Some Myths of Consolidation Settlement

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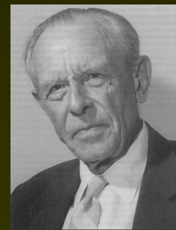
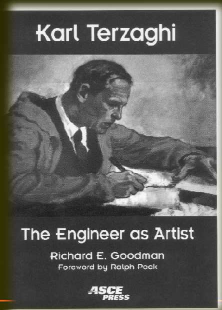
" If Civil Engineering was a game, Terzaghi had a right to lay down the rules, as he had invented and established much of the ground work. He is known as

THE FATHER OF SOIL MECHANICS "



"He developed a mechanics for soils because it was needed, just as Isaac Newton had developed the calculus to empower his studies in physics. Terzaghi was a pioneer in showing how to make dams, buildings and other structures safe even though founded on soils. This required that there be **A SCIENCE OF SOIL MECHANICS**, as well as **AN ART OF SOILS ENGINEERING**."

Richard Goodman



Photos are extracted from Prof. Goodman's book on Terzaghi: "The Engineer as Artist" ASCE 1999 publication

Myth

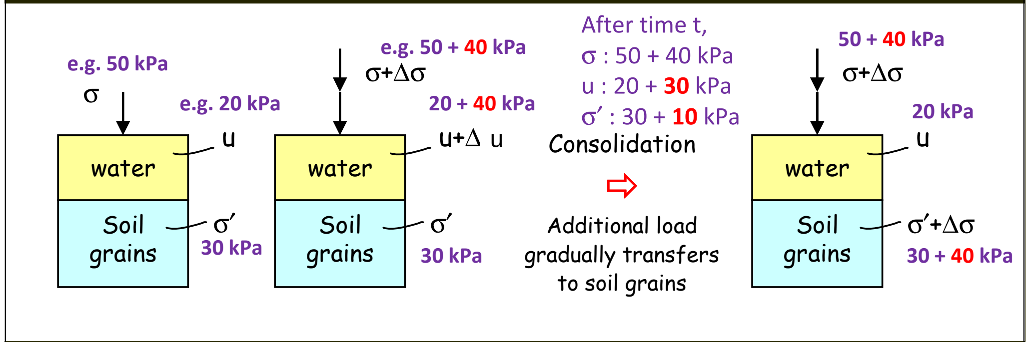
A belief or set of beliefs often unproven or false (that have accrued around a person, institution or phenomenon).

A fiction or half-truth, especially one that forms part of an ideology

1

MYTH 1 : ONE DIMENSIONAL CONSOLIDATION

Consolidation : *is the gradual reduction in volume of a (fully saturated) soil due to drainage of some of the pore water. The process continues until all excess pore water pressure set up by an increase in total stress has completely dissipated.*



When a load is applied to a soil and the soil is allowed to deform, there are normally ***3 types of compressions*** *that contribute to the overall deformation (settlements):*

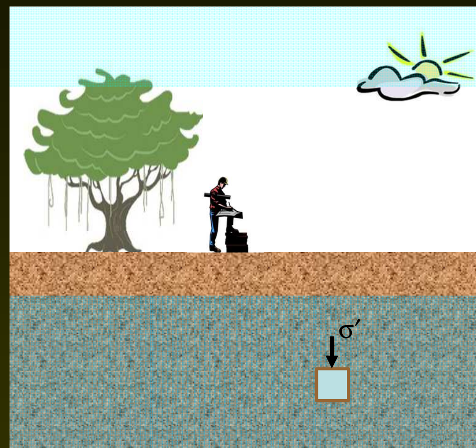
(a) Elastic deformation : *it occurs immediately on the application of load and is recoverable on removal of the load. The deformation is commonly referred to as the* ***immediate settlement****.*

(b) Primary consolidation : *This is the deformation most desingers concern about. It results from the decrease in volume due to the dissipation of excess pore water*

(c) Secondary compression : *the continued deformation of soil even after all the excess pore water has been dissipated. The deformation is sometimes called creep settlement.*

PRECONSOLIDATION PRESSURE :

The maximum effective vertical stress that has acted on the soil in the past.



Two types of soils can be identified:

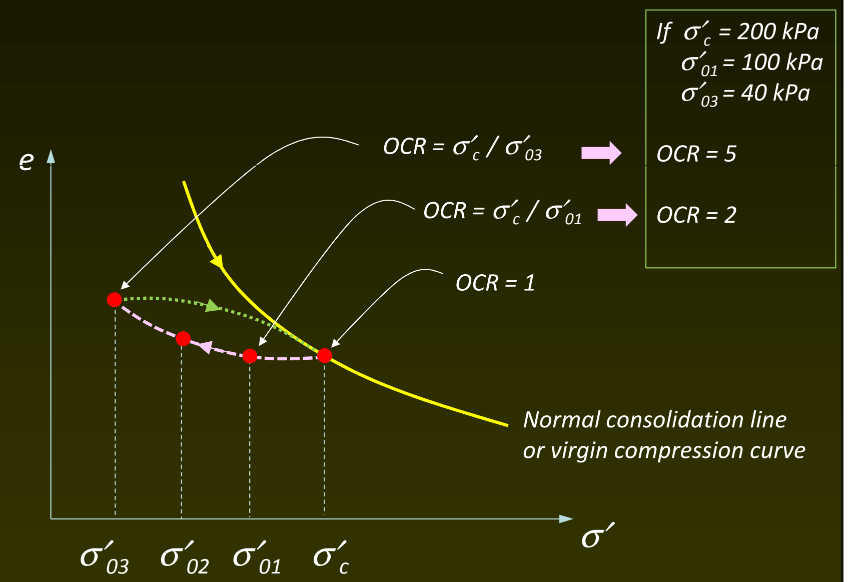
(a) Normally consolidated soil : *soil with its preconsolidation pressure equals the existing effective vertical overburden pressure.*

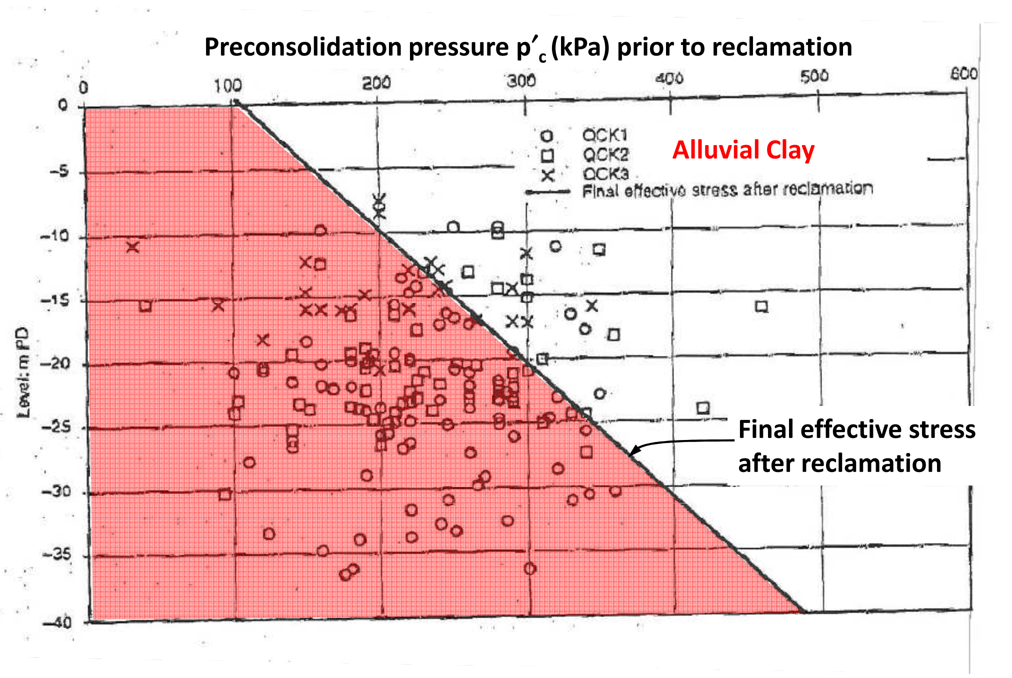
(b) Overly consolidated soil : *soil with its preconsolidation pressure greater than the existing overburden pressure.*

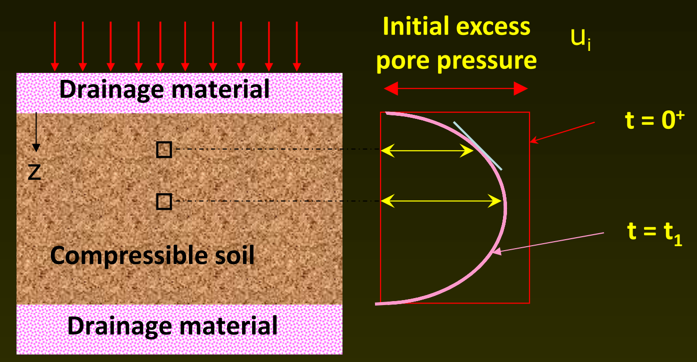
The ratio of the preconsolidation pressure (σ'_c) to the existing vertical effective overburden pressure (σ'_0) is known as over-consolidation ratio (OCR), i.e.

$$OCR = \sigma'_c / \sigma'_0$$

Over-consolidation







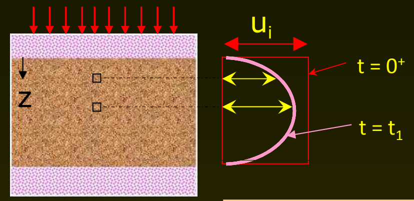
Governing equation:

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

u - excess pore pressure, z - depth, t - time

c_v - coefficient of consolidation

Fourier Series solution:



$$u = \frac{4}{\pi}\sum_{m=0}^{\infty}\frac{u_i}{2m+1}\left[\sin\frac{(2m+1)\pi z}{2d}\right]e^{-\frac{(2m+1)^2\pi^2 T_v}{4}}$$

