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On the term cohesion and its several meanings in geotechnics

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Review Article

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Cohesion intercept
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Viscous resistance

Abstract

Cohesion was defined by Coulomb in the eighteenth century as the strength of a prismatic solid body subjected to a direct tensile test. However, in geotechnics, different meanings have been ascribed to the term cohesion causing conceptual confusion. To distinguish the several meanings assigned to the term cohesion, it is common to find it followed by a complementary term to indicate its meaning. Examples include true cohesion, apparent cohesion, cohesion intercept and Hvorslev's true cohesion. Each of these terms is defined, distinguishing them by their physical interpretation. For this, it was necessary to visit some historical articles from the eighteenth and nineteenth centuries, as well as some articles from the 1930s and 1940s. In addition to the historical aspects presented, the interpretation of rock weathering as a loss of Coulomb's cohesion and the interpretation of "Hvorslev's true cohesion" as being of a viscous nature are discussed.

1. Introduction

There are effects related to the shear strength of saturated clays that are not covered by the principle of effective stresses as established by Terzaghi (1936) and are not taken into account by the existing failure criteria. Among these effects, called by Lambe & Whitman (1979, p. 314) as "complications", the strain rate effect on the undrained resistance of clays stands out. According to Bjerrum (1973), there are experimental evidences showing that the so-called strain rate effects are associated with the cohesive component of the shear resistance of clayey soils, as defined by Hvorslev (1960). However, as discussed by Schofield (1999, 2001), soils do not have cohesion in the sense used by Coulomb (1773), who was the first to define the term cohesion. Thus, how to deal with this dilemma? Do soils have cohesion or not? Moreover, is Hvorslev's cohesion, to which Bjerrum (1973) refers, identical to the Coulomb's cohesion to which Schofield (1999, 2001) refers? After all, what is the meaning of the term cohesion in geotechnics? Is there more than one meaning for it? According to the way the term has been used, the answer is yes.

In geotechnics the term cohesion has been used with different meanings, which brings conceptual confusion. In an attempt to distinguish the different meanings of the term cohesion as it appears in the technical literature, it is usual to see a complementary term associated with it. Thus, the composed terms true cohesion, apparent cohesion, cohesion intercept and Hvorslev' true cohesion usually appear. The purpose of this article is to discuss and elucidate the different meanings the term cohesion may have.

The confusion is aggravated by the use of the term cohesive soils as a synonym for clayey soils, thereby leading geotechnical engineers to the idea that such soils are "sticky". The other great group would comprise the so-called granular soils or cohesionless soils generally showing a coarse grain size, which are often seen to the naked eye. Perhaps, for this reason, the name granular. But the name granular soils is also inappropriate since clayey soils are also made up of grains (although finer and not visible to the naked eye). To overcome an inappropriate terminology resulting in conceptual confusion, replacement of the term cohesive soils by plastic soils as well as the term granular soils by non-plastic soils would be enough. Although it is not just a matter of terminology, but a conceptual aspect, this subject will not be treated here. For shortness, the authors will give here a brief and practical definition of what they mean by a plastic soil: a plastic soil might be defined as one on which liquid and plastic limits tests may be carried out, its water content being between the liquid and plastic limits.

Before explaining the different meanings of the term cohesion as used in geotechnics, it is convenient to know the origin of the term cohesion as well as some historical aspects related to the failure of solid bodies.

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2. Friction and cohesion

2.1 A brief history

A brief and interesting history of the development of material mechanics may be found in Westergaard (1952). This section will be exclusively referred to Coulomb's (1773) and Mohr's (1882, 1900) works and will show how they relate to each other concerning the strength of solid bodies.

Among the contributions to material mechanics in the 18th century, those made by Charles Augustin de Coulomb (1736-1806) stand out. The volume corresponding to the year of 1773 (published in 1776) and concerning "*Mémoires de Mathématiques et de Physique, Présentés à l'Academie Royale des Sciences, par divers Savans, et lûs dans ses Assemblées*", presents from pages 343 to 382 the article "*Essai sur une application des règles de maximis et minimis à quelques problems de statique, relatifs, à l'architecture*". In the article, problems regarding the strength of solid bodies subjected to tensile and unconfined compression tests are studied. Bending of a cantilever beam and earth pressure on a retaining wall are also studied, but these subjects are outside the scope of this article.

Coulomb (1773) carried out direct tensile tests and unconfined compression tests on rock and masonry specimens and identified two modes of failure: by separation and by shear. In the separation failure mode, as observed in direct tensile tests, failure occurs along a plane perpendicular to the direction of the tensile force and the solid body is separated into two parts (see Figure 1a). In the shear failure mode, as observed in unconfined compression tests, failure occurs along an inclined plane with respect to the direction of the compression force. In the shear failure mode, one part of the solid body slides over the other along an inclined plane of θ, where failure occurs (see Figure 1b). To quantify the results obtained from a theoretical point of view and compare them to the experimental results, Coulomb (1773) used the concepts of cohesion and frictional resistance.

By studying compression of non-slender columns, where failure occurs by shear, Coulomb (1773) theoretically determined the axial compression force that causes failure and the angle θ the failure plane makes with the horizontal.

Other major contributions to material mechanics, directly linked to Coulomb's work, were those of the German engineer Christian Otto Mohr (1835-1918). In an article published in 1882, Mohr presented a graphical representation of a three-dimensional state of stress in the form of circles, the Mohr's circles. These circles are shown in Figure 2, where σ_1, σ_2 and σ_3 are, respectively, the major, the intermediate and the minor principal stresses, which define the state of stress in the neighborhood of a point P in a solid body. For instance, in Figure 2, the circle defined by σ_1 and σ_3 is the locus of the points (σ,τ) whose coordinates are respectively

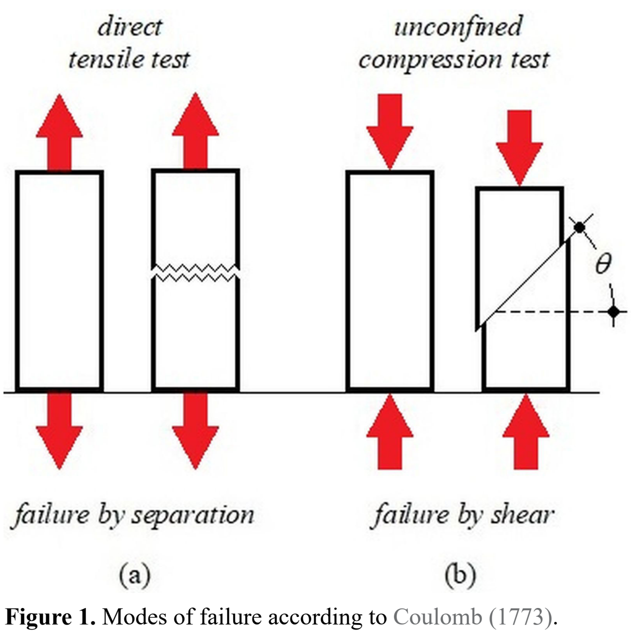

Figure 1. Modes of failure according to Coulomb (1773).

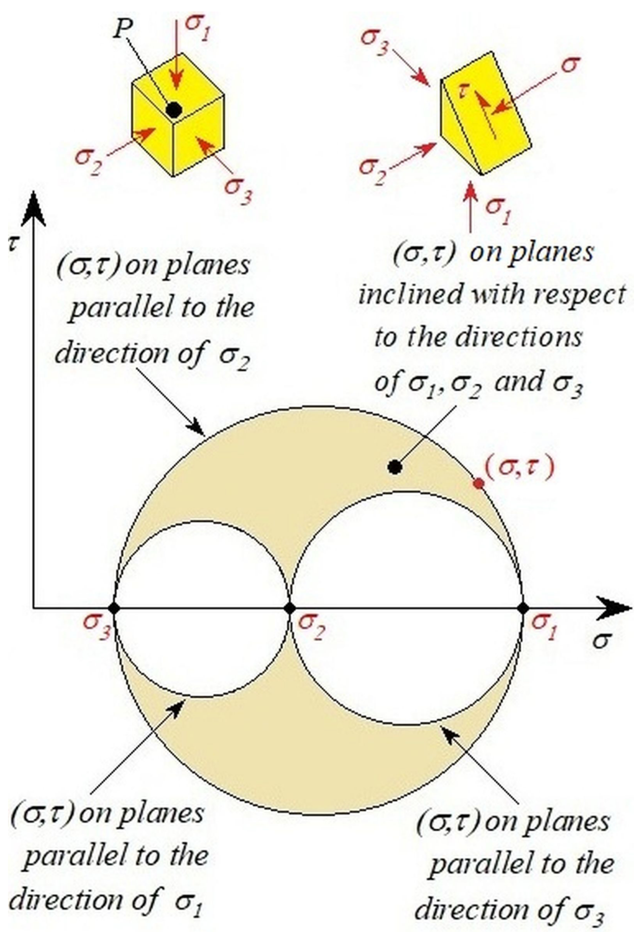

Figure 2. Mohr's circles representing a three-dimensional state of stress.

the normal and shear stresses in the neighborhood of point P acting on planes parallel to the direction of σ_2.

In 1900, based on several experimental results, Mohr published an article on elastic limit and strength of materials. He assumed as a working hypothesis that these strength limits could be determined by means of a relationship between shear stress (τ) and normal stress (σ) acting on an imaginary plane passing through a test specimen under a known state of stress. This means that, so that a state of stress does not cause failure, no point of coordinates (σ,τ) of the Mohr's circle representing the state of stress of the specimen being tested, plotted on a τ versus σ plane, should be above a limiting curve.

When plotting this limiting curve, with points (σ,τ) representing failure, it is necessary to consider that shear stress (τ) may be positive or negative. As from a physical point of view, the sign of the shear stress does not matter for the failure criterion, the limiting curve must be symmetrical with respect to the σ axis of the τ versus σ plane (Figure 3).

Mohr's circles representing possible states of stress for a given material must lie in a region of the τ versus σ plane bounded by the limiting curve for that material. Thus, Mohr's circles can at most touch the limiting curve but cannot intersect it. Hence, the name Mohr's failure envelope or Mohr's strength envelope (Figure 3).

If Mohr's hypothesis is correct, only major and minor principal stresses, which define the largest circle in a three-dimensional state of stress (see Figure 2), have influence on the definition of Mohr's failure envelope of a given material. The intermediate principal stress has no influence.

