1 Principles of Triaxial Testing

1.1 Purpose of triaxial tests

The purpose of performing triaxial tests is to determine the mechanical properties of the soil. It is assumed that the soil specimens to be tested are homogeneous and representative of the material in the field, and that the desired soil properties can in fact be obtained from the triaxial tests, either directly or by interpretation through some theory.

The mechanical properties most often sought from triaxial tests are stress–strain relations, volume change or pore pressure behavior, and shear strength of the soil. Included in the stress–strain behavior are also the compressibility and the value of the coefficient of earth pressure at rest, K_0. Other properties that may be obtained from the triaxial tests, which include time as a component, are the permeability, the coefficient of consolidation, and properties relating to time dependent behavior such as rate effects, creep, and stress relaxation.

It is important that the natural soil deposit or the fill from which soil samples have been taken in the field are sufficiently uniform that the soil samples possess the properties which are appropriate and representative of the soil mass in the field. It is therefore paramount that the geology at the site is well-known and understood. Even then, samples from uniform deposits may not "contain" properties that are representative of the field deposit. This may happen either (a) due to the change in effective stress state which is always associated with the sampling process or (b) due to mechanical disturbance from sampling, transportation, or handling in the laboratory. The stress–strain and strength properties of very sensitive clays which have been disturbed cannot be regenerated in the laboratory or otherwise obtained by interpretation of tests performed on inadequate specimens. The effects of sampling will briefly be discussed below in connection with choice of consolidation pressure in the triaxial test. The topic of sampling is otherwise outside the scope of the present treatment.

1.2 Concept of testing

The concept to be pursued in testing of soils is to simulate as closely as possible the process that goes on in the field. Because there is a large number of variables (e.g., density, water content, degree of saturation, overconsolidation ratio, loading conditions, stress paths) that influence the resulting soil behavior, the simplest and most direct way of obtaining information pertinent to the field conditions is to duplicate these as closely as possible.

Triaxial Testing of Soils, First Edition. Poul V. Lade.

However, because of limitations in equipment and because of practical limitations on the amount of testing that can be performed for each project, it is essential that:

1. The true field loading conditions (including the drainage conditions) are known.
2. The laboratory equipment can reproduce these conditions to a required degree of accuracy.
3. A reasonable estimate can be made of the significance of the differences between the field loading conditions and those that can be produced in the laboratory equipment.

It is clear that the triaxial test in many respects is incapable of simulating several important aspects of field loading conditions. For example, the effects of the intermediate principal stress, the effects of rotation of principal stresses, and the effects of partial drainage during loading in the field cannot be investigated on the basis of the triaxial test. The effects of such conditions require studies involving other types of equipment or analyses of boundary value problems, either by closed form solutions or solutions obtained by numerical techniques.

To provide some background for evaluation of the results of triaxial tests, other types of laboratory shear tests and typical results from such tests are presented in Chapter 11. The relations between the different types of tests are reviewed, and their advantages and limitations are discussed.

1.3 The triaxial test

The triaxial test is most often performed on a cylindrical specimen, as shown in Fig. 1.1(a). Principal stresses are applied to the specimen, as indicated in Fig. 1.1(b). First a confining pressure, σ_3, is applied to the specimen. This pressure acts all around and therefore on all planes in the specimen. Then an additional stress difference, σ_d, is applied in the axial direction. The stress applied externally to the specimen in the axial direction is

$$\sigma_1 = \sigma_d + \sigma_3 \tag{1.1}$$

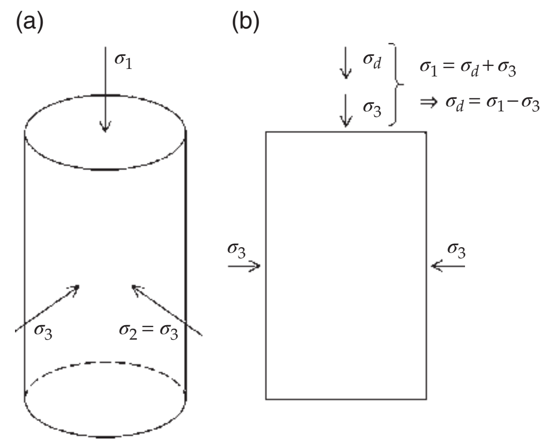


Figure 1.1 (a) Cylindrical specimen for triaxial testing and (b) stresses applied to a triaxial specimen.

and therefore

$$\sigma_d = \sigma_1 - \sigma_3 \tag{1.2}$$

In the general case, three principal stresses, σ_1, σ_2 and σ_3 may act on a soil element in the field. However, only two different principal stresses can be applied to the specimen in the conventional triaxial test. The intermediate principal stress, σ_2, can only have values as follows:

$$\sigma_2 = \sigma_3 : \text{Triaxial compression} \tag{1.3}$$

$$\sigma_2 = \sigma_1 : \text{Triaxial extension} \tag{1.4}$$

The condition of triaxial extension can be achieved by applying negative stress differences to the specimen. This merely produces a reduction in compression in the extension direction, but no tension occurs in the specimen. The state of stress applied to the specimen is in both cases axisymmetric. The triaxial compression test will be discussed in the following, while the triaxial extension test is discussed in Chapter 10.