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

where

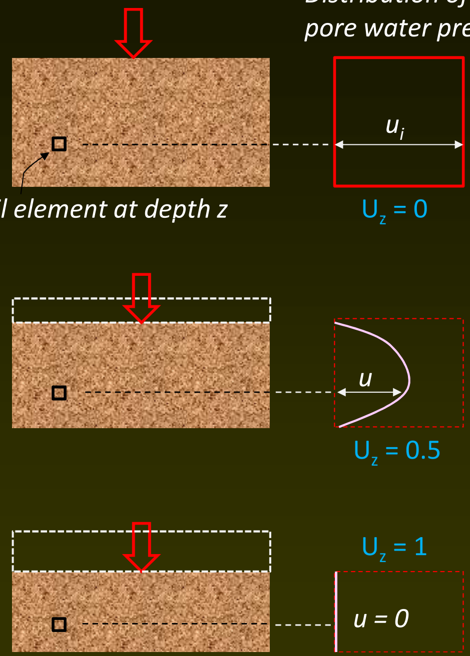
$$T_v = \frac{c_v t}{d^2}$$

*T_v is known as the **Time Factor***

Degree of consolidation is defined as:

$$U_z = \frac{u_i - u}{u_i}$$

Distribution of excess pore water pressure



Soil element at depth z

at initial stage (t = 0)

Note that U at different depths is not = 0.5

at a stage where $t = t_1$

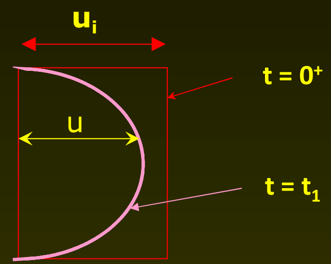
at final stage $t = \infty$

Degree of consolidation is defined as:

$$U_z = \frac{u_i - u}{u_i}$$

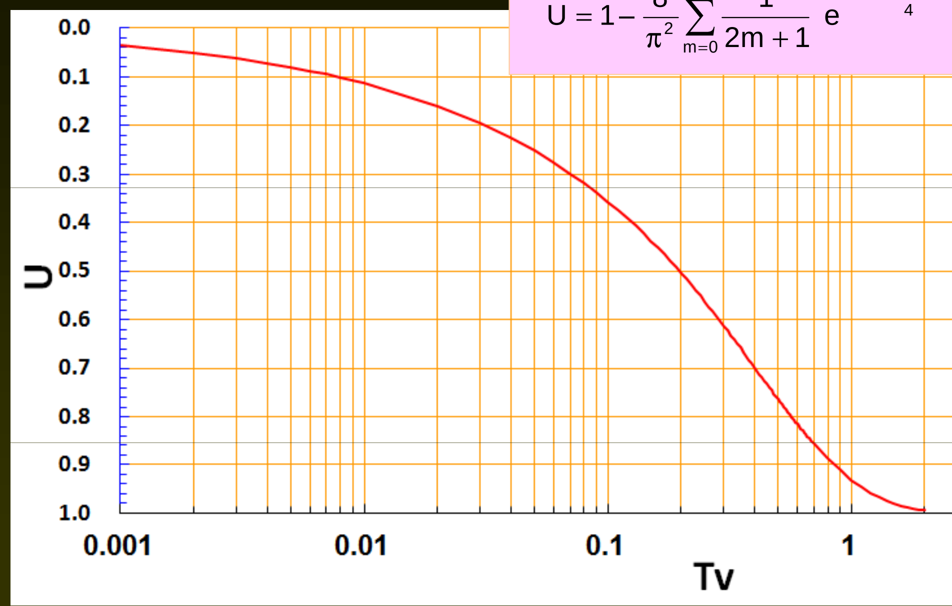
Av. Degree of consolidation:

$$U = 1 - \frac{\frac{1}{2d}\int_0^{2d} u\,dz}{u_i}$$



Hence, from Fourier Series solution:

$$U = 1 - \frac{8}{\pi^2}\sum_{m=0}^{\infty}\frac{1}{2m+1}e^{-\frac{(2m+1)^2\pi^2 T_v}{4}}$$



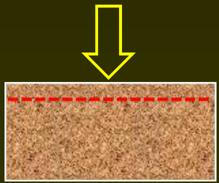
$$U = 1 - \frac{8}{\pi^2}\sum_{m=0}^{\infty}\frac{1}{2m+1}e^{-\frac{(2m+1)^2\pi^2 T_v}{4}}$$

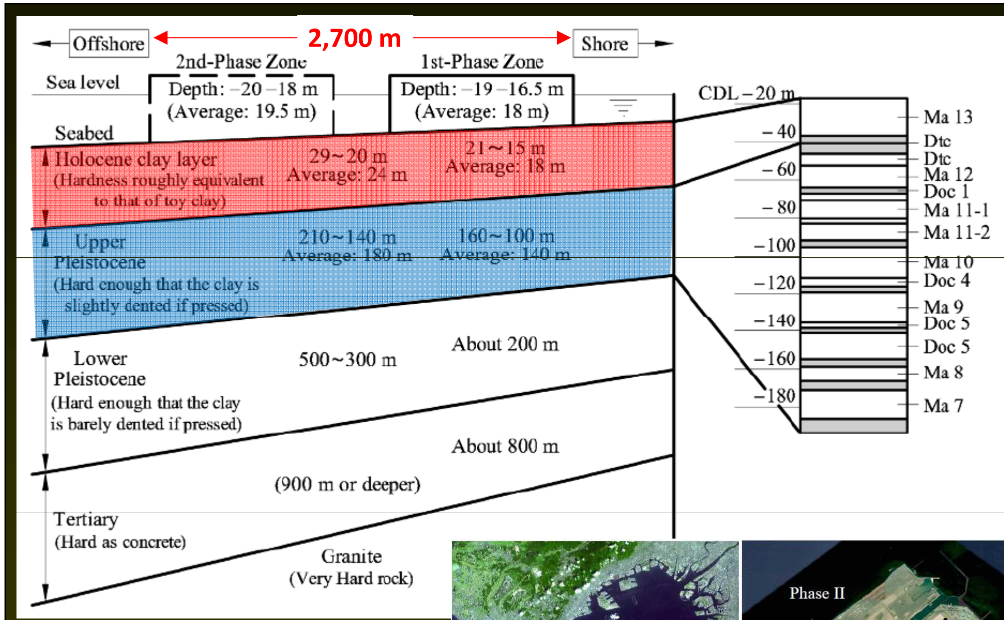
An important question to ask:

In practice , under what condition(s) that 1D consolidation theory is applicable?

I.E.

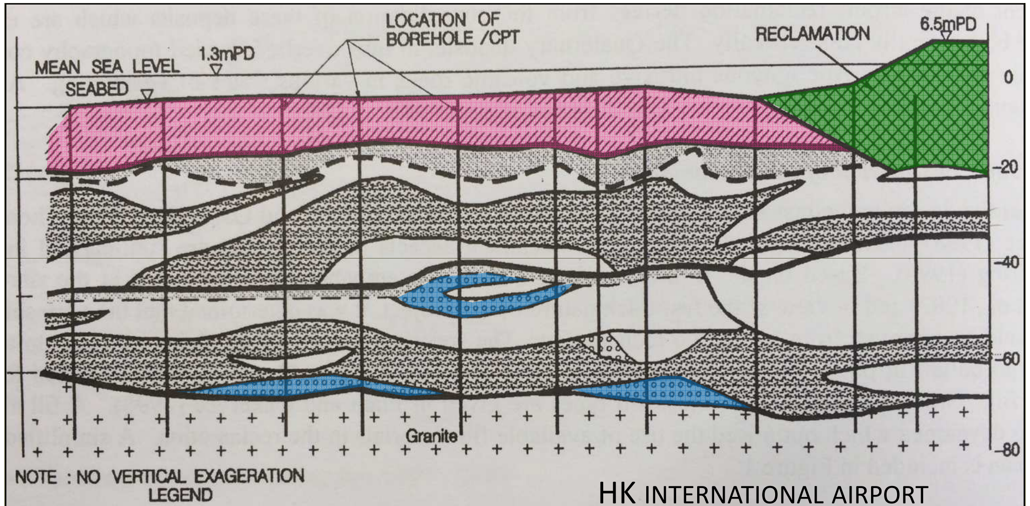
How do we know our site condition is in favour of 1D consolidation and not other types of consolidation?

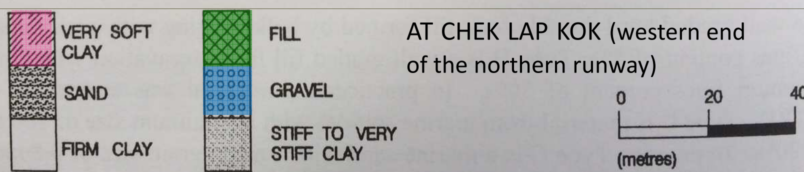




Kansai international airport

Ref: Akai and Tanaka 1999





HK international airport

AT CHEK LAP KOK (western end of the northern runway)

Source: Tosen, Pickles and Jaros (1998)

Assumptions of Terzaghi's 1D consolidation theory:

(a) soil is fully saturated

(b) soil grains and water are incompressible

(c) Darcy's law is valid

(d) soil compression and water flow in 1D only

(e) coefficient of consolidation is constant

(f) compressible soil layer is homogeneous, horizontal and of uniform thickness

(g) initial excess pore pressure due to the application of load is uniform throughout the depth of the soil layer

(h) a change in effective stress in the soil causes a corresponding change in voids ratio and their relationship is linear during any stress increment and is independent of time.

In practice, most of the assumptions in Terzaghi's 1D consolidation theory are not correct !

Erdbaumechanik
auf bodenphysikalischer Grundlage

Von

Dr. Ing. Karl Terzaghi
Zivilingenieur und Professor am amerikanischen Robert College in Konstantinopel

Mit 65 Textabbildungen

Leipzig und Wien
Franz Deuticke
1925

ERDBAUMECHANIK?

