CIV3247 – Geomechanics II

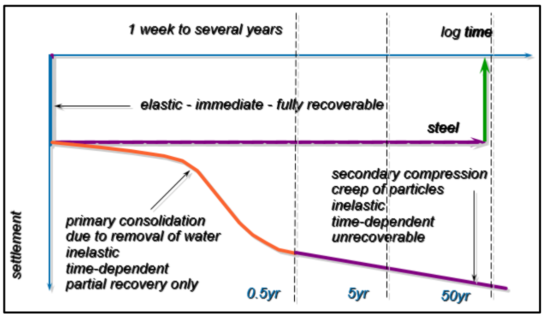
Table of Contents

1. Consolidation & Oedometer Testing

1.1 Total Settlement

- Total settlement = Elastic Deformation + Primary Consolidation + Secondary Consolidation
  - Elastic deformation:
    - Deformation of soil & rock grain
    - Compression of air & water in voids
  - Primary consolidation:
    - Drainage of water & air from voids allowing compression of soil skeleton
  - Secondary consolidation:
    - Creep movements (grains rearrange themselves to compact further after time)

- <u>Settlement Timing</u>



- Once loaded, immediate elastic settlement but is fully recoverable.

- Primary consolidation then occurs due to water removal (time dependant), can only get partial recovery, takes 1 week to several years

- Then Secondary compression occurs due to compression of soil particles by creep, un-recoverable (Need to test for consolidation settlement if footing is on clay)

1.2 Excess Pore water Pressure

- How long does it take for pore pressures to stabilize?

  Remember: When soil (saturated clay) is loaded, pore pressures (*u*) initially takes the load, after time water slowly squeezes bcas <u>*u*</u> is no longer in equilibrium. This flow then ↓ *u* back towards the pre-load value. The soil skeleton now carries ↑ load i.e effective stress (σ') ↑

Consolidation of Clay:	**Consolidation of Sand:**
- Permeability of clay is low - Drainage occurs slowly ∴ settlement & strength gained are DELAYED - Processes can be separated (elastic, primary and secondary consolidation)	- Permeability of sand is high - Drainage is instantaneous ∴ settlement is IMMEDIATE - Elastic & consolidation processes cannot be isolated - Primary Consolidation is incorporated in the elastic parameters - Secondary compression is not observed in sands

Note: We don't consider consolidation of sand bcas it occurs too quickly to consider it separately

Compaction	**Consolidation**
Compaction = solid soil particles are packed more closely together by mechanical means It is achieved through reduction of air voids	Consolidation = soil particles are packed more closely together under the application of static loading. It is achieved through gradual drainage of water

- Time for pore pressure stabilization depends on:
  1. Volume of water to be expelled, which is affected by:
     - Applied pressure
     - Soil compressibility
  2. Rate at which water is expelled, affected by:
     - Soil permeability & compressibility
     - Length of drainage path

1.3 Differential Equation for Consolidation

- **Coefficient of volume change/compressibility (m_v)**

$$m_v = \frac{\Delta h / h}{\Delta\sigma'} = \frac{change\ in\ height/height}{change\ in\ effective\ stress} = \frac{-\Delta\varepsilon}{\Delta\sigma'} = \frac{-(\Delta vertical\ strain)}{\Delta effective\ stress} = \frac{1}{D} = \frac{1}{constrained\ modulus}\ [\frac{1}{kPa}]$$

Where $\Delta\sigma'$ = - Δu (change in pore water pressure)

As C_c ↑ m_v ↑

U (%)	T (case 1)
0	0.000
10	0.008
20	0.031
30	0.071
40	0.126
50	0.197
60	0.287
70	0.403
80	0.567
90	0.848
100	∞

- **Average degree of consolidation (U)**

Average degree of consolidation = $\frac{consolidation\ at\ time\ t}{Final\ consolidation} = \frac{S_t}{S_\infty}$

For case of two-way drainage:

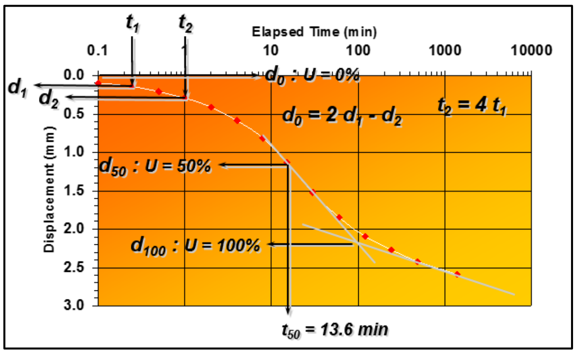
T = time factor = $\frac{c_v * t}{H_{dr}^2}$

- **Coefficient of consolidation (C_v)**

$$C_v = \frac{k}{m_v * \rho_w * g} = \frac{coef.\ of\ permeability}{coef.\ of\ compressibility * density\ of\ water * gravity} = \frac{k}{m_v * \gamma_w}$$

To calculate Cv:

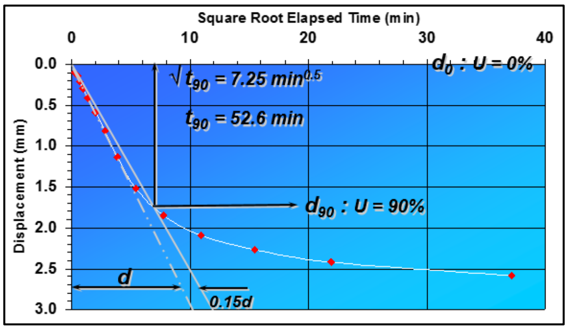
Casagrande (log time) Method



- Draw line tangent to primary consolidation (middle section)
- Draw line tangent to secondary compression (end section)
- Intersection = end of primary consolidation (d_{100}: U = 100%)
- Find d_0: choose a t_1 then find $t_2 = 4*t_1$
  so $d_0 = 2d_1 - d_2$
  then d50 = $\frac{d_0+d_{100}}{2}$ & read off t_{50}

$$\boxed{C_v = \frac{T_{50} * H_{dr}^2}{t_{50}}\ [\frac{cm^2}{s}]}$$

Taylor (square root) Method



- Draw line through initial linear section
- Add 0.15d to that line
- Intersection of new line is where the d_{90} occurs ∴ read off time

$$\boxed{C_v = \frac{T_{90} * H_{dr}^2}{t_{90}}\ [\frac{cm^2}{s}]}$$

T_{90} = time factor for 90% consolidation

H_{dr} = height to drainage

t_{90} = time taken to reach 90% consolidation

Note: - C_v for Taylor > C_v for Casagrande

- H_{dr} = distance to drainage (porous material i.e sand)

