THE IMPORTANCE OF TENSILE STRENGTH IN GEOTECHNICAL ENGINEERING

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Abstract

Many Soil mechanics textbooks contain only limited information about tensile characteristics. Šuklje's "Rheological aspects of soil mechanics" [1] is an exception, as he devoted a special chapter to this problem "Tensile and Bending Strength of Soils". Therefore, it is not a great surprise that the subject of the 13th Šuklje's Lecture is devoted to soil behaviour in tension. Tensile tests are briefly described, some results as well, with a distinction between undrained and drained tests. Practical examples of the application of the results are discussed, firstly in cases where the development of tensile cracks can be expected. Because the results of the drained tests give more information about the bonds between individual particles, some theoretical aspects of these tests are discussed as well.

Keywords

tensile cracks, tensile strength, bending, triaxial drained tensile test

1 INTRODUCTION

The behaviour of soils in tension is a subject of great interest, not only for geotechnical engineers, but also for other branches of engineering, such as agricultural or mining, where the main object is connected with tillage or with resistance during soil excavation.

From the geotechnical engineering point of view, the interest with respect to the tensile strength of soils is very often connected with the different tensile cracks that can develop in earth structures, such as embankment dams, slopes, retaining walls from reinforced soil, or with a capping clay sealing system of sanitary landfills, e.g., Vaníček [2]. Some examples are presented in the following figures. Fig. 1 shows a tensile crack that developed close to a rockfill dam crest parallel to its longitudinal axis.

Figure 1. Longitudinal crack on the surface of the clay core – Jirkov dam.

Very often, a tensile crack can be observed at the top of the slope as a first sign of the potential danger of a slope stability problem, Fig. 2.

A large tensile crack was also observed for a high retaining wall made from reinforced soil, very close behind the zone of reinforcement, Fig. 3. The water flowing into this crack started the process of wall overturning, as was observed from the shape of the quasi-homogeneous reinforced part of the wall.

Figure 2. Tensile crack at the upper part of a slope.

In the 1990s a great deal of attention was devoted to the possibility of tensile crack development in the capping sealing system of landfills, e.g., Jessberger and Stone [3], Daniel [4]. Due to the differential settlement of the deposited waste, local depression can develop there with a strong possibility of tensile crack development. These tensile cracks can influence, in a negative sense, the sealing function of this capping sealing system, Fig. 4, Vaníček [5].

The tensile tests and the obtained results are very useful tools for recognising the probability of tensile crack development, either from the view of the tensile strength or the tensile elongation at which the tensile cracks can be opened. However, these results are also very useful for the numerical modelling of the development of the tensile zone in the earth structure. In particular, the modulus of deformation in tension (or extension) determined from these tests can help to improve our knowledge about the tensile zone widening.

However, tensile tests can also help to improve our knowledge from the theoretical point of view, about which failure criteria are more general or what forces

Figure 3. Retaining wall made from reinforced soil – tensile crack behind the zone of reinforcement.

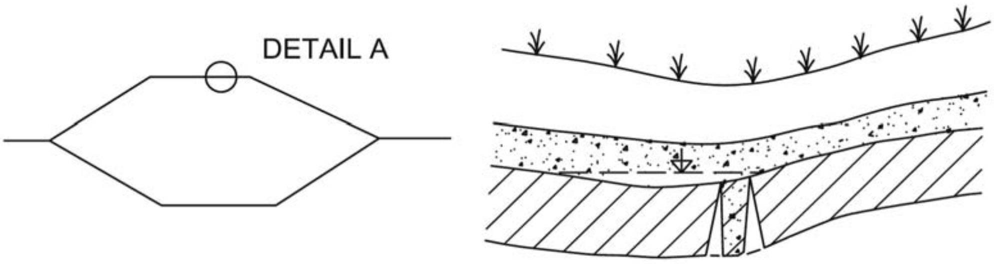

Figure 4. Deformation of the landfill surface – local differential settlements affecting the functionality of the capping clay liner.

between the individual soil particles are affecting the soil structure arrangement, e.g., Rosenquist [6], Bishop and Garga [7], Parry and Nadarajan [8]. This is especially the case with respect to the orientation of soil particles and the nature of the forces between adjacent soil particles. It is obvious that there is a certain relation between the effective tensile strength and the effective cohesion. Very often a negligible, even zero, effective cohesion is attributed to the normally consolidated clayey soils, in spite of the fact that some effective cohesion was measured.

This approach is in certain disagreement with the observation of material on the Moon's surface; where there is supposed to be no water, and so no interparticles forces where the contact is water–mineral, which are very often attributed for the measured effective cohesion. Fig. 5 shows the footprints of an astronaut's boot, where the walls are vertical. The same was observed for the walls of cuts excavated by a small dredging-machine on the Moon's surface.

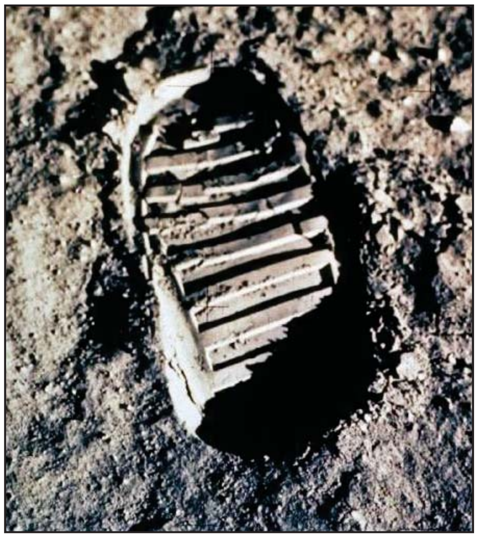

Figure 5. Footprints of an astronaut's boot on the Moon's surface with a vertical wall.

