SOIL MECHANICS

“First Part”

Third YEAR CIVIL

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helo

Course Contents:

Vertical STRESSES

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Vertical Stresses in a Soil Mass

INTRODUCTION

- Overburden stresses.
- Vertical Stress increase due to foundation loads
- Two-dimensional Problems
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- Three-dimensional Problems
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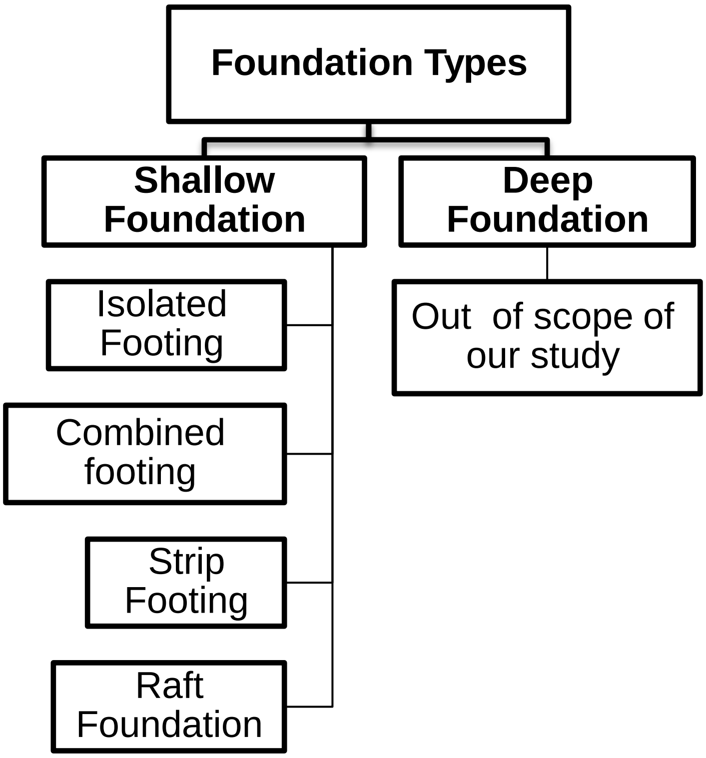
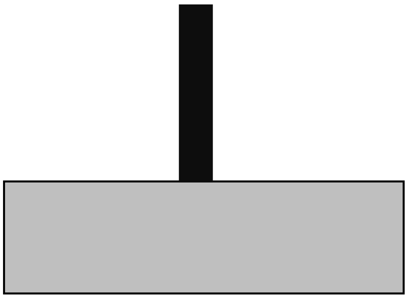
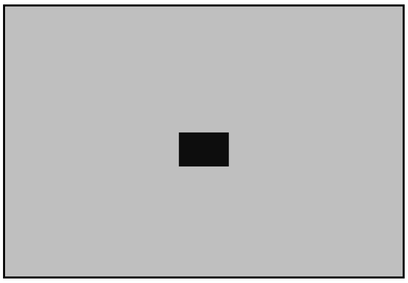
    - Stress due to a circularly loaded area
    - Stress below a rectangular area
    - Stress due to triangular load on rectangular area
- Newmark's chart
- Approximate method
- Homework



Types of Shallow Foundations:

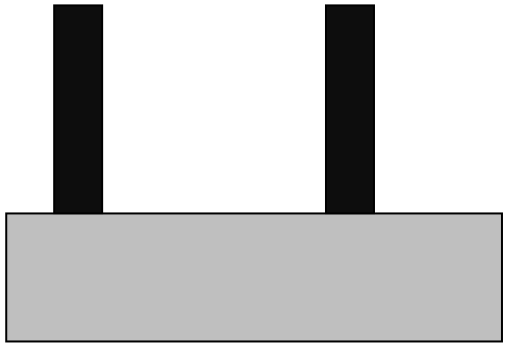


Cross section

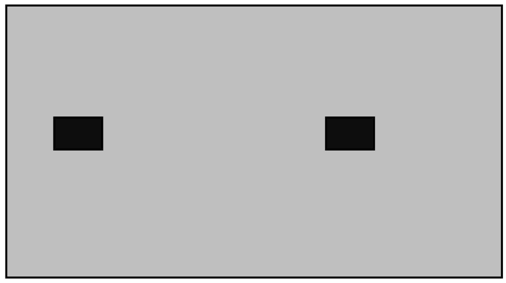


Plan

Isolated footing

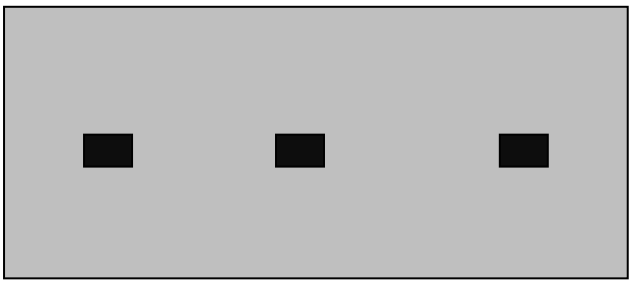


Cross section

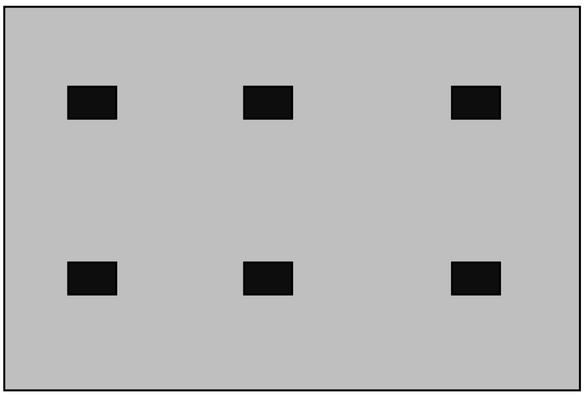


Plan

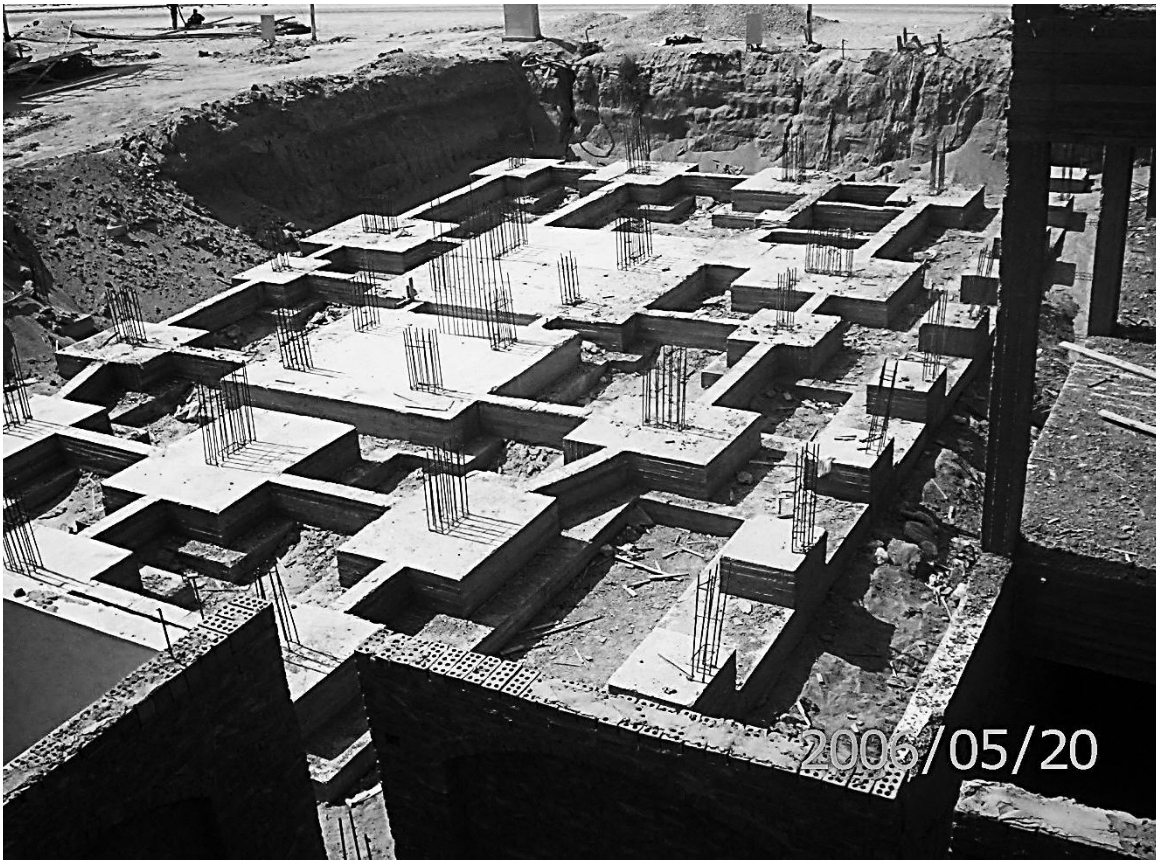
Combined footing



Strip footing



Raft foundation

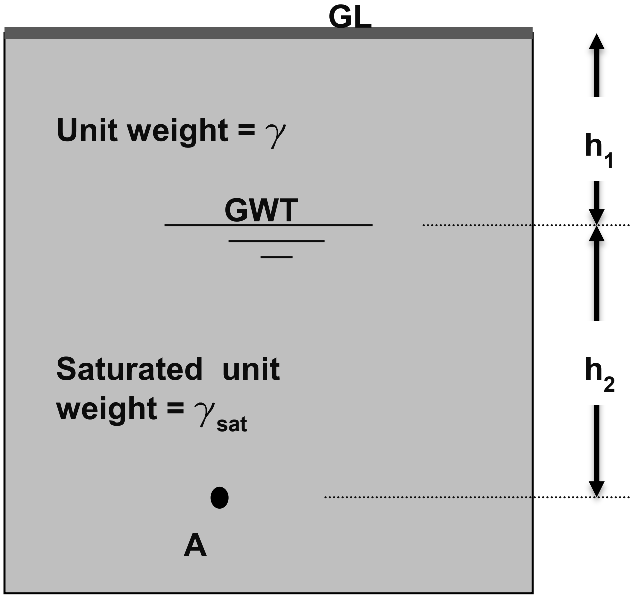


OVERBURDEN STRESSES:

- ➔ Pore water pressure
- ➔ Effective stresses
- ➔ Total stresses

Total vertical stress

$$\sigma_A = h_1\,\gamma + h_2\,\gamma_{sat}$$



The total vertical stress is carried partially by the pore water in the void spaces and partially by the soil solids at their points of contact

$$\sigma_A = u_A + \sigma'_A$$

- Changes in effective stresses will induce volume changes.
- Effective stress is also responsible for producing frictional resistance in soils and rocks

$$\sigma'_A = \sigma_A - u_A$$

$$\sigma'_A = (h_1\,\gamma + h_2\,\gamma_{sat}) - h_2\,\gamma_w$$

$$\sigma'_A = h_1\,\gamma + h_2\,(\gamma_{sat} - \gamma_w)$$

$$\sigma'_A = h_1\,\gamma + h_2\;\gamma_{sub}$$

Submerged unit weight

EXAMPLE:

Determine the total stress, pore water pressure, and effective vertical stress at A, B, and D.

At D

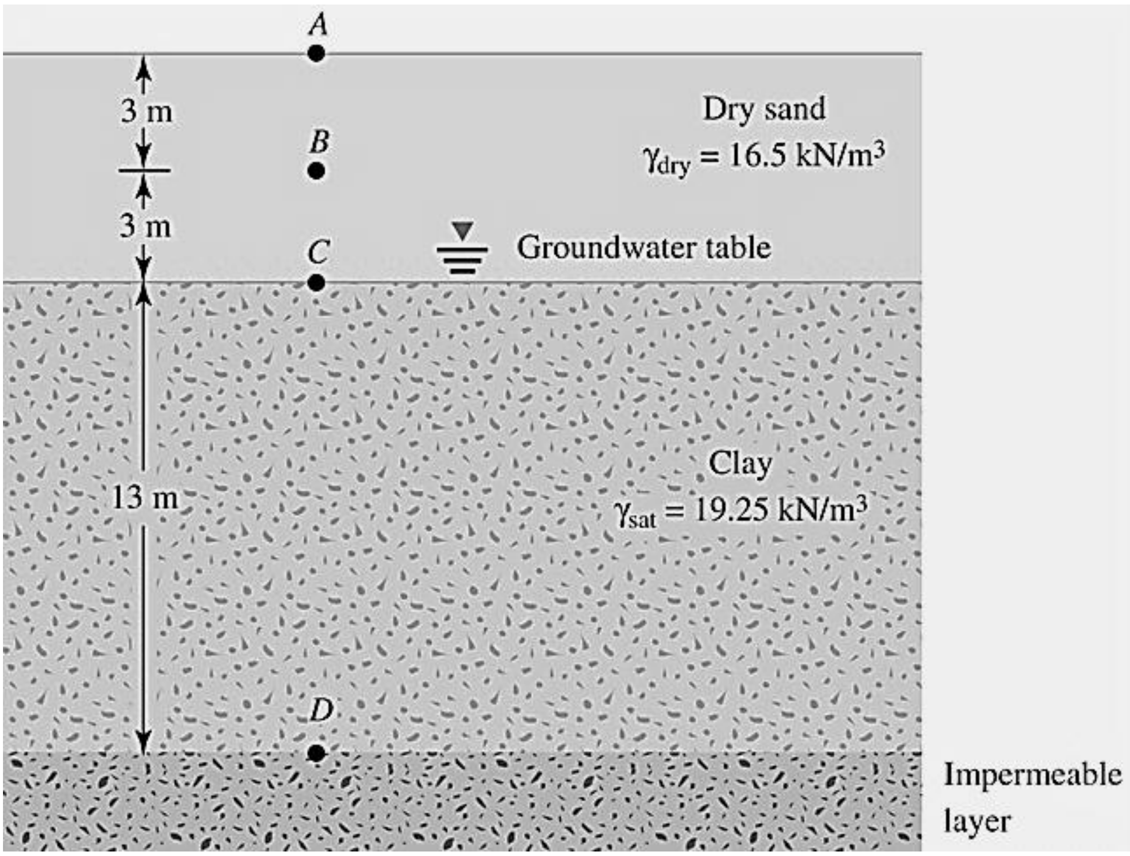
σ_D = 6x16.5+13x19.25

σ_D = 349.25 kN/m2

σ_D = 13x9.81 =127.53 kN/m2

σ_D = 6x16.5+13x(19.2 – 9.81)

σ_D = 6x16.5+13x(19.2 – 9.81) = 221.72 kN/m2



Vertical stress increase due to foundation loads:

Assumptions:

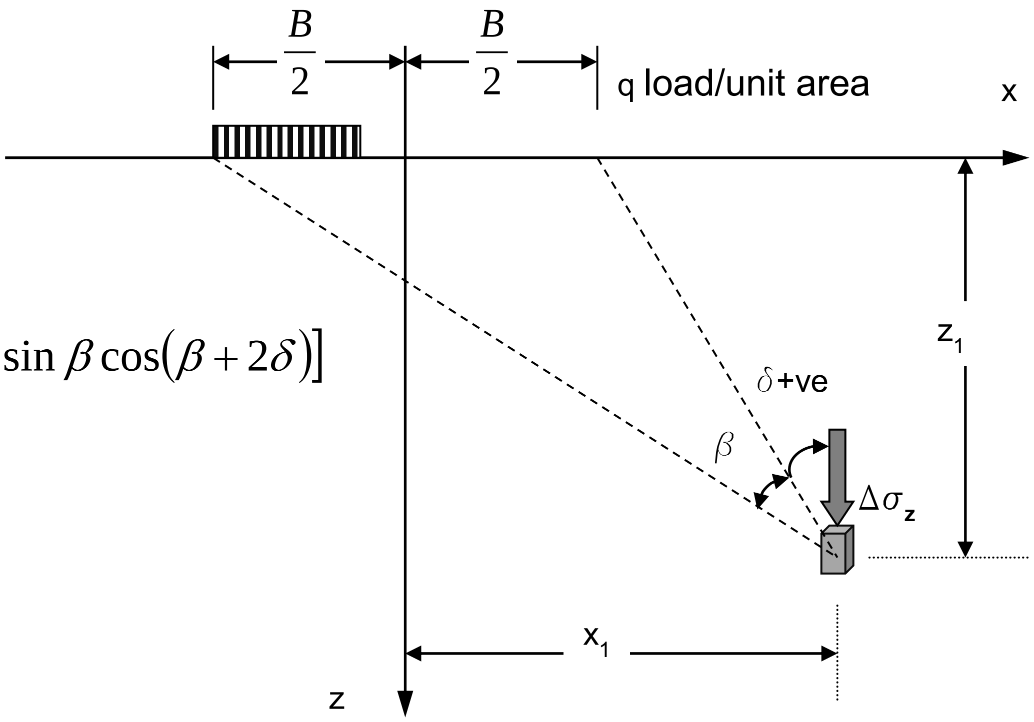
The soil mass is:

1- Elastic
2- Isotropic
3- Homogeneous
4- Semi-infinite medium

- **Rigid footing**
- **Flexible footing**

Two-dimensional problems :

Stress due to strip load

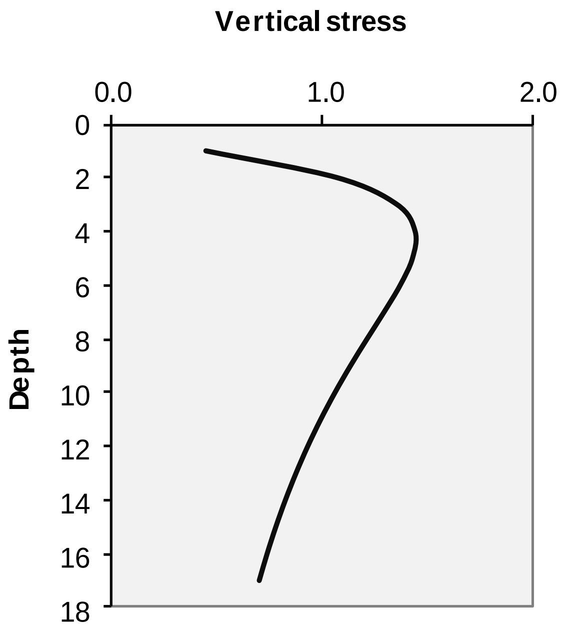


$$\Delta\sigma_z = \frac{q}{\pi}\left[\beta + \sin\beta\cos(\beta + 2\delta)\right]$$

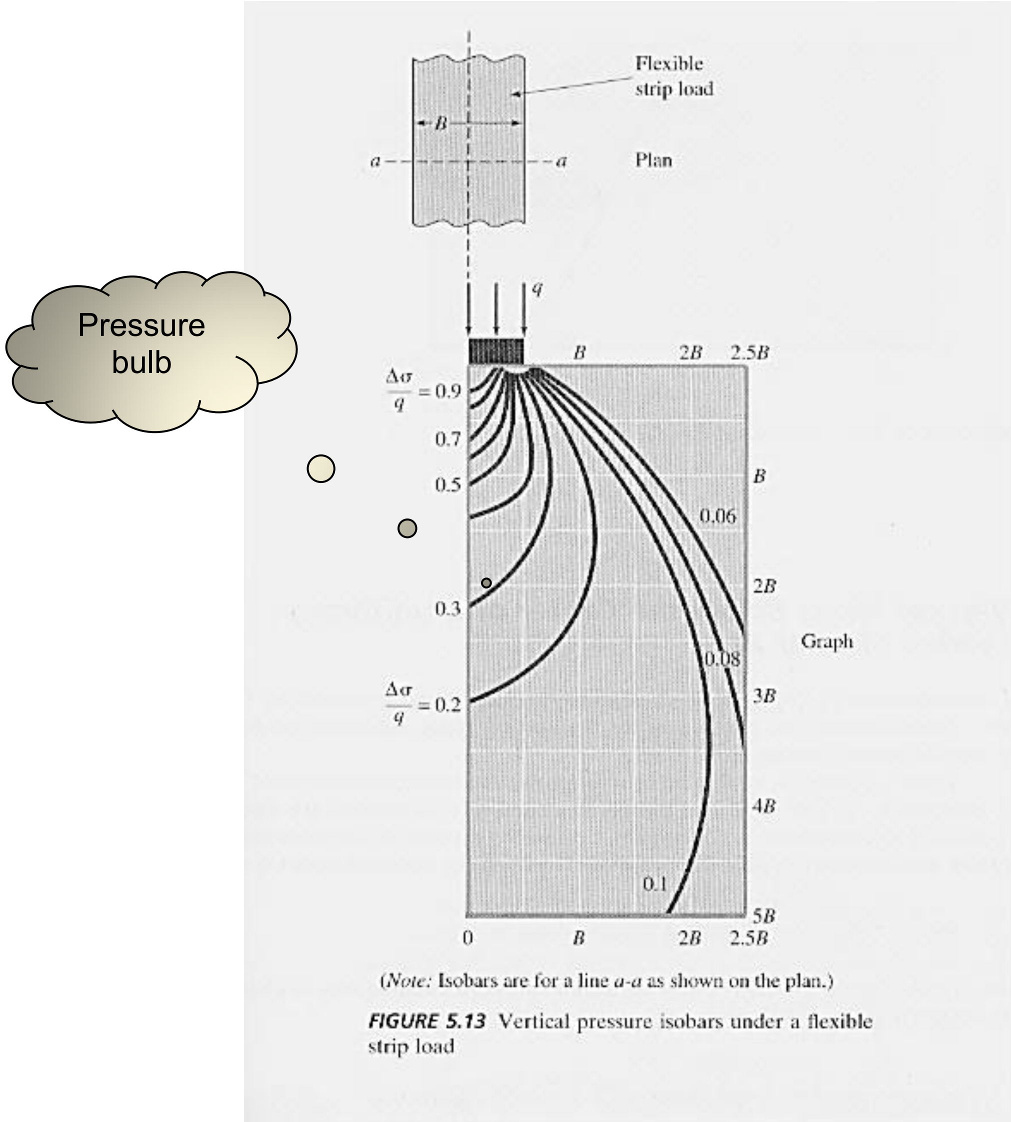
Example using EXCEL:

q t/m^2	B	x_1	z_1	β deg	β rad	δ deg	δ rad	Vertical stress
5	4	3	1	33.70716	0.588003	45.02282	0.785398	0.446353974

Variation of vertical stress with depth at distance 3.0 m from C.L of a strip load



Stress Isobars:



(*Note:* Isobars are for a line *a-a* as shown on the plan.)

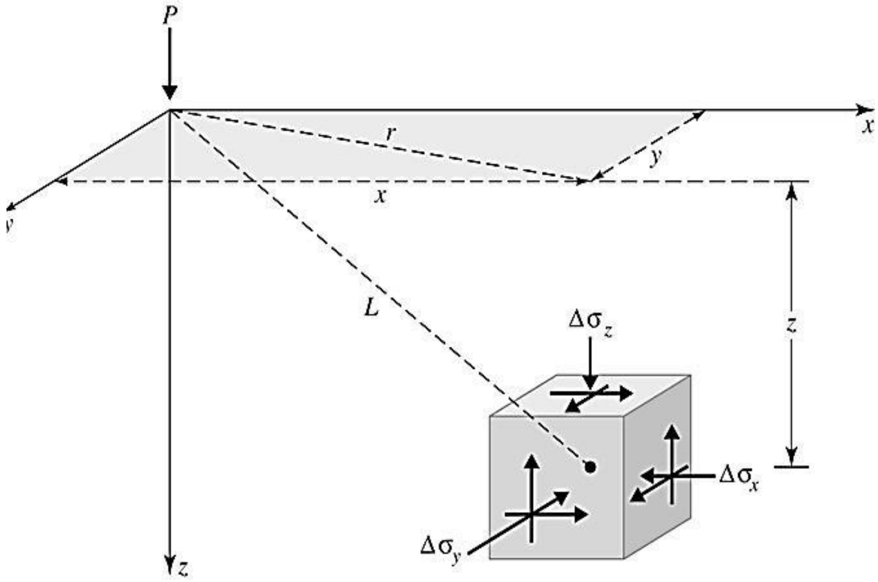
FIGURE 5.13 Vertical pressure isobars under a flexible strip load

Vertical stress isobars under a flexible strip load

Three-dimensional problems :

Stress due to vertical point load:

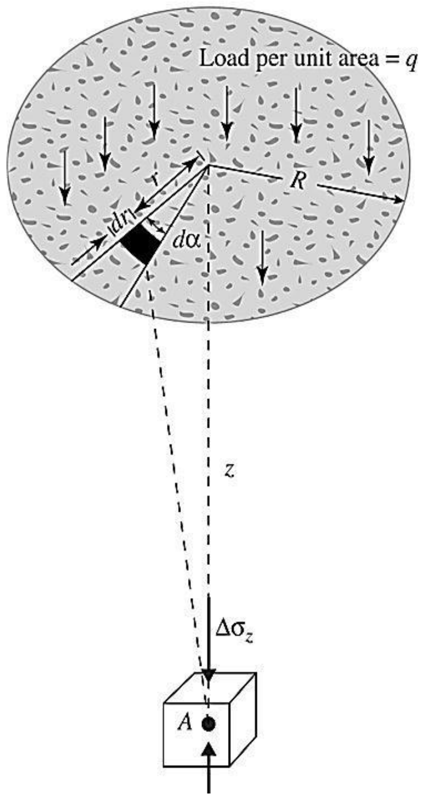
$$\Delta\sigma_z = \frac{3P}{2\pi z^2\left[1+\left(\frac{r}{z}\right)^2\right]^{\frac{5}{2}}}$$

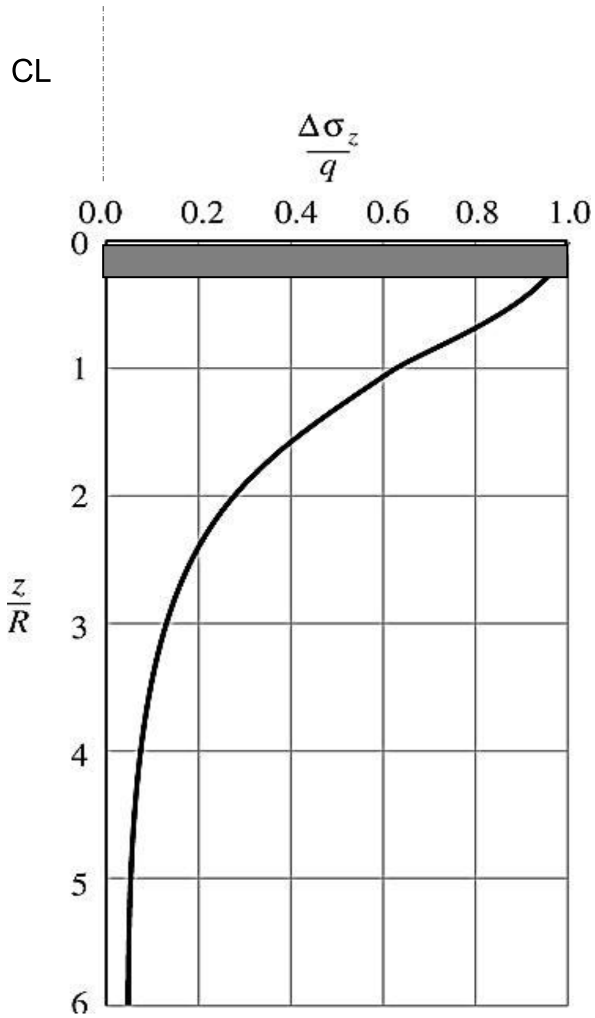


Stress due to a circularly loaded area:

CASE A : *Vertical stress under the center of circular loaded area:*

$$\Delta\sigma_z = q\left\{1-\frac{1}{\left[1+\left(\frac{B}{2z}\right)^2\right]^{\frac{3}{2}}}\right\}$$



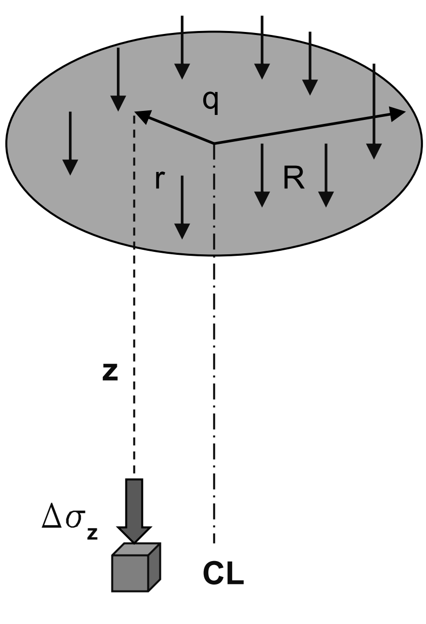


Distribution of stresses under center of

flexible circular loaded area

CASE B : *Vertical stress at any point in the soil:*

Dsz can be obtained using the next table



Variation of $D\sigma_z/q$ for uniformly loaded flexible circular area:

z/R	r/R					
	0.0	0.2	0.4	0.6	0.8	1.0
0.0	1.000	1.000	1. 000	1.000	1.000	1.000
0.1	0.999	0.999	0.998	0.996	0.976	0.484
0.2	0.992	0.991	0.987	0.970	0.890	0.468
0.3	0.976	0.973	0.963	0.922	0.793	0.451
0.4	0.949	0.943	0.920	0.860	0.712	0.435
0.5	0.911	0.902	0.869	0.796	0.646	0.417
0.6	0.864	0.852	0.814	0.732	0.591	0.400
0.7	0.811	0.798	0.756	0.674	0.545	0.367
0.8	0.756	0.743	0.699	0.619	0.504	0.366
0.9	0.701	0.688	0.644	0.570	0.467	0.348
1.0	0.646	0.633	0.591	0.525	0.434	0.332
1.2	0.546	0.535	0.501	0.447	0.377	0.300
1.5	0.424	0.416	0.392	0.355	0.308	0.256
2.0	0.286	0.280	0.268	0.248	0.224	0.196
2.5	0.200	0.197	0.191	0.180	0.167	0.151
3.0	0.146	0.145	0.141	0.135	0.127	0.118
4.0	0.087	0.086	0.085	0.082	0.080	0.075

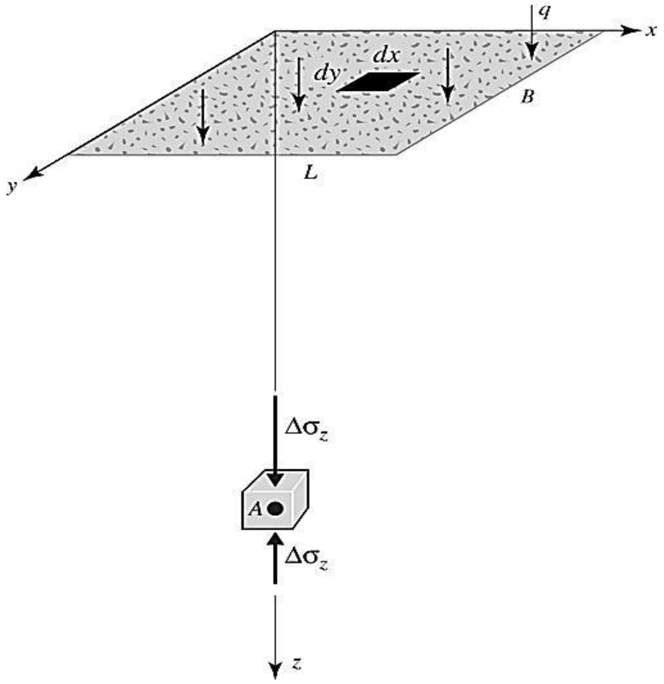
Stress below a rectangular area:

$$\Delta\sigma_z = \int_{y=0}^{L}\int_{x=0}^{B} \frac{3qz^3\,dx\,dy}{2\pi\left(x^2+y^2+z^2\right)^{\frac{5}{2}}}$$

$$\Delta\sigma_z = qI(m,n)$$

$$m = \frac{B}{z}$$

$$n = \frac{L}{z}$$



Variation of I with m and n

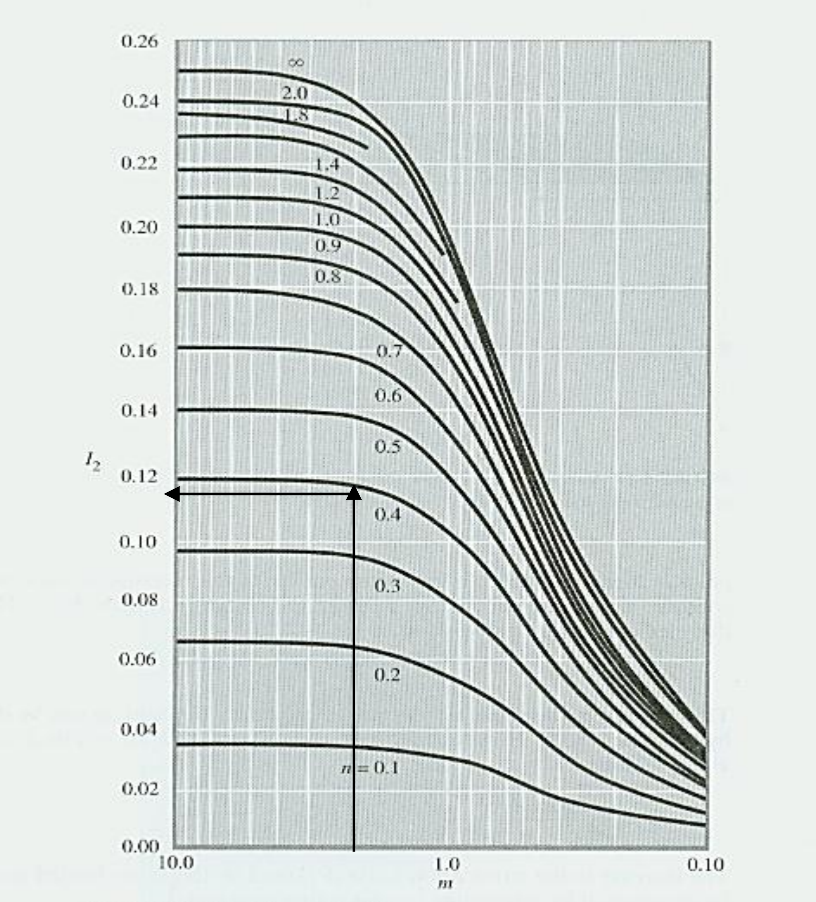
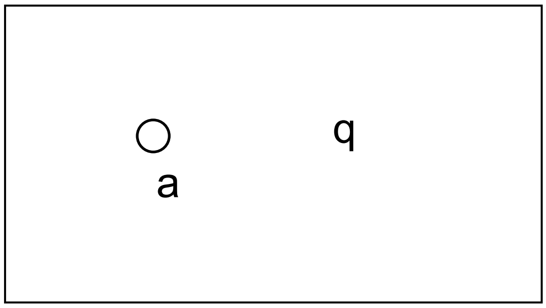
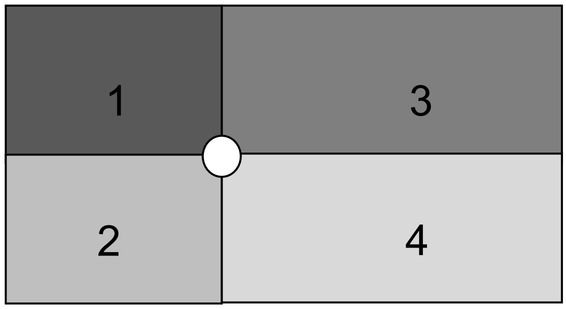