VON

O. O. PROF. DR. PAUL FILLUNGER
WIEN

IM SELBSTVERLAGE DES VERFASSERS
DRUCK UND VERTRIEB DURCH DIE
BUCHDRUCKEREI FRIEDRICH JASPER, WIEN, III., THONGASSE 12

Ref: Boer et al, Geotechnique 46, 1996

Newspaper story concerning Fillunger's suicide in 1937 (the Neue Freie Presse)

Neues Wiener
Tagblatt
Demokratisches Organ

Dienstag, den 9. März 1937.

Prof. Paul Fillunger

Der Selbstmord Professor Fillungers.

Ref: Boer et al, Geotechnique 46, 1996

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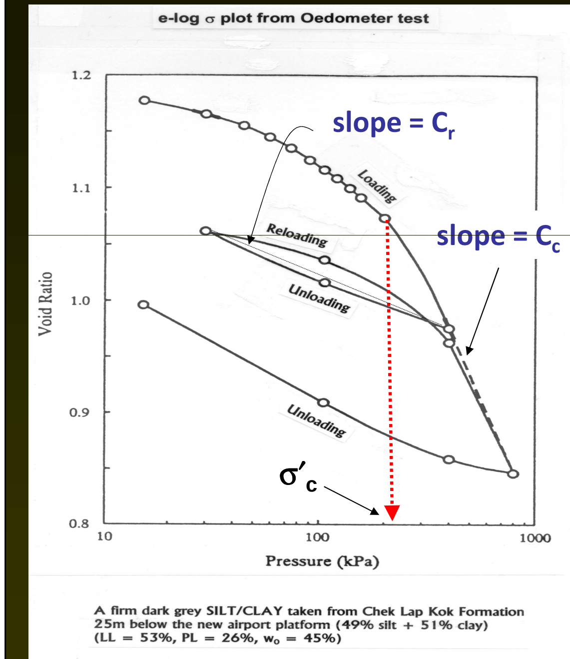
Myth 2 : Nature of consolidation parameters
(and a note on observational method)

The following compressibility-related parameters may be obtained from 1D consolidation test (i.e. oedometer test) :

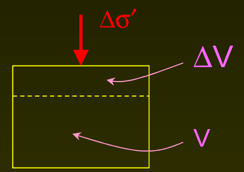
Magnitude of consolidation		*Rate of consolidation*	
m_v	coeff. of volume compressibility (for each loading/unloading stage)	c_v	coeff. of consolidation (for each loading / unloading stage)
c_c	compression index (from loading portion of e-log σ′ plot)		
c_r	re-compression index (from unloading and reloading portions of e-log σ′ plot)		
c_{sec}	coeff. of secondary compression		
e – log σ′ plot			



e – log σ′ curve showing loading and unloading data for the Chek Lap Kok airport project

SILT/CLAY from Chek Lap Kok formation 25m below the airport platform

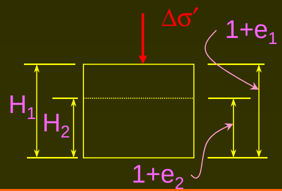
Coefficient of volume compressibility (m_v) *is defined as the volume change per unit volume per unit increase in effective stress*



$$m_v = \frac{\Delta V}{V}\frac{1}{\Delta\sigma'}$$

For 1D consolidation, area remains unchanged.

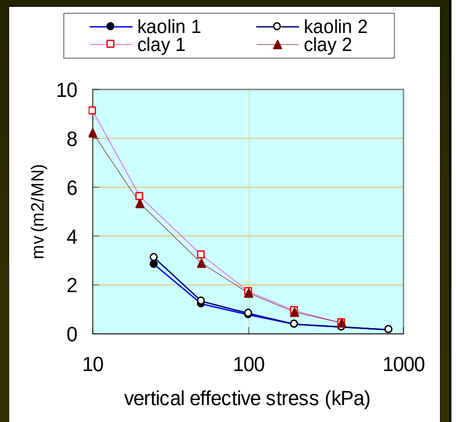
$$m_v = \frac{\Delta H}{H}\frac{1}{\Delta\sigma'} = -\frac{H_2 - H_1}{H_1}\frac{1}{\Delta\sigma'} = -\frac{e_2 - e_1}{1+e_1}\frac{1}{\sigma'_2 - \sigma'_1}$$



The normal range of m_v for alluvial deposit, clay and fine-grained volcanics found in HK is approximately ***0.05 - 5 m^2/MN*** *depending on stress level.*

Two important notes to the designers:

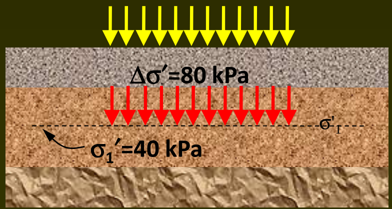
(1) m_v of a soil is ***not constant****, it depends on the stress and stress range over which it is calculated. In general, m_v decreases as stress (or depth) increases.*



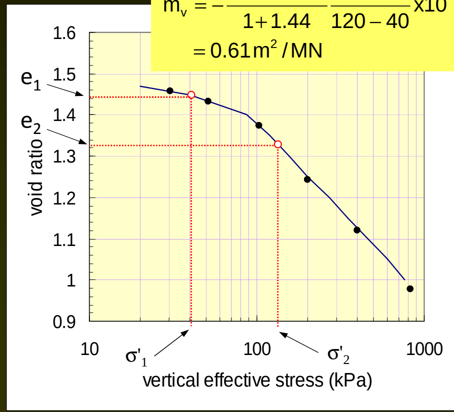
Note also m_v in fact is also a measure of the stiffness of a soil

(2) *Although Geospec 3 specifies that m_v should be calculated for each loading increment and each lab report also follows as such.*

In practice, however, the designer should calculate the most appropriate value by himself /herself according to the actual stress range encountered in the project.



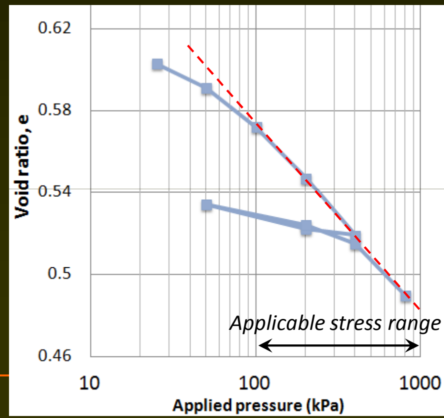
$$m_v = -\frac{1.32-1.44}{1+1.44}\,\frac{1}{120-40}\,x10^{-3}$$
$$= 0.61\,m^2/MN$$



Apart from m_v, the ***compression index (c_c)*** *can also be used to estimate the magnitude of consolidation settlement.*

c_c is the slope of the straight line portion of the virgin compression curve in the e - log σ′ plot,

$$c_c = \frac{\Delta e}{\log \frac{\sigma_1'}{\sigma_0'}}$$



c_c has the limitation that it is only applicable for the stress range that falls ***within the straight line portion*** *of the e- log σ′ plot.*

Typical c_c values for some HK soils are given below:

Soil type	Initial void ratio, e_0			
	0.4 – 0.8	0.8 – 1.2	1.2 – 2.0	>2.0
Marine clay	0.1 – 0.2	0.2 - 0.4	0.4 – 0.8	0.8 - > 1.0
Alluvial deposit	0.05 – 0.2	0.2 – 0.4	--	--
CDV	0.05 – 0.13	0.13 – 0.25	--	--

The ***coefficient of consolidation*** *c_v is defined as:*

$$c_v = \frac{k}{m_v \gamma_w}$$

(Note that is is assumed constant in Terzaghi's theory)

c_v also relates the actual time (t) and the time factor (T_v):

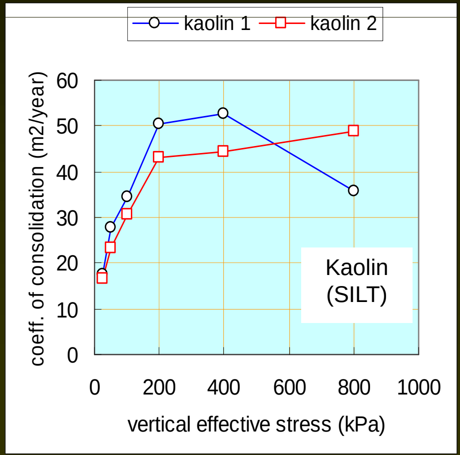
$$T_v = \frac{c_v t}{d^2}$$

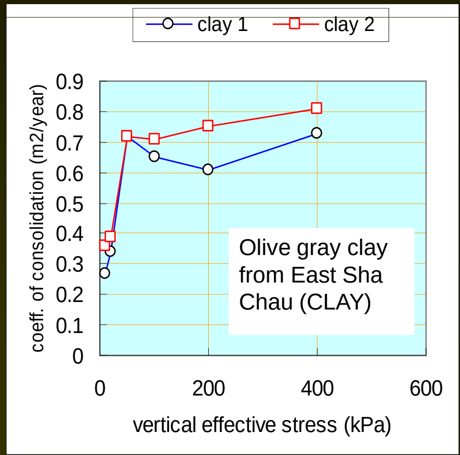
c_v may be calculated from ***root time plot*** *or* ***log time plot*** *from the oedometer test results for* ***each loading***

The normal range of c_v for HK soils varies from about ***0.1 to 50 m²/year***. *A typical laboratory c_v value for marine clay in HK is around 1 m^2/year*

Is c_v a constant as assumed by Terzaghi ?

→ *We found that c_v in fact varies with loading !!*



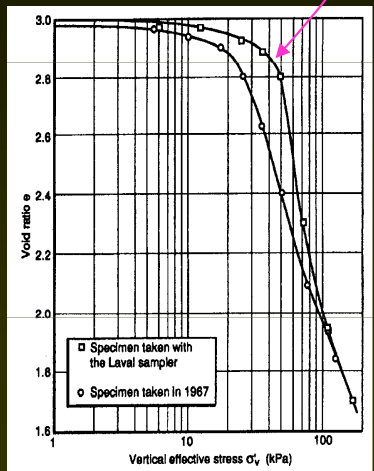


The compressibility parameters should not be treated in the same way as other soil parameters, such as shear strength parameters, c′ and ϕ'

Why?

