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Technical session 1b: Laboratory testing (II): Strength, large deformation, and hydraulic properties

Séances techniques 1b: Tests de laboratoire (II): Résistance, grandes déformations et propriétés hydrauliques

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1 INTRODUCTION

Forty papers, covering specific aspects of the behaviour of widely different materials (compacted soils, cemented and saline natural soils, sands, soft clays, gravelly soils, gypsum laden clays, claystones, among others) have been included in the Session. They are listed at the end of this report. All of them have been reviewed and their main contributions, in the author's opinion, have been summarized in the following pages. When appropriate, one or two of the most informative figures of each of the papers has been reproduced to facilitate the comprehension of the written text. Once the review was made, it became clear that no single framework or dominant set of concepts could be applied to the papers included in the Session. They were therefore grouped in a few categories: Unsaturated soils; Clay soils, long term effects; Granular soils; Soil mixtures and soil design; Chemo-mechanical interactions and Experimental techniques. Even if this grouping is made, the objectives and procedures described in each individual paper make the defined subset of papers quite heterogeneous. A common feature of all the papers reviewed is that they describe experimental results without including, with a few exceptions, constitutive model development or model performance. The review maintains this common characteristic and only marginal reference is made to phenomenological models which may be used to describe or interpret some of the data presented. In a summary section a number of relevant issues are highlighted. Finally, some issues selected for discussion are suggested.

2 UNSATURATED SOILS

2.1 *Cafaro & Cotecchia "A structure-based approach to the estimate of the water retention curve of soils"*

Despite the title, the most interesting part of the paper refers to the comparison of the soil compressibility against isotropic loading and suction increase respectively. This comparison, for soil G, a high plasticity clay, PI_G = 27.7%, is shown in Figure 1. Normally consolidated specimens were reconstituted and one-dimensionally consolidated to p' = 160 kPa. The "gross" air entry value of the soil is larger than 1000 kPa and, therefore, the compression and "suction-loading" curves represented in Figure 1 correspond to almost saturated conditions. It follows that effective stress should hold in the suction (and loading) range 160-1000 kPa. However, a stiffer response was measured for the suction increase path, when compared with the isotropic loading response (INCL curve). The authors explain this behaviour because "fabric changes of the normally consolidated clay with suction loading differ from those resulting from external isotropic loading". This fabric effect is attributed to the contraction of aggregate pores. This is inconsistent with the saturated effective stress principle and it requires further consideration. In other experimental programs, effective stress has been shown to hold for suction changes below the air entry value (Fleureau et al, 2005). Figure 1 also shows a sudden compression in the vicinity of the air entry value (identified as "drying collapse"), which, in general, has not been found in other experimental programs. The behaviour of an overconsolidated specimen (to σ'_v = 1100 kPa) is also shown in figure 1. The stiffer response upon drying is consistent with the previous loading history. Interestingly, the specimen seems to yield at the INCL line (this yield is the explanation for the "drying collapse" effect) and it becomes stiffer against increasing suctions. The fact that the air entry value of the specimen Goc coincides in this case with the preconsolidation stress is difficult to generalize. In any case, the results in Figure 1 shows that the stress history of the soils influences the water retention properties, a result stressed in the paper The paper discusses also a simplified approach to derive the water retention curve in drying processes.

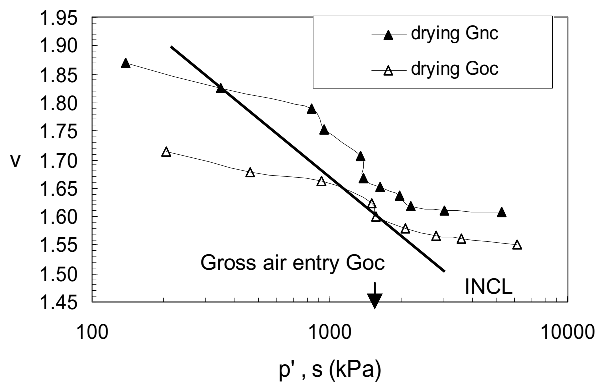

Figure 1: Comparison between normally (Gnc) and overconsolidated (Goc) reconstituted clay sample behaviour. (Cafaro and Cottecchia, 2005)