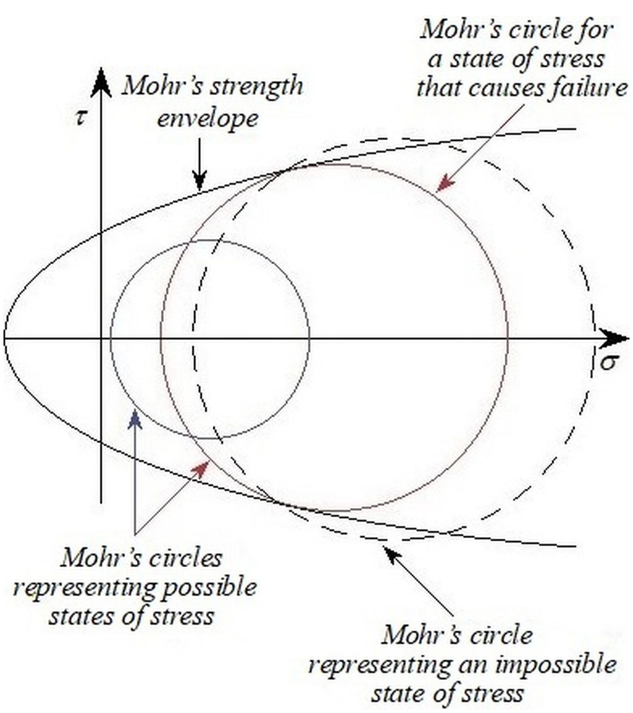

Figure 3. Mohr's circles representing possible and impossible states of stress and the limiting curve for a given material (Mohr's strength envelope).

2.2 Cohesion, friction, Coulomb's experiments, Coulomb's law and a little more history

Coulomb (1773, p. 347) described cohesion and friction resistance as reactive forces. His conception of cohesion and friction can be found in the following excerpt:

> *"Le frottement & la cohésion ne sont point des forces actives comme la gravité [...] mais seulement des forces coërcitives; l'on estime ces deux forces par les limites de leur résistance."*

Coulomb (1773, p. 347) then explained how to quantify frictional resistance, revealing an important historical fact, namely: the assignment of the law of friction not to himself but to Amontons, as reproduced below:

> *"Je supposerai ici que la résistance dûe au frottement est proportionelle à la pression, comme l'a trouvé M. Amontons."*

Here the expression "*la pression*" should be understood as the normal force applied to the solid body or, more rigorously, as the normal reaction that makes friction resistance available. Thus, friction law takes the form it is known nowadays: friction resistance between two solid bodies in contact is directly proportional to the normal reaction established between them.

Coulomb (1773, p. 348) then defined what he understood by cohesion:

> *"La cohésion se mesure par la résistance que les corps solides opposent à la desunion directe de leurs parties."*

After defining cohesion and friction resistance, Coulomb (1773) described the direct tensile tests he carried out on prismatic specimens of a rock from the Bordeaux region. He observed that in a tensile test the failure plane is perpendicular to the direction of the tensile force (failure by separation) and that the magnitude of the tensile force at failure (in case 430 lb ≅ 1.92 kN) is obtained by the product of cohesion (c) and the cross-sectional area of the specimen. Therefore, the quantity cohesion has the dimension of a stress.

Coulomb (1773) also carried out shear tests by measuring the force required to shear cantilever beams of the same rock used in the direct tensile tests, with the same cross-section, which were built-in at one end, with the shear force acting transversely to the beams' axis and neighboring their built-in end. In these tests, failure load was 440 lb (≅ 1.96 kN). Both tensile and shear tests were repeated several times, always providing slightly higher values of failure force in shear tests than in direct tensile tests. However, as the difference was small, Coulomb (1773) assumed the strengths as being equal in both tests, i.e., the same cohesion (c) value determined in the tensile test was ascribed to the strength measured in the shear tests. These concepts of cohesion and frictional resistance were adopted in the mechanism conceived by Coulomb to predict the failure load of non-slender columns subjected to axial compression forces, as explained below.

Let there be a homogeneous masonry column, with a square cross section of side a, subjected to a centered vertical load P (see Figure 4). Neglecting the column self-weight and assuming that failure takes place along the plane $ABCD$, with slope θ, mobilizing cohesion and friction resistances, what is the value of load P that causes the column to fail? And what is the value of the slope θ ?

Coulomb solved this problem by decomposing the force P into two components: $P \sin\theta$, acting along the failure plane $ABCD$ and $P \cos\theta$, normal to the same plane. To resist the $P \sin\theta$ component, the material has cohesion and friction strengths available. Cohesion strength is given by the product of cohesion (c) and the area of section $ABCD$ in Figure 4, equal to ($ca^2/\cos\theta$). Friction resistance is proportional to the normal force, having been denoted by Coulomb (1773) as ($P \cos\theta$)/ n, where ($1/n$) is the friction coefficient.

The balance of forces along plane $ABCD$ in Figure 4 when failure is imminent yields:

$$\frac{c\,a^2}{\cos\theta}+\frac{P\cos\theta}{n}=P\sin\theta \tag{1}$$

Determining the value of P from Equation 1, gives:

$$P=\frac{c\,a^2}{\left(\sin\theta\cos\theta-\dfrac{\cos^2\theta}{n}\right)} \tag{2}$$

The value of the slope θ of the failure plane $ABCD$ is the one that gives the maximum load P to which the column can resist. Mathematically, θ is determined making $dP/d\theta=0$. This procedure yields:

$$\frac{dP}{d\theta}=ca^2\frac{[\sin^2\theta-\cos^2\theta-\left(2/n\right)\cos\theta\sin\theta]}{\left[\cos\theta\sin\theta-\cos^2\theta/n)\right]^2}=0 \tag{3}$$

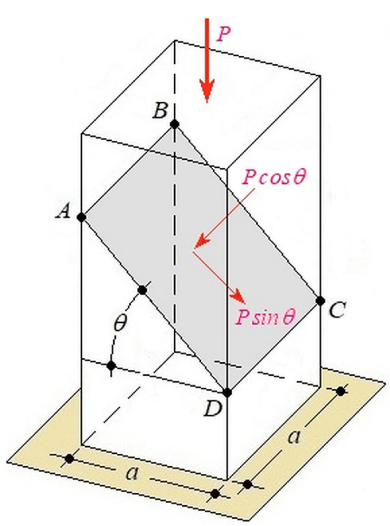

Figure 4. Maximum load P to be supported by a masonry column (Coulomb,1773).

with solution

$$\tan\theta=\frac{1+\sqrt{1+n^2}}{n} \tag{4}$$

The problem is then solved with the failure occurring by shear, along the inclined plane $ABCD$, having the slope θ given by expression 4.

But what does Equation 1 physically mean? Dividing both sides of Equation 1 by ($a^2/\cos\theta$), which is the area of the section $ABCD$ in Figure 4, yields:

$$c+\frac{1}{n}\frac{P\cos\theta}{\dfrac{a^2}{\cos\theta}}=\frac{P\sin\theta}{\dfrac{a^2}{\cos\theta}} \tag{5}$$

In Equation 5 c is cohesion. Remembering that ($1/n$) is the friction coefficient, the second term of the left-hand side of Equation 5 is the product of the friction coefficient by the normal stress on plane $ABCD$ and the right-hand side is the shear stress on plane $ABCD$.

As the friction coefficient can be expressed in the form of the tangent of an angle, the friction angle ϕ (see for example Taylor, 1948, p. 311-313), thus $(1/n)=\tan\phi$. Considering that Equation 5 refers to the failure plane at failure, the term $P \cos^2\theta / a^2$ is the normal stress on the failure plane at failure, denoted by σ_{ff}. Similarly, $P \sin\theta \cos\theta / a$ is the shear stress on the failure plane at failure, denoted by τ_{ff}. Therefore, Equation 5 can be rewritten as:

$$\tau_{ff}=c+\sigma_{ff}\tan\phi \tag{6}$$

Equations 1 and 6 are two different ways of writing what came to be known as Coulomb's law.

2.3 Mohr's failure criterion and the Mohr-Coulomb failure envelope

Mohr's failure criterion can be stated as follows: be a solid body of a given material subjected to a state of stress. For each material there is a relationship between shear stress τ and normal stress σ, expressed by a function f, so that, if an ordered pair (σ,τ), which satisfies the function $\tau=f(\sigma)$, is acting at a point P of this solid body, on a given plane that passes through P, then there will be failure of this solid body, at point P, along the referred plane. In this case, the normal and shear stresses acting on the failure plane, at failure, are denoted respectively by σ_{ff} and τ_{ff} and are related to each other by $\tau_{ff}=f(\sigma_{ff})$. Hereinafter, to indicate that Mohr's failure criterion applies to a failure plane, at failure, the notation $\tau_{ff}=f(\sigma_{ff})$ will be used.

Plotted on a τ versus σ plane, the function $\tau_{ff}=f(\sigma_{ff})$ produces a curve, which is the Mohr's failure envelope.

Since the Mohr's envelope is symmetric with respect to the σ axis, it is usual to draw only its upper part (see Figure 5).

When the function $\tau_{ff} = f(\sigma_{ff})$ has the form of Equation 6, the Mohr envelope takes the shape of a straight line. In these cases, the envelope is called the Mohr-Coulomb envelope because it combines Mohr's failure criterion and Coulomb's law. In the case of Equation 6, c and $tan\ \phi$ are, respectively, the cohesion and the friction coefficient. In materials whose failure is governed by the Mohr-Coulomb envelope, c and $tan\ \phi$ are both constants and material properties to be determined experimentally.

In the case of porous solid materials, such as rocks, when the pores are filled with water under pressure u, its influence on the strength of rocks is taken into account by the effective stress, denoted by σ' and defined by $\sigma' = \sigma - u$. The effect of pore pressure is simply to displace Mohr's circles of stresses by the value of pore pressure (u) producing the Mohr's circles of effective stresses (see for instance Obert & Duvall, 1967, p. 314-315).

Since for many rock types the strength envelope, written in terms of effective stresses, is approximately a straight line (see Obert & Duvall, 1967, p. 286), their shear strength can be expressed as:

$$\tau_{ff} = c' + \sigma'_{ff}\, tan\,\phi' \tag{7}$$

where τ_{ff} and σ'_{ff} are, respectively, the shear and the normal effective stresses on the failure plane at failure and c' and ϕ' the cohesion and the friction angle written in terms of effective stresses. Plotting Equation 7 on a τ versus σ' plane, a Mohr-Coulomb envelope is obtained in terms of effective stresses, as shown in Figure 6. In order to simplify the topic discussed here, from now on, in this section, pore pressures will be considered equal to zero.