Better specimen quality

Many site observations reveal that the actual rate and magnitude of consolidation settlements are greater than those predicted by laboratory tests.

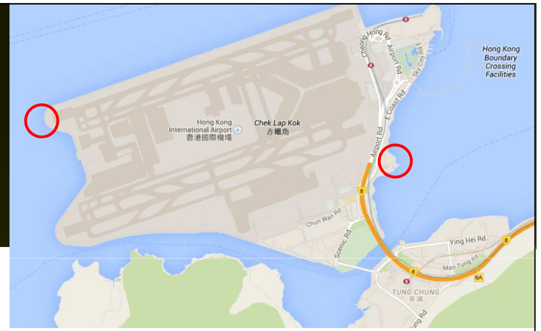


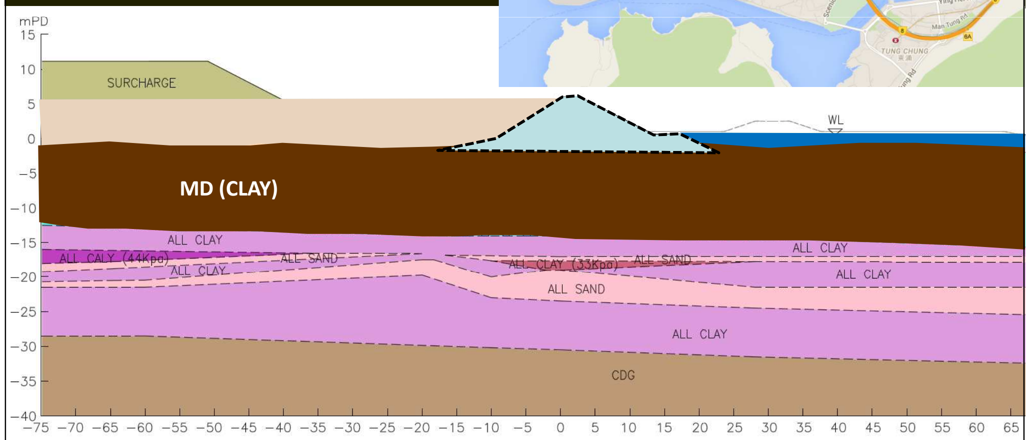
Some of the possible reasons :

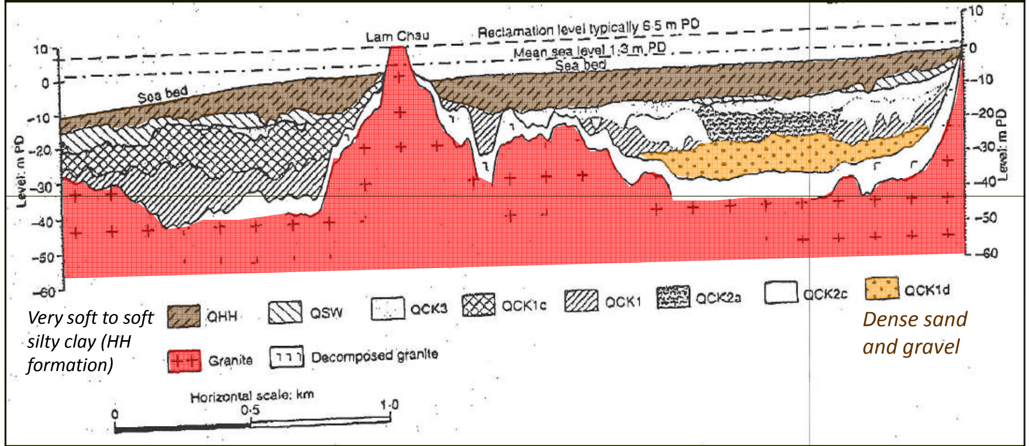
- *sample disturbance*
- *variations in the soil fabric*
- *anisotropic drainage or 3-D consolidation*

(note that in general $c_h > c_v$)

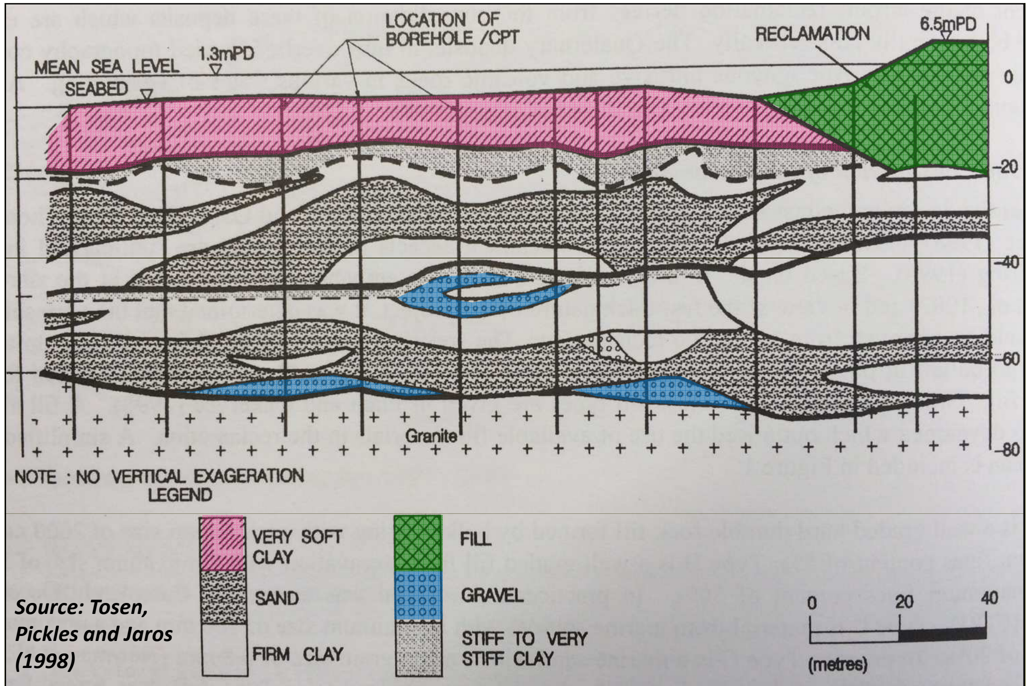
Example at Chap Lap Kok airport showing the complicated alluvial deposits formed between the CDG/granitic bedrock and the marine deposit (Hang Hau formation)



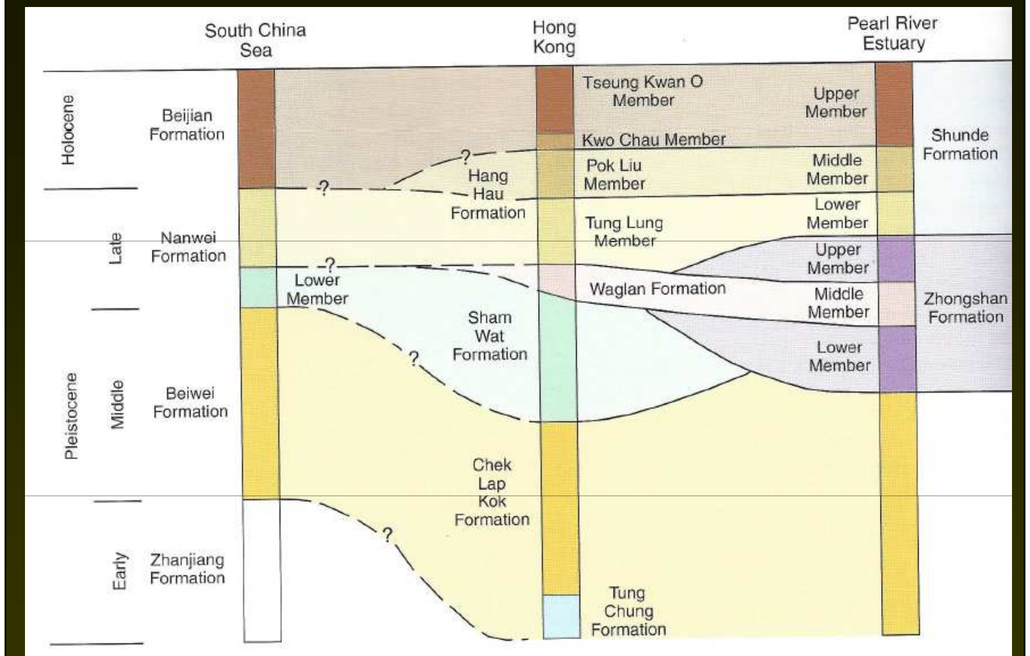




Ref: Pickles and Tosen – settlement of reclaimed land of the new HK International Airport

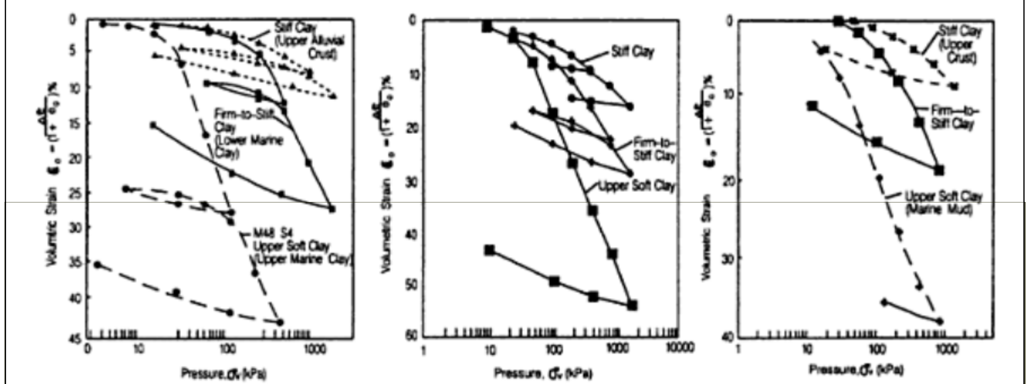


Source: Tosen, Pickles and Jaros (1998)