The test is performed using triaxial apparatus, as seen in the schematic illustration in Fig. 1.2. The specimen is surrounded by a cap and a base and a membrane. This unit is placed in a triaxial cell in which the cell pressure can be applied. The cell pressure acts as a hydrostatic confinement for the specimen, and the pressure is therefore the same in all directions. In addition,

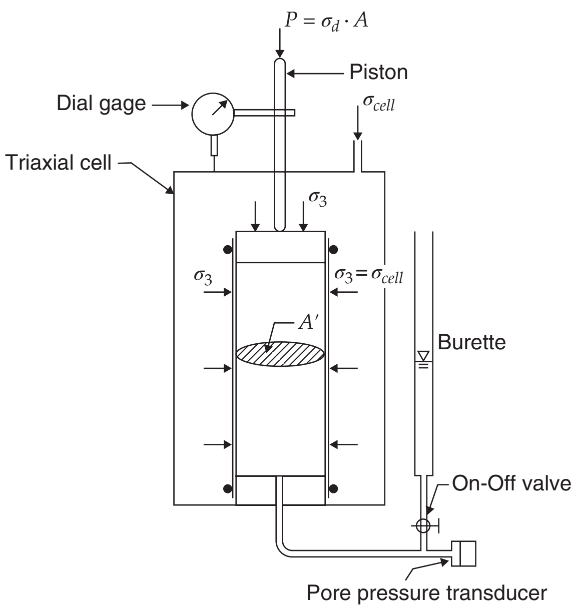


Figure 1.2 Schematic diagram of triaxial apparatus.

a deviator load can be applied through a piston that goes through the top of the cell and loads the specimen in the axial direction.

The vertical deformation of the specimen may be measured by a dial gage attached to the piston which travels the same vertical distance as the cap sitting on top of the specimen. Drainage lines are connected to the water saturated specimen through the base (or both the cap and the base) and connected to a burette outside the triaxial cell. This allows for measurements of the volume changes of the specimen during the test.

Alternatively, the connection to the burette can be shut off thereby preventing the specimen from changing volume. Instead the pore water pressure can be measured on a transducer connected to the drainage line.

The following quantities are measured in a typical triaxial test:

1. Confining pressure
2. Deviator load
3. Vertical (or axial) deformation
4. Volume change or pore water pressure

These measurements constitute the data base from which other quantities can be derived [e.g., stress difference $(\sigma_1 - \sigma_3)$, axial strain ε_1, and volumetric strain ε_v].

1.4 Advantages and limitations

Whereas the triaxial test potentially can provide a substantial proportion of the mechanical properties required for a project, it has limitations, especially when special conditions are encountered and necessitates clarification based on experimentation.

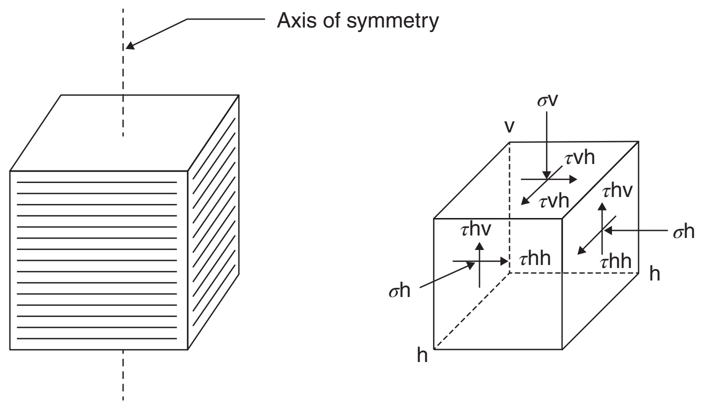
The *advantages* of the triaxial test are:

1. Drainage can be controlled (on–off)
2. Volume change or pore pressure can be measured
3. Suction can be controlled in partially saturated soils
4. Measured deformations allow calculation of strains and moduli
5. A larger variety of stress and strain paths that occur in the field can be applied in the triaxial apparatus than in any other testing apparatus (e.g., initial anisotropic consolidation at any stress ratio including K_0, extension, active and passive shear).

The *limitations* of the triaxial test are:

1. Stress concentrations due to friction between specimen and end plates (cap and base) cause nonuniform strains and stresses and therefore nonuniform stress–strain, volume change, or pore pressure response.
2. Only axisymmetric stress conditions can be applied to the specimen, whereas most field problems involve plane strain or general three-dimensional conditions with rotation of principal stresses.
3. Triaxial tests cannot provide all necessary data required to characterize the behavior of an anisotropic or a cross-anisotropic soil deposit, as illustrated in Fig. 1.3.
4. Although the axisymmetric principal stress condition is limited, it is more difficult to apply proper shear stresses or tension to soil in relatively simple tests.