Solid materials, such as concrete and rocks, have a strength that can be described as a consequence of an "imaginary pressure", called "intrinsic pressure" (Taylor, 1948), denoted by σ'_i, which remains after the formation of these materials, and providing them with a tensile strength.

It is the "intrinsic pressure", corresponding to the segment $O'O_p$ in Figure 6, that gives rise to the Coulomb cohesion. This occurs, for example, in the case of an igneous rock after its formation by cooling magma. This also occurs in the formation of a sedimentary rock, such as sandstone, when a cementing agent, through the process of diagenesis, gradually cements the grains of a sedimentary soil. A similar phenomenon occurs with concrete when the cement connects the aggregates. Cementation provides such materials with a tensile strength, denoted by σ'_t in Figure 6. This type of strength can be also quantified by the segment O_pC, as also shown in Figure 6. Such strength, denoted by c', is called, in geotechnics, true cohesion, although Coulomb originally gave the name cohesion to direct tensile strength (σ'_t).

The Mohr-Coulomb envelope in Figure 6 presents an inconsistency with the observed experimental results. The

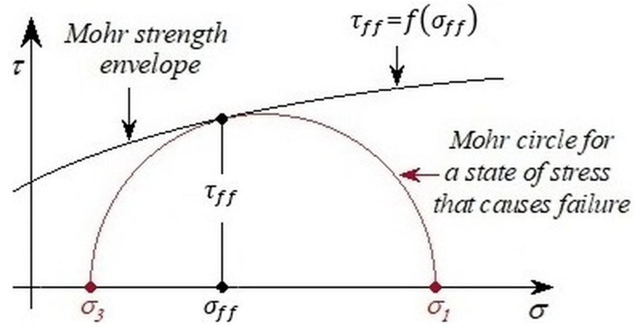

Figure 5. Mohr strength (or Mohr failure) envelope.

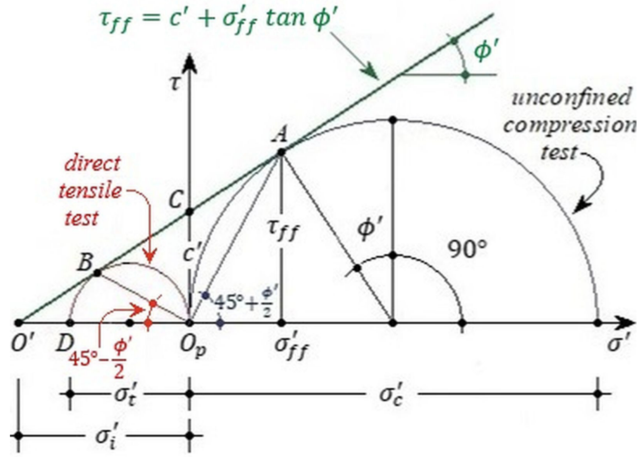

Figure 6. Mohr-Coulomb envelope for rocks.

inconsistency can be demonstrated to exist making use of the concept of pole of a Mohr circle. In Figure 6, the pole of the two Mohr circles is the point O_p. The point corresponding to failure on the Mohr circle of the unconfined compression test is point A, whose coordinates are (σ'_{ff}, τ_{ff}). By joining O_p to point A by a straight line, the direction of the failure plane in an unconfined compression test is obtained, with a slope of ($45+\phi'/2$). To determine the failure plane in the direct tensile test, one should connect point O_p to point B in Figure 6. This would result in a failure plane whose slope would be ($45-\phi'/2$). However, in the direct tensile test, failure occurs by separation, with the failure plane being orthogonal to the direction of the tensile stress (see Figure 1a). In this case, the failure plane is horizontal, and the failure envelope must necessarily touch the Mohr circle of the tensile test at point D, where failure by separation takes place. Furthermore, if the tensile strength is given by σ'_t, the strength envelope cannot have points to the left of the vertical through point D. Therefore, the failure envelope must have, in the tensile region, a curved shape, being orthogonal to the σ' axis at point D, as shown in Figure 7. Thus, the magnitude of the true cohesion c' is closely related to the tensile strength σ'_t, i.e., the greater the tensile strength σ'_t, the greater the true cohesion c'.

Table 1. Direct shear test data on dense air-dried Ipanema Beach Sand (Simões, 2015).

Initial void ratio	Displacement at failure (mm)	σ'_{ff} (kPa)	τ_{ff} (kPa)	τ_{ff}/σ'_{ff}	ϕ'^{o}	τ_{ff} given by Equation 10 (kPa)
0.493	0.75	25	23.8	0.95	43.5	---
0.496	0.55	50	42.5	0.85	40.5	---
0.495	0.80	75	63.8	0.85	40.3	---
0.494	0.82	100	82.0	0.82	39.3	---
0.491	0.75	150	122	0.81	39.0	123
0.491	0.70	300	237	0.79	38.3	236
0.490	0.95	500	395	0.79	38.2	387
0.493	1.00	750	578	0.77	37.7	575
0.488	1.20	1000	770	0.77	37.6	764
0.495	1.20	1250	975	0.78	37.9	---

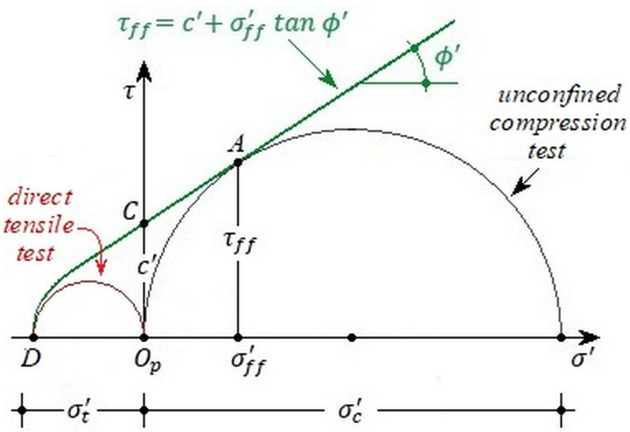

Figure 7. Curved strength envelope showing true cohesion *c'*.

2.4 Soils, rocks and true cohesion

Figure 7 is suitable for understanding the rock weathering process, transforming rocks into residual soils. In this case, true cohesion *c'* can be assigned to the remaining cementation of the material that has not been destroyed by the weathering process of the mother rock. Therefore, it can be said that the process of transforming rocks into residual soils occurs by the loss of true cohesion. As for the reverse process, called diagenesis, in which sedimentary soils undergo a lithification (cementation) process, there is a gain in true cohesion. The weathering and lithification processes are illustrated in Figure 8, in an idealized Mohr-Coulomb shear strength envelope.

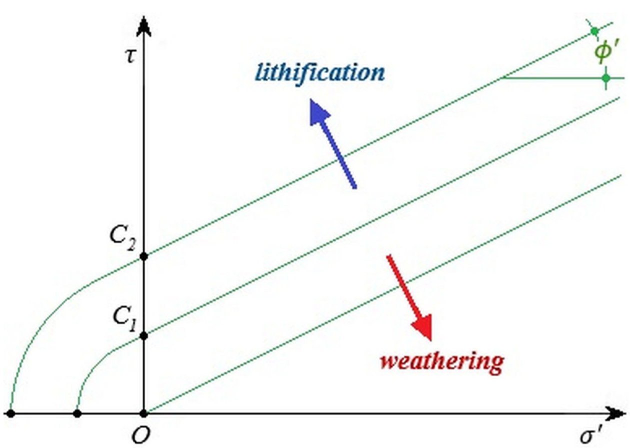

Figure 8. Weathering/lithification as a loss/gain in true cohesion (Martins, 2023).

The hypothesis that during weathering/lithification the failure envelope moves parallel to itself keeping ϕ' constant, as shown in Figure 8, is used here just to simplify the explanation of the phenomenon.

3. Effective cohesion intercept

According to Bolton (1979, p. 20):

> *"Soil to the engineer implies any uncemented and badly fitting accumulation of mineral fragments of various size, shape and composition with an appreciable percentage of interconnecting voids."*

According to this definition and what was discussed in Section 2.4., a material that has true cohesion (cementation) could not be called soil. However, assuming that the above definition is valid, without discussing whether this terminology is appropriate or not, it is concluded that, to be considered soil, meeting the given definition, an earthy material must have a failure envelope passing through the origin. This is the case of Ipanema Beach Sand, whose results of direct shear tests carried out on dense air-dried specimens with an average void ratio of 0.493, are shown in Table 1.

Figure 9 shows the strength envelope obtained with the data shown in Table 1. Although initially showing a straight-line shape, the strength envelope becomes curved downwards as the effective normal stress increases, a common feature in pure clean sands.

Increasing normal stress in direct shear tests on sands has similar effects as increasing confining stress in drained triaxial tests on sands (see Lee & Seed, 1967). Among these effects, the decrease in dilatancy stands out, with a consequent decrease in friction angle ϕ' (see Table 1) given by:

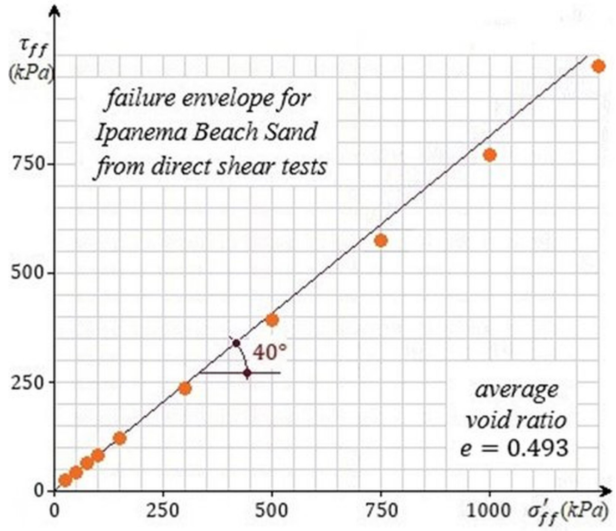

Figure 9. Strength envelope for dense air-dried Ipanema Beach Sand from direct shear tests (Simões, 2015).

