Soil's Compressibility

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Settlement

If a structure is placed on soil surface, then the soil will undergo an *elastic* and *plastic* deformation.

In engineering practice, the deformation or reduction in the soil volume is seen as ***settlement*** or heave depending on weather the load is increased or decreased.

Components of Total Settlement

$$S = S_i + S_c + S_s$$

S_i = *Immediate Settlement*

S_c = *Consolidation Settlement*

S_s = *Secondary compression*

Immediate Settlement

Immediate settlement is time – independent and results from shear strain that occur at constant volume as the load is applied to the soil.

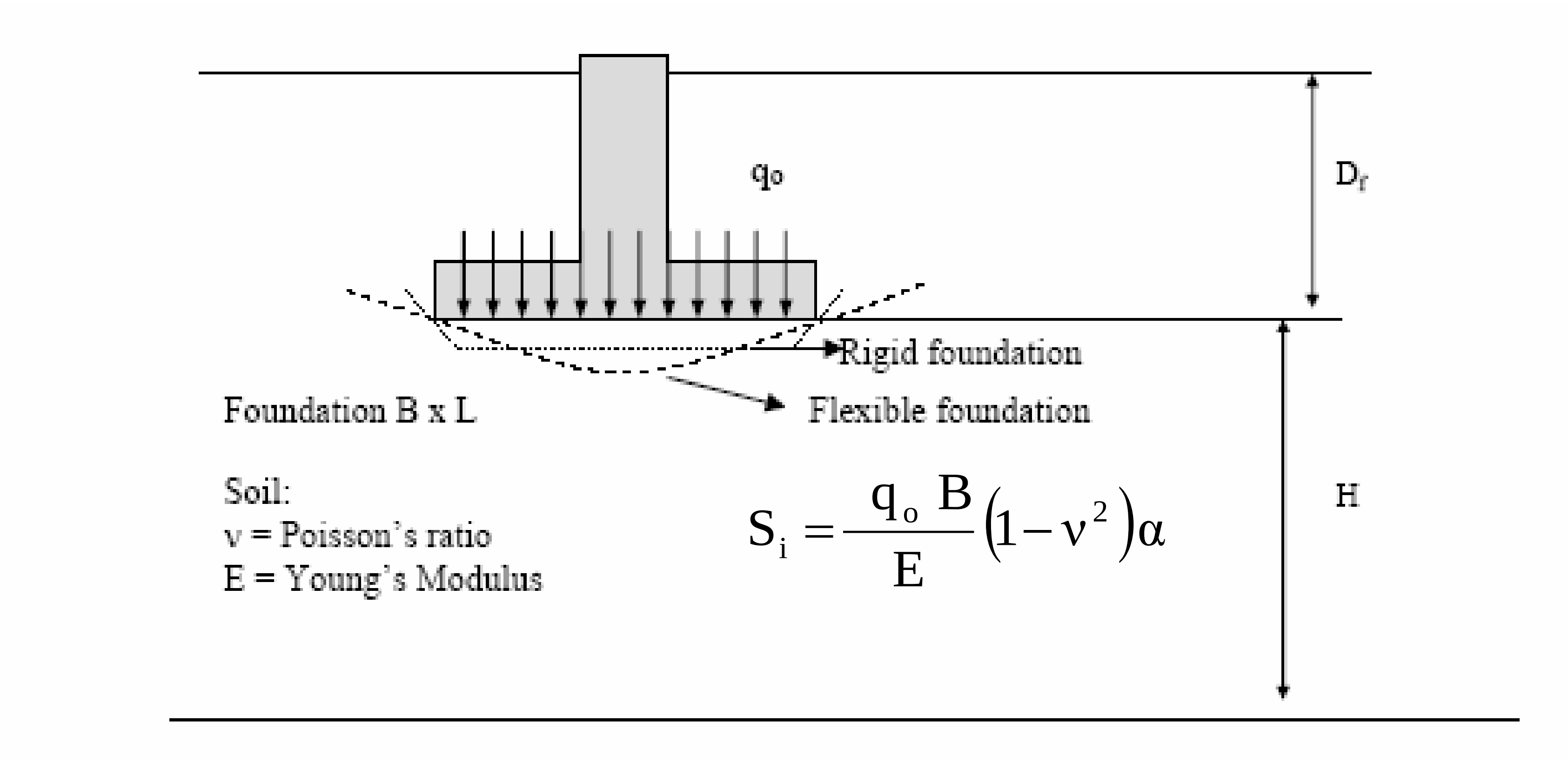
It is due to elastic Deformation of soil grain → *small and negligible*

Soil grains are not elastic → but generally calculated using elastic theory for cohesive soils

involves some assumptions such as homogeneity and isotropic which are not actually representative of natural soil properties.