- $\frac{cm^2}{s} * 3153.6 = \frac{m^2}{yr}$

- **Secondary compression (C_α)**

Caused by creep (aging phenomenon)

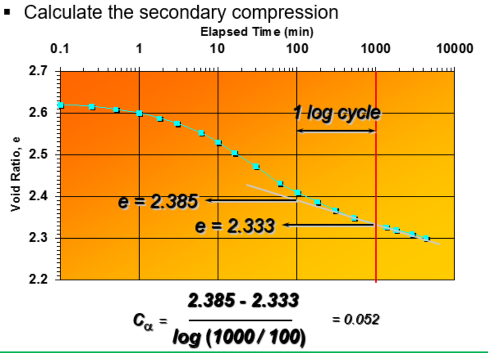
No pore pressure (u) changes

No change in effective stress (σ')

Commence after completion of primary consolidation

$$\boxed{C_\alpha = \frac{\Delta e}{\log\left(\frac{t2}{t1}\right)}\ \left[\frac{1}{1}\right] \qquad \& \qquad \rho = \frac{C_\alpha * H}{(1 + e_p)} * \log(t2/t1)}$$

▪ Calculate the secondary compression

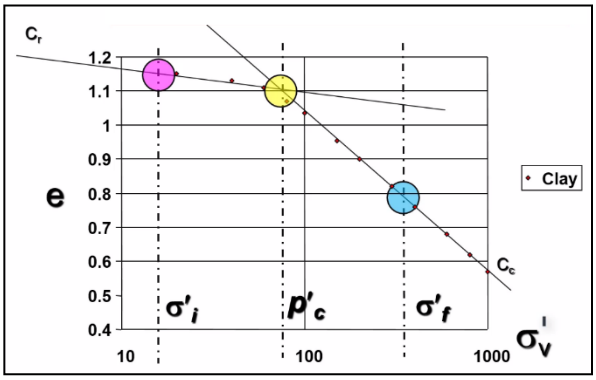


2.2 How to compute Consolidation Settlement (ρ)

- We calculate settlement from Void Ratio (e) vs. Effective stress (σ') log plot

1. Full settlement Equation

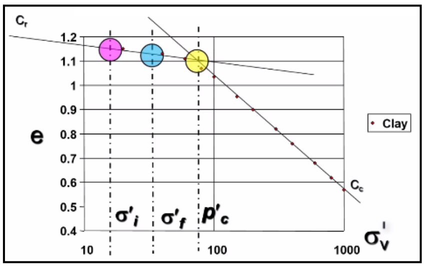
$$\rho = \sum \frac{C_r H}{1+e_o} \log \frac{p'_c}{\sigma'_i} + \frac{C_c H}{1+e_c} \log \frac{\sigma'_f}{p'_c}$$



ρ = Consolidation settlement equation [mm]
C_r = Recompression index [kPa^{-1}] (see page 6)
H = Total height of layer [m] (not height of drainage)
e_o = initial void ratio
p'_c = preconsolidation pressure [kPa] (see page 6)
σ'_i = initial effective stress [kPa] (also labelled as σ'_v)
C_c = Compression index [kPa^{-1}] (see page 6)
e_c = preconsolidation void ratio
σ'_f = final effective stress [kPa]

2. Recompression only (Overconsolidated state)

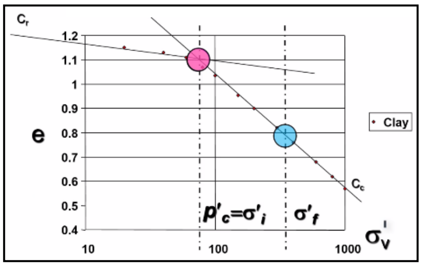
$$\rho = \sum \frac{C_r H}{1+e_o} \log \frac{\sigma'_f}{\sigma'_i}$$



- If $\sigma'_i < \sigma'_f < p'_c$
  i.e the clay soil is still overconsolidated
- Only need recompression curve portion in eqn.

3. Compression only (Normally consolidated state)

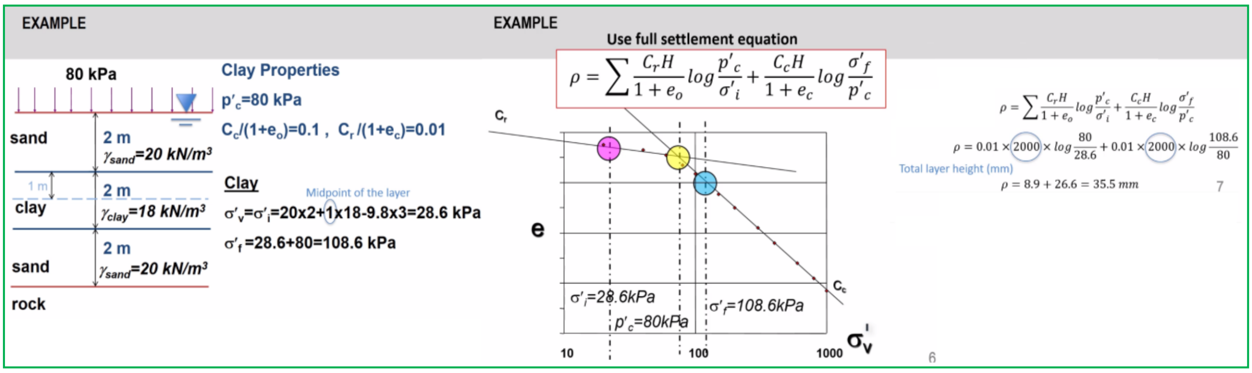
$$\rho = \sum \frac{C_c H}{1+e_c} \log \frac{\sigma'_f}{p'_c}$$



- if $p'_c \le \sigma'_I < \sigma'_f$
  i.e the clay soil is normally consolidated
- Only need compression curve portion in eqn.

Note: Overconsolidated = soil has been at higher past stress
Normally consolidated = soil hasn't been to a higher stress than what it's at presently

- E.g.