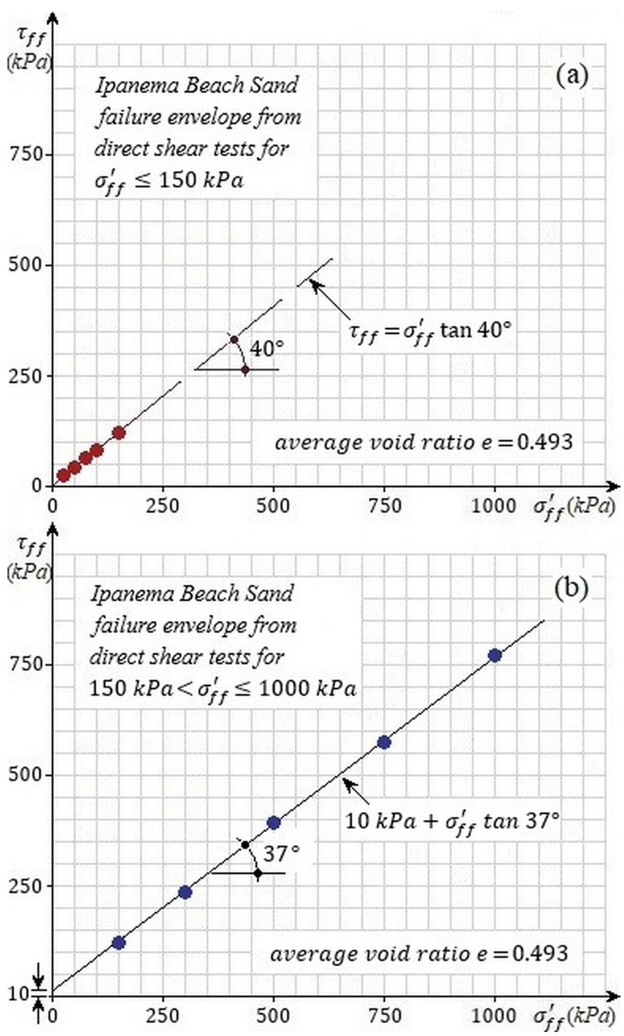

Figure 10. Strength envelopes for Ipanema Beach Sand: (a) For $\sigma'_{ff} \leq 150\,kPa$ and (b) For $150\,kPa < \sigma'_{ff} \leq 1000\,kPa$.

$$\phi' = arc\,tan\left(\frac{\tau_{ff}}{\sigma'_{ff}}\right) \tag{8}$$

Since Ipanema Beach Sand does not have cementation, $c' = 0$. Therefore, the Mohr strength envelope must pass through the origin. Moreover, if the failure envelope of Ipanema Beach Sand were of the Mohr-Coulomb type, it would be a straight line. This happens, however, only for $\sigma'_{ff} \leq 150$ kPa (see Figure 10 a) where the strength envelope can be represented by

$$\tau_{ff} = \sigma'_{ff} \tan 40° \tag{9}$$

As a consequence of the decrease in friction angle ϕ' with the increase in effective normal stress, the strength envelope becomes curved downwards departing from the straight line given by Equation 9. Therefore, for $\sigma'_{ff} > 150$ kPa, Equation 9 is no longer valid. As the use of curved envelopes is not common practice in geotechnics (except when more sophisticated softwares are used), it is usual to replace the curved section of the envelope by a straight line that best represents it in the stress range of interest (see Lambe & Whitman, 1979, p. 138-139).

To illustrate the problem, suppose that, in the case of the Ipanema Beach Sand envelope in Figure 9, one wants to determine the equation of a straight line that fits the curved strength envelope in the range 150 kPa < σ'_{ff} < 1000 kPa. To express the shear strength of that sand within the referred stress range, a straight line that fits the curved envelope in the desired range is (see Figure 10b):

$$\tau_{ff} = c' + \sigma'_{ff}\, tan\,\phi' = 10\,kPa + \sigma'_{ff}\, tan\,37° \tag{10}$$

In Equation 10, the parameter $c' = 10$ kPa does not have the physical meaning of cohesion in the sense of cementation since it is just the linear coefficient of the straight line fitted to the curved strength envelope in the range of effective stresses of interest (Pinto, 2006, p. 251). In this case, c' is called effective cohesion intercept.

The τ_{ff} values obtained by Equation 10 for 150 kPa < σ'_{ff} <1000 kPa are presented in the last column of Table 1 for comparative purposes with the measured values.

4. Apparent cohesion

4.1 Brief introduction

When a soil specimen is trimmed in the laboratory for a triaxial test, if the sample has true cohesion, the specimen remains standing. However, even if there is no true cohesion, a specimen can still remain standing. In this case, what keeps it standing, whether the soil is saturated or not, is a state of stress arising from capillary tension in the water. Under such a condition, the pressure in the pore water is lower than the atmospheric pressure. Such a negative pore water pressure, equal to the effective stress when the all-around total normal stresses are zero, gives soils a certain shear strength, sometimes called apparent cohesion. According to Taylor (1948):

> *"In some clays most of this pressure is lost in a short time if the specimen is placed under water, because submergence destroys the surface menisci, the sample swells, and the tension in the water soon is dissipated. A simple but common test to determine the degree to which this process occurs is called the slaking test, which consists simply in visually noting how fast and to what degree the sample slakes, or falls apart, when placed below water. The strength lost during a slaking test is apparent cohesion or strength due to capillary pressure"*(Taylor, 1948, p. 320).

The term apparent cohesion can be used for both saturated and unsaturated soils. Although in both cases the physical phenomenon which gives rise to the term apparent cohesion comes from the negative pore-water pressure, in the case of unsaturated soils the existence of air in the pores makes the phenomenon more complex.

In saturated soils one stress state variable, namely the effective stress, is enough to describe soil behavior. In the case of unsaturated soils, where the pore spaces are filled with water and air, two independent stress state variables are needed to describe the state of stress. These stress state variables are the net normal stress ($\sigma - u_a$) and the matric suction ($u_a - u_w$), where σ is the total normal stress, u_a is the pore-air pressure and u_w is the pore-water pressure. Thus, to define the state of stress in an unsaturated soil, two independent stress tensors are necessary (Fredlund & Rahardjo, 1993, p. 42-47). In view of the complex subject, which would require the presentation of extensive theoretical background, the apparent cohesion of unsaturated soils will not be addressed in this article.

4.2 Apparent cohesion of saturated clays in *UU* tests and the $\phi = 0$ condition

There is another condition under which the shear strength of a saturated clay can also be called apparent cohesion. Suppose a clay deposit where the water table (*W.T.*) coincides with the ground level (*G.L.*), as shown in Figure 11. Also suppose that a clay sample *A,* has been taken and immediately transported to the laboratory. The states of stresses of the sample in the field and in the laboratory are shown in Figure 11.

In the analysis presented herein, the effects of sampling operations on the sample, such as the drilling of the borehole and the insertion of the sampler, were not taken into account. For a deeper and more comprehensive study of this subject, the article by Hight (2003) is recommended. Only the effect of the change in the state of stresses caused by an idealized sampling will be analyzed here. This idealized process of removing a sample from the subsoil, keeping it intact and immediately transporting it to the laboratory will be called perfect sampling.

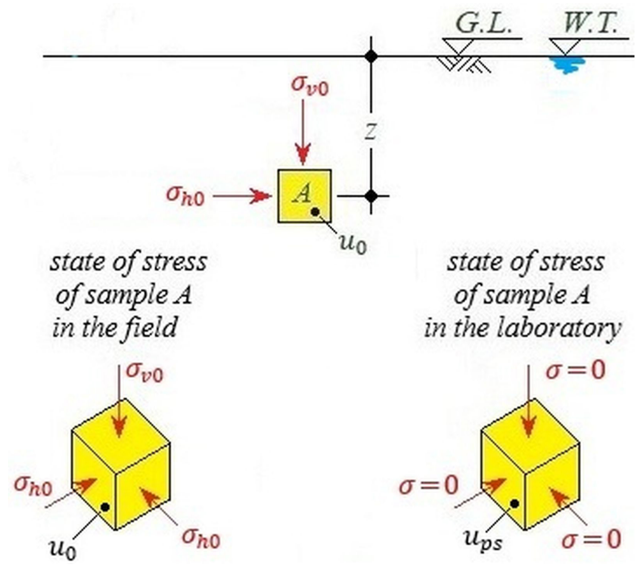

Figure 11. Perfect sampling of a saturated clay.

Considering that sample *A* is under a geostatic state of stress, the state of total stresses in sample *A* in the field can be defined by the total vertical stress σ_{v0} and the total horizontal stress σ_{h0} (see Figure 11). With K_0 being the coefficient of earth pressure at rest and u_0 the pore pressure in the field, the state of effective stresses of sample *A* in the field is given by the effective vertical stress $\sigma'_{v0} = \sigma_{v0} - u_0$ and the effective horizontal stress $\sigma'_{h0} = K_0\,\sigma'_{v0}$.

When the sample is removed from the subsoil and taken to the laboratory, the total stresses become zero in all directions. Denoting by u_{ps} and σ'_{ps}, respectively, the pore pressure and the effective stress after perfect sampling, it is concluded that $\sigma'_{ps} = -u_{ps}$, being $u_{ps} < 0$.

Assuming, for the sake of simplicity, that a saturated clay during perfect sampling does not undergo volume change and behaves as a linear-elastic and isotropic material, after perfect sampling the octahedral effective stress σ'_{oct} remains the same as that which existed in the field. As the octahedral effective stress in the field was $\sigma'_{oct} = \sigma'_{v0}\,(1+2K_0)/\,3$ and the octahedral effective stress after perfect sampling is $\sigma'_{ps} = -u_{ps}$, it is concluded that the effective state of stresses after perfect sampling is:

$$\sigma'_{ps} = \frac{\sigma'_{v0}\left(1+2K_0\right)}{3} = -u_{ps} \tag{11}$$

The effective stress in the sample after perfect sampling (σ'_{ps}), equal in magnitude to the negative pore pressure ($-u_{ps}$), explains why a saturated clay sample is able to remain standing, even though it does not have true cohesion.

It is now analyzed what happens during an unconsolidated-undrained *UU* triaxial test. A numerical example better elucidates the issue.

