CH.6

Stresses within a Soil Mass

Stresses due to:
- own weight
- external loads

Loads → deformation (change in shape & vol.)
(settlement)

- <u>Effective Stress:</u> 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)

$u = - h_c \gamma_w$ -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
  - σ_3
- Principal planes (2)

$\left.\begin{array}{l}\text{Principal stress: } \sigma_1, \sigma_3 \\ \text{Principal planes (2)}\end{array}\right\}$ No shear stress $\tau = 0$

6.3 Pole method of stress computation

origin of planes

$\sigma_y = 20$

$\sigma_x = 50$ kN/m^2

20 kN/m^2

τ
kN/m^2

σ
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

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

7.2

7.2.1 1-D Comp.

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

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

$\therefore v_s$ does not change

$$\frac{\partial v_z}{\partial Z} \cdot d_x \cdot d_y \cdot d_z \qquad \frac{\partial V_v}{\partial t} = \frac{\partial \, [v_s \, (1+e)]}{\partial t}$$

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

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

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

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

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

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

C_V

- Consolidation ratio $\quad 0 \rightarrow 1 \quad$ as $\quad e\downarrow \quad$ from e_1 to e_2

  as $\quad u_e\downarrow \quad$ from u_i to 0

  as $\quad \bar{s}\uparrow \quad$ from $\bar{s}_1$ to $\bar{s}_2$

- Degree/percentage of consolidation $\quad 0 \rightarrow 100\%$
- Time Factor (T) = C_v (t/H^2)

  length of largest drainage path

  time

- 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} = halfway between d_s & d_{100}

U = 50% $\Rightarrow$ T = 0.197

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

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

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_α : sec. comp. index = $\dfrac{\Delta e}{\Delta \log t}$

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

$= \dfrac{Ca}{C_c}$ = constant = 0.025 t 0.06 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
- penetranoter

- drained
- undrained

8.1

- Pore-pressure coeff. $B = \dfrac{\Delta u}{\Delta s_3} = 0 \rightarrow 1.$

dry   sat.

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

8.1.1

σ-ε relationship depends on – can <u>w</u> be adjusted to σ

- drained
- undrained

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

- Drained
- Undrained ------ u↑

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

8.1.2 Mohr-Coulomb Criterion

- Failure <u>is not always</u> 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 σ - ε

- Direct shear – rate ≈ 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

$\phi_{p.s.} = 2 \text{ to } 4^o + \phi_{tri}$ loose

$= 4 \text{ to } 9^o + \phi_{tri}$ dense

$\uparrow$

- o less freed0m for movement
- o greater interlocking

Coulomb Equation

$$\tau = \underset{\substack{\uparrow \\ = 0}}{c} + \sigma \tan \phi$$

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 cohose to each other without normal force pushing the two particles together.
  (True cohesion)

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

Fig. 8.27

- Drained

  - effective stress σ
  - τ vs. σ

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

  - peak
  - residual

- Undrained

CH. 9

@ rest

p. 357 <u>active</u> $\rightarrow$ $\approx$ H/500 wall movement 0.2% of H

<u>passive</u> $\rightarrow$ $\approx$ H/100 wall movement 1% of H

@ top <u>tilt</u>

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.

Foundations
- shallow — $D \le B$ - largest load-bearing area → Bearing
- Deep — $D > B$
  - Bearing
  - Friction (side)

transfer the load from columns / wall to underlying soil mass

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

  - o Prandtl (1921) $\rightarrow$ Terzaghi (1943)

  - o 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

$$q_{ult} = c\,N_c + q\,N_q + \tfrac{1}{2}\,g\,B\,N_g$$ ---------------- • General shear failure

wall

cohesion bearing capacity factors : f(ϕ)

<u>Sq.</u> Rec.

Circle