2.2 *Sung, Lee, Cho & Reddi "Soil-water characteristic curve assessment using a reference state concept"*

Sung et al. investigate the possibility of building water retention relationships from water retention data measured at the liquid limit. They tested in a pressure chamber three widely different residual soils of granitic origin, classified as CH, SC and SM. They do not provide, however, plasticity data of these soils (is the liquid limit defined for the SM soil?). Four densities (obtained by consolidating remoulded samples at their liquid limit) were tested. Void ratio was determined at the end of each equilibration period at a given suction. Water retention curves were defined in terms of the degree of saturation (or volumetric water content ratio). The correct calculation of S_r requires information on water content and void ratio. However, the retention curves presented do not correspond to a constant void ratio, information which is useful in modern constitutive formulations of coupled hydro-mechanical behaviour of unsaturated soils (Vaunat et al, 2000; Romero and Vaunat, 2000). S_r is correctly calculated but the void ratio is changing along the retention curves presented.

It is somewhat surprising the small air entry value measured for the CH soil (a few kilopascals –2 to 8 kPa- for the densities tested). Although relationships are given between the fines' content and a suction close to the air entry value, it is unlikely that a general relationship exists, irrespective of the mineralogy of the clay fraction. Nevertheless, the normalised relationships that the paper provides for the parameters of a particular equation describing the water retention curve may be useful to approximate the drying water retention envelope of soils moderately active.

One of the normalised plots is shown in Figure 2, which provides the air entry value (normalised by the value at the liquid limit) as function of void ratio (normalized with respect to the void ratio at the liquid limit). The relationship holds for the three soils tested. The idea of using a reference state (such as the liquid limit) to build water retention properties for more compacted states is interesting. But more data is required to establish the usefulness of this approach: suction tests at constant volume, imbibition paths as well as drying tests and tests on natural specimens.

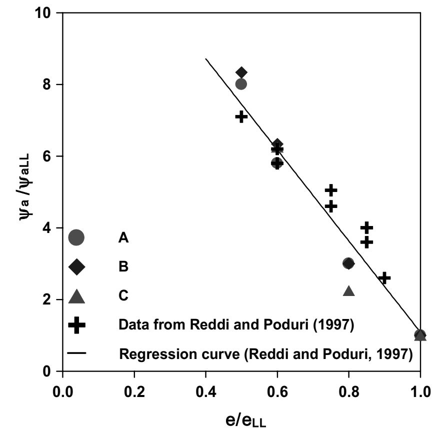

Figure 2: Relationship between the normalized air entry value, ψ_a/ψ_{aLL} versus the normalize void ratio, e/e_{LL}.(Sung et al, 2005)

2.3 *Lee, Kim & Lee "A method to estimate soil water characteristic curve for weathered granite soil"*

As in the previous paper, the motivation for the work presented is associated with the usefulness of the retention curve to derive other unsaturated soil parameters (strength and permeability are mentioned) and with the difficulties, in terms of cost and time, to determine the retention curve in the laboratory. Both aspects are probably overemphasised. In fact, some papers, presented to the session, warn against the errors in estimating strength properties, on the basis of the retention curve (Ng and Zhou, 2005). On the other hand, experimental procedures to obtain the curve are common practice in many laboratories. The authors favour the use of an Artificial Neural Network (ANN) approach to derive model parameters of empirical equations, which describe the curve. The ANN model is able to "learn" if a sufficient number of water retention curves is first analysed. The soil variables selected to perform predictions are the sand content, silt and clay content, void ratio and compaction water content. The study is performed on a number of granitic residual soils from Korea. The authors conclude that their approach is better suited than other empirical methods to peform predictions. This is illustrated in Figure 3, which shows a comparison of a water retention curve, the approximation of the ANN model, once it has been subjected to "training" by means of 13 independent data sets of the same soil and the performance of other models. Perhaps the most significant result is the difficulty of some published procedures to estimate the water retention curve on the basis of soil identification data. The authors conclude that the weathered granitic soils of Korea are not suited to perform approximations based on transfer functions, as suggested, for instance, by Fredlund et al. (1997). It is unlikely that Korean soils are very special in this regard. The conclusion is that empirical procedures have always limitations and that a procedure which incorporates information for a range of soils similar to the one used for benchmarking purposes, has obviously an increased lilkelihood of being more accurate.