Suppose that three *UU* tests are to be carried out on a saturated sample. Suppose also that in the field, at the sample depth, $\sigma_{v0} = 90$ kPa, $u_0 = 60$ kPa and $K_0 = 0.7$. Since the vertical effective stress $\sigma'_{v0} = 30$ kPa, the octahedral effective stress of this sample, in the field, is equal to 24 kPa. After perfect sampling, the sample in the laboratory will be subjected to an effective stress $\sigma'_{ps} = 24$ kPa and thus the pore pressure $u_{ps} = -24$ kPa.

Suppose now that three specimens for *UU tests* are trimmed from the sample described above and subjected to confining stresses of 50 kPa, 100 kPa and 150 kPa. Applying a confining stress of 50 kPa, without drainage, to the first specimen, there is only variation in pore pressure. As the pore pressure in the sample was −24 kPa, after applying the confining stress of 50 kPa the pore pressure becomes $u_1 = -24 + 50 = 26$ kPa, which means that the effective stress remains at the value of 24 kPa.

The example above leads to the conclusion that in *UU* tests carried out on saturated soils, whatever the confining stress, the effective state of stresses does not change. Thus, in tests 2 and 3, tested with confining stresses of 100 kPa and 150 kPa, the pore pressures after the application of such confining stresses, will be, respectively, $u_2 = -24 + 100 = 76$ kPa and $u_3 = -24 + 150 = 126$ kPa. Therefore, the starting effective stress will remain, in all tests, at 24 kPa.

Since the starting effective states of stresses in the shearing stage are the same for the three tests, the three tests will present the same Mohr circle of effective stresses at failure. Notwithstanding, the Mohr circles of total stresses at failure, although having the same diameter, will be distinct, far from each other by a distance corresponding to the difference between their confining stresses, as shown in Figure 12.

At the time when the concept of effective stress was not yet known, the failure envelope was drawn from the Mohr circles of total stresses at failure. As the Mohr circles of total stresses at failure have the same diameter, the strength envelope plotted in terms of total stresses, is horizontal yielding the condition $\phi = 0$. This result led, in the past, to the false conclusion that saturated clays behaved as "purely cohesive" materials. This terminology precedes the concept of effective stresses introduced by Terzaghi (Pinto, 2006, p. 310) and, therefore, its use should be abolished to avoid conceptual confusion. The shear strength in this case, denoted by S_u in Figure 12, is also sometimes called "apparent cohesion" (Skempton, 1948, p. 193). This behavior, called undrained, means that the strength S_u, is more properly called undrained shear strength, although it is still often mistakenly treated by the name cohesion.

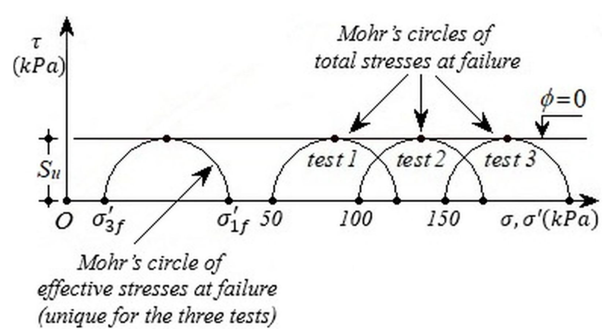

Figure 12. Typical results of *UU* triaxial tests on saturated clay.

5. Hvorslev's true cohesion

Figure 13 combines figures from Terzaghi (1938) and Gibson (1953) summarizing the results obtained by Hvorslev (1937) in a set of consolidated-drained direct shear tests carried out on a remolded saturated clay sample.

The curve $A_0B_0C_0D_0$ in Figure 13 represents the one-dimensional virgin compression followed by a rebound $D_0E_0F_0G_0H_0$, obtained from the consolidation phases that preceded the shear stages of the consolidated-drained direct shear tests. The normally consolidated specimens represented by A_0, B_0, C_0 and D_0 decreased in volume during shearing and failed respectively with the void ratios of points *A*, *B*, *C* and *D*. The bottom part of Figure 13 shows the normally consolidated failure envelope *OABCD*, which is a straight

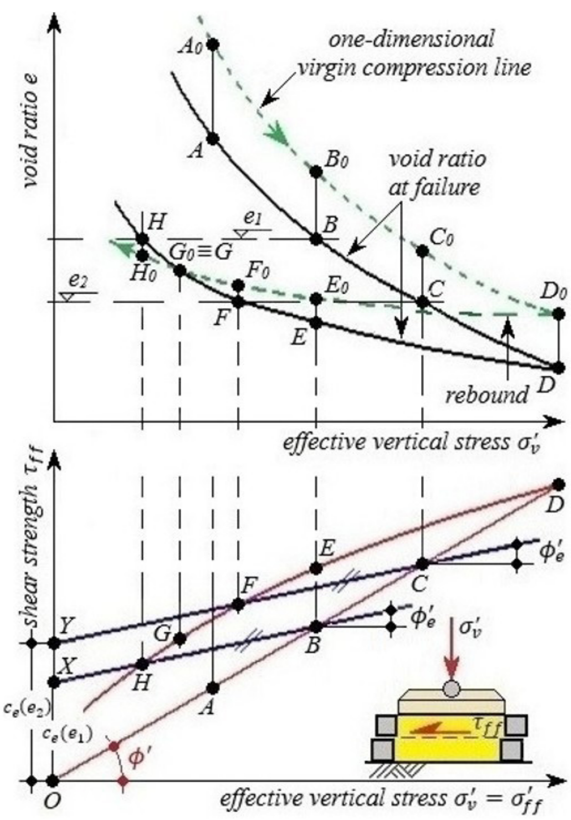

Figure 13. Definition of Hvorslev's true cohesion (c_e) and true friction angle (ϕ'_e).

line, has a slope ϕ' and passes through the origin. This means that, in the normally consolidated condition, the tested clay does not exhibit true cohesion in the physical sense as defined by Coulomb (1773). As a matter of fact, in a remolded sample, there can be no cementation between the grains. Even if some cementation had originally existed, it would have been destroyed during remolding.

For the sake of simplicity, it is assumed in Figure 13 that during the shearing phase of a consolidated-drained direct shear test, the effective vertical stress σ'_v, to which the soil consolidated, remains unchanged during shearing, i.e., the effective vertical stress at failure (σ'_{ff}) is the same as the effective vertical stress at the end of consolidation (σ'_v).

The shear strength (τ_{ff}) along the failure envelope *OABCD* can be expressed as an exclusive function of σ'_{ff}, that is, $\tau_{ff} = \sigma'_{ff}\ tan\phi'$. However, there is another variable embedded along *OABCD*, which is the void ratio at failure, i.e., along *OABCD* both effective vertical stress and void ratio vary.

Points E_0, F_0, G_0 and H_0 represent overconsolidated test specimens that provide, at failure, the strengths represented by points *E*, *F*, *G*, and *H*. The failure envelope corresponding to these overconsolidated specimens is the curve *DEFGH*, shown at the bottom of Figure 13. Along the curve *DEFGH* both effective vertical stress and void ratio also vary.

Hvorslev (1937, 1960) observed that one could draw another kind of failure envelope by connecting the points representing tests which presented the same void ratio at failure. The advantage of considering such an envelope is to eliminate the influence of void ratio on the envelope. Hvorslev (1937, 1960) realized that the equation of such an envelope could be written as:

$$\tau_{ff} = c_e\left(e\right) + \sigma'_{ff}\ tan\,\phi'_e \qquad (12)$$

In this type of failure envelope, which is a straight line, all specimens have the same void ratio at failure. An example of this type of envelope is the line *XHB* in Figure 13. Line *XHB* represents failure of specimens tested with different values of σ'_v which failed with the void ratio equal to e_1. Similarly, line *YFC* in Figure 13 represents specimens from the same clay which failed with a void ratio e_2 (with $e_2 < e_1$). The values of the linear coefficient of the straight lines *XHB* and *YFC* are respectively $OX = c_e\ (e_1)$ and $OY = c_e\ (e_2)$. Thus, it is possible to express the shear strength of a clay by Equation 12. Along this type of failure envelope proposed by Hvorslev (1937, 1960), the void ratio at failure is constant and only the stress σ'_{ff} varies.

In Equation 12, c_e is termed "effective cohesion" or "Hvorslev's true cohesion". The "Hvorslev's true cohesion" is a function of the void ratio (and of the structure of the clay), and ϕ'_e is defined as the "Hvorslev's true angle of friction".

A curious fact about Equation 12 is that it includes a value of "cohesion" associated to the overconsolidation of the sample, linked to the rebound of the compressibility curve. Here lies the question: would this overconsolidation generated by the simple mechanical action of unloading have the ability to create a cementation that would lead to a "cohesion", as defined by Coulomb (1773)? There are evidences indicating that it would not. The difference between the strengths of the test specimens represented, for example, by points *B* and *E* in Figure 13, is due solely to the difference in void ratios of the two test specimens.

Despite the algebraic similarity between Equations 12 and 7, the term "true cohesion" used by Hvorslev (1937, 1960) and Terzaghi (1938) is inconsistent with the physical meaning that Coulomb (1773) ascribed to the term cohesion. A fundamental difference between the concepts of cohesion given by Coulomb (1773) and Hvorslev (1937, 1960) is that Coulomb's cohesion (1773) originates from the "cementation" between the particles of the materials he studied (rocks, masonry, etc.), where the variation of the void ratio is not considered in the strength study. In contrast, Hvorslev's (1937, 1960) and Terzaghi's (1938) "true cohesion" depends on the void ratio, a variable that is taken into account in the study of soil strength. Furthermore, a remolded soil, where any existing cementation between grains has been destroyed, cannot exhibit true cohesion in the sense used by Coulomb (1773). Thus, "Hvorslev's true cohesion" has a different physical nature from Coulomb's "true cohesion". Therefore, the name "Hvorslev's true cohesion" is an inappropriate name for the mentioned strength parameter because, not being cohesion itself, it cannot, *ipso facto*, be called true cohesion. So, what physical phenomenon gives rise to the strength portion represented by $c_e\ (e)$ in Figure 13 and Equation 12? The answer seems to be found in classical texts of soil mechanics such as those by Terzaghi & Frölich (1936), Hvorslev (1937, 1960), Terzaghi (1938, 1941), Taylor (1942, 1948), and Bjerrum (1954).