The first limitation listed above can be overcome by applying lubricated ends on the



Require tests with application of shear stresses

$$\begin{bmatrix} \frac{1}{Eh} & \frac{-\mu hv}{Eh} & \frac{-\mu hh}{Eh} & 0 & 0 & 0 \\ \frac{-\mu hv}{Eh} & \frac{1}{Ev} & \frac{-\mu hv}{Eh} & 0 & 0 & 0 \\ \frac{-\mu hh}{En} & \frac{-\mu hv}{En} & \frac{1}{Eh} & 0 & 0 & 0 \\ 0 & 0 & 0 & \frac{1}{Ghv} & 0 & 0 \\ 0 & 0 & 0 & 0 & \frac{1}{Ghv} & 0 \\ 0 & 0 & 0 & 0 & 0 & \frac{2(1+\mu hh)}{En} \end{bmatrix} \cdot \begin{Bmatrix} \sigma h \\ \sigma v \\ \sigma h \\ \tau hv \\ \tau vh \\ \tau hh \end{Bmatrix} = \begin{Bmatrix} \epsilon h \\ \epsilon v \\ \epsilon h \\ \gamma hv \\ \gamma vh \\ \gamma hh \end{Bmatrix}$$

Figure 1.3 Cross-anisotropic soil requiring results from more than triaxial tests for full characterization.

specimen such that uniform strains and stresses and therefore correct soil response can be produced. This is discussed in Chapter 3. In addition to the limitations listed above, it should be mentioned that it may be easier to reproduce certain stress paths in other specialty equipment than in the triaxial apparatus (e.g., K_0-test).

Although the triaxial test is limited as explained under points 2 and 3 above, it does combine versatility with relative simplicity in concept and performance. Other equipment in which three unequal principal stresses can be applied or in which the principal stress directions can be rotated do not have the versatility or is more complicated to operate. Thus, other types of equipment have their own advantages and limitations. These other equipment types include plane strain, true triaxial, simple shear, directional shear, and torsion shear apparatus. All these pieces of equipment are, with the exception of the simple shear apparatus, employed mainly for research purposes. Their operational modes, capabilities and results are reviewed in Chapter 11.

1.5 Test stages – consolidation and shearing

Laboratory tests are made to simulate field loading conditions as close as possible. Most field conditions and the corresponding tests can be simplified to consist of two stages: consolidation and shearing.

1.5.1 Consolidation

In the first stage the initial condition of the soil is established in terms of effective stresses and stress history (including overconsolidation, if applicable). Thus, stresses are applied corresponding to those acting on the element of soil in the field due to weight of the overlying soil strata and other materials or structures that exist at the time the mechanical properties (stress–strain, strength, etc.) are sought. Sufficient time is allowed for complete consolidation to occur under the applied stresses. The condition in the field element has now been established in the triaxial specimen.

1.5.2 Shearing

In the second stage of the triaxial test an additional stress is applied to reach peak failure and beyond under relevant drainage conditions. The additional stress applied to the specimen should correspond as closely as possible to the change in stress on the field element due to some new change in the overall field loading situation. This change may consist of a vertical stress increase or decrease (e.g., due to addition of a structure or excavation of overlying soil strata) or of a horizontal stress increase or decrease (e.g., due to the same constructions causing the vertical stress changes). Any combination of vertical and horizontal stress changes may be simulated in the triaxial test. Examples of vertical and horizontal stress changes in the field are shown in Fig. 1.4.

Usually, it is desirable to know how much change in load the soil can sustain without failing and how much deformation will occur under normal working conditions. The test is therefore usually continued to find the strength of the soil under the appropriate loading conditions. The results are used with an appropriate factor of safety so that normal working stresses are always somewhat below the peak strength.

The stress–strain relations obtained from the triaxial tests provide the basis for determination of deformations in the field. This may be done in a simplified manner by closed-form solutions or it may be done by employing the results of the triaxial tests for calibration of a constitutive model used with a numerical method in finite element or finite difference computer programs.

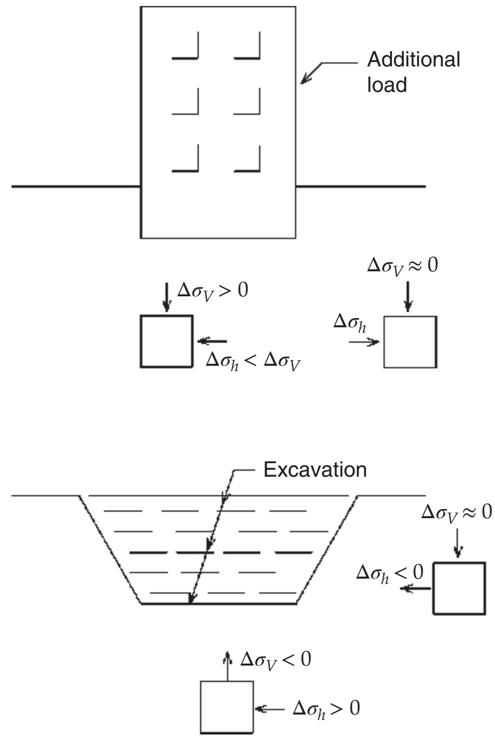


Figure 1.4 Examples of stress changes leading to failure in the field.

