prof.dr. Ivan Vaníček

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Univerzitetna izobrazba in profesionalna pot prof. Vaníčka sta tesno povezani z Oddelkom za geotehniko Češke tehniške univerze v Pragi, kjer je leta 1988 postal redni profesor. Med svojo profesionalno kariero je sodeloval tudi s tujimi univerzami: izkušnje je med letoma 1975 in 1976 nabiral na gostovanju na Imperial College-u v Veliki Britaniji pri prof. Bishopu, v Rusiji je gostoval 1 mesec v letu 1978 pri prof. Tsytovitchu in 1 mesec v letu 1983 pri prof. Ivanovu.

Poleg številnih člankov in monografij je svoje raziskovalno delo predstavil v različnih disertacijah in delih, potrebnih za akademska napredovanja: magistrsko delo v letu 1967, doktorska disertacija v letu 1975, zaključen podiplomski študij na Imperial College-u v letu 1977, izredna profesura v letu 1980, doktorat Češke akademije znanosti v letu 1985. Raziskovalne dejavnosti so povezane predvsem z nasutimi zemeljskimi pregradami in visokovodnimi nasipi, z okoljsko geotehniko (odlagališča industrijskih odpadkov, jalovišča, odlagališča komunalnih odpadkov, podzemna odlagališča) in z armiranjem zemljin z geosintetiki. Trenutno je v domačem okolju zadolžen za široko področje raziskav kot je na primer projekt "Vzdržne konstrukcije", ki ga financira Ministrstvo za šolstvo. Na ravni Evropske unije sodeluje v okviru Evropskega znanstvenega sklada na projektu "Staranje podzemnih konstrukcij", v okviru katerega se izvaja monitoring instrumentiranih podzemnih železnic v Pragi, Londonu in Barceloni – (CTU Press, 2008 in 2011).

Na pedagoškem področju pokriva področja Mehanike tal, Okoljske geotehnike in

His higher education and professional carrier is connected with the Geotechnical Department of the Czech Technical University in Prague, where he obtained full professorship at 1988. During his professional carrier he also gained foreign experiences - in UK at the Imperial College as Academic visitor, where between 1975 and 1976 he cooperated with Prof. Bishop or in Russia, where he cooperated with Prof. Tsytovitch (1978) and with Prof. Ivanov (1983) each time 1 month.

His research activities were presented apart from papers and monographs within different thesis and habilitation works for: MSc. in 1967; PhD. in 1975; DIC in 1977; Assoc. Prof. in 1980; DSc. in 1985. Research activities were strongly connected with embankment dams, dykes; with environmental geotechnics (spoil heaps, tailing dams, landfills, underground repositories) and finally with soil reinforcement with the help of geosynthetics. At the present time he is responsible on the domestic level for wider research activities as is project by Ministry of Education - "Sustainable Construction". On the European level the research cooperation proceeds under the frame of European Scientific Fund, within which metros in Prague, London and Barcelona are instrumented and monitored – "Ageing of underground structures – Prague Metro", CTU Press, 2008 and 2011.

For the teaching activities he completely elaborated subjects as Soil Mechanics, Environmental Geotechnics and Earth Structures. From nearly 20 textbooks and monographs he wrote or co-authored it is

Zemeljskih konstrukcij. Prof. Vaníček je avtor ali so-avtor skoraj 20 knjig in monografij, iz katerih lahko izdvojimo naslednje: Mehanika tal – 1982, Natezne razpoke v nasutih zemeljskih pregradah – 1987, Okoljska geotehnika – 1991, Projektiranje armiranih zemljin po metodi mejnih stanj – 2000, Sanacija odlagališč odpadkov in ekoloških bremen – 2002 (vsa dela v češkem jeziku) in nova knjiga Zemeljske konstrukcije, ki jo je leta 2008 založil Springer. Navedena dela temeljijo na raziskovalni dejavnosti in strokovnem delu tako na Češki tehniški univerzi kot tudi v njegovem konzultantskem podjetju "Geotechconsult".

Aktivnosti v pedagoških in strokovnih združenjih je pričel po letu 1989. V letu 1991 je prof. Vaníček osnoval Češko geotehniško društvo (trenutno je predsednik društva), ki je del Češke zbornice gradbenih inženirjev (prof. Vaníček je član predsedstva tega združenja). V letu 1997 je bil izvoljen za predsedujočega češkemu in slovaškemu nacionalnemu komiteju ISSMGE. Od začetka 90-ih let se je udeležil skoraj vseh ISSMGE konferenc – svetovnih, evropskih, Podonavskih – kot tudi mednarodnih kongresov s področja Okoljske geotehnike. Na marsikateri konferenci je bil v vlogi vabljenih predavateljev. Leta 2003 je bil glavni organizator Evropske konference ISSMGE v Pragi. S tem dogodkom se je začela krepiti povezava prof. Vaníčka z ISSMGE – v okviru sestankov Sveta ISSMGE, gostujočih predavanj (2007 v Albaniji), postal je predstavnik ISSMGE v skupni evropski delovni skupini "Strokovna dolžnost, odgovornost in sodelovanje v geotehniki", itd. Tesne odnose goji prof. Vaníček s predstavniki posameznih držav v združenju ELGIP.

V letu 2009 je bil izvoljen za podpredsednika ISSMGE za Evropo, s čimer je zaključil svoje delo v češkem in slovaškem nacionalnem komiteju.