Immediate/Elastic Settlement

General



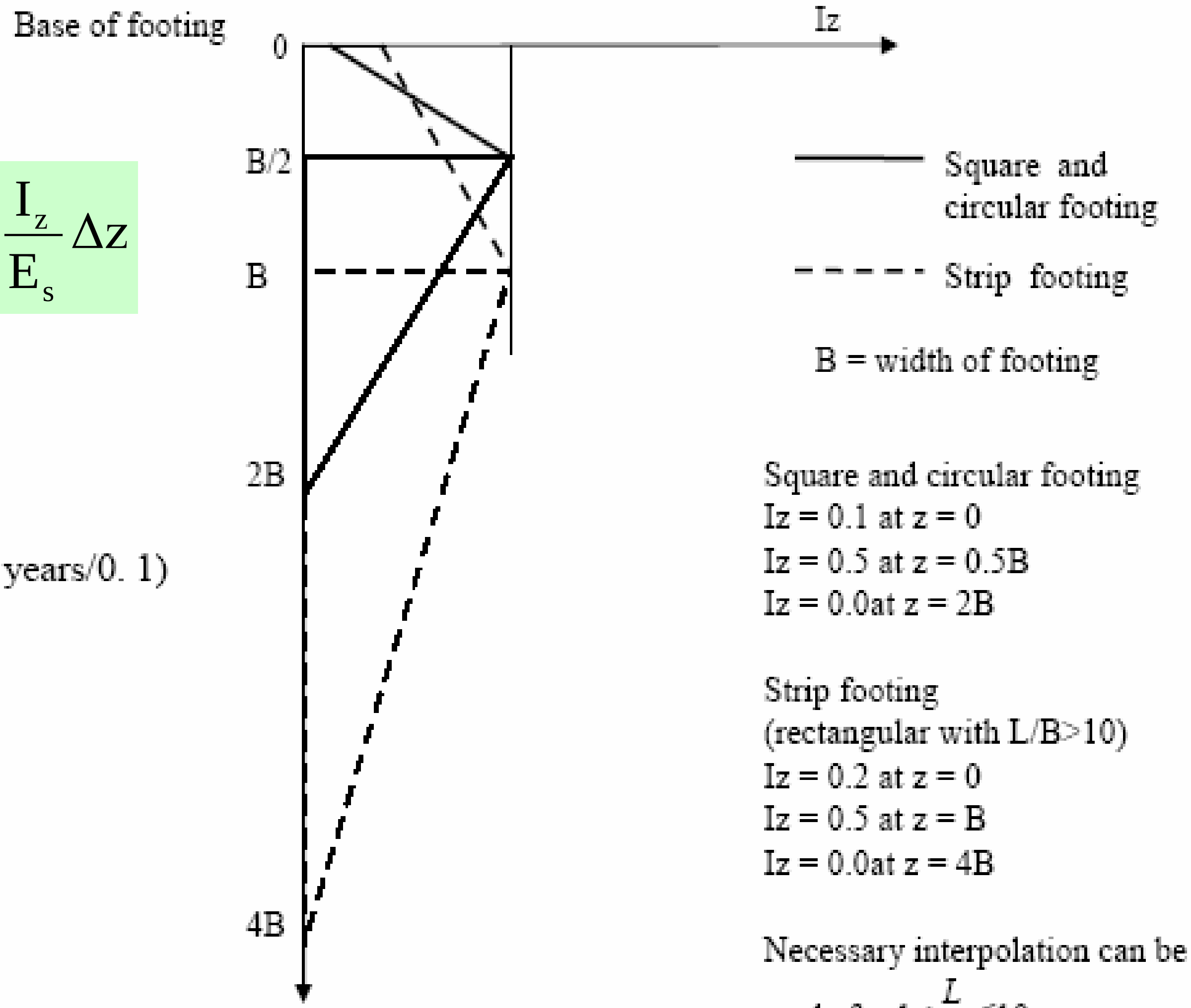
Immediate/Elastic Settlement

Schmertmann method

$$S_i = C_1 C_2 (q_o - q) \sum_0^{2B} \frac{I_z}{E_s} \Delta z$$

$$C_1 = 1 - 0.5\left(\frac{q}{q_o - q}\right)$$

$C_2 = 1 + 0.2 \log$ (time in years/0. 1)



B = width of footing

Square and circular footing
Iz = 0.1 at z = 0
Iz = 0.5 at z = 0.5B
Iz = 0.0at z = 2B

Strip footing
(rectangular with L/B>10)
Iz = 0.2 at z = 0
Iz = 0.5 at z = B
Iz = 0.0at z = 4B

Necessary interpolation can be made for $1 < \frac{L}{B} < 10$

Influence factors for vertical displacement under footing on soil layer of infinite depth

Table 4.1 Influence factors for vertical displacement under footing on soil layer of infinite depth

Shape of footing base	α			
	Center	Edge/sides	Corner	Average
Flexible foundations:				
Circle	1.00	0.64	-	0.85
Square	1.12	0.76	0.56	0.95
Rectangular $L/B = 2$	1.53	1.12	0.76	1.30
Rectangular $L/B = 5$	2.10	1.68	1.05	1.82
Rectangular $L/B = 10$	2.56	2.10	1.28	2.24
Rigid foundations:				
Circle	0.79	0.79	-	0.79
Square	0.82	0.82	0.82	0.82
Rectangular $L/B = 2$	1.12	1.12	1.12	1.12
Rectangular $L/B = 5$	1.60	1.60	1.60	1.60
Rectangular $L/B = 10$	2.00	2.00	2.00	2.00

Influence factors for vertical displacement under footing on soil layer of limited depth

Table 4.2 Influence factors for vertical displacement under footing on soil layer of limited depth

Shape of footing base	α				
	$H/B = 1$	$H/B = 2$	$H/B = 5$	$H/B = 10$	$H/B = \infty$
Flexible foundations:					
Square $L/B = 1$	0.15	0.29	0.44	0.48	0.56
Rectangular $L/B = 2$	0.12	0.29	0.52	0.64	0.76
Rectangular $L/B = 5$	0.10	0.27	0.55	0.76	1.05
Rectangular $L/B = 10$	0.04	0.26	0.54	0.77	1.28
Rectangular $L/B = \infty$	0.04	0.26	0.52	0.73	-
Rigid foundations:					
Circle	0.35	0.54	0.69	0.74	0.79

Modulus of soil

Problem related to the calculation of elastic settlement is to get the accurate estimation of modulus i.e.

- Undrained modulus (E_u)
- Drained / Deformation Modulus (E_d)

Undrained Modulus

Stress strain curve from Triaxial Test (not accurate)

Plate Load Test (Quick loading) (better)

Pressuremeter Test ($E_p \neq E_u \neq E_d$) (be careful)

Dilatometer Test (suggested)

USE CORRELATIONS

Drained Modulus

- Oedometer Test

$$E_d = \frac{1}{m_v} = \frac{(1 + e_o)}{\alpha_v} = \frac{(1 + e_o)\sigma_{av}}{0.435C_c}$$

- Plate Load Test (Slow Loading) (better)
- Correlations

Correlation for Modulus

Based on results of in-situ testing (Correlation)

For the results of CPT, Schmertman (1978) suggested to use $E_s = 2.5\ q_c$ for square or circular foundation,
and $E_s = 3.5\ q_c$ for strip footing,

For the results of standard penetration test, an empirical correlation $E_s = 766\ N\ (kN/m^2)$ was suggested

For the results of field vane shear test data (c_u):
$E_u = 500 – 1500\ c_u$

Values of Modulus & Poisson's ratio

Based on the type of soil encountered

Table 4.3 Values of Modulus and Poisson's ratio

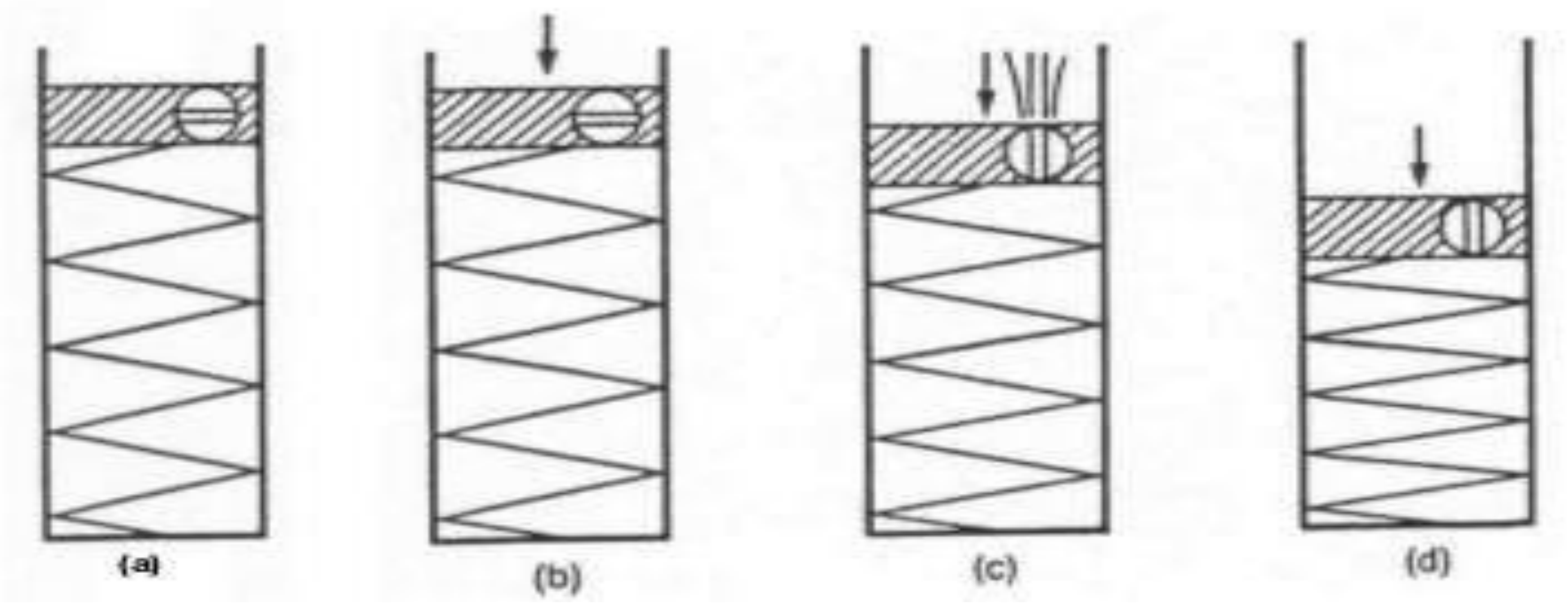
Soil Type	Cohesion (kPa)	Young's modulus E (MPa)	Poisson's ratio ν
Soft clay	< 25	2.5 - 15	0.40 – 0.50
Medium to stiff clay	25 – 100	15 – 50	0.45 – 0.50
Very Stiff to hard clay	> 100 kPa	50 – 100	0.45 – 0.50
Sandy Clay		25 – 250	
Silty Sand		5 – 20	
Loose sand		10 – 25	0.20 – 0.40
Dense sand		50 – 81	
Silt		2 - 20	