FIGURE 5.18 Variation of I_2 with m and n

Stress below any point of a loaded flexible rectangular area:





$$\Delta\sigma_{z(a)} = q\,(I_1 + I_2 + I_3 + I_4)$$

Stress due to triangular load on rectangular area:

At point E_1

$$\Delta\sigma_{z(E_1)} = \frac{q\,z^3}{2\pi L^2 B}\left\{\frac{L(z^2+k^2)}{z^2 k} + \ln\left[\frac{(n+L)(k-L)}{(n-L)(k+L)}\right] - \frac{L(m^2+n^2)}{m^2 n}\right\}$$

Where:

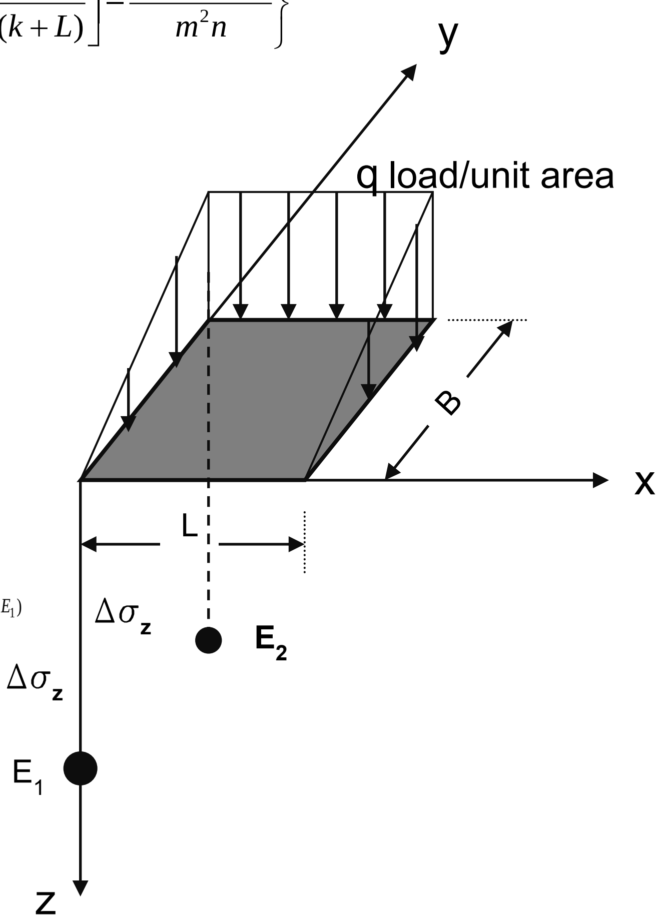
$$k = \sqrt{z^2 + L^2}$$

$$m = \sqrt{z^2 + B^2}$$

$$n = \sqrt{z^2 + B^2 + L^2}$$

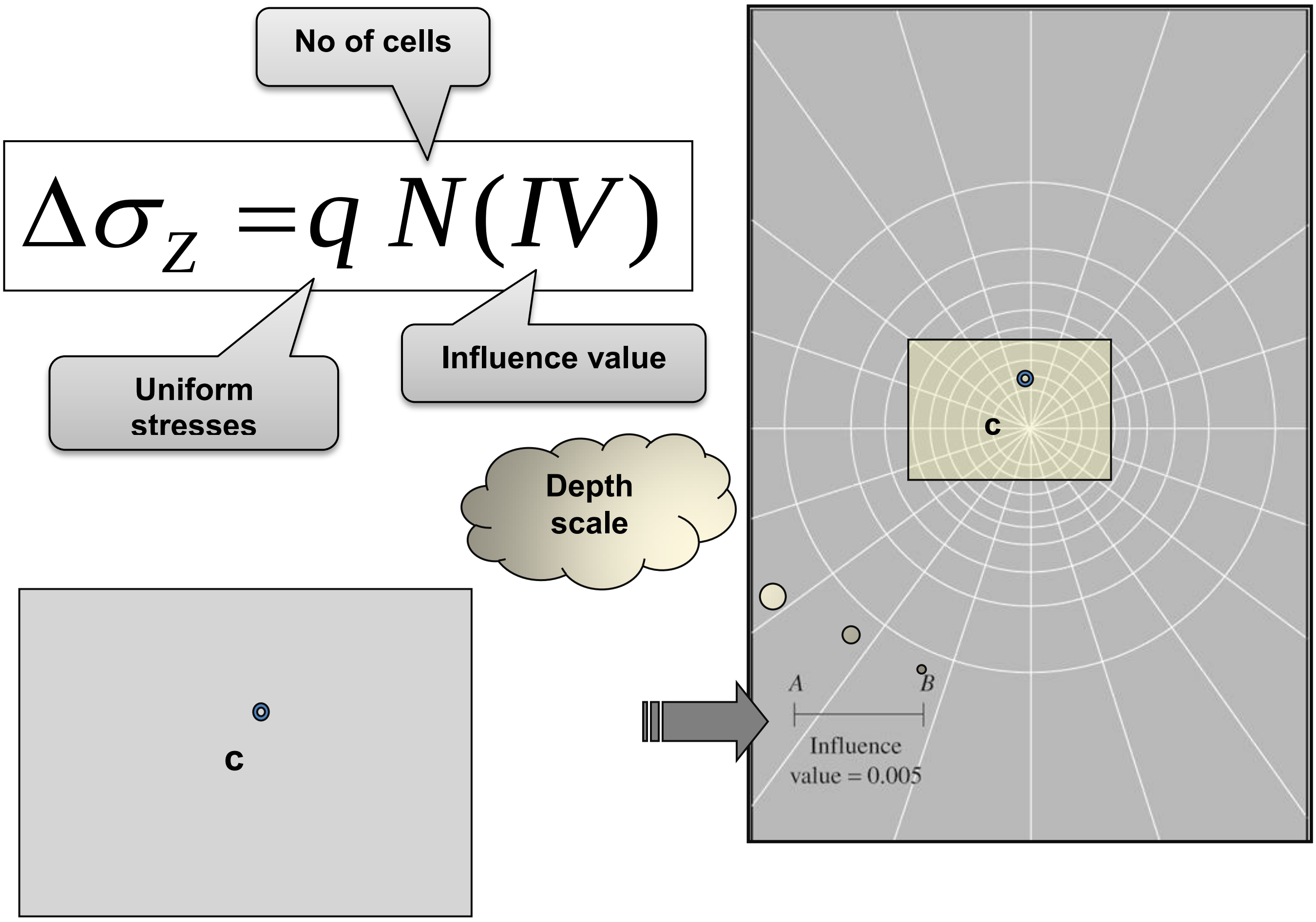
At point E_2

$$\Delta\sigma_{z(E_2)} = \frac{q}{2\pi}\left[\frac{L B z(k^2+m^2)}{k^2 m^2 n^2} + \frac{\pi}{2} - \tan^{-1}\left(\frac{z n}{L B}\right)\right] - \Delta\sigma_{z(E_1)}$$

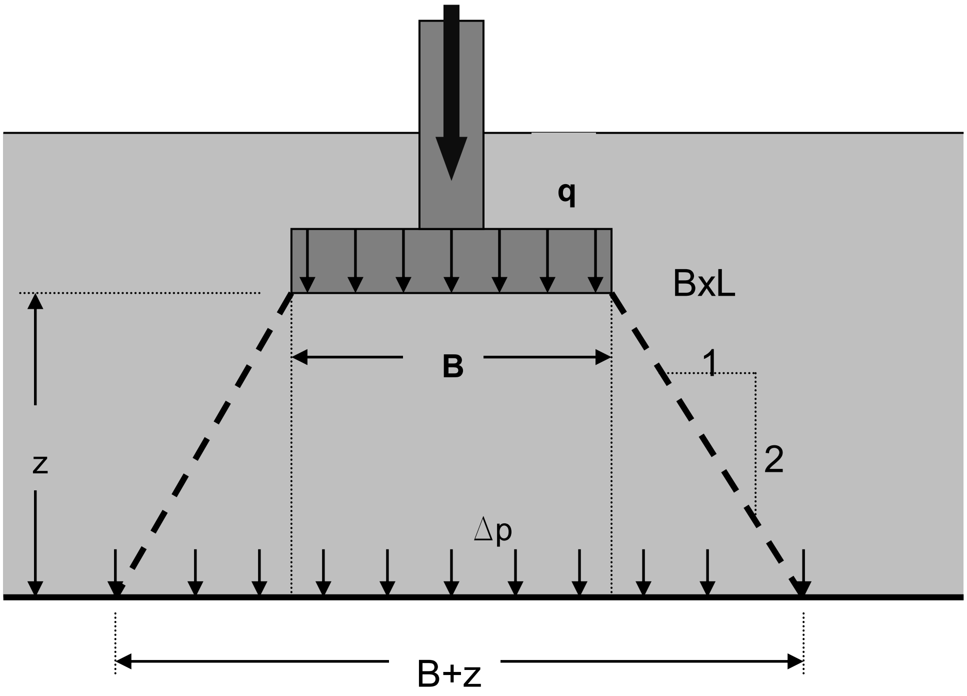


NEWMARK'S CHART:

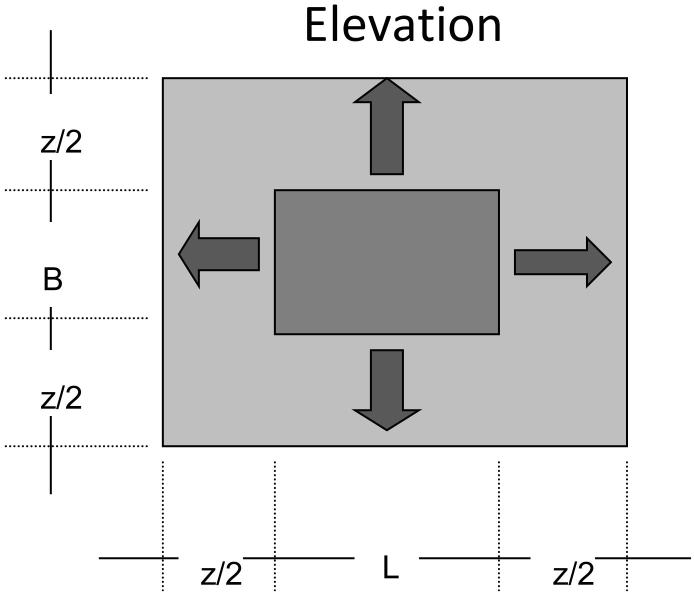
$$\Delta\sigma_Z = q\,N(IV)$$



Approximate (2:1) method:



Elevation



Plan

$$\Delta p = \frac{q\,B\,L}{(B+z)(L+z)}$$

Settlement of Soil

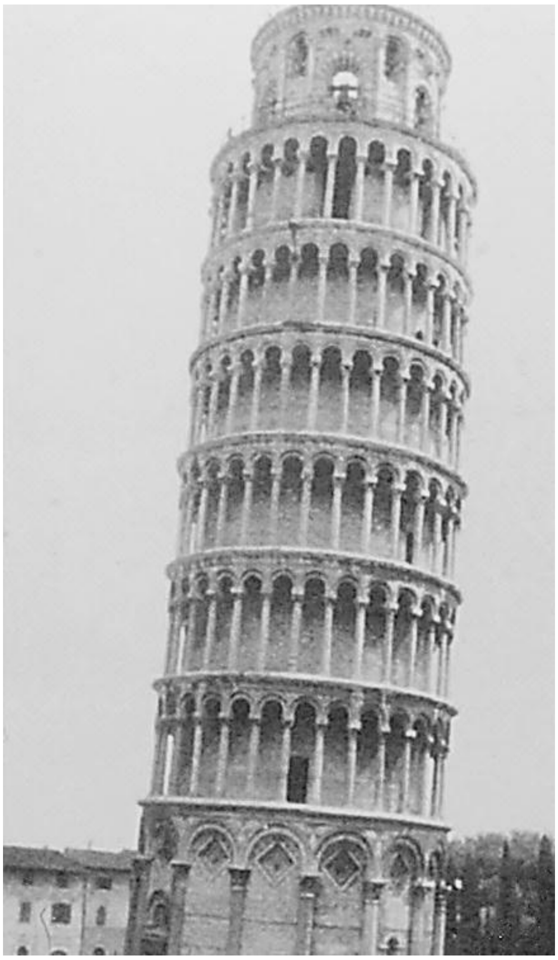
Settlement of Soil

Outline:-

- Introduction
- Types of Foundation Settlement
- Immediate Settlement
- Consolidation Settlement
- Primary Consolidation Settlement
- Secondary Consolidation Settlement

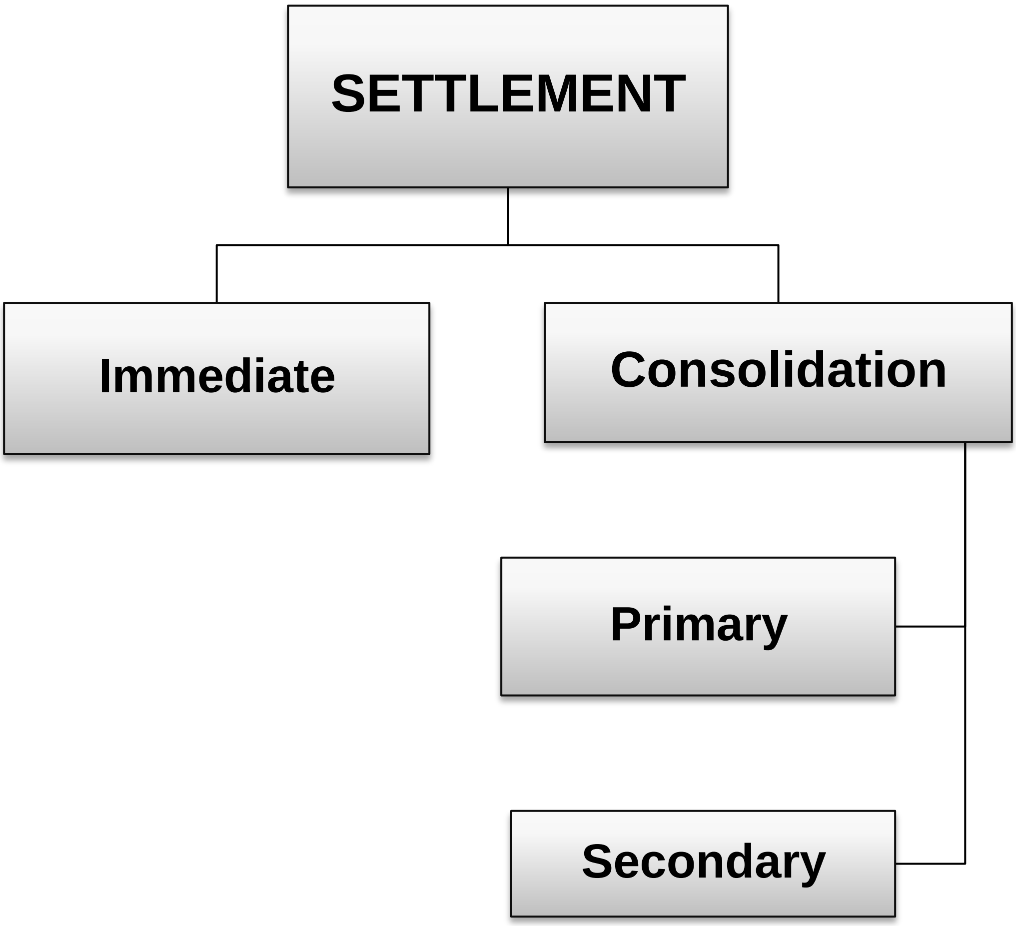
INTRODUCTION:-

- 20 m Circular Base
- 5m Out of Plumb (South)
- Weak Clay Layer at 11m



Leaning tower of Pisa

TYPES OF FOUNDATION SETTLEMENT:-

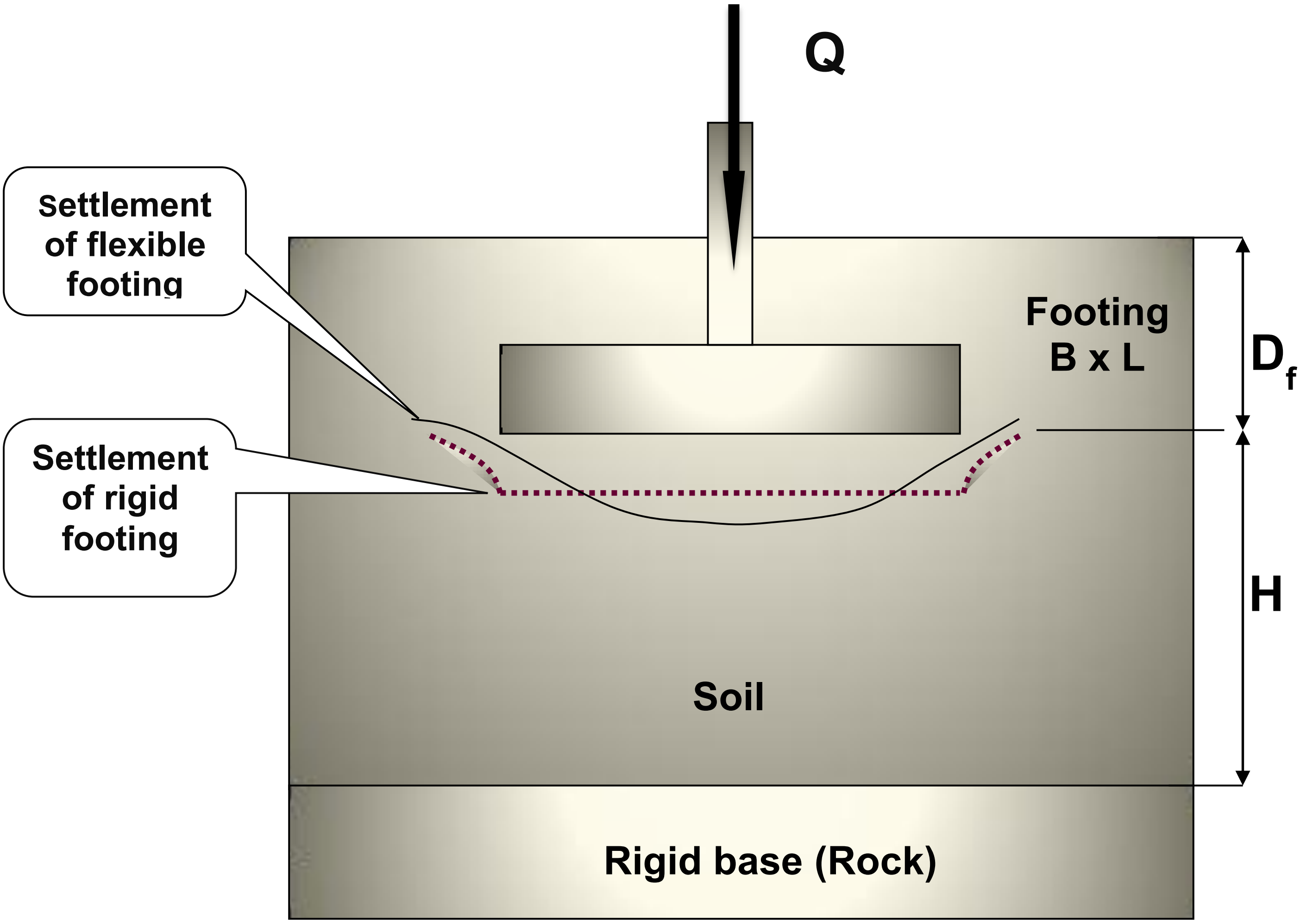


Definitions:-

- **Immediate Settlement** of a foundation: takes place during or immediately after the construction of the structure. (All soil types)
- **Primary Consolidation Settlement:** is the result of volume change of saturated clayey soils due to the expulsion of water occupying the void spaces at load application.
- **Secondary Consolidation Settlement:** occurs at the end of primary consolidation settlement due to the plastic adjustment of soil fabrics.

IMMEDIATE SETTLEMENT (ELASTIC SETTLEMENT):-

Based on the theory of elasticity:



Elastic settlement of flexible and rigid footing

Foundation on soil with infinite depth:-

If the depth of foundation Df = 0, H = ∞

The immediate settlement may be expressed as:

$$S_e = \frac{p\,B}{E_s}(1 - \mu_s^2)\; I$$

Where $p = \frac{Q}{B\,L}$ net stress

E_s = Soil modulus of elasticity

Q = Applied load

μ_s = Soil Poisson's ratio

B = Foundation width

I = Coefficient of shape and rigidity

Coefficient of shape and rigidity (I) for foundation on soil with infinite depth:-

Coef. of shape and rigidity				Shape & Rigidity
Average	**Perimeter of circle Mid of L**	**Corner**	**Center**	
0.85	0.64	--	1.00	Circle - Flexible
0.79	0.79	--	0.79	Circle - Rigid
0.95	0.76	0.56	1.12	Square - Flexible
0.82	0.82	0.82	0.82	Square - Rigid
				Rectangle - Flexible
1.30	1.12	0.76	1.53	L/B = 2
1.82	1.68	1.05	2.10	L/B = 5
2.24	2.1	1.28	2.56	L/B = 10
				Rectangle - Rigid
1.12	1.12	1.12	1.12	L/B = 2
1.60	1.60	1.60	1.60	L/B = 5
2.00	2.00	2.00	2.00	L/B = 10

Foundation on soil "with depth = H":-

Flexible Foundation:

$$S_{e(aver)} = \frac{\sigma}{E_s} H$$

Rigid foundation:

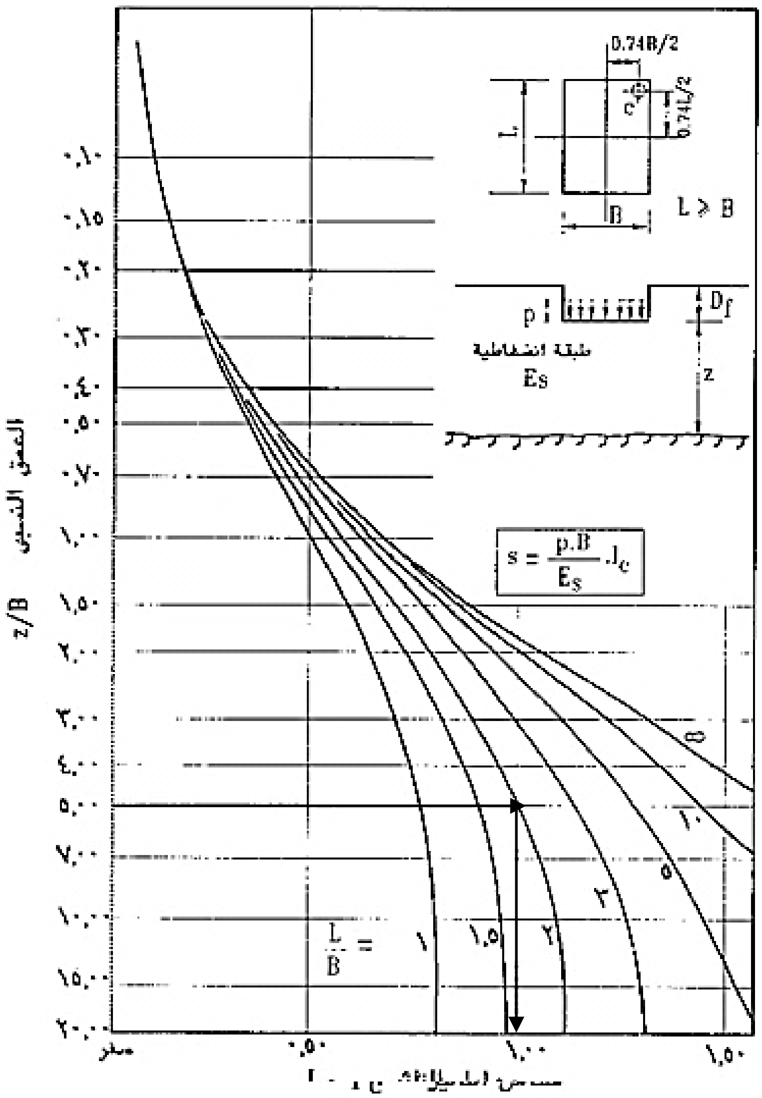
$$S_e = \frac{p\,B}{E_s} I_c$$

Where:

I_c = Influence coefficient

$p = \frac{Q}{B\,L}$ net stress

σ = Stress at mid layer



شكل (٣-٥٧) حساب الهبوط الفورى لأساس
جسىء أسفل النقطة المميزة

Foundation on multi-layered soil:-

$$S_{e(aver)} = \sum \frac{\sigma}{E_s} h_z$$

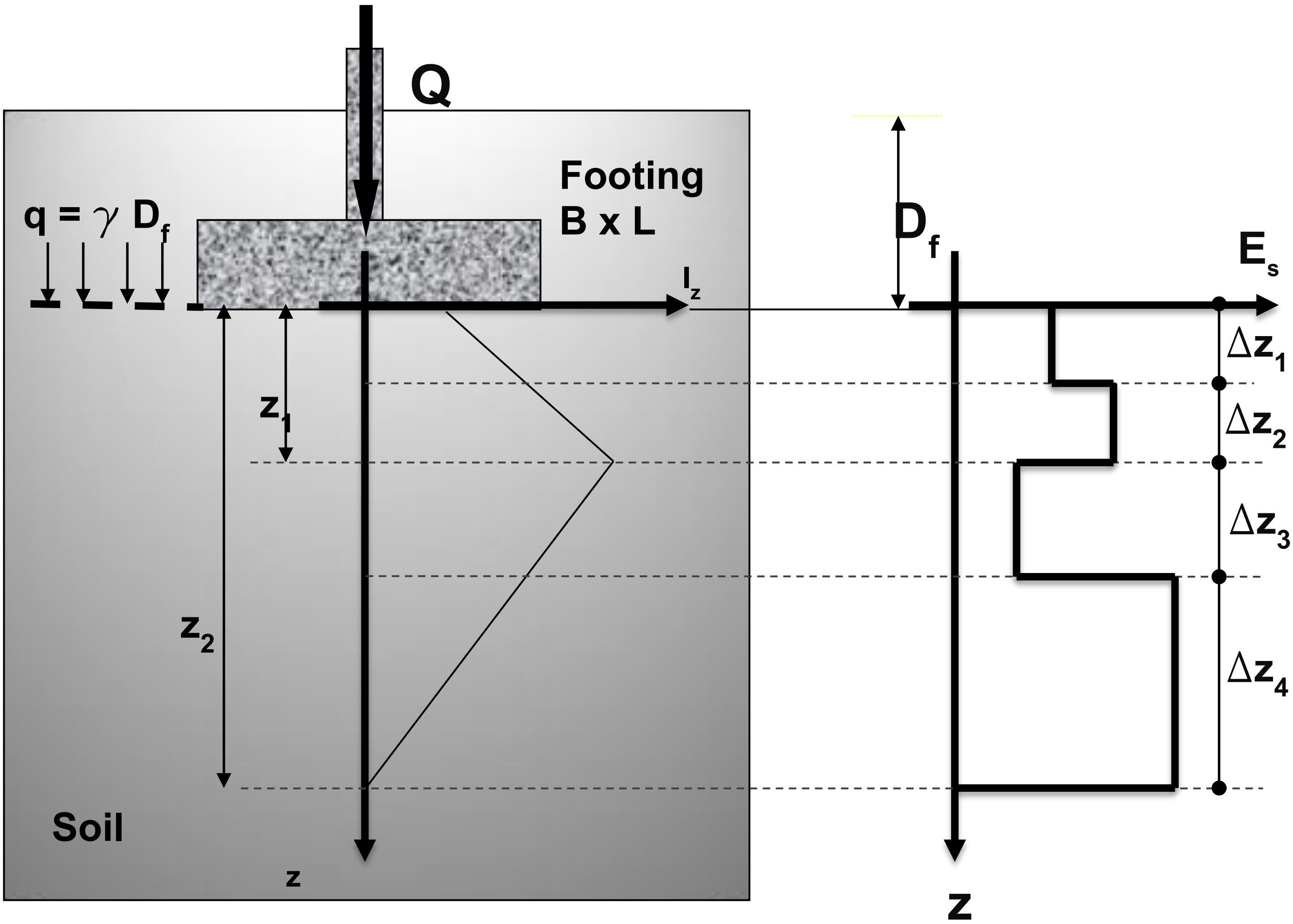
Where:

s = Stress at middle of each layer

h_z= height of each layer

Immediate settlement of sandy soil:

Use of Strain Influence Factor (SIF):



Elastic settlement calculation using SIF

According to the SIF method, the immediate settlement may be calculated as:-

$$S_e = C_1\, C_2\, (q_o - q) \sum_{0}^{z_2} \frac{I_z}{E_s} \Delta z$$

Where:

qo = Gross stress at foundation level

Iz = Strain influence factor

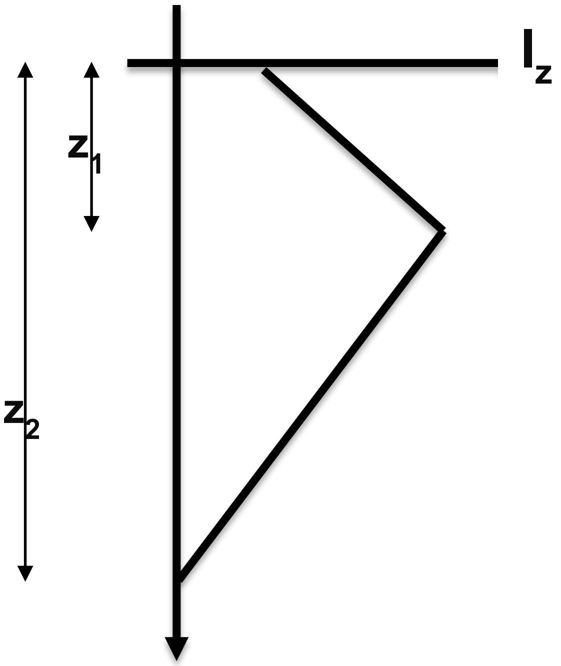
C1 = a correction factor for the depth of foundation

C1 = 1 – 0.5 [q/ (qo - q)]

C2 = a correction factor to account for the creep in soil

C2 = 1 + 0.2 log (10 time in years

- **For Square or circular foundation:**



Iz = 0.1 at z = 0

Iz = 0.5 at z = z1 = 0.5 B

Iz = 0 at z = z2 = 2 B

- **For foundation with L/B ≥ 10**

Iz = 0.2 at z = 0

Iz = 0.5 at z = z1 = B

Iz = 0 at z = z2 = 4 B

The soil modulus of elasticity may be evaluated by using:

- The Standard Penetration Test (SPT)

  Es (kN/m2) = 766 N

- The Static Cone Penetration Resistance

  Es = 2.0 qc

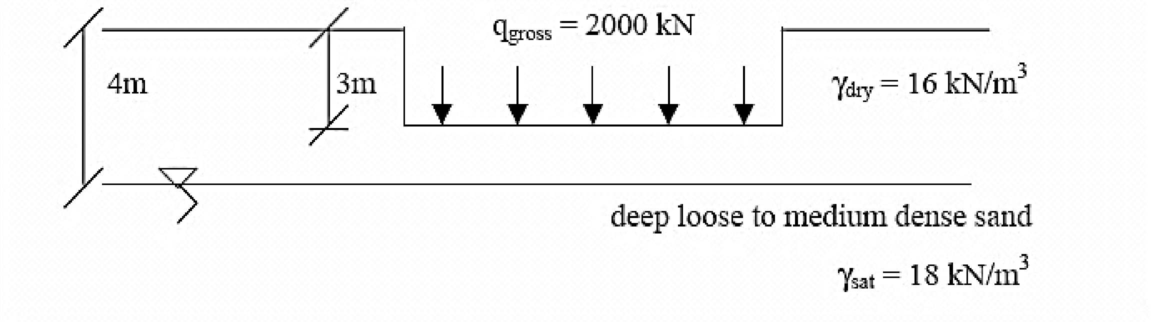
Range of material parameters for computing elastic settlement:

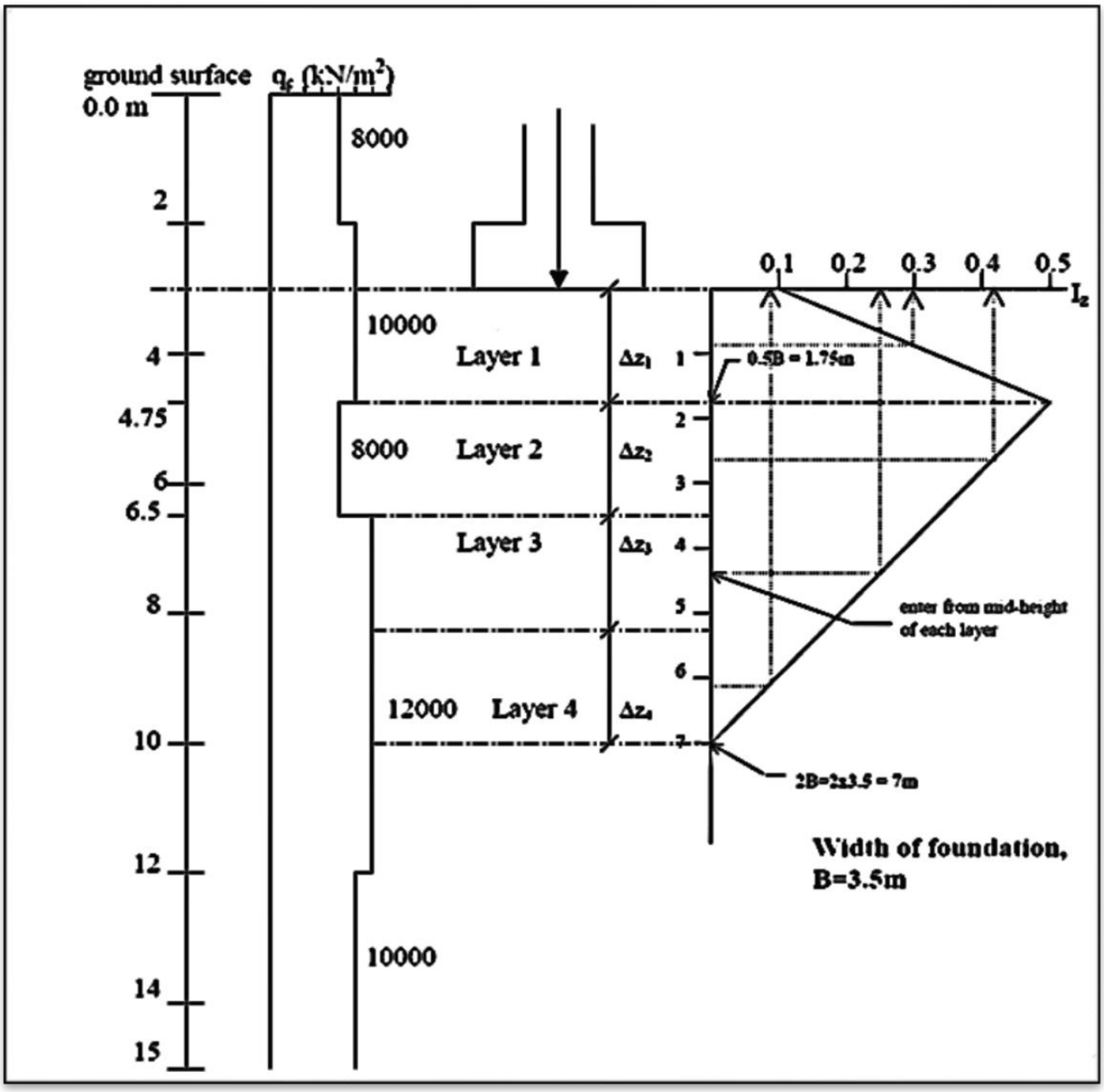
<table>
<tr><th>Type of soil</th><th>E_s MN/m^2</th><th>μ_s</th></tr>
<tr><td>Loose sand</td><td>10.35 - 24.15</td><td>0.2 - 0.4</td></tr>
<tr><td>Medium dense sand</td><td>17.25 - 27.60</td><td>0.25 - 0.4</td></tr>
<tr><td>Dense sand</td><td>34.50 - 55.20</td><td>0.3 - 0.45</td></tr>
<tr><td>Silty sand</td><td>10.35 - 17.25</td><td>0.2 - 0.4</td></tr>
<tr><td>Sand and gravel</td><td>69.00 -72.50</td><td>0.15 - 0.35</td></tr>
<tr><td>Soft clay</td><td>4.10 - 20.70</td><td rowspan="3">0.2 - 0.5</td></tr>
<tr><td>Medium clay</td><td>20.70 - 41.40</td></tr>
<tr><td>Stiff clay</td><td>41.40 - 96.60</td></tr>
</table>