1.6 Types of tests

The drainage conditions in the field must be duplicated as well as possible in the laboratory tests. This may be done by appropriate drainage facilities or preventions as discussed above for the triaxial test. In most cases the field drainage conditions can be approximated by one of the following three types of tests:

1. Consolidated-drained test, called a CD-test, or just a drained test
2. Consolidated-undrained test, or a CU-test
3. Unconsolidated-undrained test, or a UU-test

These tests are described in ASTM Standards D7181 (2014), D4767 (2014), and D2850 (2014), respectively.

Which condition of drainage in the laboratory test logically corresponds to each case in the field depends on a comparison of loading rate with the rate at which the water can escape or be sucked into the ground. Thus, the permeability of the soil and the drainage boundary conditions in the field together with the loading rate play key roles in determination of the type of analysis and the type of test, drained or undrained, that are appropriate for each case. Field cases with partial drainage can be correctly duplicated in laboratory tests if the effective stress path is determined for the design condition. However, the idea of the CD-, CU- and UU-tests is to make it relatively simple for the design engineer to analyze a condition that will render a sufficient factor of safety under the actual drainage condition, without trying to estimate and experimentally replicate the actual stress path.

It has been determined through experience and common sense that the extreme conditions are drained and undrained with and without consolidation. As a practical matter, in a commercial laboratory it is easier to run an undrained test than a drained test because it is easier and faster to measure pore pressures than volume change. Therefore, even drained parameters are more likely to be estimated from a CU-test than from a CD-test.

1.6.1 Simulation of field conditions

Presented below is a brief review of the three types of tests together with examples of field cases for which the tests are appropriate and with typical strength results shown on Mohr diagrams.

Drained tests

Isotropic consolidation is most often used in the first stage of the triaxial test. However, anisotropic consolidation with any stress ratio is also possible.

The shearing stage of a drained test is performed so slowly, the soil is so permeable and the drainage facilities are such that no excess pore pressure (positive or negative) can exist in the specimen at any stage of the test, that is

$$\Delta u = 0 \tag{1.5}$$

It follows then from the effective stress principle

$$\sigma' = \sigma - u \tag{1.6}$$

that the effective stress changes are always the same as the total stress changes.

A soil specimen always changes volume during shearing in a drained test. If it contracts in volume, it expels pore fluid (usually water or air), and if it expands in volume (dilates), then it sucks water or air into the pores. If a non-zero pore pressure is generated during the test (e.g., by performing the shearing too fast so the water does not have sufficient time to escape), then the specimen will expel or suck water such that the pore pressure goes towards zero to try to achieve equilibrium between externally applied stresses and internal effective stresses. Thus, there will always be volume changes in a drained test. Consequently, the water content, the void ratio, and the dry density of the specimen at the end of the test are most often not the same as at the beginning.

The following field conditions can be simulated with acceptable accuracy in the drained test:

1. Almost all cases involving coarse sands and gravel, whether saturated or not (except if confined in e.g., a lens and/or exposed to rapid loading as in e.g., an earthquake).
2. Many cases involving fine sand and sometimes silt if the field loads are applied reasonable slowly.
3. Long term loading of any soil, as for example:
   a) Cut slopes several years after excavation
   b) Embankment constructed very slowly in layers over a soft clay deposit
   c) Earth dam with steady seepage
   d) Foundation on clay a long time after construction.

These cases are illustrated in Fig. 1.5.

The strength results obtained from drained tests are illustrated schematically on the Mohr diagram in Fig. 1.6. The shear strength of soils increases with increasing confining pressure.

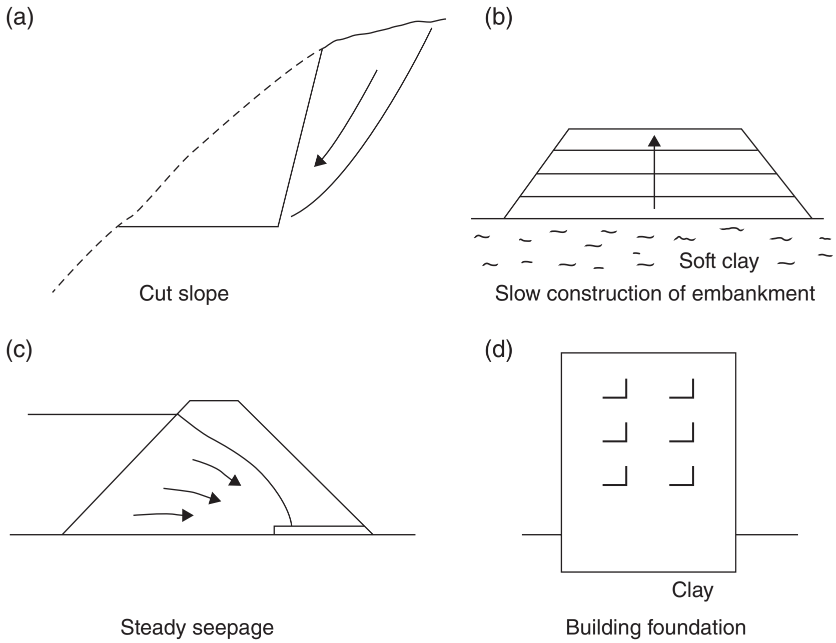


Figure 1.5 Examples of field cases for which long term stability may be determined on the basis of results from drained tests.