Consolidation Settlement

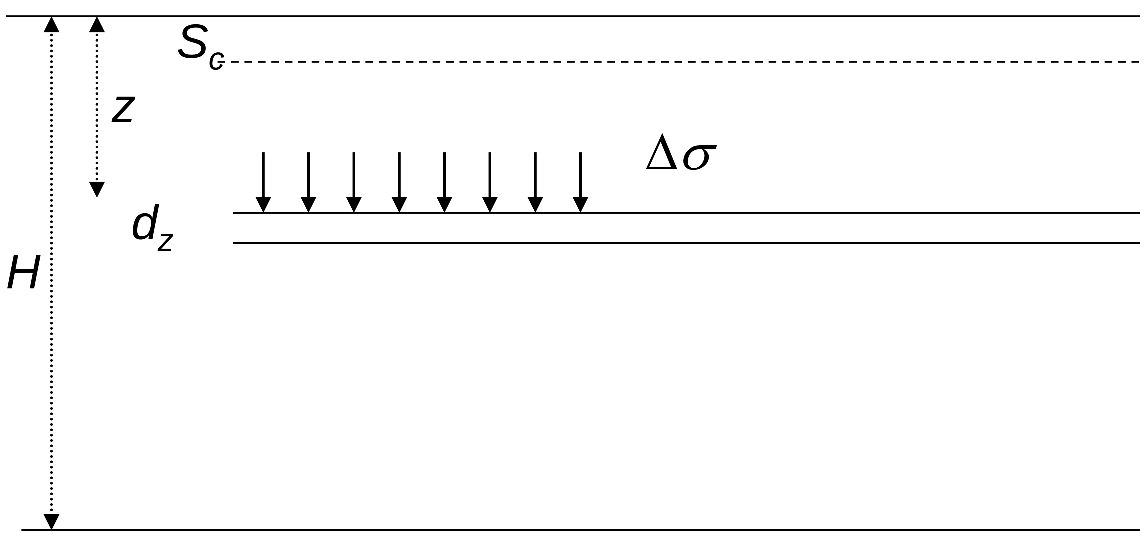
- Non-linear & irreversible
- Time dependent → Rate of settlement
- It is due to the squeezing out of water from the voids (dissipation of pore water pressure / water is incompressible)

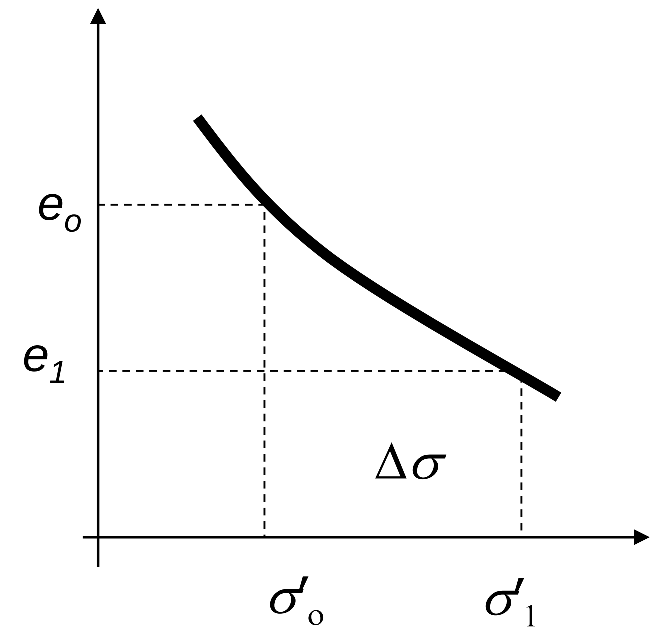
Consolidation Settlement

When saturated soil is loaded → Load is taken by the water → water dissipate → Load is taken by the soil skeleton → Deformation



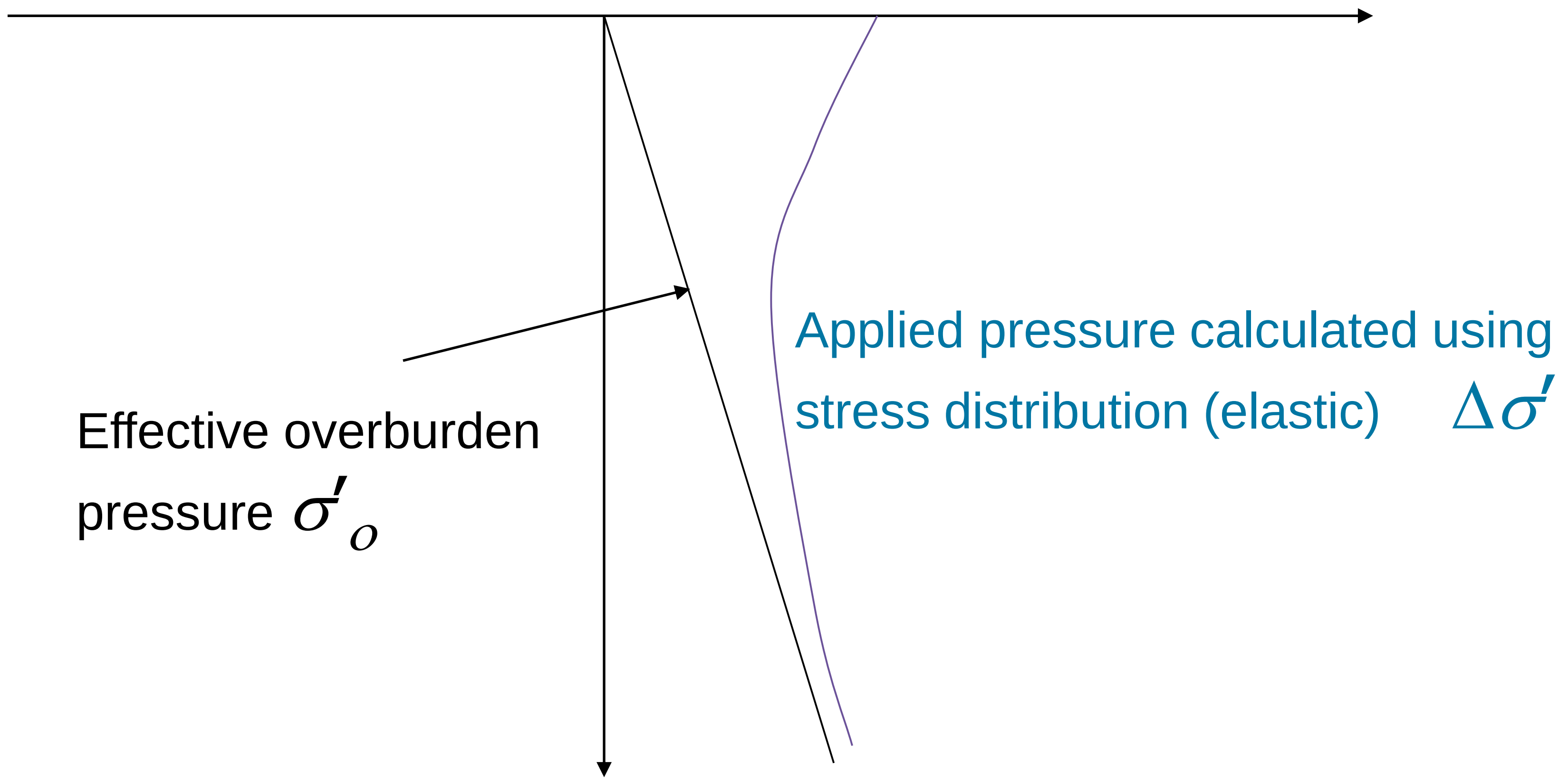
Consolidation Settlement: One-dimension





Consider a layer of saturated clay of thickness H, due to construction, the total vertical stress in an elemental layer of thickness d_z at depth z is increased by $\Delta\sigma$

