

# **Bearing Capacity Of Shallow Foundation**

## Bearing Capacity Of Shallow Foundation

- \* A foundation is required for distributing the loads of the superstructure on a large area.
- \* The foundation should be designed such that
  - a) The soil below does not fail in shear &
  - b) Settlement is within the safe limits.

## Basic Definitions :

### 1) Ultimate Bearing Capacity ( $q_u$ ) :

The ultimate bearing capacity is the gross pressure at the base of the foundation at which soil fails in shear.

### 2) Net ultimate Bearing Capacity ( $q_{nu}$ ) :

It is the net increase in pressure at the base of foundation that cause shear failure of the soil.

Thus,  $q_{nu} = q_u - \gamma D_f$  (overburden pressure)

### 3) Net Safe Bearing Capacity ( $q_{ns}$ ) :

It is the net soil pressure which can be safely applied to the soil considering only shear failure.

$$\text{Thus, } q_{ns} = q_{nu} / \text{FOS}$$

FOS - Factor of safety usually taken as 2.00 -3.00

### 4) Gross Safe Bearing Capacity ( $q_s$ ) :

It is the maximum pressure which the soil can carry safely without shear failure.

$$q_s = q_{nu} / \text{FOS} + \gamma D_f$$

5) **Net Safe Settlement Pressure ( $q_{np}$ ) :**

It is the net pressure which the soil can carry without exceeding allowable settlement.

6) **Net Allowable Bearing Pressure ( $q_{na}$ ) :**

It is the net bearing pressure which can be used for design of foundation.

Thus,

$$q_{na} = q_{ns} \quad ; \text{ if } q_{np} > q_{ns}$$

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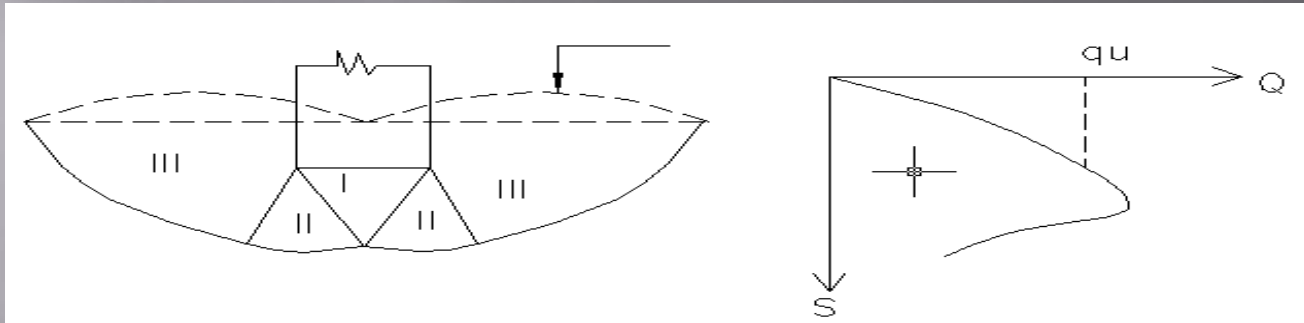
It is also known as **Allowable Soil Pressure (ASP)**.

## Modes of shear Failure :

Vesic (1973) classified shear failure of soil under a foundation base into three categories depending on the type of soil & location of foundation.

- 1) General Shear failure.
- 2) Local Shear failure.
- 3) Punching Shear failure

# General Shear failure –

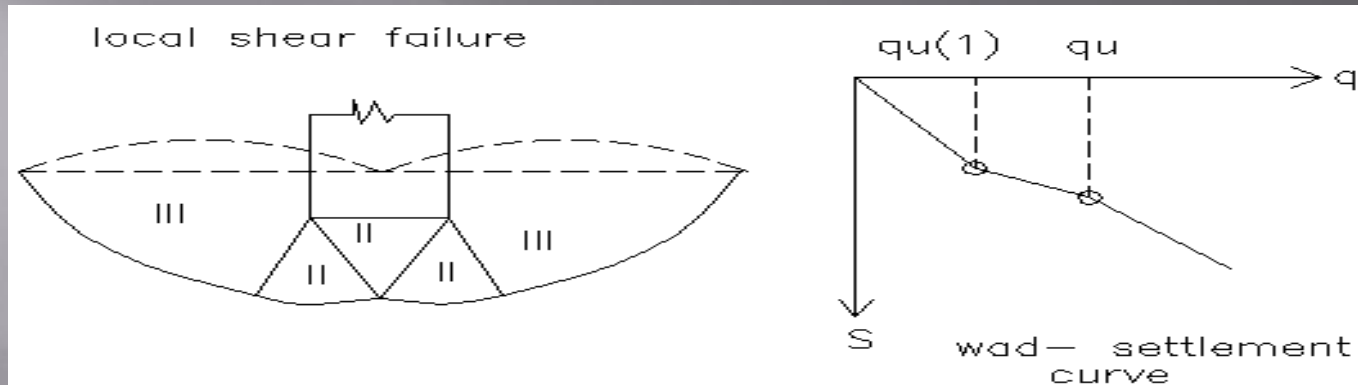


**Strip footing resting on surface  
of dense sand or stiff clay**

**Load –settlement curve**

- \* The load - Settlement curve in case of footing resting on surface of dense sand or stiff clays shows pronounced peak & failure occurs at very small strain.
- \* A loaded base on such soils sinks or tilts suddenly into the ground showing a surface heave of adjoining soil
- \* The shearing strength is fully mobilized all along the slip surface & hence failure planes are well defined.
- \* The failure occurs at very small vertical strains accompanied by large lateral strains.
- \*  $I_D > 65$  ,  $N > 35$  ,  $\Phi > 36^\circ$  ,  $e < 0.55$