2 TYPES OF TENSILE TESTS

In the literature there are descriptions of the different tensile tests that varied in different ways. There is no unification, like for other soil mechanics tests, like shear or compression tests, see Vaníček [9]. If the dimensions of the tested samples, the manner of the preparation or the time effect are not taken into account, the principal classification can be made by:

- the principle of loading,
- the drainage conditions,
- the opportunity to measure the elongation.

Furthermore, a brief description is provided about the principle of loading with remarks about two other aspects, see Fig. 6

a) Axial tensile test (direct tension test)
b) Triaxial tensile test
c) Bending test
d) Test on hollow cylinder
e) Indirect (Brazilian) tensile test.

For the axial tensile test most problems are, according to Šuklje [1], connected with the uniform distribution of the tensile stresses in the test section of the specimen. For the tested samples with a uniform cross section, different types of connection were applied, e.g., freezing, Haefeli [10], or glueing, Zeh and Witt [11].

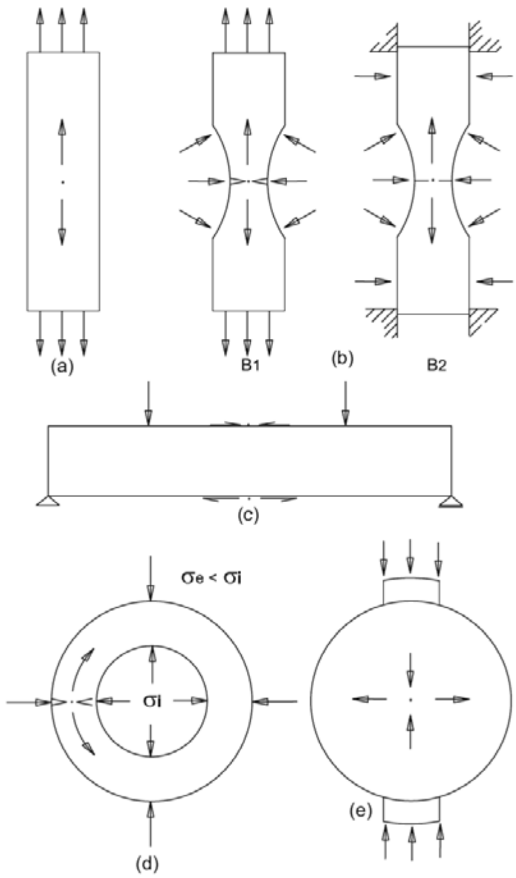

Figure 6. Division of tensile tests according to the principle of loading. a) Axial tensile tests (direct tension test). b) Triaxial tensile test. c) Bending test. d) Test on hollow cylinder. e) Indirect (Brazilian) tensile test.

For samples with wider ends (briquette) some sort of friction via connection clamps was applied. However, as in the first case, the transfer of the tension force from the widened heads to the test section must not cause concentrations of the stresses in the transition part of the specimen. Probably the largest samples were tested by Tschebotariof and De Phillipe [12]. In order to eliminate the unfavourable influence of the self-weight, the specimens are commonly tested in the horizontal position, and the effect of friction is eliminated by ball-bearing rollers, e.g., Tschebotariof and De Phillipe or by Drnovšek and Šubičeva [13]. For the unconfined tests mentioned by Hasegawa and Ikeuti [14], the weight was balanced by testing on a mercury surface. Ajaz and Parry [15] described load controlled and strain controlled direct tension tests, which were carried out by applying the pulling force through rods with universal joints to the brass grips holding the expanded ends of the tension specimen.

A triaxial tension test of type B1 will be described later for the drained tests. A type-B2 test was described by Ter-Martirosjan [16], [17] .The specificity of this test is the possibility to model a different stress state, depending on the ratio of the cross section at the end (A_E) of the sample to the cross section in the centre part of the sample (A_C). When a stiff sleeve is applied to the central part, the unconfined tensile test is modelled.

Some specificity of the tests on samples having the form of hollow cylinders and subjected to various internal and external hydrostatic pressures were described by Šuklje [1] and by Šuklje and Drnovšek [18]:

- well-defined stress states without uncontrolled stress concentration,
- the possibility of investigating the deformability and strength for various stress states,
- the possibility of taking into account the deformation anisotropy,
- the more likely possibility of carrying out long-term and drained tests.

However, up to now, very limited information was published with respect to the tensile soil behaviour.

The principle of the bending test consists of loading the tested soil beam with a pair of forces in the middle part of the sample. The advantage of such loading is the fact that in the central part of the tested beam the shearing force is zero and the bending moment is constant. It is a typical case of pure bending. The outermost fibres are either in tension or in compression.

The indirect (Brazilian) test is more often used in rock mechanics, as the sample is easily prepared from the obtained core drill and the load transfer is not so difficult. Nevertheless, some results were also published with respect to soil samples, e.g., Narain and Rawat [19] or Krishnayya et al. [20].