Initial stress & stress increment



Settlement of Normally Consolidated Soil

$$S_c = C_c \frac{H}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma}{\sigma'_o}$$

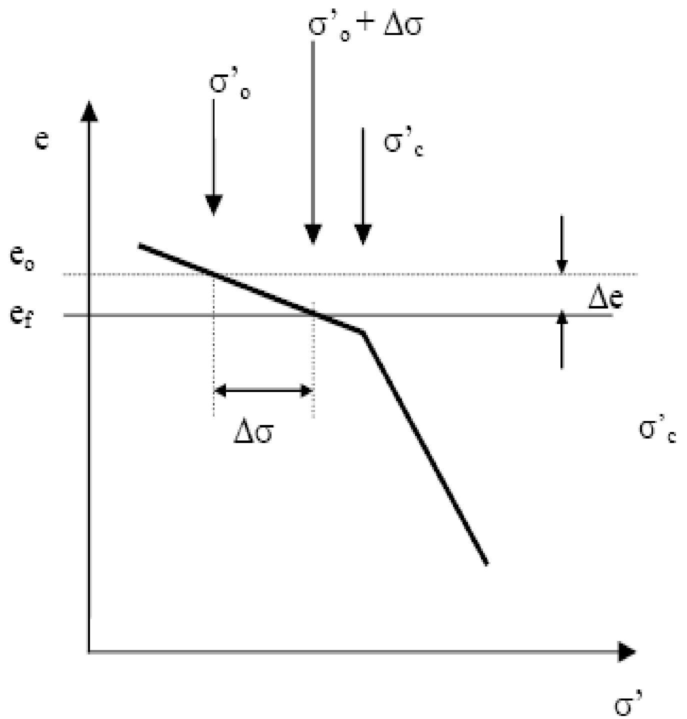
If we use m_v, then

$S_c = m_v \, \Delta\sigma' \, H$ or $S_c = a_v/(1+e_o) \, \Delta\sigma' \, H$

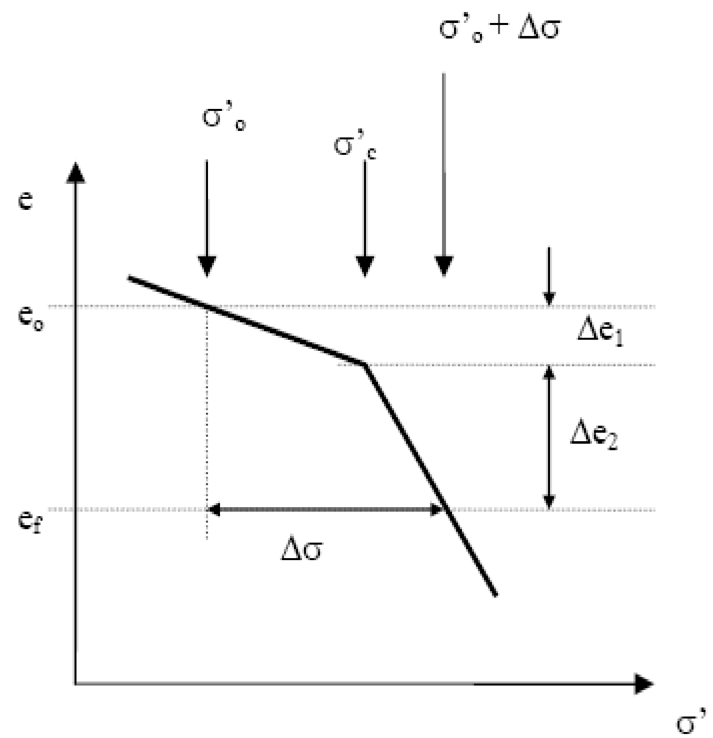
Settlement of Over-consolidated Soil

$$S_c = C_r \frac{H}{1+e_o} \log \frac{\sigma'_o + \Delta\sigma}{\sigma'_o}$$

$$S_c = C_r \frac{H}{1+e_o} \log \frac{\sigma'_c}{\sigma'_o} + C_c \frac{H}{1+e_{o1}} \log \frac{\sigma'_o + \Delta\sigma}{\sigma'_c}$$



(a)

