

CH.6

Stresses within a Soil Mass

Stresses due to: - own weight
 - external loads

Loads → deformation
(settlement)

(change in shape & vol.)

- Effective Stress: Terzaghi 1920
- Stress distribution
- Principle of superposition
(combination of loads)

$$\sigma = \bar{s} + u$$

total stress effective stress pore-water pressure

- Average stress
- Average intergranular stress σ^*

- Effective stress $\bar{s} = s * \left(\frac{A_c}{A_t} \right)$

$$u = u_h + u_e$$

hydrostatic hydrodynamic (flow)

6.1.2 Effective stress due to capillary rise

- Capillary head (h_c)

$$u = - h_c \gamma_w \quad \text{-ve pore water pressure}$$

$$\Rightarrow \bar{s} > \sigma$$

Environmental factors: - rain
- evaporation

Ignored.

6.2 Mohr Circle of Stress

2-D

σ : normal stress (comp. +ve)
 τ : shear stress (creates counterclockwise moment)

- Principal stress - σ_1
 - Principal stress - σ_3
 - Principal planes (2)
- } No shear stress $\tau = 0$

6.3 Pole method of stress computation

origin of planes

$$\sigma_y = 20$$

$$\sigma_x = 50 \text{ kN/m}^2$$

$$20 \text{ kN/m}^2$$

$$\tau$$
$$\text{kN/m}^2$$

$$\sigma$$
$$\text{kN/m}^2$$

Stresses due to external, applied loads

CH. 7

Vol. Change in Soils

7.0 Introduction

- Consolidation: slow compression due to escape of water
 \Rightarrow gradual adjustment of pore water pressure

7.1

$\sigma = \sigma_0$	$\sigma = \sigma_0$	$\sigma = \sigma_0 + \Delta\sigma$
$u = u_0$	$u = u_0 + \Delta u$	$u = u_0$
	$\Delta\sigma$	ΔH
$t = 0$	$t = t$	$t = t$

7.2

7.2.1 1-D Comp.

Rate of outflow of water – rate of inflow of water = rate of vol. change

$$v_z + \frac{\partial v_z}{\partial Z} \cdot d_x \cdot d_y - v_z \cdot d_x \cdot d_y = \frac{\partial v}{\partial t}$$

$$\frac{\partial v_z}{\partial Z} \cdot d_x \cdot d_y \cdot d_z \quad \begin{array}{l} \nearrow \\ \therefore v_s \text{ does not change} \end{array}$$

$$\frac{\partial V_v}{\partial t} = \frac{\partial [v_s (1+e)]}{\partial t}$$

$$\frac{-k}{\partial_w} \frac{\partial^2 U}{\partial Z^2} = \frac{v_s \partial e}{\partial t}$$

$$\frac{V}{1+e_0} = \frac{d_x \cdot d_y \cdot d_z}{1+e_0}$$

$$\frac{-k}{\partial_w} \frac{\partial^2 u}{\partial Z^2} \cdot d_x \cdot d_y \cdot dZ = \frac{d_x \cdot d_y \cdot dZ}{1+e_o} \cdot \frac{\partial e}{\partial t}$$

$$\frac{-k}{\partial_w} \frac{\partial^2 u}{\partial Z^2} = \frac{1}{1+e_o} \frac{\partial e}{\partial t} = a_v \partial s = -a_v \partial u$$

$$\frac{-k}{\partial_w} \frac{\partial^2 u}{\partial Z^2} = \frac{-a_v}{1+e_o} \frac{\partial u}{\partial t}$$

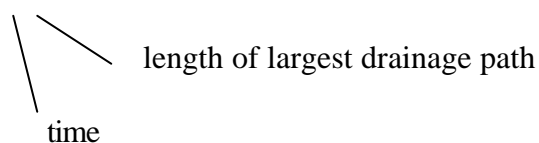
$$\frac{\partial u}{\partial t} = \frac{k}{g_w \cdot m_v} \frac{\partial^2 u}{\partial Z^2} = C_v \frac{\partial^2 u}{\partial Z^2}$$

C_v

- Consolidation ratio $0 \rightarrow 1$
 - as $e \downarrow$ from e_1 to e_2
 - as $u_e \downarrow$ from u_1 to 0
 - as $\bar{s} \uparrow$ from \bar{s}_1 to \bar{s}_2

- Degree/percentage of consolidation $0 \rightarrow 100\%$

- Time Factor (T) = $C_v (t/H^2)$



- Average degree of consolidation

7.2.3 Max Past Vert. Pressure

Max Past Overburden Pressure

Preconsolidation pressure (\bar{s}_p): The greatest effective stress to which the soil has been subjected.