▪ Example:

A soil profile consists of deep, loose to medium dense sand (gdry = 16 kN/m3 , gsat = 18kN/m3). The ground water level is at 4 m depth. A 3.5m x 3.5m square footing rests at 3 m depth. The total (gross) load acting at the foundation level (footing weight + column load + weight of soil or footing) is 2000 kN. Estimate the settlement of the footing 6 years after the construction using the strain influence factor (SIF) method (Schmertman, 1978). End resistance values obtained from static cone penetration tests are;

Depth (m)	q_c (kN/m^2)
0.00 – 2.00	8000
2.00 - 4.75	10000
4.75 - 6.50	8000
6.50 – 12.00	12000
12.00 – 15.00	10000





$$q_{net} = \frac{20000}{3.5x3.5} - 3x16 = 115.26 \text{ kPa}$$

gross pressure. initial effective overburden pressure

$$\sigma_o' = 3x16 = 48 \text{ kPa}$$

$$C_1 = 1 - 0.5\frac{48}{115.26} = 0.792$$

$$C_2 = 1 + 0.2\log\frac{6}{0.1} = 1.356$$

$$S_e = C_1 C_2 (q_o - q) \sum_0^{z_2} \frac{I_z}{E_s} \Delta z$$

Layer No	Depth(m)	Δz(m)	q_c(kPa)	Es(kPa)	I_z	$(I_z/E_s)\,\Delta z$
1	3.00-4.75	1.75	10000	20000	0.3	2.65×10^{-5}
2	4.75-6.50	1.75	8000	16000	0.416	4.55×10^{-5}
3	6.50-8.25	1.75	12000	24000	0.249	1.82×10^{-5}
4	8.25-10.00	1.75	12000	24000	0.083	0.605×10^{-5}
						$\Sigma = 9.625\times10^{-5}$

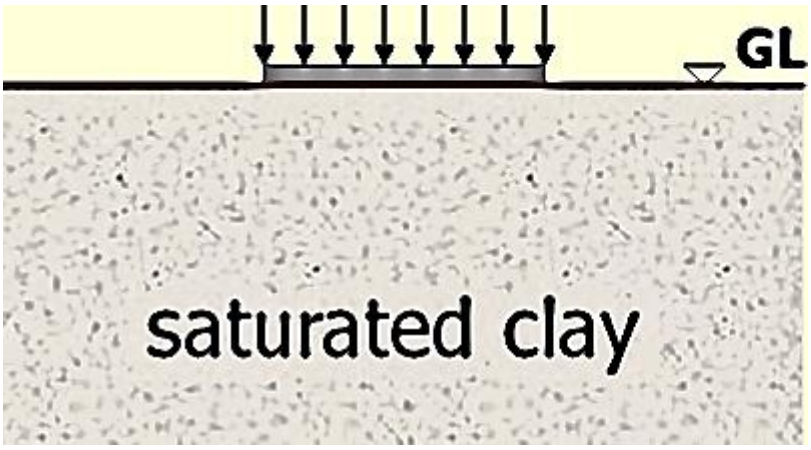
Si = (0.792) (1.356) (115.26) (9.625×10^{-5})

= 0.01191 m ⟶ Si = 11.91 mm

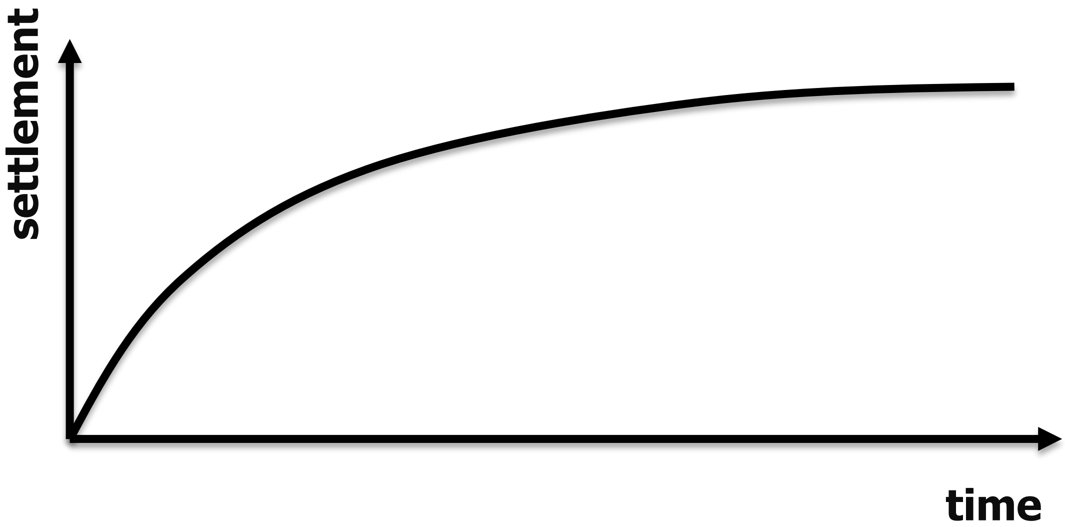
PRIMARY CONSOLIDATION SETTLEMENT:

Fundamental of consolidation:

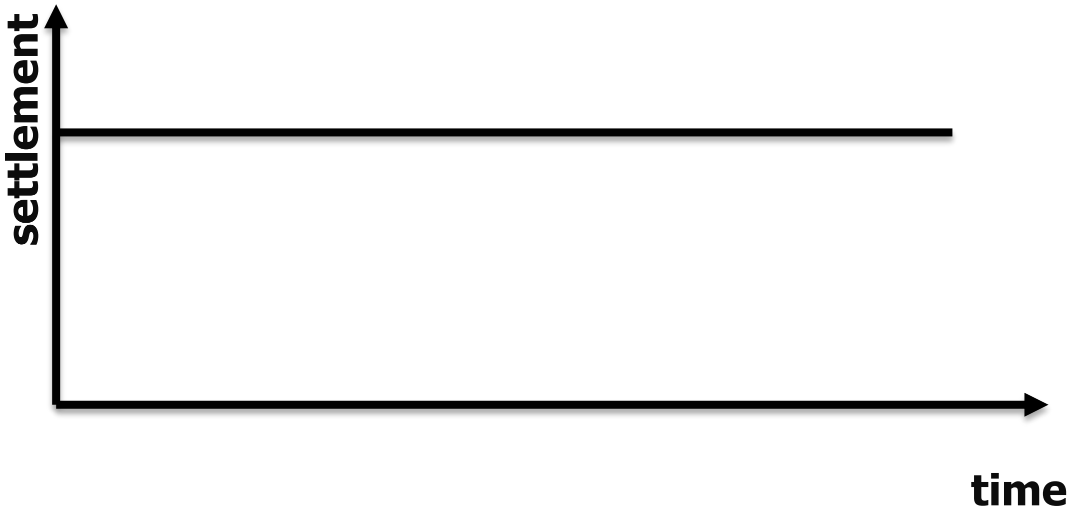
When a saturated clay is loaded externally, The water is squeezed out of the clay over a long time (due to low permeability of the clay).



This leads to settlements occurring over a long time, which could be several years.



Granular soils are freely drained, and thus the settlement is instantaneous.

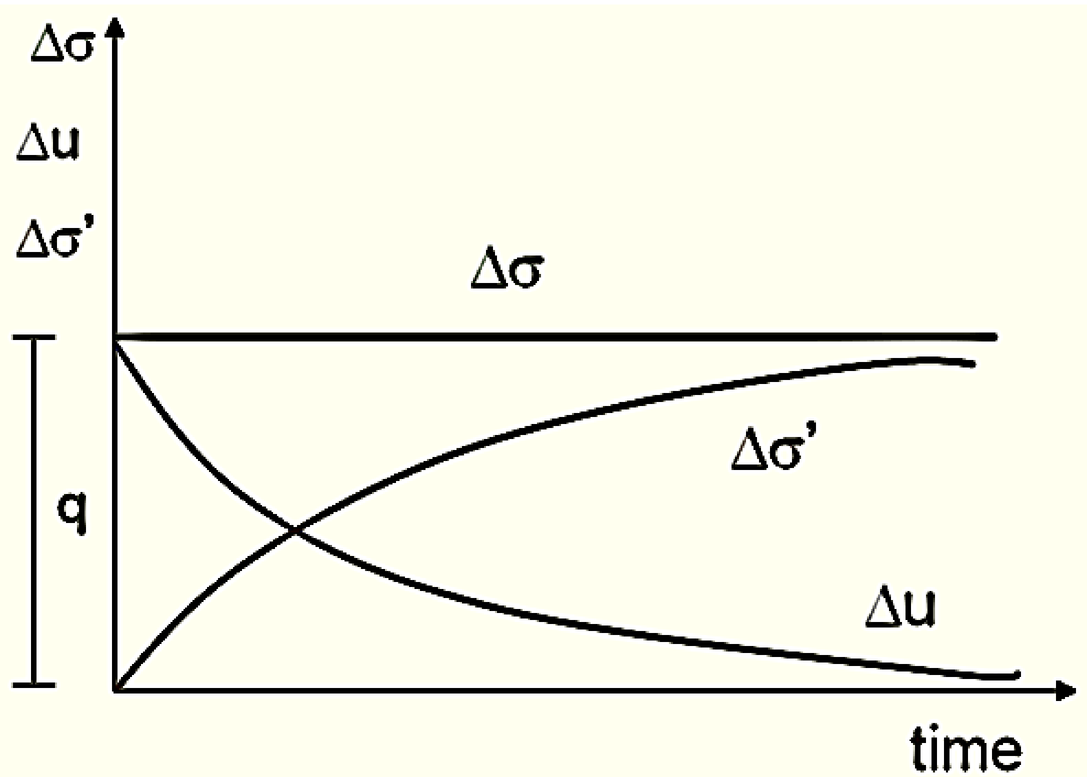


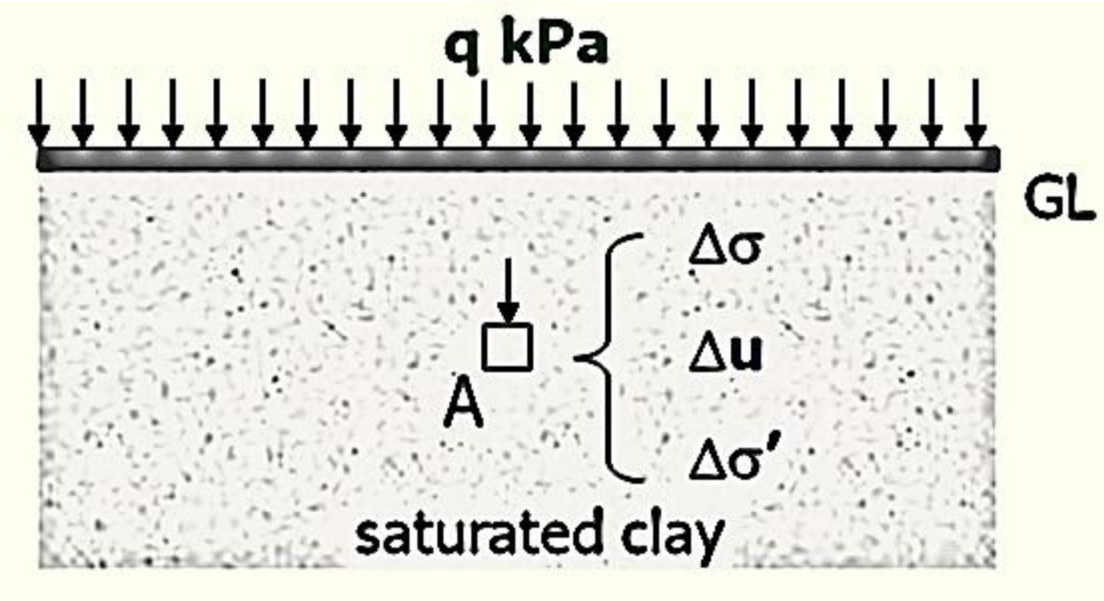
During consolidation

$\Delta\sigma$ remains the same (=q) during consolidation.

Δu decreases (due to drainage) while $\Delta\sigma'$

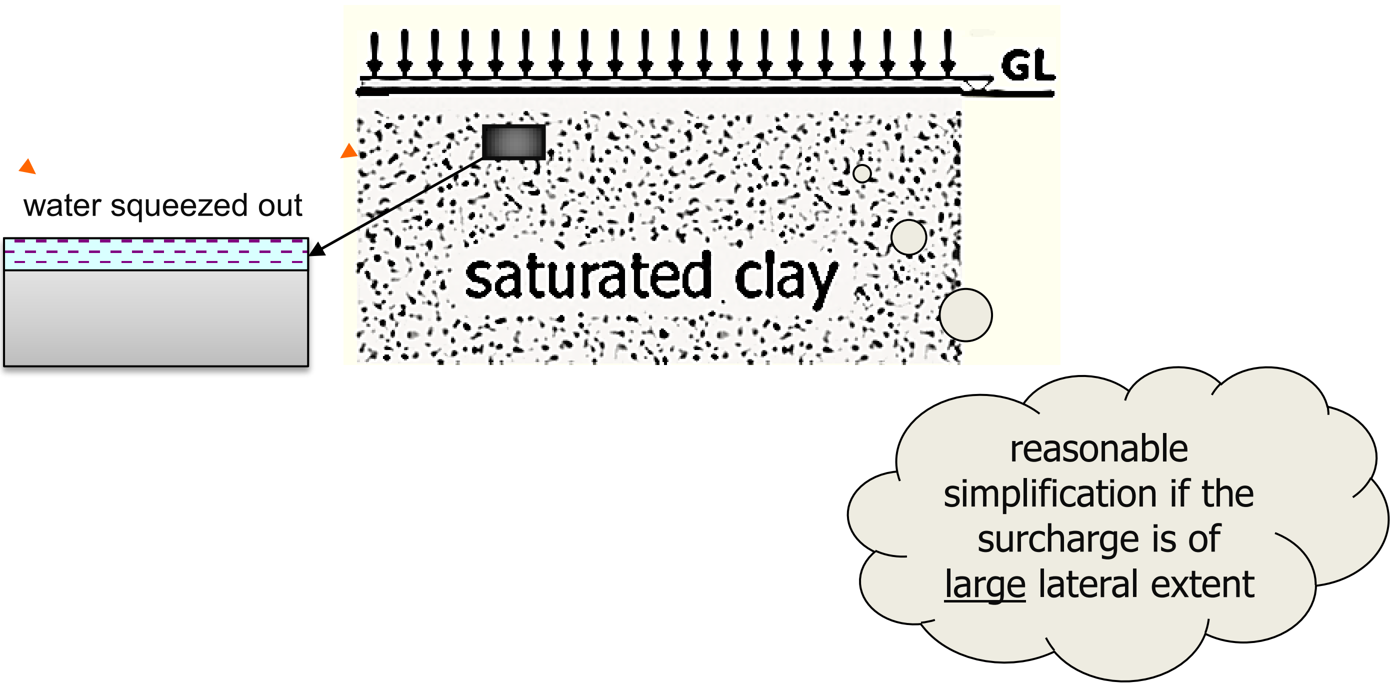
increases, transferring the load from water to the soil.



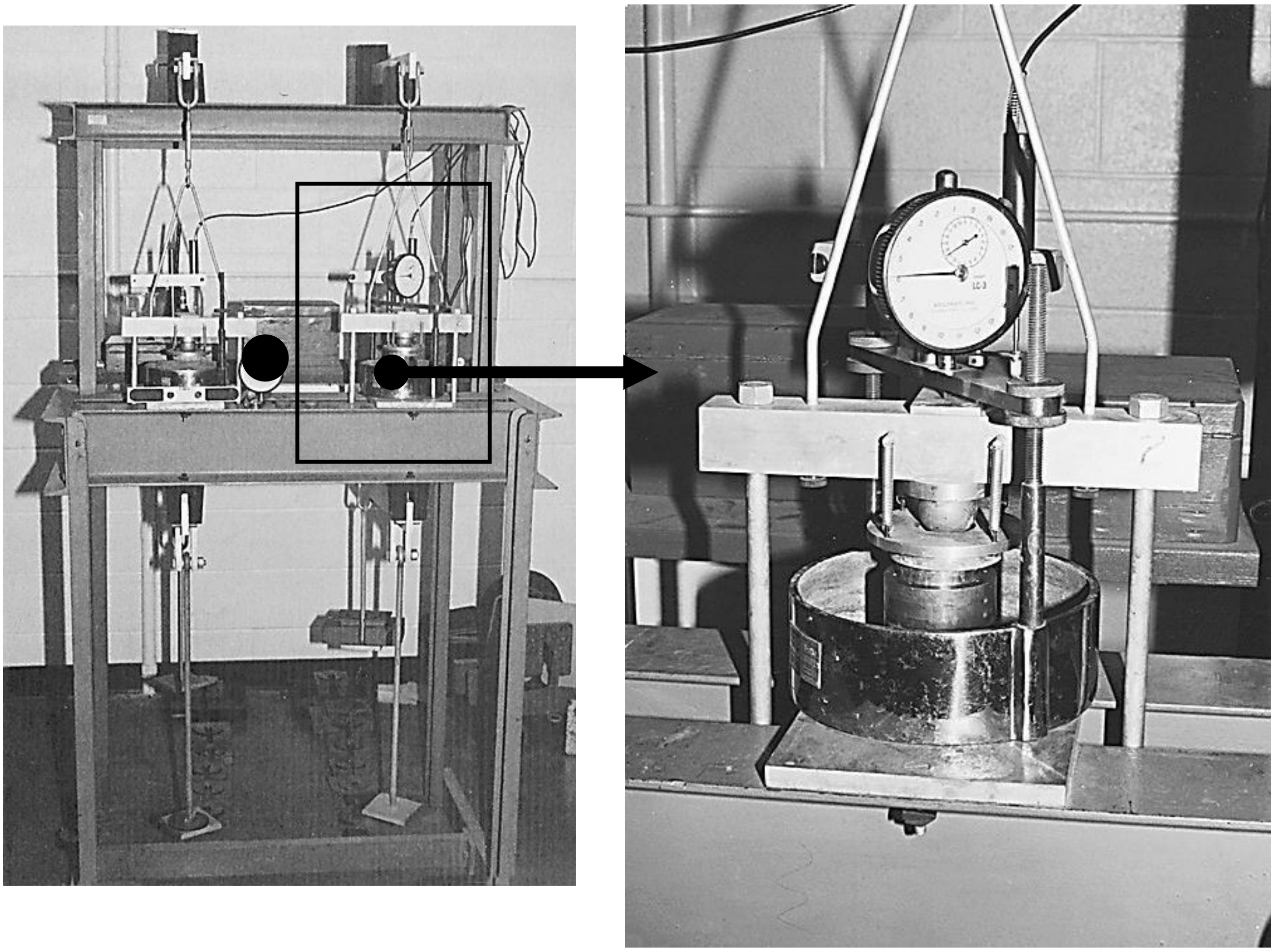


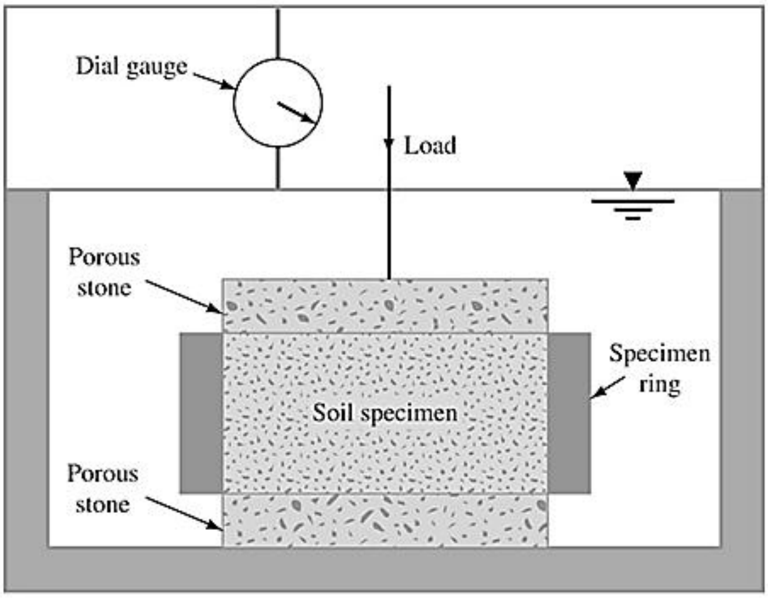
One Dimensional Consolidation:

- Drainage and deformations are vertical
- A simplification for solving consolidation problems

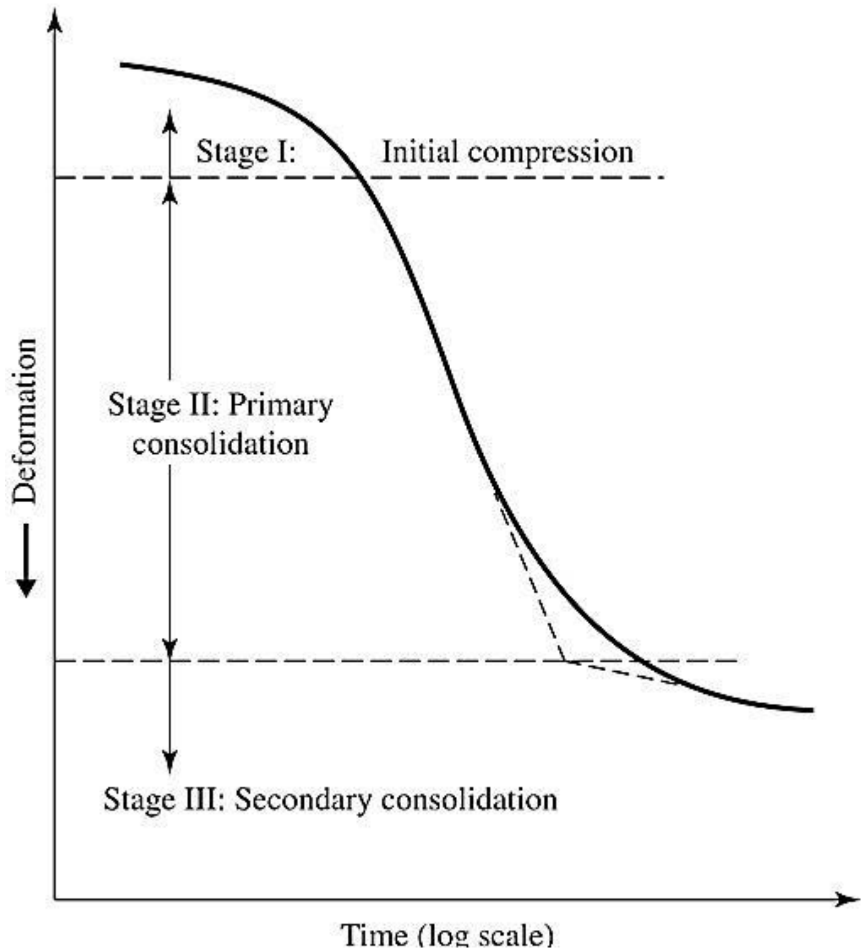


One Dimensional Consolidation Lab Test:



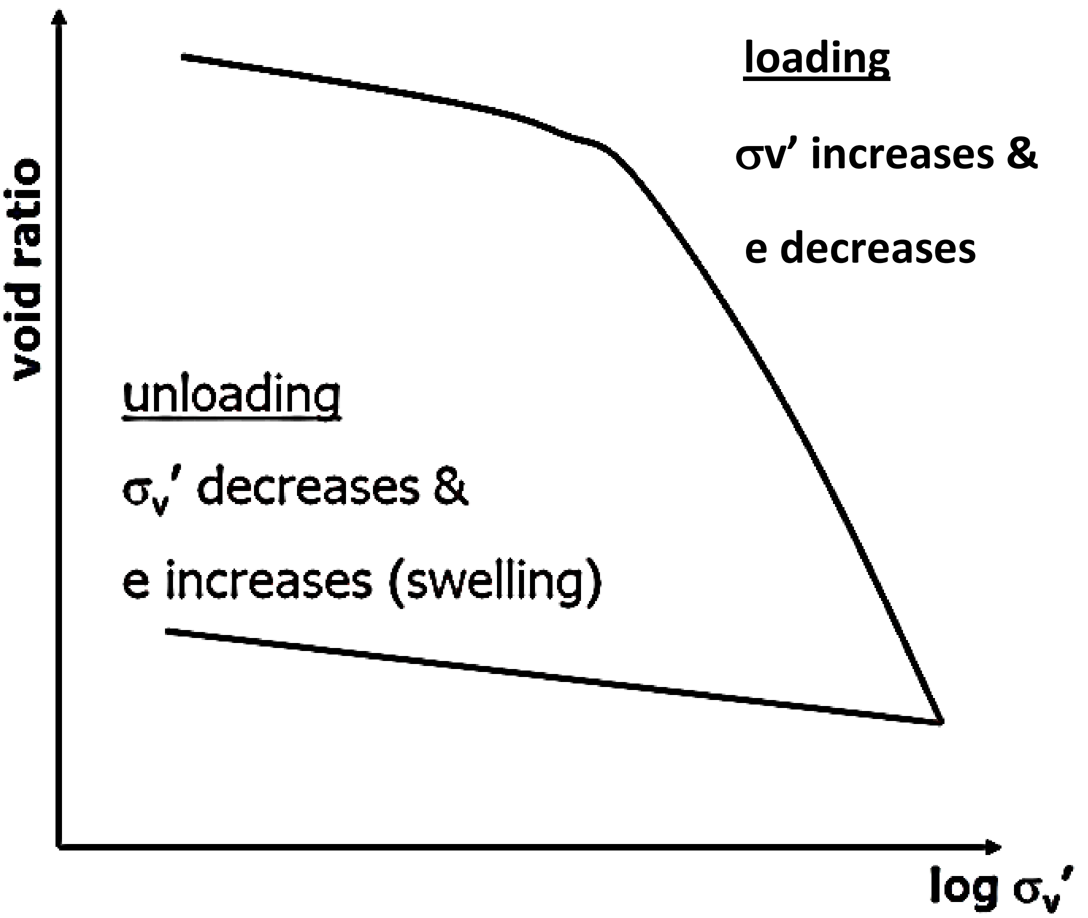


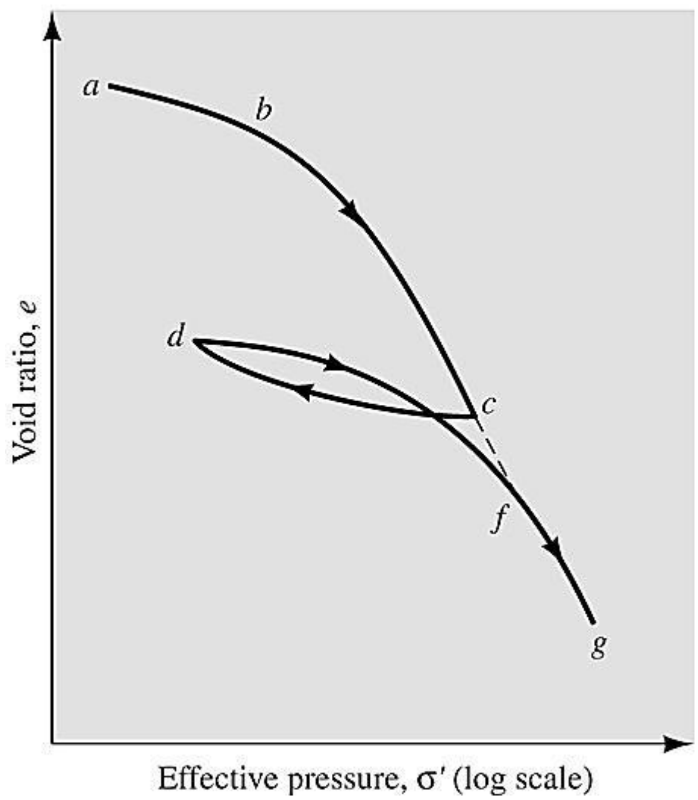
Schematic diagram of consolidation test arrangement



Time-Deformation Plot

e-log σ_v' plot





Normally Consolidated & Overconsolidated Clays:

- **Normally Consolidated**

Existing vertical effective stresses are the highest the soil has ever experienced

➔ **"Young" deposits**

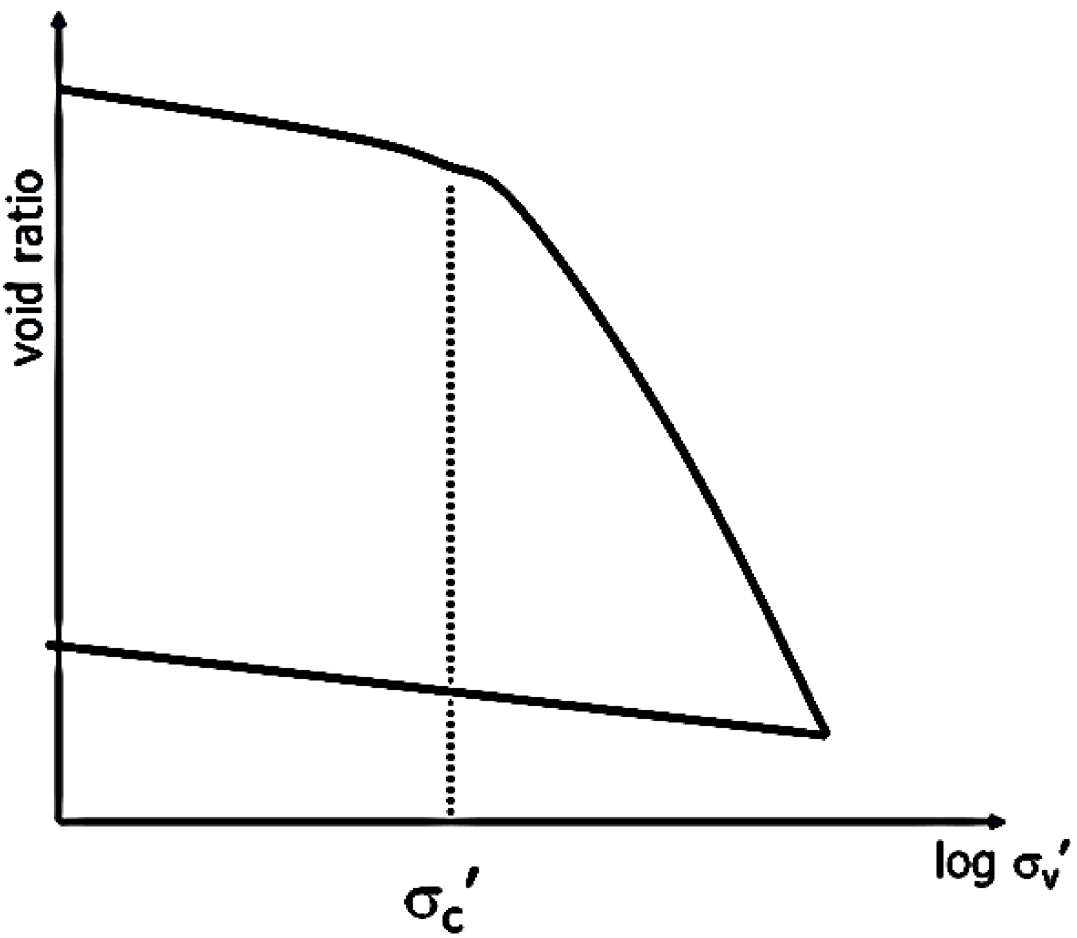
- **Overconsolidated**

Existing vertical effective stresses is less than a previous maximum

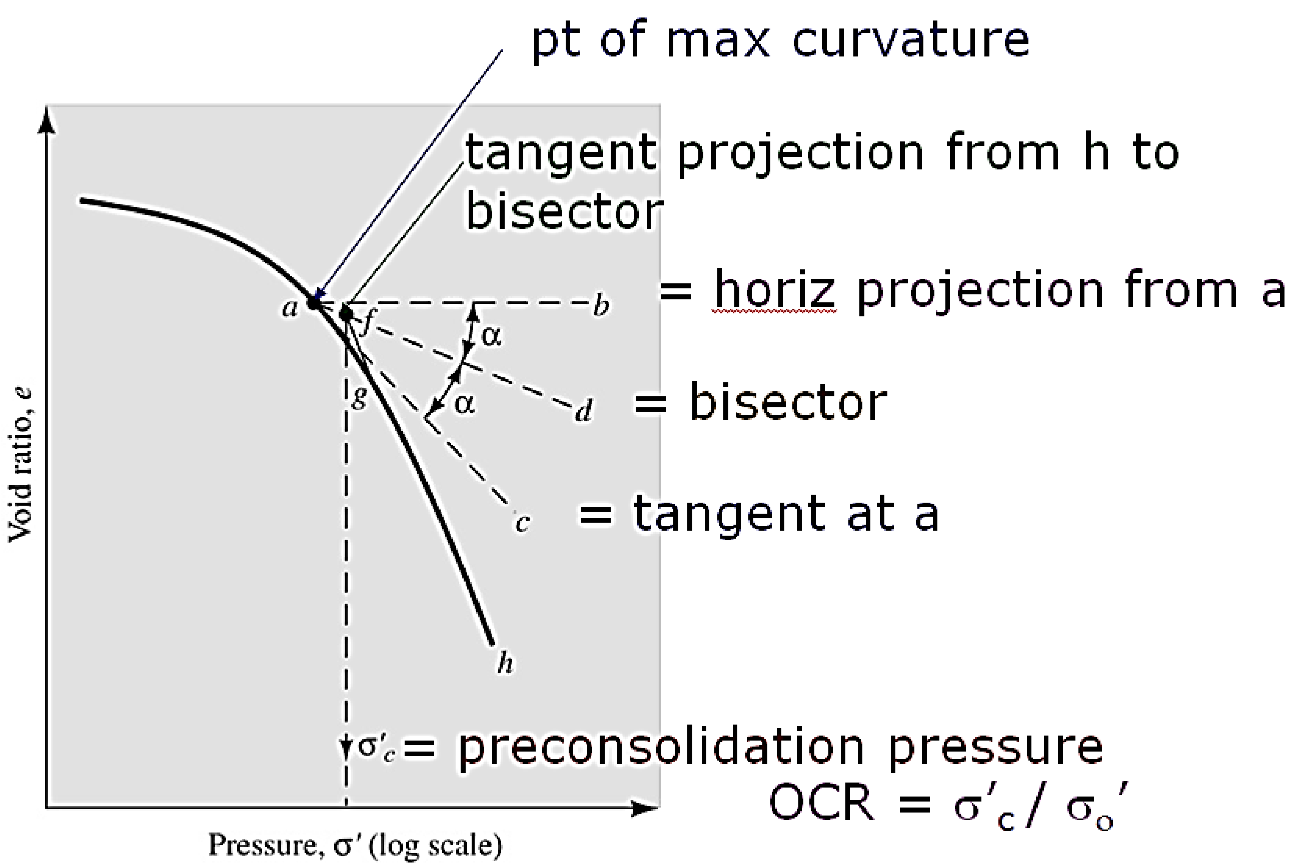
➔ **Glaciation, erosion, pre-loaded**

Preconsolidation pressure σc'