Ivan je poročen z Naďo, s katero ima dva odrasla sinova - Martina in Jiříja. Oba inženirja, geotehnični inženir in inženir hidrotehnike delujeta v lastnem konzultantskem podjetju Geosyntetika Ltd., pri čemur seveda sodelujeta tudi z očetom.

possible to mention some of them: Soil Mechanics - 1982, Tensile cracks behaviour in embankment dams - 1987, Environmental Geotechnics - 1991, Limit state design of reinforced soils - 2000, Remediation of old landfills and ecological burdens - 2002, (all in Czech) and new book "Earth Structures", which was published by Springer in 2008. These publications are also strongly based on his own research and consulting activities, which are performed either via Czech Technical University or via his own consulting firm "Geotechconsult".

His activities in learned and professional societies started mostly after 1989. In 1991 he founded the Czech Geotechnical Society, (currently chairman), which is part of the Czech Institution of Civil Engineers (member of CICE presidium). In 1997 he was elected as a chairman of the Czech and Slovak National Committee of ISSMGE. From the beginning of nineties he joined practically all conferences of ISSMGE – International, European, Danube European as well international congresses of Environmental Geotechnics. At many of these conferences he played some key role. In 2003 arranged European Conference ISSMGE in Prague. From these days his connection with ISSMGE is increasing – in the frame of Council Meetings, Touring Lectures (2007 in Albania), representing ISSMGE in Joint European Working Group "Professional Task, Responsibilities and Co-operation in Geo-Engineering", etc. Very close cooperation he also has with representatives of individual countries from the ELGIP platform.

In 2009 he was elected as Vice-president ISSMGE for Europe and ended his position in Czech and Slovak National Committee.

Ivan is married to Naďa; they have two adult sons – Martin and Jiří, both civil – geotechnical and hydro engineers, working in their own consulting firm Geosyntetika, Ltd., nevertheless they of course cooperate with dad.

Ivan VANÍČEK
Czech Technical University in Prague

IMPORTANCE OF TENSILE STRENGTH IN GEOTECHNICAL ENGINEERING

ABSTRACT: In many soil mechanics textbooks only limited information about tensile characteristics can be found. The Šuklje book “Rheological aspects of soil mechanics” is a small exception to this statement as he devoted to this problems special chapter “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 the soil behaviour in tension. Tensile tests are briefly described, some results as well, namely with distinction between undrained and drained tests. Practical examples of 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 are giving more information about bonds between individual particles some theoretical aspects of these tests are discussed as well.

POVZETEK: V mnogih učbenikih mehanike tal je mogoče najti le omejene informacije o nateznih lastnostih zemljin. Majhna izjema te trditve je knjiga dr. Luja Šukljeta "Reološki vidiki mehanike tal", v kateri je posvetil temu problemu posebno poglavje "Natezna in upogibna trdnost zemljin". Zato ni naključje, da je predavanje 13. Šukljetovega dne namenjeno obnašanju zemljin v območjih nateznih napetosti. Na kratko so opisani natezni testi kot tudi nekaj rezultatov z razlikovanjem med nedreniranimi in dreniranimi preizkusi. Obravnavani so praktični primeri uporabe rezultatov, najprej za primere, ko je mogoče pričakovati razvoj nateznih razpok. Ker rezultati dreniranih preizkusov dajejo več informacij o vezeh med posameznimi delci, so obravnavani tudi nekateri teoretični vidiki teh preiskav.

INTRODUCTION

The behaviour of soils in tension is the object of interest not only for geotechnical engineers but also for other branches of engineering, 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 tensile strength of soils is very often connected with different tensile cracks which can develop in earth structures as embankment dams, slopes, retaining walls from reinforced soil, or with capping clay sealing system of sanitary landfills, e.g. (Vaníček, 2011). Some examples are presented in following figures. Fig. 1 is showing tensile crack which developed close to the rockfill dam crest parallel to its longitudinal axis. Very often some tensile crack can be observed at the top of the slope as first signal of the potential danger of slope stability problem, Fig. 2.

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

Fig. 2. Tensile crack at the upper part of slope.

Large tensile crack was also observed for high retaining wall from reinforced soil, very closely behind the zone of reinforcement, Fig. 3. Water flowing into this crack started the process of wall overturning, as was observed from the shape of quasi-homogeneous reinforced part of the wall.

In nineties a great attention was devoted to the possibility of the tensile crack development in the capping sealing system of the landfills, e.g. (Jessberger&Stone, 1991; Daniel, 1995). Due to differential settlement of the deposited waste, local depression can develop there with high possibility of tensile cracks development. These tensile cracks can influence, in negative sense, the sealing function of this capping sealing system, Fig. 4 (Vaníček, 2002).

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

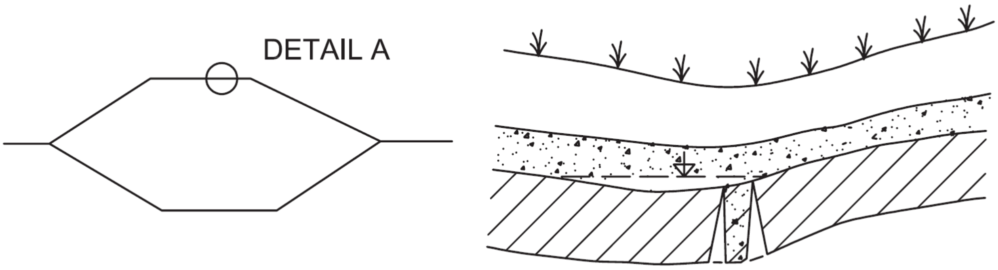

Fig. 4. Deformation of the landfill surface – local differential settlements affecting functionality of capping clay liner.

