4. Determination of shear strength of soils

Soils can be classified into two main goups based on the origin of shear strength: cohesive and cohesionless soils. Between the particles of cohesive soils there are inner cohesive forces, while the shear strength of cohesionless soils is due to the friction between particles and their clinging to each other.

While friction strength is interpreted only in presence of normal forces, there is no shear strength of cohesionless soils if the normal load is zero. By contrast, the cohesion of cohesive soils is the shear strength at zero normal load.

It is very important to emphasize that the basic rule is that the shear strength of granular soils originates only due to effective stress, in other words this shear strength is mobilized only by effective normal stress.

In cohesionless granular soils $c = 0 \frac{kN}{m^2}$, thus shear strength can be characterized by the inner friction angle. Slightly wet, partially saturated granular soils have some cohesion, too. This is created by the capillary effect of pore water, and ceases if soils become saturated or dry. Therefore this is called **pseudo (apparent) cohesion** and is ignored in calculations.

Shear tests

Shear tests are performed by constant normal stress, increasing shear stress until soil sample failure. In geotechnical practice, the most frequent method is the so-called **direct shear**, when the shear test is performed in a shear box, made of two half boxes moving away from each other (Figure 1.12 and Video 1.3).

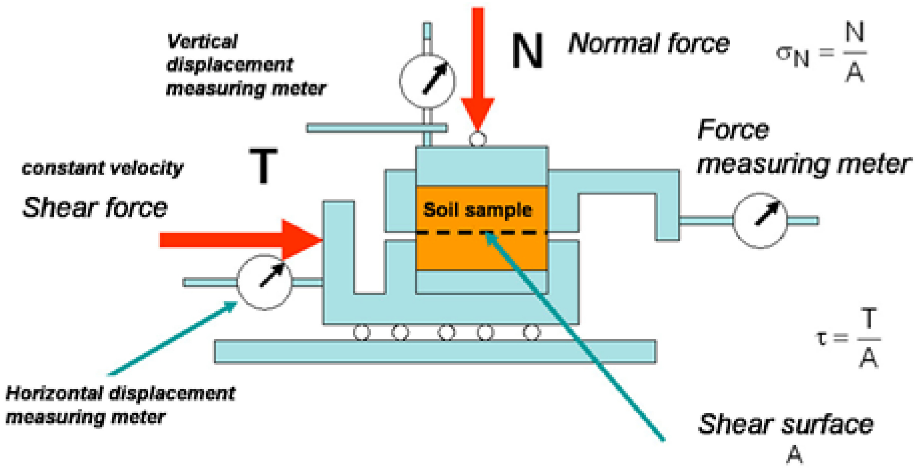


Figure 1.12: Principles of direct shear (Mecsi, 2009)

VIDEO 1.3

Video 1.3: Direct shear

In reality failures similar to direct shear are rare, but there are shear tests closer to the so-called **pure shear**. Pure shear is difficult to realize by simple experimental techniques, but there are equipment in geotechnical practice based on the principle of **simple shear**, where the smaller side wall effect results in more homogenic stress and deformation fields compared to direct shear equipment. Principles of simple shear are shown in Figure 1.13.