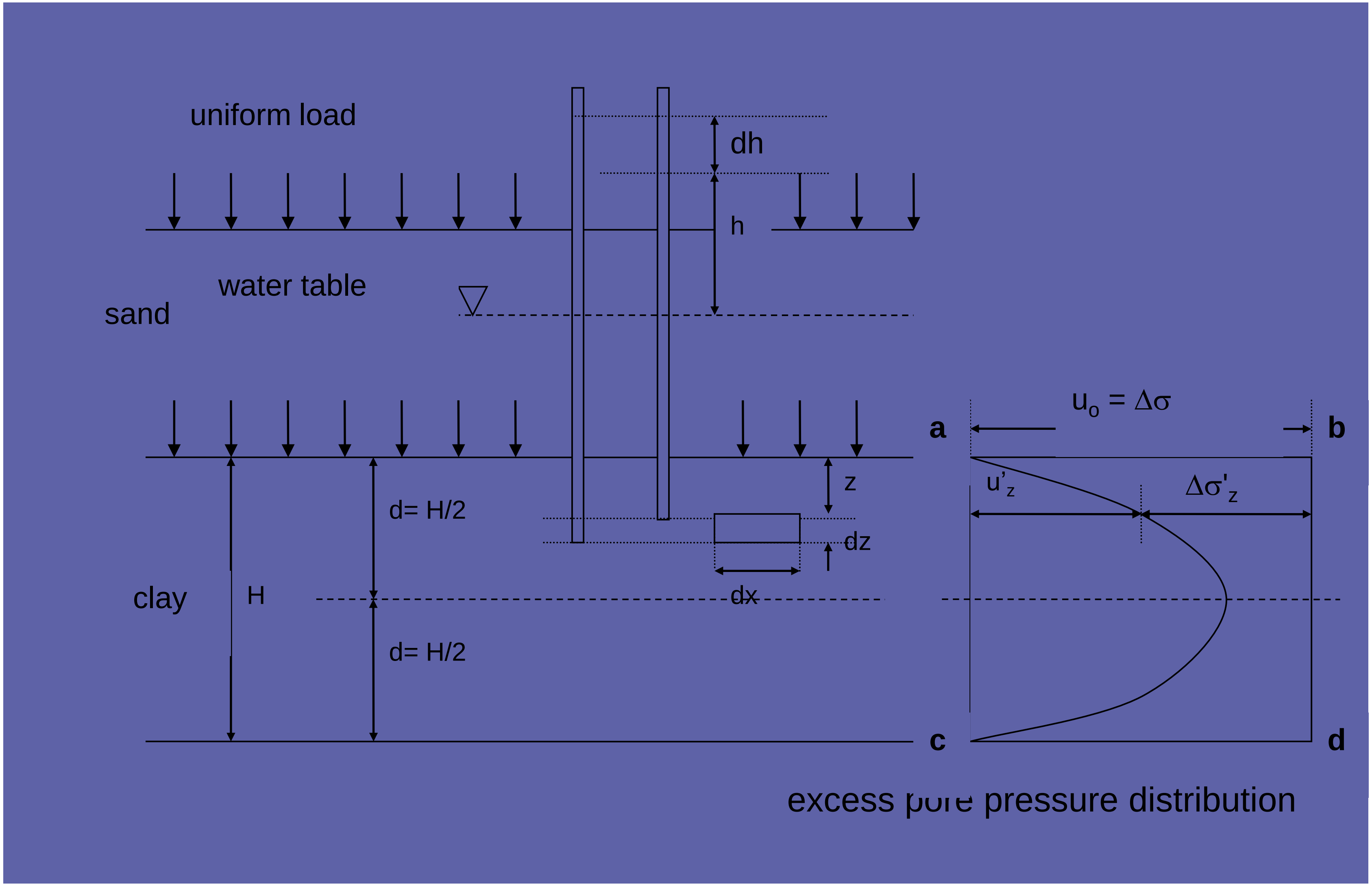


(b)

Rate of Consolidation

Terzaghi 1-D consolidation theory; **Assumptions:**

1. Compression and flow are 1-dimensional
2. The compressible layer is homogenous and saturated
3. Soil particles and water is incompressible
4. Darcy's law is valid
5. Strain due to external load is small and within the range of elasticity
6. The coefficient m_v and k remain constant throughout the process
7. There is a unique relationship, independent of time, between void ratio and effective stress



Distribution of excess pore water pressure in a clay layer subjected to uniform load

Solution to the Governing Consolidation Equation

The solution which satisfies these boundary conditions is obtained using sine harmonic Fourier series

$$\Delta u(z,t)=\sum_{m=0}^{\infty}\frac{2\Delta\mu_o}{M}\sin M\left(1-\frac{z}{H_d}\right)\exp\left(-M^2T_v\right)$$

Where $$M=\frac{\pi}{2}\left(2m+1\right)$$ and *m* is a positive integer

and T_v is Time f $$T=Cv\,\frac{t}{{H_d}^2}$$

Degree of Consolidation (U)

Degree of consolidation is the amount of consolidation completed at a particular time and depth. The ratio is equal to 0 everywhere at the beginning of consolidation and becomes unity when the consolidation completed

$$U_z = 1 - \frac{\Delta u_z}{\Delta u_o} = 1 - \sum_{m=0}^{\infty} \frac{2}{M} \sin\left(\frac{M\ z}{H_{dr}}\right) \exp\left(-M^2 T_v\right)$$

The relationship between *U*, *T* and *z* is represented by an isochrones

Degree of consolidation

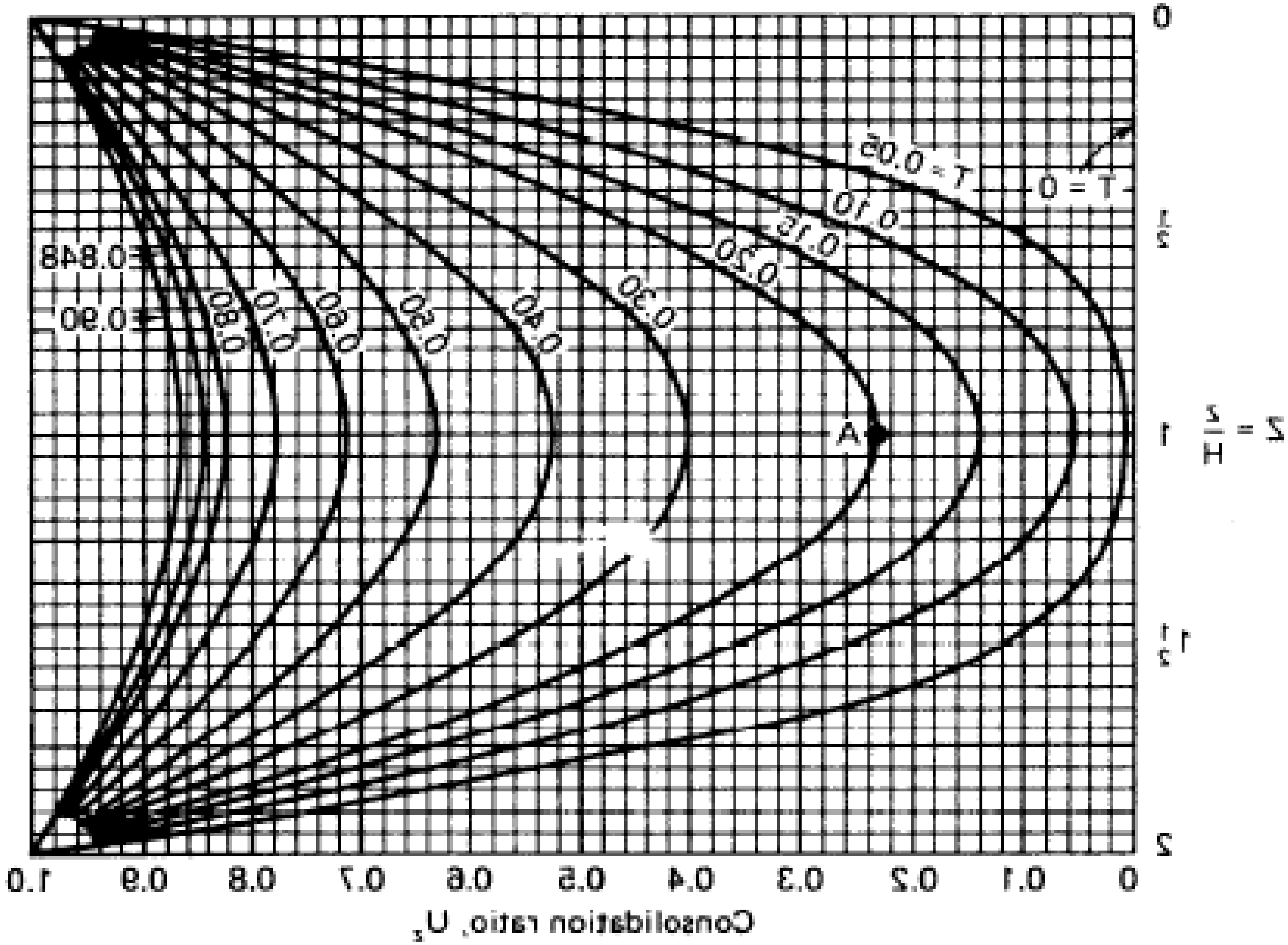
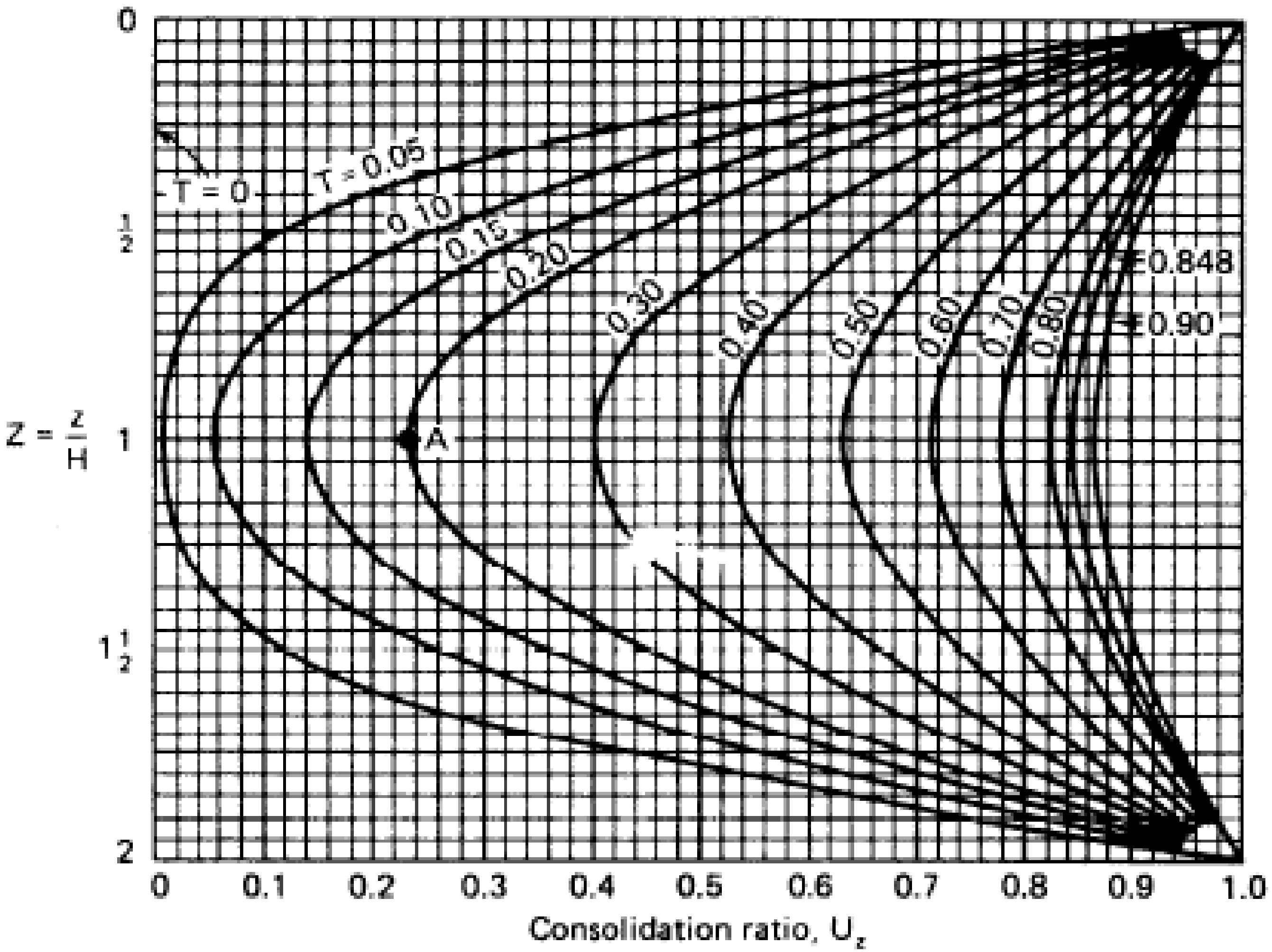