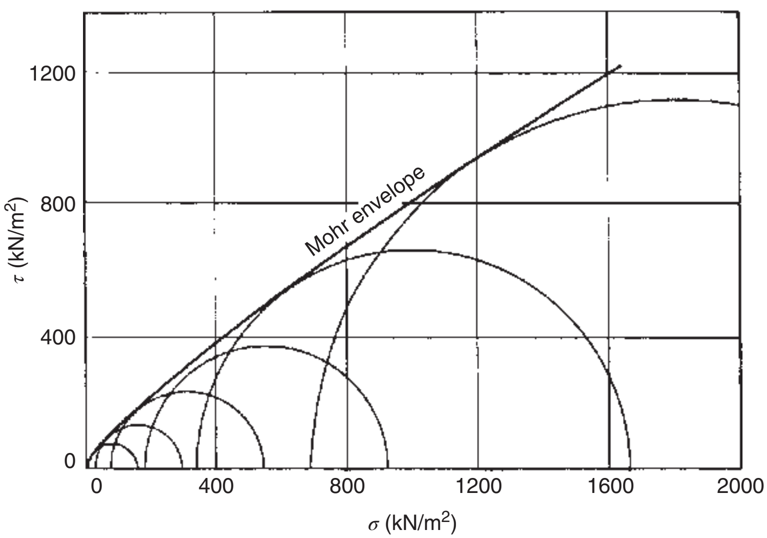


Figure 1.6 Schematic illustration of a Mohr diagram with failure envelope for drained tests on soil.

In the diagram in Fig. 1.6 the total stresses are equal to the effective stresses since there are no changes in pore pressures [Eqs (1.5) and (1.6)].

The effective friction angle, φ', decreases for all soils with increasing confining pressure, and the failure envelope is therefore curved, as indicated in Fig. 1.6. The effective cohesion, c', is zero or very small, even for overconsolidated clays. Effective or true cohesion of any significant magnitude is only present in cemented soils.

The effective stress failure envelope then defines the boundary between states of stress that can be reached in a soil element and states of stress that cannot be reached by the soil at its given dry density and water content.

Consolidated-undrained tests

As in drained tests, isotropic consolidation is most often used in CU-tests. However, anisotropic consolidation can also be applied, and it may have greater influence on the results from CU-tests than those from drained tests. The specimen is allowed to fully consolidate such that equilibrium has been obtained under the applied stresses and no excess pore pressure exists in the specimen.

The undrained shearing stage is begun by closing the drainage valve before shear loading is initiated. Thus, no drainage is permitted, and the tendency for volume change is reflected by a change in pore pressure, which may be measured by the transducer (see Fig. 1.2). Therefore the second stage of the CU-test on a saturated specimen is characterized by:

$$\Delta V = 0 \tag{1.7}$$

and

$$\Delta u \neq 0 \tag{1.8}$$

According to the effective stress principle in Eq. (1.6), the effective stresses are therefore different from the total stresses applied in a CU-test.

The pore pressure response is directly related to the tendency of the soil to change volume. This is illustrated in Fig. 1.7. Thus, there will always be pore pressure changes in an undrained test. However, since there are no volume changes of the fully saturated specimen, the water content, the void ratio and the dry density at the end of the test will be the same as at the end of the consolidation stage.

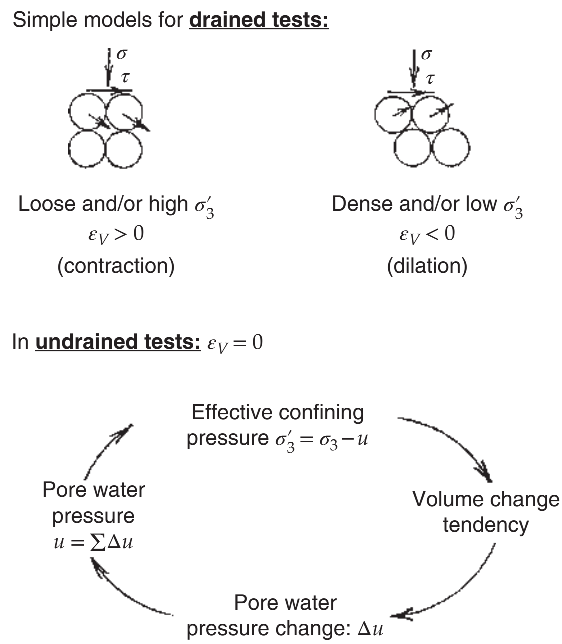


Figure 1.7 Schematic illustration of changes in pore water pressure in undrained tests.