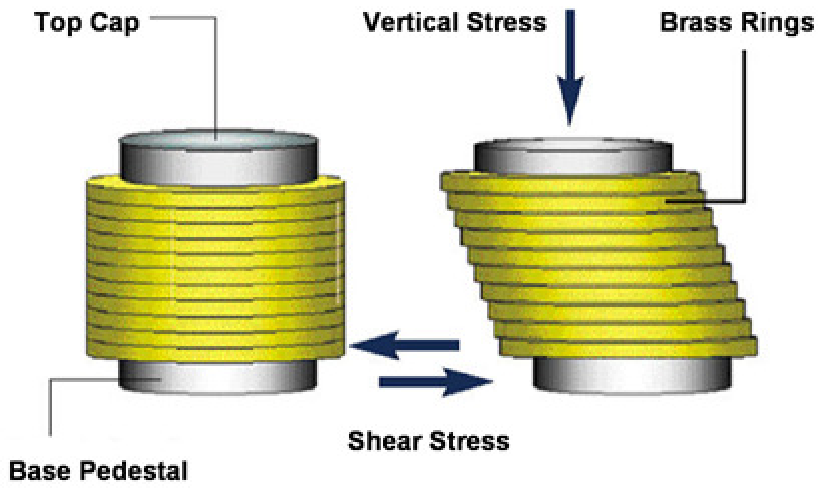


Figure 1.13: Principles of simple shear

As mobilized shear strength is a function of shear displacement, in certain cases the so-called residual shear strength must be determined as well (e.g. surface movements, soil-geosynthetics slip on each other). Residual shear strength can be determined using **torsion shear equipment**. Principles of **torsion shear** (or simply **torsion**) are shown in Figure 1.14.

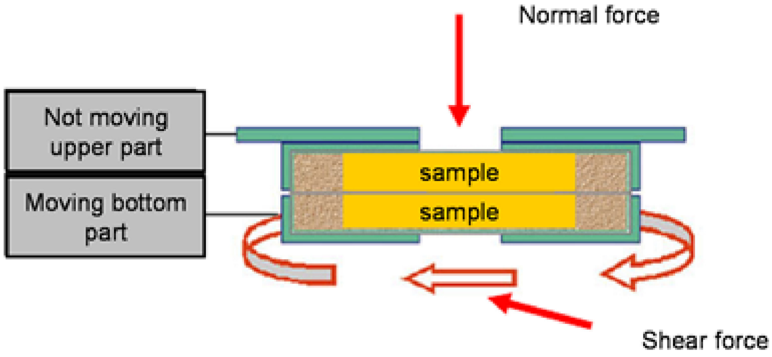


Figure 1.14: Principles of torsion shear

Processing of test results

Stresses are calculated by dividing the applied normal and shear forces with the surface of the sheared cross-sections: $\sigma = \frac{N}{A}; \quad \tau = \frac{T}{A}$

Curves of correlation between shear stresses and strains are depicted for all normal loads (Figure 1.15). From these curves we can read:

a) the proportional limit, namely the value of stress until displacements are proportional to shear stress;

b) the maximum shear stress;

c) and, if different from the maximum, the residual value of shear stress.

The Mohr-Coulomb failure lines can be drawn by depicting these values as a function of the applied normal stress. The incline of these lines gives:

a) the "proportional" friction angle;

b) the maximum of friction angle;

c) the residual value of friction angle (belonging to permanent slide).

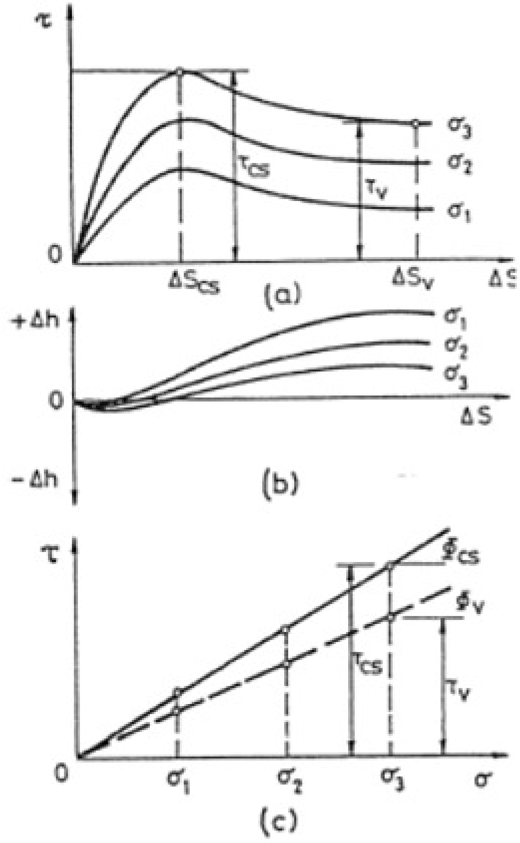


Figure 1.15: Processing of direct shear test results

Shear test results of compact and loose sands are shown in Figure 1.16.

