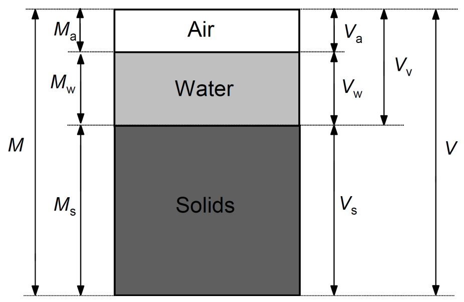
CIVE50007 SOIL MECHANICS

1 PHASE RELATIONS



Water Content: $w = \frac{M_w}{M_s}$

Specific Gravity: $G_s = \frac{\rho_s}{\rho_w}$

Degree of Saturation: $S_r = \frac{V_w}{V_v} = \frac{wG_s}{e}$

Void Ratio: $e = \frac{V_v}{V_s} = \frac{n}{1-n}$

Porosity: $n = \frac{V_v}{V} = \frac{e}{1+e}$

Specific Volume: $v = \frac{V}{V_v}\frac{1}{n} = \frac{1+e}{e}$

Bulk Density: $\rho = \frac{M}{V} = \frac{\rho_s(1+w)}{1+e} = \frac{\rho_w(G_s+S_r e)}{1+e}$

Bulk Unit Weight: $\gamma = \rho g = \frac{Mg}{V}$

Dry Density: $\rho_{dry} = \frac{\rho}{1+w}$

2 EFFECTIVE STRESS

DEFINITIONS

Phreatic Surface: Surface with zero pore water pressure

Artesian: When phreatic surface is above ground level

PRINCIPLE OF EFFECTIVE STRESS

$$\sigma' = \sigma - u$$

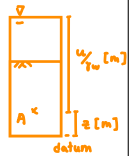
- σ' = effective stress, σ = total stress, u = pore water pressure
- If there is no flow, $u = \rho_w g h$ (hydrostatic)

3 SEEPAGE

BERNOULLI'S THEOREM

$$h = \frac{u}{\gamma_w} + z + \cancel{\frac{v^2}{2g}}$$

0



So Δh can straight away compare the height of water table $h_2 - h_1$

- h = total head, u/γ_w = pressure head, z = elevation head from datum
- $v^2/2g$ = velocity head (≈ 0 in soils)

DARCY'S LAW

$$q = Aki \Rightarrow v = \frac{q}{A} = ki$$

- q = volumetric flow rate, A = cross-sectional flow area, v = discharge/flow velocity
- k = soil permeability [m/s]
  - Typically, $k_{clay} < 10^{-8} < k_{silt} < 10^{-5} < k_{sand} < 10^{-2} < k_{gravel}$
- i = hydraulic gradient = rate of change of total head h with distance s in the flow direction:

$$i = -\frac{\partial h}{\partial s}$$

TYPICALLY ICRIT = 1

  - **Quick Conditions:** When $i_{crit} = \frac{\gamma_s}{\gamma_w} - 1$, which causes effective stress $\sigma' = 0$

FLOW NETS

EP ARE LINES WITH EQUAL PIEZOMETRIC HEAD (PRESSURE+ELEVATION)

1. If soil is anisotropic, scale the x-dimension by $\sqrt{k_y/k_x}$ and use $k = \sqrt{k_x k_y}$
2. Draw equipotential (EP) and flow lines (FL) along all boundary conditions
3. Draw FLs such that the N_{flow} flow channels formed have the same total volume
4. Draw EPs with N_{drop} equal head losses $\Delta h = \frac{H}{N_{drop}}$, perpendicular to all FLs, forming curvilinear squares
5. Calculate flow $\boxed{q = kH\frac{N_{flow}}{N_{drop}}}$ [m²/s], where H is the driving head

PERMEABILITY OF STRATIFIED SOILS

For soil of total depth d layered into n distinct beds of thickness d_i of different permeability k_i, the overall permeability k is:

Horizontal flow, parallel to strata:

$$k = \frac{1}{d}\sum_{i=1}^{n} k_i d_i$$

$kd = \sum k_i d_i$

Vertical flow, perpendicular to strata:

$$k = d\left(\sum_{i=1}^{n}\frac{d_i}{k_i}\right)^{-1}$$

$\frac{d}{k} = \sum \frac{d_i}{k_i}$

4 SOIL CONSOLIDATION

DEFINITIONS

When a saturated soil's load is raised by $\Delta\sigma$:

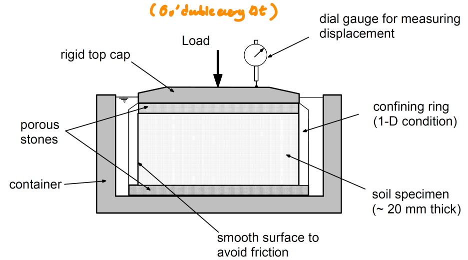
- **Drained:** Pore water readily flows out (coarse-grained)
  $\Delta u = 0$
  $\Delta\sigma = \Delta\sigma'$ directly, no Δu
- **Undrained:** Water slowly flows out (fine-grained)
  $\Delta\sigma' = 0$
  initially $\Delta\sigma = \Delta u$, over time $\Delta\sigma = \Delta\sigma'$
  - Excess pore pressure Δu causes pressure gradient, and thus, transient flow until the drained condition is reached
  - Rate of Δu dissipation depends on drainage path length (pg 30)