- Clay layer between two sand layers, w/ water table at ground surface (as sand is elastic, no consolidation settlement will need to be calculated for sand layers) & in this case clay will consolidate under double drainage (sand layers act as drainage)
- To find initial effective stress (σ_i'): σ_i' = overburden pressure of soil at midpoint of clay layer – pore water pressure
- To find final effective stress (σ_f'): $\sigma f'$ = $\sigma i'$ + uniform surface pressure

Now looking at e vs. effective stress curve:
- Soil passes through both recompression & compression curves ∴ use the full settlement eqn.

- Coefficient of consolidation **(C_v)**
  - Effect of stress on C_v
    - Small void changes (like on recompression line) result in rapid consolidation & consequent ↑ C_v
    - At preconsolidation pressure (p_c'), we get minimum values of C_v
  - Effect of soil fabric on C_v

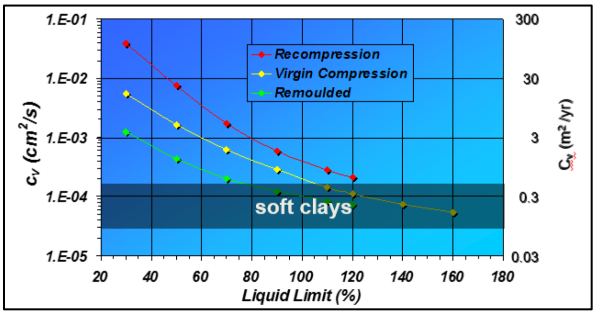
    Inhomogeneities in the soil that can change the soil coefficient of consolidation:
    - Plant roots = water travel faster in plant roots
    - Clay Fissures
    - Sand partings = coefficient of consolidation can ↑ (water travel faster in sand)
    - Sand lenses

    Usually C_v (field) > C_v (lab) ∴ Lab tests need to be corrected to account for field conditions as inhomogeneities can ↑ water travel speed

    Wick drains are often installed to allow water to travel, (shorten drainage path length to ↓ consolidation time), also placing high stresses on soil can make clay consolidate quicker

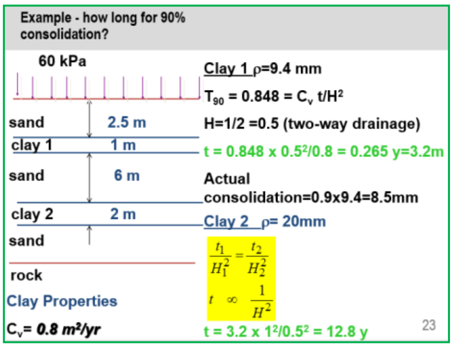
    Note: one dimensional consolidation conditions are closely applicable to field when the conditions in the field are isotropic (i.e. uniform close to homogenous deposit) the permeability is the same in all directions. Loading must be uniform and extensive on the soil

- Typical values of C_v



- Remoulded = disturbed sample
- Soft clays have ↓ C_v bcas ↑water content

- Consolidation time calculation:



For Clay 1 want time for 90% consolidation:
- Given C_v = 0.8 m²/year
- Using $\boxed{C_v = \frac{T_{90} * H_{dr}^2}{t_{90}} \text{ \& we know } T_{90} = 0.848 \text{ (see page 4)}}$
- Rearranging, $t_{90} = \frac{T_{90} * H_{dr}^2}{C_v} = \frac{0.848 * 0.5^2}{0.8}$ = 0.265 year

For clay 2:
- $t_{90} = \frac{T_{90} * H_{dr}^2}{C_v} = \frac{3.2 * 1^2}{0.8}$ = 4 years
- H_{dr} = 3.2 bcas

3.2 Quick undrained triaxial test (QU test)

- When running QU test:
  - Load undisturbed specimen into cell
  - Fill triaxial w/ water
  - ↑cell pressure (σ3), this ↑ confining pressure on specimen (want to reinstate field conditions)
  - Now keep cell pressure (σ3) constant w/ no drainage (no change in u = no change in volume when shearing)
    (Measure displacement & axial load throughout test)
  - ↑axial load through constant displacement until failure
    (Eventually sample will bulge & shear)
  - Repeat test for two other undisturbed samples from same depth & apply higher confining stress (stage test)
    (use to plot several Mohr circles)
  - Convert axial load (P) and displacement (L) into deviator stress (σ1-σ3), axial strain (ε) & cross sectional area (A):

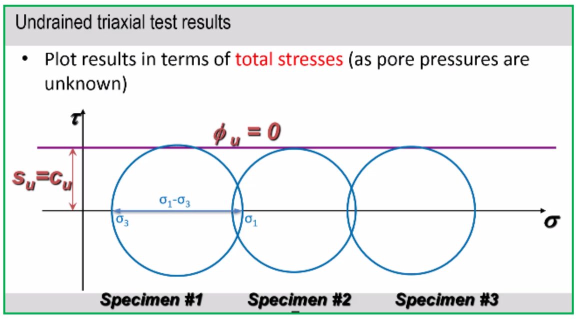
$$\boxed{A = A_0(1-\varepsilon)}$$

A = cross sectional area
A0 = initial cross sectional area
$\varepsilon = \frac{\Delta L}{L_0} = \frac{change\ in\ length}{length\ intial}$

$$\boxed{(\sigma_1 - \sigma_3) = \frac{P}{A}}$$

σ1 – σ3 = deviator stress
P = axial load
A = cross sectional area
(may need corrected area)

- ➢ Potting results from QU test



σ3 = first intersection of normal stress line = cell pressure
σ1 = second intersection on normal stress line = axial stress
Diameter of circle = deviator stress = σ1 – σ3
(remains constant)
Completing test for two other specimens we can plot results

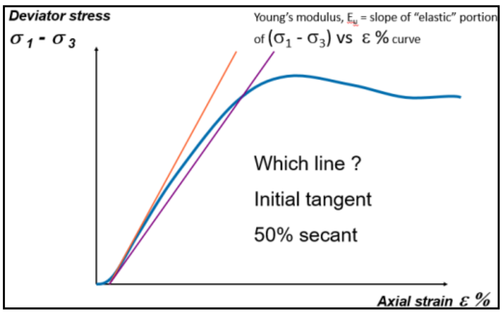
Tangent line to determine undrained friction angle (ϕ_u = 0)
Then to find undrained cohesion value ($S_u = C_u = \frac{\sigma 1 - \sigma 3}{2}$)
where S_u = undrained shear strength