## 2) Local Shear failure -



Strip footing resting on surface

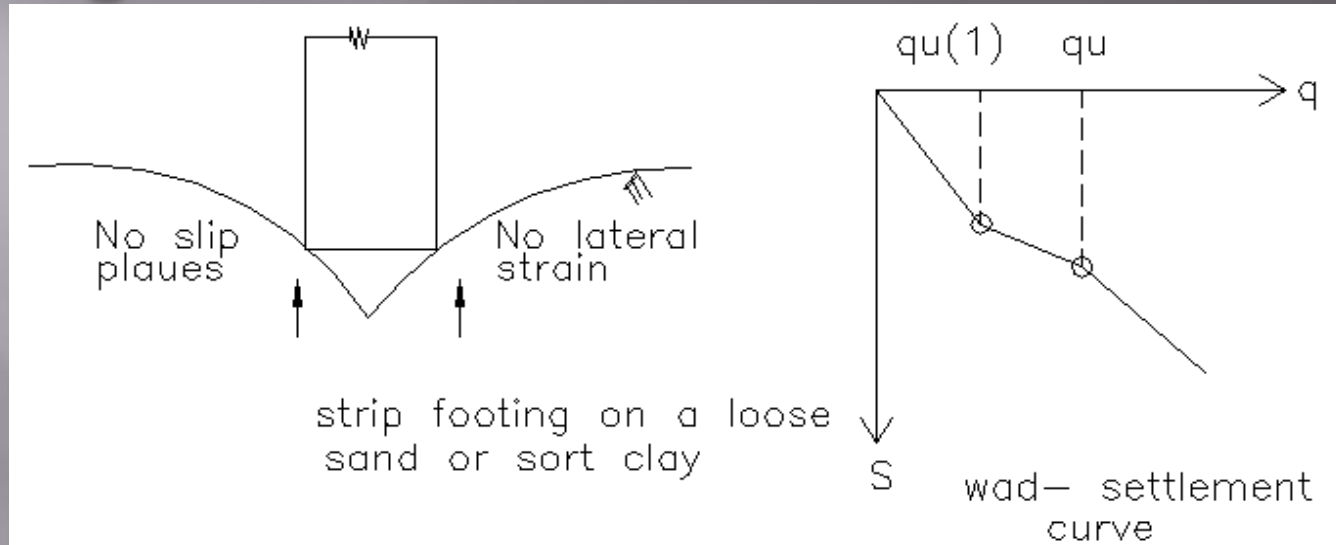
Load -settlement curve

Of Medium sand or Medium clay

- \* When load is equal to a certain value  $qu(1)$ ,
- \* The foundation movement is accompanied by sudden jerks.
- \* The failure surface gradually extend out wards from the foundation.
- \* The failure starts at localized spot beneath the foundation & migrates out ward part by part gradually leading to ultimate failure.
- \* The shear strength of soil is not fully mobilized along planes & hence failure planes are not defined clearly.
- \* The failure occurs at large vertical strain & very small lateral strains.
- \*  $I_D = 15$  to  $65$  ,  $N=10$  to  $30$  ,  $\Phi < 30$  ,  $e > 0.75$



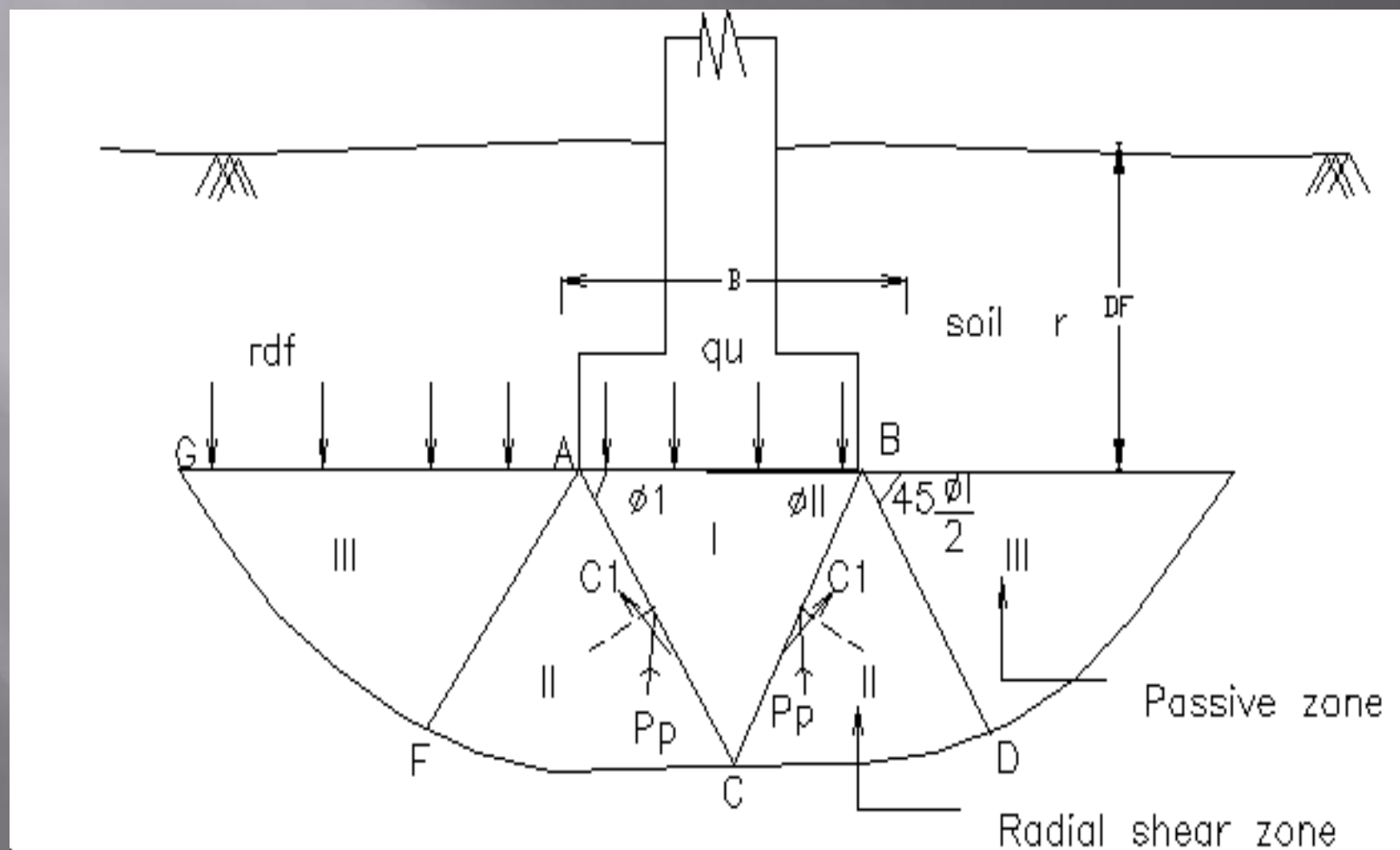
### 3) Punching Shear failure -



- \* The loaded base sinks into soil like a punch.
- \* The failure surface do not extend up to the ground surface.
- \* No heave is observed.
- \* Large vertical strains are involved with practically no lateral deformation.
- \* Failure planes are difficult to locate 222

