SHEAR STRENGTH OF SOILS

(PART-2)

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OUTLINE

- **Shear Strength Characteristics and Measurement of COHESIVE Soils**
  1. Triaxial Consolidated-Drained (CD) Test Behavior
  2. Triaxial Consolidated-Undrained (CU) Test Behavior
  3. Triaxial Unconsolidated-Undrained (UU) Test Behavior

- **Total and Effective Stress Approach**

STRENGTH CHARACTERISTICS AND MEASUREMENT

- Shear strength is measured both in the **Laboratory** and in the **Field**. Laboratory tests are made on representative soil samples and must be done in a way that simulates the conditions that will exist in the field as closely as possible, in particular the drainage and stress conditions.
- The shear strength of **Granular Soils** (clean sands and gravels) can generally be made on disturbed samples that are reconstituted in the laboratory to field densities.
- However, disturbance significantly affects the physical properties of **Cohesive Soils** (plastic silts and clays, organic soils) even if the field density is maintained, laboratory test on cohesive soils must therefore be made on undisturbed samples if the strength of a natural soil deposit is to be determined.
- The strength of proposed **Compacted Earth Embankments** is often required, and for such cases the laboratory samples must be prepared to duplicate the density, water content, and compaction method of the field soil.

Coarse- and Fine-Grained Soils

Table 5.2 Unified Soil Classification System (Based on Material Passing 76.2-mm Sieve)

Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $C_c < 1$ or $C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	GM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $C_c < 1$ or $C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	SM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 5.3)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 5.3)[e]	ML
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$; see Figure 5.3; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 5.3)	CH
			PI plots below "A" line (Figure 5.3)	MH
		Organic	$\frac{\text{Liquid limit—oven dried}}{\text{Liquid limit—not dried}} < 0.75$; see Figure 5.3; OH zone	OH
Highly organic soils	Primarily organic matter, dark in color, and organic odor			Pt

No 200 Sieve
(Grain-size 0.075mm)

[a]Gravels with 5 to 12% fine require dual symbols: GW-GM, GW-GC, GP-GM, GP-GC.

[b]Sands with 5 to 12% fines require dual symbols: SW-SM, SW-SC, SP-SM, SP-SC.

[c]$C_u = \frac{D_{60}}{D_{10}}$; $C_c = \frac{(D_{30})^2}{D_{60} \times D_{10}}$

[d]If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 5.3, use dual symbol GC-GM or SC-SM.

[e]If $4 \leq PI \leq 7$ and plots in the hatched area in Figure 5.3, use dual symbol CL-ML.

SHEAR STRENGTH OF COHESIVE SOILS

- Soils containing significant amounts of plastic fines (more than 50% by weight silt & clay) are called **Cohesive Soils**. Because of the low permeability and high compressibility of clays and plastic silts, the drainage conditions, rate of loading, and the stress history of the soil greatly affect the shear strength properties. A soil that has never been subjected to an effective pressure greater than the existing effective pressure is considered a **Normally Consolidated (NC)** Soil. If the cohesive soil has subjected to effective pressure greater than the existing pressure is considered as **Overly Consolidated (OC).**
- In cohesive soils, clay particles are surrounded by an envelope of water and particle interaction takes place through this water. Net attraction force between particles, for given clay-water system, decreases with increasing inter-particle distance, i.e. void ratio. Therefore, upon application or removal of a load, a change in pore pressure, Δu is effected. In response to Δu, pore water tends to flow into (if Δu is negative) or out (if Δu is positive) of the soil. This drainage process results in changing the inter-particle distances (void ratio or density) and therefore, the inter-particle forces (shear strength). **Therefore, the shear strength of cohesive soil changes (increases or decreases) as a result of construction activity (loading a foundation, building an embankment or making an excavation, etc.)** and it is important to take the drainage condition into account.

SHEAR STRENGTH OF COHESIVE SOILS

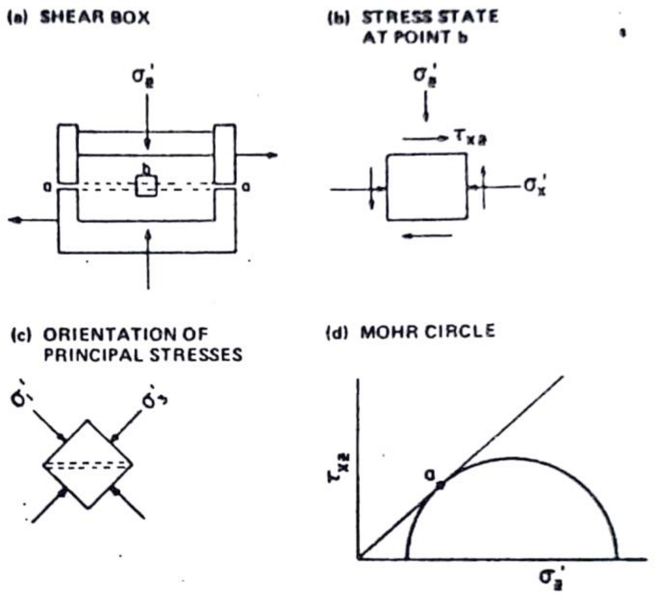
- In considering the drainage conditions two limiting cases are usually assumed:

1. **Undrained** corresponding to the **End of Construction** when full Δu develops and
2. **Drained** sufficiently long time after construction (**Long-Term**) when Δu is fully dissipated (because of low permeability of cohesive soils, this may take considerable amount of time).

- Undrained or Drained or partially drained situations are common in cohesive soils. This is a most important difference between granular soils and cohesive soils.
- Another important distinction between **NC** or lightly **OC** and **Heavily OC** clays is based on the kind of **Pore- Pressure Change (Δu)** developed in these soils during shear.

LABORATORY MEASUREMENT OF SHEAR STRENGTH

Two most commonly used laboratory shear test methods are **Direct Shear** and **the** **Triaxial Compression Test**.



These tests are able to establish the shear strength response. These tests allow variation of normal stress and measurement or control of pore pressures. The simple methods in the laboratory are designed to determine the shear strength of a sample in a particular condition, such as the water content or void ratio of the soil in situ. These methods are most often used to determine the undrained strength (s_u) of saturated cohesive soils. Simple tests include the **Unconfined Compression Test** which is a special case of the triaxial compression test in which no confining pressure is used.

TRIAXIAL COMPRESSION TESTS FOR COHESIVE SOILS:

1. **Triaxial Consolidated-Drained (CD) Test**
2. **Triaxial Consolidated-Undrained (CU) Test**
3. **Triaxial Unconsolidated-Undrained (UU) Test**

Three general stage on triaxial test are

- Sampling Stage
- Isotropic Loading Stage (saturation and confining pressure)
- Shearing Stage

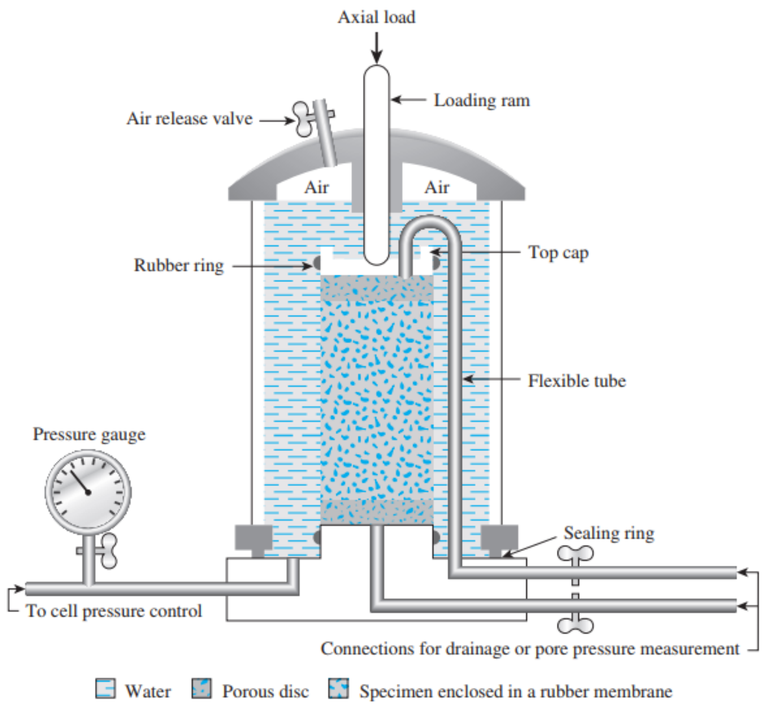
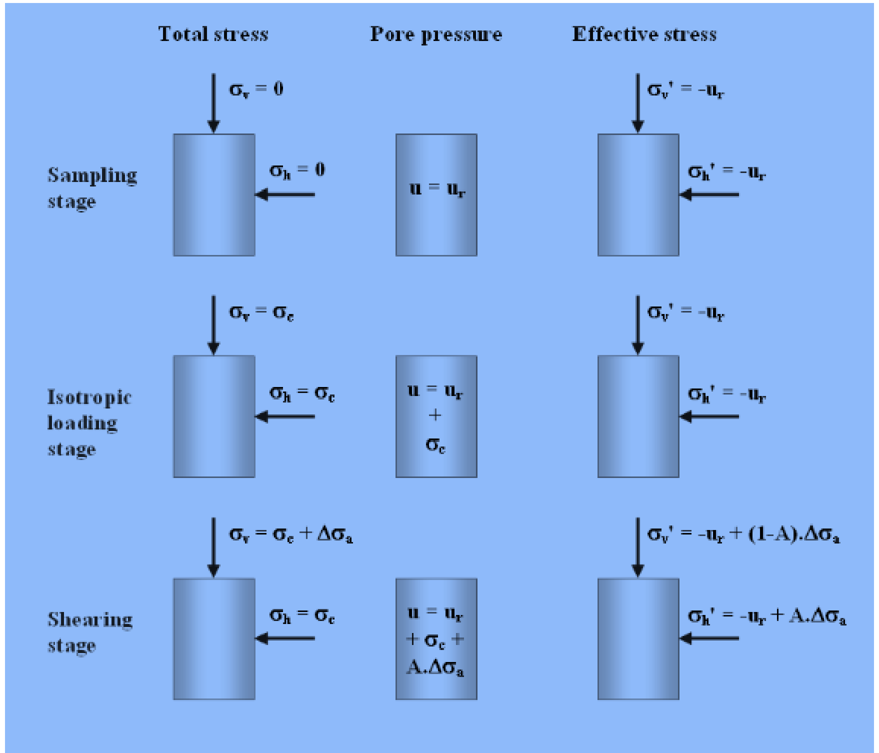


Figure 12.20 Diagram of triaxial test equipment (*After Bishop and Bjerrum, 1960. With permission from ASCE.*)



CONSOLIDATED-DRAINED (CD) TEST BEHAVIOR ON COHESIVE SOILS

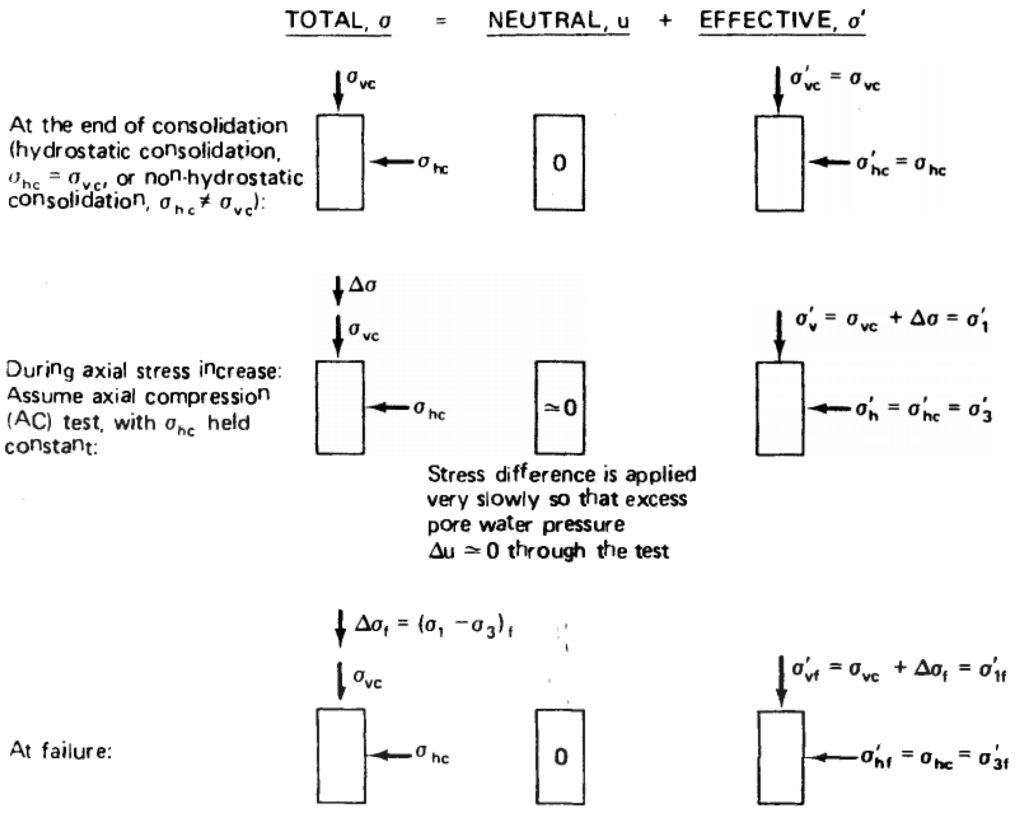


Fig. 11.23 Stress conditions in the consolidated-drained (CD) axial compression triaxial test.