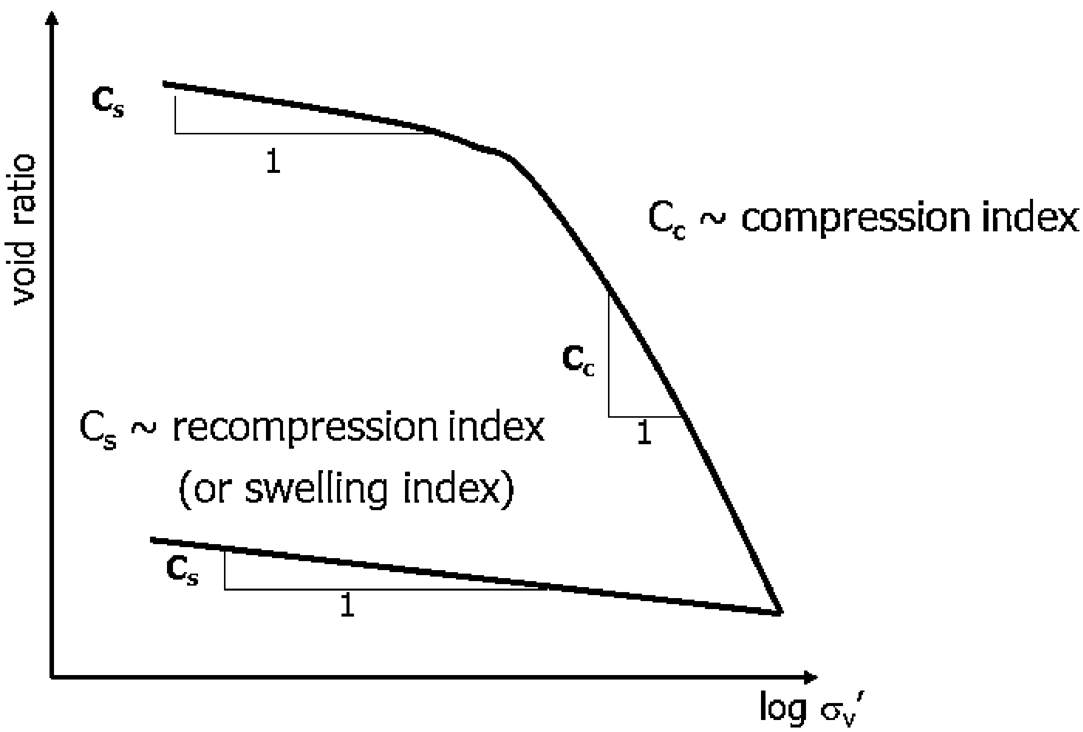
σ_c' is the maximum vertical effective stress the soil element has <u>ever</u> been subjected to



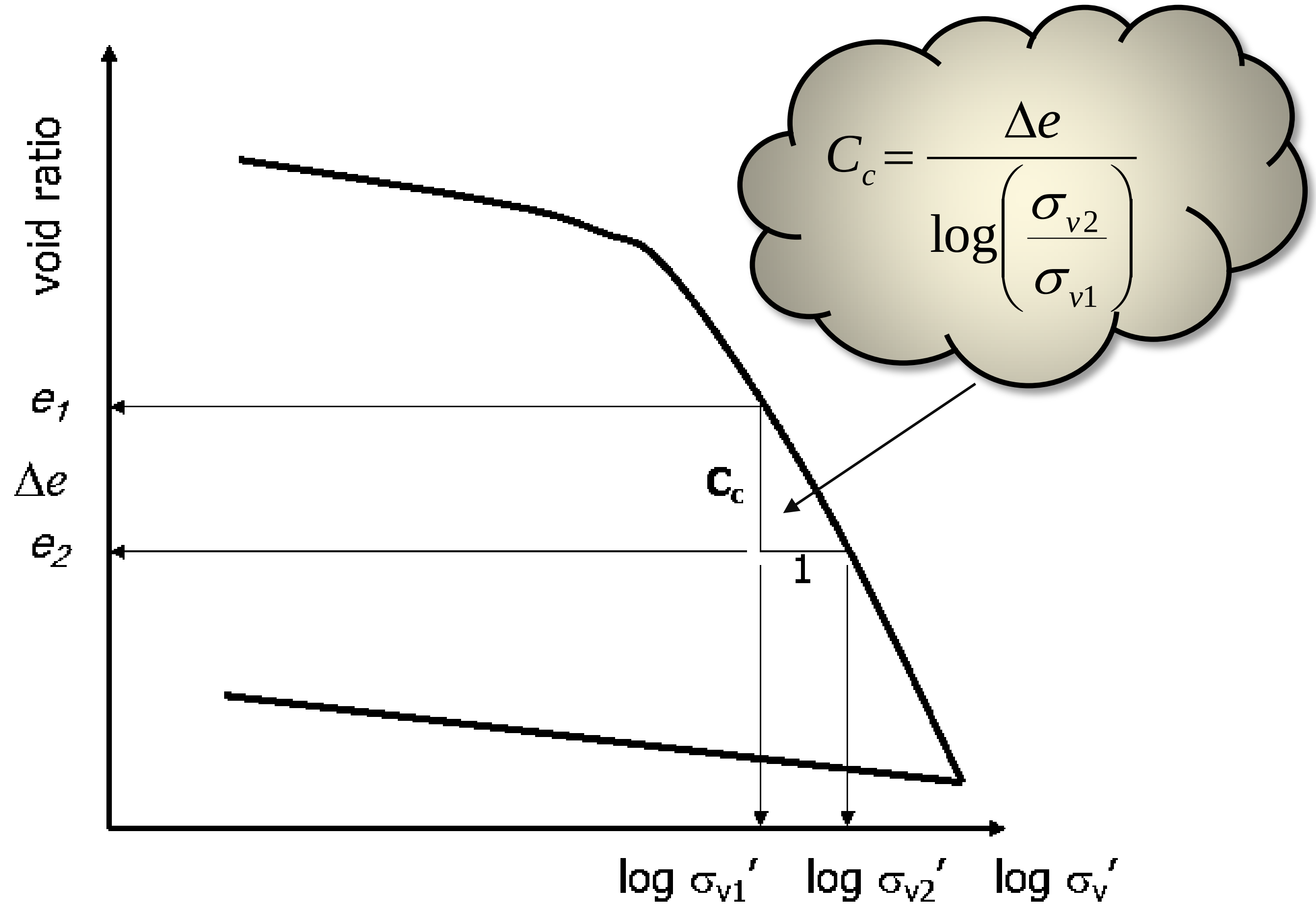
Graphical determination of σc':

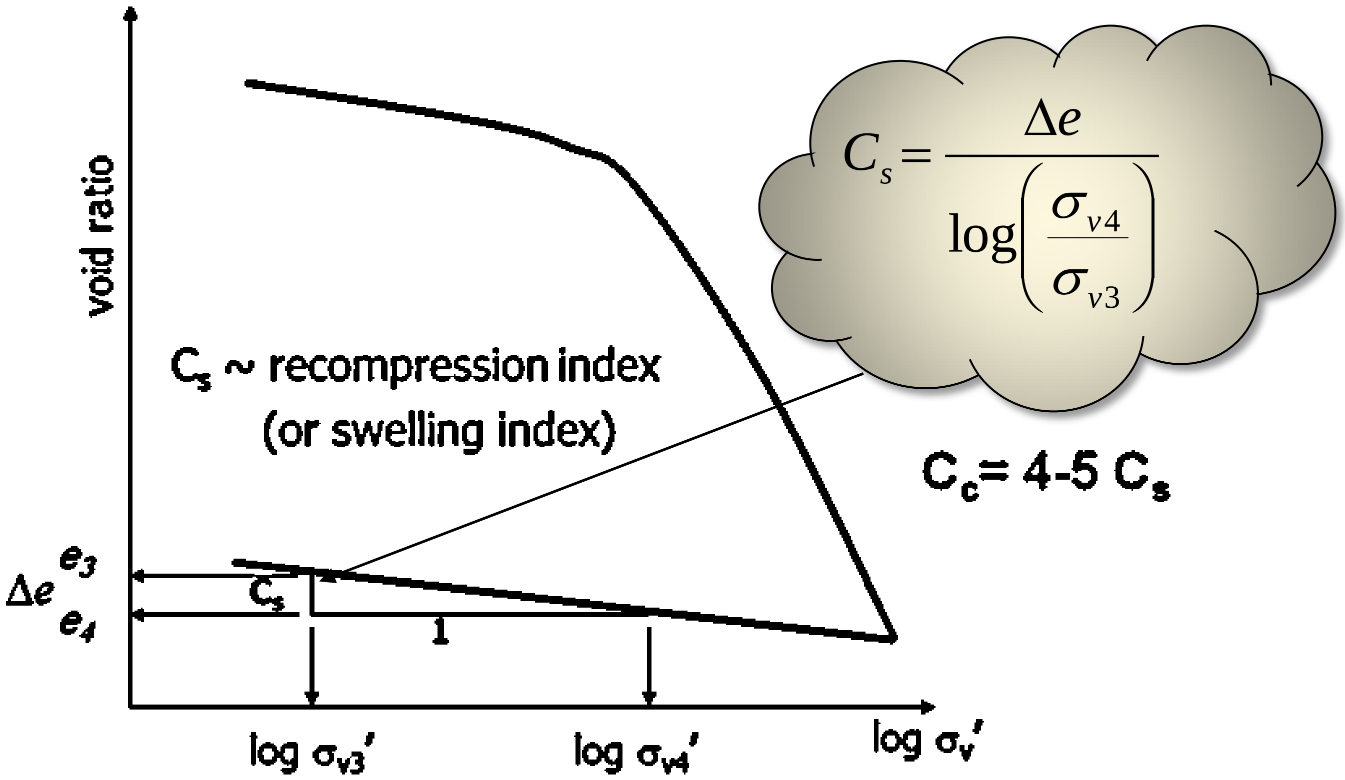


Compression and swelling indices:

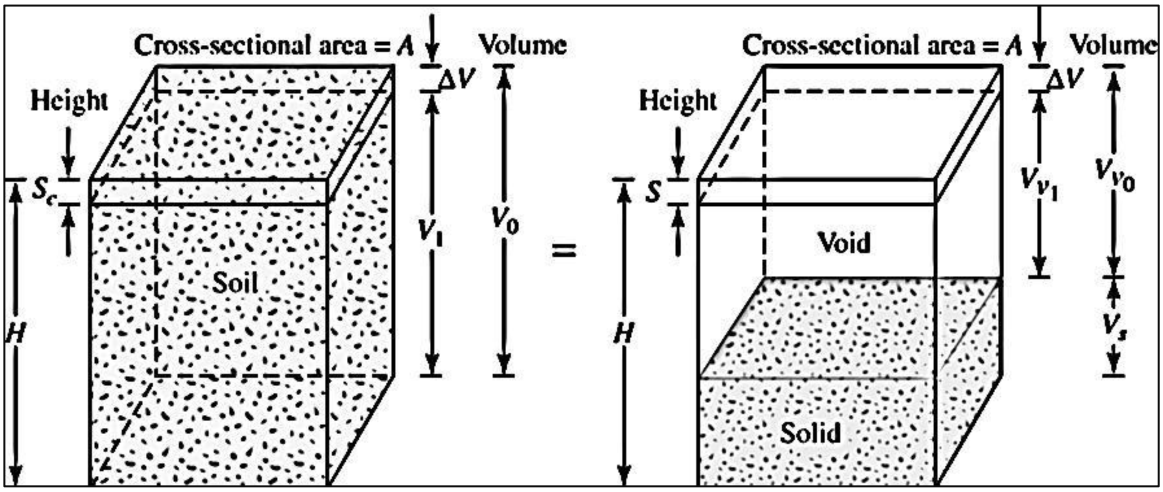


Cc ~ compression index:





SETTLEMENT CALCULATIONS:



average vertical strain = $\dfrac{\Delta H}{H}$

average vertical strain = $\dfrac{\Delta e}{1+e_o}$

Equating the two expressions for average vertical strain:

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

consolidation settlement (ΔH)

initial thickness of clay layer (H)

change in void ratio (Δe)

initial void ratio (e_o)

The key for getting Sc

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

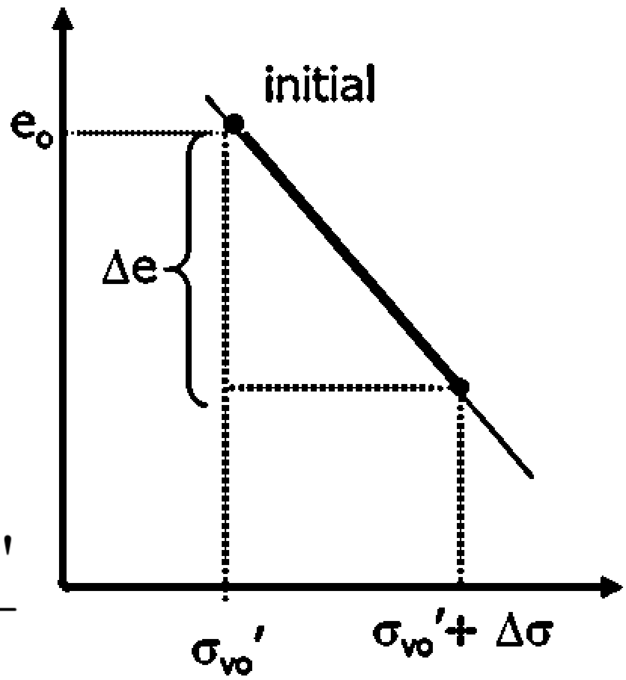
Primary consolidation settlement

If the clay is normally consolidated,

$$\Delta e = C_c \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_{vo}'}$$

⬇

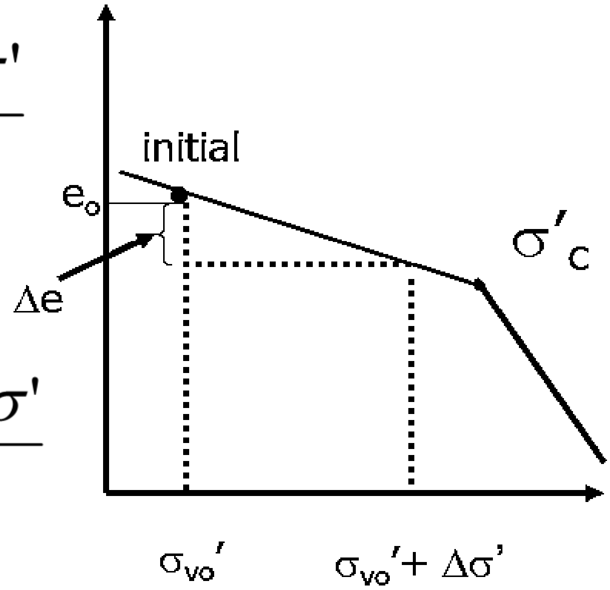
$$S_c = \frac{H\,C_c}{1+e_o} \log \frac{\sigma_{vo}' + \Delta\sigma'}{\sigma_{vo}'}$$



If the clay is overconsolidated, and remains so by the end of consolidation,

$$\sigma_{vo}'+\Delta\sigma' \leq \sigma_c'$$

$$\Delta e = C_s \log \frac{\sigma_{vo}'+\Delta\sigma'}{\sigma_{vo}'}$$

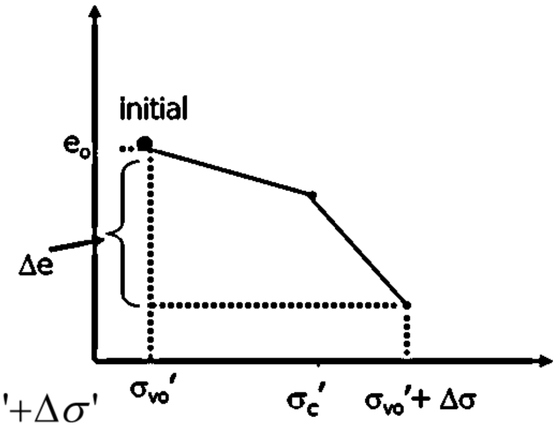


$$S_c = \frac{H C_s}{1+e_o} \log \frac{\sigma_{vo}'+\Delta\sigma'}{\sigma_{vo}'}$$

If an overconsolidated clay becomes normally consolidated by the end of consolidation,

$$\sigma_{vo}' \leq \sigma_c' \leq \sigma_{vo}'+\Delta\sigma'$$

$$\Delta e = C_s \log \frac{\sigma_c'}{\sigma_{vo}'} + C_c \log \frac{\sigma_{vo}'+\Delta\sigma'}{\sigma_c'}$$



$$S_c = \frac{H C_s}{1+e_o} \log \frac{\sigma_c'}{\sigma_{vo}'} + \frac{H C_c}{1+e_o} \log \frac{\sigma_{vo}'+\Delta\sigma'}{\sigma_c'}$$

TIME RATE OF CONSOLIDATION:

Average degree of consolidation:

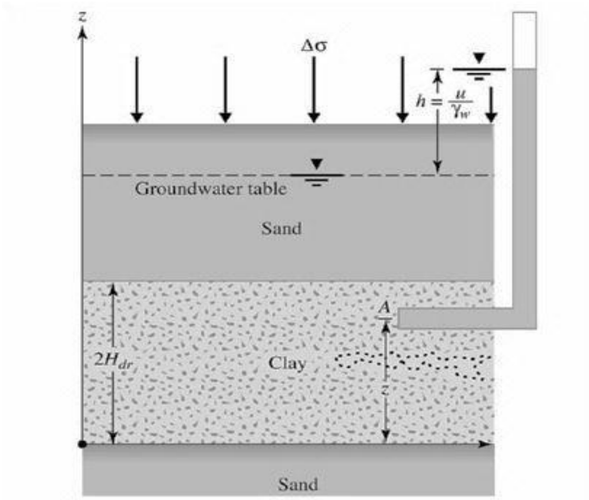
Terzaghi Equation

Excess pore water pressure

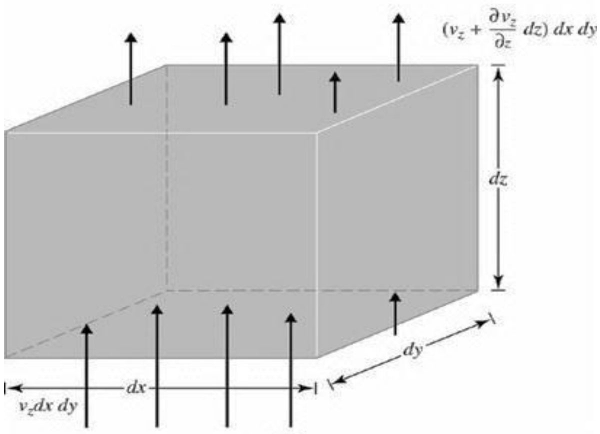
$$\frac{\partial(\Delta u)}{\partial t} = C_v \frac{\partial^2(\Delta u)}{\partial z^2}$$

Time

Coefficient of consolidation Cm^2/sec



(a)

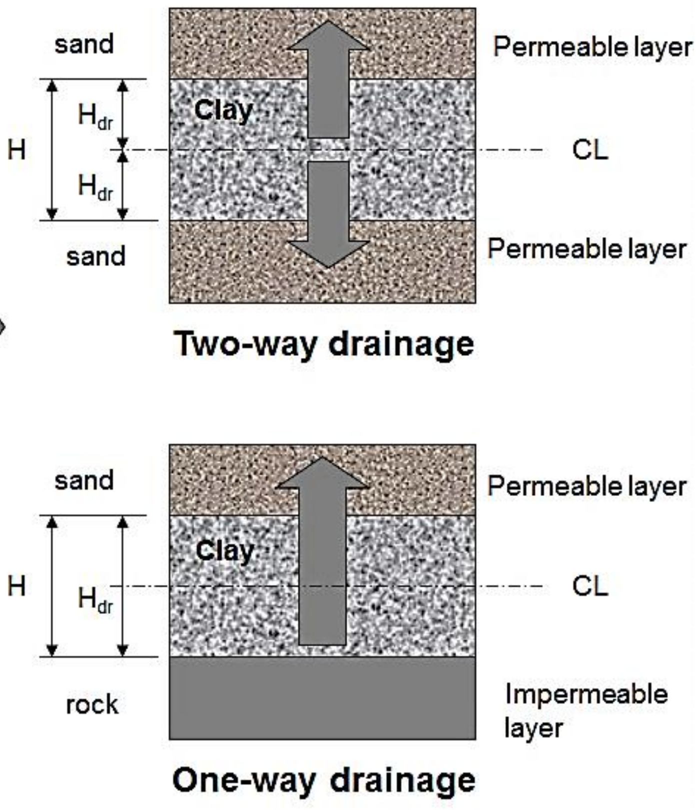


(b)

Time factor

$$T_v = \frac{C_v\, t}{H_{dr}^2}$$

Maximum drainage path



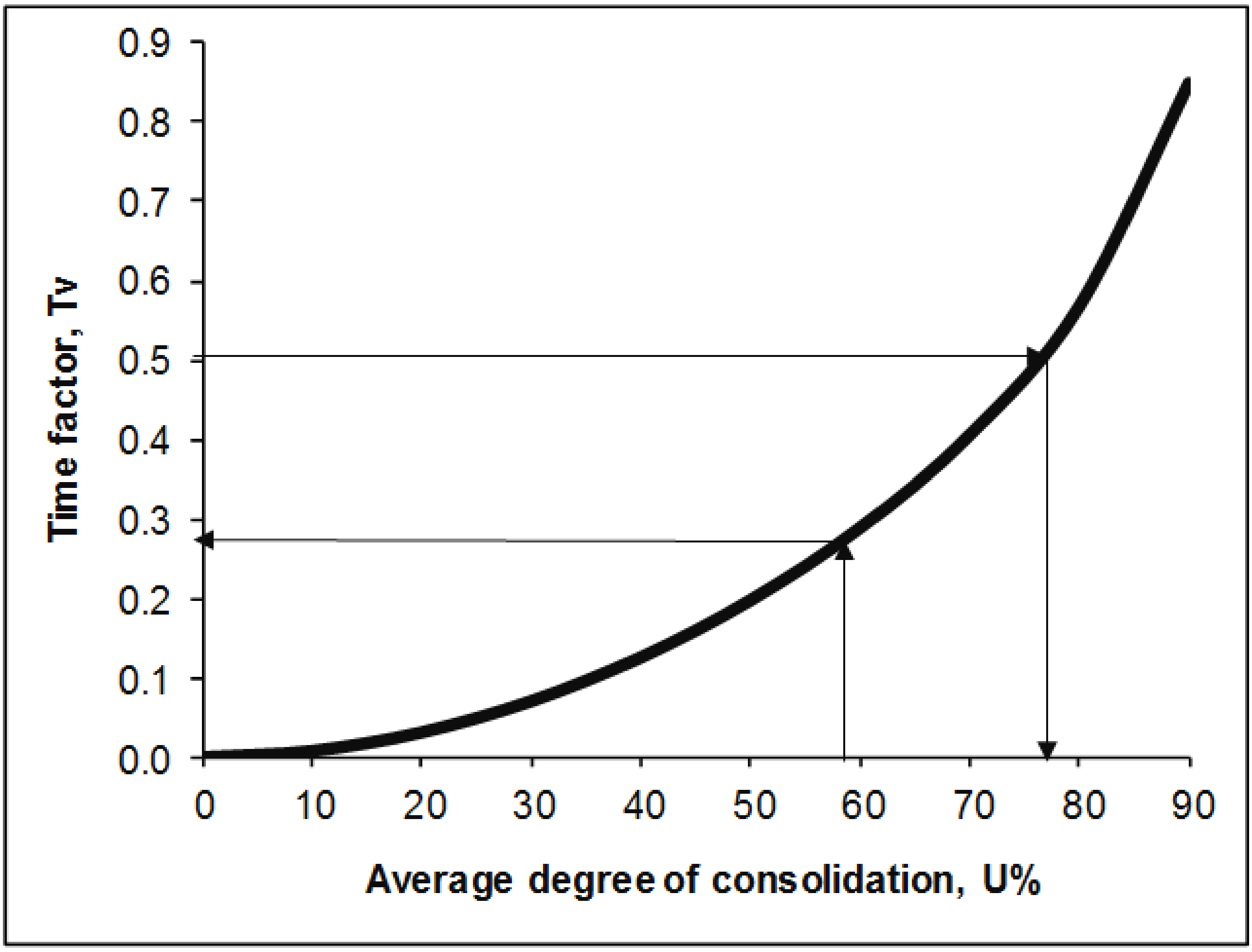
$$U = \frac{S_t}{S_{max}}$$

Settlement at time t after load application (S_t)

Maximum consolidation settlement under a given loading (S_{max})

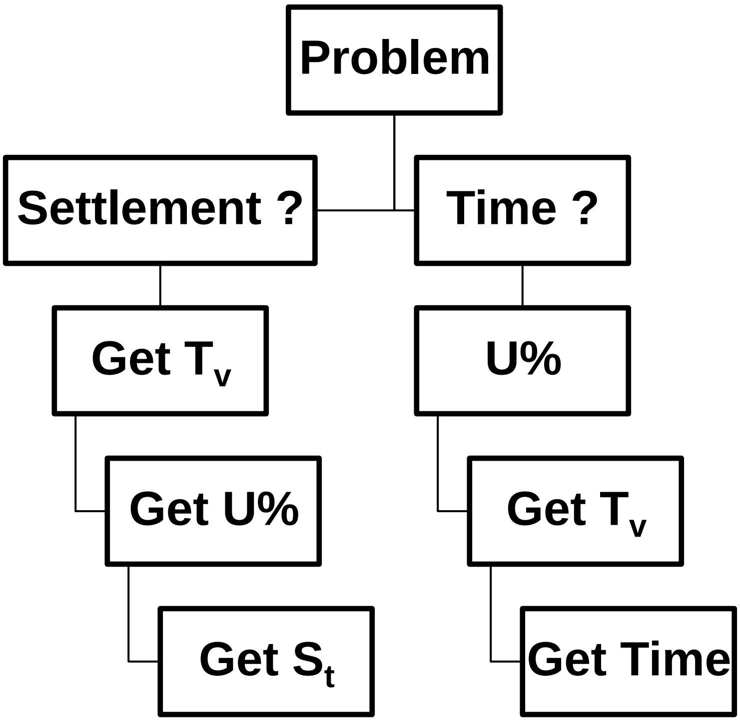
Average degree of consolidation (U)

$$T_v = \frac{\left(\frac{\pi}{4}\right)\left(\frac{U\%}{100}\right)^2}{\left[1-\left(\frac{U\%}{100}\right)^{5.6}\right]^{0.357}}$$



Plot of time factor against average degree of consolidation

Two types of problems are expected:

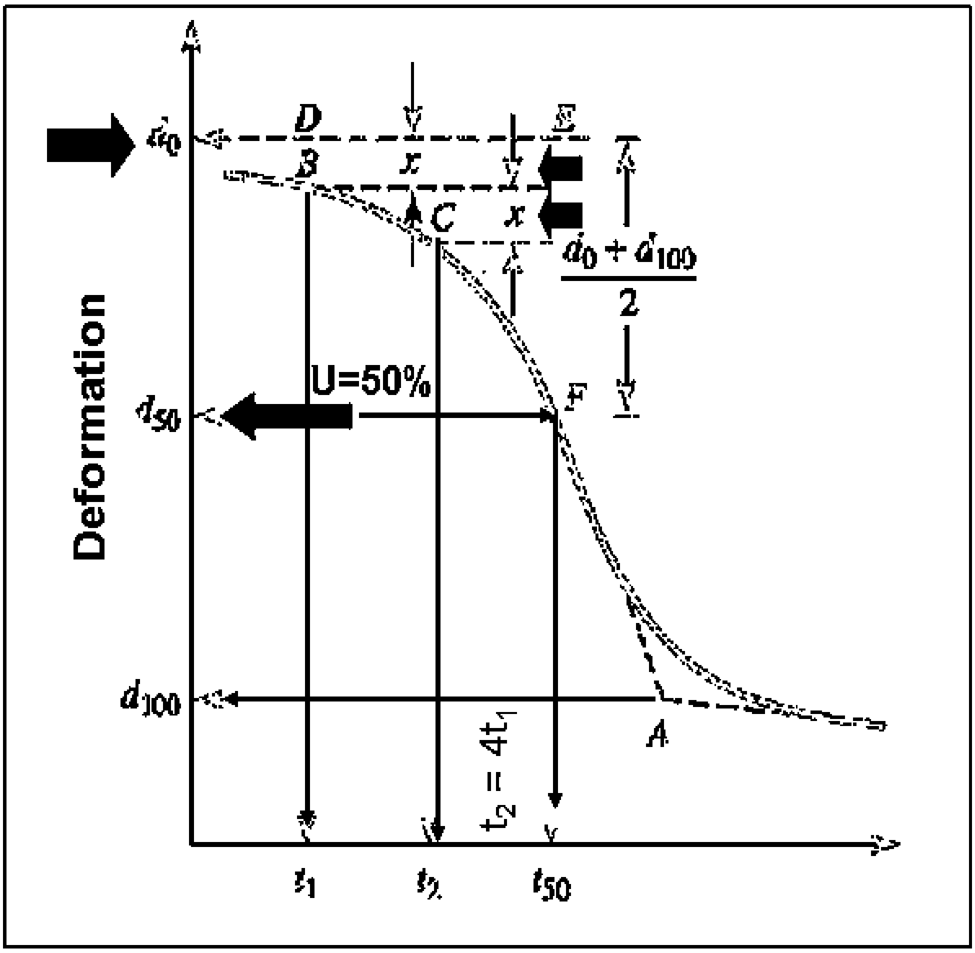


Determination of Cv:

For U = 50%

$T_v = 0.198$

$$T_v = \frac{C_v\, t}{H_{dr}^2}$$



Time (log scale)

SECONDARY CONSOLIDATION SETTLEMENT:

Occurs at the end of primary consolidation

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

Where:

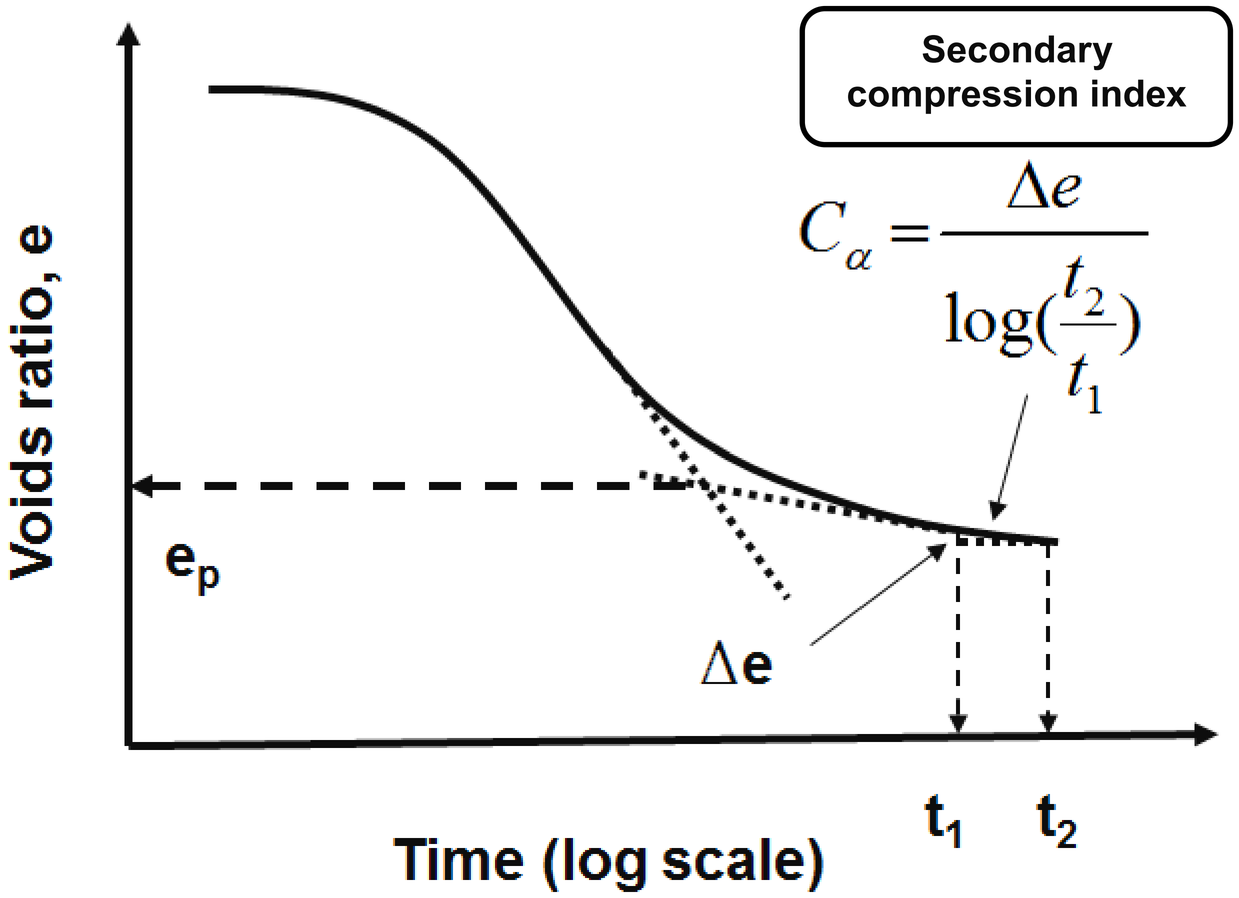
$$C'_\alpha = \frac{C_\alpha}{1+e_p}$$

ep = voids ratio at the end of primary consolidation

H = thickness of clay layer

t1 = time at the end of primary consolidation

t2 = time at which secondary consolidation is required



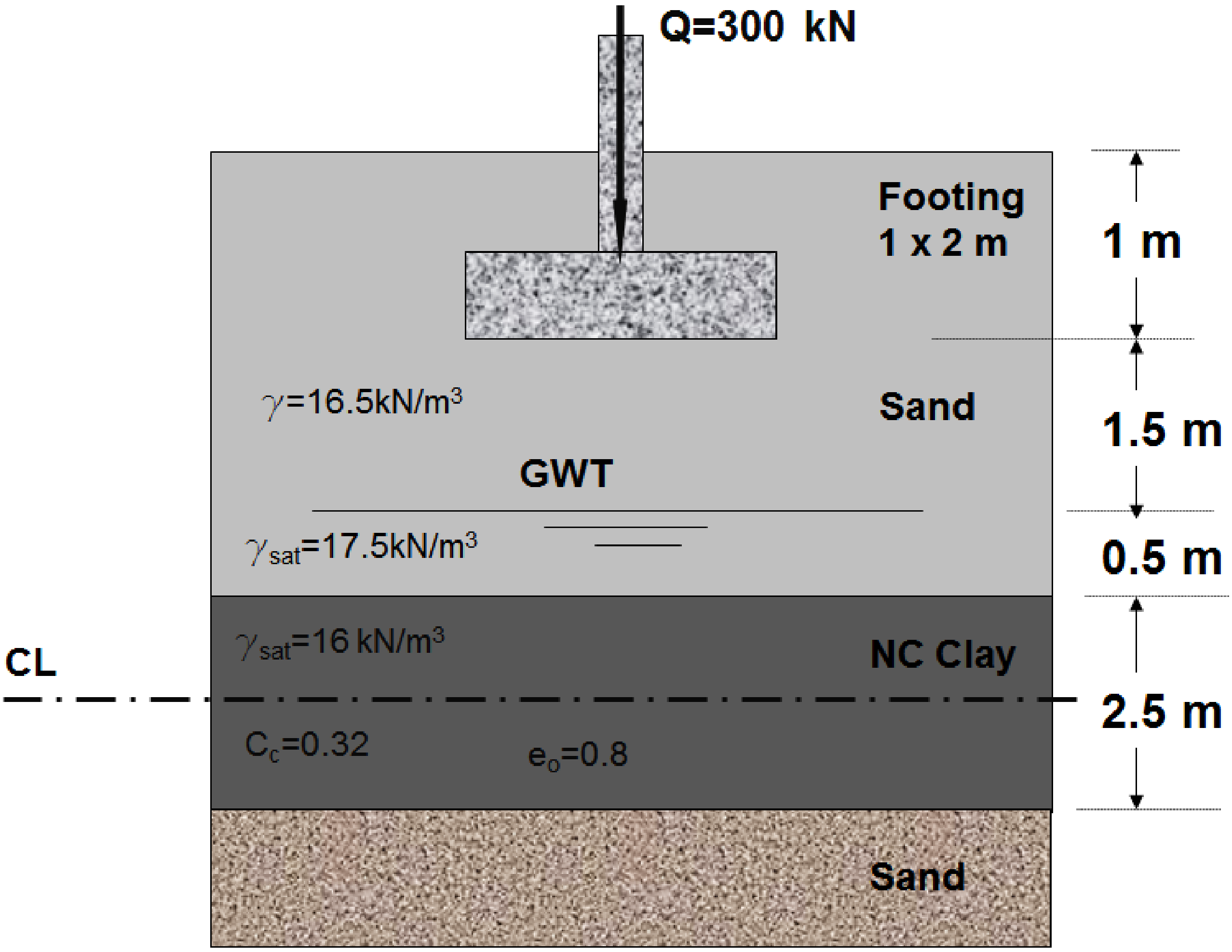
Secondary consolidation settlement is more important in organic clays and soft to very soft clays.