- ϕ_u = 0 bcas skeleton is relatively compressible so ↑ in cell pressure (σ3) is ∴ taken by pore water (u) i.e if Δσ3 = 200 kPa then Δu = 200 kPa ∴ Δσ3' = Δσ3 – Δu = 200-200 =0

- ➢ <u>QU test results</u>
  - QU test doesn't measure the pore water pressures (u) during the test
    ∴ we cannot calculate the effective stresses (σ') at failure
    only calculates total stress (σ1 & σ3)

- ➢ <u>Why is ϕ_u assumed 0?</u>
  - in QU test no consolidation is carried out before testing
    ∴ applied σ3 only leads to ↑ u
  - So for similar specimens in the stage test we get no ↑ in shear strength
    i.e deviator stress remains constant (if sample is saturated) so the diameter of Mohr circles are all equal giving ϕ_u = 0

- To determine E_u = Youngs Modulus
  - Plot deviator stress (σ1-σ3) vs. axial strain (ε_a)
  - E_u = slope in elastic portion
  - Take 50% secant as slope (purple line)

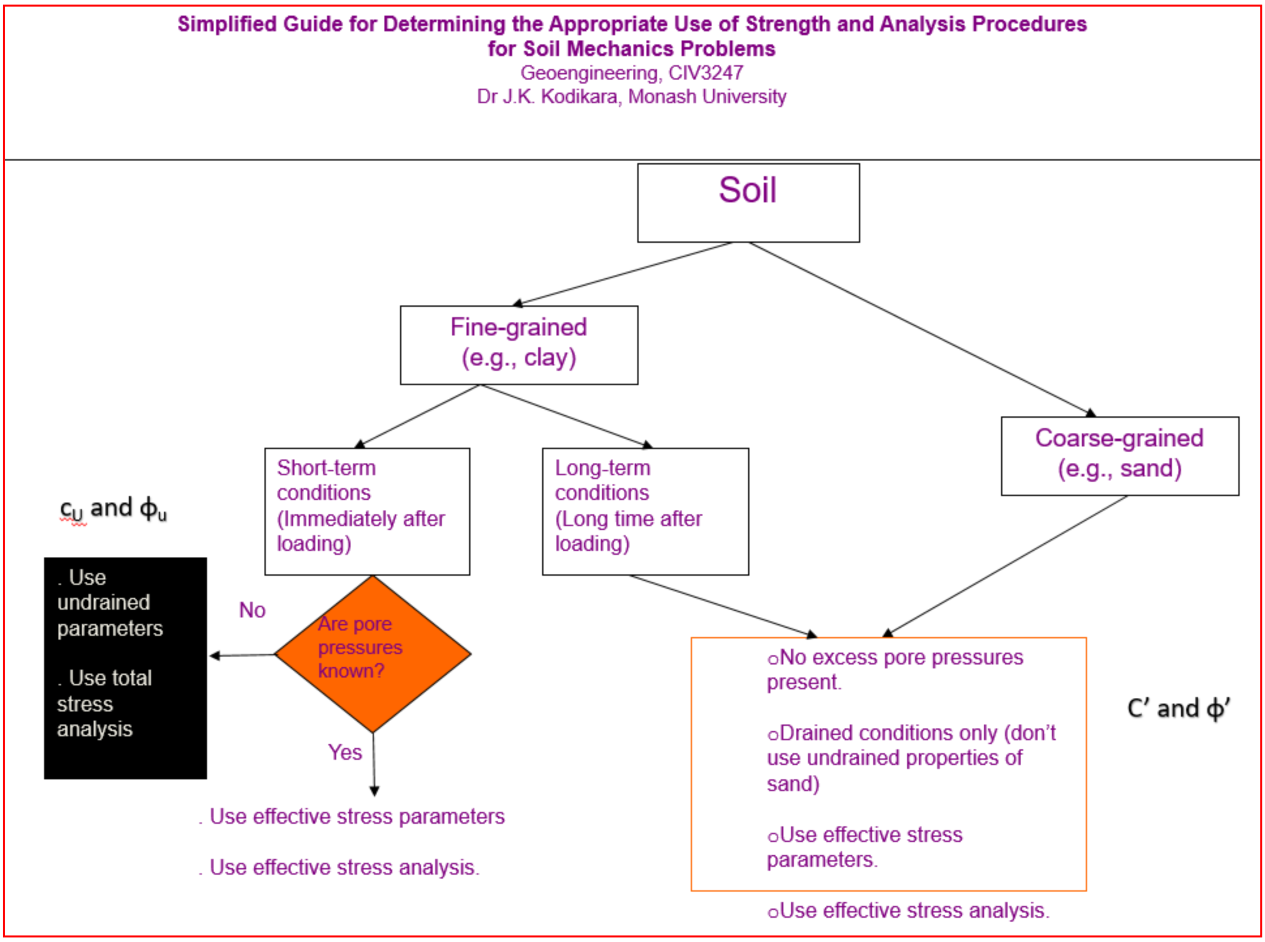


4. Triaxial testing - Drained

4.1 Drained Strength

- Drained strength is an estimate of long term strength bcas drained enables pore pressures to dissipate
  i.e this is long term strength
- Measured in a drained test (D test) OR undrained test w/ pore pressure measurements (CUPP test)

- <u>Guide to determining appropriate use of strength parameters:</u>



4.2 Triaxial testing for Effective Strength

- Two main types:
  Drained (D) i.e Triaxial Consolidation Drained (CD) test 'D test'
  Consolidation undrained w/ pore pressure measurement (CUPP test) (CU test)
- Both tests involve 3 stages/phases: (Saturation – Consolidation - Shearing)

1) **Saturation**

- Place sample in cell
- Apply cell pressure and back pressure just less than cell pressure and leave
  - Test level of saturation using "B test" where $\Delta u = B^*\Delta\sigma$
    For S = 100% (fully saturated), B = 1, $\Delta u = \Delta\sigma$, $\Delta\sigma' = 0$
- If B > 0.95 then go to consolidation stage

Note: Samples when tested must be Saturated

- Unsaturated materials behave differently due to air compressing on loading changing friction angles & effective stresses observed