- Briefly, the CD test procedure is to consolidated the test specimen under some state of stress appropriate to the field or design situation. The consolidation stress can either be isotropic (equal in all direction) or anisotropic. Another way of looking at this second case is that a stress difference or a shear stress is applied to soil.
- When consolidation is over, the "C" part of CD test is complete.
- During the "D" part, the drainage valve remain open and the stress difference is applied very slowly so that essentially no excess pore pressure develops during the test.
- Note that at the CD test, the pore water pressure is essentially zero. This means that the total stresses in the drained test are always equal to the effective stresses.

CONSOLIDATED-DRAINED (CD) TEST BEHAVIOR ON COHESIVE SOILS

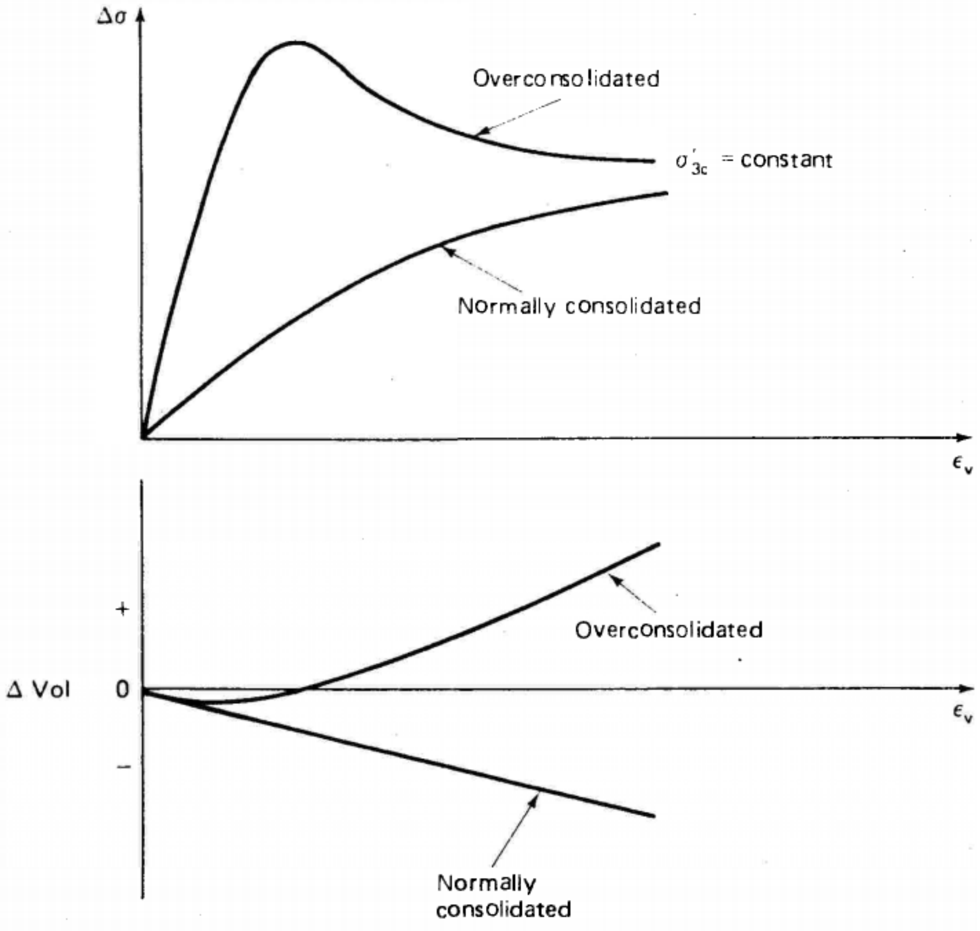


Fig. 11.24 Typical stress-strain and volume change versus strain curves for CD axial compression tests at the same effective confining stress.

- Typical stress-strain curves and volume change versus strain curves for a remolded or compacted clay are shown beside.
- Even though the two samples were tested at the same confining pressure, the over-consolidated specimen has a greater strength than the normally consolidated clay. Note also that it has a higher modulus and the maximum $\Delta\sigma$, which for the triaxial test is equal to $(\sigma_1-\sigma_3)_f$ occurs at a much lower strain than for the normally consolidated specimen.
- The over-consolidated clay **expands** during shear while the normally consolidated clay **compresses** during shear.

CONSOLIDATED-DRAINED (CD) TEST BEHAVIOR ON COHESIVE SOILS

- The Mohr failure envelopes for CD tests of typical clay soils are shown in Figure below.

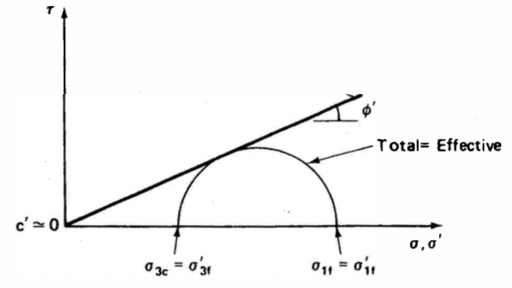


Fig. 11.25 Mohr failure envelope for a normally consolidated clay in drained shear.

- Even though only one Mohr circle is shown. The results of three or more CD tests on identical specimens at different consolidation pressures would ordinarily be required to plot the complete Mohr failure envelope. If the consolidation stress range is large or the specimens do not have exactly the same initial water content, density, and stress history, then the three failure circles will not exactly define a straight line, and an average best-fit line by eye is drawn.
- It is usually assumed that the c' parameter for normally consolidated or non-cemented clays is essentially zero for all practical purposes.

CONSOLIDATED-DRAINED (CD) TEST BEHAVIOR ON COHESIVE SOILS

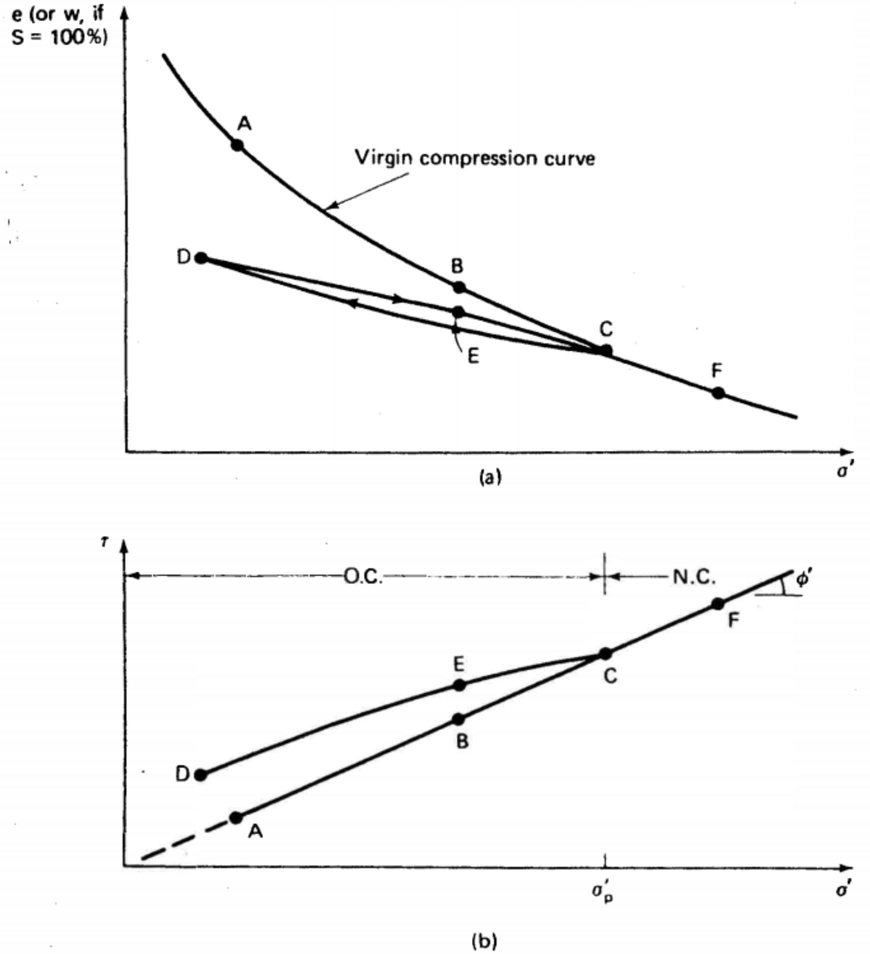


Fig. 11.26 (a) Compression curve; (b) Mohr failure envelope (DEC) for an overconsolidated clay.

- For over-consolidated clays the c' parameter is greater than zero. The over-consolidated portion of the strength envelope (ABCF). This portion (DEC) of the Mohr failure envelope is called the **pre-consolidation hump**.
- The explanation for this behavior is shown in the e versus σ' curve.
- The effects of the rebounding and reconsolidation have been in effect erased by the increased loading to point F. Once the soil has been loaded well past the pre-consolidation pressure σ'_p, it no longer "remembers" its stress history.

TYPICAL VALUES OF DRAINED STRENGTH PARAMETERS

- Average values of ϕ' for undisturbed clays range from around 20° for NC highly plastic clay up to 30° or more for silt and sandy clays. The value ϕ' for compacted clays is typically 25° or 30° and occasionally as high as 35°.
- The value of c' for NC non-cemented clays is very small and can be neglected for practical work. If the soil is over-consolidated, then ϕ' would less, and the c' intercept greater than for the NC clay.
- For stability analyses, the Mohr-Coulomb effective stress parameters ϕ' and c' are determined over the range of effective normal stresses likely to be encountered in the field.
- It has been observed (for example, Kenney, 1959) that there is not much difference between ϕ' determined on undisturbed or remolded samples at the same water content. Apparently, the development of the maximum value of ϕ' requires so much strain that the soil structure is broken down and almost remolded in the region of the failure plane.

TYPICAL VALUES OF DRAINED STRENGTH PARAMETERS

- Empirical correlations between ϕ' and the plasticity index for normally consolidated clays are shown in figure below.