In the next sections bending tests will be described in more detail for undrained tests as well the triaxial test for drained tests together with the obtained results.

3 RESULTS OF UNDRAINED TENSILE TESTS (BENDING TESTS)

Undrained tensile tests are usually performed as a very quick test. If the duration of the test is longer, the undrained conditions are satisfied by sample coating, and most often the sample was coated with a layer of petrowax and petrolatum oil.

3.1 BENDING TEST ARRANGEMENT AND EVALUATION

Bending tests were mostly conducted to investigate the state of the compacted horizontal layer in the clay core of the earth and rockfill dams, as during their deflection tensile cracks can develop there, e.g., Leonards and Narain [21], Vaníček [22].

Bending test results can be recalculated in different ways according to the specific theory, see Fig. 7:

- The theory of elasticity assumes that the deformation of the outermost fibres is the same as well the stresses in these outermost fibres are the same. The neutral axis lies in the centre of the sample.
- Navier´s hypothesis assumes planar deformation in the cross section of the tested sample (however, the neutral axis is above the centre of gravity of the profile) and also the stresses are linear to the unit deformation.
- The differential method also assumes planar deformation in the cross section of the tested sample; however, this method is not based on any preferred stress-strain law.

The first one was used by Leonards and Narain, the second one by Vaníček and the last one by Ajaz and Parry [2][23]. Vaníček took advantage of the tests performed by Šuklje, who inserted several pairs of measuring pins into the beam, the deformation of which was recorded by photographs and the displacement of the pins determined in photo-comparators. The results of the tests made by Šuklje proved that the strain plots

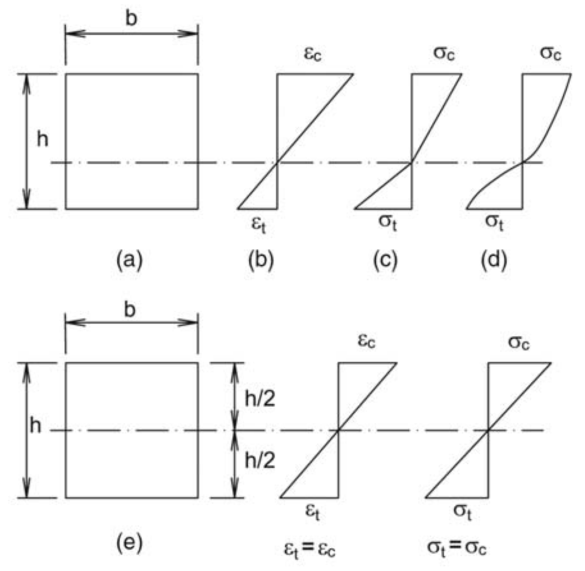

Figure 7. Bending test – fundamental assumptions. a) Beam cross section. b) Strain diagram. c) Stress diagram for Navier´s hypothesis (direct method). d) Stress diagram for differential method. e) Elastic bending theory.

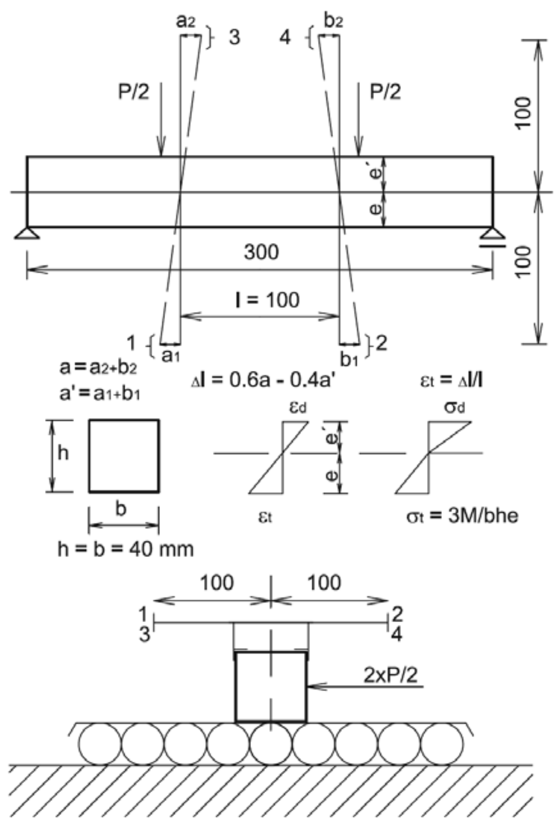

Figure 8. The layout of the arrangement and the evaluation of bending test proposed by the author.

are still approximately linear during the appearance of the first tensile cracks. Similar results were obtained by Ajaz and Parry, who applied the Cambridge radiographic technique using an embedded grid of lead shot for monitoring the strains within the beam.

The layout of the measuring device that was used by the author is presented in Fig. 8, see Vaníček [24].

The beam, 300 mm long and 40 × 40 mm in cross-section, is loaded in the horizontal direction. The weight of the beam and the friction are eliminated by means of horizontal ball-bearings. In the central part of the beam, which deforms under a constant bending moment, are fixed with two detectors connected to the beam. The deformation is measured at their ends, with roughly 5× magnification against the deformation of the outermost fibres. The soil-water mix was compressed in four layers in a special mould for the dry density and moisture content determined from the Proctor standard test. The beam was coated with a hot mixture of oil and paraffin. The loading was increased under a constant rate in the direction of the layers.

