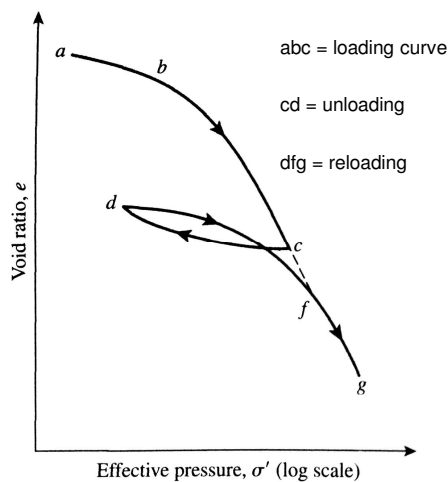
This leads to a definition of two distinct clay types in the field based on stress history:

- *Normally consolidated clay*: where present effective stress is the maximum stress the soil has experienced in its geologic history (the preconsolidation stress)
- *Overconsolidated clay*: where the present effective stress is less than the preconsolidation stress.

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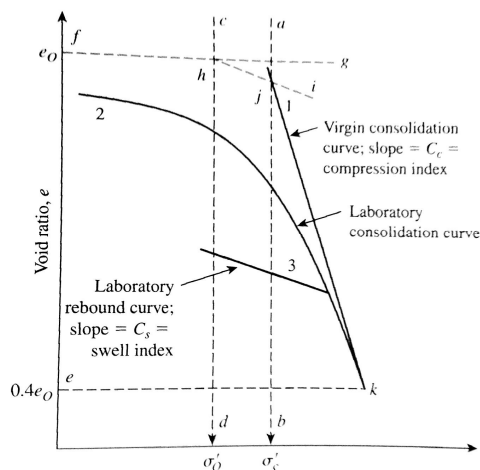
Normally Consolidated and Overconsolidated Clays

Overconsolidation can be simulated in the laboratory by loading a specimen to exceed the preconsolidation stress, then relaxing and reloading the sample:



Normally Consolidated and Overconsolidated Clays

The consolidation characteristics of a clay soil can be summarised by two independent parameters, the *Compression Index* C_c and the *Swell Index* C_s (again, the text provides details on these constructions):



Glacial clays, A) Normally consolidated and B) overconsolidated
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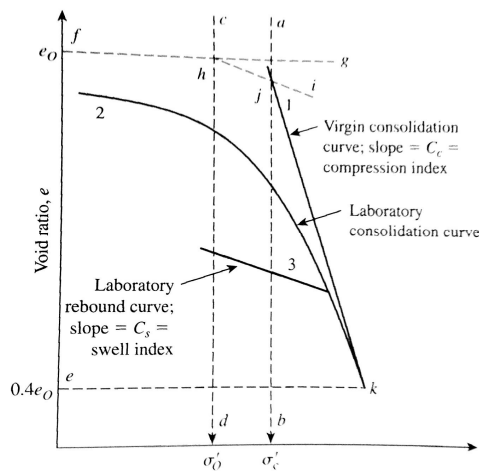
Normally Consolidated and Overconsolidated Clays

More commonly the state of consolidation is quantified by an Overconsolidation Ratio (OCR):

$$OCR = \frac{\sigma'_c}{\sigma'_o} \geq 1$$

OCR = 1 for normally consolidated soils

OCR > 2 for overconsolidated soils.



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Primary Consolidation Settlement in Normal/Overconsolidated Clays

We are now able to calculate the second term of the settlement equation, S_c or primary consolidation settlement. This requires that we identify the state of consolidation as well as the compression and swell indices C_c and C_s :

Normally consolidated clays:

$$S_c = \frac{C_c H}{1 + e_0} \log \left(\frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \right) \quad \sigma'_0 \text{ is the initial effective stress, } \sigma'_0 + \Delta \sigma' \text{ is the final stress.}$$

Overconsolidated clays:

$$S_c = \frac{C_s H}{1 + e_0} \log \left(\frac{\sigma'_0 + \Delta \sigma'}{\sigma'_0} \right) \quad \text{Note the use of the swell index } C_s:$$