2) **Consolidation**

- Open back pressure
- ↑ σ3 & u_{bp} to wanted effective stress i.e for σ3 = 700, u_{bp} = 500 ∴ σ' = 200
- Monitor change in volume from back pressure pump
  Volume change indicates the soil is consolidating
  (can apply 1D consolidation methods to approx. time till consolidation completes)
- Allow sufficient time for consolidation to complete
- Specimens volume will change as water is expelled
- Once consolidation is complete we move to phase 3

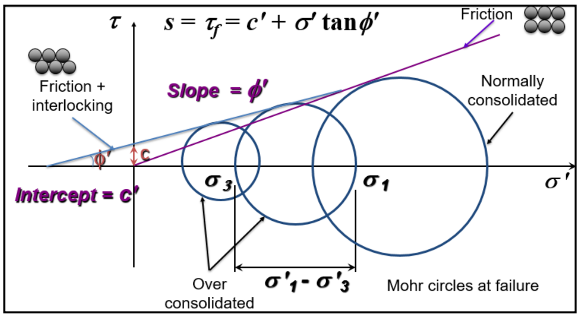
Note: Need consolidation stage bcas:
We need to test soil at known specific effective stresses & use to understand stress paths of soil

3) **Shearing**

D test	**CUPP test**
- Back pressure value is left open (this is where D test differs from CUPP) - Keep cell pressure (σ3) & back pressure (u_{bp}) constant - Load specimen at slow displacement rate to allow for pore pressure equalisation in back pressure line (loading rate depends on soil state & type of soil) - As load ↑ soil skeleton takes load as pore pressure set cannot change ∴ during shearing phase water may move in/out of specimen to keep pore pressures constant - Soil will eventually shear, bulge or fail - Drained friction angle (ϕ') can be obtained from one test - Extra stages or tests are not required to find drained friction angle - Determine $\sigma_1' - \sigma_3'$ directly from axial load - Determine E' and v' (from ε_v and ε_a)	- Close back pressure value & keep cell pressure (σ3) constant - Load specimen at slow displacement to allow for pore pressure equalisation (loading rate depends on soil state & type of soil) - As load ↑ so will u, as both soil skeleton and pore pressure takes the load - Soil will eventually shear, bulge or fail - If 3 stages are required, stop test before shearing failure occurs Repeat phase 2 and 3 at higher confining pressure two more times (Repeat phase 3 until consolidation has completed) - Determine $\sigma_1 - \sigma_3$, u then σ_1', σ_3' using pore pressure measurements - Determine E_u, v_u = 0.5 Since volume does not change $\varepsilon_v = \varepsilon_1 + 2\varepsilon_3 = 0$ $\quad \varepsilon_3 = -\frac{1}{2}\varepsilon_1$ $\quad \nu = -\frac{\varepsilon_3}{\varepsilon_1} = \frac{1}{2}$ Parameters are ϕ', c', E' & Skemptons A & B

Note: Volumetric response of soil in a triaxial is measured by amount of water flow/volume i.e $\Delta V = \Delta V_{water}$

- Mohr-Coulomb failure envelope

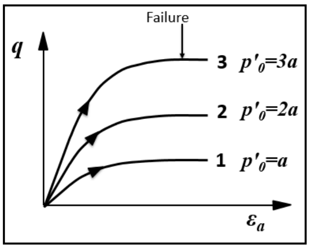


- Ensure we calculate c' & ϕ' from effective stress (σ1' & σ3') not total stress (σ1 & σ3)
  i.e in undrained test (CUPP) we get total stress
- For friction line:
  Don't have to worry about friction & interlocking of particles which is due to sample being Overconsolidated (particles close together; when shearing they move over each other)

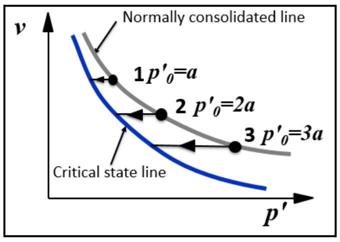
Overconsolidation makes our circles larger due to interlocking

5.6 Undrained Test (Δv=0)

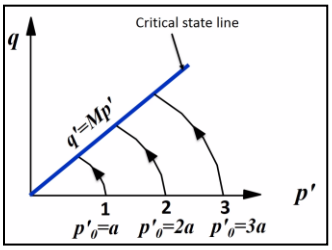
- Deviator Stress (q) vs. axial strain (ε_a)
  - 3 specimen of same soil
  - Sheared in undrained conditions
  - All specimen are normally consolidated
  - Each specimen consolidated to different initial mean stress (p')
  - Higher initial mean stress (p') the higher the failure stress
    (i.e the higher the deviator stress will be at failure)



- Specific Volume (v) vs. effective mean stress (p')
  - 3 specimen of same soil
  - Sheared in undrained conditions
    ∴ specific volume (v) remains constant throughout shearing (bcas drainage valves = closed, **no Δvolume)**
  - Same results are then plotted on q' v p' plot

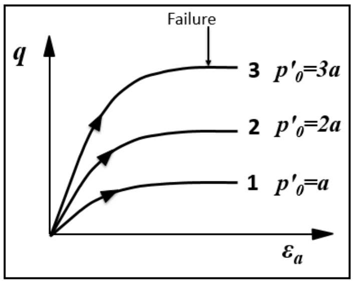


- Deviator stress (q') vs. mean stress (p')
  - Shows stress path soil takes under axial loading
  - Mean stress (p') ↓ as pore pressure (Δu) ↑ in shearing
  - All soils failures on CSL

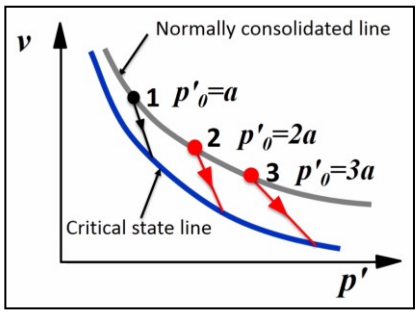


5.7 Drained test

- Deviator Stress (q) vs. axial strain (ε_a)
  - 3 specimen of same soil sheared under drained conditions
  - All specimen are normally consolidated, & consolidated to different initial mean stress (p')
  - Higher initial mean stress (p') the higher the failure stress
    (i.e the higher the deviator stress will be at failure)
  - This graph is similar to undrained case, however if soil is tested at same initial mean stress (p'), the deviator stress will be higher in drained triaxial test