Consolidation: Process of soil loading and compression

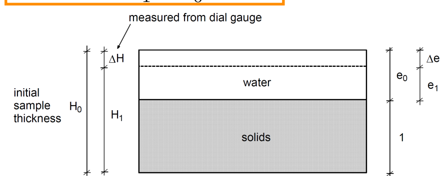
- Most revelant to fine-grained soils of low permeability
- Particles reorientate, raising inter-particle forces and reducing soil volume
- Assumes the loaded area is much larger than the soil layer's thickness

OEDOMETER TEST

Two main questions: (Two different graph to remember)
1. Fixed incremental σ_v', given $H(t) \to$ find C_v
2. t not important (σ_v' double every Δt), given $H(\sigma_v') \to$ find m_v, C_c



1. Set initial pressure at in-situ effective stress $(u \approx 0, \sigma_v' \approx \Delta\sigma_v)$ ← increase σ such that $\Delta\sigma = \Delta\sigma'$ but up to the point where any additional $\Delta\sigma$ will start to increase Δu.
2. Double the loads applied every 24 hours, assuming Δu dissipates completely in between each increment
3. Plot the sample's compression/void ratio after each increment against effective stress
   - Compression: $\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$
   - $\boxed{\text{Void ratio: } e_1 = e_0 - \Delta e}$



we want to plot e against σ_v' (we have σ_v' — doubled every day) to find e for every σ_v':
- we measure H_0 and ΔH for each $\Delta\sigma_v'$, - we calculate $e_0 = \frac{w_0 G_s}{S_r}$ then we can use formula $\frac{\Delta H}{H_0} = \frac{\Delta e}{1+e_0}$ to get Δe!

4. Calculate soil compressibility parameters

- **Coefficient of Volume Compressibility:** Volume change per unit volume per increase in effective stress

need to obtain either from e against σ_v' graph or linear interpolate

$$(m_v) = \frac{1}{1+e_0}\left(\frac{\Delta e}{\Delta\sigma'}\right) = \frac{1}{H_0}\left(\frac{H_0 - H_1}{\sigma_1' - \sigma_0'}\right)$$

+ive!

  - Typically, $m_v = 10^{-3}$ m²/kN for soft clay and 10^{-4} m²/kN for stiff clay

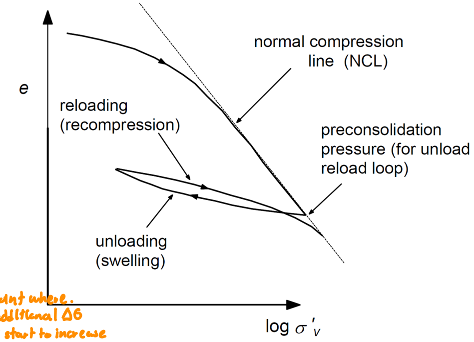
- **Long-term Settlement:** after obtaining ρ_∞, if we have U we can find ρ at any time! $\rho_t = U \times \rho_\infty$

$$m_v = \frac{1}{H}\frac{\rho_\infty}{\Delta\sigma_v'} \Rightarrow \rho_\infty = m_v H \Delta\sigma_v'$$

- **Compression Index:** Gradient of linear part of consolidation curve

$$C_c = \frac{e_0 - e_1}{\log\sigma_1' - \log\sigma_0'}$$

CONSOLIDATION CURVE



- **Normally-Consolidated (NC):** Never experienced greater pressures than what is being applied
- **Over-Consolidated (OC):** Experienced unloading at some point
  - Compresses much less than NC clay (much gentler gradient in the loop relative to NCL)
  - Exceeding preconsolidation pressure σ_c' causes a steep change in consolidation rate as the curve abruptly follows the NCL

CHECK 2.4 NOTES: BEDDING IN, PRIMARY AND SECONDARY CONSOLIDATION GRAPHS

THEORY OF 1-D CONSOLIDATION

Terzaghi's 1-D consolidation equation:

$$c_v \frac{\partial^2 u}{\partial z^2} = \frac{\partial u}{\partial t}$$

$U = 2\sqrt{\frac{T_v}{\pi}}$ for linear part.

Coefficient of Consolidation: Determines how long compression will take

T_v depends on U (from graph) — t is the time it takes

★ remember T_v and U are closely related!

(1) $\boxed{c_v = \frac{k}{\gamma_w m_v}}$, $T_v = \frac{c_v t}{H^2}$ ← H might need to ÷2 if 2-way ★

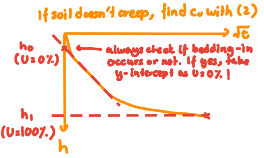
- For clays in m^2/year, $0.5 <$ stiff $< 5 <$ soft $< 25 <$ silty
- If soil does not creep, (2) $c_v = \frac{\pi d^2}{16 t_{50}}$, where t_{50} is the time when $U = 50\%$

(and uncorrected if there's bedding-in) these uses H at t_0! $d = H(t_0)$ or $\frac{1}{2}H(t_0)$ 1-way 2-way

(3) $\boxed{c_v = \frac{0.848 d^2}{t_{90}}}$ if creep (U: 90%)

Eqn 1,2,3 all are used to find Cv! (1) are used less as it require m_v, but useful if we have m_v already. (2) and (3) are used most of the time.
Eqn (4) is usually used to apply Cv to find t!