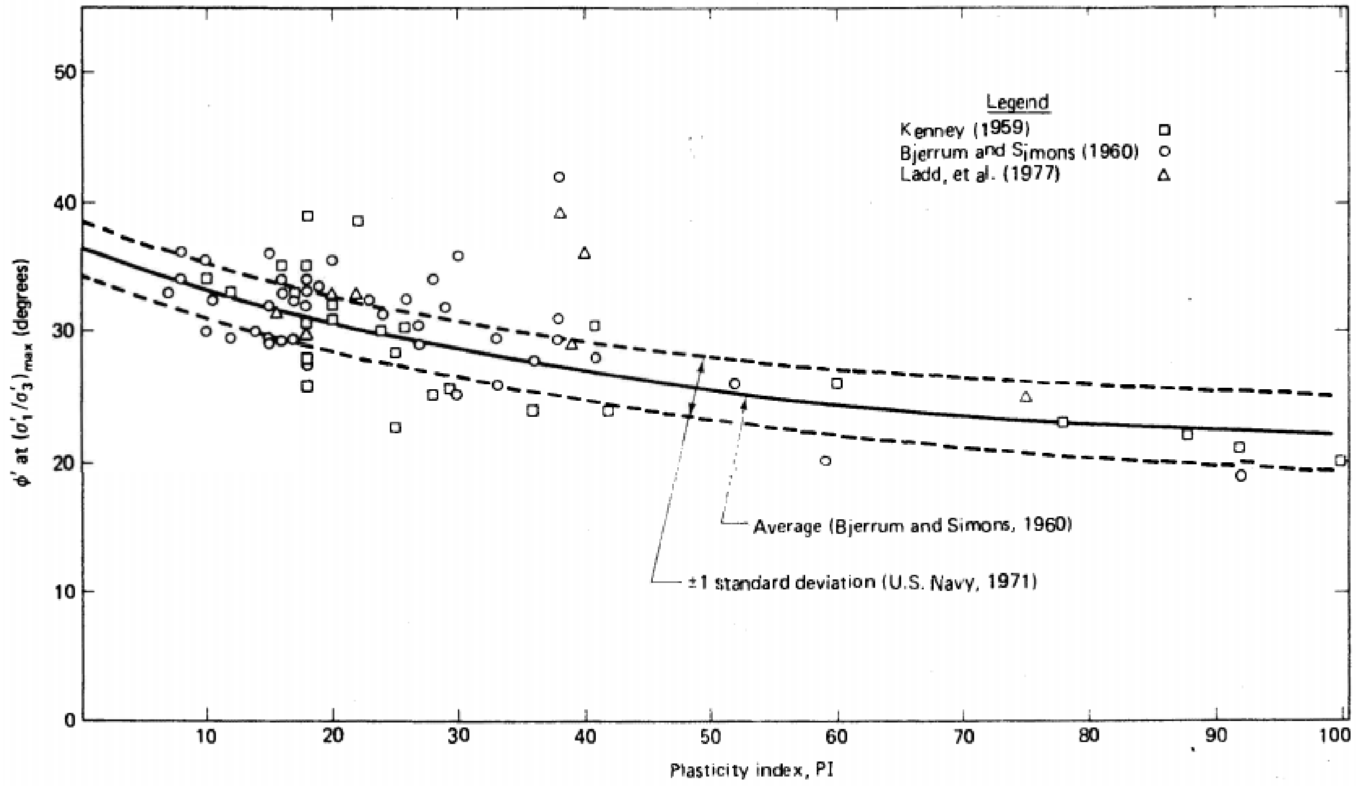


Fig. 11.27 Empirical correlation between ϕ' and PI from triaxial compression tests on normally consolidated undisturbed clays (after U.S. Navy, 1971, and Ladd, et al., 1977).

USE OF CD STRENGTH IN ENGINEERING PRACTICE

- The limiting drainage conditions modeled in the triaxial test refer to real field situations. **Examples of CD condition is long-term stability of steady seepage case for embankment dams. Other example is critical conditions of long-term stability of excavations or slopes in both soft and stiff clays**. How you actually go about making these analyses for stability can be found in textbooks on foundation and embankment dam engineering.
- You should be aware that, actually it is not easy to actually conduct a CD test on a clay in the laboratory. To ensure that no pore pressure is really induced in the specimen during shear for materials with very low permeability, the rate of loading must be very slow. The time required to fail the specimen ranges from a day to several weeks (Bishop and Henel, 1962). Such a long time leads to practical problems in the laboratory such as leakage of valves, seals, and the membrane that surrounds the sample. Consequently, since it is possible to measure the induced pore-pressures in a consolidated-undrained (CU) test and thereby calculate the effective stresses in the specimen, CU tests are more practical for obtaining the effective stress strength parameters. Therefore CD triaxial tests are not very popular in most soils laboratory.

USE OF CD STRENGTH IN ENGINEERING PRACTICE

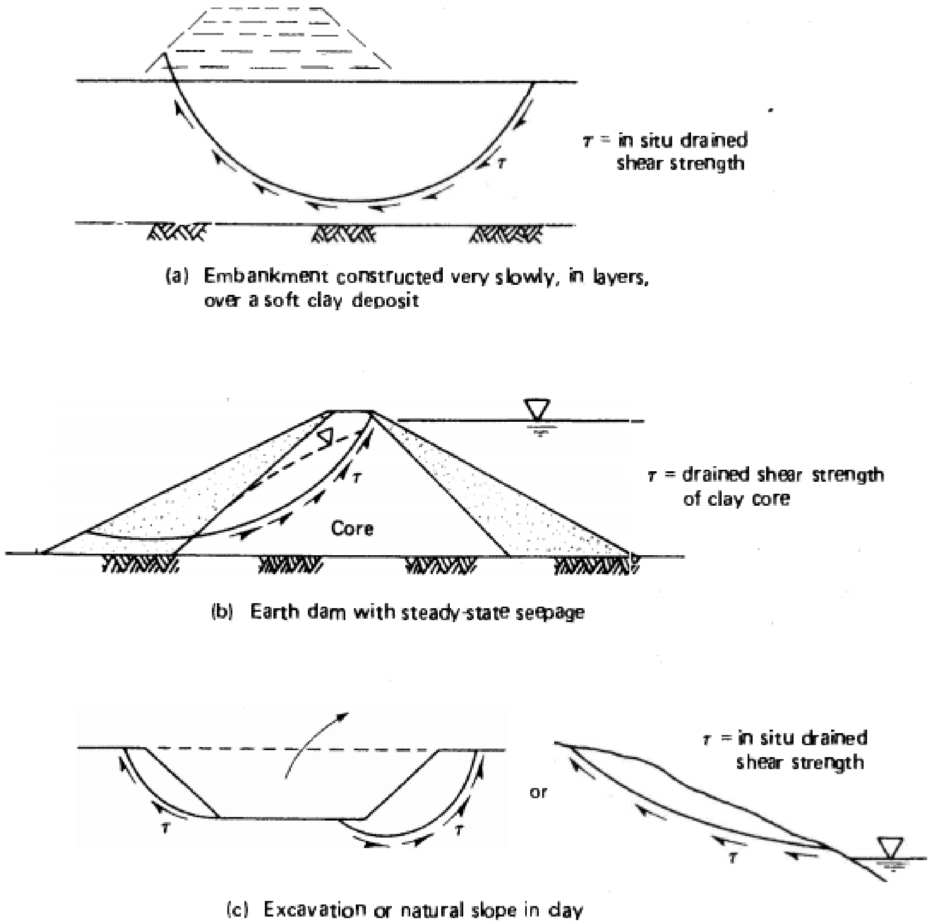
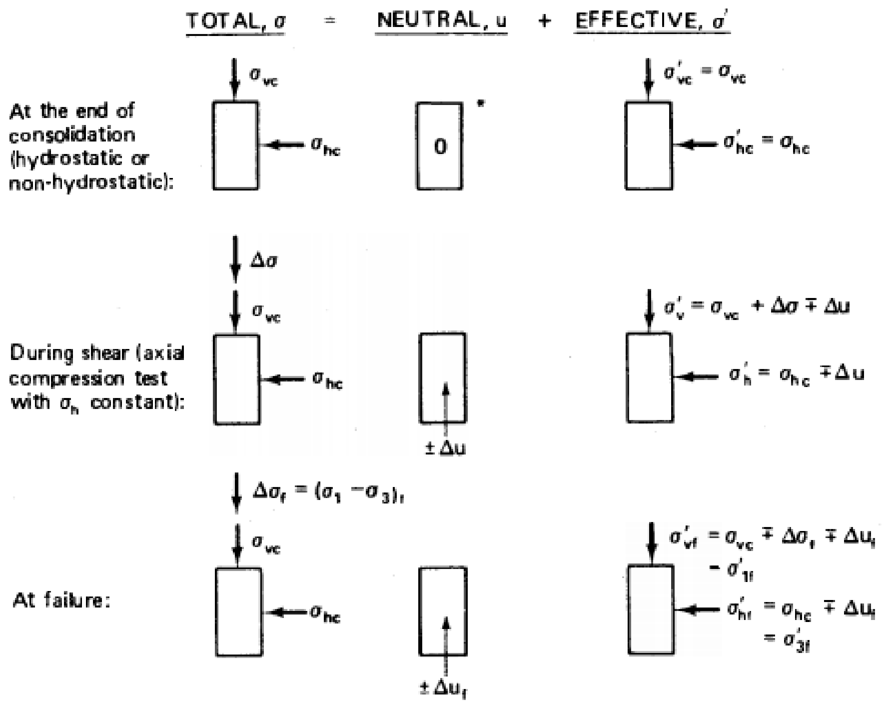


Fig. 11.28 Some examples of CD analyses for clays (after Ladd, 1971 b)

CONSOLIDATED-UNDRAINED (CU) TEST BEHAVIOR

- The test specimen is first consolidated (drainage valves open, obviously) under the desired consolidation stresses. After consolidation is complete, the drainage valves are closed, and the specimen is loaded to failure in undrained shear. The pore water pressure developed during shear are measured, and both the total and effective stresses may be calculated during shear and at failure.
- Like the CD test, the axial stress can be increased incrementally or at a constant rate of strain.
- Note that the excess-pore-water pressure (**Δu)** developed during shear can either be positive (that is, increase) or negative (that is, decrease). This happens because the sample tries to either contract or expand during shear. Remember, we are not allowing any volume change (an undrained test) and therefore no water can flow in or out of the specimen during shear. Because volume changes are prevented, the tendency towards volume change induces a pressure in the pore water.
- If the specimen *tend* to contract or consolidate during shear, then the induced (excess) pore water pressure (**Δu**) is *positive*. It wants to contract and squeeze water out of the pores, but cannot; thus the induced pore water pressure is positive. Commonly, positive excess pore pressures occur in normally consolidated clays. However it also depends on its effective consolidation pressure (σ_{hc}') . Both OCR and σ_{hc}' would determine the **Δu.**
- If the specimen tends to expand or swell during shear, the induced pore water pressure is negative. It wants to expand and draw water into the pores, but cannot. Thus the pore water pressure decreases and may even go negative. Negative pore pressures occur in overconsolidated clays.

CONSOLIDATED-UNDRAINED (CU) TEST BEHAVIOR



*In practice, to ensure 100% saturation, which is necessary for good measurements of the pore water pressure, a *back pressure* is applied to the pore water. To keep the effective consolidation stresses constant, the total stresses during consolidation are accordingly increased by an amount exactly equal to the applied back pressure, which is the same as raising atmospheric pressure by a constant amount—the effective stresses on the clay do not change.

Example: Initial conditions with back pressure:

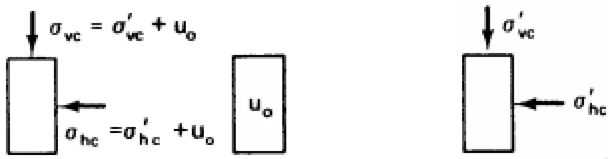


Fig. 11.29 Conditions in specimen during a consolidated-undrained axial compression (CU) test

- Note that in actual testing the initial pore water pressure typically is greater than zero. In order to ensure full saturation, a back pressure u_o is usually applied to the test specimen.
- When a back pressure is applied to a sample, the cell pressure must also be increased by an amount equal to the back pressure so that the effective consolidation stresses will remain the same.
- In practice, this may not be exactly true, but the advantage of having S=100% for accurate measurement of induced pore water pressures far outweighs any disadvantages of the use of back pressure.

CONSOLIDATED-UNDRAINED (CU) TEST BEHAVIOR

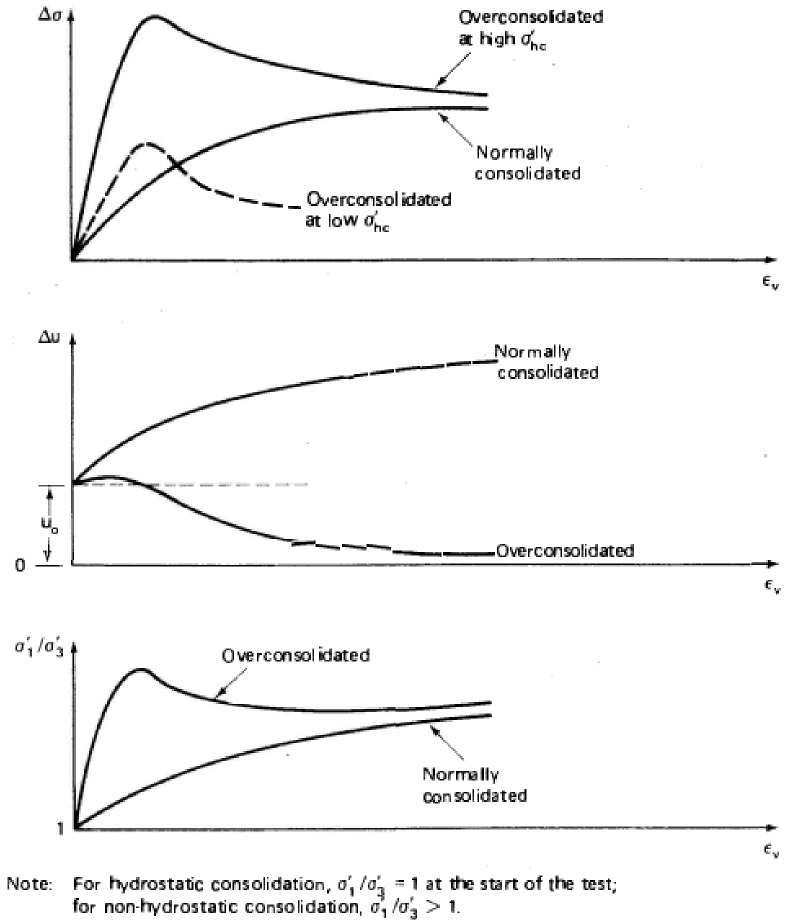


Fig. 11.30 Typical σ-ϵ, Δu, and σ'_1/σ'_3 curves for normally and overconsolidated clays in undrained shear (CU test).