The tensile tests and the obtained results are very useful tool for the recognition of the probability of tensile cracks development, either from the view of tensile strength, 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. Mainly modulus of deformation in tension (or extension) determined from these tests can help to improve the knowledge about the tensile zone widening.

Fig. 5. Foot prints of astronaut boots on the Moon surface with vertical wall.

However the tensile tests can also help to improve our knowledge from the theoretical point of view, what failure criteria are more general or what forces between individual soil particles are affecting soil structure arrangement, e.g. (Rosenquist, 1959; Bishop and Garga, 1969; Parry and Nadarajan, 1974).
Especially with respect to the orientation of soil particles and what is the nature of the forces between adjacent soil particles. It is obvious that there is a certain relation between effective tensile strength and effective cohesion. Very often the 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 with a certain disagreement with observation of material at the Moon surface; where no water is supposed to be there, and so forth no inter particles forces where the contact is water – mineral, which are very often attributed for measured effective cohesion. Fig. 5 shows the foot prints of astronaut boots, where walls are vertical. The same was observed for the walls of cuts excavated by small dredging-machine on the moon surface.

TYPES OF TENSILE TESTS

In the literature the description of the different tensile tests which varied in different way can be found. There is no unification as for other soil mechanics tests like shear or compression tests (Vaníček, 1977c). If the dimensions of the tested samples, manner of preparation or time effect are not taken into account, the principal classification can be done by:

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

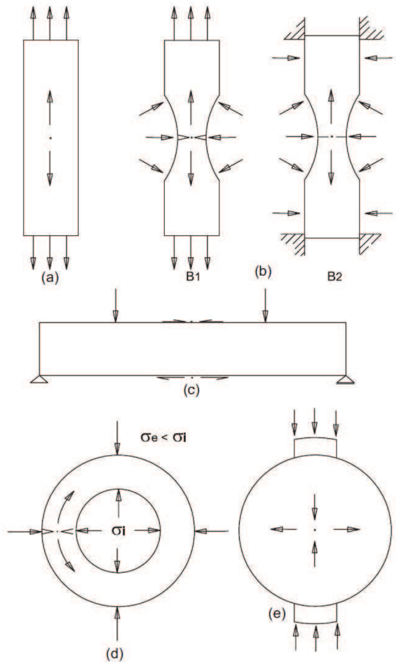

Fig. 6. Division of tensile tests according to principle of loading. A) Axial tensile tests (direct tension test). B) Triaxial tensile test. C) Bending test. D) Test on hollow cylinder. E) Indirect (Brazilian) tensile test.

For the axial tensile test the most problems are, according to Šuklje (1969), connected with the uniform distribution of tensile stresses in the test section of the specimen. For the tested samples with uniform cross section, different types of connection were applied, e.g. freezing – (Haefeli, 1951) or glueing – (Zeh and Witt, 2007).

For samples with wider ends (briquette) some sort of friction via connection clamps were 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 stresses in the transition part of the specimen. Probably largest samples were tested by Tschebotariof and De Phillipe (1953). 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 (1966). For unconfined tests mentioned by Hasegawa and Ikeuti (10) the weight was balanced by testing on the mercury surface. Ajaz and Parry (1975a) 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 tension specimen.

Triaxial tension test of the type B1 will be described later on for drained tests. Test of the type B2 was described by Ter-Martirosjan (1973, 1977) .Specificity of this test is the possibility to model different stress state depending upon the ratio of the cross section at the end (AE) of the sample to the cross section in the centre part of the sample (AC). When stiff sleeve is applied for the central part, the unconfined tensile test is modelled.

Some specificity of tests on samples having the form of hollow cylinders and subjected to various internal and external hydrostatic pressures were described by Šuklje (1969), respectively by Šuklje and Drnovšek (1965):

- well-defined stress states without uncontrolled stress concentration;
- the possibility of investigating the deformability and strength at various stress states,
- the possibility of taking into account the deformation anisotropy,
- a 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.

Principle of the bending test consists in loading of the tested soil beam by 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 bending moment is constant. It is typical case of pure bending. Outermost fibres are either in tension or in compression.

Indirect (Brazilian) test is more often used in rock mechanics, as the sample is easily prepared from the obtained core drill and load transfer is not so difficult. Nevertheless some results were published also with respect to soil samples as e.g. Narain and Rawat (1970) or Krishnayya et al (1974).

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

Results of undrained tensile tests (bending tests)

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

Bending test arrangement and evaluation

Bending tests were mostly conducted for the possibility 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, 1963; Vaníček, 1975).

Bending tests results can be recalculated by different ways according to the specific theory as, Fig. 7:
Theory of elasticity assumes that the deformation of the outermost fibres is the same and same are also stresses in these outermost fibres. Neutral axis lies in the centre of the sample.
Navier´s hypothesis assumes planar deformation in the cross section of the tested sample (however neutral axis is above the centre of gravity of the profile) and also stresses are linear to the unit deformation.
Differential method assumes also planar deformation in the cross section of the tested sample however this method is not based on any preferred stress – strain law.