Fig. 9.3 Consolidation for any location and time factor in a doubly drained layer (after Taylor, 1948).

Degree of consolidation



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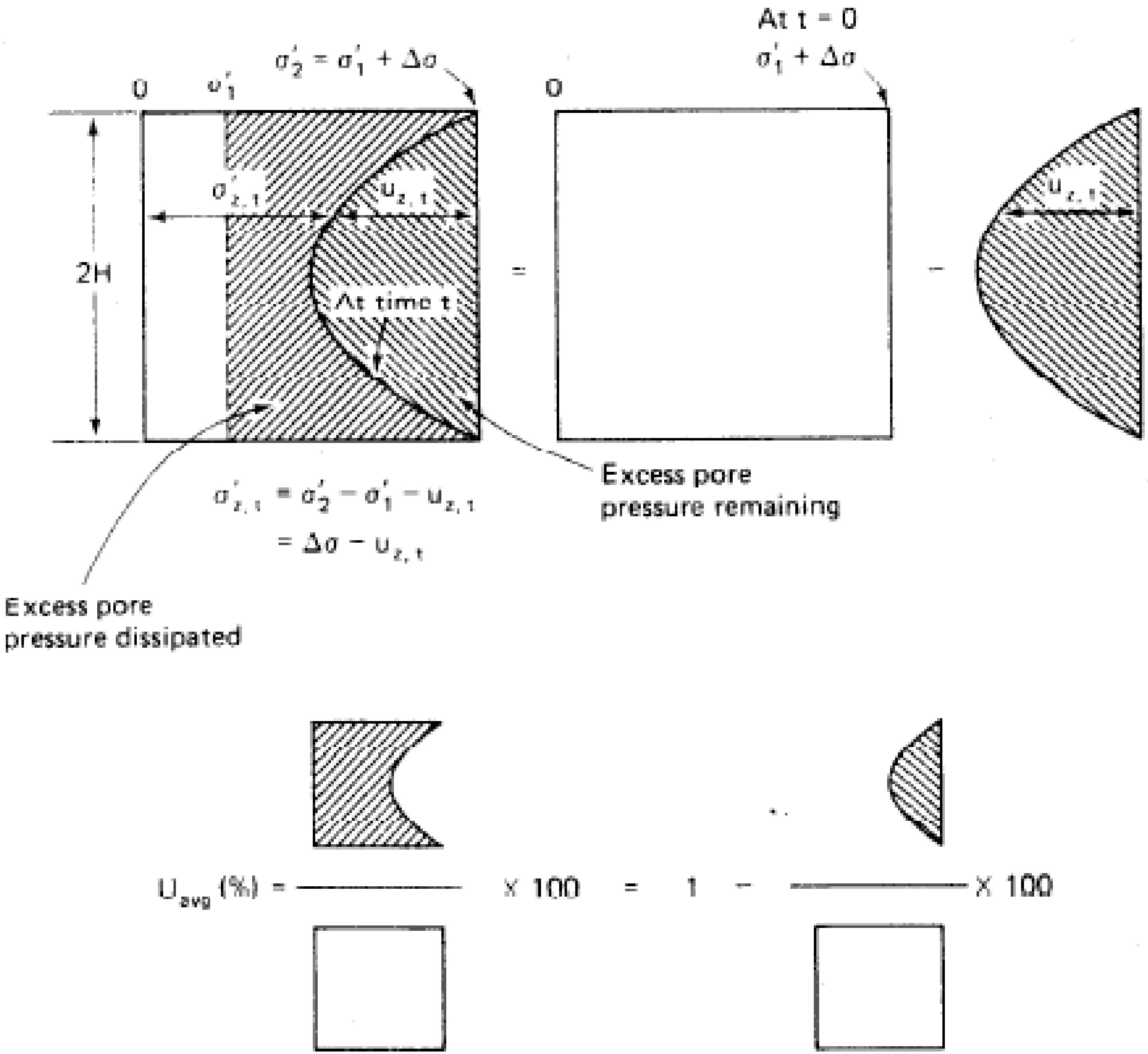


Fig. 9.4 Average degree of consolidation, U_{avg}, defined.