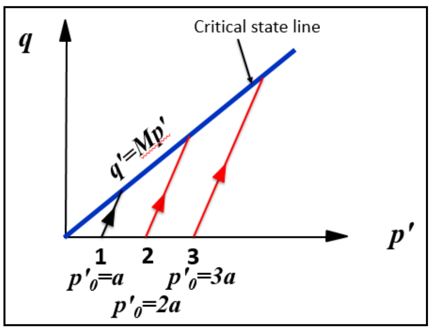


- Specific Volume (v) vs. effective mean stress (p')
  - Pore pressure (u) remains constant throughout test
  - Sheared in **drained conditions**,
    ∴ specific volume (v) ↓ as mean stress (p') ↑ (bcas drainage values are open we have Δvolume)
  - volume change takes place as drainage valves are open ∴ specific volume will ↓throughout shearing as specimen consolidate & ↑ in strength
  - Mean effective stress (p') moves from NCL towards CSL ∴ p' ↑ during shearing



- Deviator stress (q') vs. mean stress (p')
  - Shows stress path soil takes under axial loading
  - Mean stress ↑ as pore pressure remains constant during shearing
  - Gradient of drained stress path = $3q' : p'$ (3:1)
  - All soils failure close to a single curve CSL
  - CSL line:

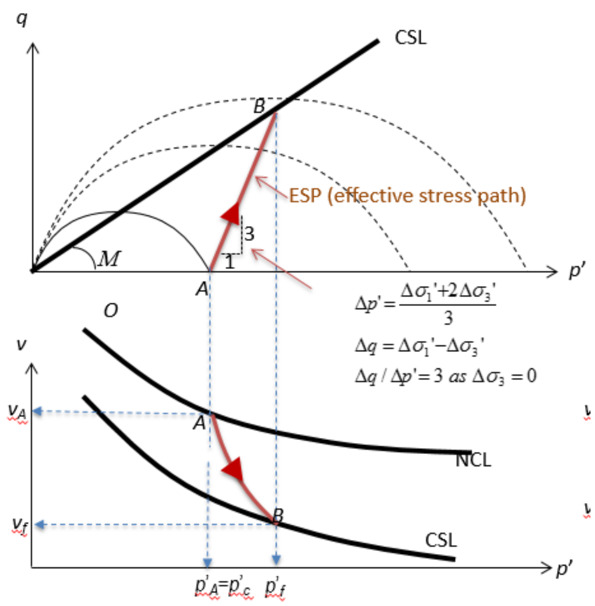
$$q' = Mp'$$
$$q' = 3(p'_f - p'_o)$$



5.11 Stress Paths to Failure

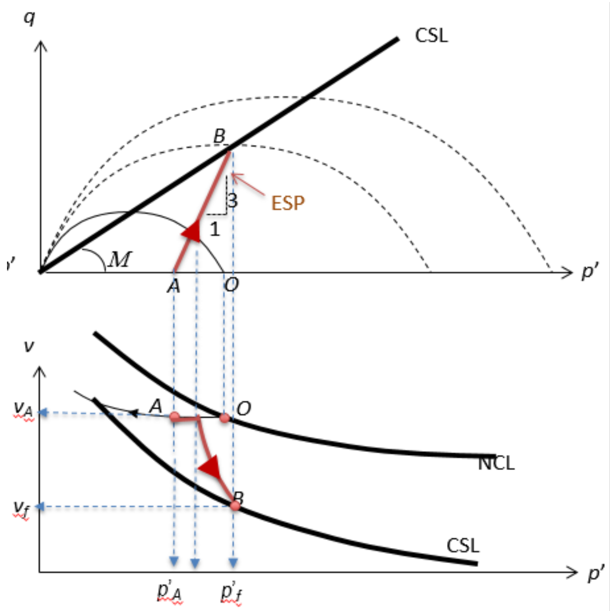
NC & Lightly OC drained axial compression test

1) Normally consolidated (NC) drained test (OCR=1)



1) NC drained test:
- Yield surface & stress path start from A
- Stress path is 3:1
In v-p' space
- Sample fails at B

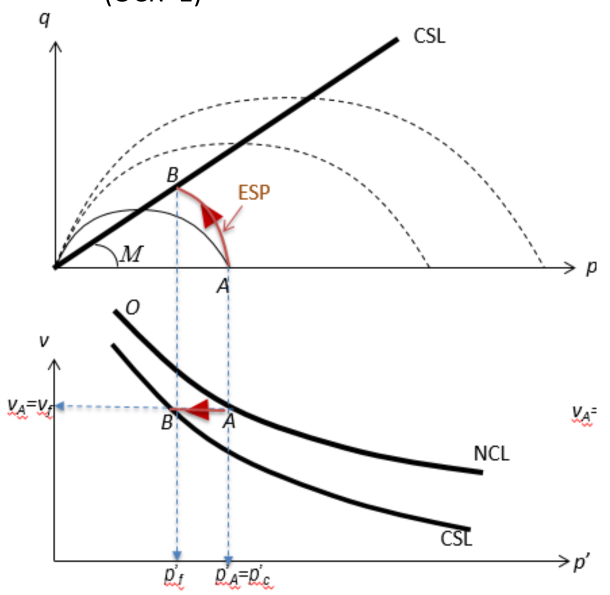
2) Lightly overconsolidated (OC) drained test (OCR≤2)



2) OC drained test:
- OCR ≤ 2 so astropical loading (no shear)
- Before yield surface soil behaviours elastically so volume doesn't change until it reaches the yield surface (see v-p' space)

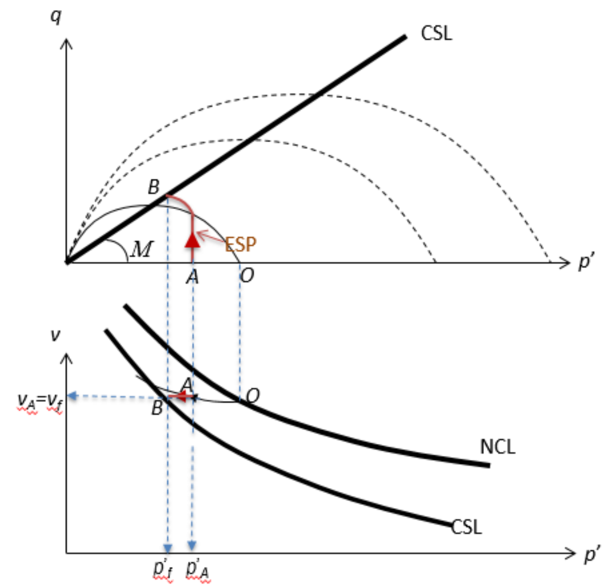
NC & Lightly OC undrained axial compression test