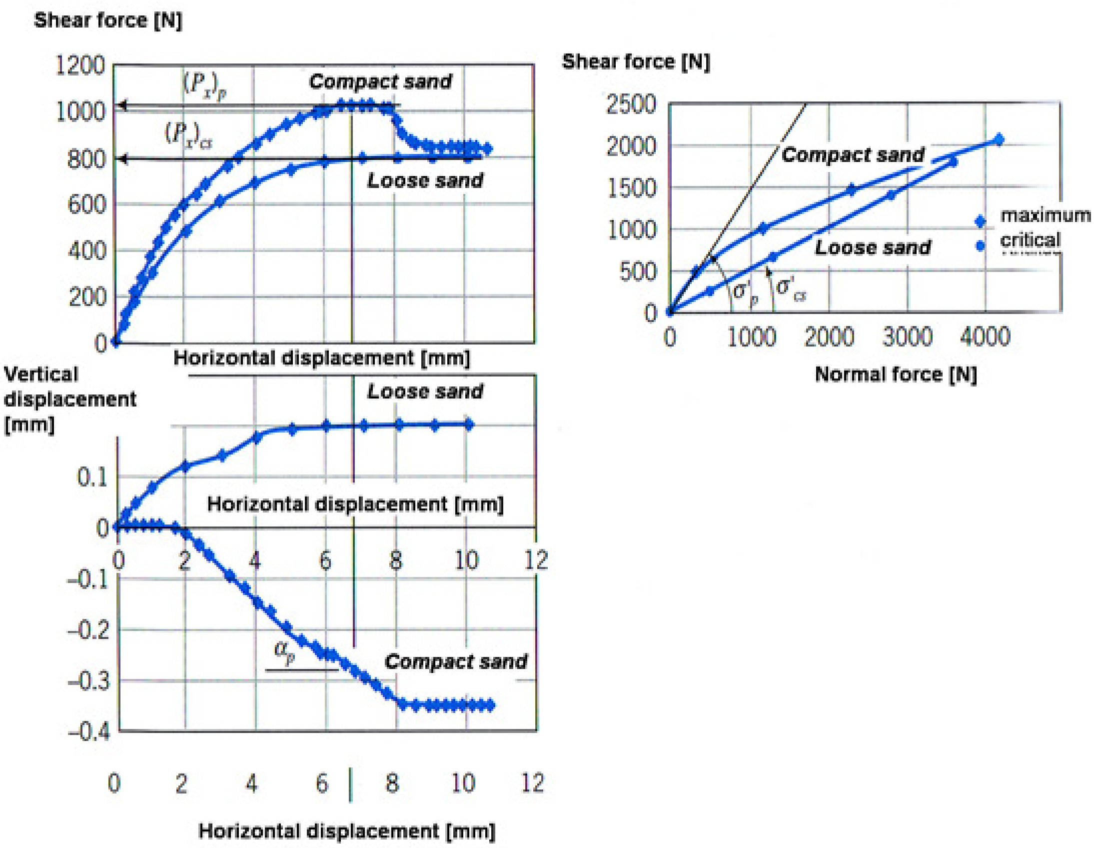


Figure 1.16: Results of shear tests of compact and loose sands

From direct shear tests, failure surface on the constraint surface between the two half boxes was only a hypothesis. Surfaces shown in Figure 1.17 and Animation 1.1 were proven by experience.