If soil doesn't creep, find c_v with (2)



I WAS TOLD THAT CREEP WON'T BE COMING OUT IN THE EXAM SO WE DON'T NEED TO KNOW HOW TO FIND CV FROM H-SQRT(T) GRAPH WHEN THERE IS CREEP (U=90%) METHOD

Average Degree of Consolidation:

$$\boxed{U = \frac{\rho t}{\rho_\infty}} = \frac{e_0 - e}{e_1 - e_0} = \frac{\sigma' - \sigma'_0}{\sigma'_1 - \sigma'_0}$$

- Notes pg. 41-42: T_v tables for a given U

5 SHEAR STRENGTH (recommend watching slides this is too summarised)

CONDITIONS

(short term)
Undrained: Sample is sealed, so volume <u>cannot change</u>

- ~~**Coulomb failure criterion:** $\tau_f = c' + \sigma'_n \tan\phi'$~~
- ~~Cohesion $c' = 0$ for sands~~
- ~~Critical in <u>short-term heavy loading</u> (eg. foundations)~~

short term is tresca failure! $\tau_f = S_u$

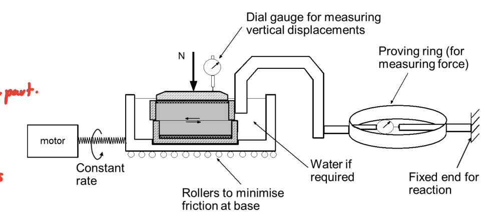
(long term)
Drained: Water can move in and out during the test, allowing $u = 0, \sigma' = \sigma$ under <u>slow</u> shearing

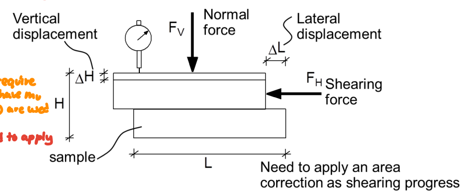
- ~~**Tresca failure criterion:** $\tau_f = c_u = S_u$~~
- ~~Critical in <u>long-term unloading</u> (eg. slope exacation)~~

long term is Mohr-Columb failure! $\tau_f = c' + \sigma_n' \tan\phi'$

SHEAR BOX TEST

only find $\tau_f = c' + \sigma' \tan\phi'$ (columb failure criterion)
doesn't find $\tau_f = S_u$ (tresca failure criterion)





Procedure

1. Consolidate identical samples at different F_V values and apply shear

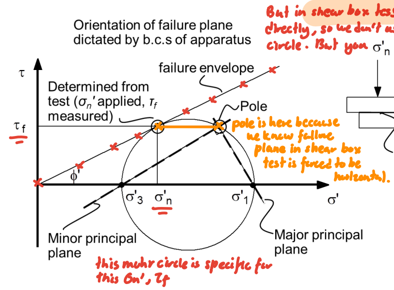
$$\sigma'_n = \frac{F_V}{L^2}, \quad \tau = \frac{F_H}{L^2}$$

   - Contact area is usually square: $A = L^2$
   - $\Delta L = v_{motor} t$, ΔH read from dial gauge
2. Plot σ'_n against τ_{max} of all results to get failure envelope
   - Dense soils have higher gradients, higher ϕ' and higher τ_{max} values
   - **Critical State:** When τ and sample volume becomes constant
     - Under very large strains, clay platelets in clays with high liquid limits and plasticity indices re-align to the shear plane, reducing strength to the <u>residual strength</u>, which has parameters with subscript r
   - **Drained Results**
     - NC & lightly-OC: Contract with $e \downarrow$
     - OC: Expand with $e \uparrow$
   - **Undrained Results**
     - NC & OC: Constant volume with constant e
   - Notes pg. 55-58: Diagrams between τ_{max}, τ, $\frac{\Delta L}{L_0}$, e and σ'_n

Mohr's Circle Of Stress

★ In triaxial test, we can determine a mohr's circle by having $\sigma_1' = \sigma_a' = \sigma_r' + \frac{F_a}{A}$ and $\sigma_3' = \sigma_r'$, then we can find τ_f at intersection between failure envelope and mohr's circle.

But in shear box test, we obtain (σ_n', τ_f) directly, so we don't usually need a mohr circle. But you can still find it by reverse calculate.
- each σ_n', τ_f has one mohr circle.