- The normally consolidated specimen develops positive pore pressure. In the over-consolidated specimen, after a slight initial increase, the pore pressure goes "negative" – in this case, negative with respect to the back pressure u_o.
- Another quantity that is useful for analyzing test results is the principal effective stress ratio σ_1'/σ_3'. Note how this ratio peaks early just like the stress difference curve for the over-consolidated clay.
- They are simply a way of normalizing the stress behavior with respect to the effective minor principal stress during the test.
- Sometimes, the maximum of this ratio is used as a criterion of failure.

CONSOLIDATED-UNDRAINED (CU) TEST BEHAVIOR

- Since we can get both the total and effective stress circles at failure for a CU test when we measure the induced pore water pressures, it is possible to define the Mohr failure envelopes in terms of both total and effective stresses from a series of triaxial tests conducted over a range of stresses.

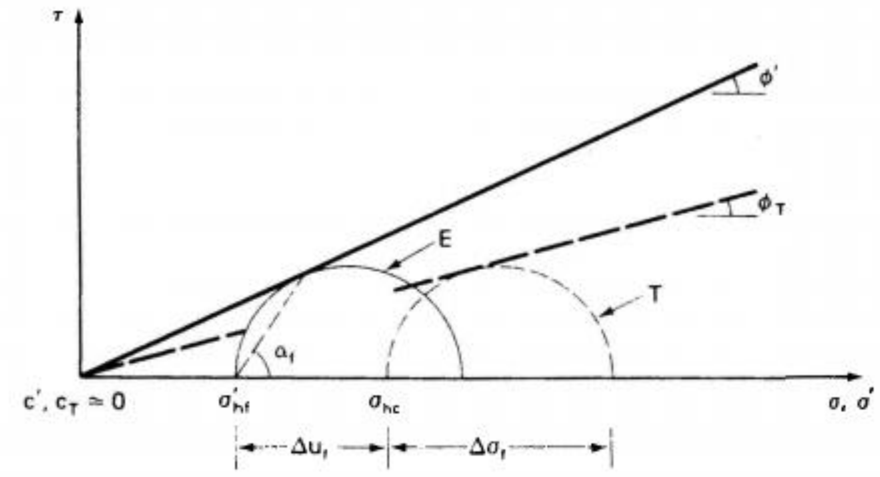


Fig. 11.31 Mohr circles at failure and Mohr failure envelopes for total (*T*) and effective (*E*) stresses for a normally consolidated clay.

- Since we can get both the total and effective stress circles at failure for a CU test when we measure the induced pore water pressures, it is possible to define the Mohr failure envelopes in terms of both total and effective stresses from a series of triaxial tests conducted over a range of stresses for normally consolidated clay.
- Note that the effective stress circle is displaced to the left, towards the origin, for the normally consolidate case, because the specimens develop positive pore pressure during shear and $\sigma' = \sigma - \Delta u$.
- Note that ϕ_T is less than ϕ' and often it is about one-half of ϕ'.

CONSOLIDATED-UNDRAINED (CU) TEST BEHAVIOR

- Things are different if the clay is overconsolidated. Since an overconsolidated specimen tends to expand during shear, the pore water pressure decreases or even goes negative. The effective stresses are greater than the total stresses, and the effective stress circle at failure is shifted to the right of the total stress circle.

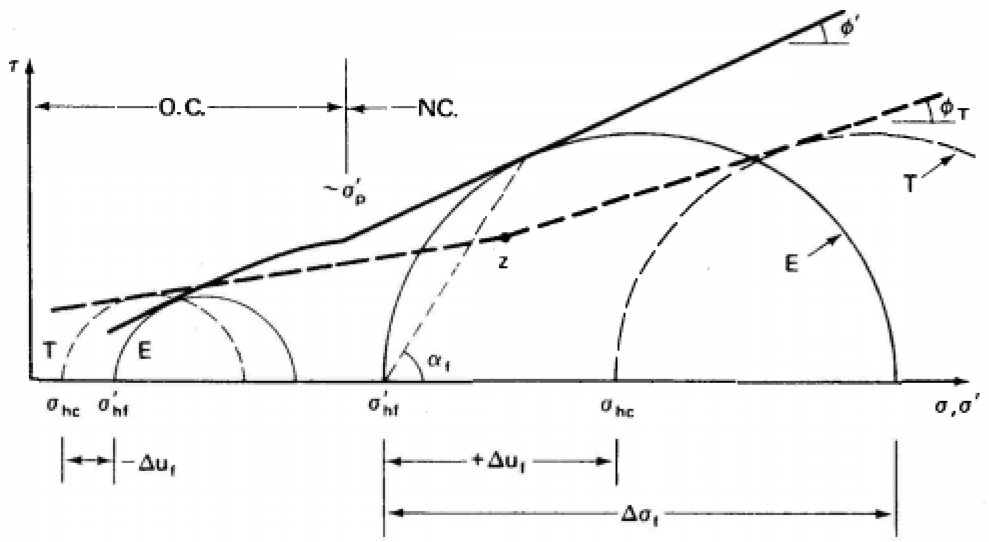


Fig. 11.33 Mohr failure envelopes over a range of stresses spanning the preconsolidation stress σ'_p

- The Mohr failure envelopes over a wide range of stresses spanning the preconsolidation stress. Thus some of the specimens are overconsolidated and others are normally consolidated.
- Note that the "break" in the total stress envelope (point z) occurs roughly about twice the σ_p' for typical clays.

CONSOLIDATED-UNDRAINED (CU) TEST BEHAVIOR

- Stress paths for the two test (NC and OC clay) are shown below.

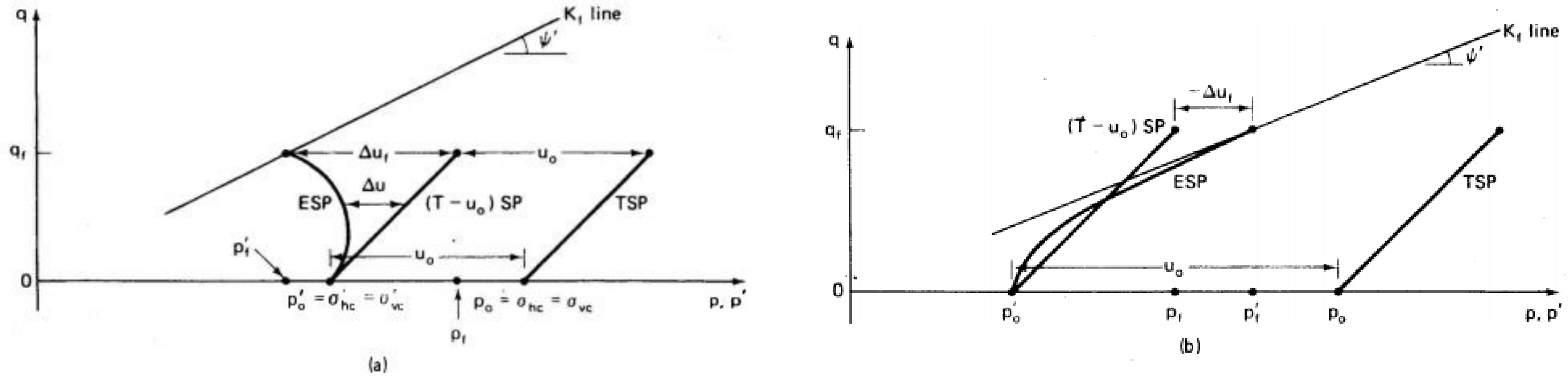
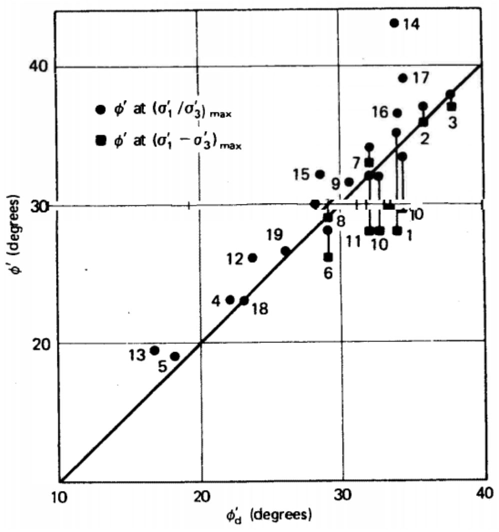


Fig. 11.34 Stress paths for the hydrostatically consolidated axial compression tests on (a) normally consolidated clays; (b) overconsolidated clays.

DIFFERENCES OF DRAINED STRENGTH PARAMETERS OBTAINED BETWEEN TRIAXIAL CU AND CD

- In our discussion so far, we have tacitly assumed that the Mohr-Coulomb strength parameters in terms of effective stresses determined by CU tests with pore pressure measurements would be the same as those determined by CD tests.
- This assumptions is not strictly correct. The problem is complicated by alternative definitions of failure.
- Depending on how the stress difference and the pore water pressures actually develop with strain, these two definitions may indicate different c's and ϕ's.
- Note that ϕ' obtained from CU tests is from 0° to 3° greater than ϕ'_d obtained from CD tests.



	Clay	State	Reference
1	Cornwall	U	Kenney
2	Cornwall	R	"
3	Bersimis	R	"
4	Weald	R	Henkel
5	London	R	"
6	Oslo	U	N.G.I.
7	Fredrikstad	U	"
8	Lodalen	U	"
9	Fornebu	U	"
10	Drammen	U	"
11	Okernbråten	U	"
12	Seven Sisters	U	Casagrande and Rivard
13	North Ridge	U	" "
14	Organic	U	Casagrande
15	Boston blue	U	"
16	Weymouth	U	Hirschfeld
17	New Haven	U	"
18	Haslemere	R	Skempton and Bishop
19	Wiener Tegel	R	Hvorslev

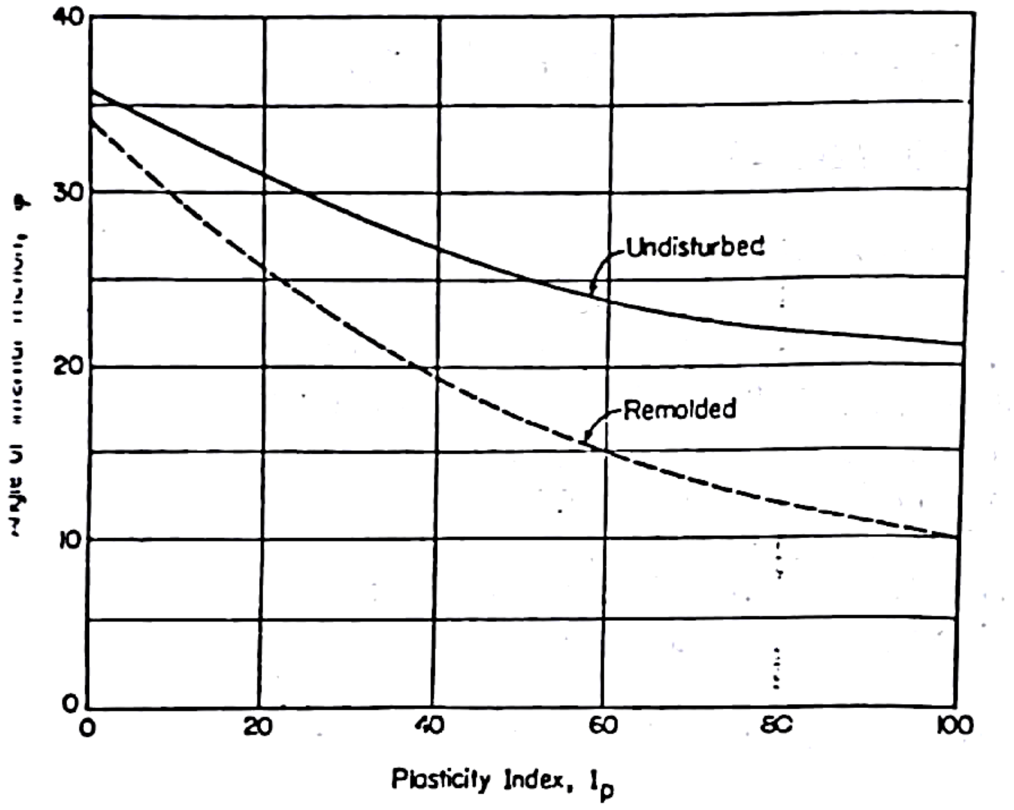
U = undisturbed
R = remolded

Fig. 11.36 Relationship between ϕ'_d determined from CD tests and ϕ' determined from CU tests with pore pressures measured. Two failure criteria are indicated for the undrained tests (after Bjerrum and Simons, 1960).