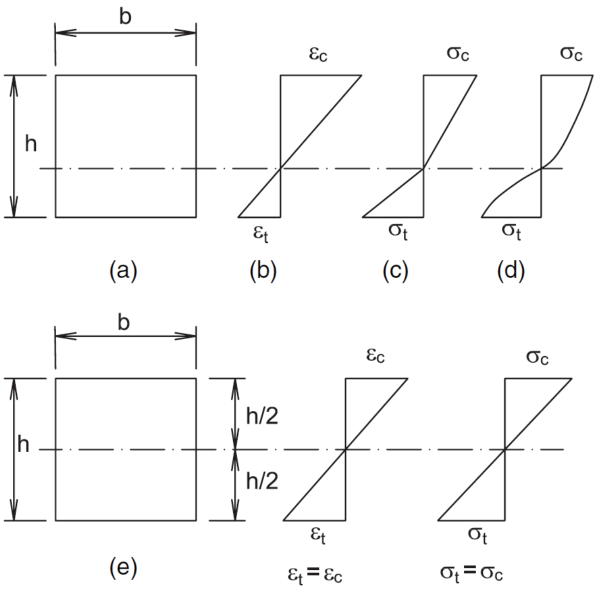

Fig. 7. Bending test – fundamental assumptions. a) Beam cross section. b) Strain diagram. c) Stress diagram for Navier´s hypothesis (direct method). d) Stress diagram for differential method. e) Elastic bending theory.

The first one was used by Leonards and Narain, the second one by Vaníček and the last one by Ajaz and Parry (1975b). Vaníček take 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 pins determined in a photo-comparators. The results of tests made by Šuklje proved that the strain plots are still approximately linear at 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 strains within the beam.

The layout of the measuring device which was used by the author is presented in Fig. 8 (Vaníček, 1977a).

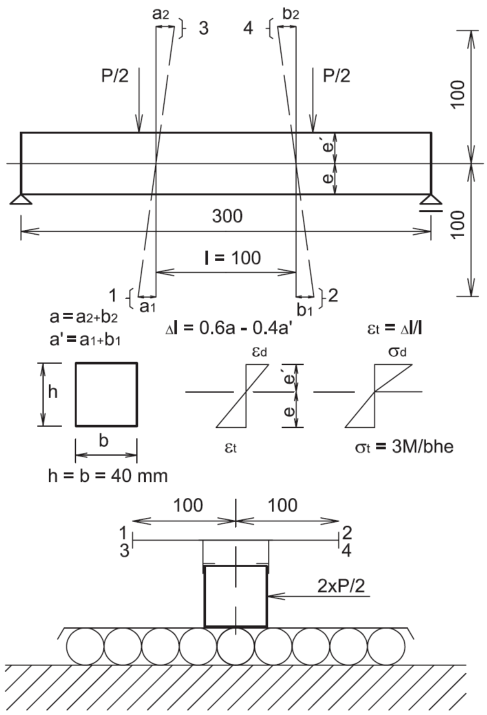

Fig. 8. The layout of the arrangement and evaluation of bending test proposed by author.

The beam 300 mm long and 40 × 40 mm in cross-section is loaded in horizontal direction. Weight of the beam and friction are eliminated by means of horizontal ball-bearings. In the central part of the beam which deforms under constant bending moment are fixed 2 detectors connected with the beam. Deformation is measured at their ends, with roughly 5× magnification against the deformation of outermost fibres. Soil-water mix was compressed in 4 layers in special mould for dry density and moisture content determined from the Proctor standard test. The beam was coated with hot mixture of oil and paraffin. Loading was increased under constant rate in the direction of the layers.

Factors influencing undrained tensile characteristics

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

The influence of moisture content

Generally the tensile strength of the tested soils decreased with an increase in moisture content. For small changes around the optimum moisture content this relationship was nearly linear. Fig. 9 shows the influence of moisture content for wider range. Result was obtained for material of the clay core from Dalešice dam (Vaníček, 1982).

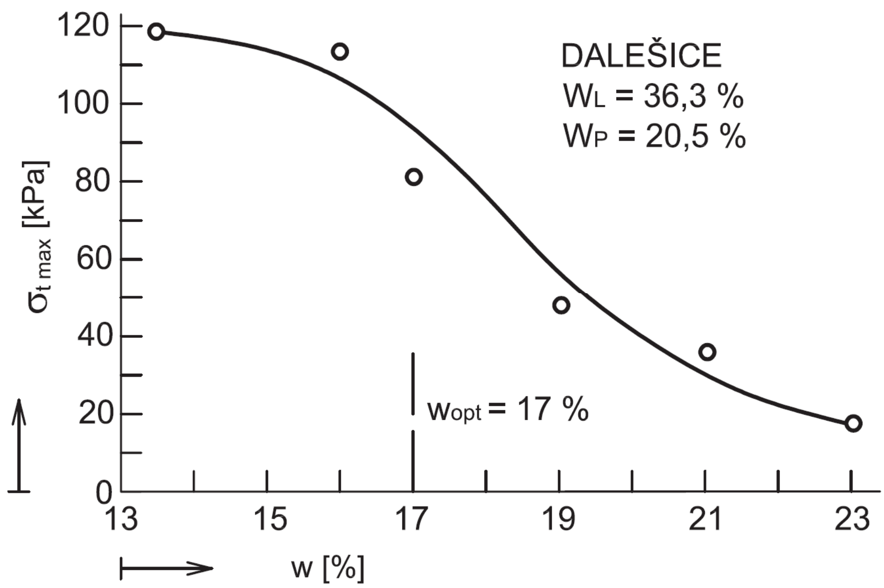

Fig. 9. Tensile strength as function of moisture content for clay core from Dalešice dam. Influence of moisture content on tensile strain at failure.