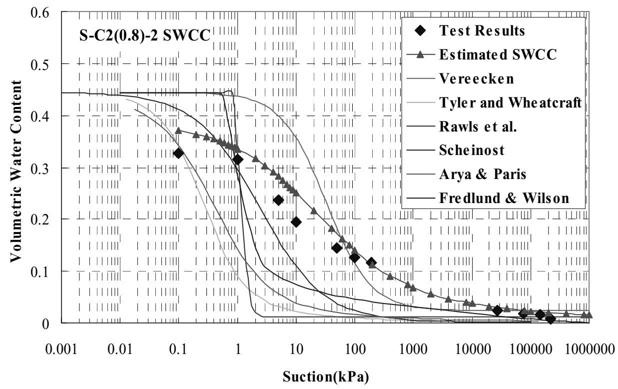

Figure 3: Comparison of SWCC obtained by test and other prediction methods. (Lee et al, 2005)

2.4 *Mrad, Cuisinier, Abdallah & Masrouri "Modélisation du comportement hydroméchanique des sols gonflants saturés sous fortes succions »*

Expansive soils exhibit some features of behaviour, which make its modelling a challenging exercise within the framework of unsaturated soil mechanics. An elastoplastic model, which integrates microstructural features (Gens & Alonso, 1992; Alonso et al., 1999), has been selected by the authors to analyse the behaviour of a high plasticity mixture of silt and bentonite (w_L = 87%; PI = 21%) compacted statically to a relatively low density (12.7 kN/m^3). A set of suction controlled oedometer tests were performed and model parameters could be identified. Of particular interest is the determination of the macrostructural yield function for isotropic conditions (LC), which is shown in Figure 4. The shape of this yield locus is similar to the LC curve found for a compacted bentonite by Lloret et al. (2003). The paper presents some simulation of oedometer tests in the high suction range and its comparison with model predictions.

2.5 *Ng & Zhou "Effects of soil suction on dilatancy of an unsaturated soil"*

The description of dilatancy (or the flow rule) is a key component for plastic models. The paper provides data on dilatancy rates measured in direct shear tests performed oon compacted specimens of residual granitic soil (w_L = 44%; w_P = 16%). Specimens were compacted dry of optimum to 82.7% of the Proctor optimum. The as-compacted suction was estimated as 60 kPa, on the basis of the data given in the paper. Specimens were saturated, placed in the box, loaded to σ_v = 50 kPa under saturated conditions, equilibrated at different suctions (up to 400 kPa) and sheared. Figure 5 shows the measured dilatancy ratio (δ_y/δ_z) as function of stress ratio (τ/τ_v) and suction. Specimens first contract, then dilate (maximum dilation corresponds

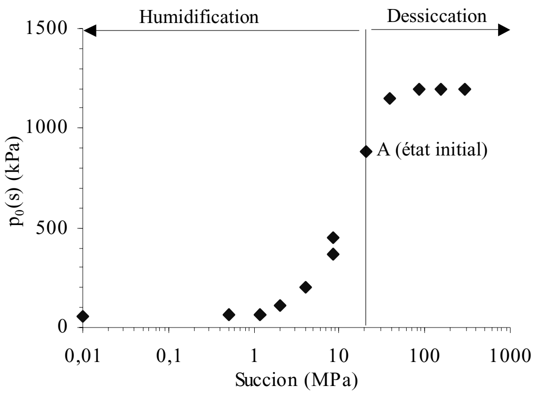

Figure 4. Variation de la pression de préconsolidation avec la succion (Mrad et al, 2005)

approximately to the relative displacement for peak strength) and finally reach constant volume conditions. Suction enhances the dilatant behaviour. However, the interpretation of these tests is not straightforward. Increasing suction compresses the soil (the compressibility of the soil was described as relatively large) and, therefore, suction and density effects contribute to its dilatancy response. Added difficulties are the heterogeneous stress and strain distributions inside the specimen during the shear process (Potts, Dounias and Vaughan, 1987;O'Sullivan and Cui, 2005). Elastic and plastic components are integrated in the given plots.