When an overconsolidated soil is recompressed past the preconsolidation stress, the calculation has two stages:

$$S_c = \frac{C_s H}{1 + e_0} \log \left(\frac{\sigma'_c}{\sigma'_0} \right) + \frac{C_c H}{1 + e_0} \log \left(\frac{\sigma'_0 + \Delta \sigma'}{\sigma'_c} \right) \quad \text{See text for derivations}$$

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Secondary Consolidation Settlement

At the end of primary consolidation some additional compression occurs because of plastic deformation of soil fabrics. This also has a logarithmic profile against void ratio, with linear slope known as the *secondary compression index* C_α . Using a plot of void ratio against time, we have:

$$C_\alpha = \frac{\Delta e}{\log(t_2/t_1)}$$

Secondary consolidation settlement S_s is then:

$$S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right)$$

$$C'_\alpha = \frac{C_\alpha}{1 + e_p}$$

e_p is the void ratio at the end of primary consolidation. H is the soil layer depth.

$$S_s = C'_\alpha H \log\left(\frac{t_2}{t_1}\right) = \frac{\Delta e H}{1 + e_p}$$

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Compression and Swell Indices

Approximate values of C_c , C_s and C_α are given in the text, along with some empirical models of these parameters:

	Liquid Limit	Plastic Limit	Compression Index C_c	Swell Index C_s
Boston blue clay	41	20	0.35	0.07
Chicago clay	60	20	0.4	0.07
Ft Gordon clay	51	26	0.12	-
New Orleans clay	80	25	0.3	0.05
Montana clay	60	28	0.21	0.05
C'_α				
Overconsolidated clays	0.001 or less			
Normally consolidated clays	0.005 to 0.03			
Organic soils	0.04 or more			

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Rate of Primary Consolidation

As well as the magnitude of settlement, the rate at which it occurs is important for construction planning - ie how long until decompressed foundation sediments are stable for construction work to commence? At what rate will sediments settle during construction?

Terzaghi's (1925) theory makes the following assumptions:

1. The clay-water system is homogenous ie a uniform mix of solid and liquid
2. Saturation is complete
3. Compressibility of soil and water are negligible, but particles may rearrange
4. The flow of water is away from the direction of compression
5. Darcy's law is valid.

The process of consolidation, ie ejection of pore water under pressure, is assumed to resemble a diffusion process:

$$\frac{\partial u}{\partial t} = c_v \frac{\partial^2 u}{\partial z^2} \quad c_v \text{ a coefficient of consolidation.}$$

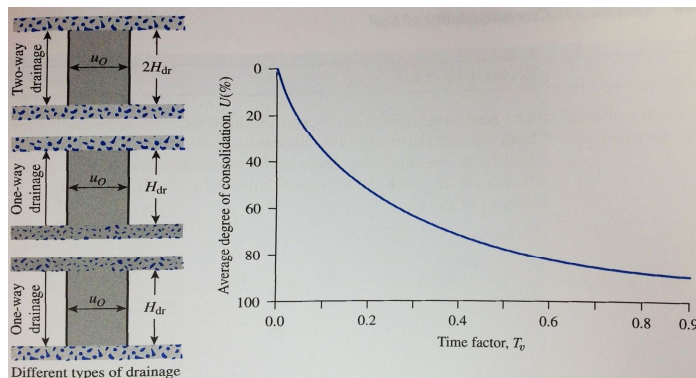
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Rate of Primary Consolidation

The coefficient of consolidation c_v is applied as a non-dimensional time factor, T_v for a given proportional consolidation, U :

$$T_v = \frac{c_v t}{H_{dr}^2} = \frac{\pi U^2 / 4}{(1 - U^{5.6})^{0.357}}$$

For a given consolidation, eg 95% ($U=0.95$), T_v is obtained and converted to time. H_{dr} is the minimum drainage distance at constant pore pressure excess u_0 .