The maximum value of tensile strength is reached for w = w_{opt} – 3.5 %. The index of plasticity was used for the comparison of different tests and soils compacted at optimum moisture content, Fig. 10 (Vaníček I. and Vaníček M., 2008).

The increase in 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 of 30 - 80 kN/m^2. Similar results were obtained by Narain and Rawat (1970) and also by Ajaz and Parry (1975a). From these results it can be seen that the undrained tensile strength is mostly caused by capillary forces.

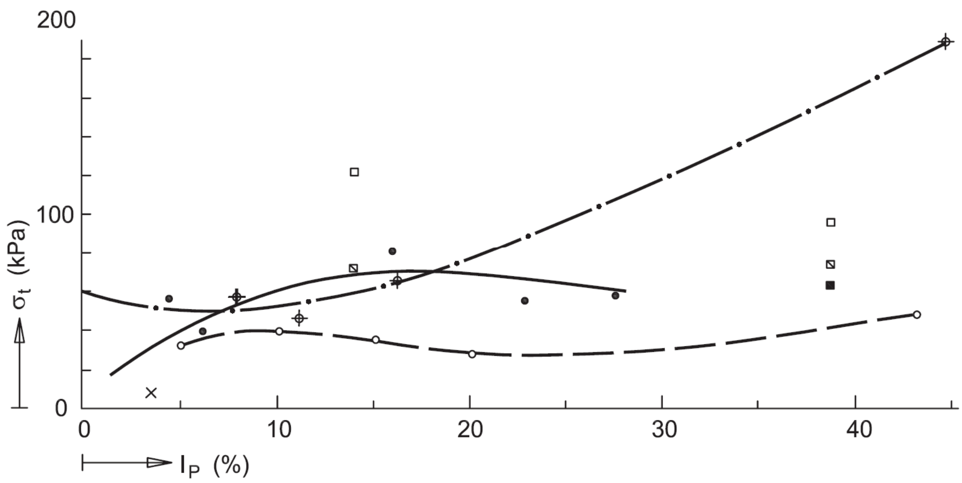

Fig. 10. Results of tensile strength for different tests and soils compacted by Proctor standard at optimum moisture content.

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

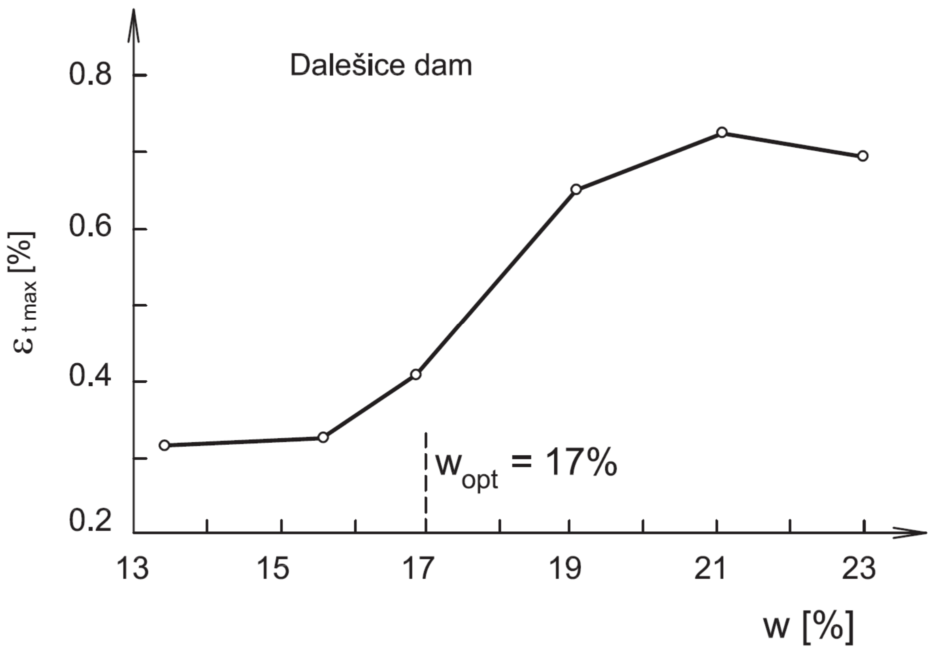

Fig. 11. Influence of moisture content on tensile strain at failure.

Again to compare the results from different authors and different soils the index of plasticity I_P was used, see Fig. 12 – Vaníček (1977b), Vaníček I. and Vaníček M. (2008).

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

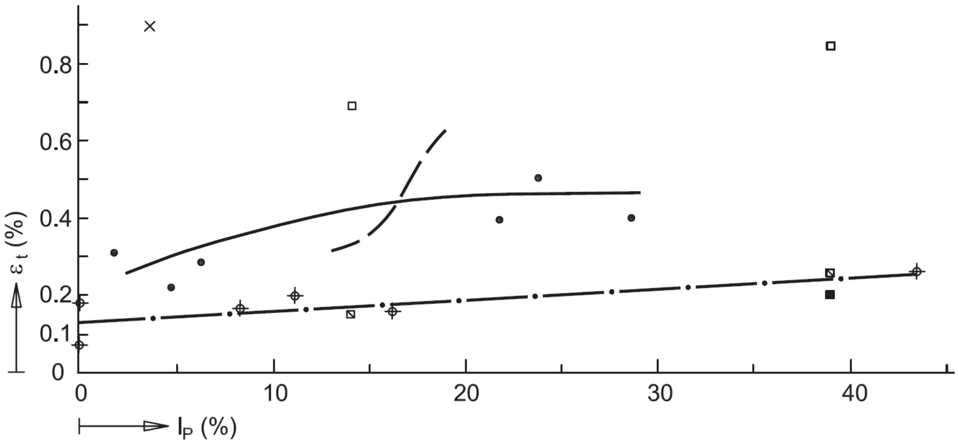

Fig. 12. Results of tensile strain at failure for different tests and soils compacted by Proctor standard at optimum moisture content.