Ref: Quaternary Geology of Hong Kong, GEO,2000

Figure 5.13 Typical oedometer results for Upper Soft Clay, Firm-to-Stiff Clay, and Stiff Clay



Soil Type	Depth (m)	Borehole Sample	γ	w	w_L	w_P
Upper Soft Clay	4 – 4.7	M48 S4	1.45	106.9	90	34
Firm-to-Stiff Clay	16 – 16.7	M41 S15	1.74	31.2	61	26
Stiff Clay	9 – 9.7	M31 S9	1.96	26.7	48	21

(a) M–Series Holes (RMP–ENCON, 1982a)

Depth (m)	Borehole Sample	γ	w	w_L	w_P
8.45	B7 S6	1.77	40.7	53	22
13.65	B7 S13	1.74	48.3	64	27
15.8	B10 S17	1.98	23.5	42	19

(b) B–SERIES HOLES (CESD, 1990)

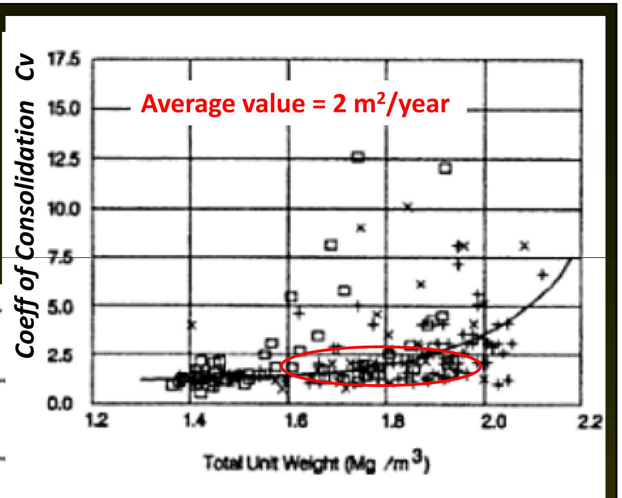
Depth (m)	Borehole Sample	γ	w	w_L	w_P
10 – 11	GMH4 S4	1.57	33.7	58	29
15.5–15.95	GMH1 S5	1.75	32.3	56	31
11 – 12	GMH10 S4	1.84	28.9	50	26

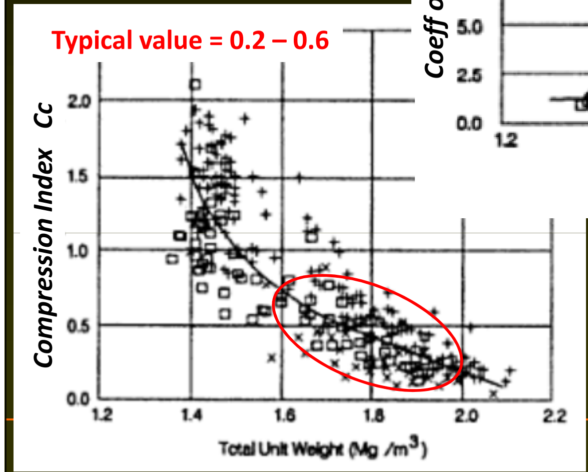
(c) GMH–Series Holes (Greiner–Maunsell, 1991a)

Notes : w – Water Content (%), w_L – Liquid Limit (%), w_P – Plastic Limit (%), γ – Bulk unit weight (t/m^3)

Ref: Site Preparation for the New Hong Kong Int Airport. Edited by G W Plant et al.

Back analysis of the piezometer data gives the c_v value of the alluvial clay in the range 4 – 30 (with an average = **16 m²/year**)





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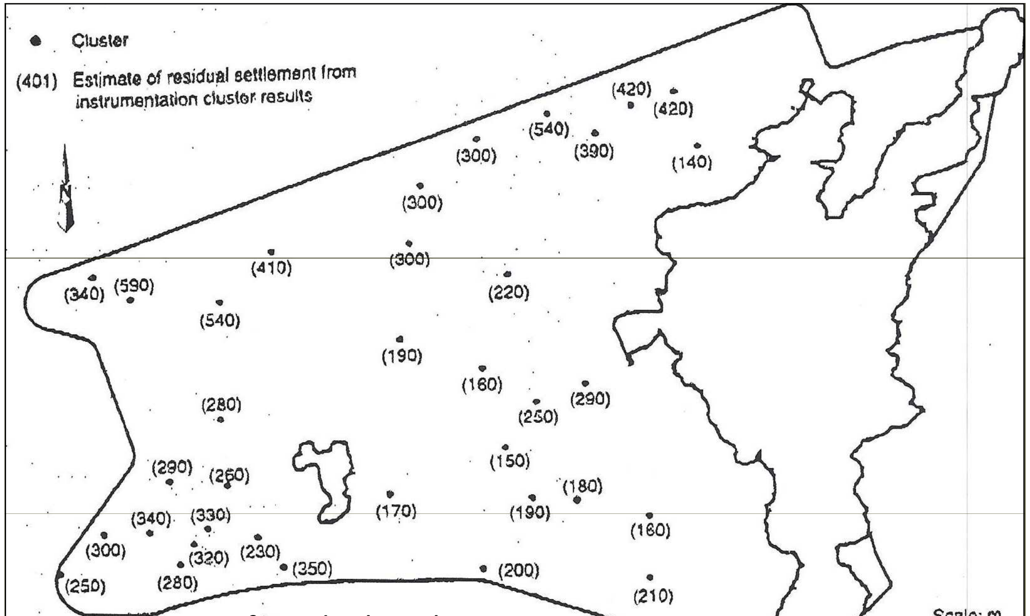
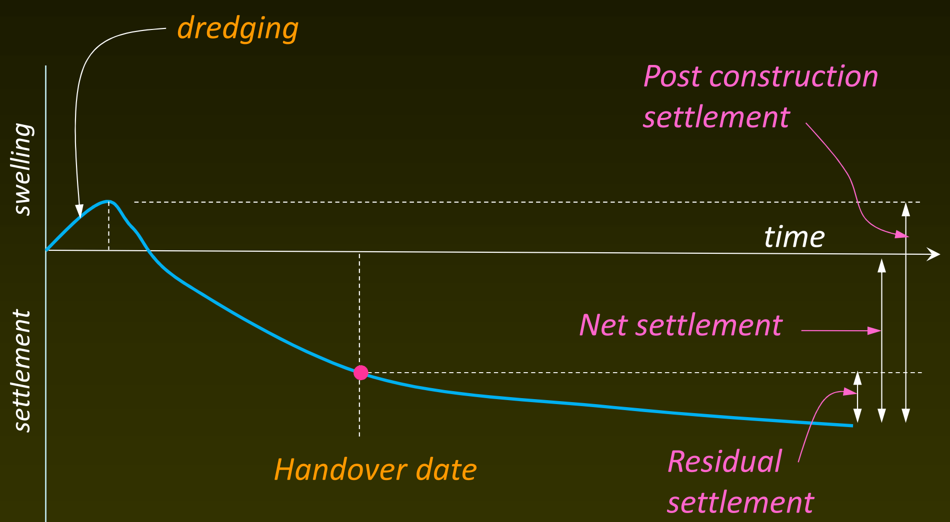


Best estimate of residual settlement in mm by observational method (Jan 1997 to 2040)

Ref: Pickles and Tosen – settlement of reclaimed land of the new HK International Airport



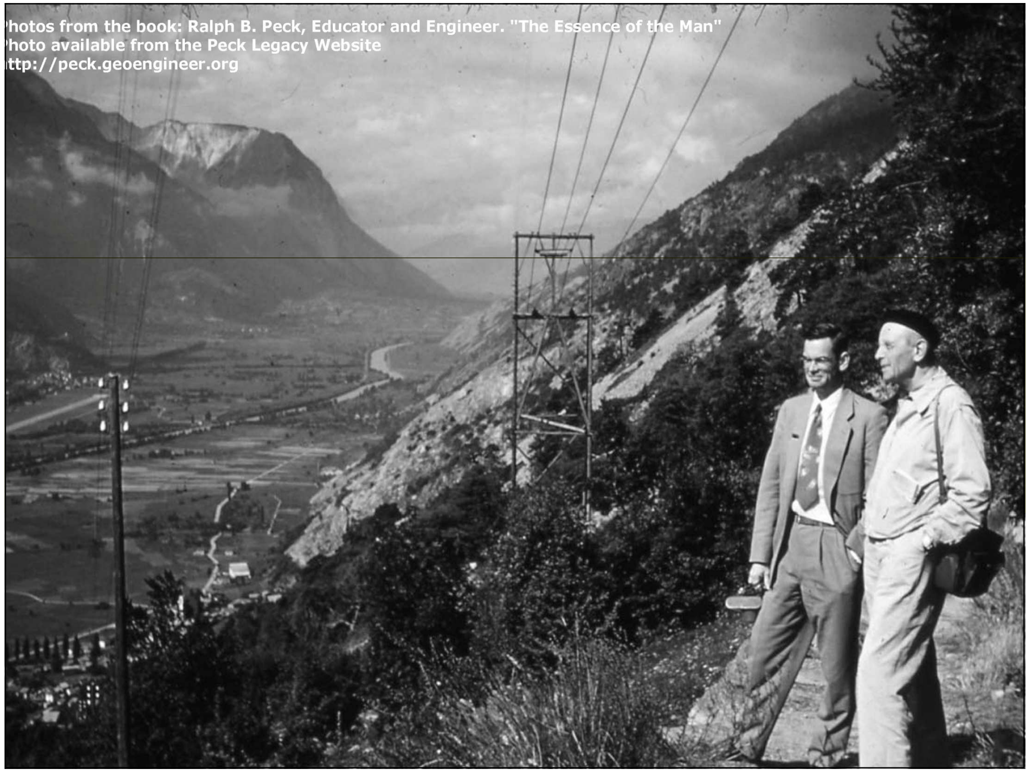
"Indeed, early in our association, Terzaghi advised me that on every new job I should start without any preconceptions and that it would be well to get all the facts by vigorous probing, quite as if soil mechanics did not exist, before attempting to make any interpretations."