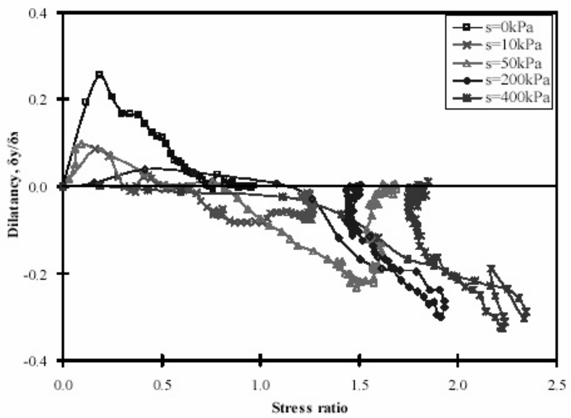

Figure 5: Experimental stress-dilatancy relationship. (Ng and Zhou, 2005)

2.6 *Nishimura & Vanapalli "Volume change and shear strength behaviour of an unsaturated soil with high soil suction"*

Only a few triaxial cells capable of handling very large suction values have been described (Blatz & Graham, 2000; Chávez & Alonso, 2003). Nishimura & Vanapalli's paper describes briefly a cell for high suction values using a vapour transfer technique to impose a desired relative humidity to the soil. They report the result of a few triaxial tests on a non-plastic silty soil of uniform grain size, statically compacted dry of optimum to a high void ratio (e = 0.93). They show that the friction angle for a suction of 39 MPa is the same as for saturated conditions (23º). The apparent cohesion increases with suction but the limited increase registered for such a high value of suction suggests that the effect of suction decreases progressively, as noticed by Escario & Saez (1986).

2.7 *Zhang, Tao & Tumay "Energy concept and soil compression"*

A convenient plot to discuss and interpret the results presented in the paper is given, in qualitative terms in Figure 6. A significant aspect of the plot is the shape of contours of constant suction. They tend to be controlled only by water content for relatively dry states and they become progressively parallel to curves characterized by S_r = constant as the soil becomes wetter.

A static compaction at constant water content (which is the procedure followed to manufacture specimens in the reported research) is also shown. The plot explains why the rate of suction decrease during compaction is affected by the initial water content. In relatively dry conditions water is confined inside the clay aggregates and no "free" water occupies the soil macroporosity. Different packing arrangements (i.e.: varying density) of a given array of aggregates should not have any effect on the soil suction. This explains the shape of lines $s = s_1$, $s = s_2$ in Figure 6. However, under wetter conditions, water begins to occupy partially the bigger pores and the capillary suction is now increasingly controlled by the degree of saturation.

The most interesting result of the paper is the dependence of compression strength of three clayey soils tested (CL, CL, CH) on the energy of compaction (Fig. 7). Unconfined strength should depend on current void ratio and suction. It seems, therefore, that the dry portion of the compaction curves (which correspond to the same energy of compaction) represent also contours of equal strength. However, one should be careful if suction is changed. For a given void ratio, a decrease in suction should lead to a reduction in strength, irrespective of the compaction energy. This is shown in Figures 8 and 9 (Rico and Del Castillo, 1976). Strength for as compacted specimens follow approximately in Figure 8 contours of equal compaction energy. However, if the soil is saturated (Fig. 9), the effect of suction dissapears and only the initial density plays a role. A general conclusion, therefore, is that suction should be incorporated in the definition of strength for a more comprehensive picture.