The following field conditions can be simulated with good accuracy in the CU-test:

1. Most cases involving short term strength, that is strength of relatively impervious soil deposits (clays and clayey soils) that are to be loaded over periods ranging from several days to several weeks (sometimes even years for very fat clays in massive deposits) following initial consolidation under existing stresses before loading. Examples of field cases in which short term stability considerations are appropriate:
   a) Building foundations
   b) Highway embankments, dams, highway foundations
   c) Earth dams during rapid drawdown (special considerations are required here, see Duncan and Wright 2005)These cases are illustrated in Fig. 1.8.
2. Prediction of strength variation with depth in a uniform soil deposit from which samples can only be retrieved near the ground surface. This is illustrated in Fig. 1.9.

The strength results obtained from CU-tests are illustrated schematically on the Mohr diagram in Fig. 1.10. Since pore pressures develop in CU-tests, two types of strengths can be derived from undrained tests: total strength; and effective strength. The Mohr circles corresponding to

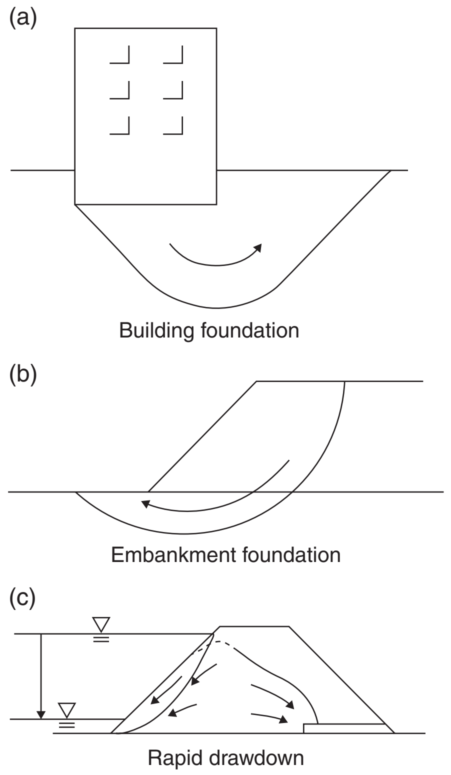


Figure 1.8 Examples of field cases for which short term stability may be determined on the basis of results of CU-tests.

these two strengths will always have the same diameter, but they are displaced by Δu from each other.

Both the total and effective stress envelopes from CU-tests on clays and clayey soils indicate increasing strength with increasing confining pressure. As for the drained tests, the effective friction angle, φ', decreases with increasing confining pressure, and the curvature of the failure envelope is sometimes more pronounced than for sands. In fact, the effective strength envelope obtained from CU-tests is very similar to that obtained from drained tests. Thus, the effective cohesion, c', is zero except for cemented soils. In particular, the effective cohesion is zero for remolded or compacted soils.

The total stress friction angle, φ, is much lower than the effective stress friction angle, φ', whereas the total stress cohesion, c, can have a substantial magnitude. The total stress friction angle is not a friction angle in the same sense as the effective stress friction angle. In the latter case, φ' is a measure of the strength derived from the applied normal stress, while φ is a measure of the strength gained from the *consolidation* stress only. If, for example, the total stress parameters are applied in a slope stability calculation in which a surcharge is suddenly added, then the surcharge will contribute to the shear resistance in the analysis (which is incorrect) as well as to the driving force, because there is no distinction between the normal forces derived from consolidation stresses and those caused by the surcharge. A better approach would be to assign undrained shear strengths (s_u) based on the consolidation stress state by using an approach that involves s_u/σ_v'.

Unconsolidated-undrained tests

In the UU-test a confining pressure is first applied to the specimen and *no drainage* is allowed. In fact, UU-tests are most often performed in triaxial equipment without facilities for drainage. The soil has already been consolidated in the field, and the specimen is therefore considered to "contain" the mechanical properties that are present at the location in the ground where the sample was taken. Alternatively, the soil may consist of compacted fill whose undrained strength is required for stability analysis before any consolidation has occurred in the field.

The undrained shearing stage follows immediately after application of the confining pressure. The shear load is usually increased relatively fast until failure occurs. No drainage is permitted during shear. Thus, the volume change is zero for a saturated specimen and the pore pressure is different from zero, as indicated in Eqs (1.7) and (1.8). The pore pressure is not measured and only the total strength is obtained from this test.

Since there are no volume changes in a saturated specimen, the void ratio, the water content and the dry density at the end of the test will be the same as those in the ground.