In overconsolidated inorganic clays the secondary compression index is very small and of less practical significance.

- Absolute settlement
- Differential settlement
- Angle of distortion

EXAMPLE:

Estimate the primary consolidation settlement for the foundation shown in Fig.



NC clay, then

CL of clay layer

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

σ_{vo}' = 2.5x16.5 + 0.5x(17.5-9.81) + 1.25x(16-9.81)

= 52.84 kN/m2

Using 2:1 approximate method at CL of clay layer

$$\Delta\sigma' = \frac{300}{(1+3.25)(2+3.25)} = 13.45 \text{ KN/m}^2$$

$$S_c = \frac{2.5x0.32}{1+0.8} \log \frac{52.84+13.45}{52.84} = 0.044\,m$$

APPLICATIONS

- Water level lowering
- New building adjacent to an existing one
- Basement
- Increasing number of floors over existing building
- Variable thickness of clay layer
- More than one clay layer

LATERAL EARTH PRESSURE

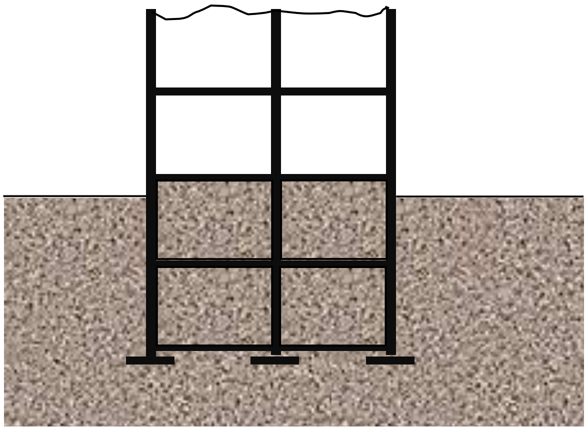
Outline:

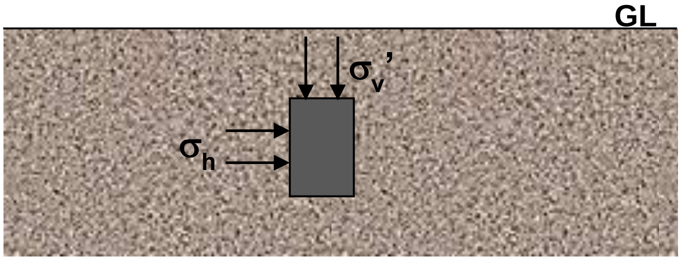
- Introduction
- Earth pressure at rest
- Active earth pressure
- Passive earth pressure

Soil nailing **Cantilever wall**

Sheet pile wall

Earth Pressure at Rest:





The ratio σ_h'/σ_v' is a constant known as coefficient of earth pressure at rest (K_o).

At Ko state, there is no lateral strains.

For normally consolidated clays and granular soils,

$$K_o = 1 - \sin\phi$$

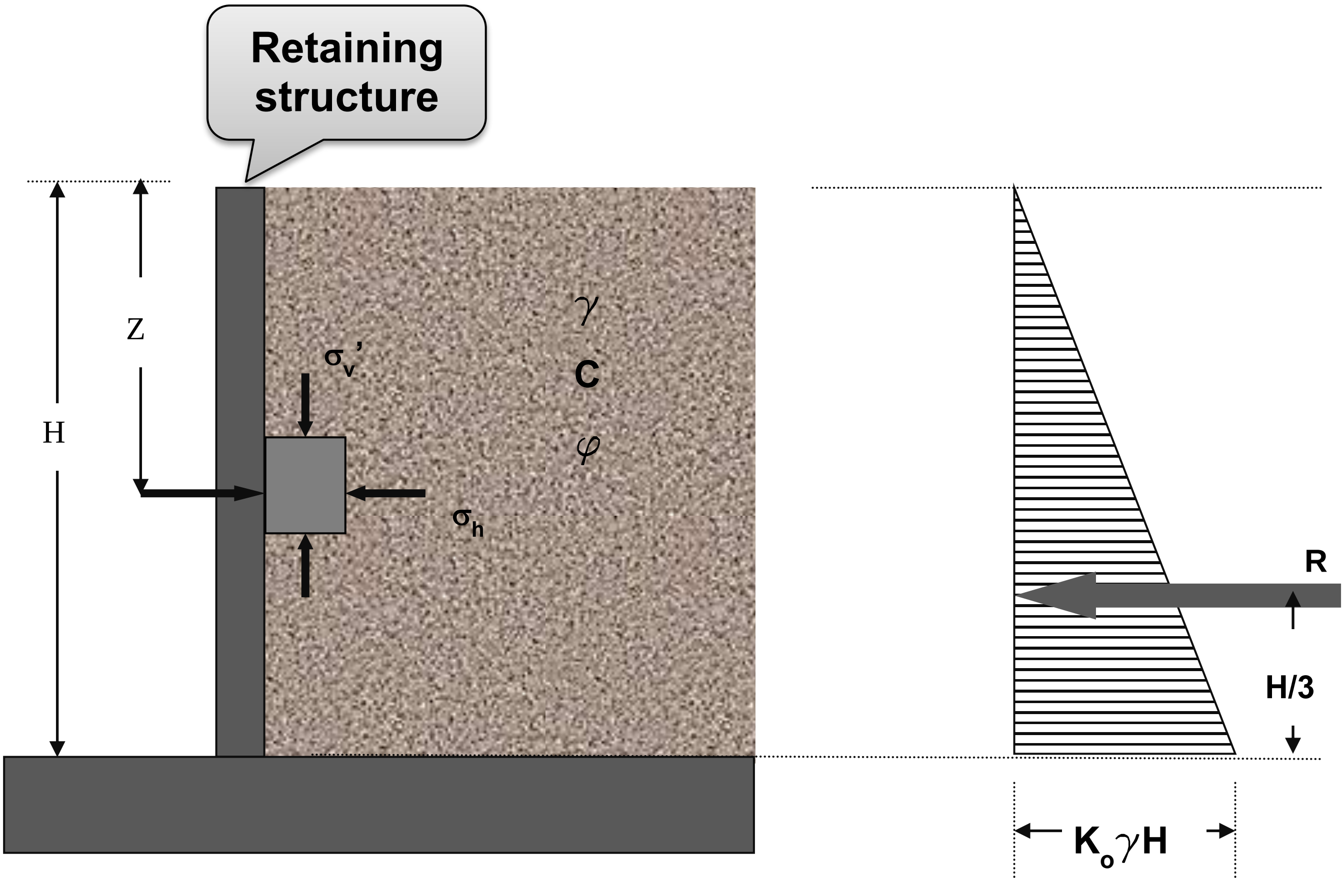
For overconsolidated clays,

$$K_{o,overconsolidated} = K_{o,normally\ consolidated}\ OCR^{0.5}$$

From elastic analysis,

$$K_o = \frac{\upsilon}{1-\upsilon}$$

Poisson's ratio



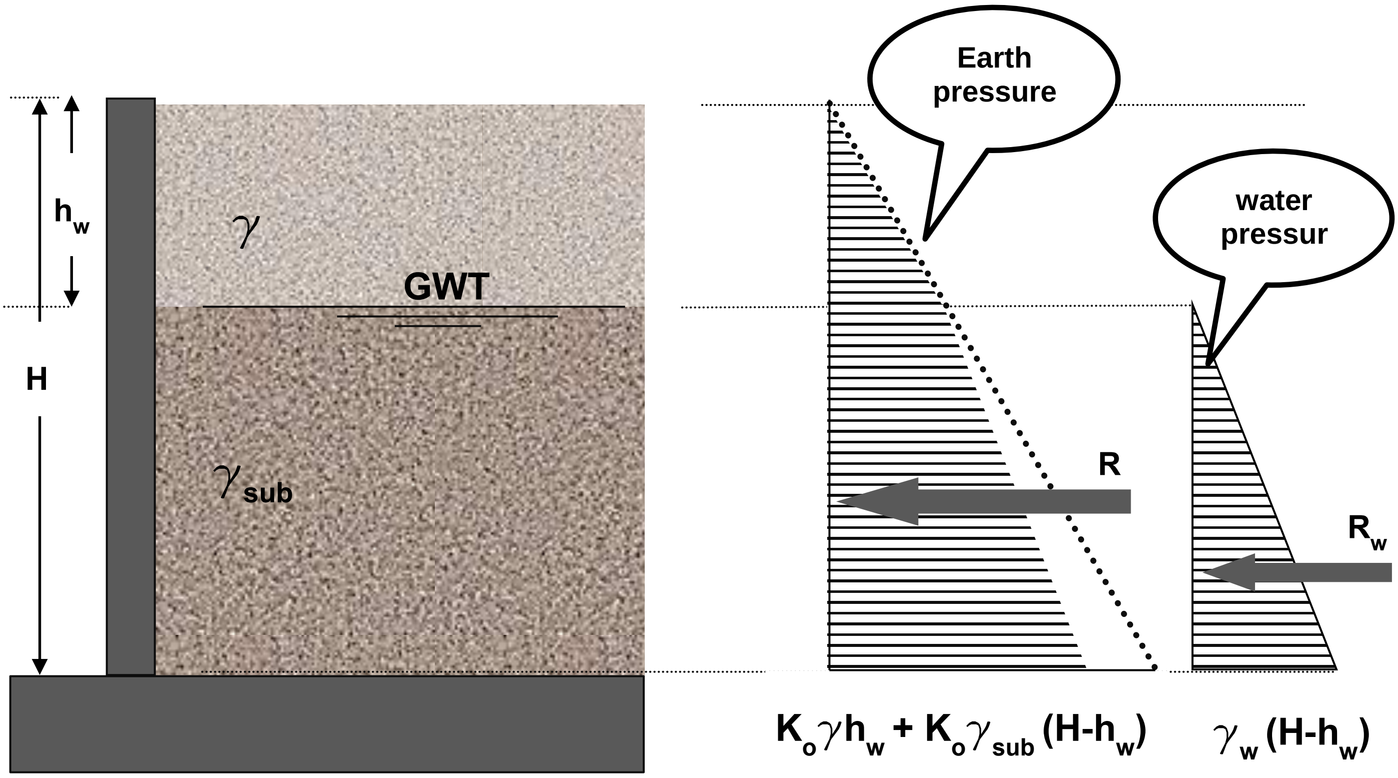
At rest earth pressure diagram

$$\sigma_o = \sigma_v^{'} K_o$$

At rest earth pressure

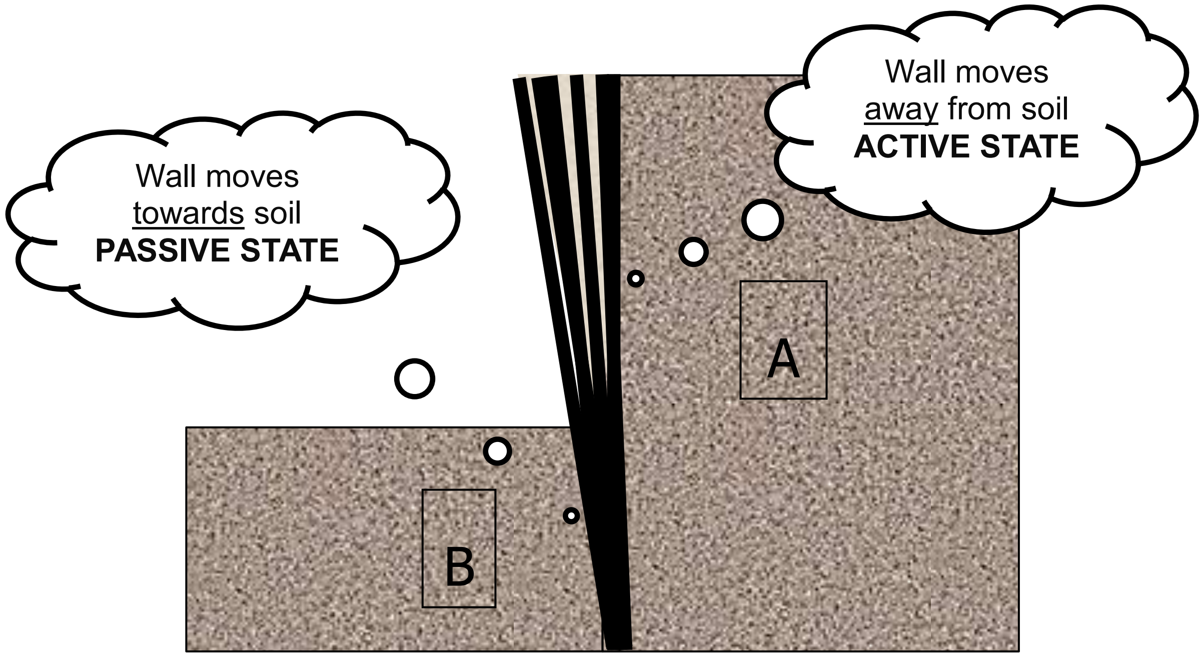
$$\sigma_v^{'} = \gamma\, z$$

Effect of water table:

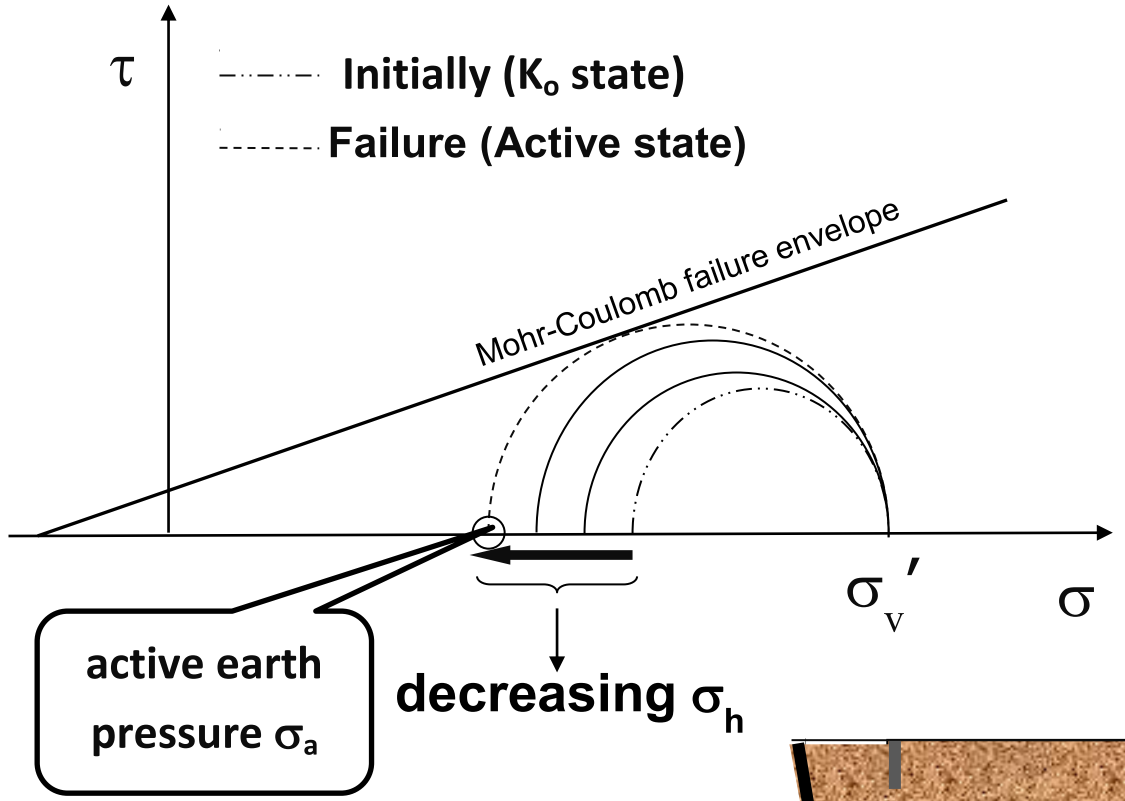


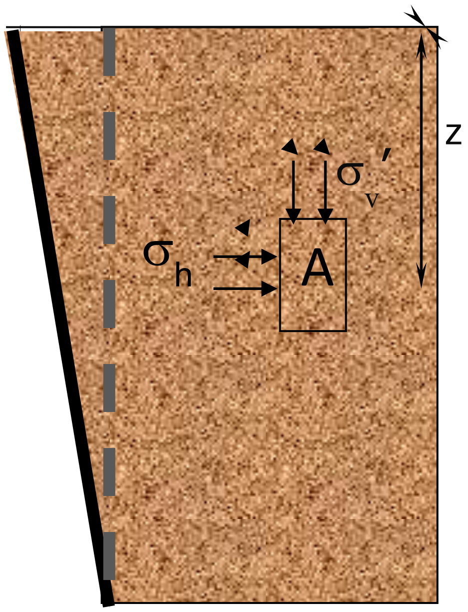
At rest earth pressure diagram

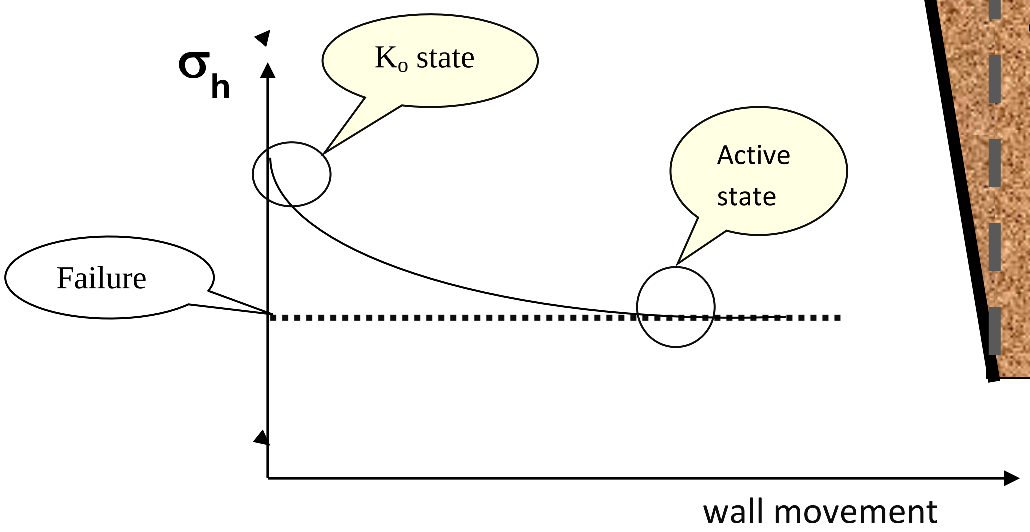
ACTIVE & PASSIVE EARTH PRESSURE

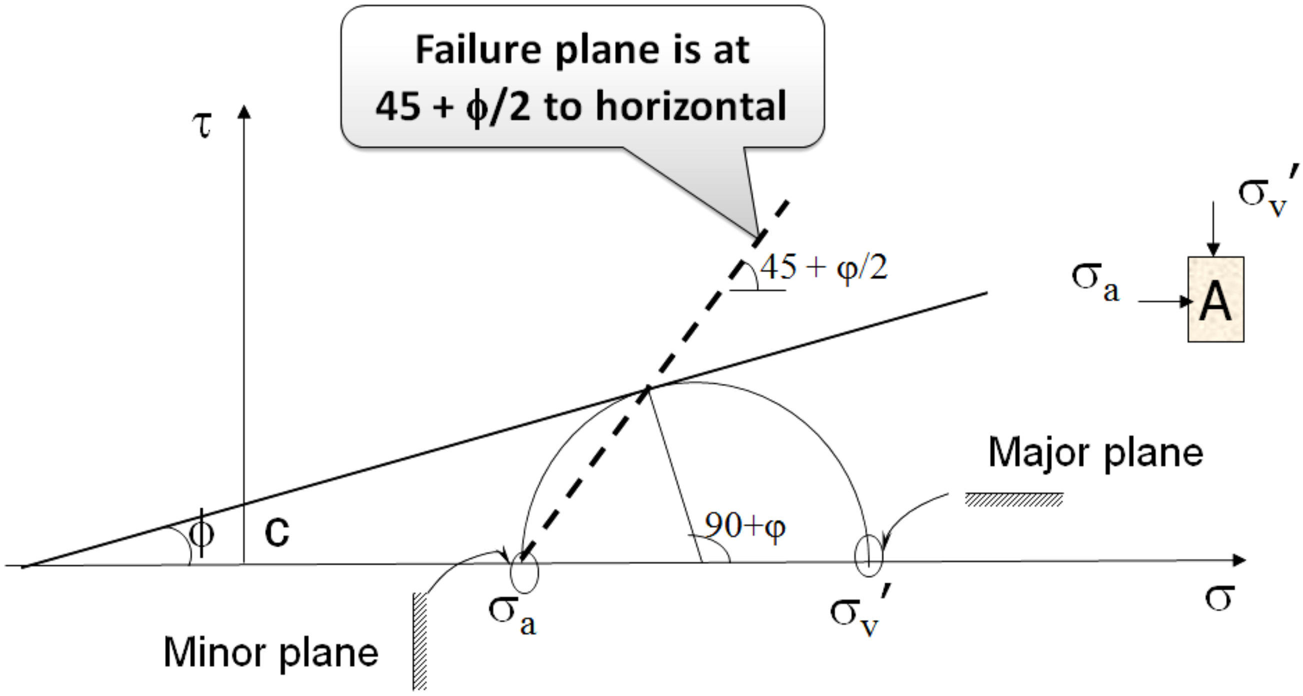


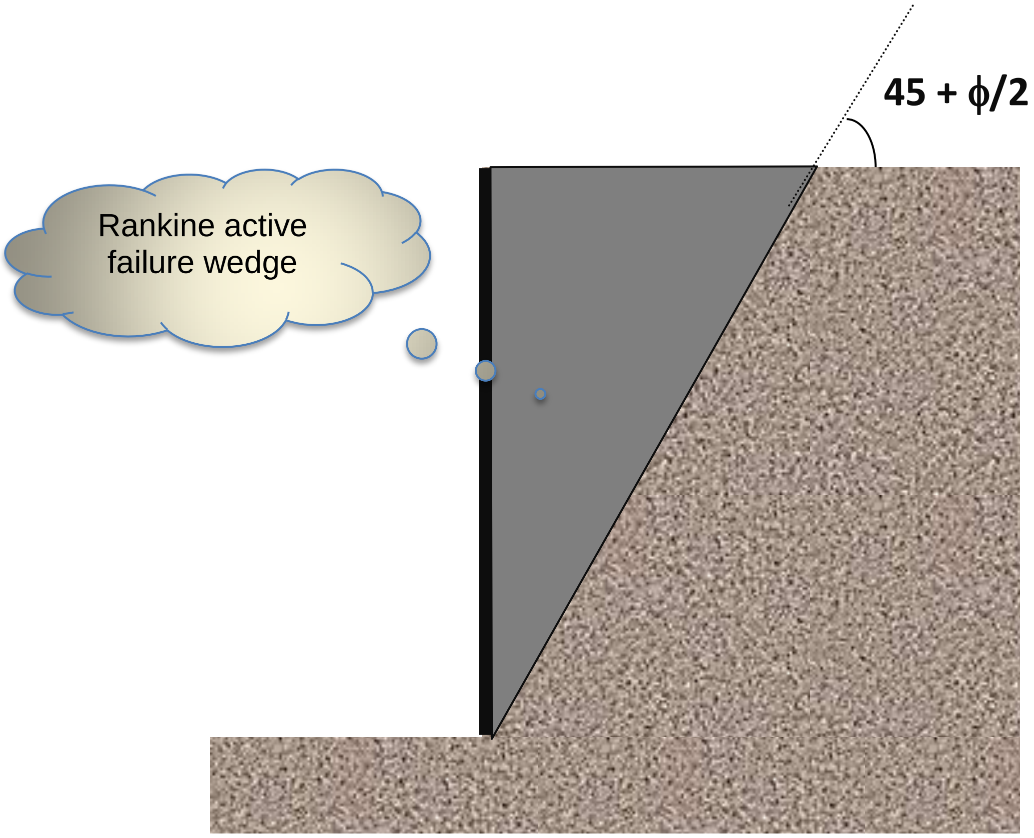
Active Earth Pressure:








Failure plane is at
45 + ϕ/2 to horizontal
τ
45 + φ/2
σv'
σa
A
Major plane
90+φ
ϕ
c
σa
σv'
σ
Minor plane


45 + ϕ/2
Rankine active
failure wedge

$$\sigma_1 = \sigma_3 \tan^2\left(45+\frac{\phi}{2}\right)+2\,c\,\tan\left(45+\frac{\phi}{2}\right)$$

Major principal stress $\sigma_1 = \sigma_v^{'}$

Minor principal stress $\sigma_3 = \sigma_a$

Thus,

$$\sigma_v^{'} = \sigma_a \tan^2\left(45+\frac{\phi}{2}\right)+2\,c\,\tan\left(45+\frac{\phi}{2}\right)$$

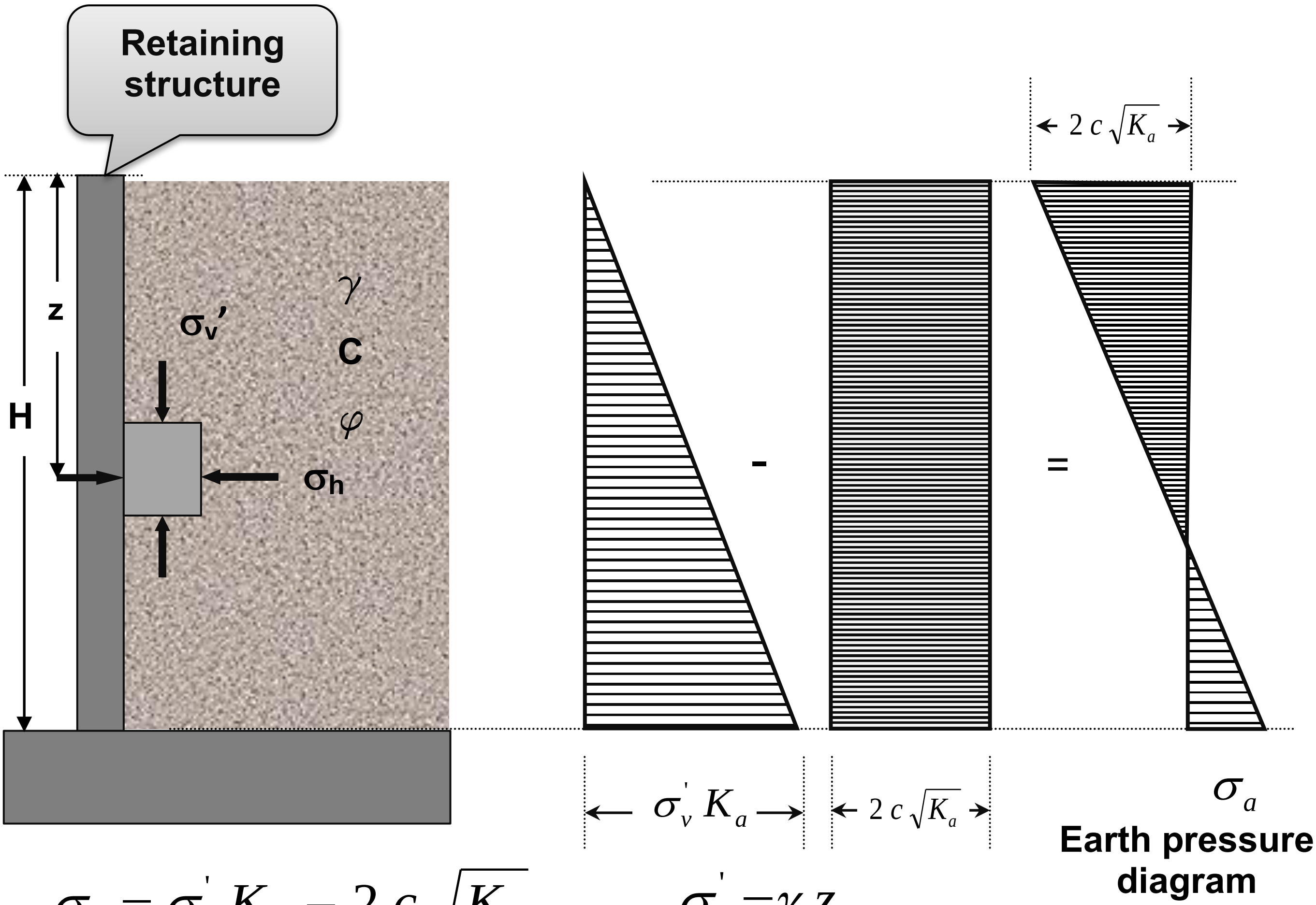
$$\sigma_a = \frac{\sigma_v^{'}}{\tan^2\left(45+\frac{\phi}{2}\right)} - \frac{2\,c}{\tan\left(45+\frac{\phi}{2}\right)}$$

$$\sigma_a = \sigma_v^{'} \tan^2\left(45-\frac{\phi}{2}\right) - 2\,c\,\tan\left(45-\frac{\phi}{2}\right)$$

Rankine active earth pressure coefficient, K_a

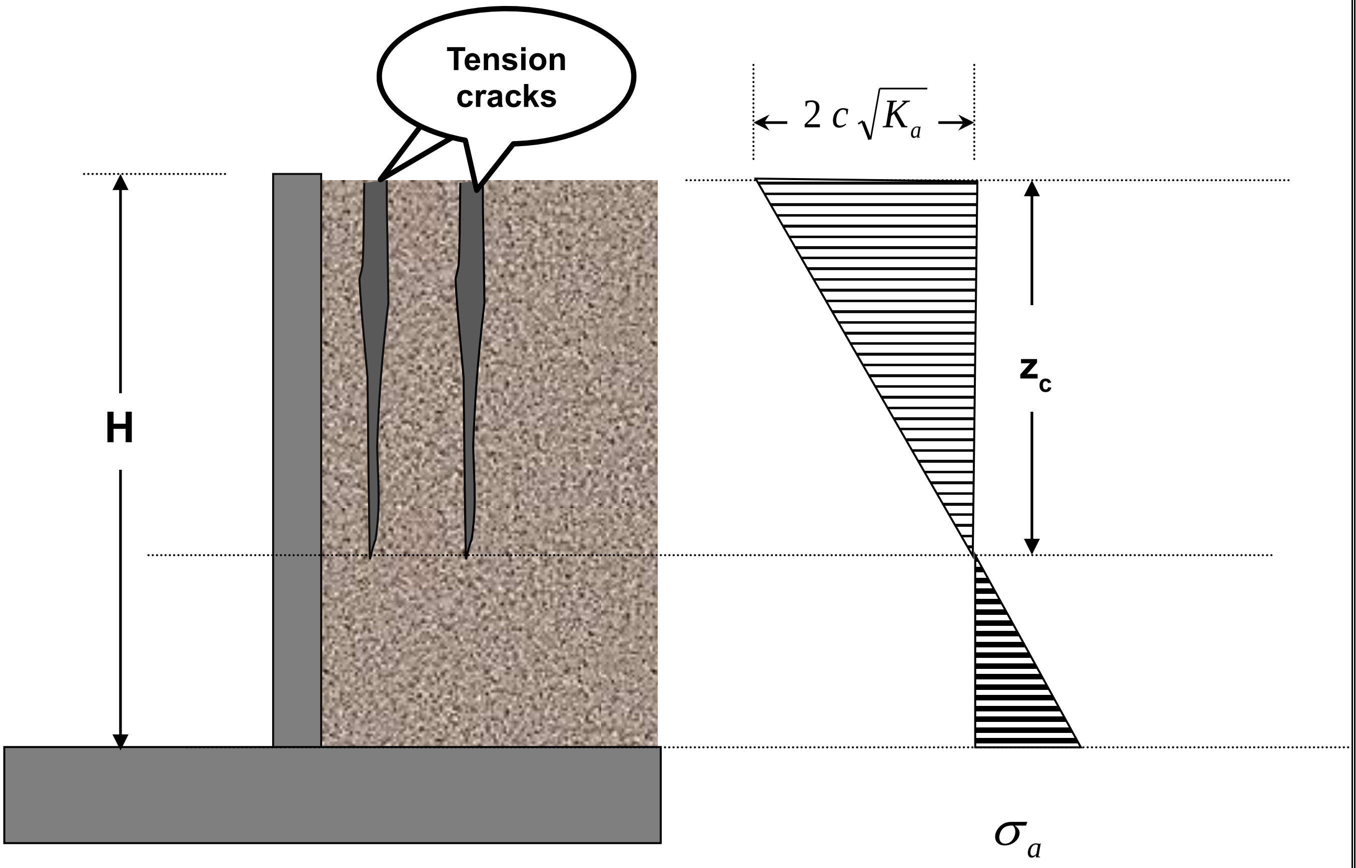
$$\boxed{\sigma_a = \sigma_v^{'}\,K_a - 2\,c\,\sqrt{K_a}}$$