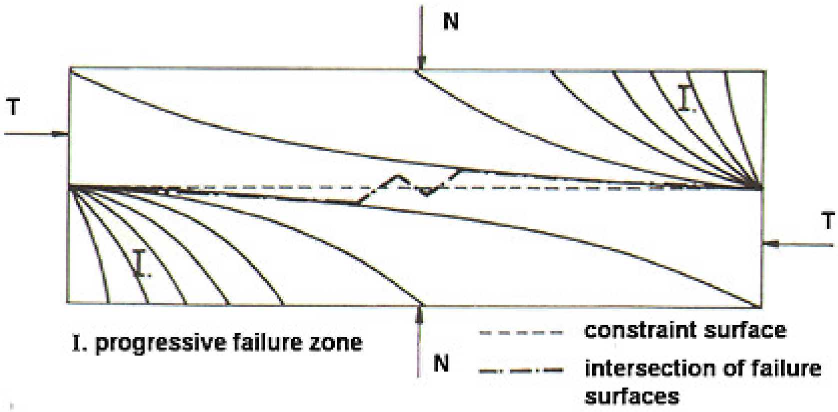


Figure 1.17: Shape of failure surfaces during direct shear

ANIMATION

Animation 1.1: Shape of failure surfaces during direct shear

Uniaxial compressive strength

This method is suitable only for the determination of the strength of cohesive soils. A cylindrical soil sample is loaded in axial direction between two compression plates, gradually increasing the load until failure. The vertical (Δh) and the horizontal (Δd) deformation of the sample is measured during loading. The method of load transfer can be fast or slow, continous or discontinous (step).

The results of the test are shown in Figure 1.18. The residual deformation after unloading is greater than the elastic deformation.

Most soils show this behaviour. The elasticity modulus calculated in this figure is not specific for soils, thus cannot be used for calculations of the deformation.

The compressive strength obtained as a final result from the test is only a comparative value between the same types of soils.