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

The influence of compaction effort

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

The influence of time

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

Results of drained tensile tests

Drained tests arrangement

Bishop and Garga (1969) described the first triaxial drained tension test without the use of end clamps, type B1 on 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 sample drops to zero. The axial effective stress throughout the centre section will at this point be negative (i.e. in tension), 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 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%) either on undisturbed samples, carefully sampled in situ, or on a remoulded sample. Differences in results are prominent. For the undisturbed samples, the measured effective tensile strength was in the range of 26.3 to 33.3 kN/m^2, whereas for remoulded samples practically zero. Time to failure for the undisturbed samples was in the range of 6.7 to 55.2 hours and tensile strain at failure in the range of 2.19 to 16.7%.

Some fundamental findings can be briefly summarized as:

- Failure of the sample has a 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 rather 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 hydraulic triaxial apparatus described by Bishop and Wesley (1975), see Fig.13.

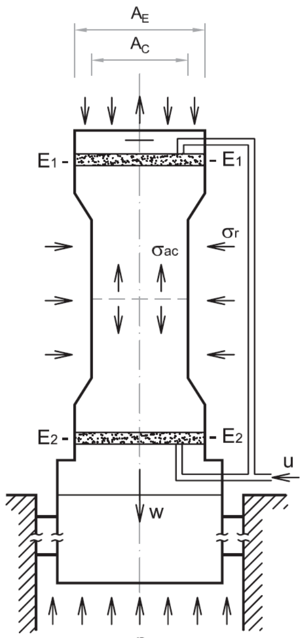

Fig. 13. Layout of the drained triaxial tension tests performed in the hydraulic triaxial apparatus. σ_r – radial stress; p – stress in loading cell; u – back pressure applied to the drainage connection; W – weight of piston; σ_{ac} - axial stress on the centre section; AE – area of end section; AC –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

During the first series the clay from the valley slopes downstream of Cod Beck Dam was used to study the behaviour of plastic clay material in a range of small compression and tension stresses and to study the influence of salinity of pore water on this behaviour (Vaníček, 1977c).

This clay (liquidity limit w_L = 44.1 %, plasticity limit w_P = 18.5 %, plasticity index Ip = 25.6 %) was deposited in a fresh water lake and minimum of salts were supposed to be in the pore water.

Remoulded samples, soil-water mix were compacted by hand into special mould, with help of wooden stick. Samples were consolidated in the triaxial apparatus up to the consolidation pressure 600 kN/m^2. However tests started when the difference between cell pressure and 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, great attention was devoted to the determination rate of loading, 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. An average value from the three lowest results is 0.07 cm^2/min (0.01 m^2 per day). Calculated required time to failure was in all cases slightly lower than real time to failure, which was between 30 to 40 hours.

A second more extensive series of tests were performed on Most clay (w_L = 53.1 %, w_P = 25.4 %, Ip = 26.7 %). Dry pulverized clay was mixed with water to 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 ends of consolidation cylindrical samples were cut out with a diameter of 38.1 mm (1.5").

Tests arrangements are described in more details by Vaníček, I. and Vaníček, M. (2008).

Discussion of the obtained results

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

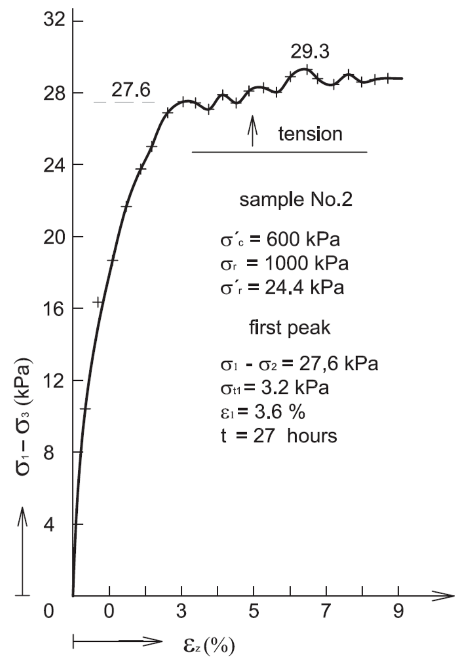

Fig. 14. The typical result of drained triaxial tension test.

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

The character of the stress-strain curve is similar to the stress-strain curve of steel in tension. We can speak about 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 rearrangement of the clay particles in this zone – the tensile loading began and the stress-strain work hardening behaviour of plastic clay was valid for this loading. It makes it possible to develop another shear failure at a different point.

Tests also helped 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 normal pressures, see Fig.16.