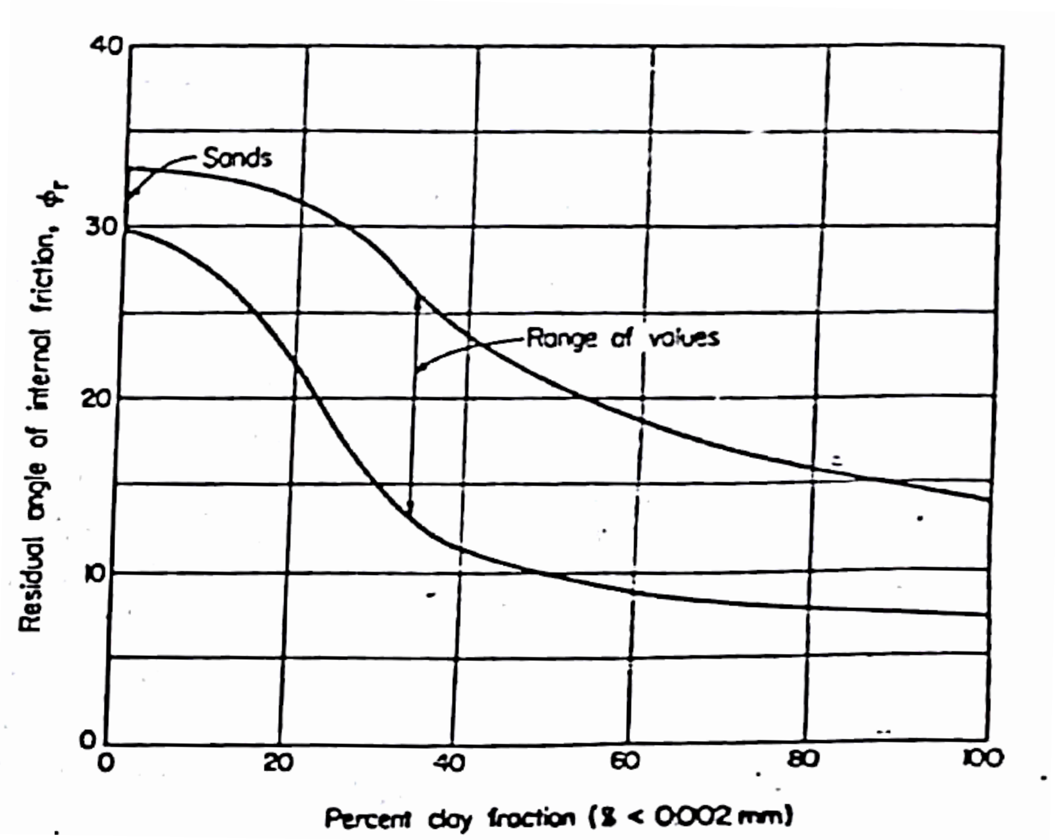
TYPICAL VALUES OF THE UNDRAINED STRENGTH PARAMETERS DETERMINED BY TRIAXIAL CU

- For normally consolidated clays, ϕ_T seems to be about half of ϕ'; thus values of 10° to 15° or more are typical. The total stress c is very close to zero.
- For over-consolidated and compacted clays, ϕ_T may decrease and c will often be significant.
- When the failure envelope straddles the pre-consolidation stress, proper interpretation of the strength parameters in terms of total stresses is difficult. This is especially true for undisturbed samples which may have some variation in water content and void ratio, even within the same geologic stratum.

CORRELATION BETWEEN Φ' AND $Φ_R$ – PLASTICITY INDEX AND PERCENT CLAY



(a)

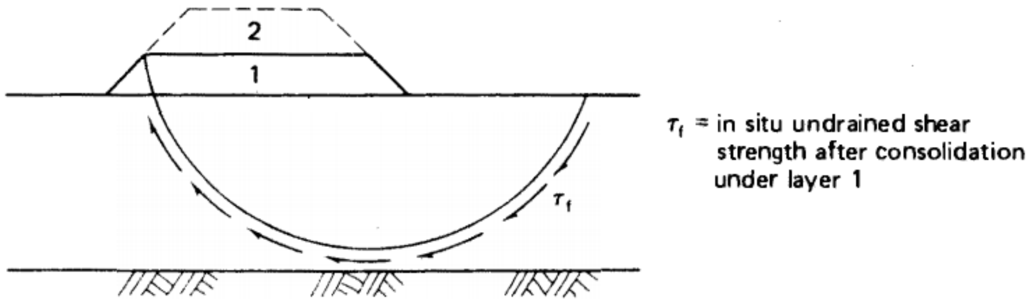


(b)

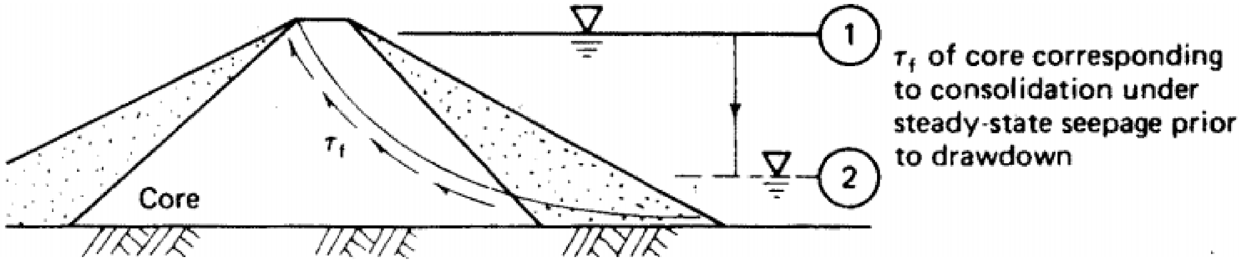
Figure 24. Correlation between: a) ϕ' and Plasticity Index for Normally Consolidated Clays and b) ϕ_r and Percent Clay

USE OF CU STRENGTH IN ENGINEERING PRACTICE

- This test is commonly used to determine the shear strength parameters in terms of both total and effective stresses. CU strength are used for stability problems where the soils have first become fully consolidated and are at equilibrium with the existing stress system. Then, for some reason, additional stresses are applied quickly with no drainage occurring.
- Practical examples include rapid drawdown of embankment dams and the slopes of reservoirs and canals.
- Also in terms of effective stresses, CU test results are applied to the field situations similar as CD tests.



(a) Embankment raised (2) subsequent to consolidation under its original height, (1).



(b) Rapid drawdown behind an earth dam. No drainage of the core. Reservoir level falls from 1 → 2.

UNCONSOLIDATED-UNDRAINED (UU) TEST BEHAVIOR

- In this test, the specimen is placed in the triaxial cell with the drainage valves closed from the beginning. Thus, even when a confining pressure is applied, no consolidation can occur if the sample is 100% saturated.
- The sample is loaded to failure in about 10 to 20 min; usually pore water pressures are not measured in this test. This test is a total stress test and it yields the strength in terms of total stresses.

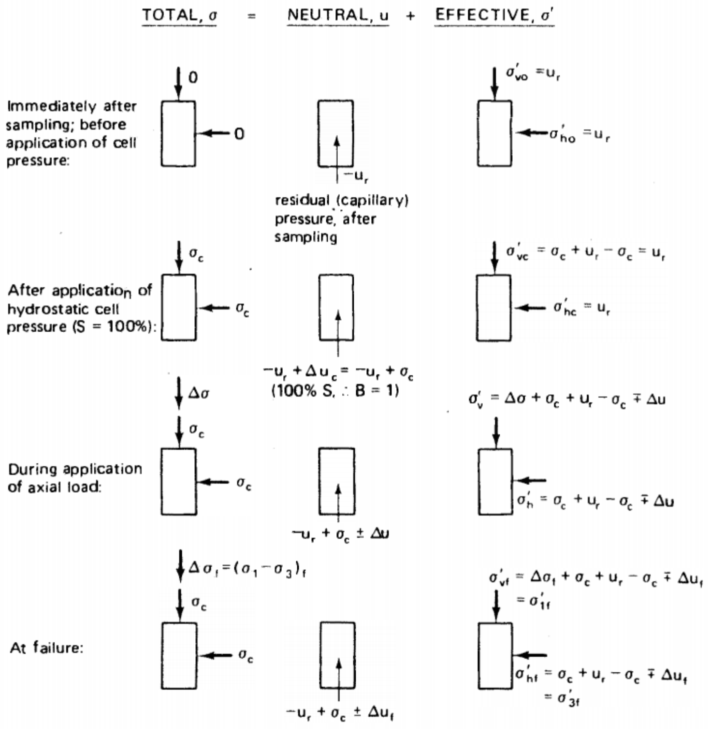


Fig. 11.38 Conditions in the specimen during the unconsolidated-undrained (UU) axial compression test.

UNCONSOLIDATED-UNDRAINED (UU) TEST BEHAVIOR

- The test is quite conventional in that hydrostatic cell pressure is usually applied and the specimen is failed by increasing the axial load, usually at a constant rate of strain.
- Note that initially for undisturbed samples, the pore pressure is negative, and it is called the residual pore pressure – u_r, which results from stress release during sampling.
- Since the effective stresses initially must be greater than zero (otherwise the specimen would simply disintegrate) and the total stresses are zero (atmospheric pressure = zero gage pressure), the pore pressure must be negative.
- When the cell pressure is applied with the drainage valves closed, a positive pore pressure Δu_c is induced in the specimen which is exactly equal to the applied cell pressure σ_c.
- All the increase in hydrostatic stress is carried by the pore water because:

1. The soil is 100% saturated
2. The compressibility of the water and individual soil grains is small compared to the compressibility of the soil structure
3. There is a unique relationship between the effective hydrostatic stress and the void ratio

UNCONSOLIDATED-UNDRAINED (UU) TEST BEHAVIOR

- Typically, stress-strain curves for UU tests are not particularly different from CU or CD stress-strain curves for the same soils. For undisturbed samples, especially the initial portion of the curve (initial tangent modulus), are strongly dependent on the **quality** of the undisturbed samples.
- Also, the sensitivity affects the shape of these curves; highly sensitive clays have sharply peaked stress-strain curves.
- The maximum difference often occurs at very low strains, usually less than 0.5%.

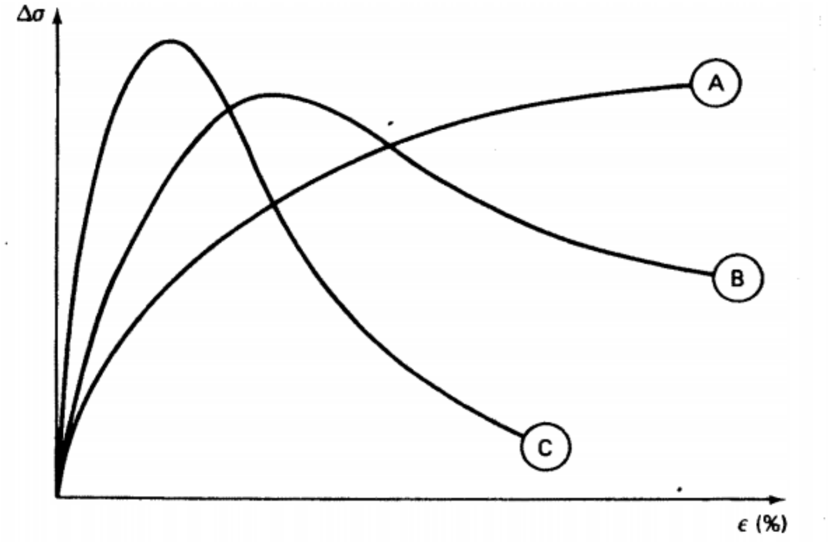


Fig. 11.39 Typical UU stress-strain curves for (*A*) remolded and some compacted clays, (*B*) medium sensitive undisturbed clay, and (*C*) highly sensitive undisturbed clay.