RANKINE ACTIVE EARTH PRESSURE



$$\sigma_a = \sigma_v' K_a - 2\,c\,\sqrt{K_a} \qquad \sigma_v' = \gamma\, z$$

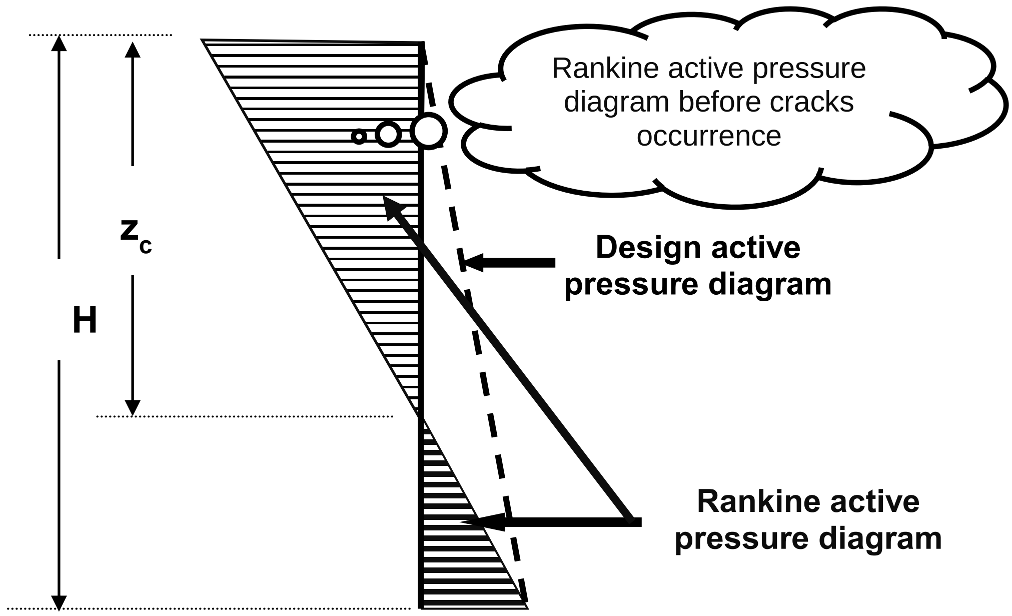
 helo



Earth pressure diagram

$$0 = \gamma\, z_c\, K_a - 2\, c\, \sqrt{K_a} \qquad z_c = \frac{2\,c}{\gamma\,\sqrt{K_a}}$$

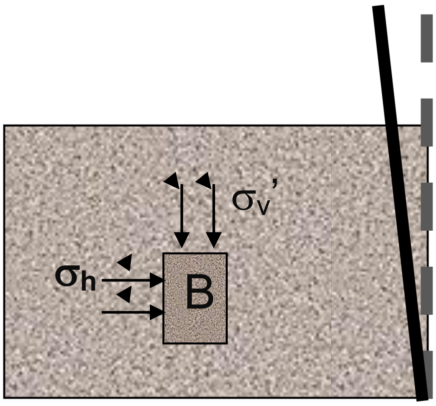
z_c = depth of tension crack



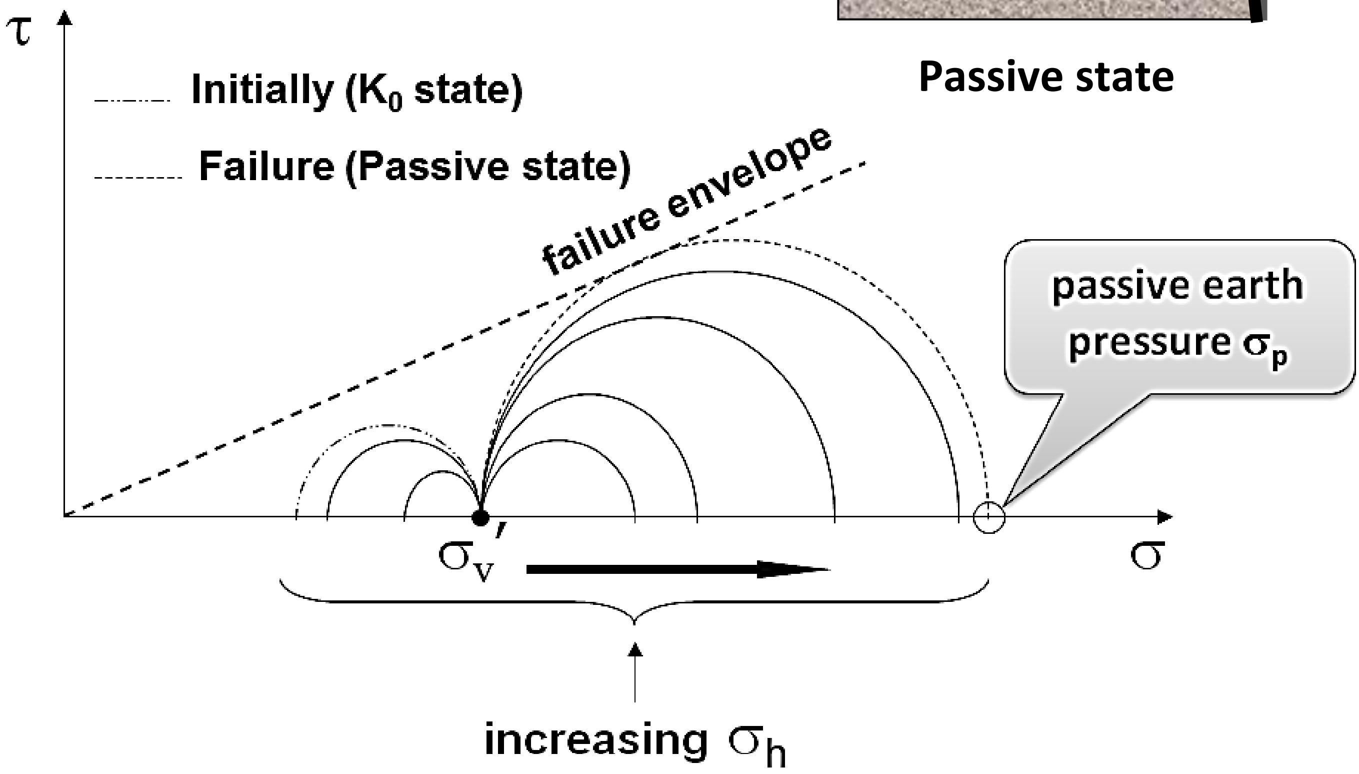
Rankine active pressure diagram for clay backfill behind retaining structure

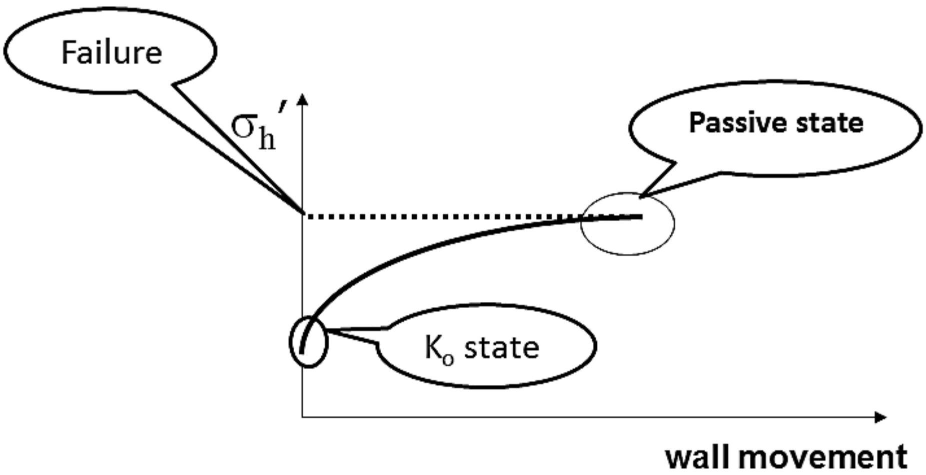
Passive Earth Pressure:

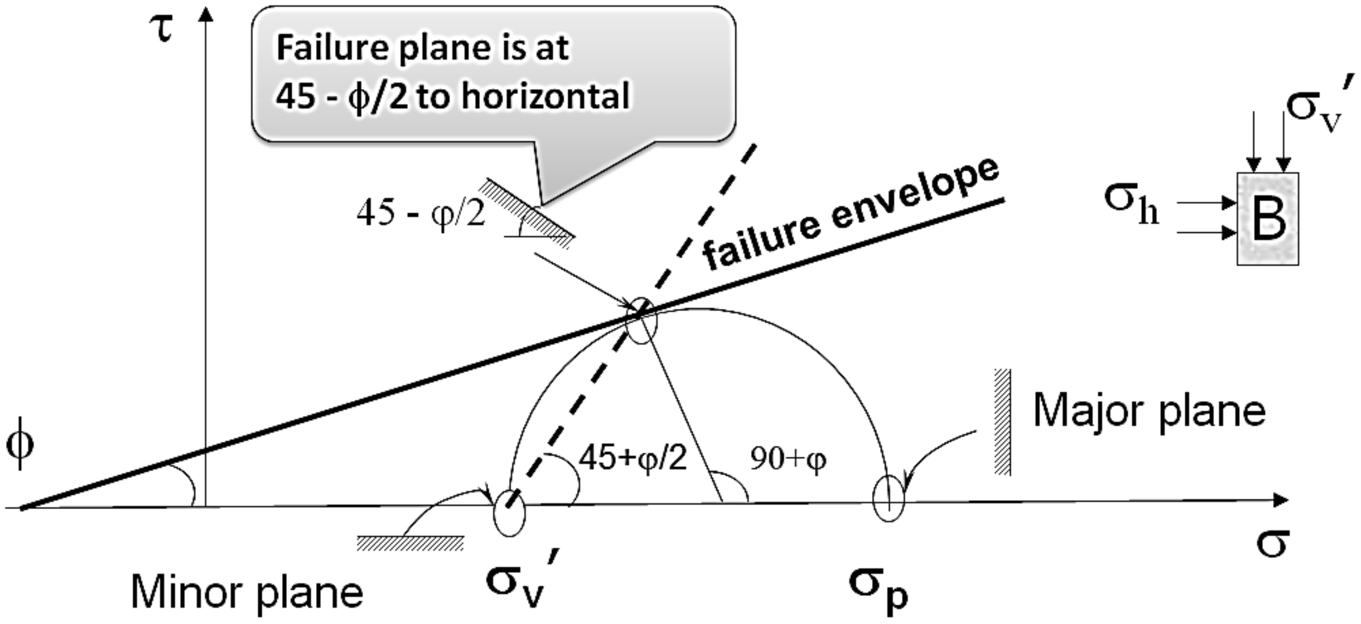
As the wall moves towards the soil,

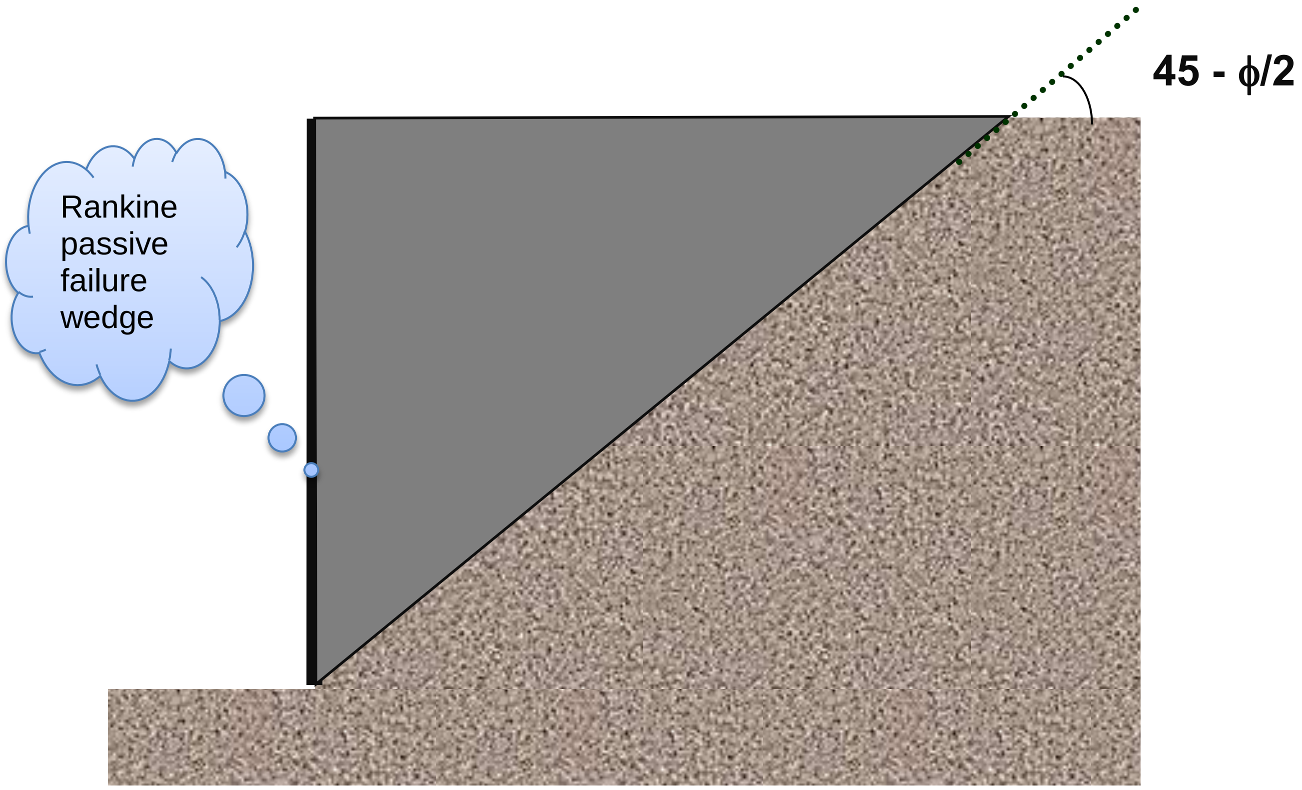


Passive state






τ
Failure plane is at
45 - ϕ/2 to horizontal
45 - φ/2
failure envelope
σv'
σh
B
ϕ
45+φ/2
90+φ
Major plane
σ
Minor plane
σv'
σp


45 - ϕ/2
Rankine passive failure wedge

$$\sigma_1 = \sigma_3 \tan^2\left(45 + \frac{\phi}{2}\right) + 2\,c\,\tan\left(45 + \frac{\phi}{2}\right)$$

Major principal stress $\sigma_1 = \sigma_p$

Minor principal stress $\sigma_3 = \sigma_v^{'}$

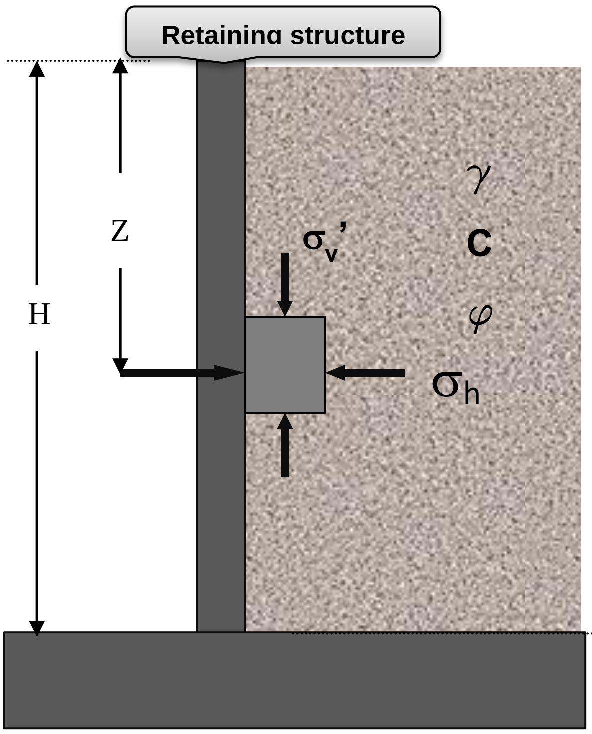
Thus,

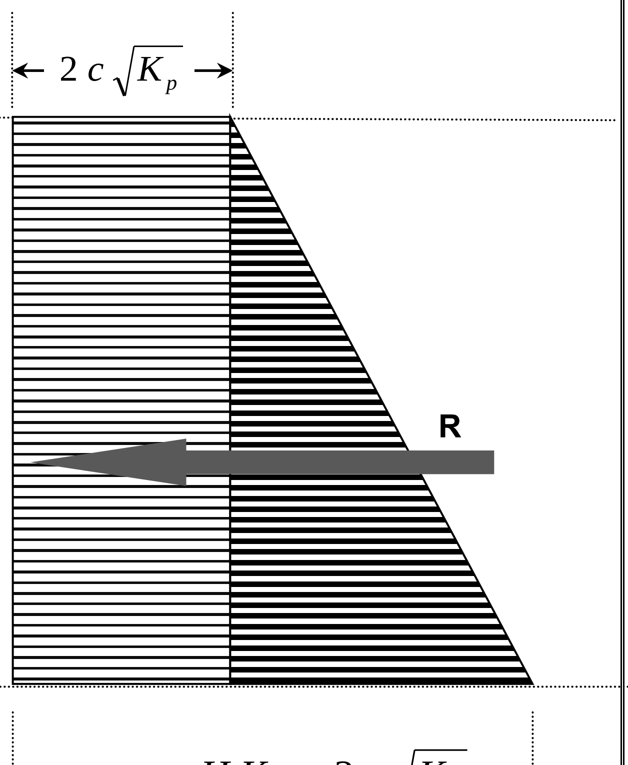
$$\sigma_p = \sigma_v^{'} \tan^2\left(45 + \frac{\phi}{2}\right) + 2\,c\,\tan\left(45 + \frac{\phi}{2}\right)$$

Rankine passive earth pressure coefficient, K_p

$$\sigma_p = \sigma_v^{'} K_p + 2\,c\,\sqrt{K_p}$$

PASSIVE EARTH PRESSURE



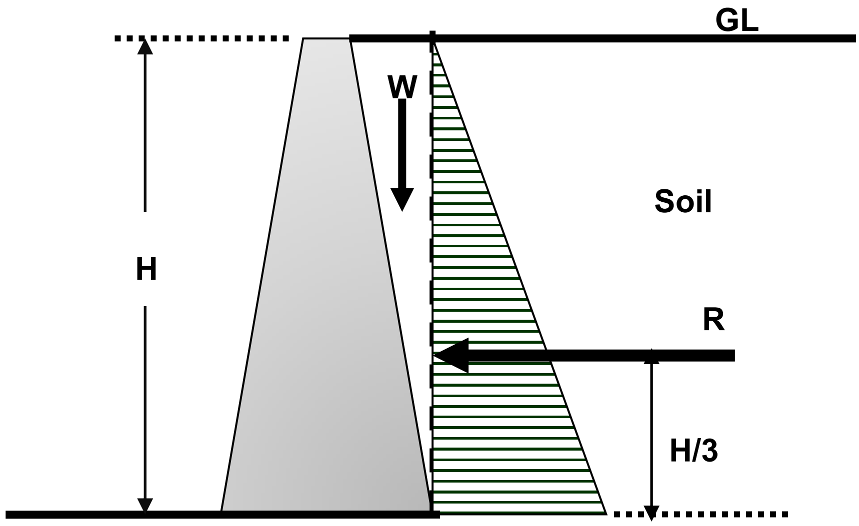


$$\sigma_p = \sigma_v^{'} K_p + 2\,c\,\sqrt{K_p} \qquad \sigma_v^{'} = \gamma\,z$$

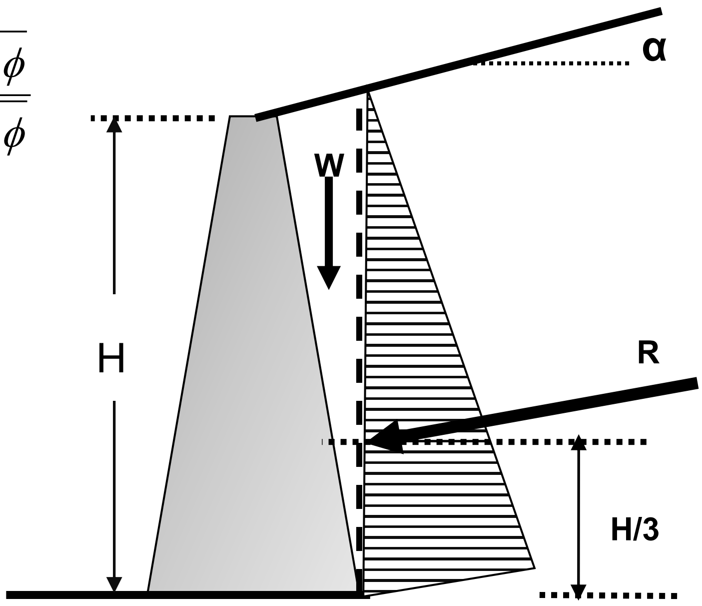
$$\sigma_p = \gamma\,H\,K_p + 2\,c\,\sqrt{K_p}$$

Earth pressure diagram

Case of inclined retaining structure:



Rankine active earth pressure for inclined granular backfill



$$K_a = \cos\alpha \frac{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi}}{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}}$$

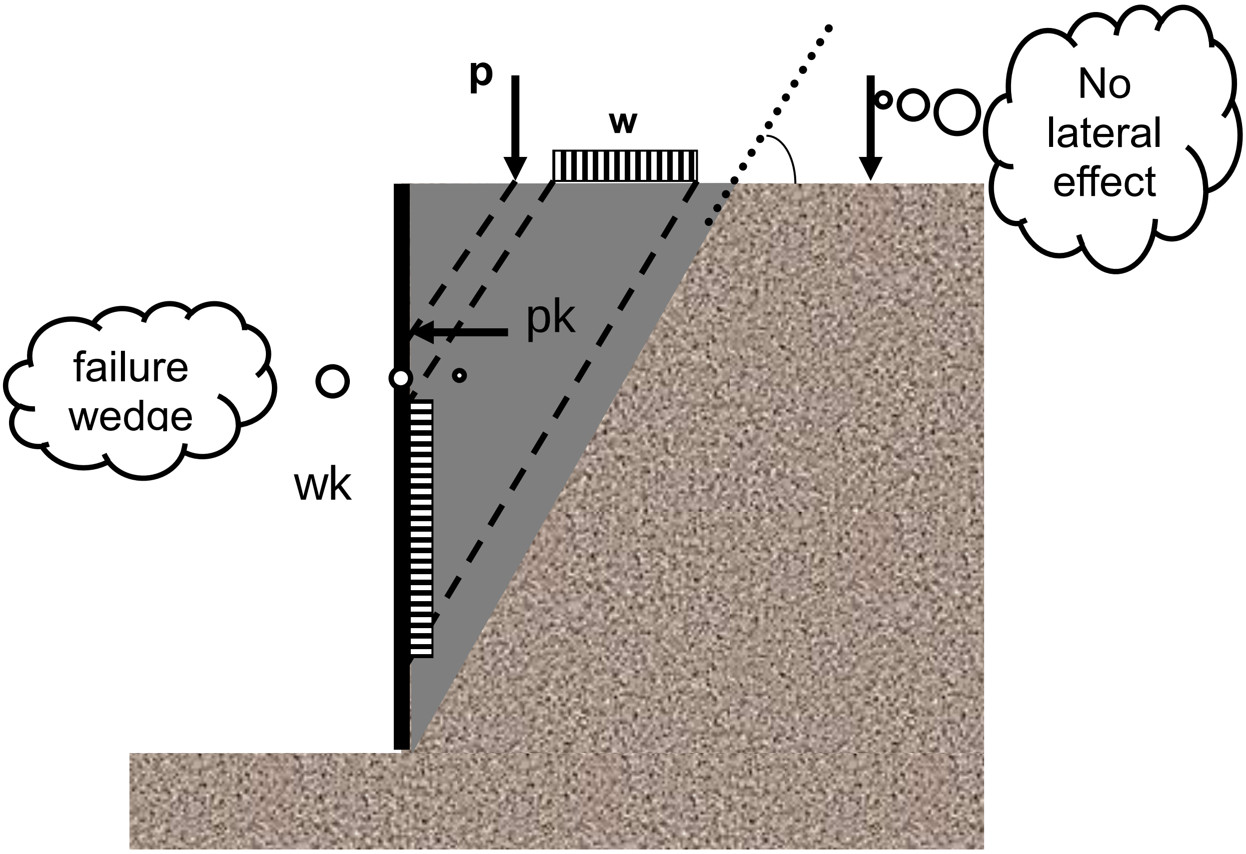
$$\sigma_a = \gamma\, z\, K_a$$

$$R = \frac{1}{2}\,\gamma\, H^2\, K_a$$

For the passive earth pressure

$$K_p = \cos\alpha \frac{\cos\alpha + \sqrt{\cos^2\alpha - \cos^2\phi}}{\cos\alpha - \sqrt{\cos^2\alpha - \cos^2\phi}}$$

Active & passive earth pressure due to surcharge:



EXAMPLE:

For the retaining wall shown, assume that the wall can yield sufficiently to develop active state. Draw the pressures diagrams and determine the active force per unit length of the wall as well as the location of the resultant line of action.

$K_{a(1)}$ = $\tan^2(45-15) = 0.333$

$K_{a(2)}$ = $\tan^2(45-18) = 0.26$

At z = 3 m

$$\sigma_v^{'} = 16\,x\,3 = 48\,kN/m^2$$

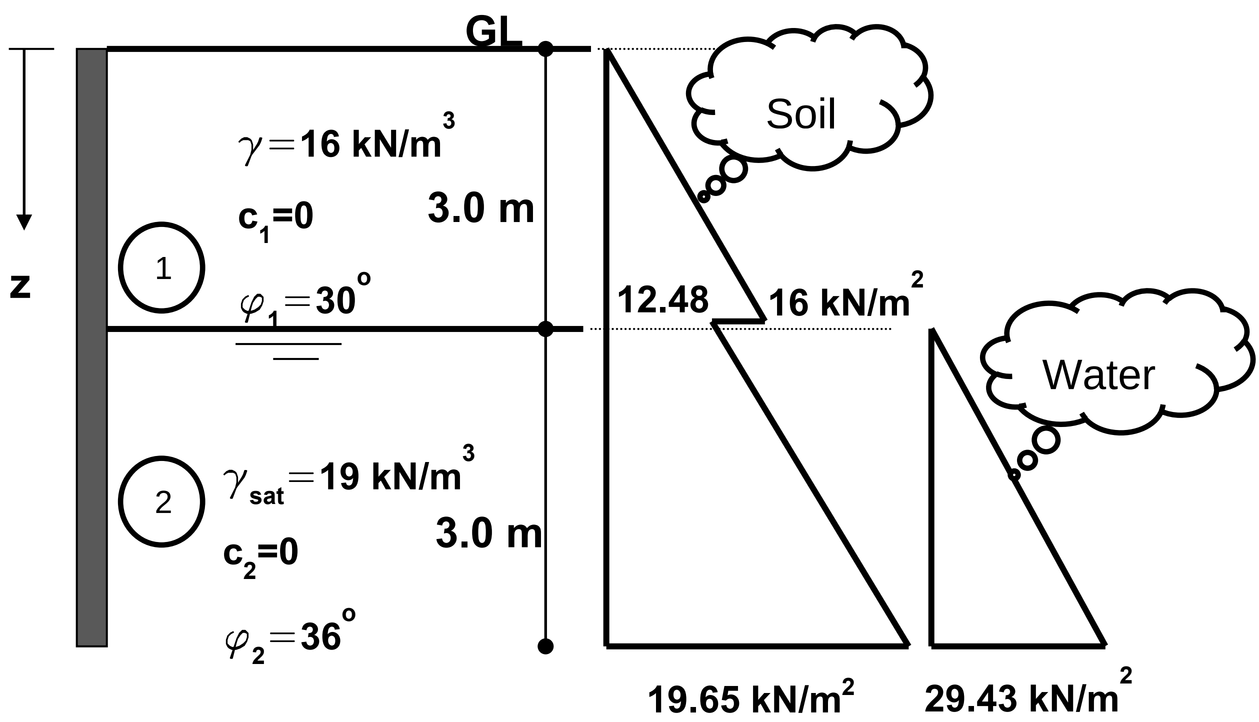
$$\sigma_{a-}^{'} = 0.333\,x\,48 = 16\,kN/m^2$$

$$\sigma_{a+}^{'} = 0.26\,x\,48 = 12.48\,kN/m^2$$

At z = 6 m

$$\sigma_v^{'} = 16\,x\,3 + (19-9.81)(3) = 75.57\,kN/m^2$$

$$\sigma_a^{'} = 0.26\,x\,75.57 = 19.65\,kN/m^2$$



SOIL INVESTIGATION

SOIL INVESTIGATION

PURPOSE OF SOIL INVESTIGATION:

1. Selecting the type and depth of foundation
2. Evaluating the load-bearing Capacity of the foundation
3. Estimating the expected settlement
4. Determining the location of the water table
5. Predicting lateral earth pressure
6. Establishing construction methods

SOIL INVESTIGATION PROGRAM:

- Collection of preliminary Information
- Visual inspection
- Site Investigation

Collection of preliminary information:

- Information regarding type of structure
- For buildings, approximate columns loads and spacing, basement requirements are required.
- For bridges, approximate span lengths loads on piers and abutments are required.
- General idea about of soil around the proposed site.

Visual inspection:

Visual inspection of the site is required to obtain the following data:

1. General topography of the site.
2. Soil stratification from deep cuts.
3. Type of vegetation.
4. Ground water levels.
5. Reported problems in nearby buildings.

Site investigation:

- Planning
- Making test boreholes
- Collecting soil samples

PLANNING:

The following tables:-

جدول (١-١٣) متطلبات تحديد عدد الجسات بالمواقع المختلفة

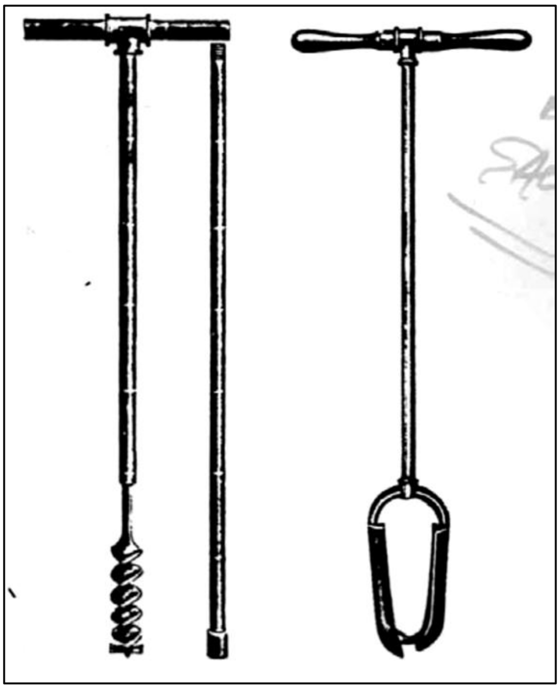
مناطق البحث	تخطيط الجسات
المواقع العمرانية	تخطط الجسات الإسترشادية على شبكة من ٢٠٠×٢٠٠ متراً إلى ٤٠٠×٤٠٠ متراً وفى حالة الأبحاث التفصيلية يزداد عدد الجسات للحصول على قطاعات جيولوجية دقيقة .
مواقع البناء المعتادة	يتم إختيار الجسات بصفة عامة بواقع جسة لكل ٣٠٠ مترمربع ، ولا يقل عددها عن جستين فى حالة ما يكون المسطح من ١٠٠ متر إلى ٣٠٠ متر مربع . أما فى حالة المبانى التى يقل مسطحها عن ١٠٠ متر مربع كغرف الحراسة وغرف المولدات الكهربائية فيكتفى بعمل جسة واحدة مع الاسترشاد بالجسات المجاورة إذا أمكن ذلك ، وإلا فيتم عمل جستين على الأقل . وفى حالة المنشآت ذات المسطحات الكبيرة يتحدد عدد الجسات بواقع جسة لكل من ٣٠٠ متراً مربع إلى ٥٠٠ متراً مربعاً ، وذلك حسب المعلومات المتوفرة عن تكوينات التربة فى المنطقة . وفى حالة المنشآت الموزعة على مسطحات كبيرة مثل القرى السياحية أو ماشابه فيتوقف عدد الجسات على المسافة بين المنشآت المختلفة وحجم هذه المنشآت مع الإسترشاد بما تقدم.
السدود وخزانات المياه والترع والجسور والحوائط الساندة	يتم إختيار الجسات بحيث تكون المسافة بينها فى حدود من ٥٠ متراً إلى ٢٠٠ متراً. وتقل المسافة بين الجسات عن منطقة منتصف المنشأ لتصبح حوالى ٣٠ متراً فى حالة السدود أو المناطق الأكثر تحميلاً من الناحية الإستاتيكية . وفى جميع الأحوال يتوقف عدد الجسات على طول المنشأ ونوعية طبقات التربة المتوقعة.
خطوط المياه والصرف الصحى	يتم عمل جسة كل ٢٥٠ متراً فى الأراضى الزراعية، وجسة كل ١٥٠ متراً أو أقل فى الأراضى الصحراوية، وجسة فى كل من أماكن غرف المحابس وغرف التفتيش والمطابق.
خطوط كهرباء الضغط العالى وأبراج الاتصالات	يتم عمل جسة أو إثنين بموقع كل برج ويعتمد ذلك على مساحة الأساس المتوقع للبرج .وفى حالة الأبراج ذات الشد فتعمل جسة فى موقع الشداد إذا لزم الأمر.
أكتاف الكبارى	يتم إختيار الجسات بحيث تكون المسافة بينها فى حدود من ١٠ إلى٢٠ متراً طبقاً لتجانس طبقات التربة وبعدد لايقل عن جستين بموقع كل كتف .
الخزانات الأرضية ذات الأبعاد الكبيرة	يتم إختيار الجسات بحيث تكون المسافة بينها فى حدود من ١٠ إلى٢٠ متراً طبقاً لتجانس طبقات التربة .