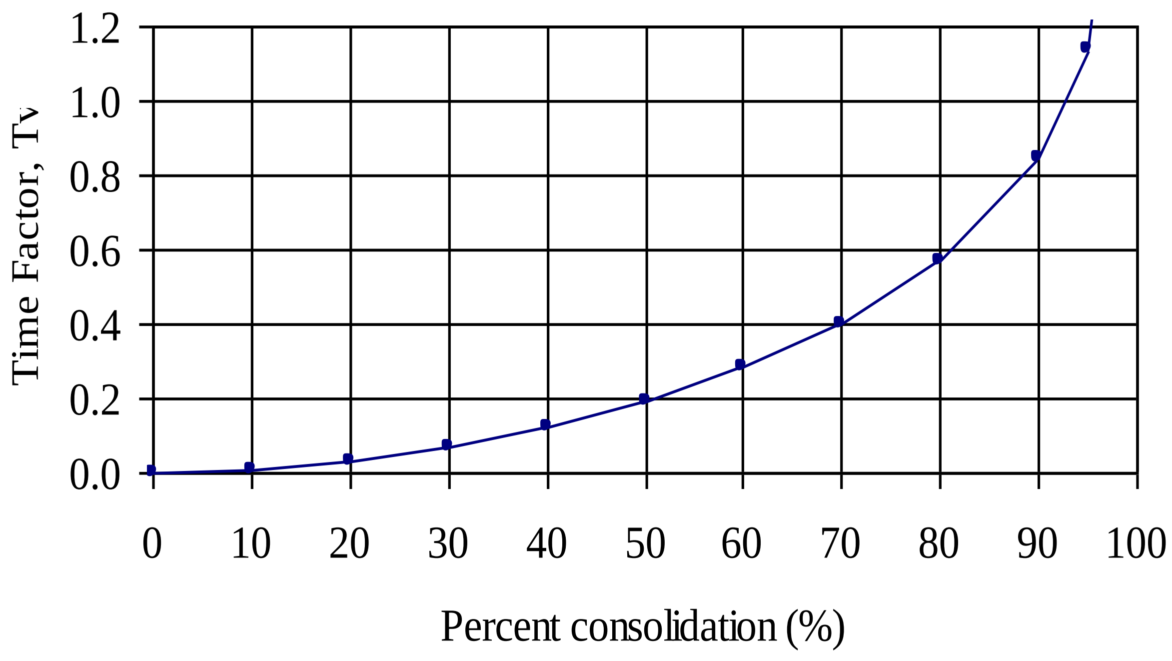
Average degree of consolidation

Geotechnical engineer is interested in the average degree of consolidation NOT the degree of consolidation at a particular depth, therefore

$$U = 1 - \frac{\Delta u_z}{\Delta u_o} = 1 - \sum_{m=0}^{\infty} \frac{2}{M^2} \exp(-M^2 T_v)$$

Relationship between

T_v* and *U



Relationship between T_v and U

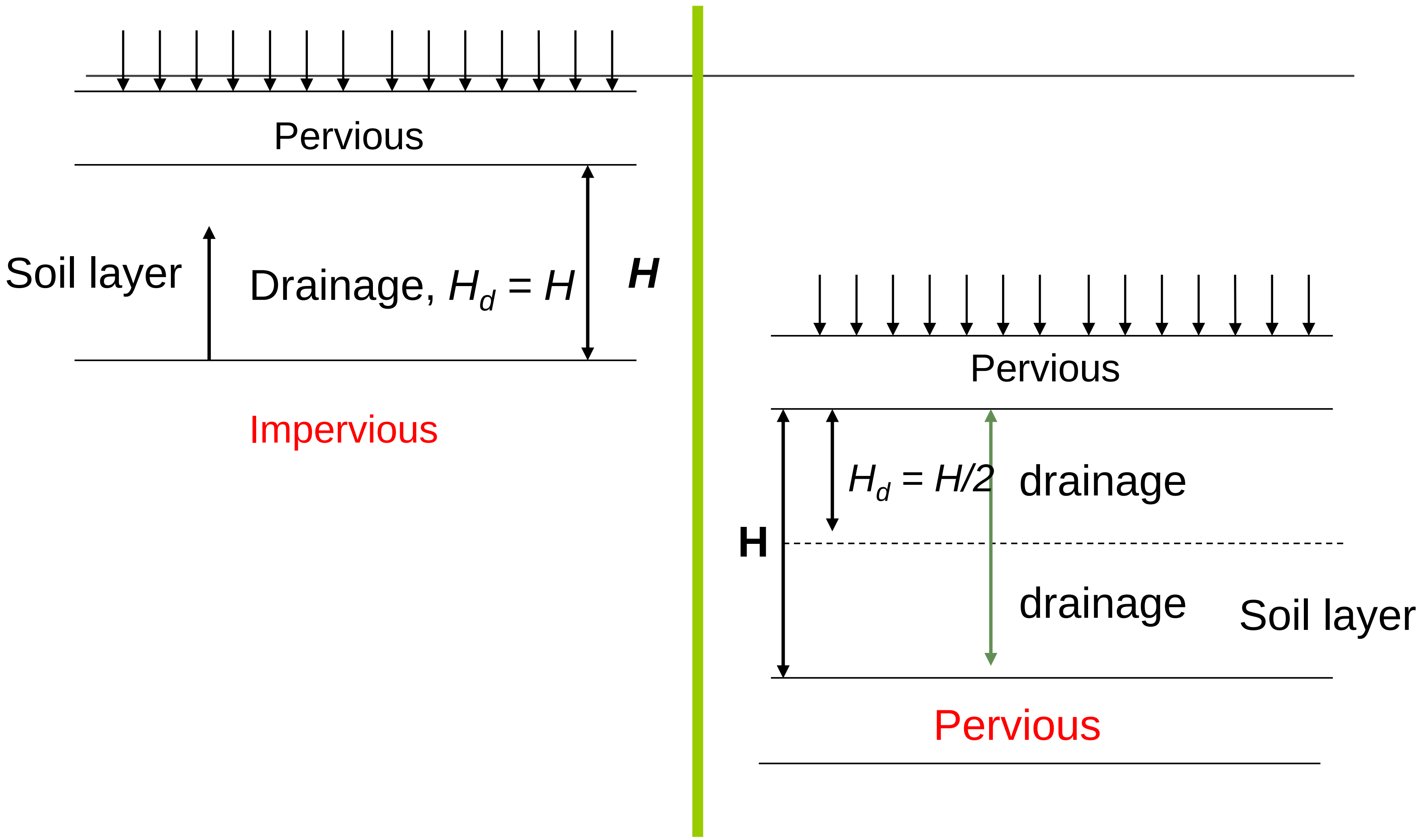
Consolidation Percentage	U average	T_v Double drainage
10%	0.1	0.008
20%	0.2	0.031
30%	0.3	0.071
40%	0.4	0.126
50%	0.5	0.196
60%	0.6	0.287
70%	0.7	0.403
80%	0.8	0.567
90%	0.9	0.848
95%	0.95	1.136
100%	1.0	α

Cassagrande: $U < 60\% \rightarrow T_v = (\pi/4)\, U^2$

$U > 60\% \rightarrow T_v = -0.933 \log(1-U) - 0.085$

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

Single drainage vs Double drainage



Secondary Compression (Creep)

Secondary compression *is a form of soil creep that is largely controlled by the rate at which the skeleton of compressible soils (clay, silt, and peat) can yield and compress (**Time dependent**)*

General perceptions

- *Secondary* compression *occurs at a* ***slower rate*** *than primary consolidation*
- *Usually occur* ***after*** *the primary consolidation, but can also start during primary consolidation.*
- ***Difficult to separate*** *secondary* ***compression*** *from the primary one especially for thick layer*

Evaluation of Secondary Compression according to extension of Cassagrande Method (C_α/C_c concept)