UNCONSOLIDATED-UNDRAINED (UU) TEST BEHAVIOR

- The Mohr failure envelopes for UU tests are shown in Figure below for 100% saturated clays.
- All test specimens for fully saturated clays are presumably at the same water content (and void ratio), and consequently they will have the same shear strength since there is no consolidation allowed. Therefore all Mohr circles at failure will have the same diameter and the Mohr failure envelope will be a horizontal straight line.
- The UU test gives the shear strength in terms of total stresses, and the slope $\phi_T = 0$.

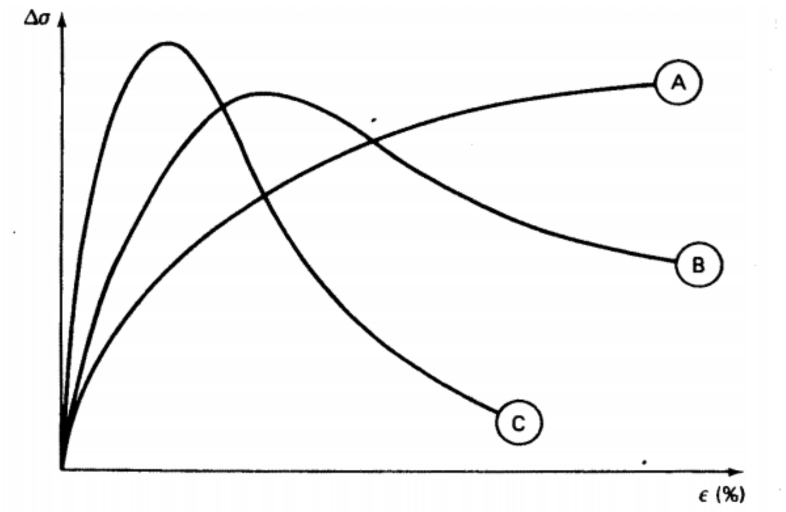


Fig. 11.39 Typical UU stress-strain curves for (*A*) remolded and some compacted clays, (*B*) medium sensitive undisturbed clay, and (*C*) highly sensitive undisturbed clay.

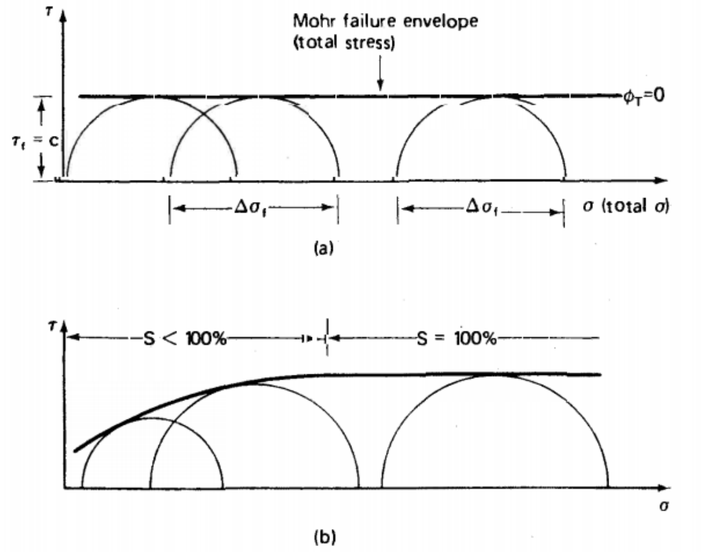


Fig. 11.40 Mohr failure envelopes for UU tests: (a) 100% saturated clay; (b) partially saturated clay.

UNCONSOLIDATED-UNDRAINED (UU) TEST BEHAVIOR

- For partially saturated soils, a series of UU tests will define an initially curved failure envelope until the clay becomes essentially 100% saturated due simply to the cell pressure alone.
- Even though the drainage valves are closed, the confining pressure will compress the air in the voids and decrease the void ratio.
- As the cell pressure is increased, more and more compression occurs and eventually, when sufficient pressure is applied, essentially 100% saturation is achieved. Then, as with the case for initially 100% saturated clays, the Mohr failure envelope becomes horizontal as shown on the right side.

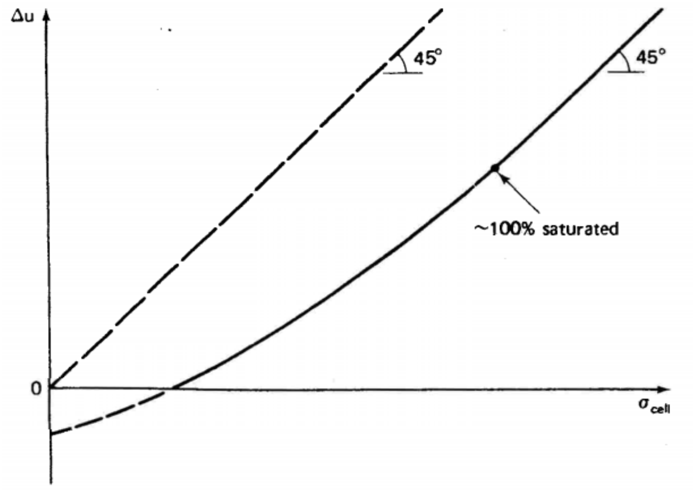
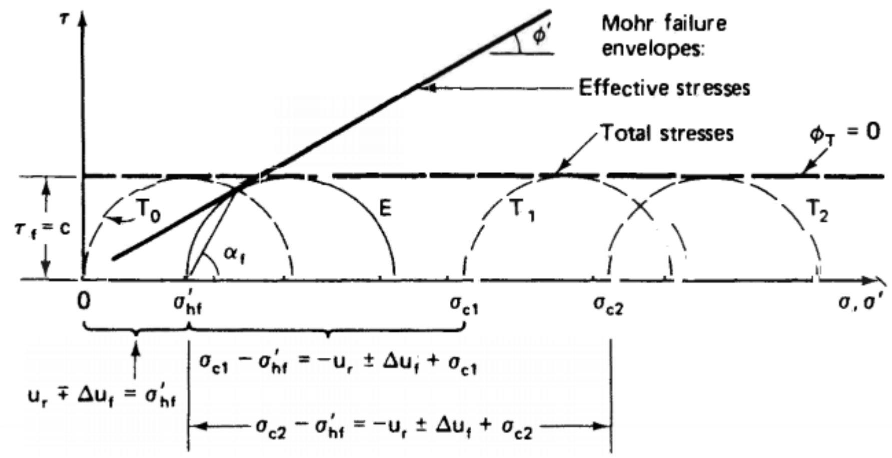


Fig. 11.41 Results obtained from a PH test on a partially saturated compacted clay (after Skempton, 1954, and Hirschfeld, 1963).

- Another way of looking at the compression of partially saturated clays is shown in Figure beside.
- As the cell pressure is increased incrementally, the measured increment of pore pressure increases gradually until at some point for every increment of cell pressure added, an equal increment of pore water pressure is observed.
- At this point, the soil is 100% saturated and the soil (experimental) curves becomes parallel to the 45° line.

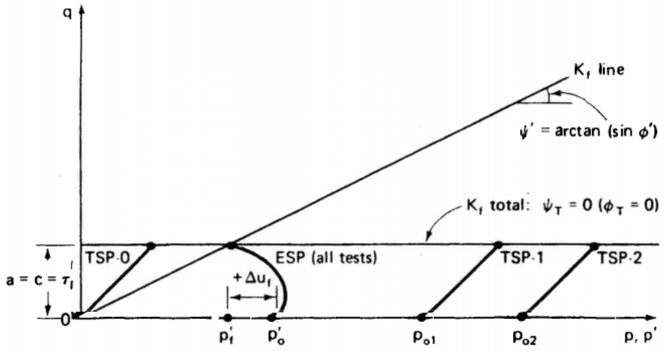
UNCONSOLIDATED-UNDRAINED (UU) TEST BEHAVIOR

- In principle, it is possible to measure the induced pore water pressures in a series of UU tests although it is not commonly done. Since the effective stresses at failure are independent of the total cell pressures applied to the several specimens of a test series, there is only one UU effective stress Mohr circle at failure.
- Stress paths for the UU tests on normally consolidated clay are shown below



Note: σ'_{hf} is the same for all three total stress circles!

Fig. 11.42 UU test results, illustrating the unique effective stress Mohr circle at failure.



	Test	Initial Conditions p_o	Initial Conditions q_o	At Failure p_f	At Failure q_f
Total stresses	0	0	0	$\frac{\Delta\sigma_f}{2}$	$\frac{\Delta\sigma_f}{2}$
	1	σ_{c1}	0	$\frac{\Delta\sigma_f + 2\sigma_{c1}}{2}$	$\frac{\Delta\sigma_f}{2}$
	2	σ_{c2}	0	$\frac{\Delta\sigma_f + 2\sigma_{c2}}{2}$	$\frac{\Delta\sigma_f}{2}$
Effective stresses		p'_o	q_o	p'_f	q_f
	All	u_r	0	$\frac{\Delta\sigma_f + 2u_r - 2\Delta u_f}{2}$	$\frac{\Delta\sigma_f}{2}$

Fig. 11.43 Stress paths for UU tests on a normally consolidated clay. Same tests as in Fig. 11.42.

TYPICAL VALUES OF UU STRENGTHS

- The undriained strength of clays varies widely. Of course, ϕ_T is zero, but the magnitude of Su can vary from almost zero for extremely soft sediments to several MPa for very stiff soils and soft rocks.
- Often, the undrained shear strength at a site is normalized with respect to the vertical effective overburden stress, σ'_{vo} at each sampling point.
- Then the Su/ σ'_{vo} ratios are analyzed and compared with other data. This point is covered in more detail later in this chapter.

USE OF THE UNDRAINED (UU) SHEAR STRENGTH IN ENGINEERING PRACTICE

- Like the CD and CU tests, the undrained or UU Strength is applicable to certain critical design situations in engineering practice.
- These situations are where the engineering loading is assumed to take place so rapidly that there is no time for the induced excess pore water pressure to dissipate or for consolidation to occur during the loading period.
- We also assume that the change in total stress during construction does not affect the in situ undrained shear strength (Ladd, 1971).

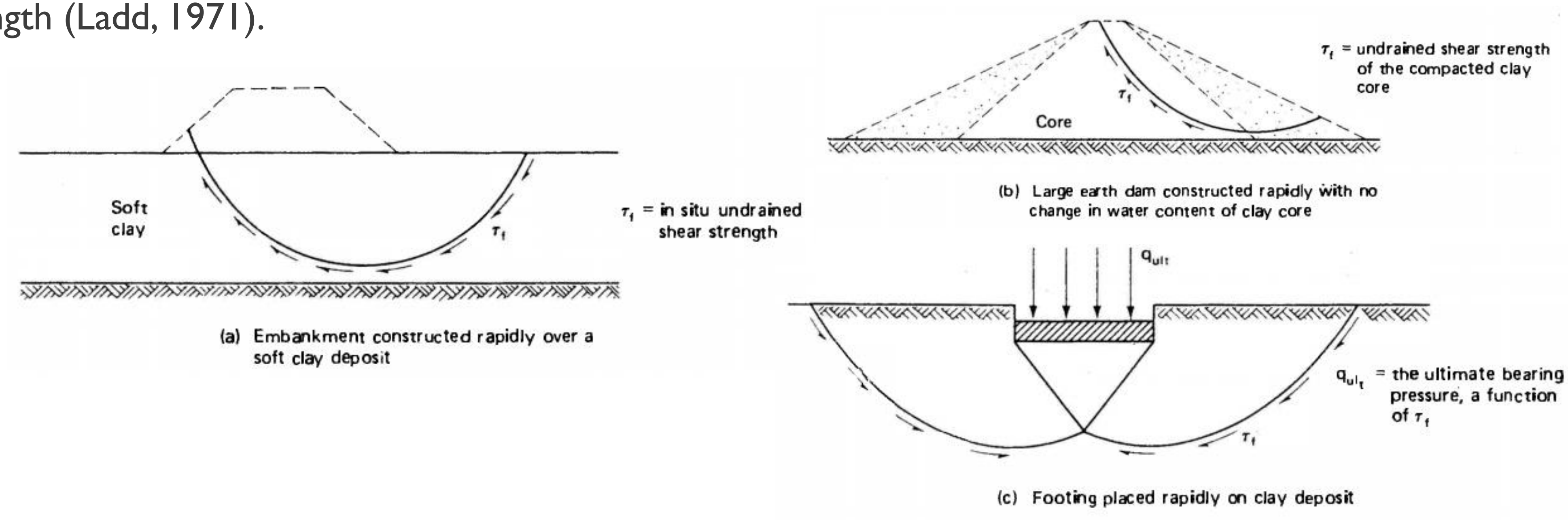


Fig. 11.57 Some examples of UU analyses for clay (after Ladd, 1971b).