جدول رقم (١-١٤) متطلبات تحديد أعماق الجسات بالمواقع المختلفة

مناطق البحث	أعماق الجسات
المواقع العمرانية	يتم عمل الجسات بعمق لايقل عن ١٠,٠٠ متراً كما يتم عمل حفر إستكشافية مفتوحة بأعماق لاتقل عن ٥,٠٠ متراً ما أمكن للحصول على عينات فى حالتها الطبيعية ، ويعمل جسات عميقة بواقع ١٠% من عدد الجسات المطلوبة .
مواقع البناء المعتادة	فى جميع الأحوال لاتقل أعماق الجسات عن ١٠,٠٠متراً. وتزاد أعماق الجسات طبقاً لطبيعة التربة بالموقع وطبيعة المنشأ وأحماله . وفى حالة الأساسات العميقة (مثل الخوازيق) يجب أن يصل عمق الجسات إلى ٥,٠٠ متراً على الأقل عن النهاية المتوقعة لإرتكاز الخوازيق.
السدود وخزانات المياه والترع والجسور والحوائط الساندة	يجب أن لاتقل أعماق الجسات عن مرتين الإرتفاع الحر للحائط مقاساً من منسوب الأرض أمام الحائط . وفى حالة الجسور لاتقل عمق الجسة عن مرة ونصف العرض الكامل لقطاع الجسر خلال الطبقات المتجانسة ، مع زيادة هذا العمق فى حالة ظهور طبقات ضعيفة . كما يجب أن تصل أعماق الجسات إلى عمق أكبر من عمق مستوى سطح الإنهيار المحتمل فى حالة دراسة الميول، أو أن تصل الجسات إلى عمق كافى للوصول للطبقات الصلبة .
خطوط المياه والصرف	لايقل عمق الجسة عن ٥ متر وبحيث يكون عمق الجسة أسفل الراسم السفلى للماسورة بقدر ٦ مرات قطر الماسورة أو ٣ متر أيهما أكبر. وفى حالة الأعمال الصناعية على الخط لايقل عمق الجسة عن ١٠ متراً.
خطوط كهرباء الضغط العالى وأبراج الاتصالات	لا يقل عمق الجسة عن ١٥ متراً. وفى حالة أبراج الشد والأبراج ذات إرتفاع أكبر من ١٠٠ متراً يكون عمق الجسة ٢٠ متراً على الأقل .
أكتاف الكبارى	لايقل عمق الجسات عن ١٠,٠٠ متراً وتزاد أعماق الجسات طبقاً لطبيعة التربة .
الخزانات الأرضية ذات الأبعاد الكبيرة	يتم عمل الجسات بأعماق كبيرة خاصة فى حالة وجود التربة اللينة أو الرخوة . وعلى كل حال فيتم عمل ٢٠% من عدد الجسات على الأقل أو جستين بعمق لايقل عن قطر الخزان أو البعد الأصغر للخزانات المستطيلة.

Boring Types:

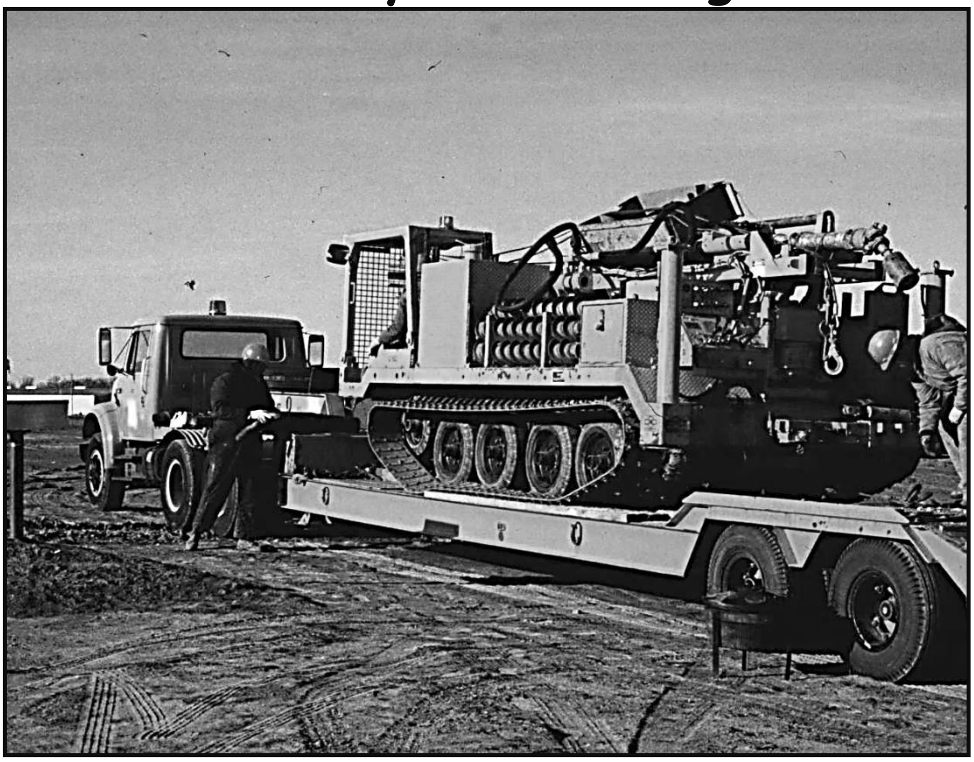
- Auger boring
- Continuous flight auger
- Wash boring
- Rotary drilling

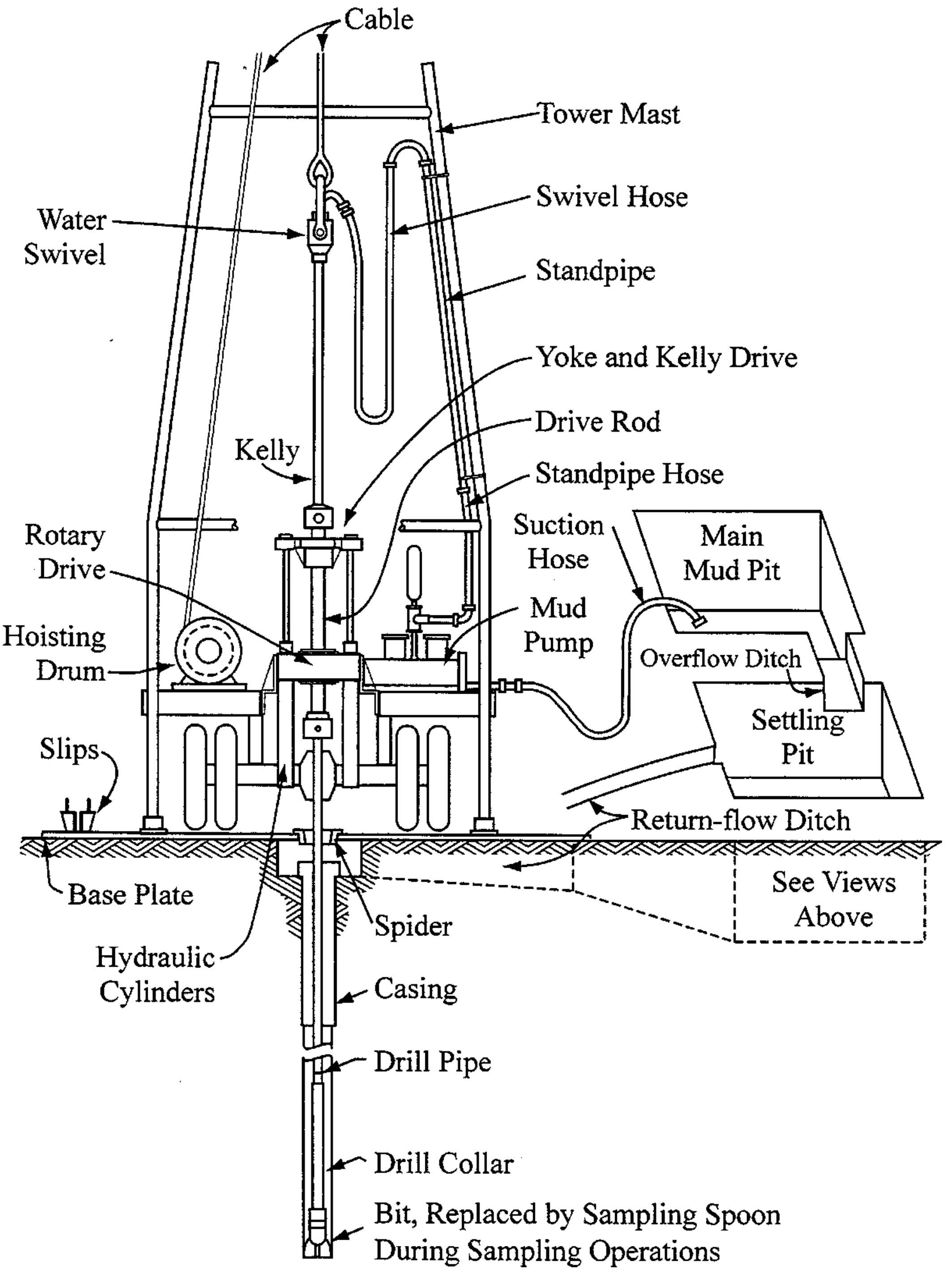


Hand Auger

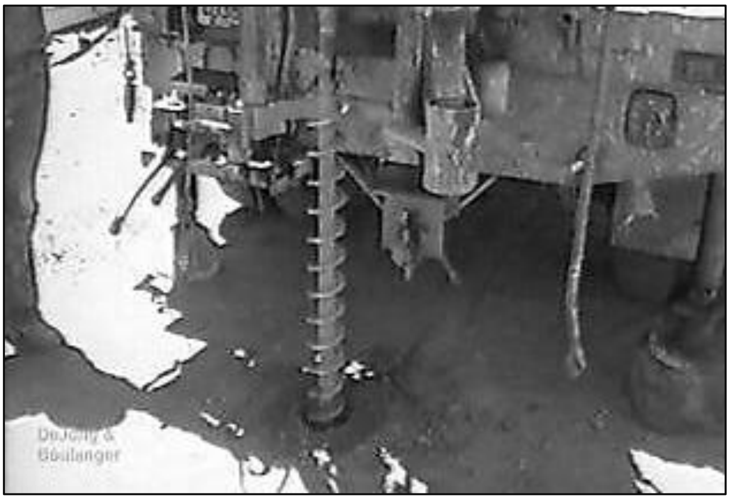
Auger Drilling

Rotary wash boring





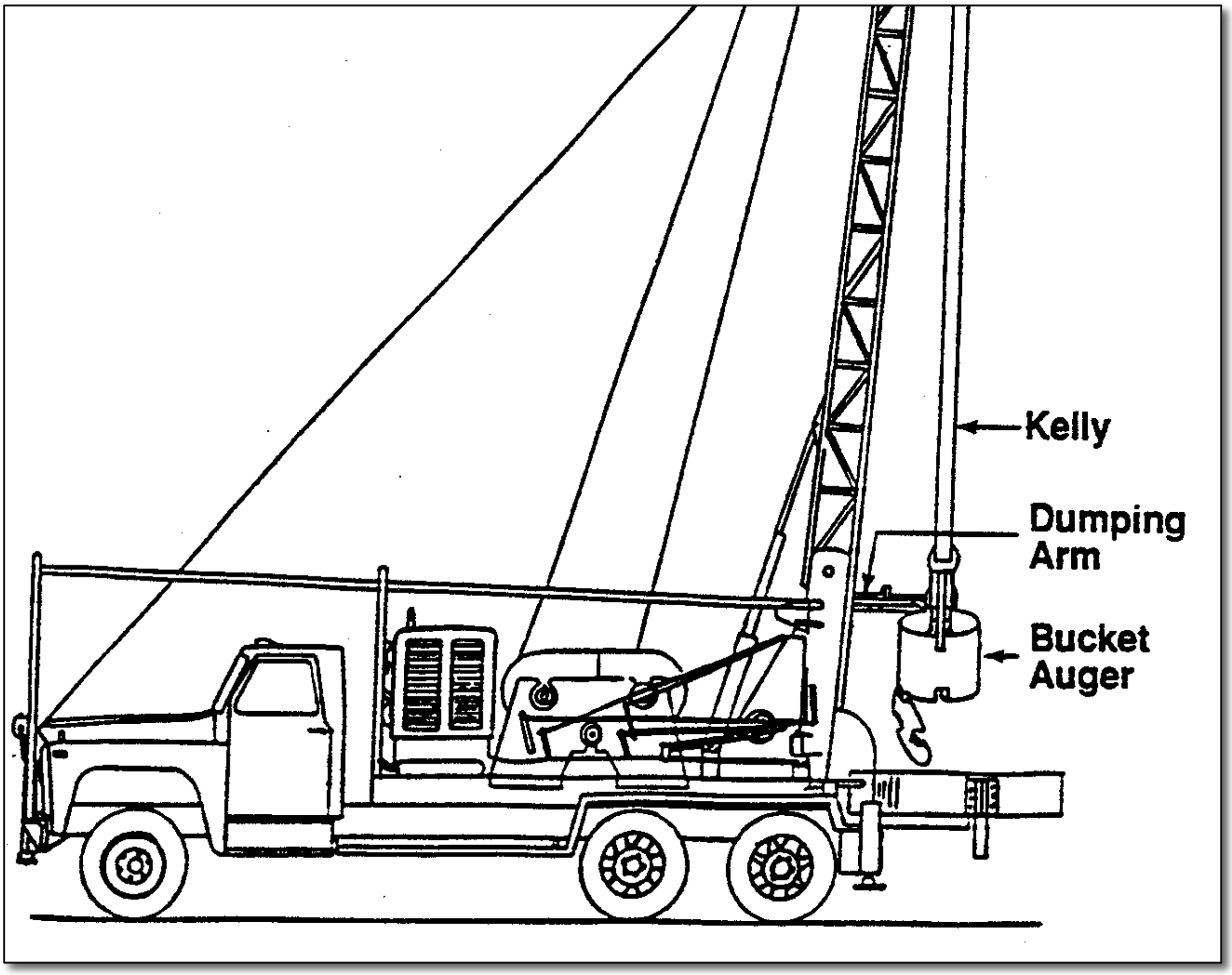
Rotary wash boring



Solid Augers

Hollow stem augers

Rotary wash borings



Bucket auger borings

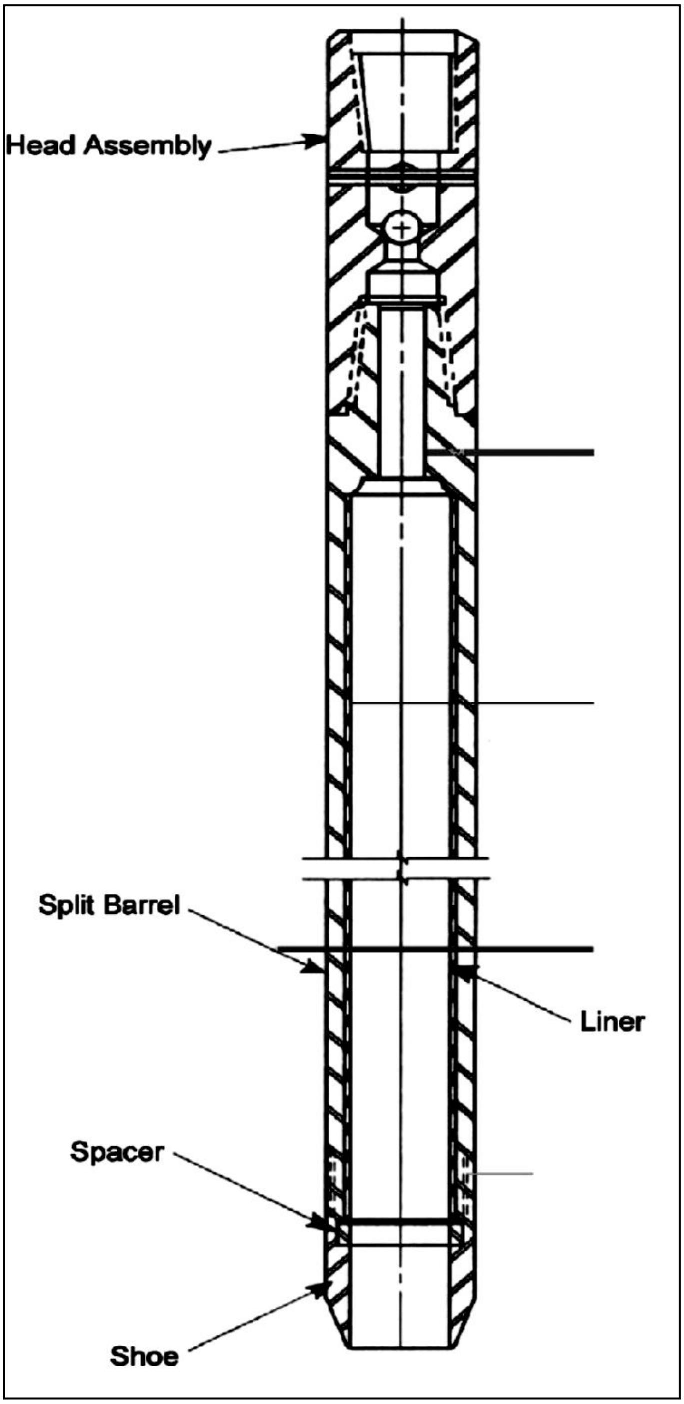
SOIL SAMPLING:

Three types of soil samples:

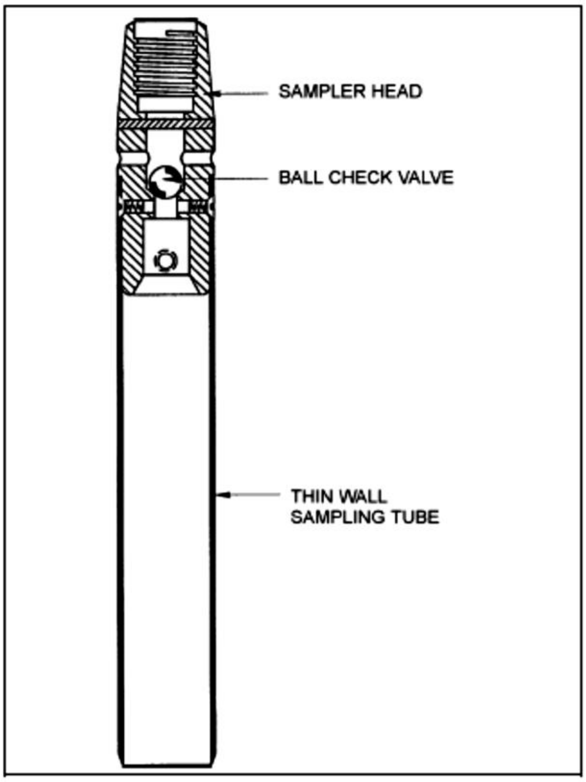
- **Disturbed samples**
  - ♦ Bulk samples (from auger cuttings or test pit excavations)
  - ♦ Drive samples (e.g., split-barrel)
- **Partially undisturbed**
  - ♦ Continuous Hydraulic Push
- **Undisturbed samples**
  - ♦ Push Tubes (Shelby, Piston, Laval, Sherbrook)
  - ♦ Rotary & Push (Denison, Pitcher)
  - ♦ Block Samples

SAMPLER TYPES:

- Split-Spoon Sampling
- Scraper Bucket
- Thin Wall Tube
- Piston Sampler



Split-Spoon Soil Sampler



Schematic diagram of thethin-wall tube sampler

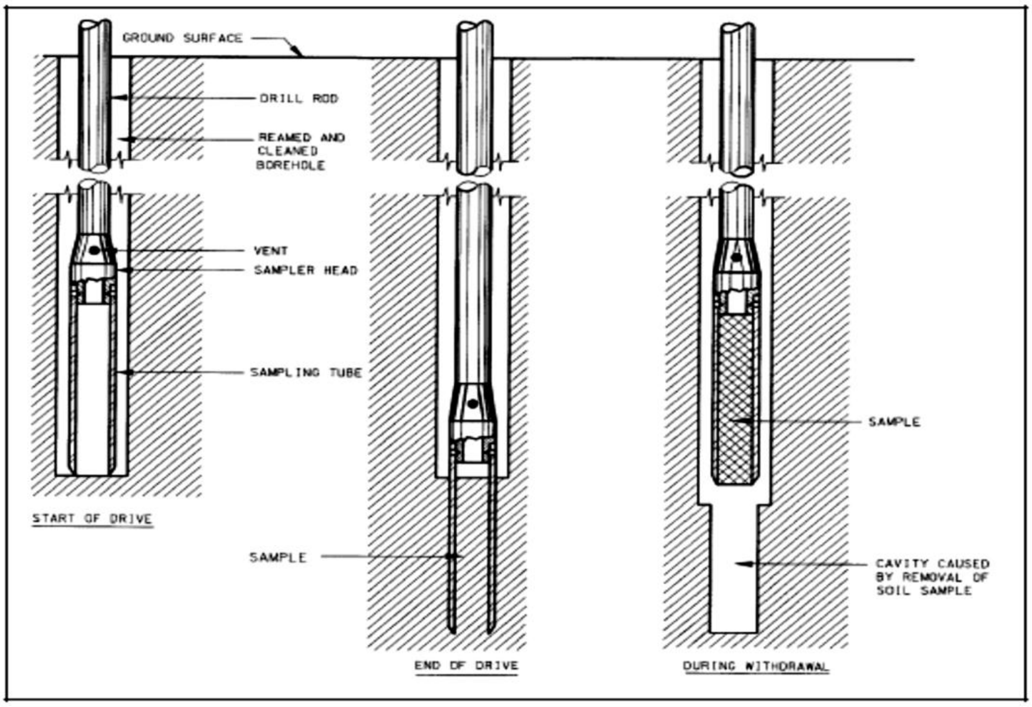
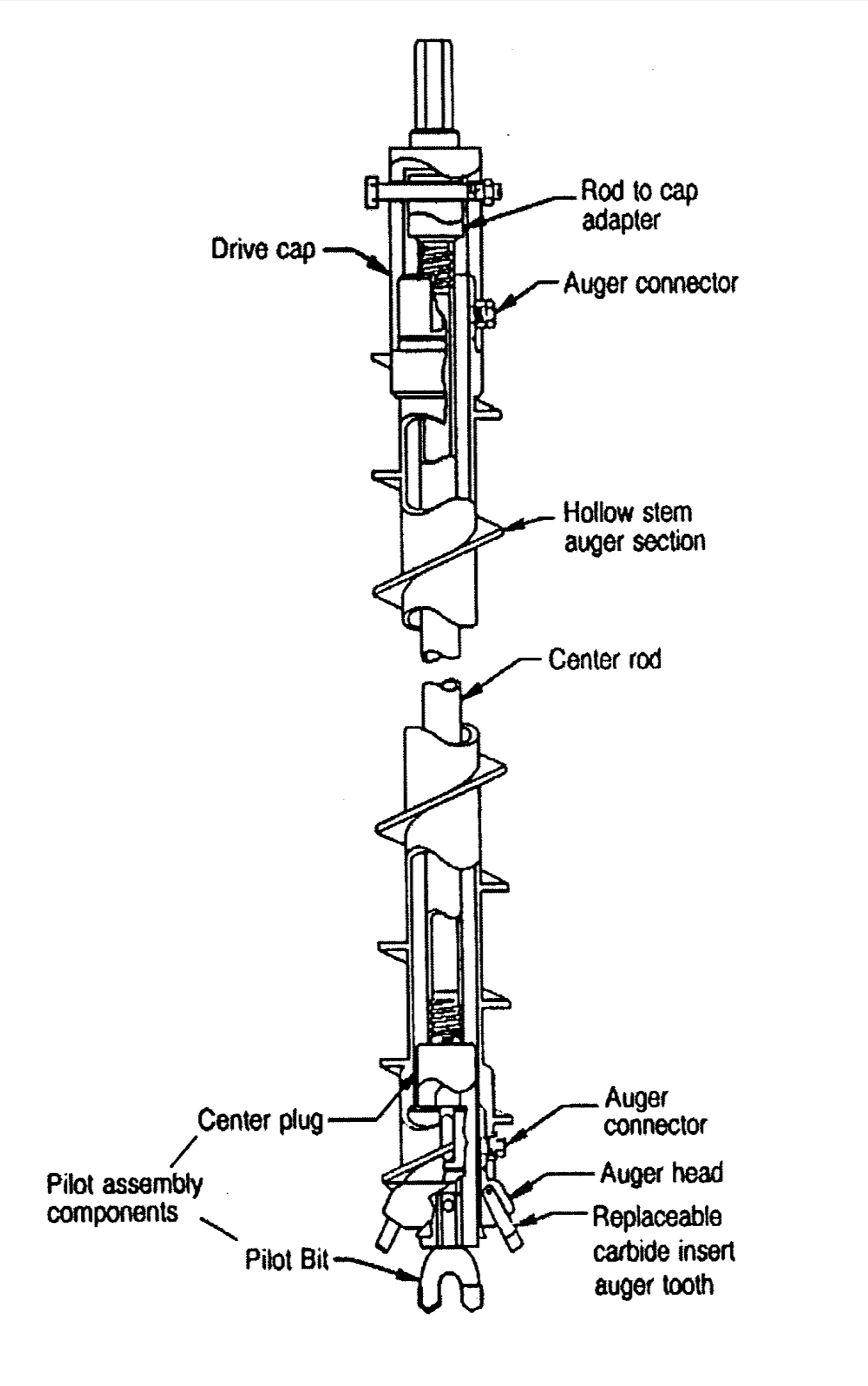
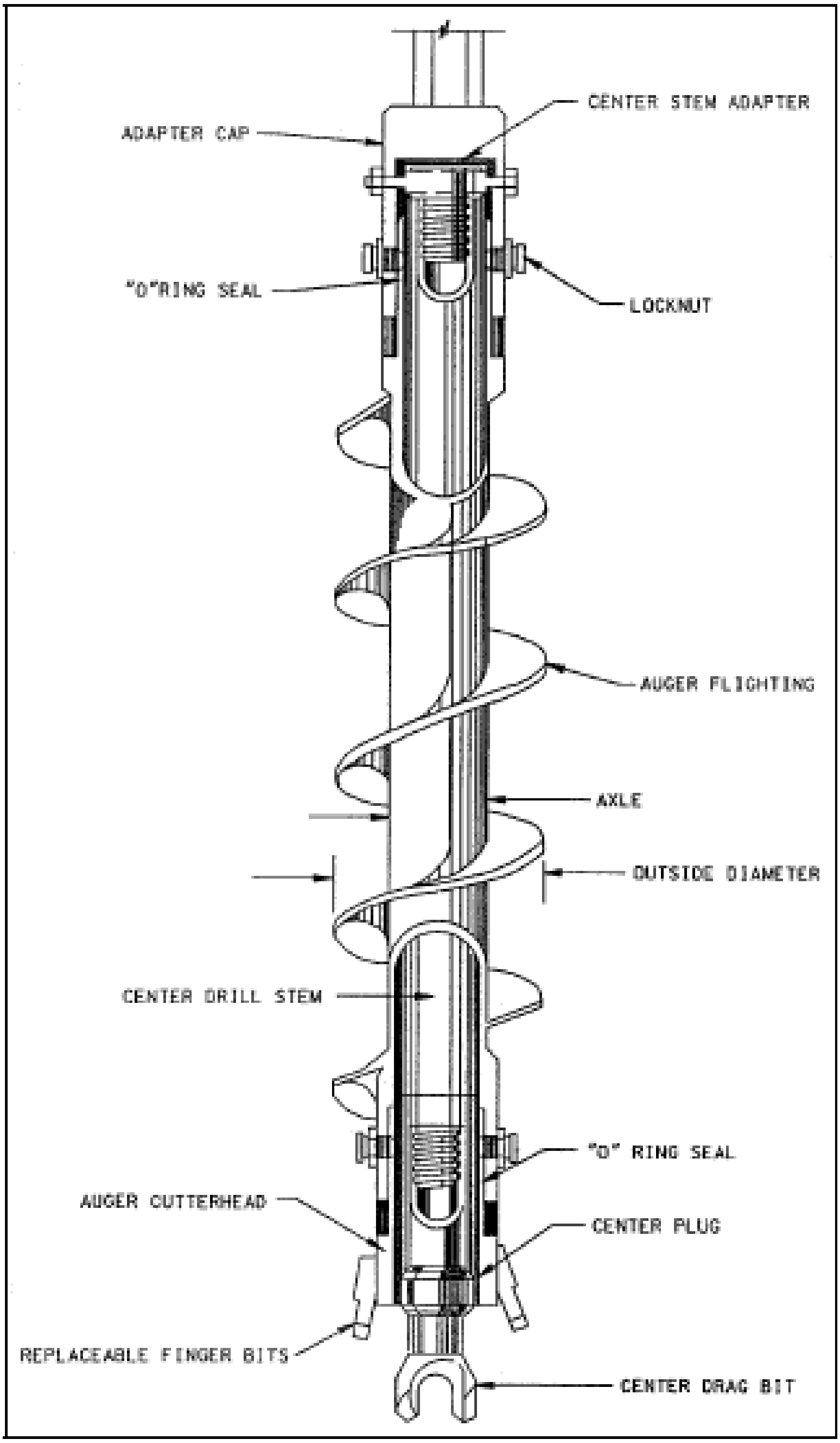


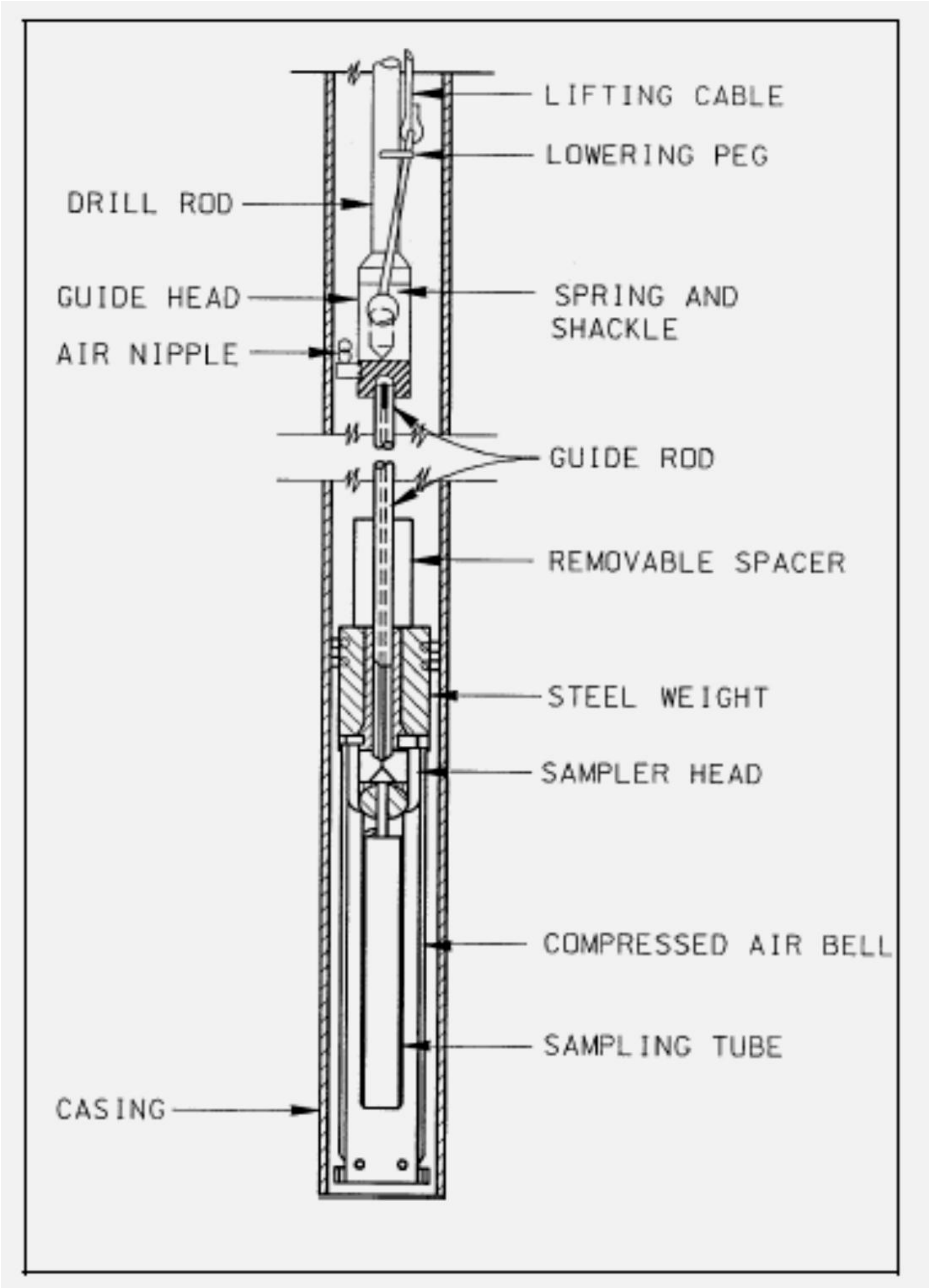
Diagram of the sampling operation using the thin-wall tube sampler



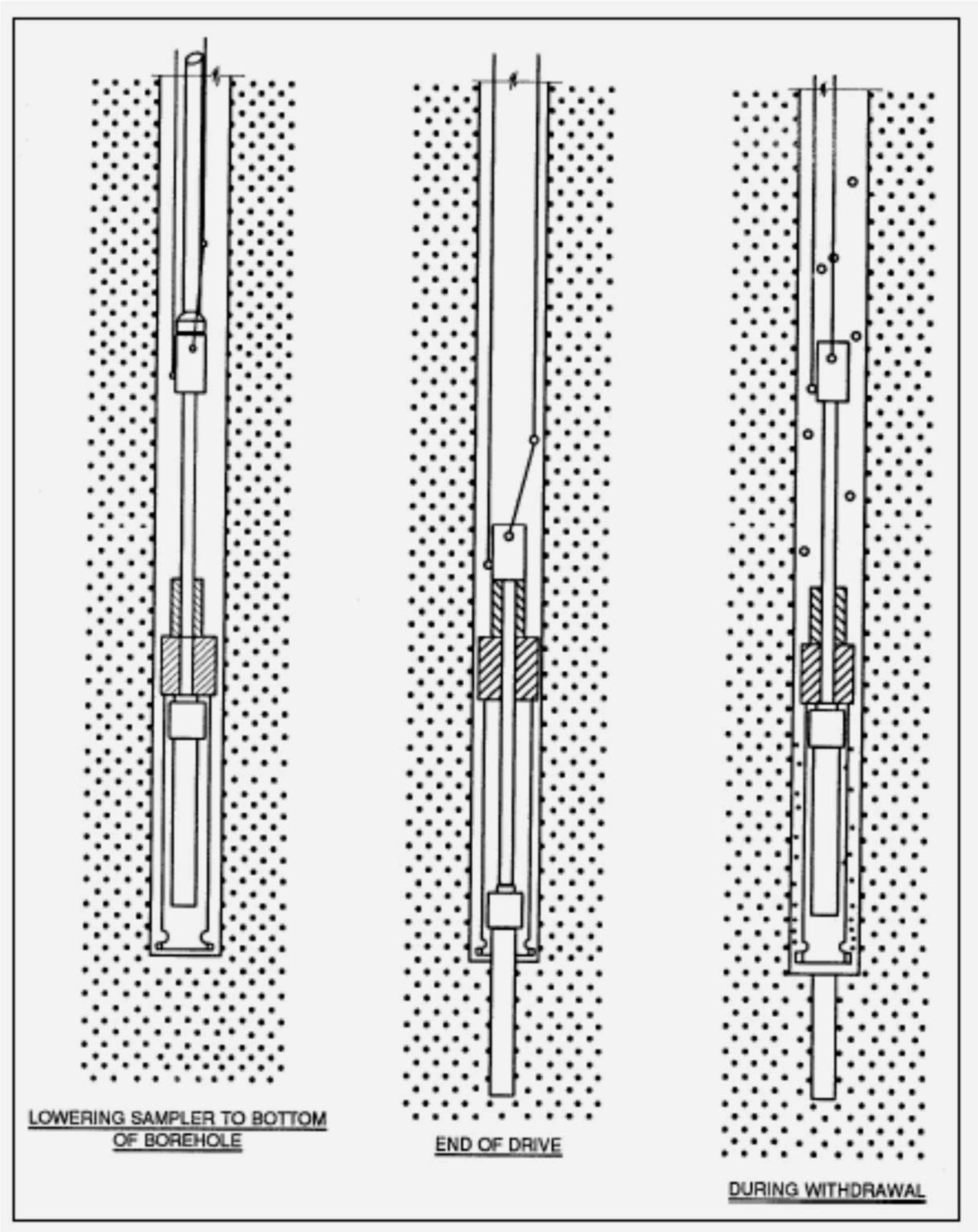
Typical components of a hollow-stem auger



Isometric drawing of the hollow-stem auger with the center drag bit which can be used with soil sampling



Schematic drawing of the Bishop sand sampler



Schematic diagram illustrating the operation of the Bishop sand sampler

OBSERVATION OF WATER TABLES:

- The water levels in a borehole during field exploration should be recorded.
- In soils with high hydraulic conductivity, the water level in a borehole will stabilize about 24 hrs after completion of the boring.
- The depth of the water table can be recorded by lowering a chain or a tape into the borehole.
- In highly impermeable layers, the water level in a borehole requires longer time to stabilize, hence a piezometer is used for accurate measurement of water level.