## Terzaghi's Bearing Capacity Analysis –

Terzaghi (1943) analysed a shallow continuous footing by making some assumptions –



- \* The failure zones do not extend above the horizontal plane passing through base of footing
- \* The failure occurs when the down ward pressure exerted by loads on the soil adjoining the inclined surfaces on soil wedge is equal to upward pressure.
- \* Downward forces are due to the load ( $=q_u \times B$ ) & the weight of soil wedge ( $1/4 \gamma B^2 \tan \theta$ )
- \* Upward forces are the vertical components of resultant passive pressure ( $P_p$ ) & the cohesion ( $c'$ ) acting along the inclined surfaces.

For equilibrium:

$$\Sigma F_v = 0$$

$$\frac{1}{4} \gamma B^2 \tan \theta + qu \times B = 2P_p + 2C' \times Li \sin \theta'$$

where  $Li$  = length of inclined surface  $CB$   
( =  $B/2 / \cos \theta'$  )

Therefore,

$$qu \times B = 2P_p + BC' \tan \theta' - \frac{1}{4} \gamma B^2 \tan \theta' \text{ ----- (1)}$$

The resultant passive pressure ( $P_p$ ) on the surface  $CB$  &  $CA$  constitutes three components ie.  $(P_p)_r$ ,  $(P_p)_c$  &  $(P_p)_q$ ,

Thus,

$$P_p = (P_p)_r + (P_p)_c + (P_p)_q$$

$$q_u \times B = 2[(P_p)_r + (P_p)_c + (P_p)_q] + Bc' \tan \phi' - \frac{1}{4} \gamma B^2 \tan \phi'$$

$$\begin{aligned} \text{Substituting; } 2(P_p)_r - \frac{1}{4} \gamma B^2 \tan \phi' &= B \times \frac{1}{2} \gamma B N_r \\ 2(P_p)_q &= B \times \gamma D N_q \\ \& 2(P_p)_c + Bc' \tan \phi' &= B \times C' N_c; \end{aligned}$$

We get,

$$q_u = C' N_c + \gamma D_f N_q + 0.5 \gamma B N_r$$

This is Terzaghi's Bearing capacity equation for determining ultimate bearing capacity of strip footing. Where  $N_c$ ,  $N_q$  &  $N_r$  are Terzaghi's bearing capacity factors & depends on angle of shearing resistance ( $\phi$ )

$\phi$	General Shear Failure			Local Shear Failure		
	Nc	Nq	Nr	Nc'	Nq'	Nr'
0	5.7	1.0	0.0	5.7	1.0	0.0
15	12.9	4.4	2.5	9.7	2.7	0.9
45	172.3	173.3	297.5	51.2	35.1	37.7

## Important points :

- \* Terzaghi's Bearing Capacity equation is applicable for general shear failure.
- \* Terzaghi has suggested following empirical reduction to actual  $c$  &  $\phi$  in case of local shear failure  
Mobilised cohesion  $C_m = 2/3 C$   
Mobilised angle of  $\phi_m = \tan^{-1} (2/3 \tan \phi)$

Thus,  $N_c'$ ,  $N_q'$  &  $N_r'$  are B.C. factors for local shear failure

$$q_u = C_m N_c' + \gamma D_f N_q' + 0.5 \gamma B N_r'$$

- \* **Ultimate Bearing Capacity for square & Circular footing -**  
Based on the experimental results, Terzaghi's suggested following equations for UBC –

Square footing  $q_u = 1.2c' N_c + \gamma D_f N_q + 0.4 \gamma B N_r$

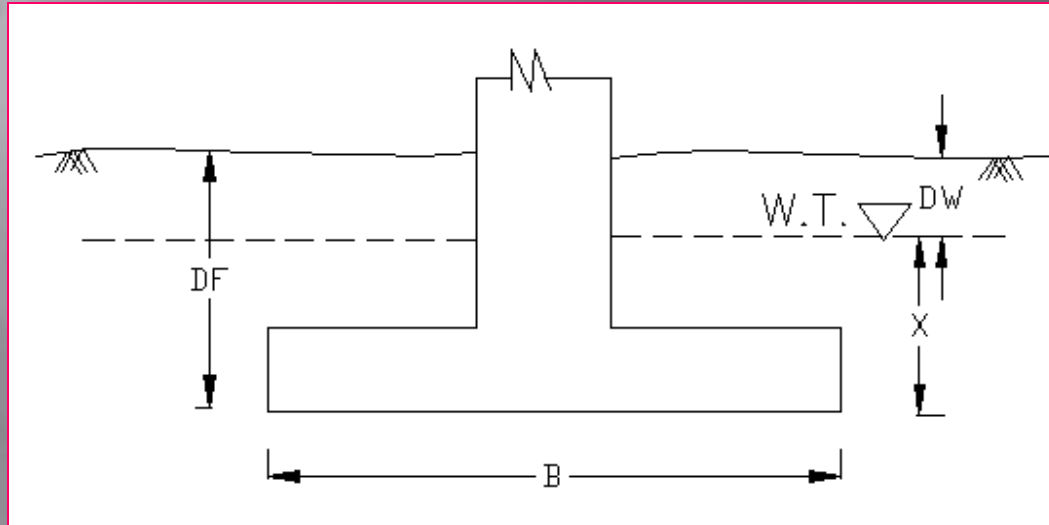
Circular footing  $q_u = 1.2c' N_c + \gamma D_f N_q + 0.3 \gamma B N_r$

# Effect of water table on Bearing Capacity :

- \* The equation for ultimate bearing capacity by Terzaghi has been developed based on assumption that water table is located at a great depth .
- \* If the water table is located close to foundation ; the equation needs modification.