3) Normally consolidated (NC) undrained test (OCR=1)



3) NC undrained test
- effective stress path in undrained (ΔV=0, see v-p' plot) so for soil to reach CSL the p' ↓ to CSL

4) Lightly overconsolidated (OC) undrained test (OCR≤2)



4) OC drained test
- start from O unload it to A, volume is unchanged
- p' is unchanged in elastic region (below yield surface) then ESP curves towards the CSL

7. Rock Slope Stability

7.1 Introduction

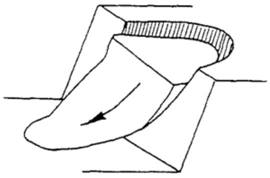
- Soil slopes are reasonably modelled assuming failure occurs on a circular slip surface
- Failure of rock slopes are controlled not by the strength of the intact rock, but by the orientation, position strength, etc. of the many discontinuities that are contained in the rock mass
  ∴ there are several forms of failure that must be considered

Note: Rock mass strength is the strength of mass of rock including the rock material & discontinuities.
Intact rock strength is the strength of the rock material without discontinuities.

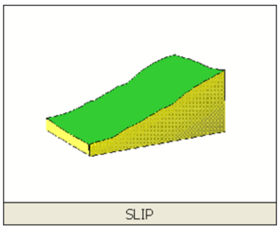
7.2 Rock Slope Stability Analysis

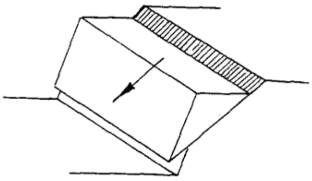
- Depending on the nature of the rock mass itself, the height/steepness of the slope, & ground water conditions, a no. of widely differing potential modes of failure may be identified:
  i.e plane sliding, wedge sliding, circular failure, toppling (or any combination below) each requires different analysis
- Behaviour of a rock slope is dominated by the **presence of discontinuities** (joints, faults, bedding, foliation etc)

Failure Modes

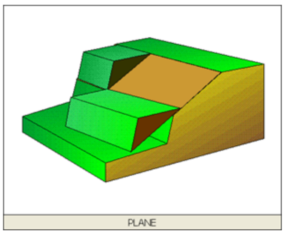


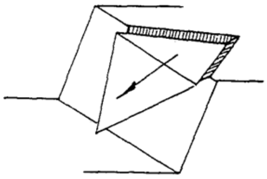
a. Circular failure in overburden soil, waste rock or heavily fractured rock with no identifiable structural pattern.



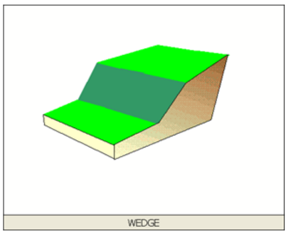


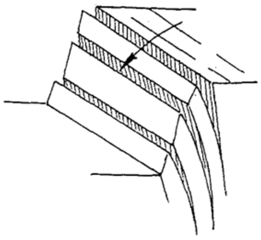
b. Plane failure in rock with highly ordered structure such as slate.



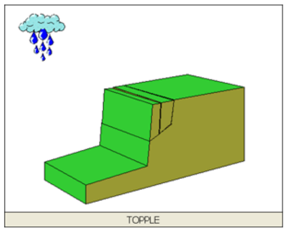


c. Wedge failure on two intersecting discontinuities.





d. Toppling failure in hard rock which can form columnar structure separated by steeply dipping discontinuities.



Circular failure = due to lots of discontinues
Plane failure = joints in similar direction (i.e. slate)
Wedge failure = two intersecting discontinuities
Toppling failure = rocks form a column structure & topple over each other

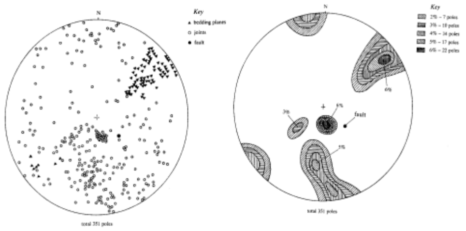
- Why are some steep/high slopes stable while some low/flat slopes unstable?
  Due to unfavourable orientations of controlling discontinuities in the slope, & material properties existing on those discontinuities i.e discontinuities dominate failure

- Trigger Mechanisms
  - Water pressures in joints reducing normal stress > FRICTION CAPACITY
    Water in clay (or other infill material) reduces friction between joints
  - Reduction in negative pore pressure - less tensile total stress capacity.
  - Undercutting - natural / excavation
  - Shocks - earthquake, blasting

- Remedial Measures
  - Realign slope strike to more favourable joint orientation
  - Flatten slope (often restricted by land able to be acquired)
  - Reinforcement - rock anchors to stable material
  - Control of joint water pressure w/ drainage