3.2 FACTORS INFLUENCING THE UNDRAINED TENSILE CHARACTERISTICS

In this section the influence of such factors as the initial moisture content and the compaction energy on the tensile characteristics will be described, together with time effects. This means factors that can be controlled during the construction of the clay core of the earth and rockfill dams.

3.2.1 The influence of moisture content

Generally, the tensile strength of the tested soils decreased with an increase in the moisture content. For small changes around the optimum moisture content this relationship was nearly linear. Fig. 9 shows the influence of the moisture content for a wider range. The result was obtained for the material of the clay core from the Dalešice dam, see Vaníček [25].

The maximum value of the tensile strength is reached for $w = w_{opt} - 3.5$ %. The index of plasticity was used for the comparison of the different tests and soils compacted at the optimum moisture content, Fig. 10, see Vaníček, I. and Vaníček, M. [26].

The increase in the tensile strength with the index of plasticity is not convincing. For most samples with an index of plasticity lower than 30, the maximum tensile strength is in the range 30–80 $kN.m^{-2}$. Similar results were obtained

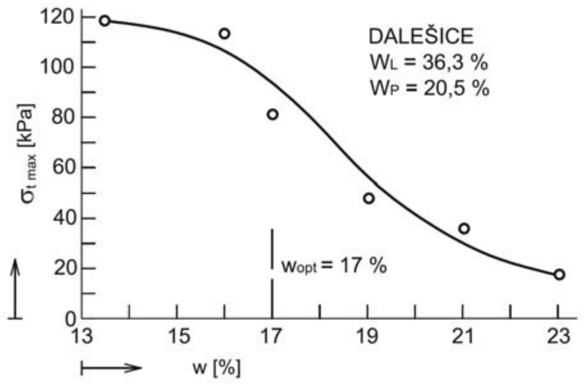

Figure 9. Tensile strength as a function of moisture content for the clay core from the Dalešice dam. Influence of moisture content on the tensile strain at failure.

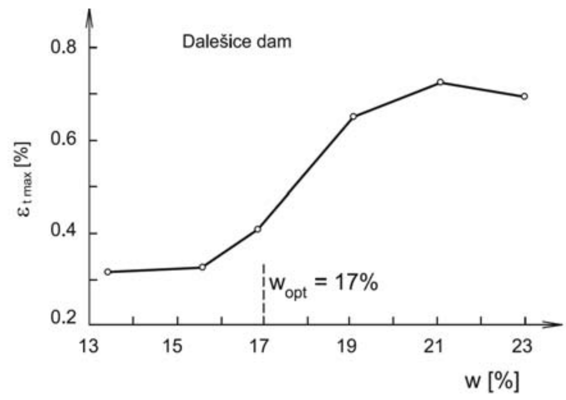

Figure 11. Influence of moisture content on the tensile strain at failure.

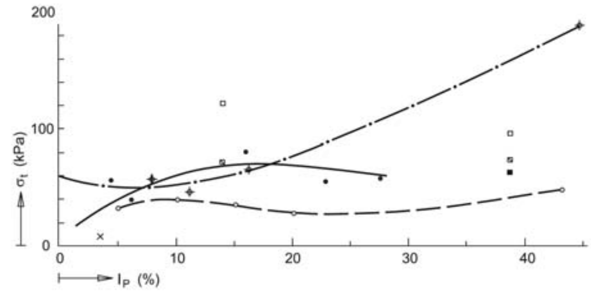

Figure 10. Results of the tensile strength for different tests and soils compacted by the Proctor standard at the optimum moisture content.

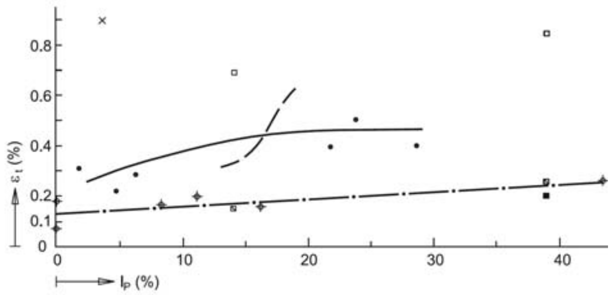

Figure 12. Results of the tensile strain at failure for different tests and soils compacted by the Proctor standard at the optimum moisture content.

by Narain and Rawat [19] and also by Ajaz and Parry [15]. From these results it is clear that the undrained tensile strength is mostly caused by capillary forces.

For small changes of moisture content around the optimum the tensile strain at failure increases linearly with the moisture content. For a wider range of moisture content for Dalešice clay this relation is shown in Fig.11. The increase of the failure tensile strain is relatively steep for a small change in the moisture content from the optimum. The gradient decreases at the end of the observed range and for $w = w_{opt} + 6\%$ first marked the occurrence of the decreasing of the maximum tensile strain.

Again, to compare the results from different authors and for different soils the index of plasticity I_p was used, see Fig.12 – Vaníček [27], Vaníček I. and Vaníček, M. [26].

The individual points represent tests on compacted samples by energy using the Proctor standard with the optimum moisture content. It is clear that the values of the maximum tensile strains are mostly in the range 0.2–0.6 % and increase with the value of the plasticity index. But this increase is not so significant as to unilaterally lead to an application just on plastic materials. The changes in the maximum tensile strain are more affected by the moisture content. It is shown in the same picture for the material from Dalešice dam (I_p = 15.8) using a dashed line.