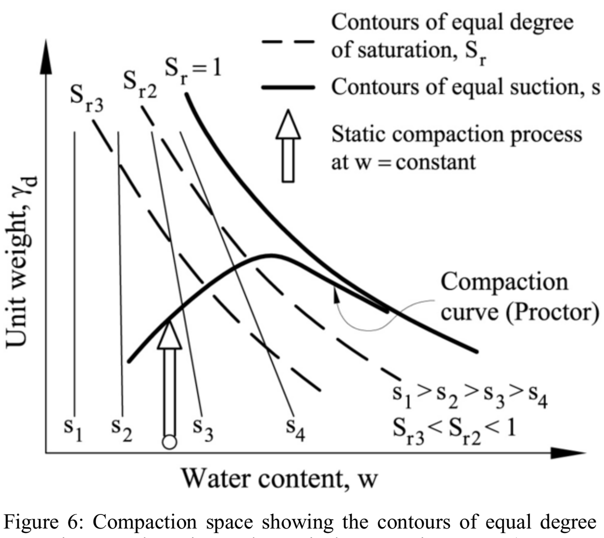

Figure 6: Compaction space showing the contours of equal degree of saturation, equal suction and a typical compaction curve (constant energy). The arrow indicates a static compaction process at constant water content.

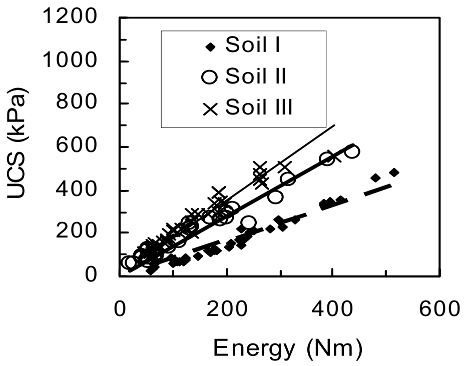

Figure 7: Unconfined Compression Strength versus Absorbed Energy. (Zhang et al, 2005)

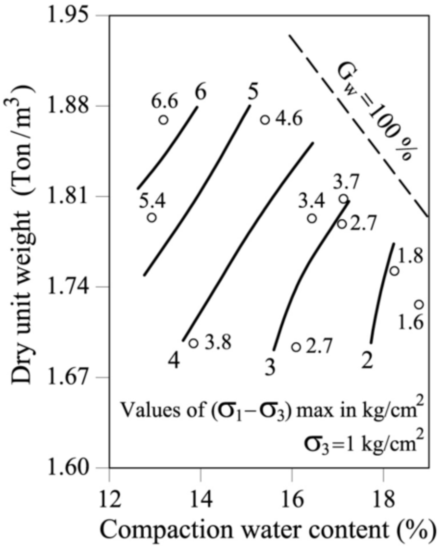

Figure 8: Contours of equal compression strength, in rapid tests, without previous saturation under a confining stress of 1 kg/cm^2 (Rico and Del Castillo, 1976)

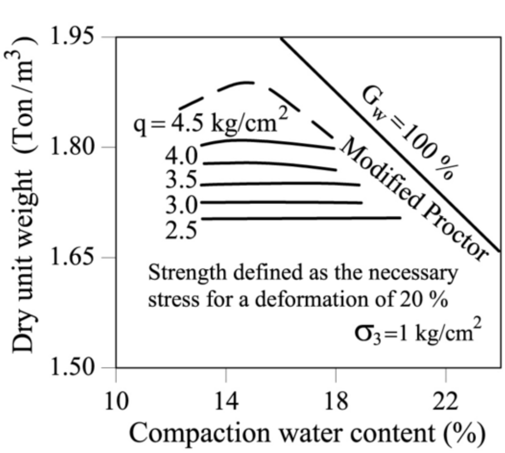

Figure 9: Contours of equal confining strength in rapid tests pwith revious saturation (Strength measured at large deformations) (Rico and Del Castillo, 1976)

2.8 *Chae, Kim, Lee & Kim "Shear characteristics of an unsaturated compacted granite soil"*

Chae et al. present triaxial test data on compacted specimens of a non-plastic silty sand (SM) (% < 2μ = 17.15%). Specimens were compacted at 92% of the optimum (Normal Proctor ?), dry density and at water contents on the dry side (w = 6.5%), optimum (w = 11.6%) and wet (18.5%) of optimum.