- Inclination of failure plane determined by shear box
- At failure, circle is tangential to the failure envelope

Advantages

- Simple to set up
- Good for sands, as density can vary
- Can perform interface tests

Disadvantages

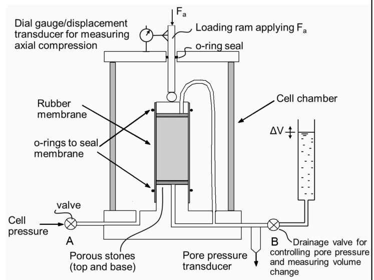
- No pore water pressure control
- Effective stress known only if $u = 0$
- Non-uniform shear stresses over contact area
- Shear area changes throughout

TRIAXIAL TEST

HOW TO FIND INTITIAL STRESS IN THE SOIL SAMPLE
given σ_0 and u_0 (in-situ)
find σ_1 and u_1 (sample)
1. find $\sigma_0' = \sigma_0 - u_0$
2. $\sigma_1 = 0$ ($\Delta\sigma = \sigma_1 - \sigma_0$)
3. $\Delta\sigma' = \Delta\sigma - \Delta u$ — find this
$\Delta\sigma' = 0$!
4. $u_1 = u_0 + \Delta u$
$\therefore u_1 = u_0 + \Delta u$
$\sigma_1 = 0$
$\sigma_1' = \sigma_0'$

Aim: To find τ_f (shear stress at failure)

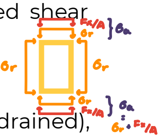
Two type of τ_f can be found, 1. $\tau_f = S_u$ and 2. $\tau_f = c' + \sigma' \tan\varphi'$
- τ_f doesn't depends on σ' - uses U-U test
- τ_f depends on σ' - uses C-U test



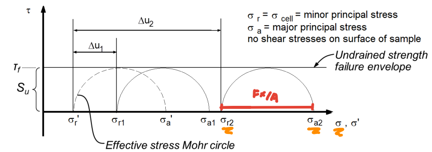
Setup

1. Install soil sample to 2:1 H:D ratio with porous plattens within an impermeable latex membrane, sealed with O-rings
2. Fill cell chamber with water and increase cell pressure $\sigma_{cell} = \underline{\sigma_r}$ radial stress
   - Isotropic loading ($\Delta\sigma \uparrow = \Delta u \uparrow, \Delta\sigma' = 0$)
   - If sample is fully saturated, no volume change occurs

Unconsolidated-Undrained (U-U): Apply F_{ax} with loading ram until undrained shear strength S_u is reached σ_a is axial stress.



- Deviator Stress/Principal Stress $F_{ax}/A = \sigma_a - \sigma_r$
- Corrected Area A: $\frac{A_0}{1-\varepsilon_a}$ (undrained), $A_0\left(\frac{1-\varepsilon_v}{1-\varepsilon_a}\right)$ (drained), where $\varepsilon_a = \frac{\Delta H}{H_0}$ and $\varepsilon_v = \frac{\Delta V}{V_0}$



(we don't actually "find" σ' mohr circle but we can calculate it from σ mohr circle → but need the hassle of finding it!)

- At failure: $\tau_f = S_u = \frac{\sigma_a - \sigma_r}{2}$, $\phi_f = 0$
- S_u increases with quicker shearing of drier, smaller samples

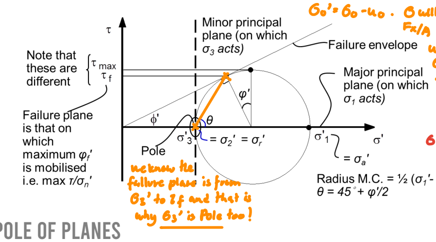
Consolidated-Undrained (C-U): Apply back pressure $P_{back} = u$ to obtain in-situ σ', then apply F_{ax} with drainage closed as in U-U test

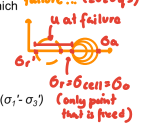
which is a range so we can get multiple mohr's circle.

- Sample stays fully saturated
- At failure: $\tau_f = \frac{1}{2}(\sigma'_a - \sigma'_r)\sin 2\theta, \sigma'_n = \frac{\sigma'_a + \sigma'_r}{2} + \frac{\sigma'_a - \sigma'_r}{2}\cos 2\theta$, where $\theta = 45° + \frac{\phi'}{2}$
- Note: $\tau_{max} = \frac{\sigma'_a - \sigma'_r}{2} \neq \tau_f$

when doing C-U:
★ Initial effective stress σ_0' is the one we try to imitate in-situ's σ'! so with σ_0' known: u_0=UBP, σ_0=σ_{cell} are set such that $\sigma_0' = \sigma_0 - u_0$. σ will be constant while F_a/A is added... u will increase while σ' will decrease till failure... (2020 Q3)

Mohr's Circle Of Stress

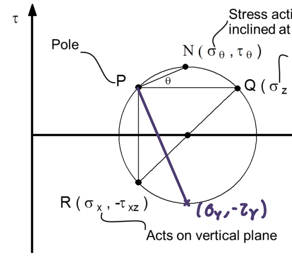


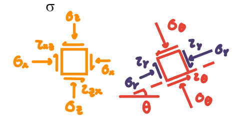


POLE OF PLANES

Pole: Intersection point P of lines extending from Q and R

- θ = orientation of the plane that N's stress state acts on

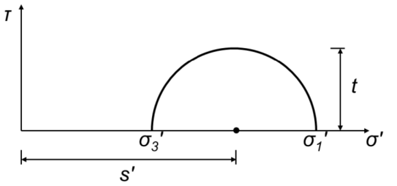




STRESS INVARIANTS

Stress Invariants: $s = s'$ and $t = t'$ be the centre and radius of the Mohr circle:

$$t = t' \begin{cases} t = \frac{1}{2}(\sigma_1 - \sigma_3) & , \quad s = \frac{1}{2}(\sigma_1 + \sigma_3) \\ t' = \frac{1}{2}(\sigma_1' - \sigma_3') & , \quad s' = \frac{1}{2}(\sigma_1' + \sigma_3') \end{cases}$$