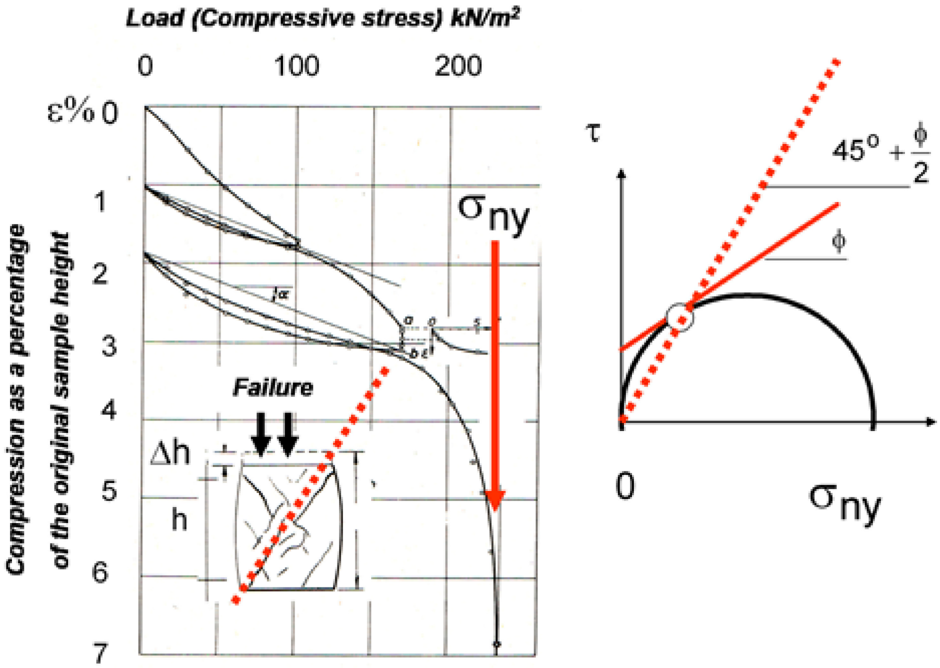


Figure 1.18: Results of the uniaxial compressive strength test

Triaxial compression test

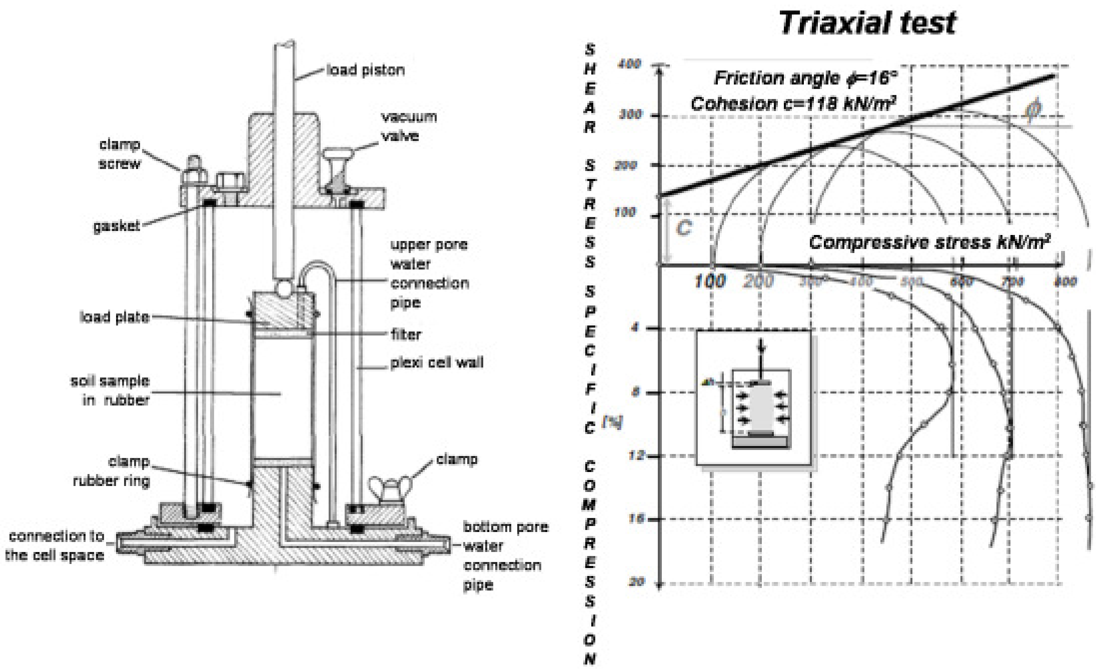
A cylindrical soil sample in a compression cell is loaded by hydrostatic pressure first, than loaded by vertical pressure until failure during the triaxial test.

The Mohr circle of the tension condition resulting in failure is drawn, and the test is performed repeatedly at different hydrostatic pressures. The cohesion and the inner friction angle of the soil can be read from the envelope of the Mohr circles of failure, the so-called Coulomb failure line.

The apparatus used for triaxial tests is shown in Figure 1.19 and Video 1.4. The main parts of the apparatus are the cylinder (made of glass or transparent plastic) and the well-sealed base and cover plates. The soil sample, surrounded by rubber, is placed in the liquid-filled (usually water-filled) space on the base plate sealed with rubber rings. There are filters below and above the sample, connected to capillaries.

These capillaries communicate through the bores of the base plate with the volume change measuring device and the pore water pressure measuring device.

The vertical load is transferred by the piston pressing on the top cap, and the loading force is measured. The measured displacement of the piston shows the vertical displacement of the sample. The side pressure can be controlled by a pressure regulator.