i) When water table is located above the base of footing -



\* The effective surcharge is reduced as the effective weight below water table is equal to submerged unit weight.

$$q = D_w \cdot r + x \cdot \gamma_{sub}$$

$$\text{put } x = D_f - D_w$$

$$q = \gamma_{sub} D_f + (\gamma - \gamma_{sub}) D_w$$

Thus,

$$q_u = c' N_c + [\gamma_{\text{sub}} D_f + (\gamma - \gamma_{\text{sub}}) D_w] N_q + 0.5 \gamma_{\text{sub}} B N_r$$

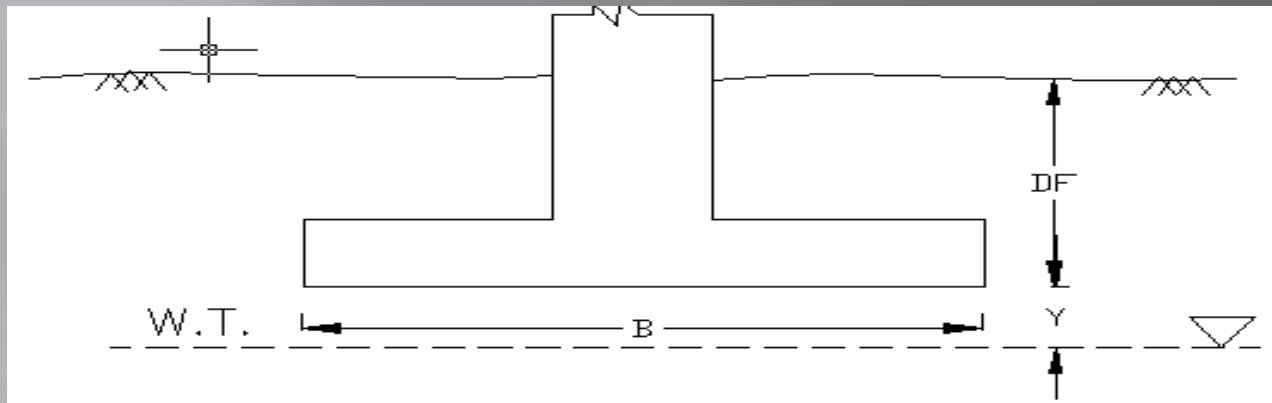
When,  $D_w = 0$

$$q_u = c' N_c + \gamma_{\text{sub}} N_c + 0.5 \gamma_{\text{sub}} B N_r$$

& when  $x = 0$

$$q_u = c' N_c + \gamma D_f N_q + 0.5 \gamma_{\text{sub}} B N_r$$

ii) When water table is located at depth  $y$  below base :



\* Surcharge term is not affected.

\* Unit weight in term is  $\gamma = \gamma_{sub} + \frac{y}{B} (\gamma - \gamma_{sub})$

Thus,

$$q_u = c' N_c + \gamma D_f N_q + 0.5 B \gamma N_r$$

When  $y = B$  ; W.T. at  $B$  below base of footing.

$$q_u = c' N_c + \gamma D_f N_q + 0.5 B \gamma N_r$$

Hence when ground water table is at  $b \geq B$ , the equation is not affected.

## Hansen's Bearing Capacity Equation :

Hansen's Bearing capacity equation is :

$$q_u = cN_c S_c d_c i_c + qN_q S_q d_q i_q + 0.5 \gamma B N_r S_r d_r i_r$$

where,

$N_c, N_q$ , &  $N_r$  are Hansen's B.C factors which are some what smaller than Terzaghi's B.C. factors.

$S_c, S_q$  &  $S_r$  are shape factors which are independent of angle of shearing resistance;

$d_c, d_q$ , &  $d_r$  are depth factors ;

$i_c, i_q$  &  $i_r$  are inclination factors

The same form of equation has been adopted by I.S. 6403 –1971 & may be used for general form as

$$q_{nu} = c N_c S_c d_c i_c + q(N_q-1)S_q d_q i_q + 0.5 \gamma B N_r S_r d_r i_r W'$$

## Settlement of foundation :

### a) Settlement under loads

Settlement of foundation can be classified as-

1. **Elastic settlement ( $S_i$ ):** Elastic or immediate settlement takes place during or immediately after the construction of the structure. It is also known as the distortion settlement as it is due to distortions within foundation soil.

2. **Consolidation settlement ( $S_c$ ):** Consolidation settlement occurs due to gradual expulsion of water from the voids at the soil. It is determined using Terzaghi's theory of consolidation.

3. **Secondary consolidation settlement ( $S_s$ ):** The settlement occurs after completion of the primary consolidation. The secondary consolidation is non-significant for inorganic soils.

Thus,

$$\text{Total settlement (s)} = S_i + S_c + S_s$$

b) Settlement due to other causes

1. Structural collapse of soil.
2. Underground erosion.
3. Lowering of water table. .
4. Thermal changes.
5. Subsidence etc.

## Elastic settlement of foundation :

### a) On Cohesive soils

According to Schleicher, the vertical settlement under uniformly distributed flexible area is,

$$S_i = q B \left( 1 - \frac{\mu^2}{E_s} \right) I$$

where

$q$  - uniformly distributed load.

$B$  - characteristic length of loaded area,

$E_s$  - modulus of elasticity of the soil.

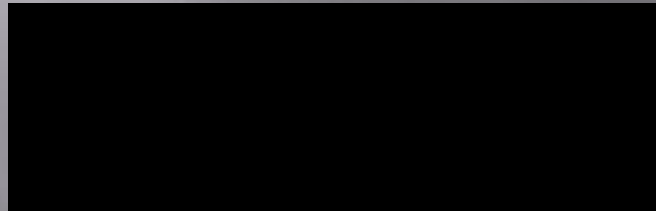
$\mu$  - poisson's ratio.

$I$  - influence factor which dependent upon elastic properties of base & shape at base.  
Alternatively, the value of  $\left[ 1 - \frac{\mu^2}{E_s} \right] I$  can be determined from the plate load test.



## b) On Cohesionless Soils

According to Stuartmann & Hartman immediate settlement on Cohesionless soils is given by -



Where,  $C_1$  -Correction factor for depth of foundation embedment

$C_2$  - correction factor for creep in soils.