Before testing, specimens were equilibrated at suctions of 0.5, 1 and 2 kg/cm^2. The non-plastic character of the soil is an indication of very limited or inexistent fabric effects. Therefore, irrespective of the origin, the shear behaviour should be essentially controlled by the suction at equilibrium (and by the confining stress). This is actually one of the conclusions of the work. On the other hand, suction has, for this sandy soil, a reduced effect if compared with the confining stress. This is also a conclusion, illustrated in Figure 10, which shows the deviatoric stress-strain and dilatancy curves measured for the series of drained tests performed. Suction increases the dilatancy effects, but in a very moderate way.

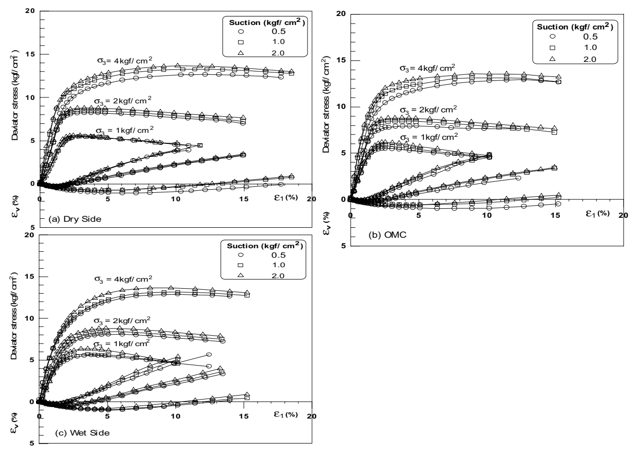

Figure 10: Consolidated drained test on unsaturated granite soils. (Chae et al, 2005)

Apparently, all the tests were performed at a very high strain rate (0.1%/min), which is several orders of magnitude faster than the rate of other tests reported in the literature for silts and low plasticity soils (Delage, 2004). This brings the issue of the real nature of the "drained tests" performed. Even if the fine's content of the tested soil is small (17.15%), the soil response to a fast shearing rate is probably only partially drained. The "apparent cohesions" reported for the drained tests are probably a consequence of the partially undrained behaviour (Fig. 11). The authors also reached the conclusion that the effective stress angle was not affected by suction, a result that could be expected in view of the non-existent plasticity of the soil.

The paper, however, raises the important aspect of the effect of initial soil fabric on its subsequent behaviour, when suction changes are induced. This is relevant in practice. In fact, one may wonder if there is any potential collapse in a soil compacted on the wet side, but later dried and then loaded (when the height of the embankment increases, for instance). The granitic soil tested by Chae et al. is not sensitive to this stress path, but clayey soils are. The reason (see figure 12 for a reference plot in the space suction vs. net mean stress) is that drying of clayey soils generate large micropores in the path IA which may resist the application of external load (Path AB). However they are prone to collapse in path BC (wetting, once the net mean stress has been applied).

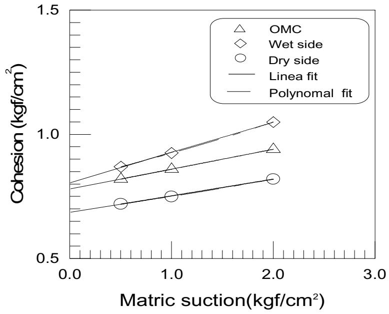

Figure 11: Relationships between the matric suction and the cohesion for consolidated drained tests. (Chae et al, 2005)

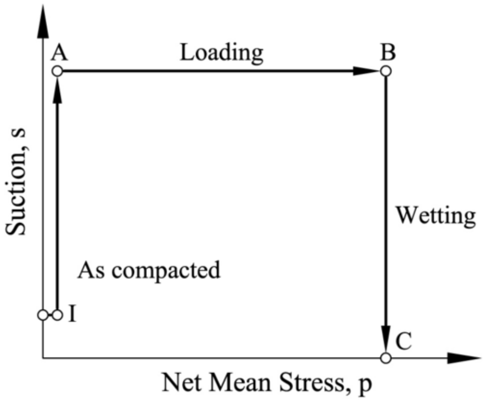

Figure 12: Stress paths to illustrate the possibility of generating collapse strains in a soil compacted wet of optimum.