R.B. Peck

Observational Method : a learn-as-you-go method

"In the engineering for such works as large foundations, tunnels, cuts, or earth dams, a vast amount of effort and labour goes into securing only roughly approximate values for the physical constants that appear in the equations. Many variables, such as the degree of continuity of important strata or the pressure conditions in the water contained in the soils, remain unknown. Therefore, the results of computations are not more that working hypotheses, ***subject to confirmation or modification during construction****."*

Quoted from an early version in the Introduction to Terzaghi and Peck's book "Soil mechanics in engineering practice" (but never published)



A drawback of the observational method:

If the situation is complex, or is not yet appreciated, designer may measure the wrong quantities and may come to dangerously incorrect conclusions.

In a case study of the subsidence of chemical plant (see Peck's Rankine Lecture in 1969, Geotechnique)

Peck : *Now about that report on the subsidence ..*
Terzaghi : *Yes ? I think you have missed the boat. It is obvious that the settlement is in the bedrock.*
Peck : *But that is impossible. The bedrock is too thick.*

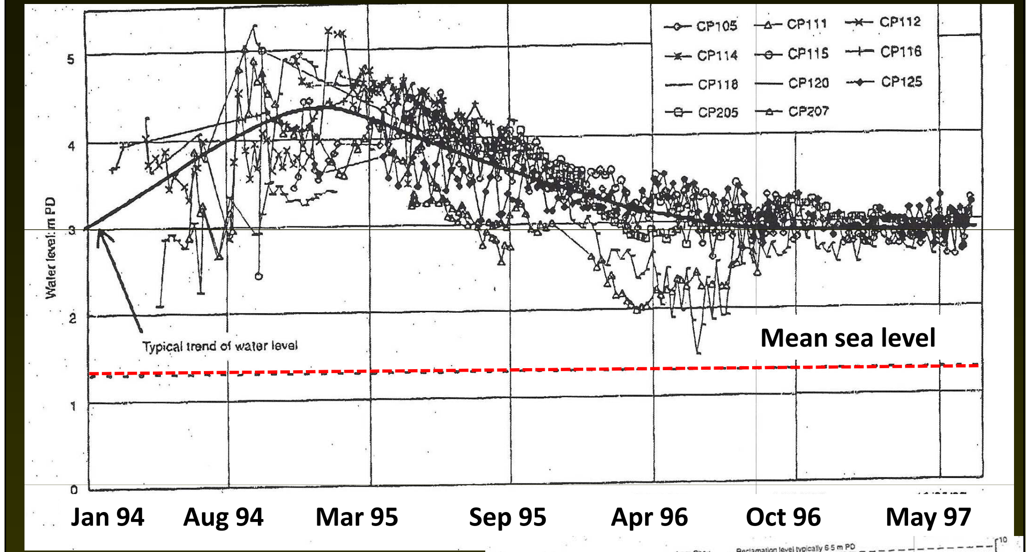
Terzaghi : *How do you know it is impossible? You didn't establish any reference points at the surface of the bedrock, did you?*

Peck : *No, but the general magnitudes of the observed*

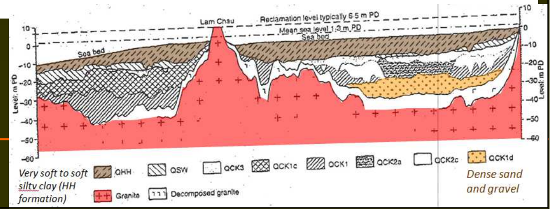
Terzaghi : *Didn't you notice that the real pattern of differential settlement is much more abrupt and erratic than the computed one?*

Peck : *Yes, but I think this difference is caused by the presence of erratic, compressible organic deposits near the ground surface.*

Terzaghi : *What is the evidence?* ***You have forced the evidence to fit your preconceived notions****.*



Piezometric level measured at dense sand and gravel layer beneath alluvial clay



Ref: Pickles and Tosen – settlement of reclaimed land of the new HK International Airport

3

Myth 3 : Secondary compression

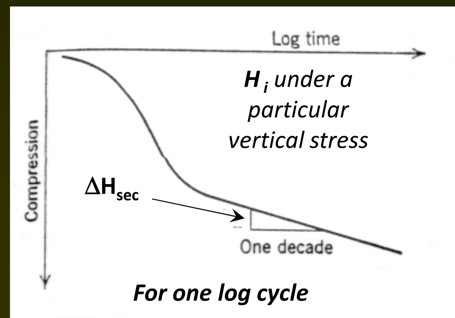
Secondary compression – c_{sec}

Geospec 3 defines ***the coefficient of secondary compression (C_{sec})*** *as : the ratio of the change in height, δH_{sec}, to the initial height, H_i, of a test specimen at the start of the primary consolidation under a particular vertical stress increment over one log cycle of time during the secondary compression phase*

$$C_{sec} = \frac{\Delta H_{sec}}{H_i} \frac{1}{\Delta \log_{10} t} = \frac{\Delta H_{sec}}{H_i}$$

For one log cycle

This definition has the beauty that you may calculate the settlement due to secondary compression directly

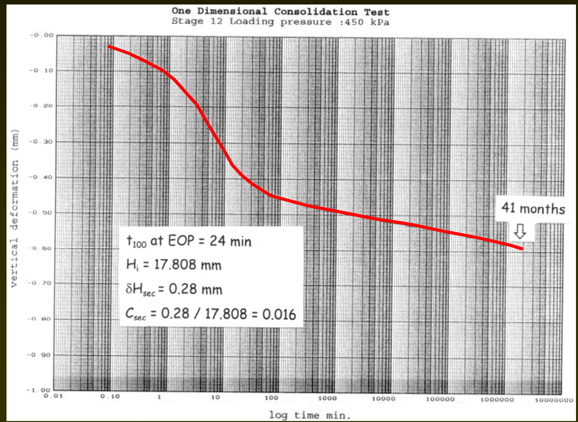


Geospec 3 specifies that the compression curve should cover at least one complete cycle of log time. e.g. from 100 min to 1000 min or from 1000 min to 10000 min

(1,000 min ~ 16.7 hours

10,000 min ~ 7 days)

$$C_{sec} = \frac{\Delta H_{sec}}{H_i} \frac{1}{\Delta \log_{10} t} = \frac{\Delta H_{sec}}{H_i}$$



An example for an alluvial clay at Chep Lap Kok airport is shown. (why 41 months were required??)

- *Geospec 3 distinguishes c_{sec} from the* ***secondary compression index, C_α*** *which is defined as $\Delta e / \Delta \log t$ (i.e. the slope of the compression curve)*
- *An important question to the designer: how do I know whether secondary compression is significant in my project/design?*
- *Some hints:*
  - *(a) past experience on similar soils*
  - *(b) highly sensitive soils*
  - *(c) soils with high organic contents*
  - *(d) soils with high c_c value also has high c_α value* ***(most practical hint)***

- *It is quite surprise to note that the ratio c_α / c_c is approximately a constant for most soil (e.g. see Mesri and Godlewski 1977)*
- *The following table gives you some information:*

Materials	C_α / C_c
Granular soils including rockfill	0.02 ± 0.01
Shale and mudstone	0.03 ± 0.01
Inorganic clays and silts	0.04 ± 0.01
Organic clays and silts	0.05 ± 0.01
Peat	0.06 ± 0.01

The graph shows an example of an alluvial clay from Chep Lap Kok formation (Lo et al):

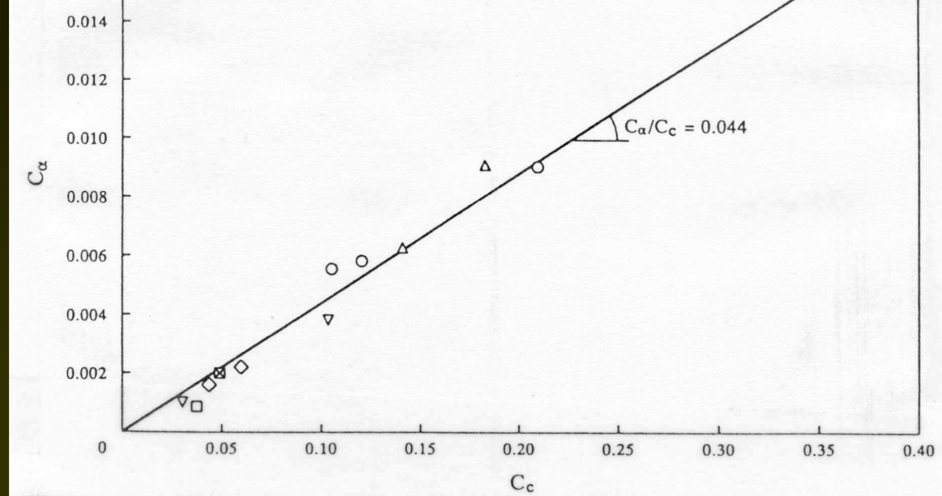


Table 5.4: Summary of the predicted settlement from the Master Plan (Greiner-Maunsell, 1990)

Geotechnical Strata	Settlement Allowance (m)	Reference
Chek Lap Kok Formation	0.13 (0.15)(1)	Table 5.3 (Approach A8)(1)
Completely Weathered Granite (CDG)	0.03	Figure 5.22
Filling material	0.10	Estimated
Secondary consolidation (for 50 years)	0.15	See Section 5.6.1
Future ground improvement works including compaction by temporary traffic	0.10	Estimated
Total	0.51 (0.53)(1)	

Note (1) Values in brackets include consolidation settlement of the potential 2 m of retained Upper Soft Clay

Two more questions on secondary compression

- WHY NOT SECONDARY CONSOLIDATION
- WHEN DOES SECONDARY COMPRESSION COMMENCE

After the completion of primary consolidation ?