N.C. Normally Consolidated $\bar{s}_o = \bar{s}_p$

O.C. Overconsolidated

* Procedure for determining \bar{s}_p (Fig. 7.4)

0. Draw the consolidation curve
1. Select by eye point of max. curvature (A)
2. Draw horiz. line & tangent line through point (A)
3. Draw a third line bisecting the angle between the two lines in (2)
4. Draw a line (upward extension of the st. line portion of the curve)
5. The intersection of line (3) with (4) gives location B: most probable value for \bar{s}_p .
6. Min value: horizontal from e and # (4).
7. Max value: when line (4) exist the curve.

- Effect of sample disturbance

* Reconstructing insitu compression curve

- Loan-unload-reload cycle

C_r : recompression index

C_c : compression index

7.2.4 Curve-fitting

Terzaghi Theory of Consolidation

- Assumptions

- 1
- 2
- 3 p. 257
- 4
- 5

* \sqrt{t} Method

1. Get tangent (initial slope) $\rightarrow d_s$
2. Extend a st. line from d_s with 1.15 slope of initial tangent

$$d_{90} \rightarrow \sqrt{t_{90}}$$

$$u = 90\% \quad \rightarrow \quad T = 0.848$$

* Log t Method

1. Draw tangents to the two st. line parts
2. Intersection $\rightarrow d_{100}, t_{100}$
3. d_s is located by selecting two points on the initial part --- t & $4t$

$$d_s = d_t - (d_{4t} - d_t)$$

$$d_{50} = \text{halfway between } d_s \text{ \& } d_{100}$$

$$U = 50\% \Rightarrow T = 0.197$$

$$C_v \text{ from } \sqrt{\quad} > C_v \text{ from log}$$

- primary compression / consolidation
- secondary compression / consolidation
- r primary compression ratio: $r = \frac{d_s - d_{100}}{d_o - d_{100}} = 0.7 \pm .2$

7.2.5 Seco. Comp.

- soil compression after dissipation of excess water pressure
- takes place at a constant effective stress
- st. line relationship between comp. & $\log t$

$$C_{\alpha} : \text{sec. comp. index} = \frac{\Delta e}{\Delta \log t}$$

$$C_{\alpha E} : \text{modified sec. comp. index} = \frac{\Delta e_v}{\Delta \log t}$$

$$= \frac{C_a}{C_c} = \text{constant} = 0.025 \quad \text{t} \quad 0.06 \quad \text{for organic}$$

7.3 Constant-rate-of-strain Consolidation

CH. 8

Shear Strength of Soils

8.0

* Shear strength: ability of soil to sustain load without “large” deformation or failure

Tests – Lab - - direct shear
- unconfined compression
- triaxial compression
- vane shear
- penetrometer

- drained
- undrained

8.1

- Pore-pressure coeff. $B = \frac{\Delta u}{\Delta \sigma_3} = 0 \rightarrow 1.$
dry sat.

- Deformation – slippage between soil particles
 - deformation behavior
 - shear strength
 - measurements

8.1.1 σ - ϵ relationship depends on – can w be adjusted to σ

- drained
- undrained

* Dilatancy: increase in volume of dense soils with increasing deformation.

- Drained
- Undrained ----- $u \uparrow$

$$\bar{A} = \frac{\Delta u}{\Delta s_1}$$

8.1.2 Mohr-Coulomb Criterion

- Failure is not always clearly defined

← max. stress

- peak
- @ 15 to 20% strain
- @ strain impaired by the structure

- ½ Mohr Circle

- total stresses
- effective stresses
- stresses acting on soil sample @ failure
- Mohr failure envelope

- * slope = ϕ
- * intercept = c cohesive
= 0 for cohesionless

8.2 Measurement of $\sigma - \epsilon$

- Direct shear – rate \approx 1% per min.
- Triaxial tests - drained
 - undrained
- Vane shear test - lab
 - field
- Simple shear test
- Hollow cylinder & plane strain test
- Field methods: - SPT
 - CPT
 - PMT