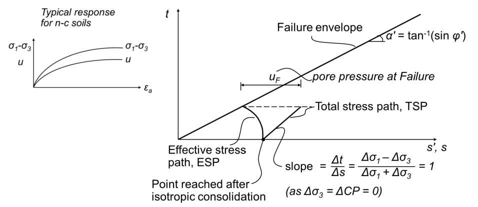


STRESS PATHS

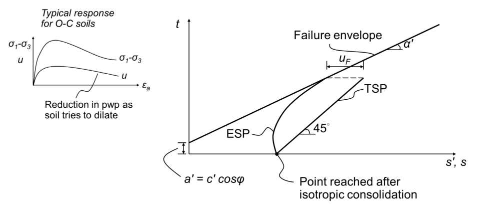
Stress Path: The lines that σ and σ' follow as shearing occurs

- u = horizontal distance between effective and total stress paths (ESP and TSP)

NC Clay (C-U test):



OC Clay (C-U test)



6 GEOTECHNICAL ANALYSIS

REQUIREMENTS FOR A SOLUTION

Equilibrium: $\sum F_{ext} = \sum F_{int} \Rightarrow \sum F = 0$

$$\sum F_x = 0 \Rightarrow \frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{yx}}{\partial y} + \frac{\partial \tau_{zx}}{\partial z} + \gamma = 0$$

$$\sum F_y = 0 \Rightarrow \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} + \frac{\partial \tau_{zy}}{\partial z} = 0$$

$$\sum F_z = 0 \Rightarrow \frac{\partial \tau_{xz}}{\partial x} + \frac{\partial \tau_{yz}}{\partial y} + \frac{\partial \sigma_z}{\partial z} = 0$$

- τ_{ij} is the stress component parallel to axis j, acting on a plane perpendicular to axis i
- Self weight γ acts in the x-direction
- Compressive stresses are positive

Compatibility: No overlapping or generation of holes

$$\varepsilon_x = \frac{\partial u}{\partial x}, \quad \varepsilon_y = \frac{\partial v}{\partial y}, \quad \varepsilon_z = \frac{\partial w}{\partial z}$$

$$\gamma_{xy} = \frac{\partial v}{\partial x} + \frac{\partial u}{\partial y}$$

$$\gamma_{yz} = \frac{\partial w}{\partial y} + \frac{\partial v}{\partial z}$$

$$\gamma_{xz} = \frac{\partial w}{\partial x} + \frac{\partial u}{\partial z}$$

- Compressive strains are positive

Constitutive Behaviour: Material's stress-strain behaviour

$$\begin{Bmatrix} \Delta\sigma_x \\ \Delta\sigma_y \\ \Delta\sigma_z \\ \Delta\tau_{xy} \\ \Delta\tau_{xz} \\ \Delta\tau_{zy} \end{Bmatrix} = [\boldsymbol{D}]_{6\times 6} \begin{Bmatrix} \Delta\varepsilon_x \\ \Delta\varepsilon_y \\ \Delta\varepsilon_z \\ \Delta\gamma_{xy} \\ \Delta\gamma_{xz} \\ \Delta\gamma_{zy} \end{Bmatrix}$$

- Can be expressed in terms of total or effective stresses
- Commonly used to relate increments of stress and strain with $[\boldsymbol{D}]$ depending on stress history, due to non-linear behaviour of soil

- **Plane strain:** Reduction to 2D cross-sections if object is very long along z-axis

$$\varepsilon_z = \gamma_{yz} = \gamma_{zx} = 0 \Rightarrow \Delta\tau_{xz} = \Delta\tau_{zy} = 0$$

  - Non-zero stress changes: $\Delta\sigma_x$, $\Delta\sigma_y$, $\Delta\sigma_z$, $\Delta\tau_{xy}$

- **Axial symmetry:** No displacement in θ-direction and displacements in r- and z-axes are independent of θ

$$\varepsilon_r = \frac{\partial u}{\partial r}, \quad \varepsilon_z = \frac{\partial v}{\partial z}, \quad \varepsilon_\theta = \frac{u}{r}$$

$$\gamma_{rz} = \frac{\partial v}{\partial r} + \frac{\partial u}{\partial z}, \quad \gamma_{r\theta} = \gamma_{z\theta} = 0$$

  - Non-zero stress changes: $\Delta\sigma_r$, $\Delta\sigma_z$, $\Delta\sigma_\theta$, $\Delta\tau_{rz}$

Boundary Conditions: Force and displacement constraints depending on the situation

7 LIMIT EQUILIBRIUM METHOD

1. Assume a failure criterion holds everywhere along an arbitrary failure surface
2. Treat the failing soil block as a rigid body and apply equilibrium balance
3. Derive an expression for the desired quantity
4. Optimise to find the critical angle and quantity value