With respect to the possibility of tensile crack development, that the stiffness of the soils stressed by tension is also important. In all cases the tensile modulus of deformation decreases while the moisture content increases. The question is how much the stiffness can be decreased with an acceptable increase in the moisture content. The increasing flexibility and decreasing strength are partly limited to the differences in stresses, which have a tendency to create in the earth structure. Simultaneously, they can be the reason for higher relative and total deformations, which on the other hand can help to create tensile cracks. Therefore, for each particular case there is an optimum value of the flexibility and strength, which will minimize the potential risk of crack development.

3.2.2 The influence of compaction effort

In accordance with the expectation, the higher compaction energy being applied for the optimum moisture content corresponding to this energy, e.g., a comparison of the energy typical for the Proctor standard and the Proctor-modified tests, the material flexibility is substantially reduced. When for the same initial moisture content a higher compaction effort is applied, after that the tensile strength and the tensile strain at failure are increasing. However, the impact on the secant modulus at the failure is negligible. This statement is very important because when the results of the compaction are better from the point of view of the maximum tensile strain, and so after that the shear strength and compression are also better, the shear parameters and the modulus of the deformation are higher.

3.2.3 The influence of time

Tests performed by author for the core of the Bulgarian dam Rosino showed that the influence of time is a complicated problem. On one hand the maximum elongation is increasing with the time of the test's duration, i.e., for very slow tests, from which a justifiable statement can be deduced, the clay core cracking is more probable for quick loading. However, also important is the time of the delay between the sample preparation and the testing, as normally a high pressure is used during the beam's formation. After unloading, a negative pore pressure can reach high values, which can increase the tensile undrained strength.

4 RESULTS OF THE DRAINED TENSILE TESTS

4.1 ARRANGEMENT OF THE DRAINED TESTS

Bishop and Garga [7] described the first triaxial drained tension test without the use of end clamps, type B1 in Fig. 6. A sample with a reduced centre section is enclosed by a rubber membrane. The end caps will only become detached from the ends under the action of an axial tensile force *T* when the average effective stress at the ends of the sample drops to zero. The axial effective stress throughout the centre section will, at this point, be negative (i.e., in tension), while the magnitude of this tensile stress is dependent on the ratio of the end-section and mid-section areas.

A controlled rate of the strain tension test with a constant cell pressure will thus be a test with $\sigma_1' = \sigma_2'$ = constant, and with σ_3' decreasing (until the peak stress difference $\sigma_1' - \sigma_3'$ is reached).

Bishop and Garga tested London blue clay (w_L = 75 %, w_p = 29 %) on undisturbed samples, carefully sampled in situ or on a remoulded sample. The differences in the results are clear. For the undisturbed samples, the measured effective tensile strength was in the range 26.3–33.3 kN.m^{-2}, whereas for remoulded samples it was practically zero. The time to failure for the undisturbed samples was in the range 6.7–55.2 hours and the tensile strain at failure was in the range 2.19–16.7 %. Some of the fundamental findings can be briefly summarized as:

- Failure of the sample has the character of a brittle material,
- Tensile stress at failure is almost independent of the value of σ_1' in the range examined,
- The variability of the maximum strain is relatively large.

However, there are some signals that the rate of loading for this type of clay was still insufficient to be fully drained.

The author completed two series of drained triaxial tests using the hydraulic triaxial apparatus described by Bishop and Wesley [28], see Fig.13.

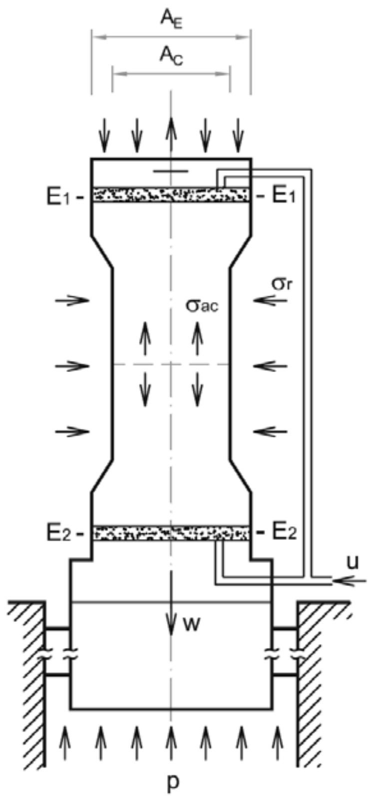

Figure 13. Layout of the drained triaxial tension tests performed in the hydraulic triaxial apparatus. σ_r – radial stress; *p* – stress in loading cell; *u* – back pressure applied to the drainage connection; *W* – weight of piston; σ_{ac} - axial stress on the centre section; A_E – area of end section; A_C – area of centre section.

During the first series, clay from the valley slopes downstream of the Cod Beck Dam was used to study the behaviour of plastic clay material in the range of small compression and tension stresses and to study the influence of the salinity of pore water on this behaviour, see Vaníček [9].

This clay (liquidity limit w_L = 44.1 %, plasticity limit w_p = 18.5 %, plasticity index I_p = 25.6 %) was deposited in a freshwater lake and the minimum of salts was supposed to be in the pore water. The remoulded samples, soil-water mix were compacted by hand into a special mould, with the help of a wooden stick. The samples were consolidated in the triaxial apparatus up to a consolidation pressure of 600 kN.m^{-2}. However, the tests started when the difference between the cell pressure and the back pressure was as low as from 5 to 25 kN.m^{-2}. The load cell was connected to the top cap and the test was started by applying a change in the pressure in the lower pressure cell.