Example: Target: $U=0.95$,
 $H_{dr}=2.5\text{m}$, $c_v=1 \times 10^{-6}$

Then $T_v = 1.2$

Given $H_{dr} = 2.5 \text{ m}$ and $c_v = 0.000001 \text{ m}^2/\text{s}$,

$$t = 1.2(2.5)^2 / 0.000001 = 7,500,000 \text{ s} = 87 \text{ days}$$

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Rate of Primary Consolidation

The coefficient of consolidation c_v can be determined either in the field or the laboratory:

Site	$c_{v(\text{lab})}$ (m ² /sec)	$c_{v(\text{in situ})}$ (m ² /sec)	$c_{v(\text{in situ})}/c_{v(\text{lab})}$
Ska-Edeby IV	5.0×10^{-9}	1.0×10^{-7}	20
Oxford (1)			4–57
Donnington			4–7
Oxford (2)			3–36
Avonmouth			6–47
Tickton			7–47
Over causeway			3–12
Melbourne			200
Penang	1.6×10^{-8}	1.1×10^{-6}	70
Cubzac B	2.0×10^{-8}	2.0×10^{-7}	10
Cubzac C	1.4×10^{-8}	4.3×10^{-7}	31
A-64	7.5×10^{-8}	2.0×10^{-6}	27
Saint-Alban	1.0×10^{-8}	8.0×10^{-8}	8
R-7	6.0×10^{-9}	2.8×10^{-7}	47
Matagami	8.0×10^{-9}	8.5×10^{-8}	10
Berthierville		4.0×10^{-8}	3–10

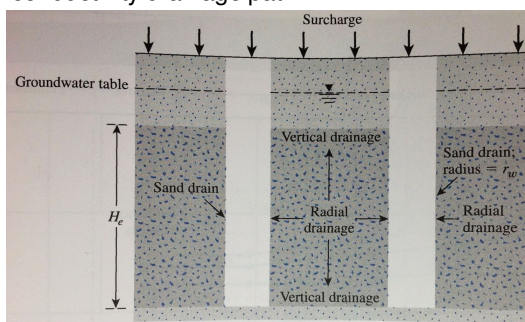
It is apparent that the consolidation coefficient is highly uncertain. Times to consolidation may be highly variable.

Methods of Accelerating Consolidation

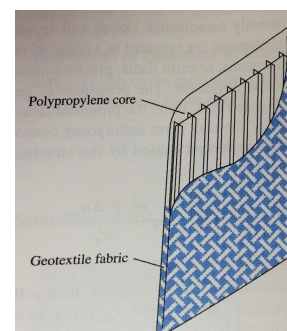
Sand drains and prefabricated vertical drains are used in the field to accelerate consolidation.

Sand drains are produced by drilling holes through a clay layer at regular intervals, then backfilling with sand. A surcharge applied at the ground surface increases the pore pressure; consolidation commences as water drains horizontally and vertically via the sand drains.

Prefabricated vertical drains (wick or strip drains) are manufactured from synthetic polymers and geotextile fabrics and embedded vertically in soil to provide a high-conductivity drainage path.

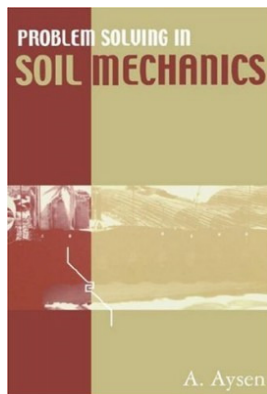
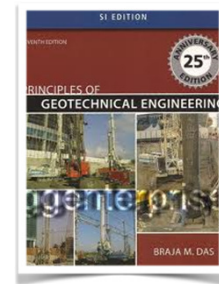


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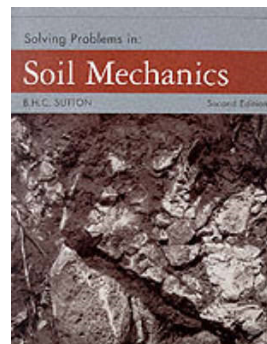
Readings

Chapter 11.3 - 11.12; 11.15-11.18 Das

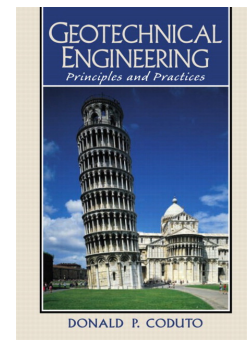


<http://www.ebay.com/ctg/Problem-Solving-Soil-Mechanics-A-Aysen-2004-Paperback-/52853454>

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