$q$  - pressure at the level of foundation

$q$  - surcharge ( $\gamma D_f$ )

$E_s$  - modulus of elasticity =  $766 N (KN/m^2)$  from SPT  
=  $2q_c$  from SCPT

## Settlement of foundation on Cohesionless Soils

Settlement of foundations on Cohesionless soils are generally determined indirectly using the semi-empirical methods.

### 1. Static Cone Penetration method

In this, the sand layer is divided into small layers such that each small layer has approximately constant value of the cone resistance. The average value of the cone resistance of each small layer is determined.

The settlement of each layer is determined using the following equation-

$$S = H/C \text{ Log } (\sigma_{0 + \Delta \sigma}) / \sigma_0$$

$$\text{Where, } c = 1.5 q_c / \sigma_0$$

in which  $q_c$  - static cone resistance  
 $\sigma_0$  - mean effective overburden pressure,  
 $\Delta \sigma$  - Increase in pressure at center of layer  
due to net foundation pressure.  
 $H$  - thickness of layer.

The total settlement of the entire layer is equal to the sum of settlements of individual layers.

## 2. Standard Penetration Test

IS 8009 (part I) 1976 gives a chart for the calculation of settlement per unit pressure as a function of the width of footing & the standard penetration number.

## 3. Plate Load Test

The settlement of the footing can be determined from the settlement of the plate in the plate load test.

\* The difference between the magnitudes of settlements at any two points is known as differential settlement.

\* If there is large differential settlement between various part of a structure, distortion may occur due to additional moments developed.

\* The differential settlement may caused due to tilting of a rigid base, dishing of flexible base or due to non uniformity of loading.

\* If  $S_1$  &  $S_2$  are the settlements at two points, then differential settlement is

$$\Delta = S_1 - S_2$$

$$\text{Angular distortion} = (S_1 - S_2) / L = \Delta$$

- \* It is difficult to predict the differential settlement.
- \* It is generally observed indirectly from the maximum settlement.
- \* It is observed that the differential settlement is less than 50% of the maximum settlement in most of the cases.

The differential settlement can be reduced by providing rigid rafts, founding the structures at great depth & avoiding the eccentric loading.

## Allowable Settlement

- \* The allowable maximum settlement depends upon the type of soil, the type of foundation & the structural framing system.
- \* The maximum settlement ranging from 20mm to 300mm is generally permitted for various structures.
- \* IS 1904-1978 gives values of the maximum & differential settlements of different type of building.

	Sand & hard Clay			Plastic clay		
	Max.Settle.	Diff.Settl	Angular distortion	Max.Settle	Diff. Settle.	Angular distortion
<b>Isolated foundation</b>						
i) steel struct	50mm	0.0033L	1/300	50mm	0.0033L	1/300
ii) RCC struct	50mm	0.0015L	1/666	75mm	0.0015L	1/666
<b>Raft foundation</b>						
i) steel struct	75mm	0.0033L	1/300	100mm	0.0033L	1/300
ii) Rcc struct.	75mm	0.002L	1/500	100mm	0.002L	1/500

**Theoretically, no damage is done to the superstructure if the soil settles uniformly.**

**However, settlements exceeding 150mm may cause trouble to utilities such as water pipe lines, sewers, telephone lines & also is access from streets.**

## Consolidation Settlement :

- \* Compressibility of soil is the property of the soil due to which a decrease in volume occurs under compressive forces.
- \* The compression of soils can occur due to-
  - A) Compression of solid particles & water in the voids.
  - B) Compression & expulsion of air in the voids.
  - C) Expulsion of water in the voids.
- \* The compression of a saturated soil under a steady pressure is known as consolidation. It is entirely due to the expulsion of water from the voids. Due to the expulsion of water, the solid particles shift from one position to the other by rolling & sliding & thus attain a closer packing causing a reduction in volume.



## Consolidation of laterally confined soil:

When a pressure  $\Delta \sigma$ , is applied to a saturated soil sample of unit cross-sectional area, the pressure is shared by the solid particles & water as

$$\Delta \sigma + u = \Delta \sigma,$$

Initially, just after the application of pressure, the entire load is taken by water in form of excess hydrostatic pressure ( $u$ ), thus,

$$0 + (u, ) = \Delta \sigma,$$

The excess hydrostatic pressure so developed sets up a hydraulic gradient, & the water starts escaping from the voids. As the water escapes, the applied pressure is transferred from the water to the solids. Thus at  $t = t_f$ ,

$$\Delta \sigma + 0 = \Delta \sigma,$$

As the effective stress increases the volume of soil decreases & consolidation completes under  $\Delta \sigma$ , load.

Laboratory Consolidation Test

- \* The consolidation test is conducted in a laboratory study the compressibility of soil.

- \* Consolidation test apparatus, known as consolidometer or an odometer consists a loading device & a cylindrical container called as consolidation cell. Consolidation cell are of two types, i) free ring or floating ring cell &  
ii) fixed ring cell

- \* The internal diameter of the cell is 60 mm & thickness of sample taken is usually 20 mm.