Because the tests were performed as consolidated drained tests, a lot of attention was devoted to the determination rate of the loading, the time to failure, and to ensure that the excess (change) of pore pressures had a chance to dissipate (to equalize). The coefficient of the consolidation was calculated from the consolidation stage. The average value from the three lowest results is 0.07 cm^2 . min^{-1} (0.01 m^2 per day). The calculated required time to failure was, in all cases, slightly lower than the real time to failure, which was 30–40 hours.

A second, more extensive, series of tests was performed on Most clay (w_L = 53.1 %, w_p = 25.4 %, I_p = 26.7 %). Dry pulverized clay was mixed with water to form a clay slurry and this clay slurry was consolidated in a large oedometer with a diameter of 250 mm and a height of 150 mm under effective stresses of 100 or 300 kN.m^{-2}. Afterwards, at the end of the consolidation, cylindrical samples were cut out with a diameter of 38.1 mm (1.5").

The test arrangements are described in more detail by Vaníček, I. and Vaníček, M. [26].

4.2 DISCUSSION OF THE OBTAINED RESULTS

The typical result from a drained triaxial tension test is shown in Fig.14. For the presented case (sample No 2) the first peak in the deviator of stresses occurred for a tensile stress in the central part of the sample equal to 3.2 kN/m^2 and for an elongation of 3.6%.

After this first peak, a small neck in the central part was observed, but the deviator of stresses rose again

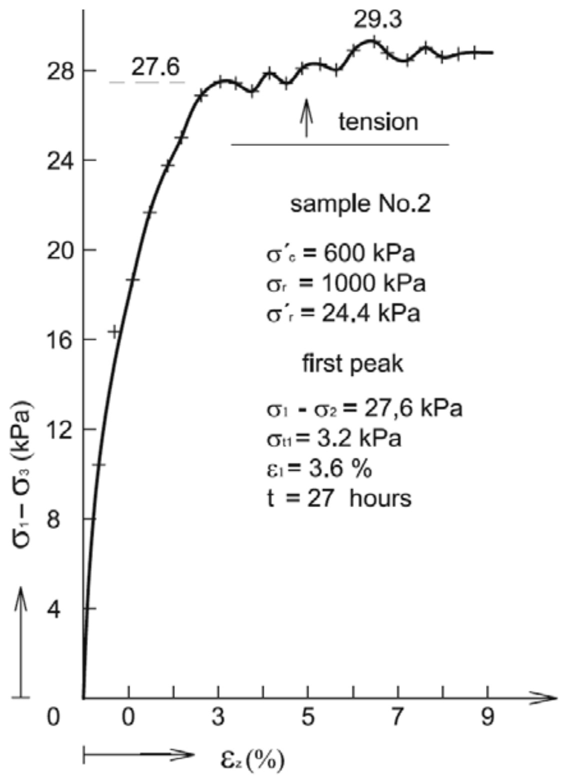

Figure 14. The typical result of drained triaxial tension test.

and a second neck was observed Fig.15. This untypical behaviour – when the failure did not continue in the first neck with the highest concentration of stresses – was observed in soils for the first time.

Figure 15. Photo of tested Most clay with visible two necks.

The character of the stress-strain curve is similar to the stress-strain curve of steel in tension. So, we can speak about a phase of strain hardening. It is rather difficult to explain this special behaviour. But with a high probability, the first failure is due to shear strength. This failure is accompanied by a fall in the deviator of stresses. After that – probably after the rearrangement of the clay particles in this zone – the tensile loading began and the stress-strain work hardening behaviour of the plastic clay was valid for this loading. This makes it possible to develop another shear failure at a different point.

The tests also helped to prove the character of Mohr's circles in the range of small positive (small compression) and negative (tension) stresses. Probably, the Mohr-Coulomb line is not valid for the range of stresses for negative values of the normal pressures, see Fig.16.

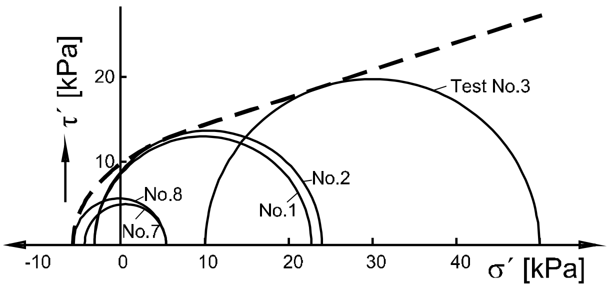

Figure 16. Mohr's circles for tri-axial compression and tension tests (for first failure).

The described sample No 2 was mixed during the phase of preparation only with distilled water. Samples No 4, 5 and 6 were mixed with brine (NaCl and KCl – dissolved in distilled water) so that samples No 4 and 5 contained 2.5 g of salts per litre of pore water and the ratio of the cations K/Na was 0.2 for sample No 4 and 0.8 for sample No 5. The pore water for sample No 6 contained 5 g of salts per litre with the ratio K/Na =0.2 of potassium (K) and for sodium (Na) it was 0.2. The results indicated a small, positive influence of the water salinity on the tests results, as the effective cohesion was slightly higher (10.5–12.5 $kN.m^{-2}$) than for sample No 2 (9 $kN.m^{-2}$).