8 STRESS FIELD METHOD

ASSUMPTIONS

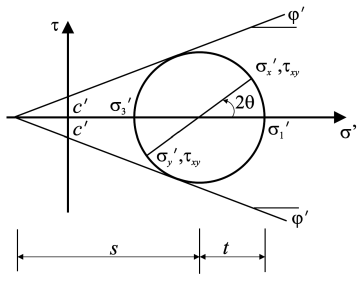
- 2D plane strain conditions

$$\frac{\partial \sigma_x}{\partial x} + \frac{\partial \tau_{xy}}{\partial y} = 0, \quad \frac{\partial \tau_{xy}}{\partial x} + \frac{\partial \sigma_y}{\partial y} = \gamma$$

- Mohr-Coulomb failure criterion everywhere in the soil

$$\sigma_1' - \sigma_3' = 2c' \cos\phi' + (\sigma_1' + \sigma_3') \sin\phi'$$

  - Rewriting the criterion in terms of s and t:

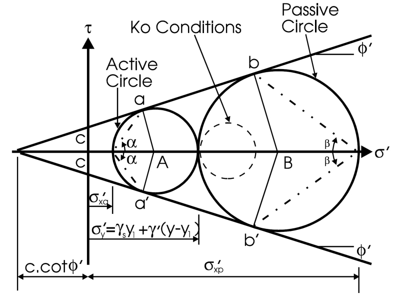


$$s = c' \cot\phi' + \frac{\sigma_1' + \sigma_3'}{2} = c' \cot\phi' + \frac{\sigma_x' + \sigma_y'}{2}$$

$$t = \frac{\sigma_1' - \sigma_3'}{2} = \sqrt{\left(\frac{\sigma_x' - \sigma_y'}{2}\right)^2 + \tau_{xy}^2}$$

$$\Rightarrow t = s \sin\phi'$$

RANKINE STRESS STATES



Active State (Extension)

- Earth Pressure Coefficient: $K_a \approx 0.2 - 0.3$
- Earth Pressure:
  - Flat Ground:

$$\sigma_{xa}' = \sigma_y' \tan^2\left(\frac{\pi}{4} - \frac{\phi'}{2}\right) - 2c \tan\left(\frac{\pi}{4} - \frac{\phi'}{2}\right)$$

$$= \sigma_y' K_a - 2cK_{ac}$$

  - Slope of Inclination β:

$$\sigma_{xa}' = \sigma_y' \cos\beta \frac{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'}}{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'}}$$

- Dotted circle inverts and expands towards the left
- Maximum stress obliquity/Failure planes inclined at $\pm\alpha = \pm(\frac{\pi}{4} + \frac{\phi'}{2})$ to the direction of x-axis

Passive State (Compression)

- Earth Pressure Coefficient: $K_p \approx 3.0$
- Earth Pressure:
  - Flat Ground

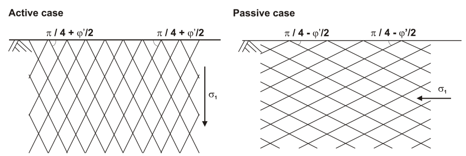
$$\sigma_{xp}' = \sigma_y' \tan^2\left(\frac{\pi}{4} + \frac{\phi'}{2}\right) + 2c \tan\left(\frac{\pi}{4} + \frac{\phi'}{2}\right)$$

$$= \sigma_y' K_p + 2cK_{pc}$$

  - Slope of Inclination β:

$$\sigma_{xp}' = \sigma_y' \cos\beta \frac{\cos\beta + \sqrt{\cos^2\beta - \cos^2\phi'}}{\cos\beta - \sqrt{\cos^2\beta - \cos^2\phi'}}$$

- Dotted circle expands towards the right
- Maximum stress obliquity/Failure planes inclined at $\pm\beta = \pm(\frac{\pi}{4} - \frac{\phi'}{2})$ to the direction of x-axis



9 ENGINEERING GEOLOGY

DEFINITIONS

Soil: Material with unconfined compressive strength (UCS) < 0.6 MPa

Rock: Material with UCS > 0.7 MPa

Rockmass: Intact rock and its fractures

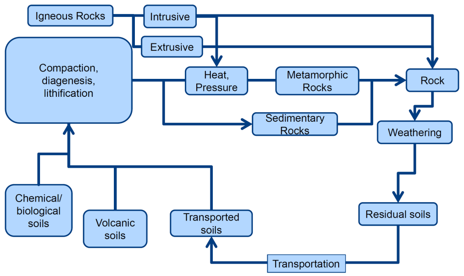
Igneous (intrusive): From subsurface magma

Igneous (extrusive): From magma ejected above ground

Sedimentary: From compaction & lithification of sediment

Diagenesis: Aggregation of sediment into sedimentary rock

Metamorphic: From intense temperature and pressure



10 SITE INVESTIGATION

AIMS

- Document strength and behaviour of rocks and engineering soils in the area
- Identify potential ground hazards to reduce risk
- Ensure ground is fit for construction

STAGE 1: INITIAL SCOPING

Desk Studies: To interpret existing data and reduce scope of investigation

- Geological & hydrological maps, historical photographs, seismic data, environmental records, aerial photographs, satellite remote sensing data, etc
- **Benefits:** Reduces costs, highlights problems early and aids design process