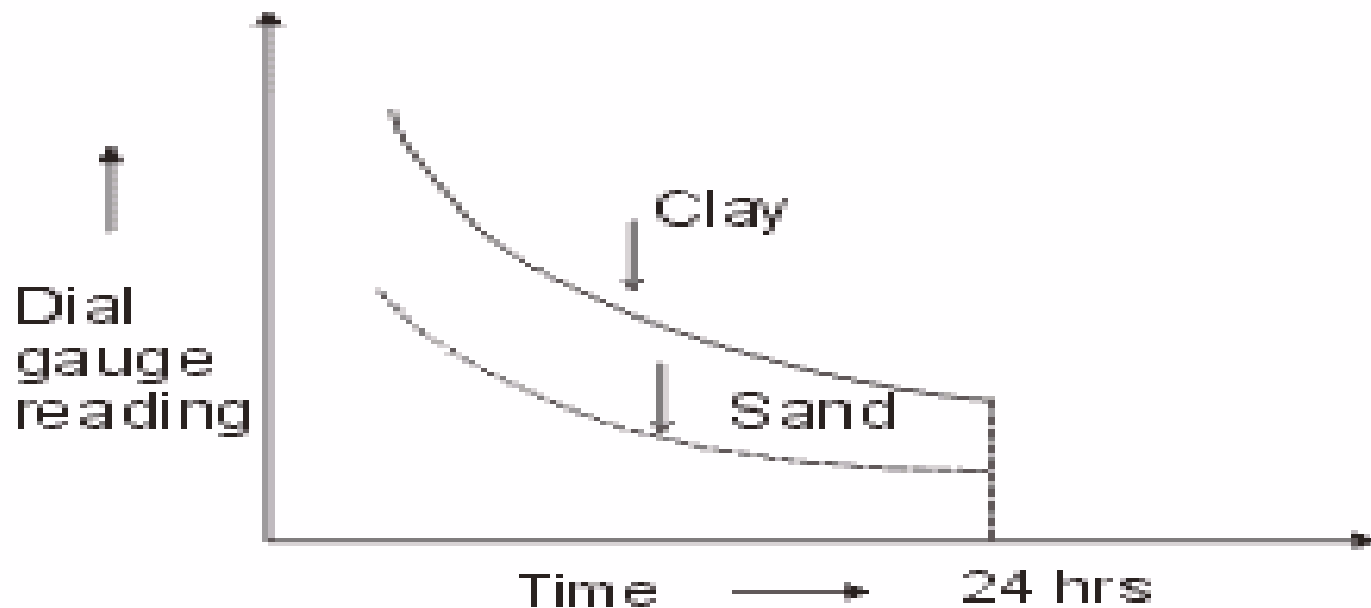
- \* The consolidometer has arrangements for application of the desired load increment, saturation of sample & measurement of change in thickness of sample at every stage of consolidation process.

- \* An initial setting load of about  $5 \text{ kN/ m}^2$  is applied to sample.
- \* The first increment of load to give a pressure of  $10 \text{ KN/ m}^2$  is then applied to the specimen, the dial gauge readings are taken after 0.25, 1.0, 2,4,9,16,..... etc up to the 24 hours.
- \* The second increment of load is then applied. The successive pressures usually applied are 20,40, 80, 160 & 320  $\text{KN/ m}^2$  etc till the desired maximum load intensity is reached.  
( Actual loading on soil after construction of structure)
- \* After consolidation under final load increment is complete, the load is reduced to  $\frac{1}{4}$ th of final load & allowed to stand for 24 hours. The sample swells & reading of dial gauge is taken when swelling is complete. The process is repeated till complete unloading.  
Immediately after complete unloading, the weight of ring & sample is taken. The sample is dried in over for 24 hours & its dry mass  $M_s$  is taken

## Consolidation test results

### 1) Dial gauge reading time plot :

- Plotted for each load increment
- Required for determining the coefficient of consolidation.
- Useful for obtaining the rate of consolidation in field.



**Fig.4**

## 2) Final void ratio – effective stress plot:

- Plotted for entire consolidation process under desired load.
- Required for determination of the magnitude of the consolidation settlements in field.