However, from the practical point of view the main conclusion is that the character of the drained and undrained tensile tests for the compacted or pre-consolidated samples is very different. The behaviour for the undrained tests is close to the brittle character, while the elongation at failure is rather small compared to the elongation for drained tests, which is roughly 10 times higher, in the range 2–6 %, while the effective tensile strength is roughly 10 times lower, between 3 and 8 kPa. This means that during drained loading the material is much more flexible and the probability of the development of tensile cracks is significantly lower.

5 APPLICATION OF THE RESULTS ON SPECIFIC EARTH STRUCTURES

In the Introduction some examples were mentioned where the tensile zones and the tensile cracks can be expected and can play a very negative role in the behaviour of these earth structures. The three basic such examples are described in more detail.

5.1 EARTHFILL AND ROCKFILL DAMS

The problem of tensile cracks in the sealing part of fill dams is probably the most sensitive and the most discussed. L. Šuklje in his book mentions "A thorough analysis of the tensile strain states is needed when constructing the clay cores of earth dams. Tensile fissures in such cores can represent a dangerous starting point for the erosive action of seepage water and, therefore, they have to be avoided". However, in the same year, Casagrande [29] expressed his views on the increasing height of dams. He thinks that in the case of high rock dams in valleys with steep sides it is impossible to avoid the tensile zones and transversal cracks in the crest of a dam that is near the sides of a valley. It does not depend on the kind of building material. So it is necessary to protect the dam against the effects of cracks.

Tensile cracks can be initiated either by differential settlement or by hydraulic fracturing. In some cases seismic effects and desiccation also have to be taken into account. Transversal cracks in the direction of the seepage path are the most dangerous. The transversal cracks in a dam crest are usually caused by the differential settlement of the dam body. The advantage is that these cracks are observable, while the internal transversal cracks caused by hydraulic fracturing are not, and therefore they are much more dangerous.

Numerical methods, especially when the results of the tensile tests are utilized, can be a very useful tool for tensile zone prediction and specification. A subsequent parametrical study can give a better view of these zones if different types of soils are used or the selected soil is modified, e.g., with respect to the initial moisture content. In any case, the geometrical profile of the dam body can be rearranged as well.

When tensile crack development cannot be avoided, attention must be concentrated on the crack behaviour when the water starts to seep through it. A high swelling potential plays a positive role, while the high susceptibility to erosion plays a negative one. Therefore, the overall approach to the design of fill dams has been changing over recent decades. The question connected with the possibility of crack development is more important than the problem of slope stability. Therefore, a new, logical scheme for the design of fill dams was proposed by Whitman [30] or in modified form by Vaníček [31].

Vaníček [25] also describes the steps that were performed when the measurement in the Dalešice dam body showed a greater elongation than that measured during the laboratory tensile tests.

From the practical point of view the results from the undrained and drained tests can be used with respect to the dam construction speed or the speed of reservoir filling. For example, transversal cracks at the dam crest are very sensitive to the speed of reservoir filing, as this filling causes deformation of the upstream stabilization zone by its saturation.

5.2 SANITARY LANDFILLS

Due to the different physical, chemical and biological processes inside the deposited material, the settlement of the landfill surface is sometimes very high. In previous times it has very often been pointed out that surface clay sealing systems can embody differential settlement, causing tensile cracks, see Jessberger and Stone [3], and Daniel [4]. As an example, a crater with a diameter $\Delta L = 5$ m with the maximum depression $\Delta s = 0.25–0.5$ m is mentioned as a typical case observed on the landfill surface, see Vaníček [5].

This differential settlement corresponds to an elongation of 0.1–1.0 %, which can cause tensile crack development with preferential infiltration into the landfill body. Daniel, for example, indicates that many, if not all, covers for municipal solid waste landfills have areas with a distortion of this magnitude or larger. This is also one of Daniel's arguments for giving preference to a geosynthetic clay liner (GCL) in the capping system.

But on the basis of his own experience with tensile tests, the author is not so sceptical towards the utilization of clay liners in landfills, mainly for the following reasons:

- The maximum tensile elongation for cohesive soils compacted for optimum moisture content according to the Proctor standard test is really in the range 0.1–1.0%, but grows with a moisture-content increase. Because the compaction of the clay liners is usually performed for a moisture content higher than optimum (due to the decrease of permeability), this aspect is on the positive side.
- Up to now, the mentioned results were obtained for tensile tests that can be labelled as undrained tests. For the real conditions the crater development on the landfill surface is not such a quick process, it is time dependent. For the capping clay liners, partly drained conditions can be expected and therefore also a lower probability of tensile crack development.
- The last chance for improvement is the swelling potential – after the potential opening of the tensile crack and the first water infiltration through it, the crack can be closed by the swelling potential of the clay minerals as was also mentioned for the cracks in the fill dam body.

Therefore, we can conclude that a potential risk of tensile crack development exists, but it is not as high as was believed at the beginning of the last decade.

The possibility of tensile crack development in the capping clay liner by desiccation from the bottom, as mentioned, e.g., by Daniel [4], is still a matter of discussion.