USE OF THE UNDRAINED (UU) SHEAR STRENGTH IN ENGINEERING PRACTICE

- For these cases, often the most critical design condition is ***immediately after*** the application of the load (at the end of construction) when the induced pore pressure is the greatest but before consolidation has had time to take place.
- Once consolidation begins, the void ratio and the water content naturally decrease and the shear strength increases. So the embankment or foundation becomes increasingly safer with time.
- One of the more useful ways to express the undrained shear strength is in terms of the S_u/σ_{vo}' ratio for NC clays. In natural deposits of sedimentary clays the undrained shear strength has been found to increase with depth, and thus it is proportional to the increase in effective overburden stress with depth.

TYPICAL VALUES AND EMPIRICAL METHODS FOR SHEAR STRENGTH PARAMETERS OF COHESIVE SOILS

- The undrained shear strength (s_u) of cohesive soils depends, among other factors, on water content, density, soil texture, clay mineralogy, soil microstructure, stress history, etc. and it may vary from as low as 0.25 tons/ft^2 (25 kPa – very soft) to more than 2 tons/ft^2 (200 kPa – hard). Table below gives the empirical relationship between standard penetration resistance (N) and unconfined compressive strength ($q_u = 2s_u$).

Table 1 Empirical relationship between SPT and several soil properties†

N	Consistency	Field identification	γ_{sat}, kN/m^3	q_u, kPa
< 2	Very soft	Easily penetrated several centimeters by fist	16–19	< 25
2–4	Soft	Easily penetrated several centimeters by thumb	16–19	25–50
4–8	Medium	Moderate effort required to penetrate several centimeters with thumb	17–20	50–100
8–16	Stiff	Readily indented by thumb	19–22	100–200
16–32	Very stiff	Readily indented by thumbnail	19–22	200–400
> 32	Hard	Difficult to ident with thumbnail	19–22	> 400

TYPICAL VALUES AND EMPIRICAL METHODS FOR SHEAR STRENGTH PARAMETERS OF COHESIVE SOILS

- The undrained strength (s_u) for normally consolidated clays can be estimated from the relationship.

$$\frac{s_u}{\sigma_o'} = 0.11 + 0.0037 I_p \quad I_p > 10$$

  Where σ_o' = effective overburden pressure and I_p = plasticity index.

- For over-consolidated clays it has been shown that the normalized undrained strength can be estimated from.

$$\frac{\left(\frac{S_u}{\sigma_o'}\right)_{OC}}{\left(\frac{S_u}{\sigma_o'}\right)_{NC}} = OCR^{0.8}$$

  Where OCR is the over-consolidation ratio

- The drained (or effective) cohesion intercept, c', may vary between 0 and 0.2 tons/ft^2 (20 kPa), whereas φ' (drained or effective friction angle) ranges from more than 30° for clays with low plasticity index to less than 15° for clays with higher plasticity index. φ' values for normally consolidated clays are as function of plasticity index Ip.

HEAVILY OVER CONSOLIDATED CLAY

- Most heavily over consolidated clay show stress – strain relations that suggest general strain – softening. If the peak strength is used to describe failure, an effective stress failure envelope as shown by line A’B’C is obtained from a CD test. The failure envelope is approximately a straight line and if extrapolated to the axis of σ’=0, there is a cohesion intercept (c’). Therefore, the failure envelope is given for the over-consolidated range of stresses (σ_3’ < σ_c’) as

$$s = c' + \sigma' \tan \phi'$$

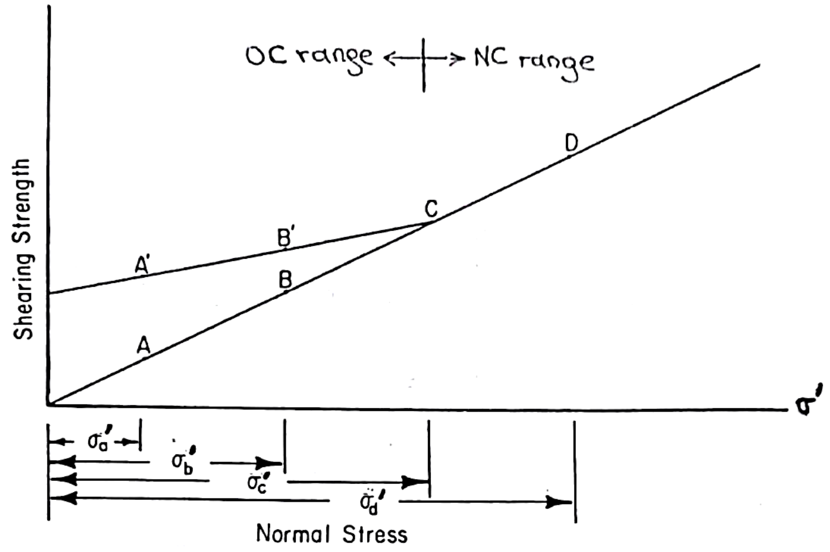


Figure 23. Strength Envelope for an Overconsolidated Clay

ANALYSIS APPROACH

- **Total Stress Approach**

  Short term condition (Undrained)

  (End of construction)

- **Effective Stress Approach**

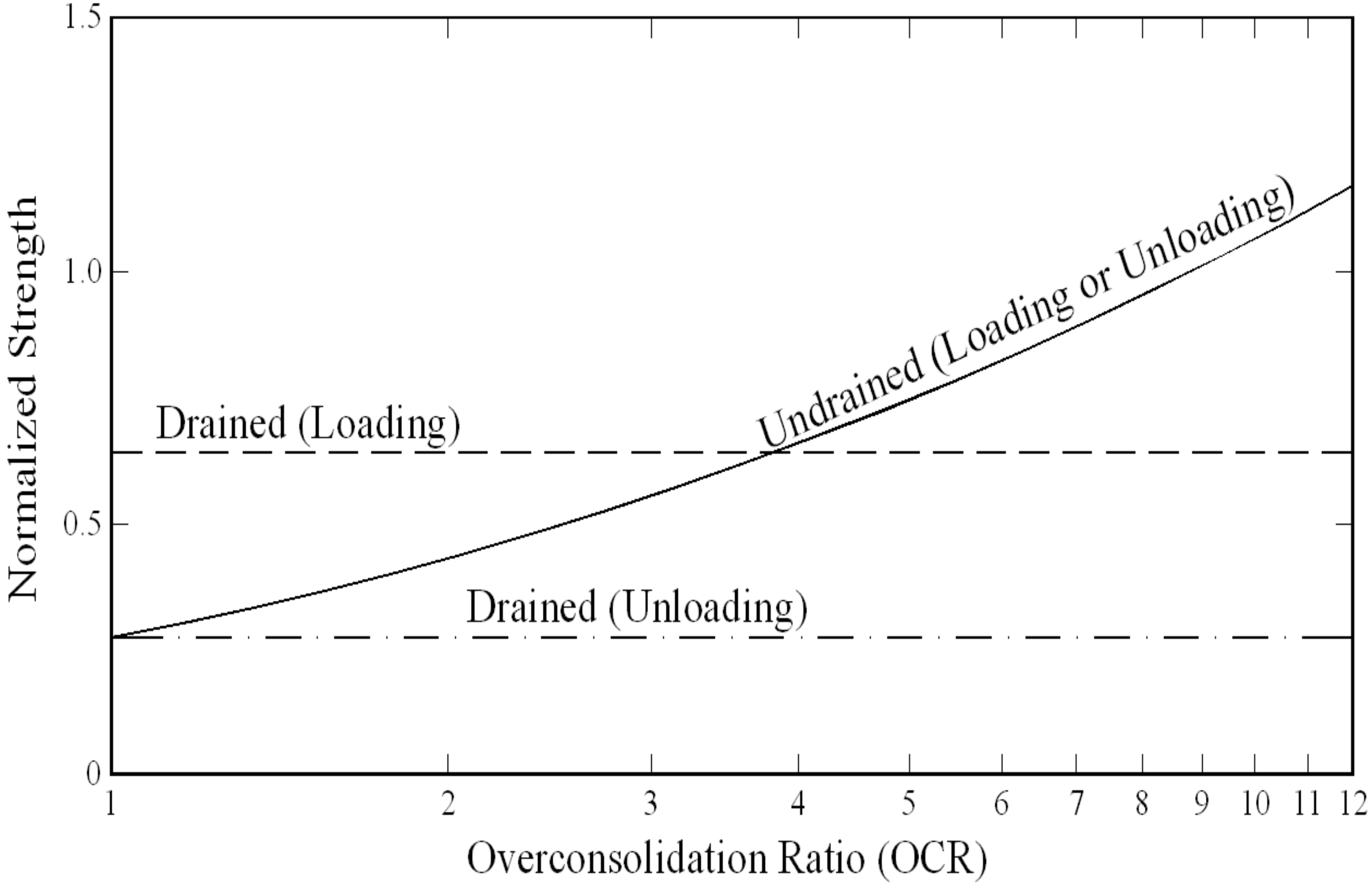
  Long-term (Drained)

  Also applicable for Undrained (Short -Term)

Effect of OCR on Drained and Undrained Strength of Clay (Edil, 1982)

There are 3 variable aspects that contribute to critical conditions in geotechnical stability problems in saturated cohesive soils:

(a) Over-Consolidation Ratio (OCR)
(b) Stress-Path
(c) Pore-water Condition (Undrained or Drained)



Choice of Shear Strength Parameters from Laboratory Tests

- Short term stability (End of construction)
  - Triaxial test
    - UU test → Undrained Strength , S_u
    - CU test → C_{cu} dan ϕ_{cu}
  - Unconfined compression test → Unconfined strength, q_u

- Long term stability
  - Direct shear test → C' dan ϕ'
  - Triaxial test
    - CD test
    - CU test dg pengukuran tek. air pori
    - → C' dan ϕ'
  - Ring shear test → C'_r dan ϕ'_r residual

Pemilihan Tipe Parameter Kuat Geser dari Tes Triaksial (Lee, 1996)

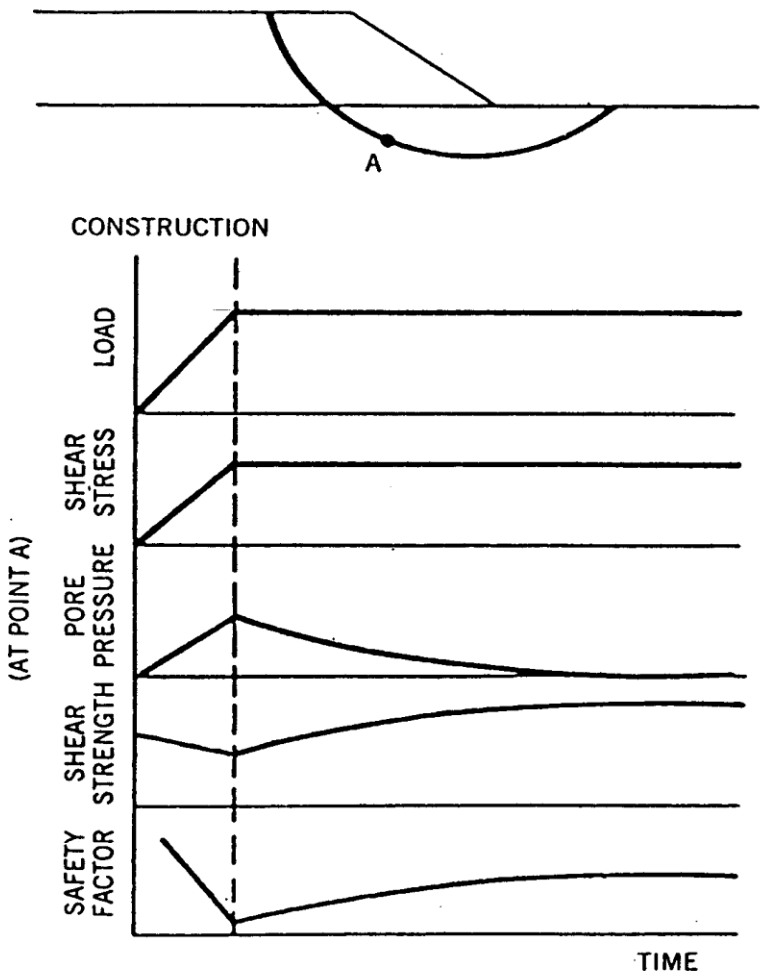
Jenis Tanah	Jenis Konstruksi	Jenis Tes dan Kekuatan Geser
Kohesif	Jangka pendek (short term/end of construction)	Test UU atau CU untuk *undrained strength* dengan level tegangan insitu yang sesuai.
	Konstruksi bertahap (staged construction)	Test CU untuk *undrained strength* dengan level tegangan yang sesuai.
	Jangka panjang (long term)	Test CU dengan pengukuran *pore pressure*, atau tes CD untuk parameter kuat geser efektif.
Granular	Semua jenis	Parameter *strength* ϕ' didapatkan dari tes lapangan atau tes *direct shear*.
c-ϕ material	Jangka panjang (long term)	Tes CU dengan pengukuran *pore pressure* atau tes CD untuk parameter kuat geser efektif.