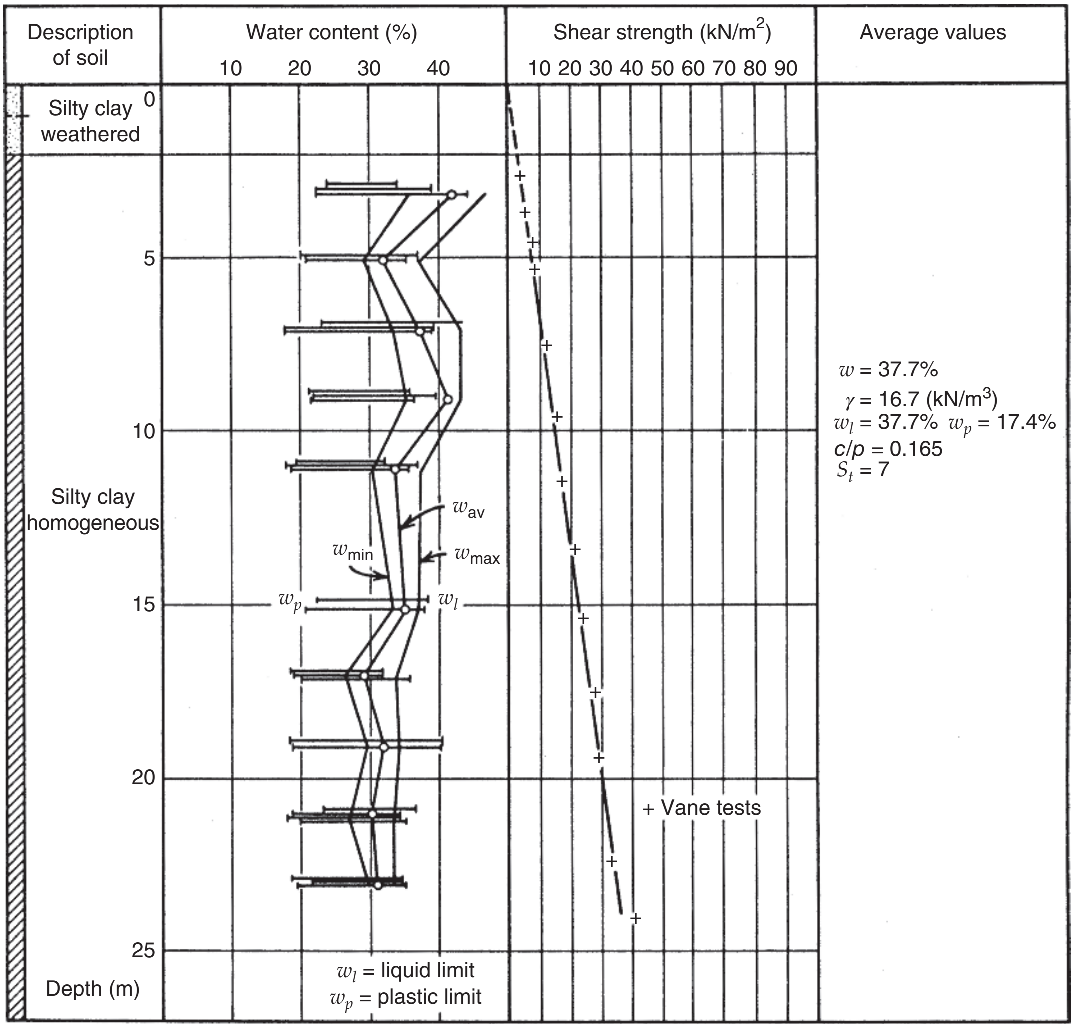


Figure 1.9 Strength variation with depth in uniform soil deposit of Norwegian marine clay. Reproduced from Bjerrum 1954 by permission of Geotechnique.

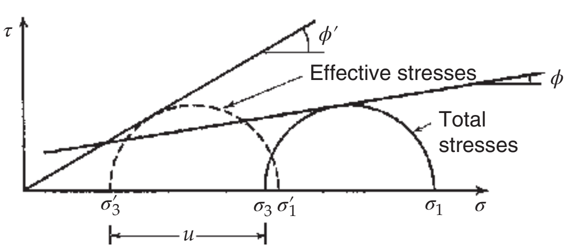


Figure 1.10 Schematic illustration of a Mohr diagram with total stress and effective stress failure envelopes from CU-tests on soil (after Bishop and Henkel 1962).

The following field conditions may be simulated in the UU-test:

1. Most cohesive soils of relatively poor drainage, where the field loads would be applied sufficiently rapidly that drainage does not occur. Examples of field cases for which results of UU-tests may be used:
   a) Compacted fill in an earth dam that is being constructed rapidly
   b) Strength of a foundation soil that will be loaded rapidly

c) Strength of soil in an excavation immediately after the cut is made

These cases are illustrated in Fig. 1.11.

2. Undisturbed, saturated soil, where a sample has been removed from depth, installed in a triaxial cell, and pressurized to simulate the overburden in the field.

The strength results obtained from UU-tests on *saturated* soil are illustrated schematically on the Mohr diagram in Fig. 1.12. The strength obtained from UU-tests on saturated soil is *not* affected by the magnitude of the confining pressure. This is because consolidation is not allowed after application of the confining pressure. Thus, the actual effective confining pressure in the saturated soil does not depend on the applied confining pressure, and the same strength is therefore obtained for all confining pressures. Consequently, the total strength envelope is horizontal corresponding to $\varphi = 0$, and the strength is therefore characterized by the undrained shear strength:

$$s_u = \frac{1}{2}(\sigma_1 - \sigma_3) \tag{1.9}$$

This is indicated in Fig. 1.12.