5.3 RETAINING WALLS FROM REINFORCED SOIL

Most geotechnical engineers count on the possibility of tensile-zone development at the top of the slope, which can finally lead to tensile crack opening. Fig.17 shows the results of step-by-step numerical modelling of the pit excavation performed by Dunlop and Duncan (1970) [32], where the first tensile zones were modelled, even for a slope stability that is higher than usually demanded $F = 1.5$.

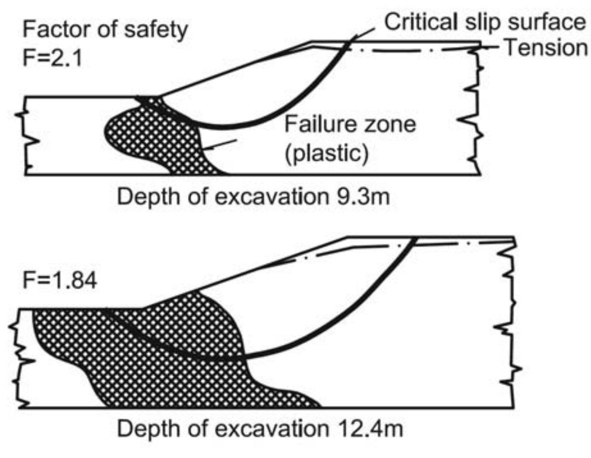

Figure 17. Development of the failure (plastic–tensile) zone during the process of excavation.

However, a similar case was observed for a retaining wall made from reinforced soil, see Fig. 3.

In fact the retaining wall from reinforced soil is a quasi-homogeneous gravity wall, for which different limit states of failure have to be checked. Therefore, a great attention is devoted to the ultimate limit state of failure along slip surfaces passing, not only through the reinforced part (the so-called internal stability) but also behind the zone of reinforcement (external stability). The limit state of overturning is often neglected. Nevertheless, this limit state can be very important, especially in the case that the tensile cracks are opened and filled with water.

For the case shown in Fig. 3 the wall was about 10 m high, the length of the reinforcing elements (geogrids) were in the upper part about 7 m. The soil used for this part was similar to the rest of the embankment; however, this soil was lime stabilized, and therefore the stiffness of this quasi-homogeneous block was much higher than the surrounding soil.

This different stiffness together with the different settlement of the quasi-homogeneous block plays the most important role in crack development just behind the zone of reinforcement. The differential settlement of this block is caused by the different settlement of the block corners. For the outer corner, where the 3D deformation prevails, the settlement is always higher than for the inner corner, where it is roughly a 1D deformation, see Fig. 18. In the given case there were additional factors, the ground behind the outer corner was inclined, and just below this corner some backfilled pipes were situated.

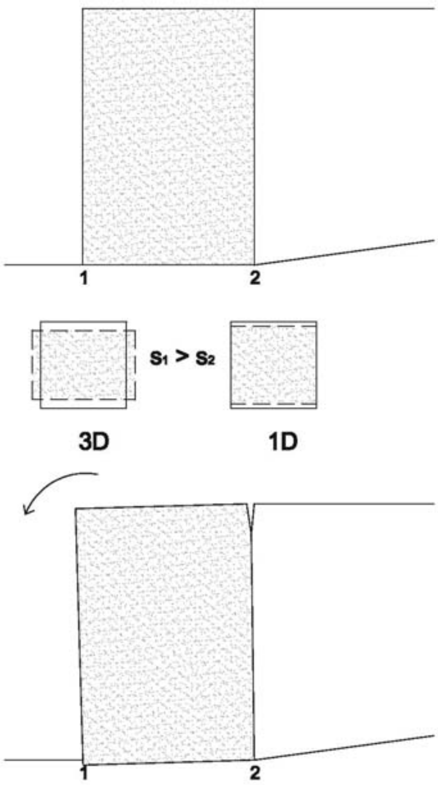

Figure 18. Deformation of the quasi-homogeneous block from the reinforced soil.

6 CONCLUSION

The main purpose of this paper is to show the importance of the tensile characteristics of soils. The interest in this problem over the last half century has a sinusoidal character, and the peaks of interest were connected with tensile zones or cracks for soil slopes, for cracks in the sealing layers of fill dams, cracks in the capping clay liner of sanitary landfill and, finally, with the cracks behind the zone of reinforcement for the retaining walls from reinforced soil.

The author therefore presented his practical experience, either with tensile tests or with practical applications for the cases mentioned above.

The results of the undrained bending tests performed on compacted clays used for the dam clay core show that the tensile characteristics are strongly influenced by the capillary forces, the tensile strength is in the range 30–80 kPa and the maximum elongation at failure is about 0.2–0.6 % and can be significantly influenced by the moisture content or by the compaction effort. The results of the triaxial drained test show very different behaviour: the effective tensile strength is about 10 times lower and the maximum elongation about 10 times higher. Even if the tensile effective strength is relatively small, this nevertheless shows in the internal forces between the individual clay particles. The salinity of the pore water has a positive effect on the small strength increase. However, the most important factor is the stress-strain curve character (shape), showing in the stress-strain hardening. After the first peak was exceeded (probably as a result of shearing in the weakest point), the stress went up again (probably as the result of resistance against particle separation in this weakest point) causing shear failure at the second weakest point.

The summary shows that tensile tests deserve more attention in the future. There is a lot of opportunity for new findings, especially when the testing devices and monitoring possibilities are rapidly improving with time.

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