According to Terzaghi (1941), clay particles are surrounded by a layer of highly viscous adsorbed water. As the distance from the surface of the clay particles increases, the viscosity of the adsorbed water decreases until, at a certain distance *"d"*, the adsorbed water becomes free water. This conception creates the possibility of two types of contact between clay particles: solid-to-solid type contacts, through the solid adsorbed water and viscous type contacts, through the viscous adsorbed water (see Figure 14).

During shearing, when clay particles move relative to each other, in addition to overcoming the frictional resistance present at solid-to-solid contacts, it is necessary to overcome the viscous resistance of the adsorbed water that surrounds the particles in both types of contact. The phenomenon is described by Terzaghi & Frölich (1936) as follows:

> *"La resistance au glissement des particules n'est pas produite uniquement par la résistance de frottement mais aussi par la viscosité des couches separatrices, entourant la zone de frottement"* (Terzaghi & Frölich, 1936, p. 18-19).

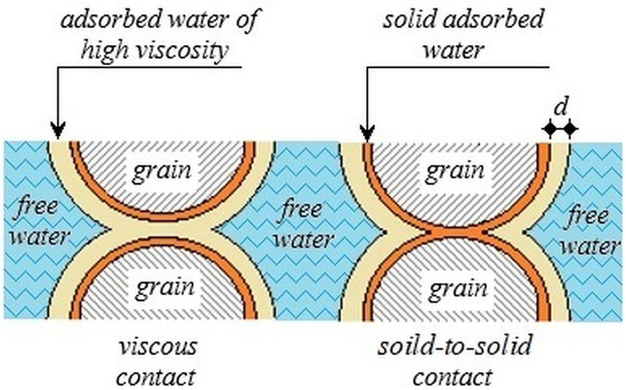

Figure 14. Illustration of adsorbed water and types of contact between clay particles (adapted from Terzaghi, 1941).

In a reevaluation of his 1937 work, Hvorslev (1960) points out that there are three components in the shear strength (τ_{ff}) of a soil, namely: the effective friction component (τ_ϕ), the effective cohesion (c_e), and the dilatancy component (τ_d), that is to say:

$$\tau_{ff} = \tau_\phi + c_e + \tau_d \tag{13}$$

According to Hvorslev (1960), cohesive soils (which the authors of this article suggest calling plastic soils) are those that exhibit the component of effective cohesion (c_e). It is interesting to know how Hvorslev (1960) conceives effective cohesion, also referred to as Hvorslev's true cohesion:

"Most cohesive soils possess an apparent structural viscosity and their deformations are of visco-elastic character. The corresponding strength component may be called the "viscous component", but factors other than viscosity seem to be involved, and the more inclusive term "rheological component" and the notation c_v are proposed. It will be assumed that c_v forms a part of the effective cohesion component, c_e, because the effective friction component, τ_ϕ, of a remolded clay does not seem to be affected by the increased rate of deformation after failure, provided the soil structure is not changed; However, this assumption is in need of further experimental corroboration. The value of c_v converges on zero with increasing time or decreasing rate of deformation, whereas c_e at the same time approaches an ultimate value, c_u, which may be called the "ultimate cohesion component. By definition, the following relation exists at any given test duration or rate of deformation $c_e = c_u + c_v$ "(Hvorslev, 1960, p. 182-183).

There is still the following excerpt transcribed from Hvorslev (1960) that is of fundamental importance for the physical understanding of Hvorslev's true cohesion meaning:

"[...] the basic assumption is made that the cohesion and rheological components are constant when (1) the void ratio or water content of saturated clays is constant, (2) the rate of deformation or test duration is constant, and (3) there is no significant difference in the geometric structure of the clays during a given test series" (Hvorslev, 1960, p. 183).

It is interesting to observe the importance that Hvorslev (1960) ascribes to the influence of the rate of deformation on the c_e value, reaffirming his conception of the viscous nature of Hvorslev's true cohesion. However, paradoxically, according to Figure 13 and Equation 12, c_e is expressed only as a function of the void ratio. Thus, Equation 12, which does not take into account the influence of the rate of deformation, cannot represent the physical phenomenon which, according to Hvorslev (1960) himself, is of a viscous origin!

Taylor (1948) was the first to physically interpret the "effective cohesion" as being of viscous origin and to quantify it. The following excerpt, taken from Taylor (1948), summarizes his conception regarding the effect of the rate of deformation on the shear strength of clays:

"The effect of speed of shear on the strength is believed to be caused by the viscous or plastic characteristics of material in the adsorption zones in the vicinity of points of contact or near contact of clay particles" (Taylor, 1948, p. 379)

According to Taylor (1948, p. 377-378), all viscous and plastic materials exhibit a resistance that depends on the distortion rate $\left(\dot{\gamma} = \partial\left(\varepsilon_1 - \varepsilon_3\right)/\partial t\right)$. In clays, this resistance, referred to by Taylor (1948) as "plastic resistance", is an example.

In the shear phase of a *CIUCL* (Consolidated Isotropically Undrained Compression Loading) triaxial test, $\varepsilon_1 = \varepsilon_a$, $\varepsilon_3 = \varepsilon_r$ and $\varepsilon_v = \varepsilon_a + 2\varepsilon_r$, where ε_a, ε_r and ε_v are the axial, radial and volumetric strains, respectively. Since during the shear phase of a *CIUCL* test, $\varepsilon_v = 0$, it follows that $\varepsilon_r = -(\varepsilon_a / 2)$. Thus, $\gamma = \varepsilon_a - \varepsilon_r = (3/2)\varepsilon_a$ and $\dot{\gamma} = (3/2)\dot{\varepsilon}_a$. This result shows that to study the influence of the distortion rate $(\dot{\gamma})$ on the undrained shear strength of a clay via *CIUCL* tests, it is sufficient to study the influence of the axial strain rate $\dot{\varepsilon}_a$.

Figure 15 shows the effect of the strain rate $(\dot{\varepsilon}_a)$ on the undrained strength of a normally consolidated sample of Sarapuí II clay, measured in a *CIUCL* triaxial test. Results of this kind are typical, having already been obtained by many authors before.

Two points raised by Bjerrum (1954) support the hypothesis of "Hvorslev's true cohesion" (c_e) being due to the viscous action of adsorbed water: its increasing magnitude with the decrease in void ratio (see Figure 13) and its dependence on strain rate, as illustrated in Figure 15.

In summary, all the aforementioned articles indicate that the so-called "Hvorslev's true cohesion" is of a viscous nature and, therefore, different from the nature of "Coulomb's cohesion".

Assuming that "Hvorslev's true cohesion" is of a viscous nature, caused by the resistance to distortion of the adsorbed viscous water, and that the viscosity of the adsorbed water increases as it approaches the surface of the particles, it can be concluded that "Hvorslev's true cohesion" must be a function not only of the void ratio but also of the rate at which shear occurs, i.e., the rate of distortion.

Starting from the conception outlined above, Martins (1992, 2023) showed that, in a plastic soil, the shear stress on the failure plane at failure (τ_{ff}) can be considered to be internally resisted by the sum of two components: a friction component (τ_ϕ) and a viscous component (τ_η), that is:

$$\tau_{ff} = \tau_\phi + \tau_\eta \tag{14}$$

The friction component (τ_ϕ) is given by:

$$\tau_\phi = \sigma'_{ff} \tan\phi'_e \tag{15}$$

where σ'_{ff} is the effective normal stress on the failure plane at failure and ϕ'_e is the "Hvorslev's true angle of friction".

The viscous component (τ_η) is given by:

$$\tau_\eta = \eta(e) f\left(\dot{\gamma}_{ff}\right) \tag{16}$$

Where η (e) is defined as the coefficient of soil viscosity (see Martins, 2023), a function of the void ratio and of the soil structure, and $f\left(\dot{\gamma}_{ff}\right)$ is a function of the distortion rate in the failure plane at failure.

By adding Equations 15 and 16, one finally arrives at Expression 17, which gives the shear strength as the sum of two components: one from frictional origin and the other from viscous nature, namely:

$$\tau_{ff} = \sigma'_{ff} \tan\phi'_e + \eta(e) f\left(\dot{\gamma}_{ff}\right) \tag{17}$$

As previously presented, "Hvorslev's true cohesion", c_e, is a function of the void ratio, of the rate of distortion, and of the soil structure, which are, according to Hvorslev (1960), the three variables upon which "Hvorslev's true cohesion" depends. However, since c_e is written in Equation 12 as an exclusive function of the void ratio, Expression 12 is not able to take into account the effect of the strain rate, as shown in Figure 15, ability that Equation 17 has. In the case of Figure 15, for example, after failure, with $\varepsilon_a \cong 10\%$ and the void ratio kept constant ($e_c = 2.13$), when all friction resistance has already been mobilized, it was enough to increase the rate of strain for the shear strength to also increase.

By equating Expressions 12 and 17, one obtains

$$c_e = \eta(e) f\left(\dot{\gamma}_{ff}\right) \tag{18}$$

Equation 18 leads to the conclusion that assuming shear strength of a plastic soil is composed of a friction component and a viscous component, the "Hvorslev's true cohesion", c_e, corresponds to the viscous component and is a function not only of the void ratio but also of the rate of distortion. As for the structure effect, it can be taken into account through the parameter η (*e*). Thus, it can be concluded that Equation 17 can advantageously replace Equation 12 and that Hvorslev's true cohesion cannot be treated as cohesion because it is of viscous nature.

Finally, it remains to show the relationship between Expression 17 and Figure 13. To do this, observe Figure 16, where for the Hvorslev *YFC* envelope, $c_e = c_e(e_2) = OY$.