2.9 *Cui, Delage, Marcial, Terperau & Marchadier "Sur la susceptibilité à l'effondrement des loess du Nord de la France"*

Unsaturated low plasticity carbonated loess occupy a large area in the north of France. They create potential problems to infrastructure and, in particular, to embankments of high-speed railways. The authors have performed simple collapse tests on natural specimens trimmed from block samples taken at several depths in excavated pits. They compare simple and double oedometer techniques (Jennings & Knight, 1957) and they found a close agreement between them (Fig. 13). The plot shows the collapse measured under a confining vertical load of 200 kPa, due to the total wetting of the specimen. It should be added, however, that simple a double oedometer tests are peformed at constant water content. Thus, suction decreases continuously during the loading process. Therefore, vertical deformations measured during the loading stage incorporate already some collapse. The remaining collapse potential is, thus, reduced, especially under high loads. Figure 13 shows that collapse under a given load increases with the initial water content of the soil, a result which is consistent with the prediction of current elastoplastic models for unsaturated soil behaviour such as BBM. This is qualitatively illustrated in Figure 14, which shows in a (σv, suction) plane the position of three specimens (A, B and C) at different water contents (and, therefore, at different suctions) and at a common vertical stress of 200 kPa. The loading-collapse yield curves plotted (LCA, LCB and LCc) provide the collapse potential upon wetting, which is given for sample A, for instance, by the stress change from p^*_{oA} to 200 kPa. This stress change increases as the initial water content decreases (high suctions). The authors check also the accuracy of six collapse criteria. The results vary widely from a criterium that declares all four specimens tested as collapsible to another one, which declares three specimens out of four as non-collapsible. It is clear that collapse empirical criteria, based on soil indexes offer a very limited reliability. In addittion, some criteria declare as non-collapsible soils which experience a collapse deformation smaller than 1%. However, this deformation may have dangerous consequences under a high-speed railway. More information on loess formations in France has been recently offered by Delage (2005).

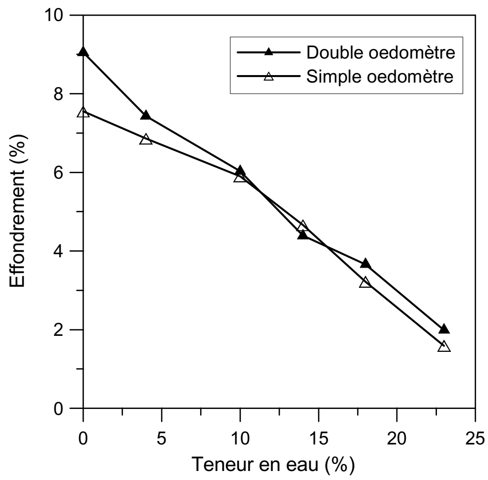

Figure 13: Effondrement du sol à 2,2 m à différentes teneurs en eau initiales, identifié par les méthodes du simple et double oedomètre. (Cui et al, 2005)

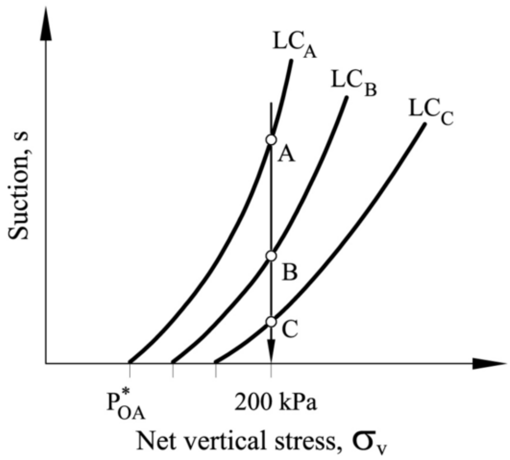

Figure 14: Stress paths of specimens wetted from different initial suctions at a common vertical stress (200 kPa)