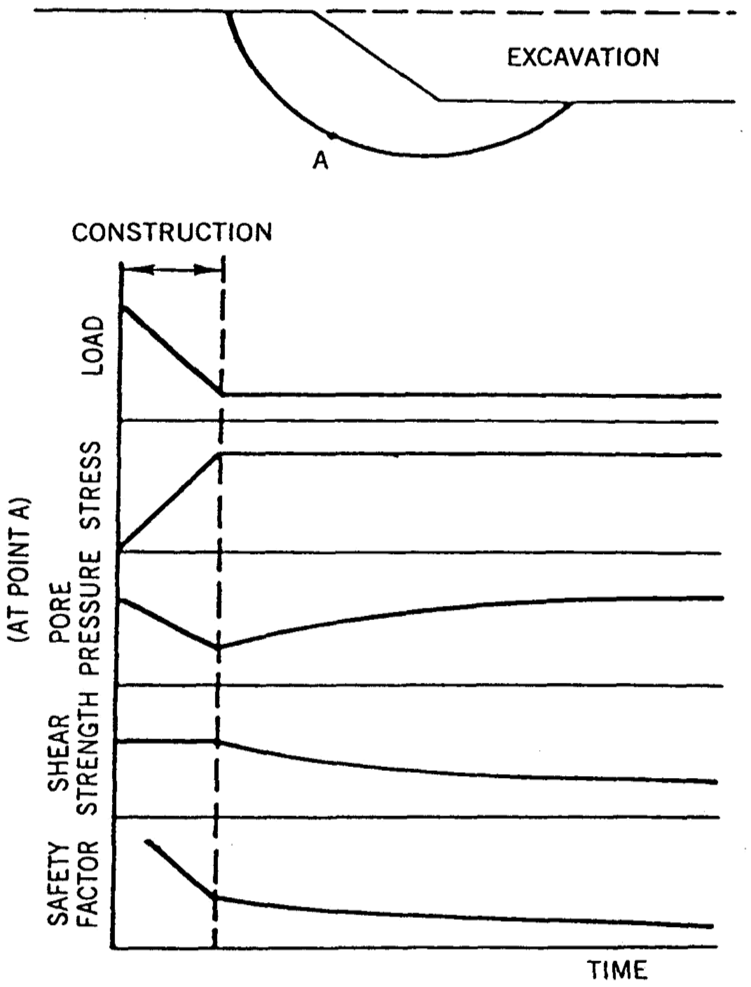
Kondisi Kritis untuk Stabilitas pada Tanah Lempung Jenuh (Lee, 1995)

	Jenis Tanah	
	Soft (NC) Clay	**Stiff (Highly OC)**
	Timbunan	
Kondisi kritis Catatan:	Kasus Unconsolidated Undrained (UU) tanpa drainase Gunakan $\phi = 0$, $c = \tau_{ff}$ dengan koreksi yang sesuai.	Kemungkinan kasus UU tapi cek juga kasus Consolidated Drained (CD) Stabilitas biasanya bukan problem utama.
	Galian atau *Natural Slope*	
Kondisi kritikal Catatan:	Bisa keduanya, kasus UU atau CD. Jika tanah sangat sensitif, dapat beralih dari kondisi drained ke undrained.	Kasus CD (drainase penuh). Gunakan analisis tegangan efektif *dg equilibrium pore presssure*; jika clay agak fissured, c' dan juga mungkin ϕ' dapat menurun sbg fungsi

Kondisi stabilitas untuk sebuah timbunan dan galian lereng pada tanah lempung jenuh NC

(Sumber Edil T.B., 1982; Bishop and Bjerrum, 1960)





USE OF DRAINED VERSUS UNDRAINED STRENGTH FOR COHESIVE SOILS

- For unloading such as in excavations, cuts or river or coastal erosion of slopes, the drained strength is always equal (only for NC clay, OCR =1) or less (for OC clay, OCR > 1) than the undrained strength. Therefore, in the long-term as the flow of water into soil takes place in response to reduced pore pressures resulting from unloading (negative Δp in equation $q_f = a' + p'_f \tan \alpha'$) the strength drops and the critical strength to be used in the analysis. Simple implication of this fact is that an open cut which can be cut to stand initially may cave in after some time.
- For loading such as embankment construction, foundations, etc., the undrained strength is less than the drained strength for OCR values less than 2 to 4 and therefore, the end-of-construction (short-term) undrained condition is the most critical.
- As the positive pore pressures (due to positive Δp and positive α) dissipate the strength increases to the drained value in the long-term. If OCR is greater than 2 to 4, then the drained strength become less than the undrained. This is because α is negative for heavily over-consolidated soils and results in negative Δu even though Δp is positive due to loading. In such cases the long-term strength become critical similar to the unloading conditions.

STRESS-DEFORMATION AND STRENGTH-CHARACTERISTICS OF SATURATED COHESIVE SOILS

- Basically, the same things happen when clay soils are sheared. In drained shear, whether the volume changes are dilation or compression depends not only on the density and the confining pressure but also on the stress history of the soil. Similarly, in undrained shear the pore pressures developed depend greatly on whether the soil is normally consolidated or over-consolidated.
- Typically, engineering loads are applied much faster than the water can escape from the pores of a clay soil, and consequently excess hydro static or pore pressures are produced.
- The primary difference in behavior between sands and clays is the compressibility of soils, is in the time it takes for these volume changes to occur.
- The time aspect strictly depends on, or is a function of, the difference in permeability between sands and clays. Since cohesive soils have much lower permeability than sands and gravels, it takes much longer for the water to flow in or out of a cohesive soil mass

STRESS-DEFORMATION AND STRENGTH-CHARACTERISTICS OF SATURATED COHESIVE SOILS

- What happens when the loading is such that a shear failure is imminent?
- Since the pore water cannot carry any shear stress, all the applied shear stress must be resisted by the soil structure. Put another way, **the shear strength of the soil depends only on the effective stresses** and not on the pore water pressures.
- This does not mean that the pore pressure induced in the soil are unimportant. On the contrary, as the total stresses are changed because of some engineering loading, the pore water pressures also change, and until equilibrium of effective stresses occurs instability is possible.
- These observations lead to two fundamentally different approaches to the solution of stability problems in geotechnical engineering:

1. Total stress approach
2. Effective stress approach.

STRESS-DEFORMATION AND STRENGTH-CHARACTERISTICS OF SATURATED COHESIVE SOILS

- In the total stress approach, we allow no drainage to take place during the shear test, and we make the assumption, admittedly a big one, that the pore water pressure and therefore the effective stresses in the test specimen are identical to those in the field. The method of stability analysis is called the total stress analysis and it utilize the *undrained shear strength* (Su) of soil.

- The second approach to calculate the stability of geotechnical engineering problem uses the shear strength in terms of effective stresses. In this approach, we have to measure or estimate the excess hydrostatic pressure, both in the laboratory and in the field. Then, if we know or can estimate the initial and applied total stresses, we may calculate the effective stresses acting in the soil.

STRESS-DEFORMATION AND STRENGTH-CHARACTERISTICS OF SATURATED COHESIVE SOILS

- Since we believe that shear strength and stress-deformation behavior of soils is really controlled or determined by the effective stress, the second approach (effective stress analysis) is philosophically more satisfying.

- But, it does have its practical problems. For example, estimating or measuring the pore pressures, especially in the field, is not easy to do.

- The effective stress analysis utilizes the drained shear strength or the shear strength in terms of effective stresses. The drained shear strength is ordinarily only determined by laboratory tests.

NOTES ON HEAVILY OVER-CONSOLIDATED CLAY

- In a CU or UU test, the sample is maintained at constant volume during shearing. Since a heavily over-consolidated clay has a tendency to dilate (similar to a dense granular soil in this respect), the pore pressure change, Δu, will become negative. In other words, coefficient α for **tan α** is negative for heavily OC clays while it is positive for NC clays and lightly OC clays. Because Δu becomes negative during undrained shear on a heavily OC clay, the effective stresses are larger than the total stresses, and hence the undrained strength will be greater than the drained strength. In the field the negative pore pressures over a long period will draw water into the pores, <u>causing an increase in void ratio and a decrease in strength</u>.
- Furthermore, many heavily over-consolidated clays contain a network of very small cracks and fissures. Their shear strength is often time dependent and difficult to describe.

Total and Effective Stress Approach for Solution of Stability Problems in Geotechnical Engineering

Untuk solusi praktis dalam masalah stabilitas geoteknik, dilakukan 2 pendekatan analisis, yaitu:

- ***Total Stress Analysis*** ***(Undrained Condition)***
- ***Effective Stress Analysis*** ***(Drained Condition)***

$$\tau_{ff} = (\sigma_{ff} - u) \tan \phi' + c' \quad ; \quad u = u_o + \Delta u$$

Undrained → **Drained**

Δu time **$\Delta u = 0$**

(End of Construction) **(Long-Term)**

Dengan kriteria keruntuhan Mohr-Coulomb kita dapat menghitung tegangan-tegangan pada bidang runtuh pada saat keruntuhan terjadi dan mengevaluasi Factor of Safety (FoS):

$$FoS = \tau_{ff\ (yang\ ada)} / \tau_{f\ (yang\ bekerja)}$$

Effective Stress Analysis

- **Consistently: use of effective stress and effective strength and modulus parameters.**
- **Consequently: Need to calculate $u = u_o + \Delta u$ for any condition from undrained to drained conditions (It is also applicable for short-term (end of construction) stability analysis as long as Δu is available).**
- **Practical for Long-Term stability condition, since u is easy to evaluate and $\Delta u = 0$.**

Total Stress Analysis

- **Consistently: use total stress and total strength and modulus parameters.**
- **Advantage: no need to compute pore water pressure u**
- **Very practical Short-Term (end of construction) stability condition.**

Available FE Program For Geotechnical And Soil-Structure Interaction Analysis

NLSSIP (Duncan)	**Hyperbolic**
SOILSTRUCT (Duncan)	**Hyperbolic**
SIGMA/W (Geo-Slope International)	**: Elastic-Perfectly Plastic (Mohr-Coulomb)** **- Cam-Clay**
CRISP-90	**: Cam-Clay**
PLAXIS	**: - MC Elastic-perfectly Plastic, Drucker-Prager** **- Hardening, Soft Soil Models**
ABAQUS	**-Hyperbolic, Elastic-perfectly Plastic, Cam-Clay**
MIDAS	**-Cam-Clay, Drucker-Prager, Hardening**

Undrained Analysis With Effective Parameters

It is possible to specify undrained behavior in an effective stress analysis using effective model parameters. This is achieved by identifying the type of material behavior (or material type) of a soil layer as undrained.

The presence of pore water pressure in a soil body, usually caused by water, contributes to the total stress level. According to Terzaghi's principle, total stress $\underline{\sigma}$ can be divided into effective stress $\underline{\sigma}'$ and pore pressures $\underline{\sigma}_w$:

$$\sigma_{xx} = \sigma_{xx}' + \sigma_w$$

$$\sigma_{yy} = \sigma_{yy}' + \sigma_w$$

$$\sigma_{zz} = \sigma_{zz}' + \sigma_w$$

$$\sigma_{xy} = \sigma_{xy}$$

$$\boldsymbol{\sigma_w = u = u_o + \Delta u}$$

u = pore water pressure (pwp)

u_o = initial pwp (hydrostatic or seepage)

Δu = excess pwp due to change in stress

Notes on Practical Analysis Using PLAXIS

Short -term Analysis (During Construction) Undrained	**Long-term Analysis (Operation) Drained**

• Effective Stress Analysis (Undrained)
  • Material Type **Undrained A**, use : c’, φ’, E’ and ν’ ; Excess pore pressure (**Δu) ≠ 0**
 	
• Effective Stress Analysis (Undrained)
  • Material Type **Undrained B** , use : E’ and ν’, c=Su, φ= 0˚ ; Excess pore pressure (**Δu) =0**
 	
• Total Stress Analysis (Undrained)
  • Material Type **Undrained C**, use : c=Su, φ= 0˚, Eu and ν = 0.495 ; Excess pore pressure (**Δu)= 0**
	• Effective Stress Analysis (Drained)
	  • Material Type Drained, use : c’, φ’, E’ and ν’ ; Excess pore pressure (**Δu)= 0**

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