Site Visits: To visually assess the site's features

- Slope angles, signs of movement, rockface outcrops (eg. cliffs, quarries), land-use, vegetation changes, groundwater conditions (eg. springs), site access for workers/equipment (eg. overhead cables, road widths), existing structural damage
- **Benefits:** Understand locals' problems and build stakeholder relations

STAGE 2: FIELDWORK

Geomorphologic Mapping: To assess surface conditions by field surveys & aerial/satellite photographs

- Distribution & geometry of rock deposits
- Surface features (eg. runoff & drainage patterns, slope angles)
- Extent of permafrost
- Benefits: Cheap

Geophysical Surveys: To search for hidden geological features for siting boreholes/pits and interpolate borehole information

- Electromagnetic, electric resistivity, gravitational, magnetic, seismic methods, ground-penetrating radar, ground motion monitoring

Trial Pits & Trenches: To assess sub-surface conditions by digging out ground cross-section

- Benefits: Assess 3D nature of rock, easy to extract samples for testing in lab or in-situ

Boreholes: To assess subsurface conditions by drilling deep cores

- Spacing
  - Buildings: 10 - 30 m
  - Road Lines: 30 - 300 m
  - Landslides: > 5 in line for profile
- Depth
  - Generally, 1.5 × foundation width + 10 m control hole and 3 m below rockhead
- Drilling Methods
  - Auger Rotary Drilling
    - Rotating auger progressively digs soil out
    - Completely destroys soil fabric
    - Soils only, shallow depths
  - Cable Percussion
    - Hammers core barrel into the ground
    - Yields disturbed samples
    - Soils & weak rocks, 15 - 40 m depth
    - Cheap and commonly used
  - Rotary Drilling Rig
    - Long rotating drill with cooling lubricant to flush chippings to the surface
    - Yields intact cores
    - Hard rocks, > 100 m depth
    - Deep, intact samples, but expensive

Laboratory Tests: To characteristic rock properties on-site or in laboratory

- Point load tests, index testing, unconfined compressive strength tests, Schmidt Hammer, Brazilian, box & ring shear, triaxial tests

STAGE 3: REVIEW

- Monitor excavation and construction

CORE LOGGING

Material: Rock properties

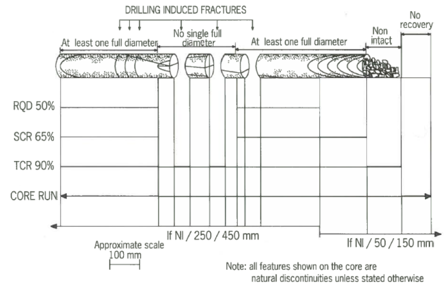
- Unconfined compressive strength (UCS)
- Bedding/layering
- Colours & name

Discontinuity: Fractures in the rock

- Fracture spacing, orientation & frequency
- Fracture persistence, size and shape
- Fracture roughness
- Overall fracture sets & block size

Mass: Combined rockmass properties

- Total Core Recovery (TCR): Core material recovered over total core run
- Solid Core Recovery (SCR): Intact core material with ≥1 full diameter over total core run
- Rock Quality Designation (RQD): Total intact core pieces >100 mm long over total core run
- Fracture Index/Spacing: Number/Spacing of fractures per unit length
- Rock Mass Rating (RMR): Overall summary rating of all rock properties
  - Other ratings: Q Index, Geological Strength Index



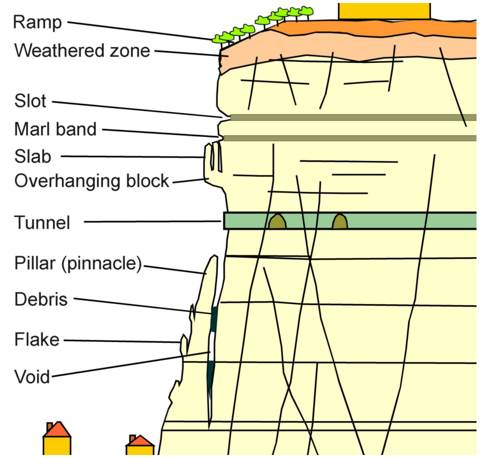
11 ROCKMASS INSTABILITY

Hazard: Anything that could cause human harm

Vulnerability: Chance for human harm

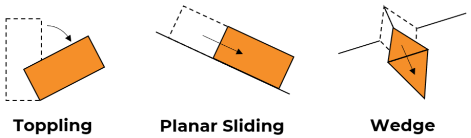
Risk: Hazard × Vulnerability

CLIFF ANATOMY



CLIFF COLLAPSE

Failure Types



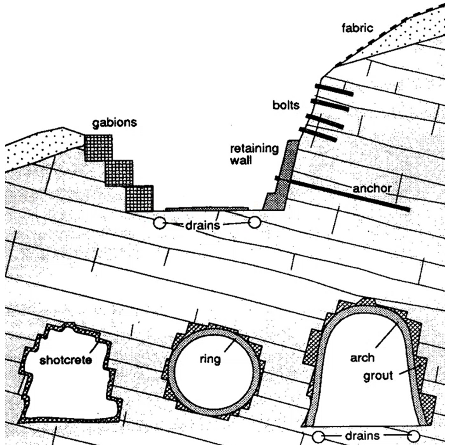
Physical Causes

- Low cohesion in unconsolidated material
- Close fracturing (bedding, joints, schistosity) in solid rock
- Impermeable layers that obstruct flow and provide lubricated slip surfaces
- Presence of 'slippery' rocks (eg. clay, coal)
- Downslope dip of potential slip planes