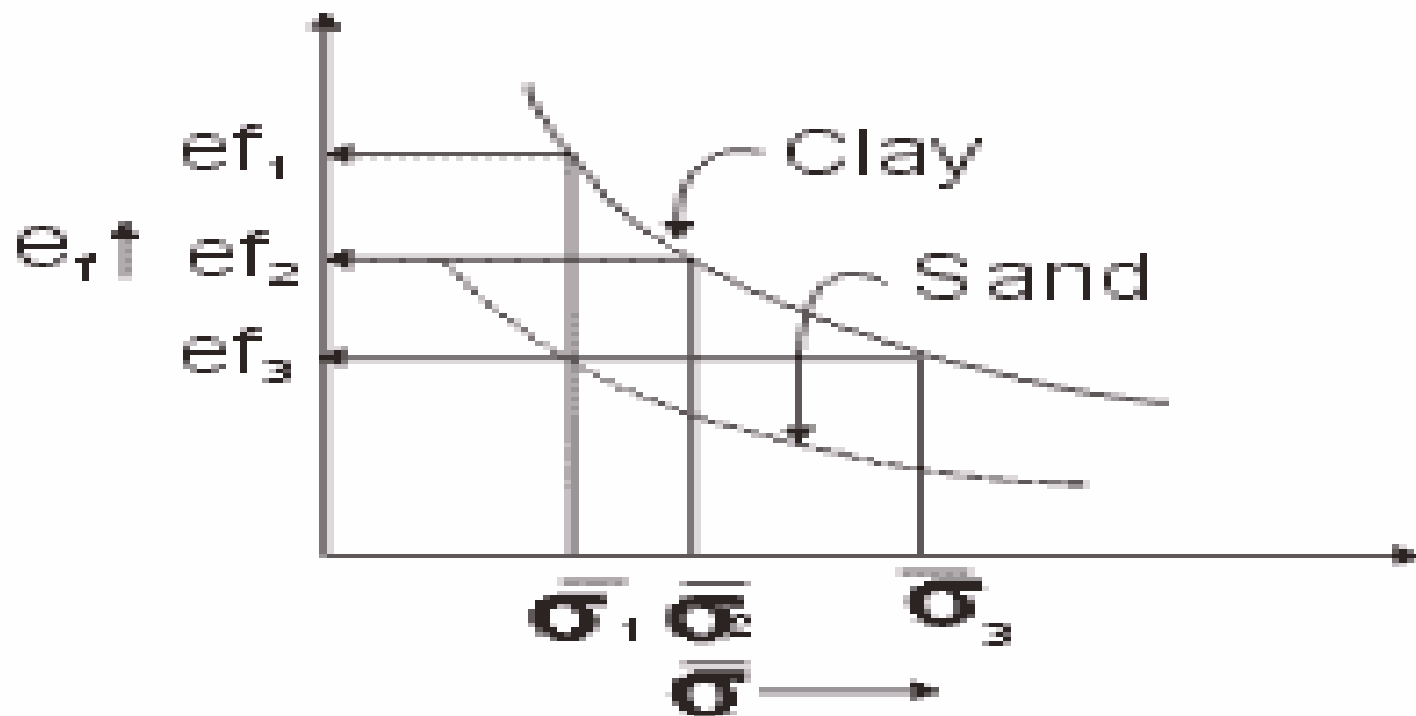
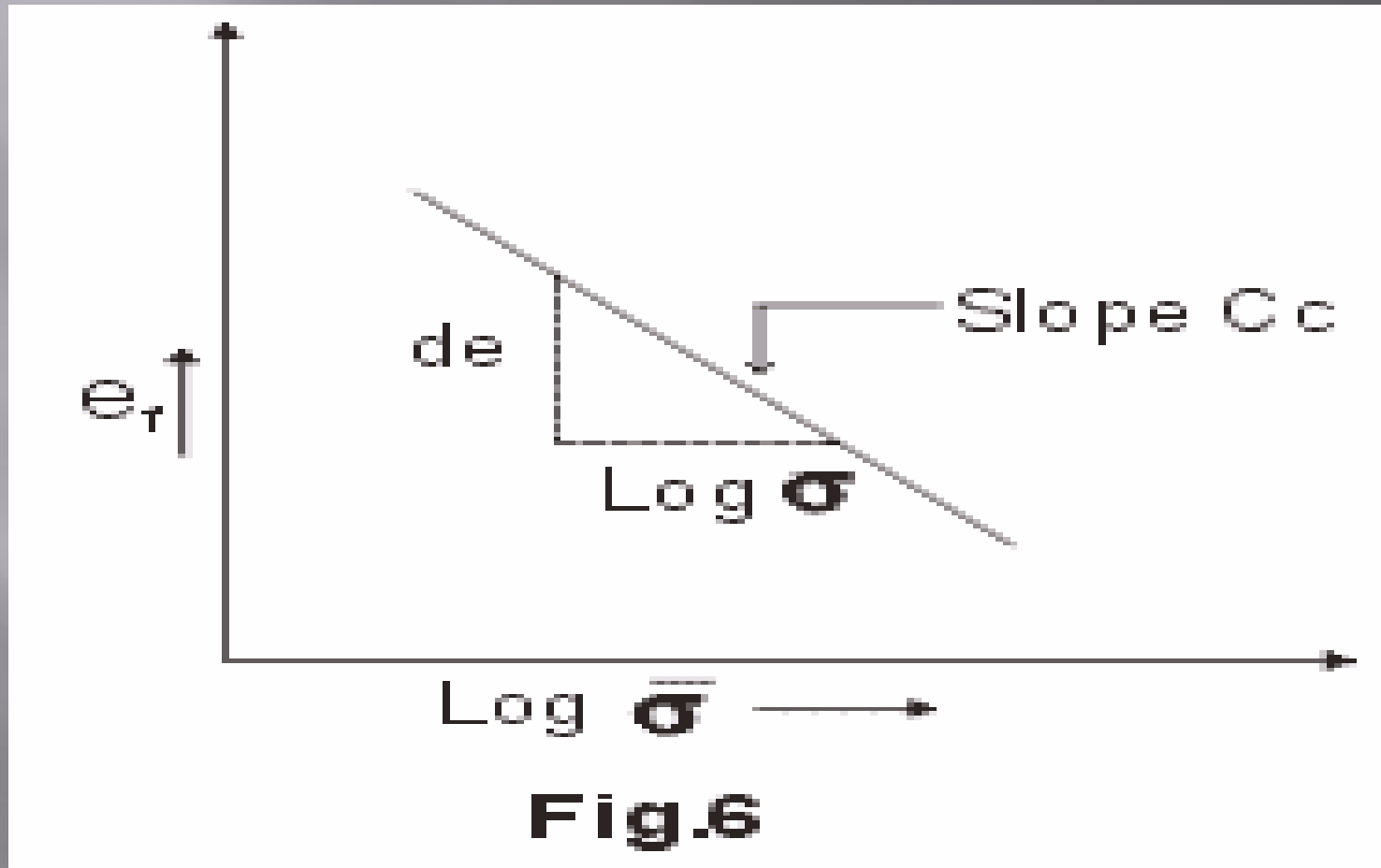