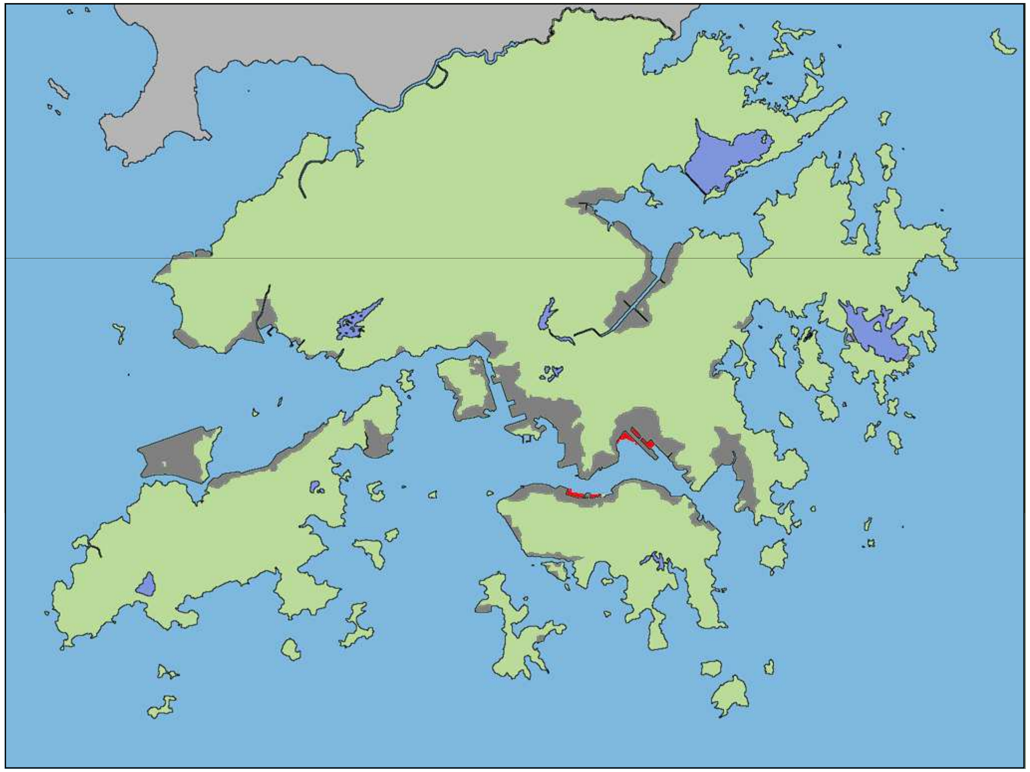
At some time of the primary consolidation ? e.g. at U = 0.6

Almost concurrently starts with primary consolidation ?

Other possible scenario ?

4

MYTH 4 : CREEP IN RECLAMATION FILL



CREEPING OF THE FILL MATERIALS MAY CONTRIBUTE SIGNIFICANTLY TO THE OVERALL SETTLEMENT AND THIS ASPECT SHOULD NOT BE TAKEN LIGHTLY

Table 5.4: Summary of the predicted settlement from the Master Plan (Greiner-Maunsell, 1990)

Geotechnical Strata	Settlement Allowance (m)	Reference
Chek Lap Kok Formation	0.13 (0.15)[1]	Table 5.3 (Approach A8)[1]
Completely Weathered Granite (CDG)	0.03	Figure 5.22
Filling material	0.10	Estimated
Secondary consolidation (for 50 years)	0.15	See Section 5.6.1
Future ground improvement works including compaction by temporary traffic	0.10	Estimated
Total	0.51 (0.53)[1]	

Note (1) Values in brackets include consolidation settlement of the potential 2 m of retained Upper Soft Clay

The physical mechanisms leading to creep of fill materials are not well understood. It may be due to the degradation of the point-to-point contacts between particles and the subsequent rearrangement of the fill particles due to e.g. vibrations or other input energy.

The creep rate of fill materials could be comparable with that of secondary compression of the soil.

The creep rate of fill can be modelled as a linear relationship with log time :

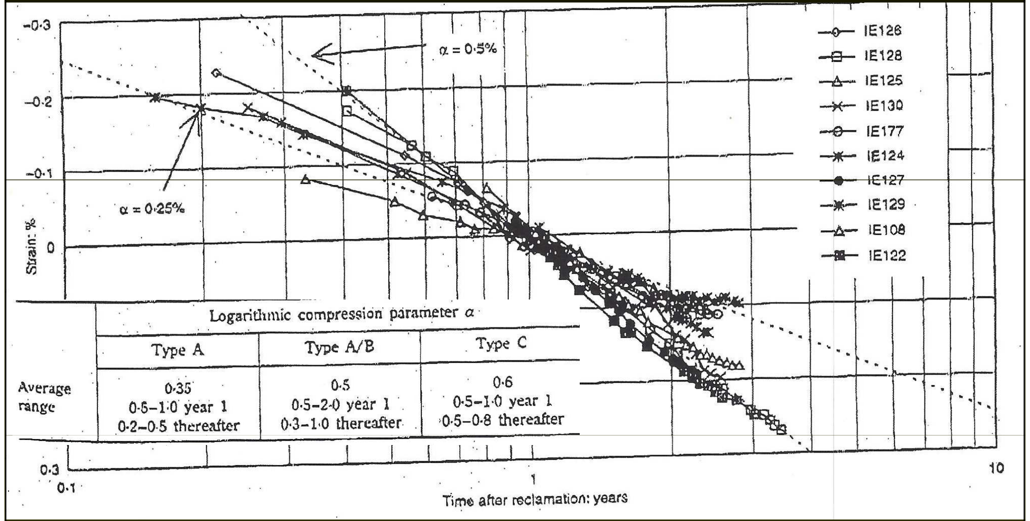
Settlement due to creep of fill $= H \alpha \log(t_2/t_1)$
between time t_1 and t_2

where α is the log creep compression rate (%)

Port Works Design Manual (CED 2002) gives the following recommendation:

For granitic fill : 1% - 2%
but may reduce to 0.5% - 1% after fill treatment

Measured creep in fill type A/B (rock fill + CDG) along the northern runway



	Logarithmic compression parameter α		
	Type A	Type A/B	Type C
Average range	0.35 0.5–1.0 year 1 0.2–0.5 thereafter	0.5 0.5–2.0 year 1 0.3–1.0 thereafter	0.6 0.5–1.0 year 1 0.5–0.8 thereafter

From observational method, it was established that the creep of the reclamation fill comprised about 50% of the residual settlement

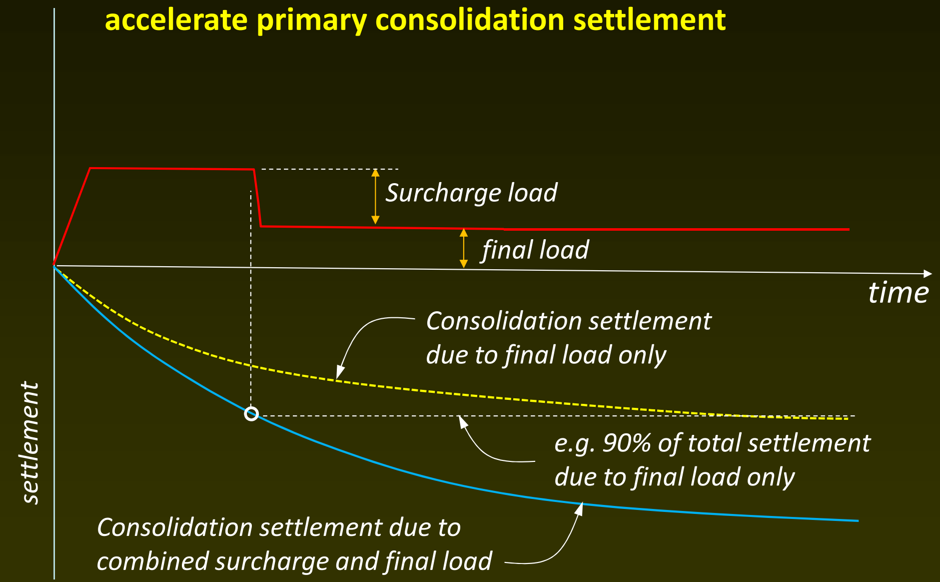
Ref: Pickles and Tosen – settlement of reclaimed land of the new HK International Airport

5

MYTH 5 : GROUND TREATMENT BY MEANS OF SURCHARGE LOADING AND VERTICAL DRAINS

WHAT IS THE MECHANISM OF SPEEDING UP SETTLEMENT BY MEANS OF SURCHARGE LOADING AND VERTICAL DRAINS ?

Schematic diagram showing how surcharge loading can accelerate primary consolidation settlement

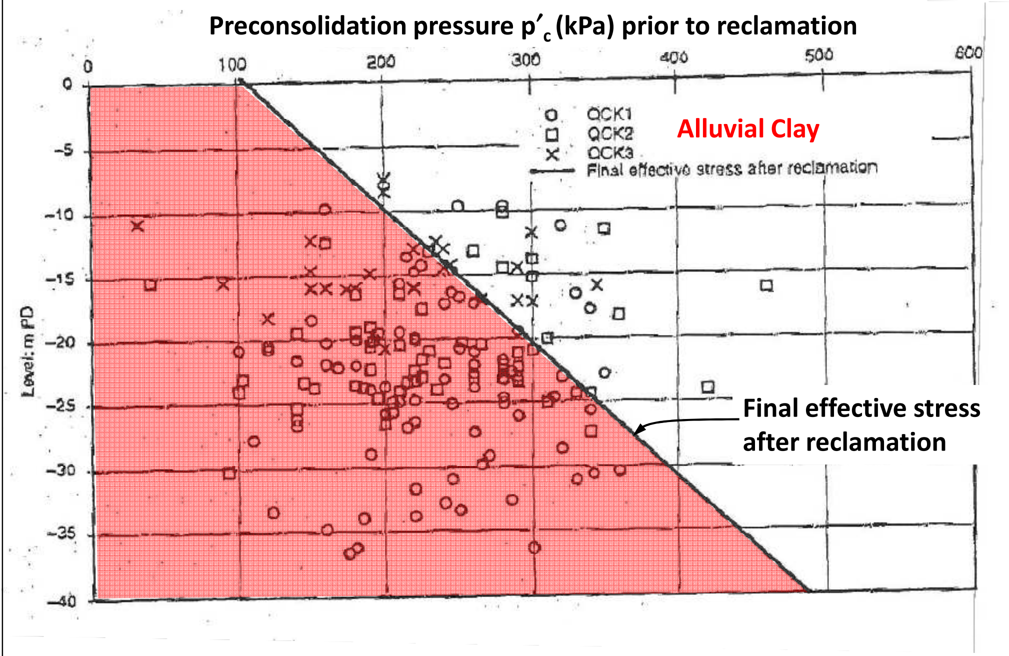


The duration of surcharge load application can be notionally determined from the previous slide.