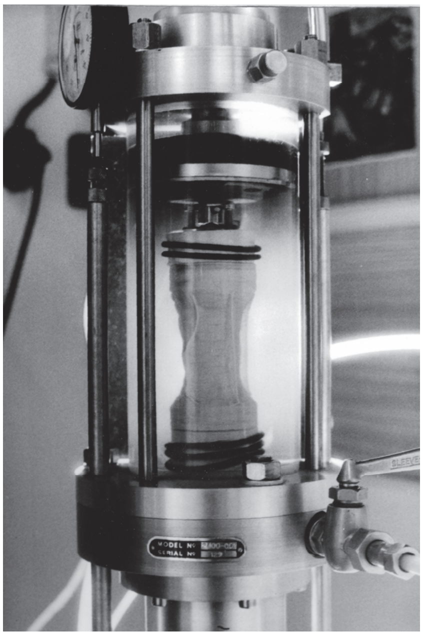

Fig. 15. Photo of tested Most clay with visible two necks

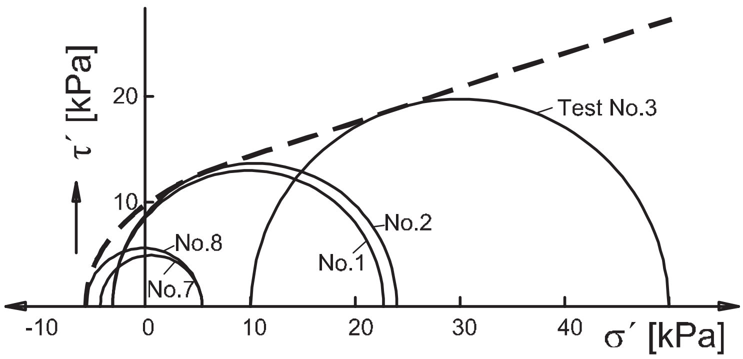

Fig. 16. Mohr's circles for triaxial compression and tension tests (for first failure).

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. Pore water for sample No. 6 contained 5 g of salts per litre with ration K/Na = 0.2 of potassium (K) and sodium (Na) was 0.2. The results indicated small positive influence of the water salinity on the tests results, as effective cohesion was little bit 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 drained and undrained tensile tests for compacted or pre-consolidated samples is very different. Behaviour for undrained tests is close to the brittle character, elongation at failure is rather small compared to the elongation for drained tests, which is roughly 10 times higher, in the range of 2 – 6 %, while the effective tensile strength is roughly 10 times lower, between 3 and 8 kPa. So it means that during drained loading the material is much more flexible and the probability of development of tensile cracks is significantly lower.

Application of results on specific earth structures

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

Earthfill and rockfill dams

Problem of tensile cracks in the sealing part of fill dams is probably most sensitive and most discussed. L.Šuklje in his book is mentioning: "A thorough analysis of the tensile strain states is needed when constructing 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 at the same year, Casagrande (1969) expressed his view on an 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 which is near the sides of 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 also seismic effect and desiccation have to be taken into account. Transversal cracks in the direction of seepage path are most dangerous. Transversal cracks in a dam crest are usually caused by 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 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. Subsequent parametrical study can give better view on these zones if different types of soils are used or the selected soil is modified, e.g. with the respect of initial moisture content. Certainly the geometric profile of the dam body can be rearranged as well.

When the tensile cracks development cannot be avoided, the attention must be concentrated on the crack behaviour when water starts to seep through it. High swelling potential is playing positive role while the high susceptibility to erosion the negative one. Therefore the overall approach to the design of fill dams is changing during last decades. The question connected with the possibility of crack development is more important than the problem of slope stability. New logical scheme for the fill dams design were therefore proposed by Whitman (1984) or in modification form by the author (Vaníček, 1988).

Author (Vaníček, 1982) also describes the steps which were performed when the measurement in the Dalešice dam body showed larger elongation then that measured during laboratory tensile tests.

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

Sanitary landfills

Due to the different physical, chemical and biological processes inside deposited material, 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 (Jessberger and Stone, 1991; Daniel, 1995). As an example a crater with the dia ΔL = 5 m with maximum depression $\Delta s = 0.25 – 0.5$ m is mentioned as a typical case observed on the landfill surface (Vaníček, 2002).

This differential settlement corresponds to the elongation of 0.1 to 1.0 %, which can cause the tensile crack development with preferential infiltration into landfill body. Daniel (1995) for example indicates „that many, if not all, covers for municipal solid waste landfills have areas with distortion of this magnitude or larger". This is also one of Daniel's arguments for giving a preference to the GCL – geosynthetic clay liner in the capping system.

But on the basis of own experience with tensile tests the author is not so sceptical towards the utilization of clay liners in landfills, mainly for the following reasons:
Maximum tensile elongation for cohesive soils compacted for optimum moisture content according to the Proctor standard test is really in the range of 0.1 – 1.0%, but grows with moisture content increase. Because the compaction of the clay liners is usually performed for moisture content higher than optimum (due to decrease of permeability), this aspect is on the positive side.

Up to now mentioned results were obtained for the tensile tests which can be labelled as undrained tests. For the real conditions the crater development on the landfill surface is not such a quick process, is time dependant. For the capping clay liners partly drained conditions can be expected and therefore also the lower probability of tensile cracks development.

Last chance for improvement is the swelling potential – after the potential opening of the tensile crack and first water infiltration through it the crack can be closed by the swelling potential of clay minerals as was mentioned also for cracks in the fill dam body.

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

The possibility of tensile cracks development in capping clay liner by desiccation from the bottom, as mentioned e.g. by Daniel (1995), is still matter of discussion.