8.3 Shear Strength & Soil Materials

8.3.1 Cohesionless soils

$\phi \Rightarrow$ properties affecting ϕ Table 8.4

$$\begin{aligned}\phi_{p.s.} &= 2 \text{ to } 4^\circ + \phi_{tri} && \text{loose} \\ &= 4 \text{ to } 9^\circ + \phi_{tri} && \text{dense}\end{aligned}$$

↑

o less freedom for movement

o greater interlocking

Coulomb Equation

$$\begin{aligned}\tau &= c + \sigma \tan \phi \\ &\uparrow \\ &= 0\end{aligned}$$

- ϕ - sliding
- dilatancy
- crushing & rearranging

8.3.2 Cohesive soils

- attractive forces
- interlocking

- Cohesion: shear resistance that can be mobilized between two adjacent particles that stick or cohere to each other without normal force pushing the two particles together.
(True cohesion)

$$f = 0.5 \Rightarrow (\phi_u)_{\text{quartz}} = 26^\circ$$

Fig. 8.27

- Drained

- effective stress σ
- τ vs. σ

$$t = \bar{c} + \bar{s} \tan f \quad \text{Mohr Coulomb envelope}$$

- peak
- residual

- Undrained

CH. 9

@ rest

p. 357 active $\rightarrow \approx H/500$ wall movement 0.2% of H
 passive $\rightarrow \approx H/100$ wall movement 1% of H } @ top tilt

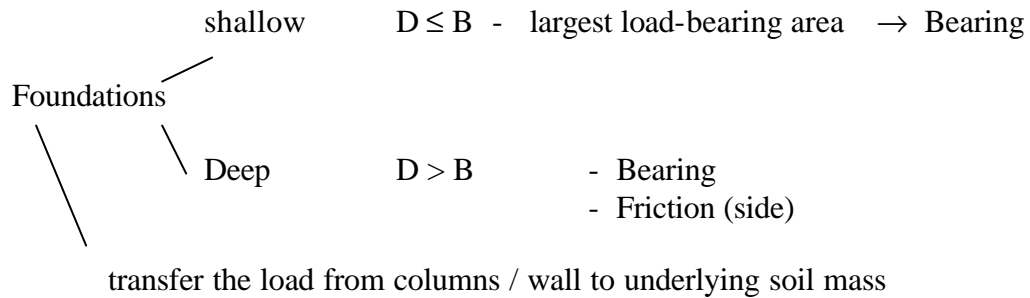
< Rankine - smooth wall
 Coulomb

CH. 10

Bearing Capacity

B.C. : ability of the soil to carry a load without failure?

Failure – excessive settlement without any increase in applied load.



10.1 Factor of Safety

F.S. : ratio of calculated bearing pressure to the applied bearing pressure (q_a)

ultimate bearing capacity, $q_{ult.} = f(\phi, c)$

$$FS = \frac{q_{ult.}}{q_a} \approx 3.0$$

allowable bearing capacity

allowable bearing pressure

Foundation Design - settlement
- strength (B.C.)

1. Find q_{all}
2. Calculate S
3. Check $S_{allowable}$
4. If (3) is not ok, reduce (1)

10.2 Bearing Cap. Theory for Shallow Foundations (Terzaghi)

- * Four stages of failure
 - movement
 - cracking
 - cone : down + outward
 - cont. to surface

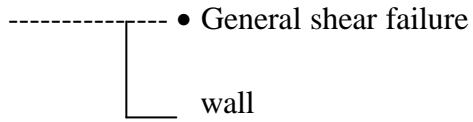
- * B.C. of shallow foundations
 - Prandtl (1921) → Terzaghi (1943)
 - Assumptions
 - homogeneous
 - $D_f \leq B$
 - above water table upto Zone II
 - vertical concentric load
 - neglect cohesion/friction forces along sides of footing

- * Three zones - plastic equilibrium
 - I. Elastic equilibrium
 ϕ to horizontal
 - II. Radial shear
 - III. Passive Rankine

$$q_{ult} = c N_c + q N_q + \frac{1}{2} \mathbf{g} B N_g$$

cohesion

bearing capacity factors : $f(\phi)$



Sq. Rec.

Circle