However, designer usually forgets two important points in adopting surcharge loading:

(1) Surcharge is effective in normally consolidated soil and only effective in overly consolidated soil if the preconsolidation pressure is exceeded.

(2) Before the surcharge is removed, the remaining excess pore water pressure must be smaller than the design load (final load).

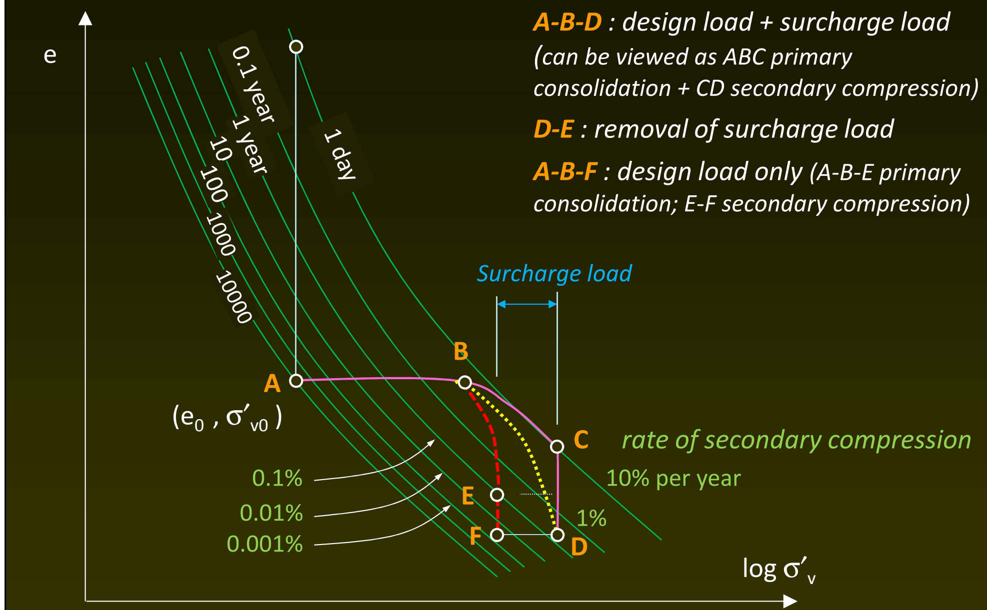


Pre-loading is also beneficial to secondary compression in terms of magnitude and rate.

A-B-D *: design load + surcharge load (can be viewed as ABC primary consolidation + CD secondary compression)*

D-E *: removal of surcharge load*

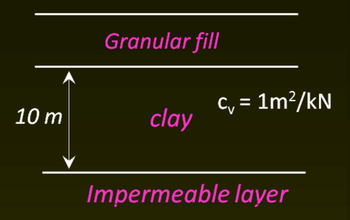
A-B-F *: design load only (A-B-E primary consolidation; E-F secondary compression)*



The working principle of vertical drains is illustrated in the next slide.

Q1 : how long will it take to complete 90% of the total settlement?

$U = 0.9$, From chart, $T_v = 0.85$
As $T_v = (c_v)t/(d^2)$,
$0.85 = 1 \times t / (10^2)$, $\therefore t = 85$ years

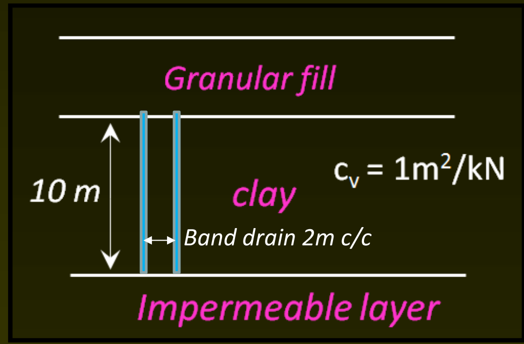


With the installation of band drain of 2m c/c :

$d = 1$ m, assume $c_h = c_v$,

$0.85 = 1 \times t / (1^2)$, $\therefore t = 0.85$ years !

Hence, installation of effective band drain may shorten the consolidation time by 100 times.



Installation of vertical drains is effective in speeding up the primary consolidation settlement only if certain criteria are met (e.g. see Port Works Design Manual)

- *Tensile strength of the drain to withstand the stress induced dduring installation*
- *Transverse permeability*
- *Soil retention and clogging resistance*
- *Vertical discharge capacity under confining pressure*
- *Performance in folded condition*
- *Durability*

End

Supplementary Slides

1916 Terzaghi lectured at the Imperial Institute of Engineering, Constantinople, Turkey

1918 Terzaghi taught in the Robert College in Constantinople until 1925 when he moved to MIT. He developed the mathematical theory of consolidation between 1918-23

1923 He published the work on consolidation (which together with his principle of effective stresses) provided the basis for the development of soil mechanics

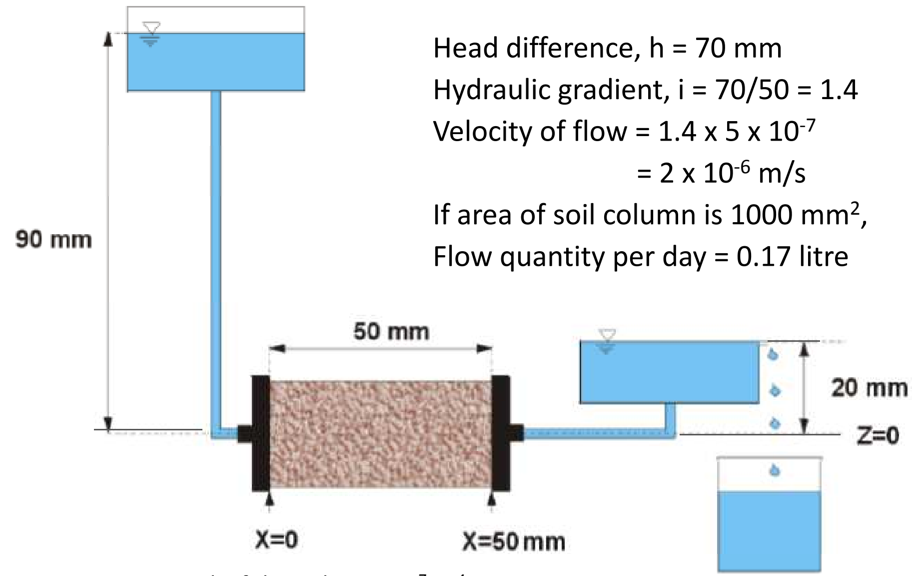
1925 He published the book "Erdbaumechanik"

what drives the flow of water?

Water flows from high elevation to low elevation and from high pressure to low pressure, hence ***difference in total head*** *(i.e. gradient in potential energy) drives the water flow*

Hydraulic gradient = total head / length of soil

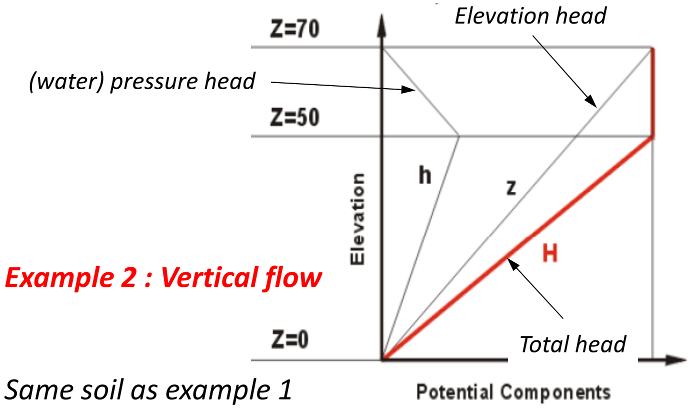
Example 1 : Horizontal flow

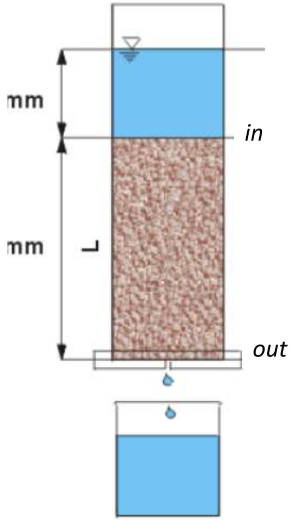


Head difference, h = 70 mm
Hydraulic gradient, i = 70/50 = 1.4
Velocity of flow = 1.4 x 5 x 10^{-7}
= 2 x 10^{-6} m/s
If area of soil column is 1000 mm^2,
Flow quantity per day = 0.17 litre

k of the soil = 5 x 10^{-7} m/s
(i.e. fine silt such as fine CDV or kaolin)

Example 2 : Vertical flow





Same soil as example 1

Difference in hydraulic head across the soil length :

$(h+z)_{in} - (h+z)_{out}$ i.e. (20+50) – (0+0) = 70 mm

Hydraulic gradient, i = 70/50 = 1.4

Velocity of flow = $1.4 \times 5 \times 10^{-7} = 2 \times 10^{-6}$ m/s

Comments : (1) the velocity of flow is the same irrespective of whether the flow is vertical or horizontal. (2) Flow is governed by the ***total head****.*