Since the UU-strength of a *saturated* soil is unaffected by the confining pressure, a UU-test may be performed in the unconfined state. This test is referred to as an unconfined compression test. In order that the unconfined compression test produces the same strength as would be obtained from a conventional UU-test, the soil must be:

1. Saturated
2. Intact
3. Homogeneous

Soils such as partly saturated clay (not saturated), stiff-fissured clays (not intact, fissures may open when unconfined), and varved clays (not homogeneous, cannot hold tension in pore water) do not fulfill these requirements

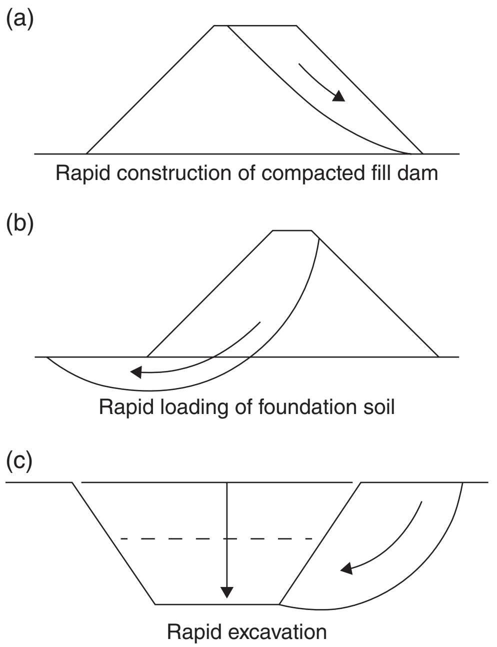


Figure 1.11 Examples of field cases for which short term stability may be determined on the basis of results of UU-tests.

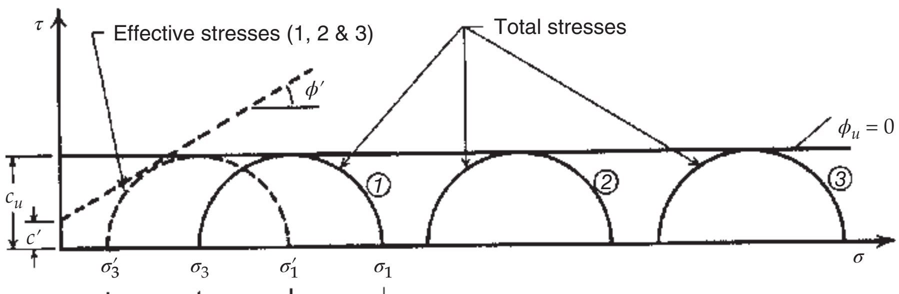


Figure 1.12 Schematic illustration of a Mohr diagram with results of UU-tests on saturated soil (after Bishop and Henkel 1962).

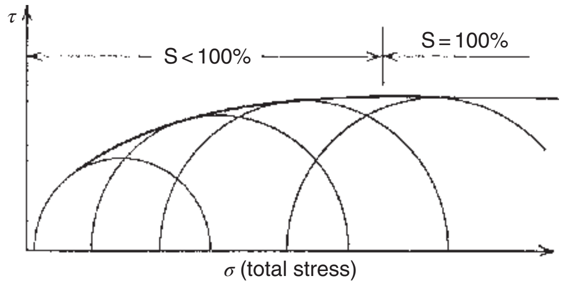


Figure 1.13 Schematic illustration of strength of partly saturated soil obtained from UU-tests.

and should not be tested in the unconfined compression test.

For those soils which qualify for and are tested in the unconfined compression test, the undrained *shear strength* is:

$$s_u = \frac{1}{2} \cdot q_u \tag{1.10}$$

in which q_u is the unconfined *compressive strength*:

$$q_u = (\sigma_1 - \sigma_3)_{\max} = \sigma_{1\max} \tag{1.11}$$

This is also indicated in Fig. 1.12.

For partly saturated soils the Mohr failure envelope is curved at low confining pressures, as seen in Fig. 1.13. As the air voids compress with increasing confinement, the envelope continues to become flatter. When all air is dissolved in the pore water, the specimen is completely saturated, and the envelope becomes horizontal. The undrained shear strength obtained at full saturation depends on the initial degree of saturation.

1.6.2 Selection of test type

The application of soil properties in analyses of actual geotechnical problems are outside the scope of the present treatment. However, it is important to know in which type of analysis the soil properties are to be used before any testing is initiated. Thus, different types of analyses (total stress or effective stress, short term or long term) may require results from different types of tests or results from different methods of interpretation of the results. In other words, the analysis that is appropriate for each particular field condition dictates the type of triaxial test to be performed.

Generally, soils that tend to contract will develop positive pore pressures during undrained shear resulting in lower shear strength than that obtained from the corresponding drained condition. Short term stability involving undrained conditions would be most critical for such soils. On the other hand, soils that tend to dilate will develop negative pore pressures during undrained shear resulting in higher shear strength than that obtained from the corresponding drained condition. Long term stability involving drained behavior would be most critical for these soils. Field conditions involving partial drainage should be analyzed for the most critical condition(s). For example, an earth dam usually undergoes several different stability analyses corresponding to different phases of construction and operating conditions. Some guidelines may be obtained from the examples given above.