Figre 1.19: The tiaxial test and the processing of the results of the measurements

VIDEO 1.4

Video 1.4: The tiaxial test

Animation 1.2: The triaxial test (SOIL MECHANICS, A. Verruijt, TU Delft, 2001)

The drainage of water (assuming saturated condition) from the soil sample can be controlled by the shut-off and opening of a tap on the tube. The system can be closed or opened. This depends on the allowance of the consolidation of the sample or the development of the pore water pressure inside the sample. The test can be performed in three different ways:

a) Unconsolidated fast test (Unconsolidated Undrained (UU)): In an unconsolidated undrained test the sample is not allowed to drain. The sample is compressed at a constant rate (hydrostatic pressure) with the tap closed. The vertical stress is increased with the tap closed as well. The speed of compression is steady, 0.5-1.0% of the height of the sample per minute. The pore water pressure is measured during this test.

b) Consolidated fast test (Consolidated Undrained (CU)): In a consolidated undrained test the sample is not allowed to drain. The consolidation of the sample under hydrostatical pressure (with the tap opened) must take place before increasing the vertical stresses, than the vertical pressure is increased until failure with the tap closed. The pore water pressure is measured during this part of the test.

c) Slow (consolidated) test (Consolidated Drained (CD)): the full consolidation under hydrostatical pressure the vertical load is increased gradually, and the consolidation of the sample is allowed to take place in every load step. During both the hydrostatical and the vertical load steps the pore water pressure is measured. The consolidation can be considered completed when the pore water pressure measuring device shows zero. This usually takes a long time, thus test can be run for days.

Tests of unsaturated samples can be performed without the measurement of the pore water pressure. In this case, the results of the tests can be graphed only as a function of the total stresses. The result of this is also the apparent friction angle.

Figure 1.20 shows the determination of the shear strength as a function of the total and the effective stresses.

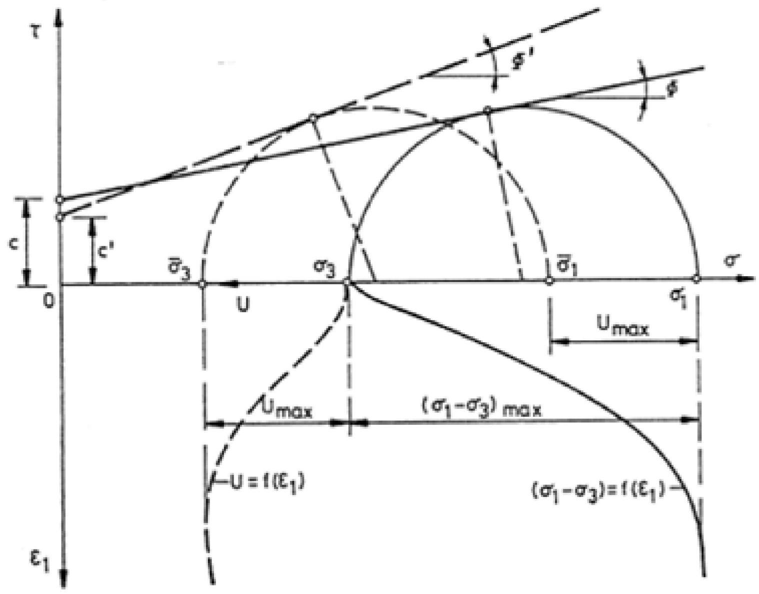


Figure 1.20: The determination of the Coulomb failure line as a function of the total and the effective stresses

ANIMATION

Animation 1.3: Drained triaxial test (SOIL MECHANICS, A. Verruijt, TU Delft, 2001)

ANIMATION

Animation 1.4: Undrained triaxial test (SOIL MECHANICS, A. Verruijt, TU Delft, 2001)

Knowing the value of total stress and pore water pressure (both measured during the test) the Mohr circles of the effective principal stresses and the Coulomb failure line can be drawn as a function of the effective stresses. It is obvious that the diameter of the Mohr circle of the principal stresses is the same in the two cases, but their location is different, so it follows that the shear strength parameters determined as a function of the total and the effective stresses will not be the same ($\phi' > \phi$; $c' < c$), and thus the Coulomb failure line will be a function of the method of the test (UU, CU or CD).