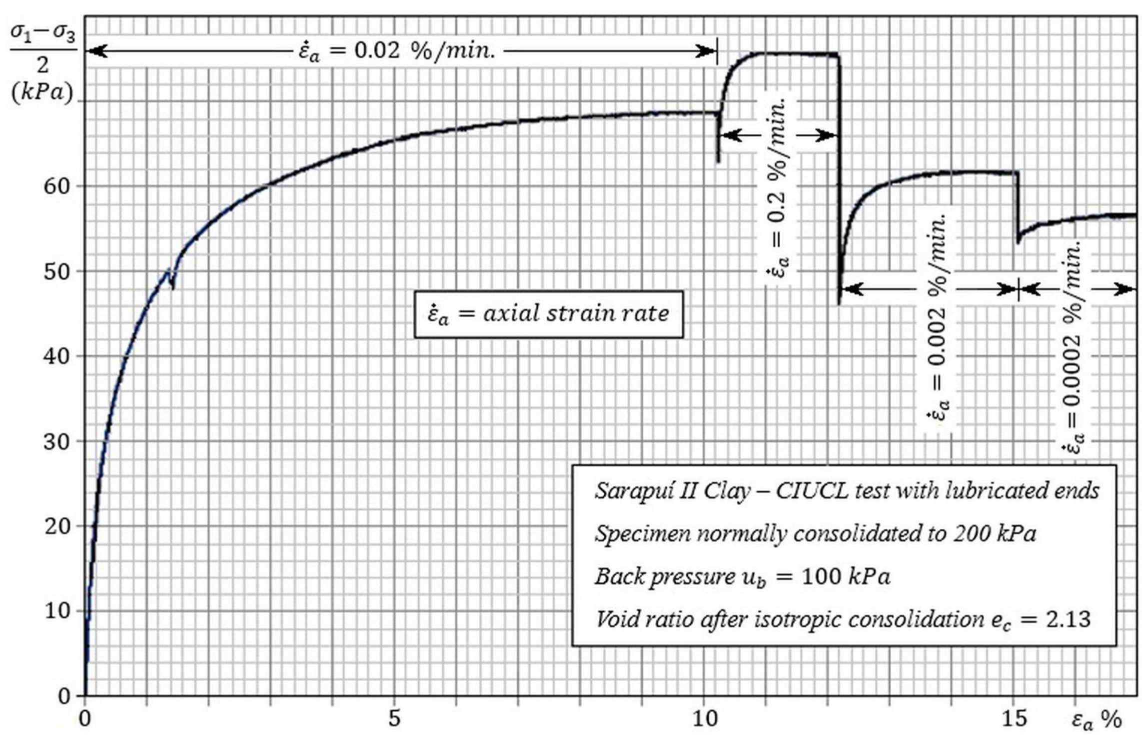

Figure 15. Strain rate effect on the undrained strength of Sarapuí II Clay (Martins, 2023).

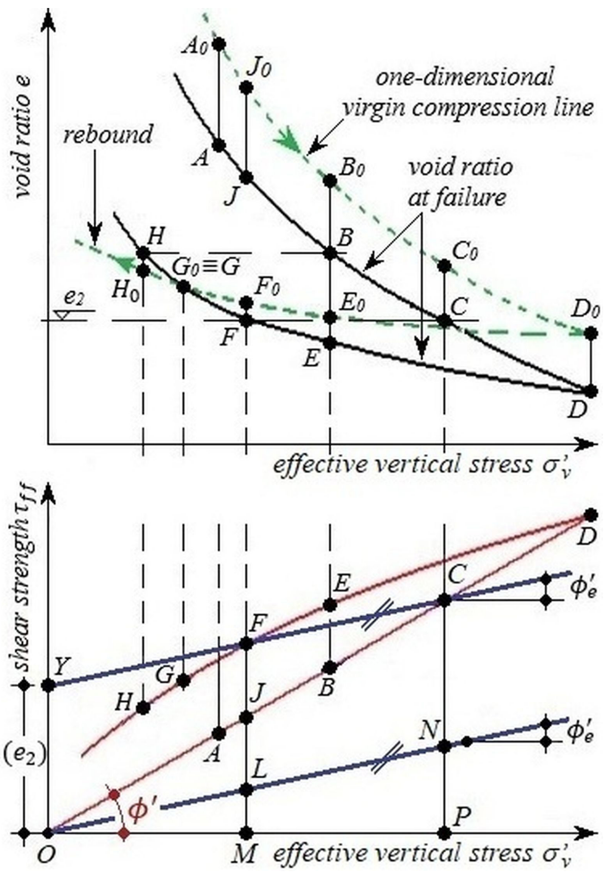

Figure 16. Relationship between Expression 17 and Figure 13.

According to Equation 12, the friction component along the *YFC* envelope is $\sigma'_{ff} \tan\phi'_e$. This resistance due to friction is represented by line *OLN* in Figure 16.

The expression for the normally consolidated envelope *OAJBCD* in Figure 16 is

$$\tau_{ff} = \sigma'_{ff} \tan\phi' \tag{19}$$

By equating Expressions 19 and 13, one obtains:

$$\sigma'_{ff} \tan\phi' = \tau_\phi + c_e + \tau_d \tag{20}$$

In the case of normally consolidated soils, which exhibit contractive behavior, the shear strength component corresponding to dilatancy is zero, that is, $\tau_d = 0$. Thus, Equation 20 can be rewritten as:

$$\sigma'_{ff} \tan\phi' = \tau_\phi + c_e \tag{21}$$

By substituting the friction component $\tau_\phi = \sigma'_{ff} \tan\phi'_e$ into Equation 21, one obtains:

$$\sigma'_{ff} \tan\phi' = \sigma'_{ff} \tan\phi'_e + c_e \tag{22}$$

This means that in the shear strength of a normally consolidated clay there is embedded a portion corresponding to "Hvorslev's true cohesion." However, since the normally consolidated failure envelope passes through the origin, a normally consolidated soil does not possess cohesion in the sense used by Coulomb, which shows, once again, that "Hvorslev's true cohesion" cannot actually be cohesion in the sense used by Coulomb. This aspect can be illustrated by analyzing point *C* of Figure 16. Point *C* is on the normally consolidated failure envelope and has a shear strength equal to *PC*, the sum of friction resistance *PN* and "Hvorslev's true cohesion" $c_e = c_e(e_2) = NC$. However, as shown in Figure 15, the strength of normally consolidated clay also depends on the strain rate. Thus, according to Equation 17, the shear resistance *PC* in Figure 16 is more appropriately written as the sum of the friction component $\tau_\phi = PN = \sigma'_{ff}(P) \tan\phi'_e$ and the viscous component $\tau_\eta = NC = \eta(e_2) f(\dot{\gamma}_{ff})$.

The reasoning above can be applied to any point of the normally consolidated failure envelope *OABCD*, as long as the shear rate is kept constant. At point *J*, for example, the shear resistance is *MJ*, the sum of the friction resistance $\tau_\phi = ML = \sigma'_{ff}(M) \tan\phi'_e$ and the viscous resistance $\tau_\eta = LJ = \eta(e_J) f(\dot{\gamma}_{ff})$, where e_J is the void ratio corresponding to point *J*.

Analogous reasoning can be applied to compare the shear strength at point *F*, on the overconsolidated failure envelope, with that at point *J*. Points *J* and *F* are at failure under the same σ'_{ff}. Since both tests exhibit contractive behavior (see Figure 16), the dilatancy component (τ_d) in both tests is zero. As both tests fail under the same σ'_{ff}, the friction component of both is the same, given by segment *ML* in Figure 16. Thus, in light of Expression 17, the difference lies in the viscous component $\eta(e) f(\dot{\gamma}_{ff})$. Assuming that the rate of deformation in both tests is the same, since the void ratio of the test represented by point *F* is lower than that represented by point *J*, the viscous strength given by *LF* is greater than the viscous strength given by *LJ*. Hence, since the frictional strength component at points *F* and *J* are equal, the shear strength at point *F* must be greater than the shear strength at point *J*.

6. Summary and conclusions

There are two modes of failure: by separation (observed in direct tensile tests) and by shear (observed in unconfined compression tests).

Coulomb (1773) defined the cohesion of a material as its strength measured in a direct tensile test.

Shear tests on cantilever beams in the transverse direction to their axis, neighboring the built-in end, provided practically the same strength obtained in the direct tensile tests. Coulomb (1773) then assumed that the strengths in the tensile and shear tests were equal to each other and equal to the cohesion.

According to Coulomb (1773), a non-slender prismatic solid body subjected to unconfined compression fails by shear along an inclined plane. The shear and normal stresses on the failure plane at failure, respectively σ_{ff} and τ_{ff}, are related to each other by $\tau_{ff} = c + \sigma_{ff} \tan \phi$, where c is the cohesion and $\tan \phi$ is the friction coefficient. In shear failure, it is necessary to overcome the resistances due to cohesion (c) and due to friction ($\sigma_{ff} \tan \phi$). This is Coulomb's law.

Mohr (1900) set a failure criterion which can be stated as: For each solid material there is a relationship between shear stress τ and normal stress σ, expressed by a function f, so that, if an ordered pair (σ,τ), which satisfies the function $\tau = f(\sigma)$, is acting at a point P of this solid body, on a given plane that passes through P, then there will be failure of this solid body, at point P, along the referred plane. In this case, the normal and shear stresses acting on the failure plane, at failure, are denoted respectively by σ_{ff} and τ_{ff} and are related to each other by $\tau_{ff} = f(\sigma_{ff})$.

Plotted on the τ versus σ plane, the function $\tau_{ff} = f(\sigma_{ff})$ represents, for a given material, a boundary line beyond which there cannot exist points (σ,τ) representing stresses that could not be resisted by the material in question. This boundary line is called the Mohr failure envelope.

When the failure envelope assumes the shape of a straight line, which can be written as $\tau_{ff} = c + \sigma_{ff} \tan \phi$, it is called the Mohr-Coulomb failure envelope, as it associates the Mohr failure criterion with Coulomb's law.

Many rocks have their shear strength expressed by the Mohr-Coulomb failure envelope. Such an envelope is written in terms of effective stresses as $\tau_{ff} = c' + \sigma'_{ff} \tan\phi'$. This failure envelope presents an inconsistency because it indicates, in direct tensile tests, that the failure plane is not orthogonal to the tensile stress. For consistency to exist, the strength envelope must be curved in the tensile region and intercept the σ' axis orthogonally.

The weathering process of rocks represents a loss of cohesion in the sense of the term used by Coulomb (1773). Lithification is the converse process.

When in the range of compressive stresses, where failure occurs by shear, the failure envelope is not a straight line, it is common to fit to the section of the curved envelope, within the range of effective normal stresses of interest, a line whose equation is given by $\tau_{ff} = c' + \sigma'_{ff} \tan\phi'$. In these cases, the parameter c' is called "effective cohesion intercept" and does not represent cohesion in the sense given by Coulomb, being merely the linear coefficient of the line fitted to the curved section of the envelope.

When a saturated clay sample is taken from the field and exposed to the atmosphere, the pressure in the pore water is lower than atmospheric pressure. Under such a condition, the effective stress in the sample is equal in magnitude to the negative pore pressure, which gives clays samples a certain shear strength. This explains why a saturated clay sample is able to remain standing, even though it does not have true cohesion. Such shear strength is sometimes called apparent cohesion.