Fig.5

### 3) final void ratio – log $\sigma$ plot



## 4) Loading, unloading & reloading plot

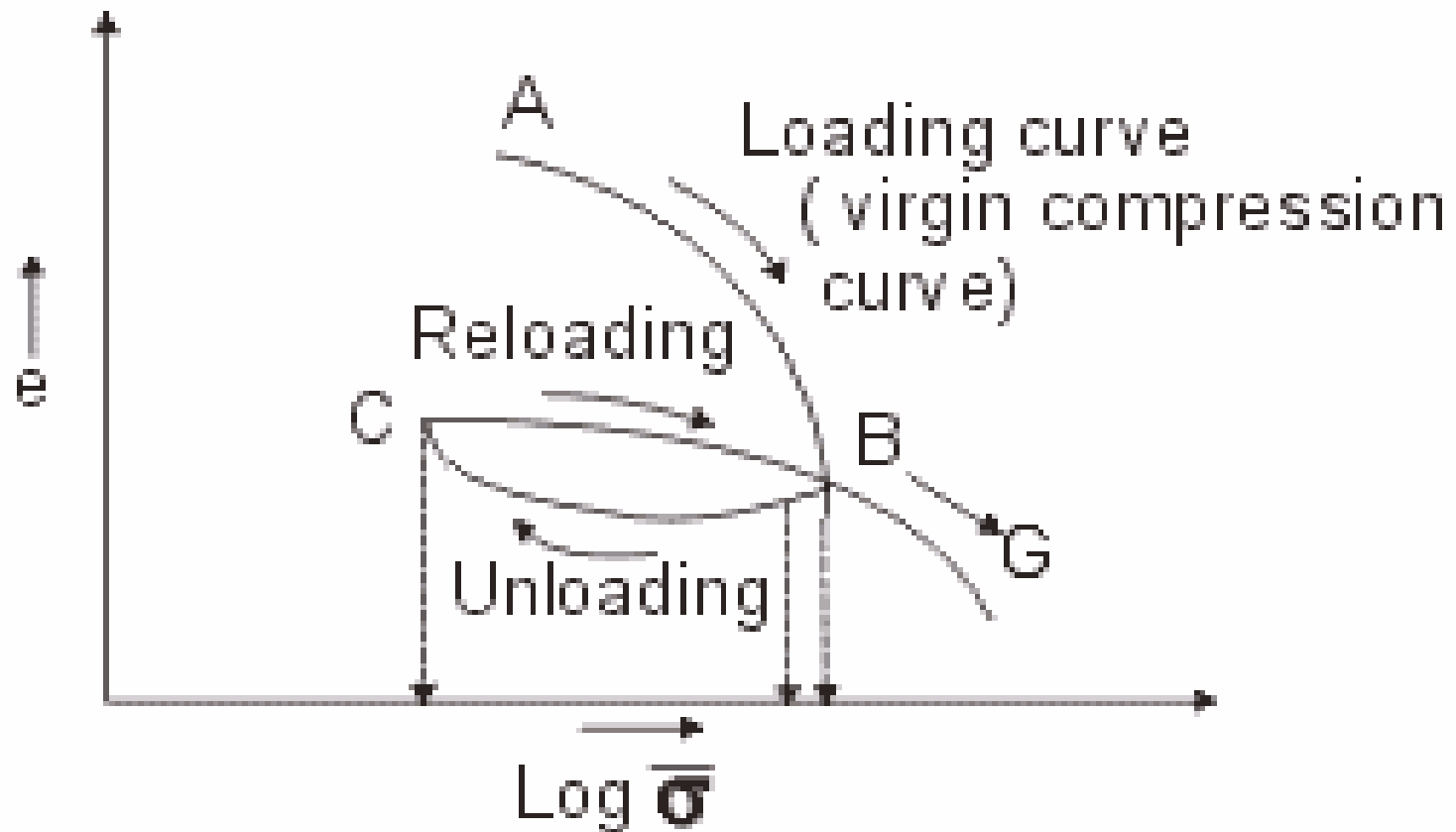


Fig.7

## Important Definitions

1) Coefficient of compressibility ( $a_v$ ) is defined as decrease in void ratio per unit increase in effective stress.

$$a_v = -de/d\sigma = -\Delta e / \Delta \sigma \quad (\text{slope of } e - \sigma \text{ curve units} - \text{m}^2 / \text{KN})$$

2) Coefficient of volume change ( $m_v$ ) is defined as the volumetric strain per unit increase in effective stress.

$$m_v = -(\Delta v / v_o) / \Delta \sigma \quad \text{in which,}$$

$v_o$  – initial volume,

$\Delta v$  – change in volume

$\Delta \sigma$  - change in effective stress

a)  $m_v = -(\Delta e / 1 + e_o) / \Delta \sigma$

b) for one dimension,  $\Delta v = \Delta H$

$$m_v = -(\Delta H / H_o) / \Delta \sigma$$

$$\text{also } m_v = a_v / (1 + e_o)$$

in which,  $e_o$ - initial void ratio.

$\Delta e$  - change in void ratio.

$H_o$  initial thickness.

$\Delta H$  – change in thickness.



3) Compression index (  $C_c$  ) is equal to the slope of the linear portion of the void ratio versus  $\log \sigma$  plot.

$$C_c = - \Delta e / \log_{10} (\sigma_0 + \Delta \sigma) / \sigma_0$$

in which,  $\sigma_0$  = initial effective stress.

$\Delta \sigma$  - change in effective stress.

Empirical relationship after Terzaghi & Peck;

a) for undisturbed soils  $C_c = 0.009 ( W_L - 10 )$

b) for remoulded soils  $C_c = 0.007 ( W_L - 10 )$

c) Also  $C_c = 0.54 ( e_0 - 0.35 )$

$$C_c = 0.0054 ( 2.6 w_o - 35 )$$

- Normally consolidated soil :- A normally consolidated soil is one which had not been subjected to a pressure greater than the present existing pressure. The portion AB of loading –unloading curve represent the soil in normally consolidated condition.
- Over consolidated soil: - A soil is said to over consolidated if it had been subjected in the past to a pressure in excess of the present pressure. The soil in the range CD (loading –unloading curve) when it recompressed represent overconsolidated condition.

- Normally consolidated soils & Overconsolidated soils are not different types of soils but these are conditions in which a soil exists.

- Preconsolidation Pressure- The maximum pressure to which an overconsolidated soil had been subjected in the past is known as preconsolidation pressure or overconsolidation pressure ( $\sigma_c$ )

- When a soil specimen is taken from a natural deposit, the weight of overlying material is removed. This causes an expansion soil due to reduction in pressure. Thus the specimen is generally preconsolidated.

## Final Settlement Of Soil Deposit In The Field

- For computation of final settlement, the coefficient of volume change or compression index ( $C_c$ ) is required. For time rate of computation, the Terzaghi's theory is used.

- Final settlement using coefficient of volume change :

Let  $H_0$  = initial thickness of clay deposit. Consider a small element of thickness  $\Delta z$  at depth  $z$ .

$\Delta\sigma$  = effective pressure increment causing settlement.

Then  $\Delta H = m_v H_0 (\Delta\sigma)$

Representing the final settlement as  $\Delta s_f$  & taking  $H_0 = \Delta z$

Thus, total settlement of the complete layer,



alternatively

## Final Settlement Using Void Ratio

The value of  $\Delta e$  corresponding to the given load increment is read off from  $e - \sigma$  plot & substituted in –

$$\Delta H = H_o (\Delta e / 1 + e_o)$$

i.e  $S_f = H_o (\Delta e / 1 + e_o) \dots \dots \dots (1)$

where  $e_o$  is the initial void ratio. The usual practice is not to use  $\Delta e$  but to use the coefficient of compression ( $C_c$ ) or coefficient of recompression index ( $C_r$ )

a) **Normally Consolidated Soils** - The compression index of a normally considered soil is constant.

$$S_f = \frac{C_c}{1+e_0} H_o \text{Log}_{10} \left\{ \frac{\sigma_{0+\Delta\sigma}}{\sigma_0} \right\}$$

b) **Pre Consolidated Soils** -

$$S_f = \frac{C_r}{1+e_0} H_o \text{Log}_{10} \left\{ \frac{\sigma_{0+\Delta\sigma}}{\sigma_0} \right\}$$

The above equation is applicable when  $(\sigma_{0+\Delta\sigma}) < \sigma_c$ .