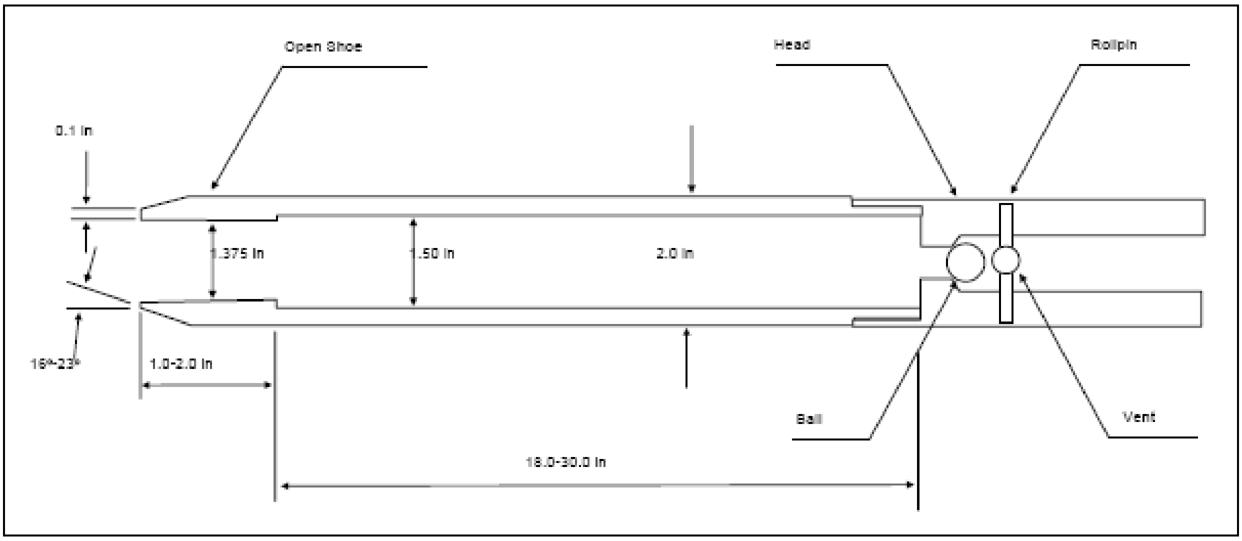
Field Testing:

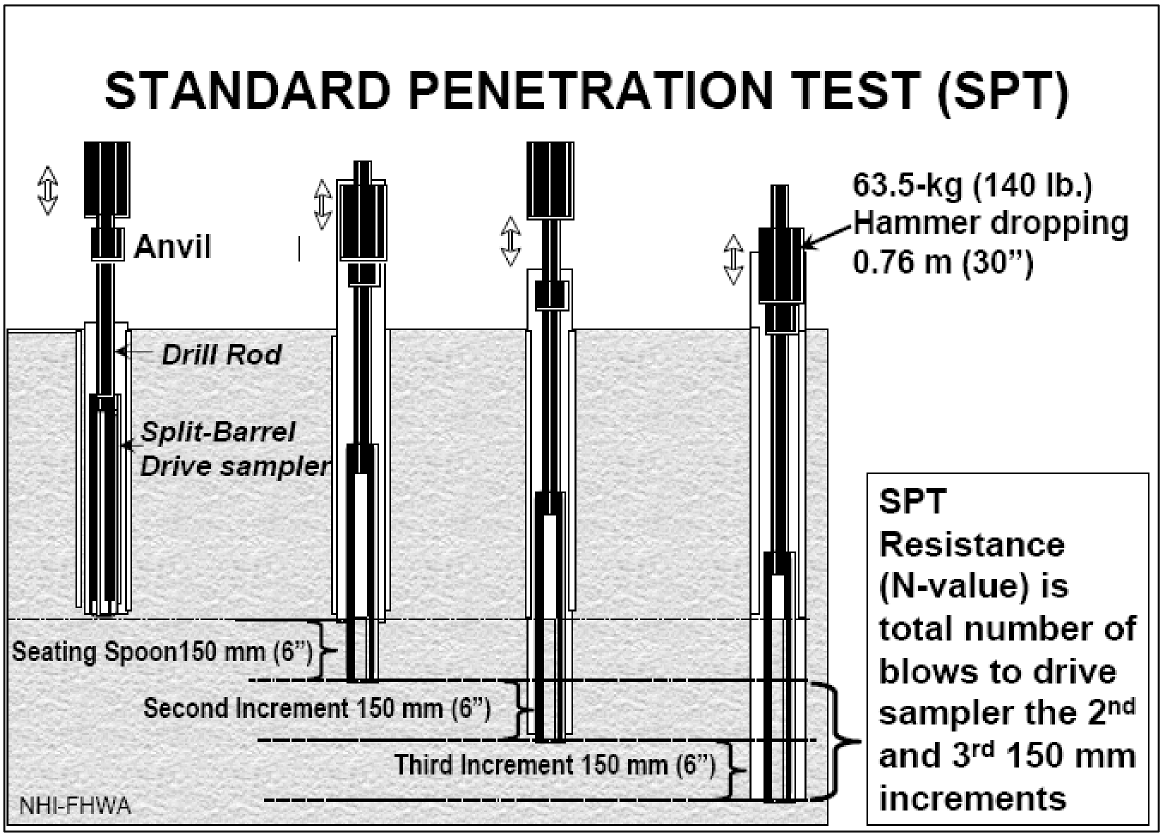
- Standard Penetration Test
- Vane Shear Test
- Cone Penetration Test
- Pressuremeter Test
- Dilatometer Test

Standard Peneteration Test (SPT):

- General equipment and procedure
- Corrections
- Correlation between SPT and soil properties

Split-Spoon Sampler





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Correlation between Spt and Soil Properties:

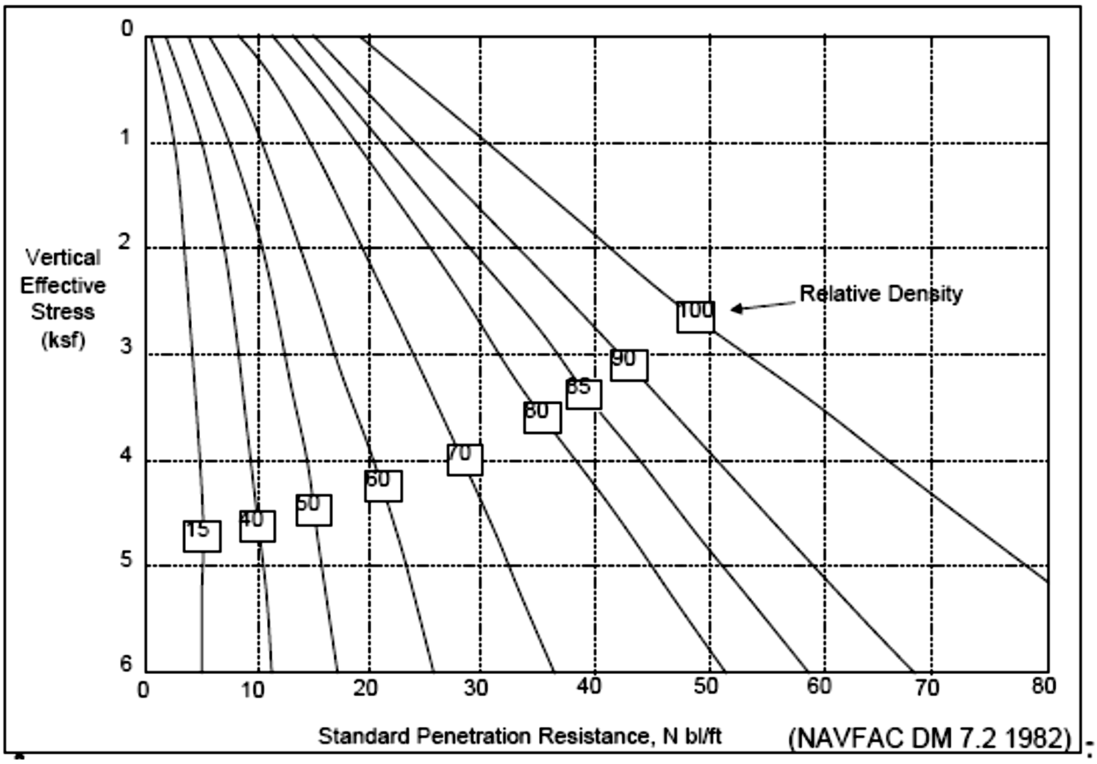
- Relative density
- Effective stress friction angle
- Undrained shear strength
-

CORRELATIONS

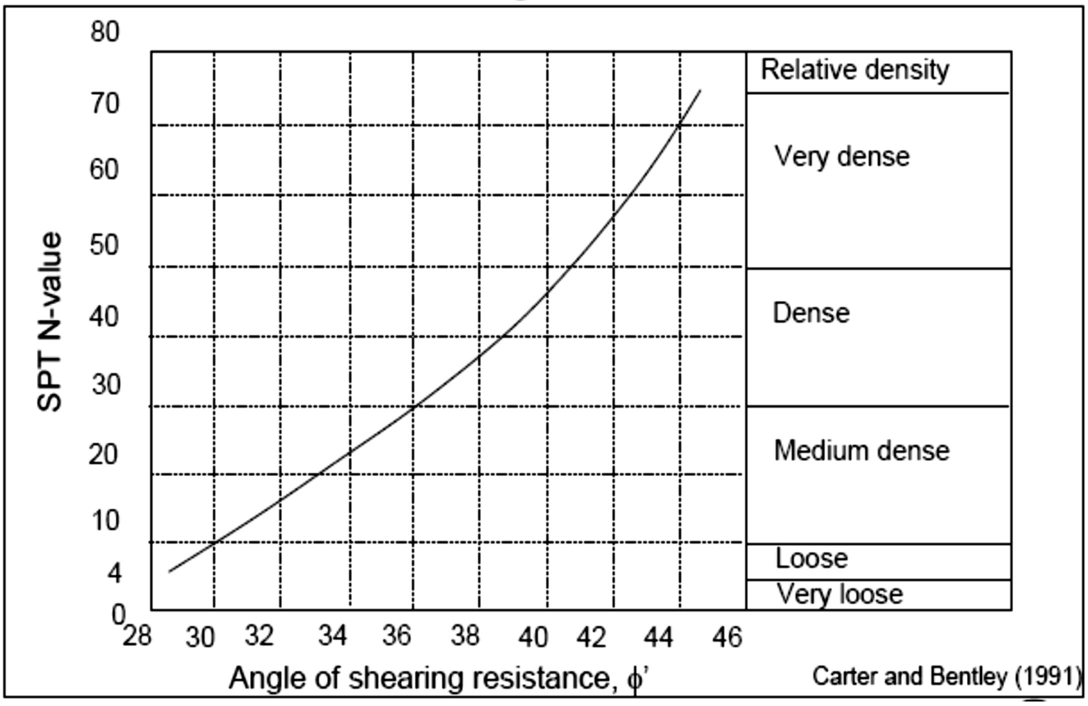
Meyerhoff (1956)

State of Packing	Relative Density	Standard Penetration Resistance (N)	Static Cone Resistance (q_c)	Angle of Internal Friction (ϕ')
	Percent	Blows / ft	Tsf or kgf/cm²	Degrees
Very Loose	< 20	< 4	< 20	< 30
Loose	20 – 40	4 –10	20 – 40	30 – 35
Compact	40 – 60	10 –30	40 – 120	35 – 40
Dense	60 – 80	30 – 50	120 – 200	40 – 45
Very Dense	> 80	> 50	> 200	> 45

Relative Density



Effective Stress Friction Angle



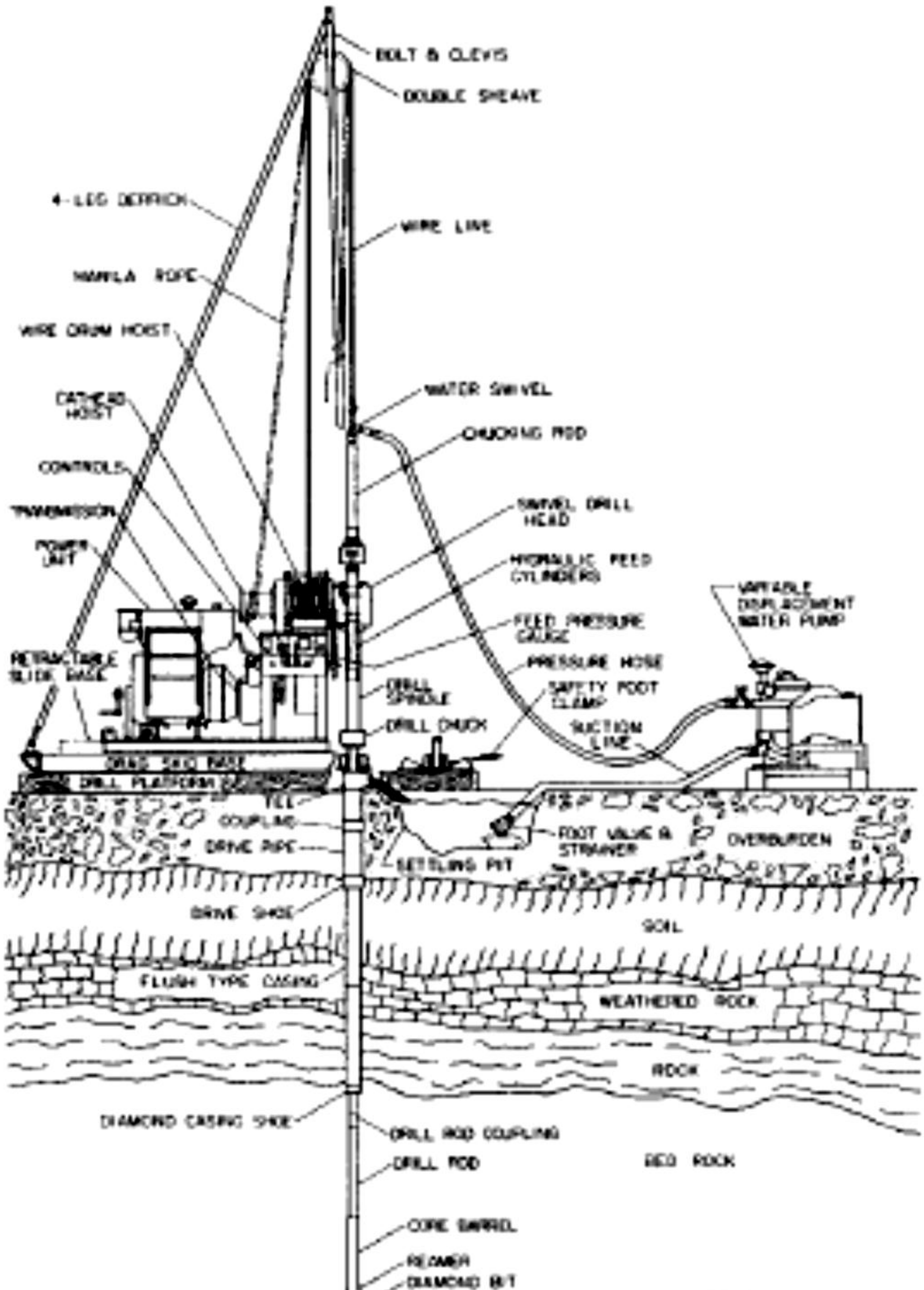
CORRELATIONS

Undrained Shear Strength

Soil Consistency	SPT N	S_u (psf)	S_u (kPa)
Very Soft	< 4	< 250	< 12
Soft	2 – 4	250 – 500	12 – 25
Medium	4 - 8	500 – 1000	25 – 50
Stiff	8 – 15	1000 – 2000	50 – 100
Very Stiff	15 – 30	2000 – 4000	100 – 200
Hard	> 30	> 4000	> 200

Terzaghi et al. (1996)

CORING OF ROCKS:



Rock core-drill set-up

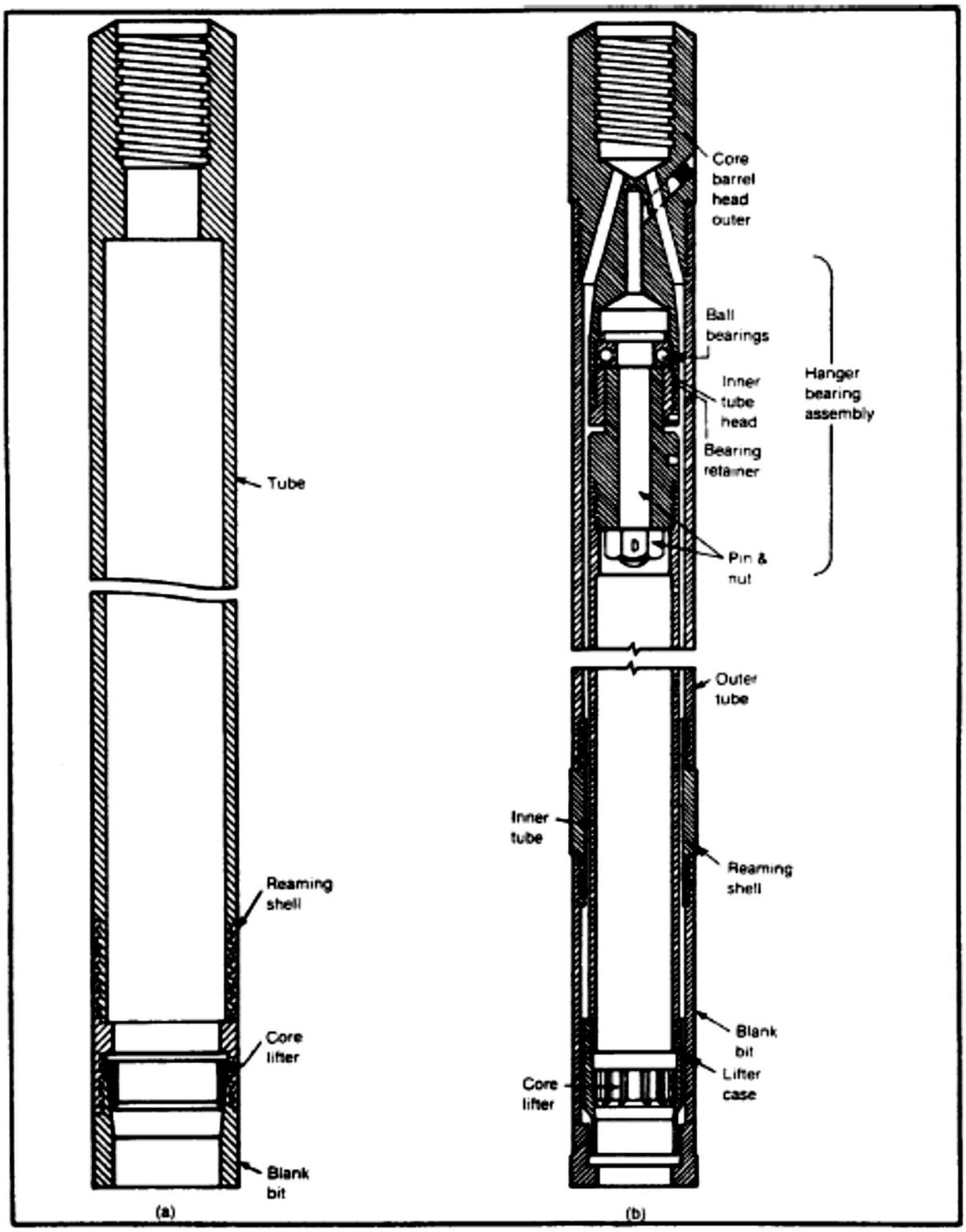
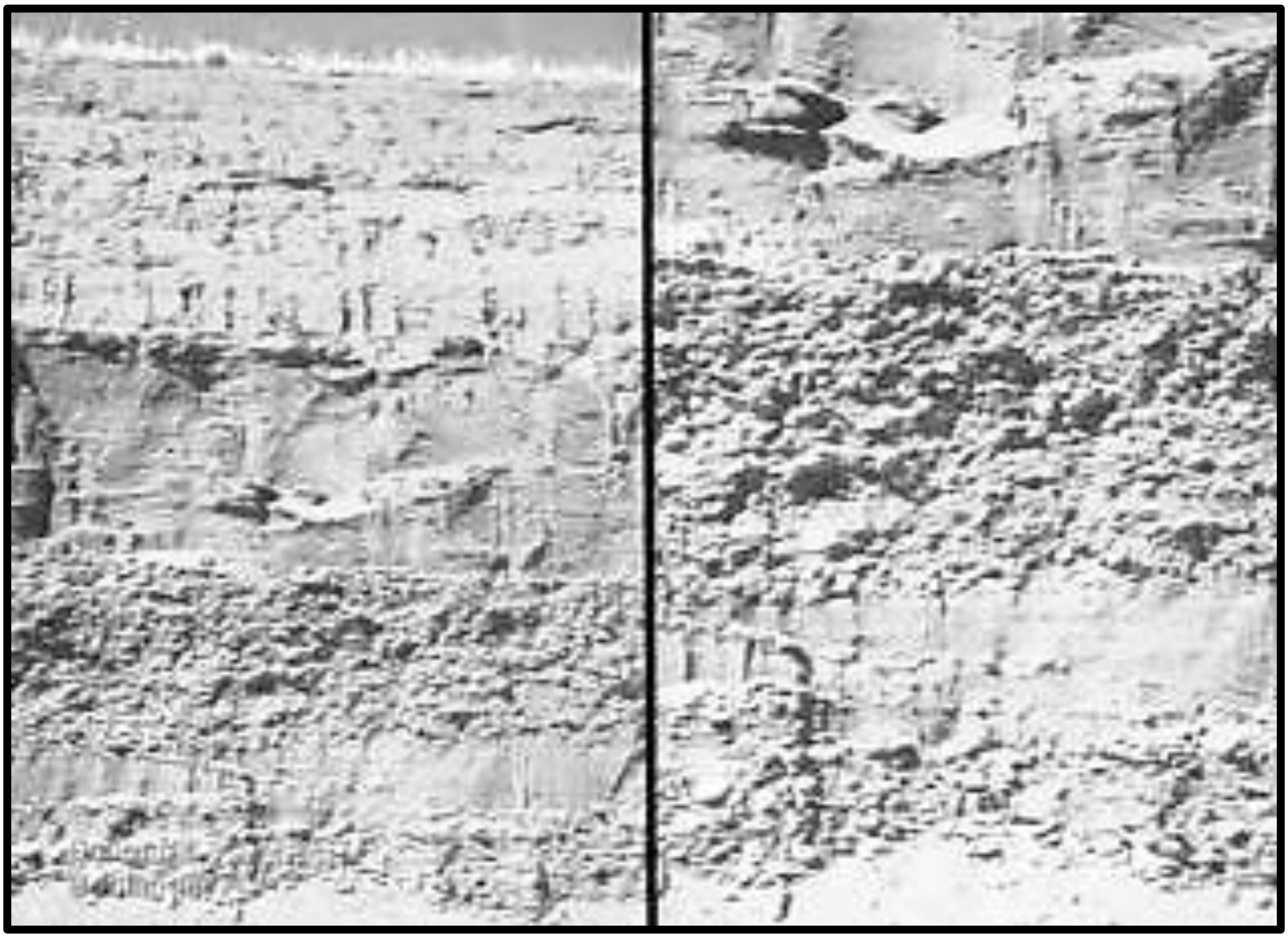


Diagram of two-types of core barrels

(a) Single tube (b) Double tube

STRATIFICATION:



Preparation Of Boring Logs:

The data gathered from each borehole is presented in the so called *boring log*. The driller should include the following data in the site:

1. Drilling company.
2. Number and type of boring.
3. Date of boring.
4. Subsurface stratification by visual observation.
5. Water levels.
6. SPT results.
7. Number, type, and depth of samples.

After completion of lab tests, the geotechnical engineer prepare the final log that include the data from the driller field log and test results.

Subsoil Explolration Report:

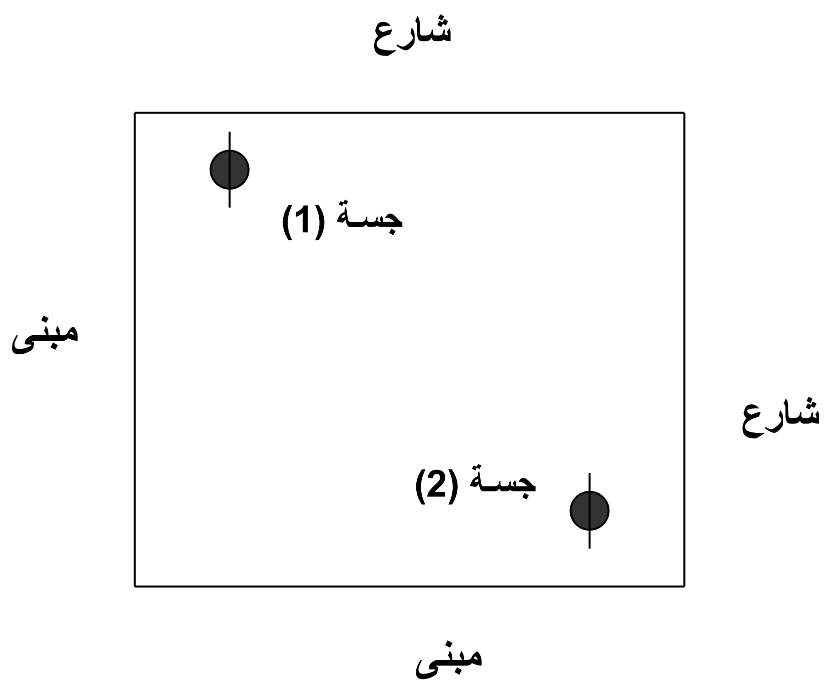
- *After all the required information has been compiled, a soil exploration report is prepared.*
- *Each report should include the following items:*

1. The scope of the investigation.
2. A description of the proposed structure.
3. A description of the location of the site.
4. Geological setting of the site.
5. Details of the field exploration.

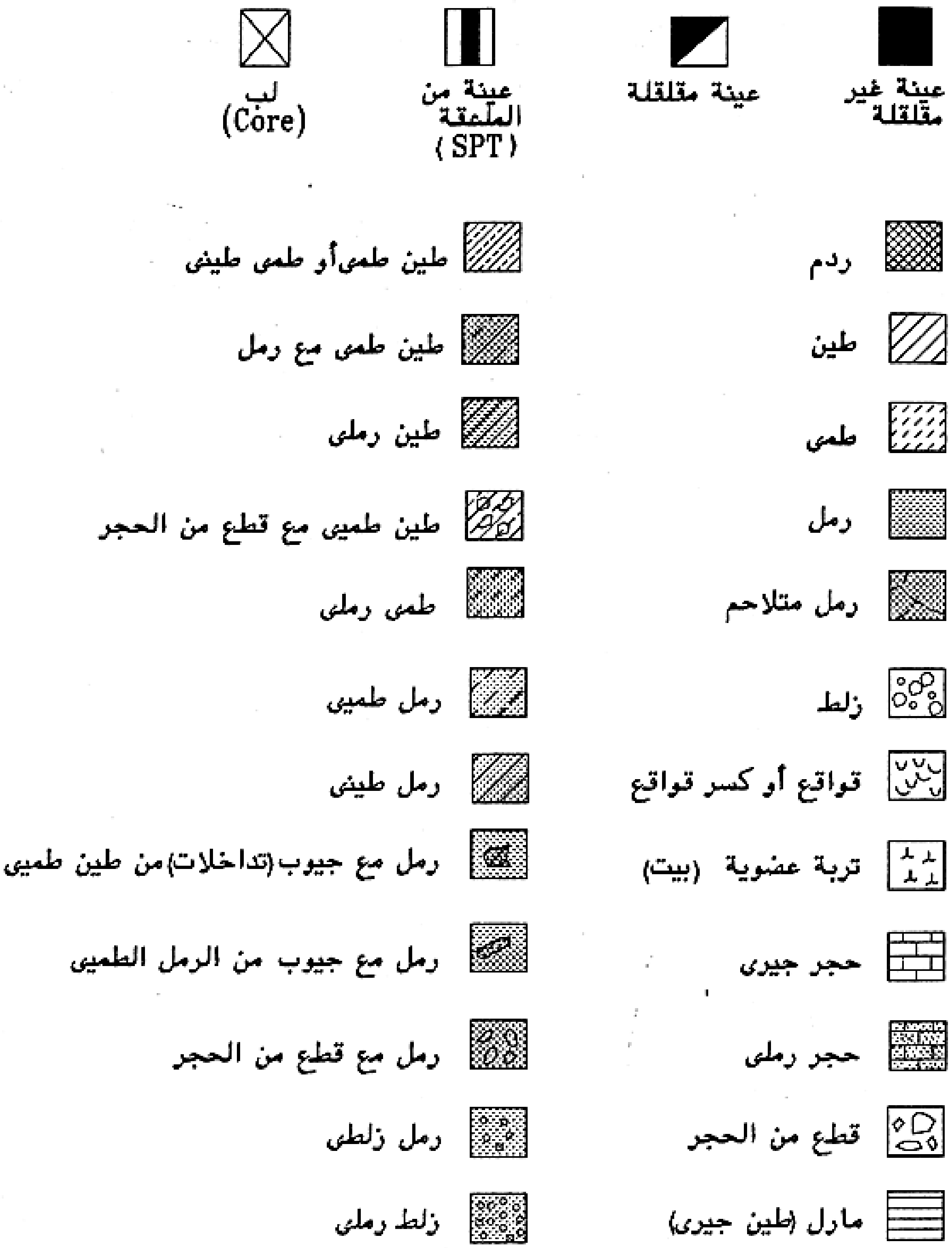
6. General description of the subsoil condition.
7. Water-table conditions.
8. Foundation recommendations.
9. Conclusions and limitations of the investigation.

- *Some graphs should be attached to the to the report as:*

1. Site location map.
2. A plan view of the locations of the borings.
3. Boring logs.
4. Laboratory test results.



شكل(1) – الموقع العام وأماكن الجسات



شكل (١-٣) توضيح أنواع العينات وأنواع التهشير فى قطاع الجسات

مشروع :	جسة رقم:	ملف رقم :
منسوب سطح الارض :	عمق المياه الارضية الابتدائى : النهائى :	تاريخ : تاريخ :
مهندس المعمل :	مهندس الموقع :	الحفار:
تاريخ :	المعدات :	لوحة رقم من

العمق (متر)	عينة رقم	عينة نوع	عدد الثلاثات القياسية (N)	الطبقة السمك (متر)	الطبقة القطاع	التصنيف والتوصيف	الضغط الغير محاط ميجانيوتن/م٢	معامل جودة الصخر (RQD) %	نسبة الاستخلاص (Recovery) %	الكثافة الطبيعية كيلونيوتن/م٣
	١			١،٢٠		ردم مكون من رمل وطمى وزلط متدرج - بنى				
١										
	٢			١،٧٠		رمل متوسط الى كبير الحبيبات متوسط الكثافة مع بعض الزلط الدقيق-أصفر				
٢										
	٣		٢٠							
٣										
	٤			٣،٦٠		حجر رملى متوسط الصلابة ـ متوسط التجوية ـ كتل رقيقة ـ عالى التكسير متوسط الحبيبات ـ بنى مصفر	١٢،٥	١٠	٤٠	٢١
٤										
	٥							٢٥	٥٥	٢٣
٥										
	٦						٢١،٠	٢٠	٦٠	٢٢
٦										
	٧						١٦،٠	١٥	٤٥	٢٠
٧										
	٨			٥،٥٠		حجر جيرى متوسط الصلابة ـ ضعيف الجودة ـ متوسط التجوية ـ أبيض مصفر	١٣،٥	١٥	٤٠	١٦
٨										
	٩							٣٠	٥٥	٢٢
٩										
	١٠						٢٠،٠	٣٥	٦٠	٢١
١٠										
	١١							٤٠	٧٠	٢٢
١١										
	١٢						٣٦،٠	٥٥	٨٥	٢٣
١٢	١٣							٤٠	٦٥	٢٢
						نهاية الجسة على عمق ١٢،٠٠ متر				
١٣										
١٤										
١٥										

شكل (١ – ١) تسجيل المعلومات والبيانات فى قطاع نموذجى لجسة فى طبقات صخرية

منسوب سطح الارض:	عمق المياه الارضية الابتدائى : النهائى :	تاريخ : تاريخ :
مهندس المعمل :	معندس الموقع :	الحفار:
تاريخ:	المعدات :	لوحة رقم من

عينة رقم	عينة نوع	عدد الدقات القياسية (N)	الطبقة السمك متر	الطبقة القطاع	التصنيف والتوصيف	الضغط الغير محاط كيلونيوتن/م٢	الكثافة الطبيعية كيلونيوتن/م٣
١ ٢			١,٥٠		ردم مكون من الطين والطمى والرمل ـ رمادى داكن		
٣ ش١ ٤ ش٢ ٥		٥ ٦ ٧	٥,٢٠		طين طمبى متوسط التماسك ـ بنى	٦٠ ٨٠ ٧٠ ٩٠ ٨٠	١,٦٠ ١,٨٠
٦ ٧		١١	١,٨٠		رمل دقيق الحبيبات متوسط الكثافة و آثار من الطمى ـ بنى		
٨ ٩ ١٠ ١١ ١٢ ١٣		١٨ ٢٤ ٢٧	٦,٥٠		رمل متدرج متوسط الكثافة و آثار من زلط دقيق-اصفر نهاية الجسة على عمق ١٥,٠٠متر		

شكل (١-٢) تسجيل المعلومات والبيانات فى قطاع نموذجى لجسة فى طبقات تربة