Failure Triggers

- External Vibrations: Earthquakes, traffic
- Loading: Buildings, snow, debris, vegetation
- Toe Removal: Loss of underlying support
- Soil Strength Loss: Weathering, porewater changes, old slip surface reactivation

Excavation Stabilisation Measures



- **Prevent toppling by securing slope**
  - Netting
  - Rock bolts
- **Prevent sliding by obstruction**
  - Anchored retaining walls
  - Gabions
- **Pore water pressure control**
  - Drains at base of excavation
- **Tunnel stabilisation**
  - Shotcrete
  - Segmented concrete rings/arches
  - Grout filling

12 STEREOGRAPHIC PROJECTION

Stereographic Projection: Projection of a hemisphere's 3D surface onto its 2D equatorial plane

Great Circle: Any circle on a sphere's surface such that they share the same centre

Pole: Point of intersection between the hemisphere and a line that passes through the sphere's centre and is perpendicular to the fracture plane

Dip: Inclination angle below horizontal plane

Dip Direction: 3-digit bearing from North

REPRESENTATION

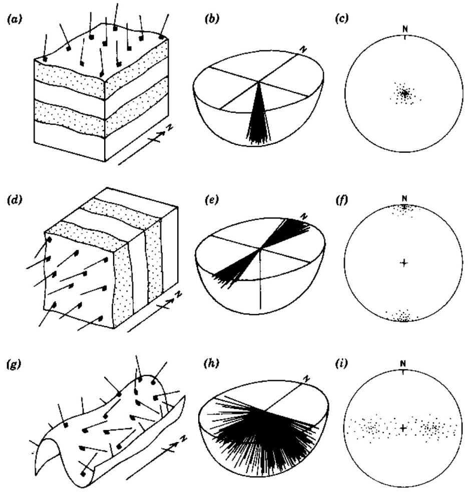
For a dip of x to bearing y:

1. Place tracing paper on top of stereonet and pin the centres together
   - Latitude and longitude lines are all spaced 2° apart
2. Count y° clockwise from the North pole and mark the perimeter
3. Rotate the tracing paper to align this mark with the E-W axis
4. Count from the mark towards the stereonet's centre along the E-W axis
   - **Line:** Single point x° from the mark
   - **Plane:** Entire longitudinal line at x°
   - **Pole:** Single point (90 + x)° from the mark representing the entire plane
5. Rotate the tracing paper back to its original position

Note: Pole projections are typically used to avoid clutter

POLE PROJECTION PATTERNS

- **Horizontal Bedding:** Clustering near the centre
- **Vertical Bedding:** Clustering near opposite edges
- **Cylindrically-Folded Bedding:** Clustering along a line
  - Line is parallel to the actual folding direction



KINEMATIC ANALYSIS

Assumptions

- Shear strength of sliding surface is only from friction angle
- Cohesion is zero

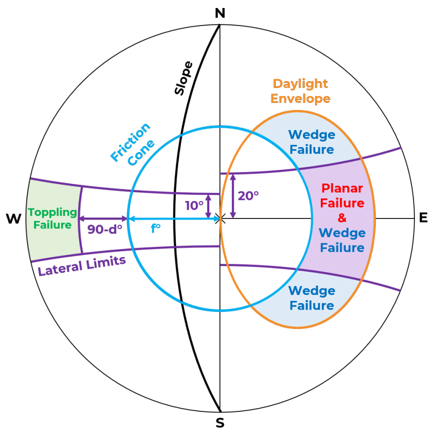
Procedure

1. Set reference axis (eg. road) as the stereonet's N-S axis
2. Draw the great circle of the desired slope with dip d°
3. **Daylight Envelope:** Connect the poles for all points along this great circle
4. **Friction Cone:** Draw a circle of radius f° in the stereonet centre, where f is the friction angle of the slope
5. **Lateral Limits:** Draw latitude lines 10° and 20° from either side of the stereonet centre perpendicular to the slope
   - Imposed by adjacent blocks on the slope preventing movement

6. **Planar Failure:** Poles that lie in the daylight envelope between the lateral limits, but outside the friction cone (purple region)

7. **Toppling Failure:** Poles between 10° lateral limits on the convex side of the great circle and (90 - d)° in that direction from the friction cone (green region)
   - Extra (90 - d)° is to compensate for part of the slope's great circle 'overlapping' with the friction cone

8. **Wedge Failure:**
   (a) Group poles that lie on the concave side of the slope's great circle
   (b) Draw the great circles of these groups' centroids and find their intersection points
   (c) If the poles of the intersection points of these great circles lie in the blue or purple regions, wedge failure is possible.



Limitations

- Only tells you if block failure is possible, but not when it will occur
- Does not consider external forces from water & reinforcements (eg. rock bolts)

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