Retaining walls from reinforced soil

Most of the geotechnical engineers count with the possibility of tensile zone development at the top of the slope, which can finally lead to the tensile cracks opening. Fig.17 is showing the results of step by step numerical modelling of the pit excavation performed by Dunlop and Duncan (1970), where first tensile zone were modelled even for slope stability higher than usually demanded F = 1.5.

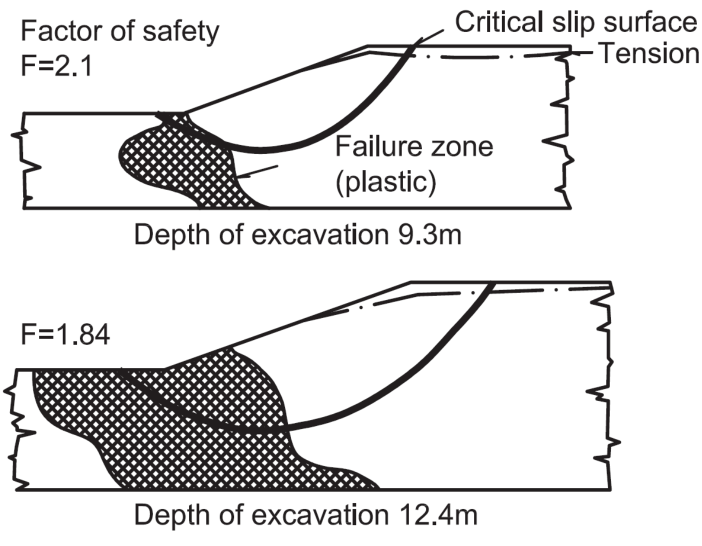

Fig. 17. Development of failure (plastic – tensile) zone during the process of excavation.

However the similar case was observed for the retaining wall from reinforced soil, see Fig. 3. In fact the retaining wall from reinforced soil is 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 (so called internal stability) but also behind the zone of reinforcement (external stability). Limit state of overturning is often neglected. Nevertheless this limit state can be very important especially in the case that tensile cracks are opened and filled by water.

For the case displayed in Fig. 3 the wall was about 10 m high, length of reinforcing elements (geogrids) were in the upper part about 7 m. Material used for this part was similar as for the rest of the embankment, however was lime stabilized there, so that the stiffness of this quasi-homogeneous block was much higher than surrounding soil.

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

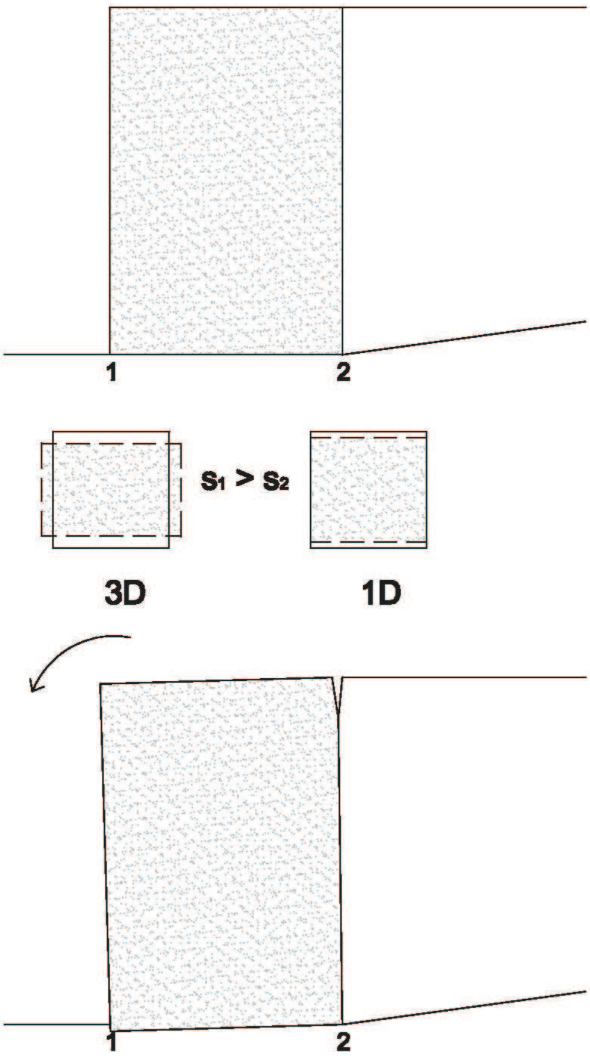

Fig. 18. Deformation of the quasi-homogeneous block from the reinforced soil.

CONCLUSION

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

The author therefore presented his practical experience either with tensile tests performance but also with practical application on cases mentioned above.

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 capillary forces, tensile strength is in the range of 30 – 80 kPa and maximum elongation at failure about 0.2 – 0.6 % and can be significantly influenced by moisture content or by compaction effort. The results of triaxial drained test show very different behaviour, effective tensile strength is about 10 times lower and maximum elongation about 10 times higher. Even if the tensile effective strength is relatively small nevertheless shows on internal forces between individual clay particles. The salinity of the pore water has positive effect on small strength increase. However what is the most important is the stress-strain curve character (shape), showing on the stress-strain hardening; after the first peak was exceeded (probably as the 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 the tensile tests deserve more attention in the future. There is a large space for new findings, especially when the testing devices and monitoring possibilities are strongly improving with time.

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