7.3 Kinematic Analysis

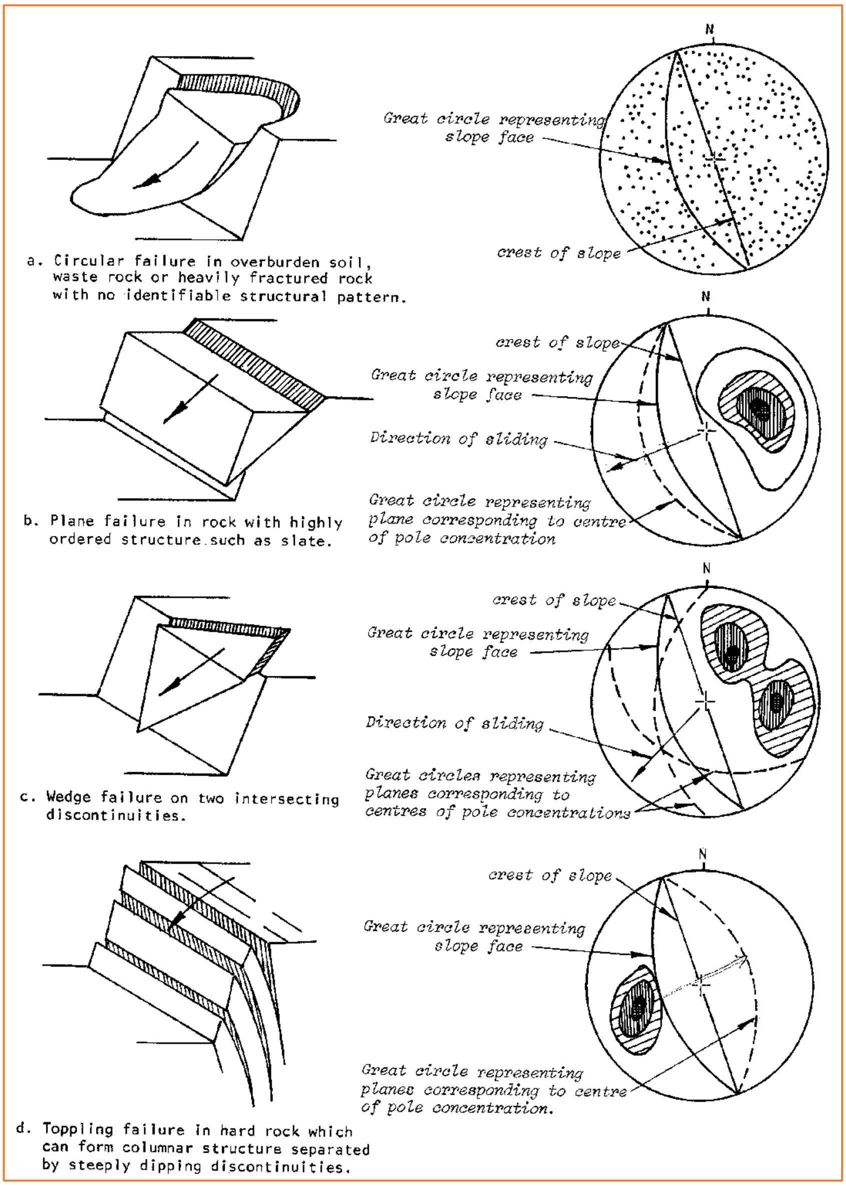
- Used to simplify discontinuity data:
    - Identify critical discontinuities/sets (plot joints, beddings, faults etc. as Dip Direction/Dip)
    - Likely failure modes
    - Identify slopes that are likely to give problems
- Involves no spatial information - **deals only with angles**



- Assumes that discontinuity must daylight joint face to fail
- Must be followed by more careful analysis
    - i.e cant assume slope **will** fail, this is just a general approach

➢ **Statistical Analysis of Joint Planes**

- Plot discontinuity data based on joint poles
    - i.e plot poles and observe their concentration
- Analyse concentration of poles to predict failure mode (i.e. planar, wedge etc.):



Note: Circular slip analysis shows a random scatter if poles w/ no identifiable structural pattern and is ∴ difficult to determine if circular slip will occur. Rock mass properties are more important than material properties in fractured rock i.e rock slopes will generally fail based on the discontinuities and their orientations.

9. Retaining Walls

9.1 Introduction

- Simplest way to hold two areas of soil (or rock) with differing ground levels in a stable state is to join them with a slope of appropriate angle (this is wasteful of valuable surface area
- Vertical boundary between two soils is ∴ required i.e some type of earth retaining structure or retaining wall is necessary
- Design of retaining walls remains one of the least satisfactory aspects of modern Geoengineering, partly because of the ill-defined nature of the forces actually acting on such structures
- Earth retaining structures provide slope support for weak soils, high/steep slopes, displacement control & erosion control

▪ Retaining structures failure mechanisms

- Retaining walls commonly fail by:

    Sliding on base
    Overturning about toe
    Bearing capacity failure or Overall slope failure
    Excessive deformation and settlement
    Structural failure
    Loss of toe resistance (sheet piles)

➢ To ↓ the likelihood of failure:

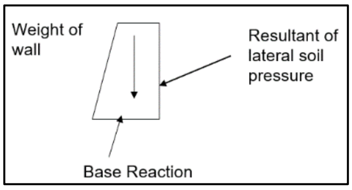
1) Provide anchor blocks (Soils withstand much higher passive pressures than active, ∴ use anchors to transition soil into passive conditions)
2) Install shear key to ↑ base resistance
3) ↑ height of passive soil (∴ ↑ passive pressure)

▪ Retaining Wall Types

- Gravity = utilise the weight of a coherent mass
    Typically upto 3m
    Large costs and require space ∴ not very practical or economic
    Utilize geotextiles and coarse grained aggregates to prevent pore pressure building up)
- Cantilever = use of an extended lever arm to resist the toppling, giving a more slender structure
    Uses less materials, efficient for taller structures bcas lever arm allows force to resist overturning
- Sheet piled = generally temporary supports, thin sheets
    Used for cuts in excavations, hammered in & interlock between each sheet)
    Required to be embedded to a depth below excavation (allows stabilizing soil pressures)
- Soldier Pile = transfer lateral forces to deeper soil
    Large RC piles with thinner connecting walls
- Contiguous and Secant Pile = similar to soldier but closely spaced
    Contiguous pile wall = bore piles then fill w/ RC (used for stiff sandy clays where there's less chance of material falling between piles)
    Secant = drill primary piles (not reinforced) then drill second pile and use RC

➢ Gravity Retaining Wall

-Design as coherent wall body to resist 3 forces:



If base reaction > weight of wall otherwise wall fails by bearing capacity
If base friction cannot sustain resultant soil pressure, the wall fails by sliding
If weight of wall about the toe < moment due to soil pressure, wall may overturn

9.2 Factor of Safety

FOS for sliding = $\frac{F_S}{F_D} = \frac{Stabilizing\ Force}{Disturbing\ Force} = \frac{shear\ strength\ (\tau)*b}{sum\ of\ earth\ pressures}$

FOS for overturning = $\frac{M_R}{M_O} = \frac{Restoring\ Moment}{Overturning\ Moment} = \frac{Gravity\ wall\ provides\ restoring\ effect}{Soil\ pressures\ cause\ overturning}$ where M = force*distance