When saturated specimens are subjected to *UU* triaxial tests, the applied confining stress does not affect the state of effective stress. As a consequence, the shear strength will be the same, regardless of the applied confining stress and the Mohr circle of effective stresses at failure will be unique. The Mohr circles of total stresses at failure will be separated from each other by a distance equal to the difference in confining stresses. At the time when the concept of effective stress was unknown, the resistance envelope was drawn tangent to the total stress circles, resulting in a horizontal envelope ($\phi = 0$). This situation led to the false conclusion that these soils were "purely cohesive". In this case, the most appropriate name for the measured strength is undrained strength (S_u).

Hvorslev's true cohesion has a different physical meaning from Coulomb's cohesion. According to Hvorslev himself, what he called true cohesion has characteristics of viscous resistance since it depends on the void ratio and the rate of deformation (and on the soil structure). Thus, Hvorslev's true cohesion cannot be expressed as an exclusive function of the void ratio.

Hvorslev's true cohesion should be interpreted as being of a viscous nature. The strength of a plastic soil can be written as the sum of a component due to friction and a component due to viscosity (Equation 17).

Declaration of interest

The authors have no conflicts of interest to declare. All co-authors have observed and affirmed the contents of the paper and there is no financial interest to report.

Authors' contributions

Ian Schumann Marques Martins: Conceptualization, Data curation, Formal Analysis, Funding acquisition, Investigation, Methodology, Project administration, Resources, Supervision, Validation, Visualization, Writing - original draft, Writing - review & editing. Michel da Cunha Tassi: Conceptualization, Data curation, Formal Analysis, Investigation, Validation, Visualization, Writing - review & editing. Mirella Dalvi dos Santos: Conceptualization, Methodology, Writing - review & editing. Mauricio do Espirito Santo Andrade: Conceptualization, Methodology, Visualization, Writing - review & editing. Vitor Nascimento Aguiar: Conceptualization, Methodology, Visualization, Writing - review & editing.

Data availability

All data produced or examined in the course of the current study are included in this article.

Declaration of use of generative artificial intelligence

This work was prepared without the assistance of any generative artificial intelligence (GenAI) tools or services. All aspects of the manuscript were developed solely by the authors, who take full responsibility for the content of this publication.

List of symbols and abbreviations

$1/n$	Coulomb's friction coefficient
a	Size of the side of a squared cross-sectional prismatic column
c	Coulomb's cohesion
c_e	Hvorslev's true cohesion
c'	Cohesion written in terms of effective stress or effective cohesion intercept
d	Thickness of the adsorbed water layer
e	Void ratio
K_0	Coefficient of earth pressure at rest
P	Centered vertical load applied to a column
S_u	Undrained strength
u	Pore pressure
u_a	Pore air pressure
u_0	Pore pressure in the field
u_{ps}	Pore pressure after perfect sampling
u_w	Pore water pressure
$\dot{\gamma}$	Distortion rate
$\dot{\gamma}_{ff}$	Distortion rate in the failure plane at failure
ε_1	Major principal strain
ε_3	Minor principal strain
ε_a	Axial strain
$\dot{\varepsilon}_a$	Axial strain rate
ε_r	Radial strain
ε_v	Volumetric strain
η	Coefficient of soil viscosity
θ	Slope of failure plane in an unconfined compression test
σ	Normal stress
σ_1	Major principal stress
σ_2	Intermediate principal stress
σ_3	Minor principal stress
σ_c	Confining stress
σ_h	Total horizontal stress
σ_v	Total vertical stress
σ_{ff}	Normal stress on the failure plane at failure
σ_{v0}	Total vertical stress in the field
σ'	Normal effective stress
σ'_{ff}	Normal effective stress on the failure plane at failure
σ'_i	Intrinsic pressure
σ'_t	Tensile strength
σ'_v	Vertical effective stress
σ'_{h0}	Horizontal effective stress in the field
σ'_{oct}	Octahedral effective stress
σ'_{ps}	Effective stress after perfect sampling
σ'_{v0}	Vertical effective stress in the field
τ	Shear stress
τ_d	Component of shear strength due to dilatancy
τ_{ff}	Shear stress on the failure plane at failure
τ_η	Component of shear strength due to viscosity
τ_ϕ	Component of shear strength due to friction
ϕ	Friction angle
ϕ'	Friction angle written in terms of effective stress
ϕ'_e	Hvorslev's true angle of friction

References

Bjerrum, L. (1954). *Theoretical and experimental investigations on the shear strength of Soils* (Publication, No. 5). Oslo: Norwegian Geotechnical Institute.

Bjerrum, L. (1973). Problems of soil mechanics and construction on soft clays: state-of-the-art report to session IV. In *Proceedings of 8th International Conference on Soil Mechanics and Foundation Engineering* (Vol. 3), Moscow. ISSMGE. Retrieved in October 19, 2025, from https://www.issmge.org/uploads/publications/1/37/1973_06_0004.pdf

Bolton, M. (1979). *A guide to soil mechanics*. London: The Macmillan Press.

Coulomb, C.A. (1773). Sur une application des règles de maximis & minimis à quelques problems de statique, relatifs à l' architecture. In *Mémoires de Mathematique et de Physique, présentés à l'Académie Royale des Sciences, par divers Savans, et lûs dans ses Assemblées* (pp. 343-382). Paris: Académie Royale des Sciences.

Fredlund, D.G., & Rahardjo, H. (1993). *Soil mechanics for unsaturated soils*. New York: John Wiley & Sons.

Gibson, R.E. (1953). Experimental determination of the true cohesion and true angle of internal friction in clays. In *Proceedings of 3rd International Conference on Soil Mechanics and Foundation Engineering* (Vol. 1, pp. 126-130), Zurich. ISSMGE. Retrieved in October 19, 2025, from https://www.issmge.org/uploads/publications/1/42/1953_01_0029.pdf

Hight, D.W. (2003). Sampling effects in soft clay: an update on Ladd and Lambe (1963). In *Proceedings of the Symposium on Soil Behavior and Soft Ground Construction* (pp. 86-121). ASCE. https://doi.org/10.1061/40659(2003)4.

Hvorslev, M.J. (1937). Über die Festigkeitseigenschaften gestörter bindiger Böden. *Ingeniörvidenskabelige Skrifter*, 45, 1-159.

Hvorslev, M.J. (1960). Physical components of the shear strength of saturated clays. In *Research Conference on Shear Strength of Cohesive Soils* (pp. 169-273). Boulder: University of Colorado.

Lambe, T.W., & Whitman, R.V. (1979). *Soil mechanics, SI version.* New York: John Wiley & Sons.

Lee, K.L., & Seed, H.B. (1967). Drained strength characteristics of sands. *Journal of the Soil Mechanics and Foundations Division*, 93(6), 117-141. https://doi.org/10.1061/JSFEAQ.0001048.

Martins, I.S.M. (1992). *Fundamentos de um modelo de comportamento de solos argilosos saturados* [Doctoral thesis]. Federal University of Rio de Janeiro.

Martins, I.S.M. (2023). The 8th Victor de Mello lecture: role played by viscosity on the undrained behaviour of normally consolidated clays. *Soils & Rocks*, 46(3), e2023006123. https://doi.org/10.28927/SR.2023.006123.

Mohr, O.C. (1882). Ueber die darstellung des spannungszustandes und des deformationszustandes eines körperelementes und über die anwendung derselben in der festigkeitslehre. *Der Civilingenieur*, 28(2), 113-156.

Mohr, O.C. (1900). Welche umstande bedingen die elastizitatsgrenze und den bruch eines materiales? *Zeitschrift des Vereines Deutscher Ingenieure*, 44(45), 1524-1530.

Obert, L., & Duvall, I.D. (1967). *Rock mechanics and the design of structures in rock*. New York: John Wiley & Sons, Inc.

Pinto, C.S. (2006). *Curso básico de mecânica dos solos* (3. ed.). São Paulo: Oficina de Textos.

Schofield, A.N. (1999). A note on Taylor's interlocking and Terzaghi's "true cohesion" error. *Geotechnical News*, 17(4), 56-58.

Schofield, A.N. (2001). Re-appraisal of Terzaghi's soil mechanics. In *Proceedings of the 15th International Conference on Soil Mechanics and Geotechnical Engineering* (pp. 2473-2480), Istanbul. Retrieved in October 19, 2025, from https://www.issmge.org/uploads/publications/1/30/2001_04_0003.pdf

Simões, F.B. (2015). *Caracterização geotécnica da Areia da Praia de Ipanema* [Bachelor's dissertation]. Federal University of Rio de Janeiro (in Portuguese).

Skempton, A.W. (1948). A study of the immediate triaxial test on cohesive soils. In *Proceedings of the 2nd International Conference on Soil Mechanics and Foundation Engineering* (pp. 192-196), Rotterdam. Retrieved in October 19, 2025, from https://www.issmge.org/uploads/publications/1/43/1948_01_0038.pdf

Taylor, D.W. (1942). *Research on consolidation of clays* (Serial, No. 82). Cambridge: Department of Civil and Sanitary Engineering, Massachusetts Institute of Technology.

Taylor, D.W. (1948). *Fundamentals of soil mechanics*. New York: John Wiley & Sons.

Terzaghi, K. (1936). The shearing resistance of saturated soils and the angle between the planes of shear. In *Proceedings of the 1st International Conference on Soil Mechanics and Foundation Engineering* (pp. 54-56), Cambridge, Mass. Retrieved in October 19, 2025, from https://www.issmge.org/uploads/publications/1/44/1936_01_0017.pdf

Terzaghi, K., & Frölich, O.K. (1936). *Théorie du tassement des couches argileuses.* Paris: Dunod.

Terzaghi, K. (1938). The Coulomb equation for the shear strength of cohesive soils. In K. Terzaghi (Ed.), *From theory to practice in soils mechanics: selections from the writings of Karl Terzaghi* (pp. 174-180). New York: John Wiley & Sons.

Terzaghi, K. (1941). Undisturbed clay samples and undisturbed clays. *Journal of the Boston Society of Civil Engineers*, 28(3), 211-231.

Westergaard, H.M. (1952). *Theory of elasticity and plasticity* (Harvard Monographs in Applied Science, No. 3). Cambridge: Harvard University Press.