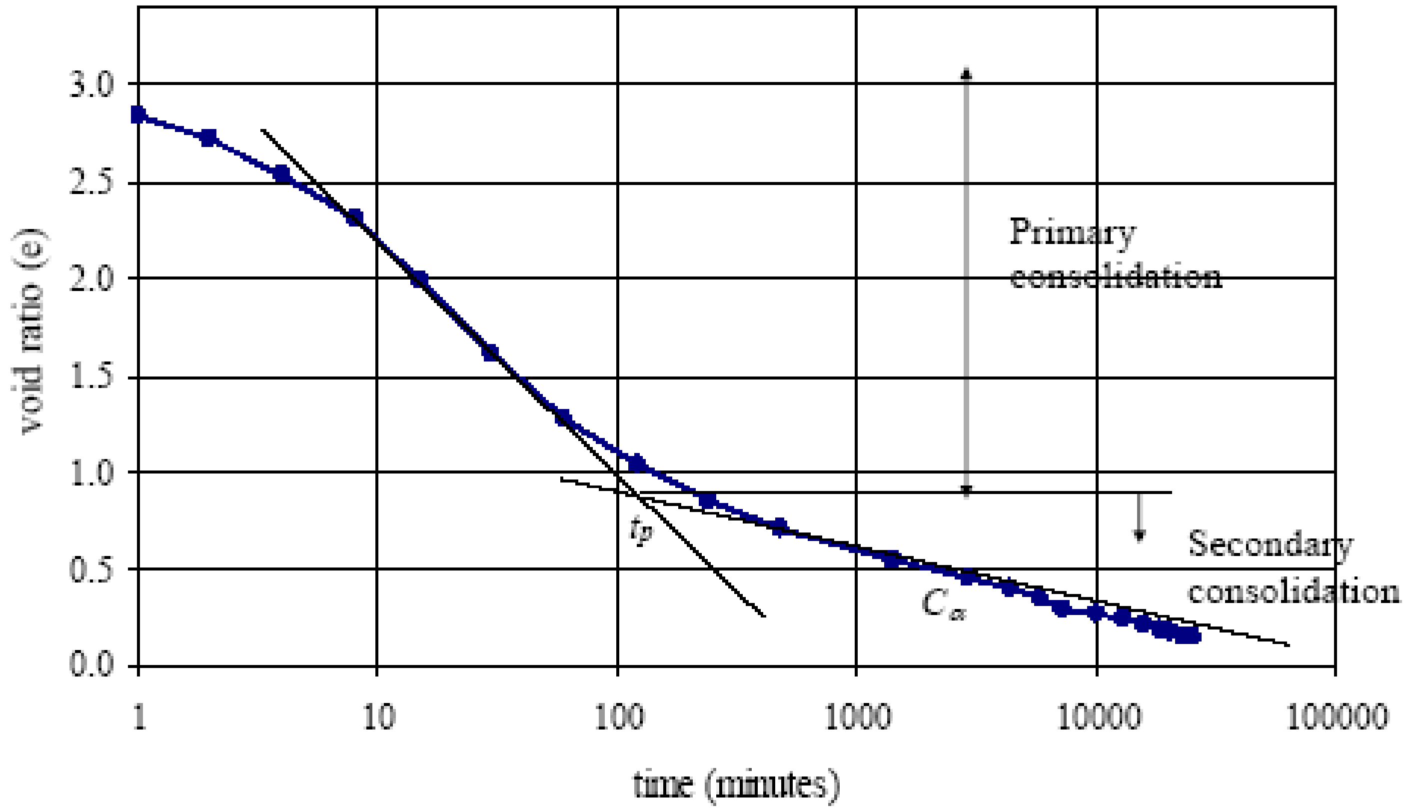


Figure 4.15 Void ratio vs time for primary and secondary settlement

Change of void ratio Δe from time t_p to t is given as:

$$\Delta e_s = C_\alpha \log\left(\frac{t}{t_s}\right)$$

The secondary compression axial strain ε_σ corresponding to Δe is

$$\varepsilon_s = C_{\alpha\varepsilon} \log\left(\frac{t}{t_s}\right)$$

In these cases, C_α is the secondary compression index which is analogous to the C_c while $C_{\alpha\varepsilon}$ is the modified secondary compression index, thus:

$$C_{\alpha e} = \frac{C_\alpha}{1+e_{op}}$$

The secondary compression settlement is

$$s_s = H_o \frac{C_\alpha}{1+e_{op}} \log\left[\frac{t}{t_p}\right]$$

where:

H_o : the thickness of layer at the end of primary consolidation

t_p: the beginning of secondary consolidation

t : time of interest

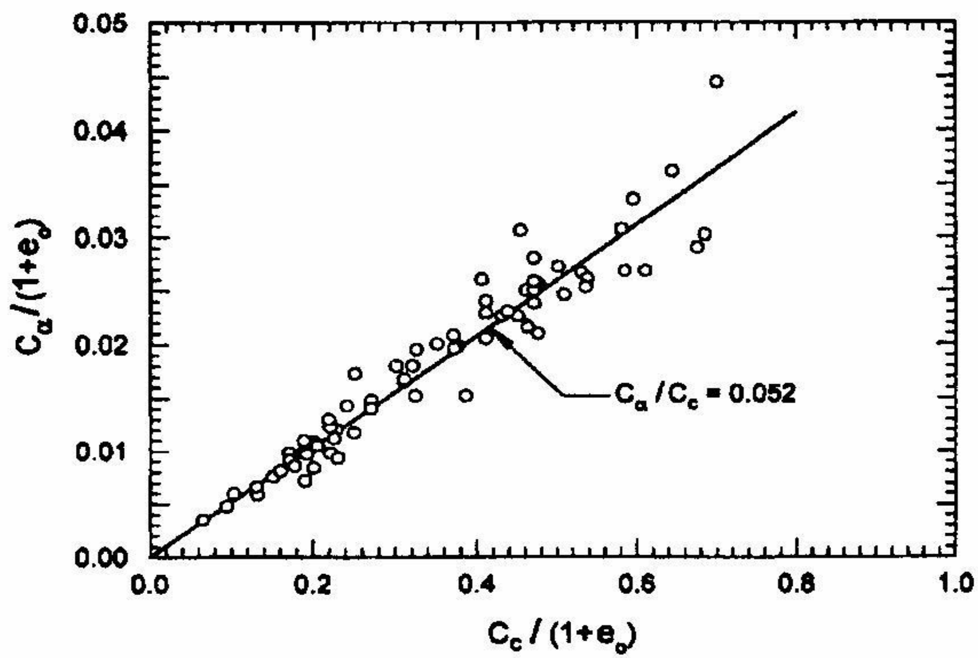


FIG. 10. Relationship between Secondary Compression Index and Compression Index for Middleton Peat

Research Findings:

1. C_α is not independent of time
2. C_α is not independent of the thickness of the soil layer
3. C_α is independent of LIR, as long as the primary consolidation occurs
4. The ratio C_α/C_c is approximately constant for many NC soils over the normal range of engineering stresses (Holtz and Kovacs, 1981).

TABLE 2—*Degree of consolidation (%) at which the secondary compression begins.*

Pressure Range (kPa)	Load Increment Ratio	Beginning of the Secondary Compression (Degree of consolidation %)			
		Kaolinite	Red soil	Silty Soil	Peat
25–75	2	98.5	99	97	86
75–150	1	92.5	97	93	73
150–225	0.5	70	69	75	64
225–300	0.33	65	59	63	62
300–375	0.25	59	58	56	60
375–450	0.20	56	57	54	59

Thank you