Triaxial cells can be used – besides for shear strength tests - to determine the hydraulic conductivity of low permeability soils in environmental geotechnics. The main advantage of these tests is the possibility of preventing the side wall leakage, which is the main source of error in the determination of the hydraulic conductivity of low permeability soils (e.g. clays, liner layers). Figure 1.21 shows the determination of the hydraulic conductivitiy of a clay sample.

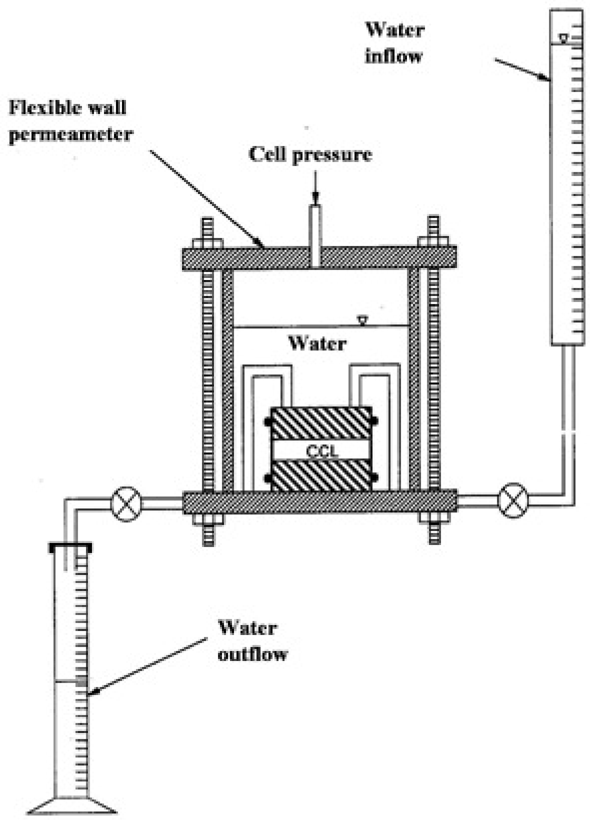


Figure 1.21: The determination of the hydraulic conductivity of clays in a triaxial cell

Problems of shear strength tests of soils

The shear strength of granular soils is a relatively simple problem. Granular soils behave almost always as "open systems" (depending on the shear speed) and their failure can be examined by the analysis of the effective stresses. Their inner friction angle is $\phi' = 28-42°$ and depends on the:

- Grain size: $\phi' = 38-40°$ of gravels, and $\phi' = 30-34°$ of sands is typical;
- The continuity of grain size distribution: ϕ' can change by $6°$ depending on the c_U (inequality coefficient);
- The compactness: the difference between the ϕ' of the loosest and the most compact soils can be $6°$;
- The roughness of the surface of the soil particles: this can result in about $3°$ difference in ϕ'.

The shear strength of dense sands can decrease because of the dilatation after the large shear deformations and can fall back to the residual value (see also Figure 1.16). The decrease of the inner friction angle can be characterized by the dilatation angle, which can be as much as $6°$.

The cohesion of granular soils is zero both in saturated and in dry condition, but in unsaturated condition they can have a cohesion of $5\,kPa$ because of the capillary effect. This cohesion can be ignored by calculations, because saturation or drying-up eliminates it.

The shear strength of cohesive soils is a more difficult problem. The test results are affected by several factors:

- They behave as an opened or closed system depending on the speed of load;
- The shear strength depends on the load speed because of the viscous properties;
- They tend to creep because of the same reason, and the creeping can lead to failure;
- The preload can increase shear resistance;
- The strength of preloaded clays can decrease, as well as the strength of compact sands.

The literature gives many measurement results, and there are empirical functions to describe certain effects (Szepesházi, 2008).

A tananyag kifejlesztése a TÁMOP 4.1.2-08/1/A-2009-0033 pályázat keretében valósult meg.