2.10 *Skutnik & Garbulewski "Suction-swelling relations for Warsaw clays"*

The determination of swelling potential of clays is a classical subject that may be approached from different perspectives. Soil index parameters such as activity, plasticity, clay content, dry density, natural water content and others, have been connected with the potential of volume change. Simple laboratory tests (two are described in the paper and applied to Warsaw clays: the COLE and the CLOD tests) provide also an estimation of the expansion potential. This is shown in Figure 15. The shaded areas correspond to different sites. In general, the high plasticity Warsaw clays present a moderate to high swelling potential. These are traditional engineering approaches, which higlight the mineralogical content of the clay as a key factor. But it is convenient to emphasise that any clay soil is potentially expansive. For a given soil mineralogy, the expansion is essentially controlled by three variables: the initial packing (void ratio), the initial suction and the applied confinig stress. Suction controlled testing allows a more precise identification of swelling and this is also shown in the paper.

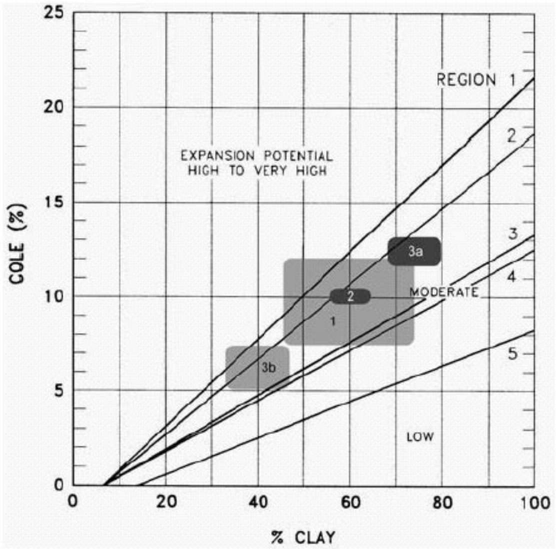

Figure 15: Identification of the expansion potential for the tested soils acc. to the COLE value classification chart. (Skutnik and Garbulewski, 2005)

2.11 *Vesga & Vallejo "Strength of an unsaturated kaolinite clay under suction pressures"*

The main idea put forward by the authors is that the pure tensile strength of a given plane in an unsaturated soil should be equal to the normal effective stress on that plane. Then, a Brasilian test provides a direct estimation of the effective stress associated with the specimen suction. The unconfined compressive strength of the unsaturated soil is then interpreted as the triaxial strength for a confining stress given by the mean effective stress provided by the water suction. Figure 16 reproduces both stress values for specimens of reconstituted kaolinite (w_L = 58%; w_P = 28%). Tests were performed without suction control. For the impervious material tested both tests were almost certainly undrained. Given the different volumetric response in compression and tension, the suction prevailing at the time of failure in the compressed specimens will not be equal to the suction in the tensile plane of the Brasilian test. This is a relevant aspect of the tests performed, not discussed in the paper. Therefore, the proposed strength envelope, on the basis of the Brasilian and compression tests, provides only an approximation to a consistent strength envelope, in terms of effective stresses.

2.12 *Jimoh "Shear strength/moisture content models for a laterite soil in Ilorin, Kwara state, Nigeria"*

The author presents the results of "conventional quick laboratory shear strength tests" on samples of lateritic soils from Nigeria. Specimens were prepared at different moisture contents and sheared. Values of "cohesion, c" and "friction angle, ϕ" for each soil and for different water contents are indicated. Unfortunately no experimental details of the tests performed and the specimen preparation are given in the paper.
Testing details are, however, very important to interpret results, especially if the soil is unsaturated. Regression equations are used to calculate c and ϕ as a function of water content.

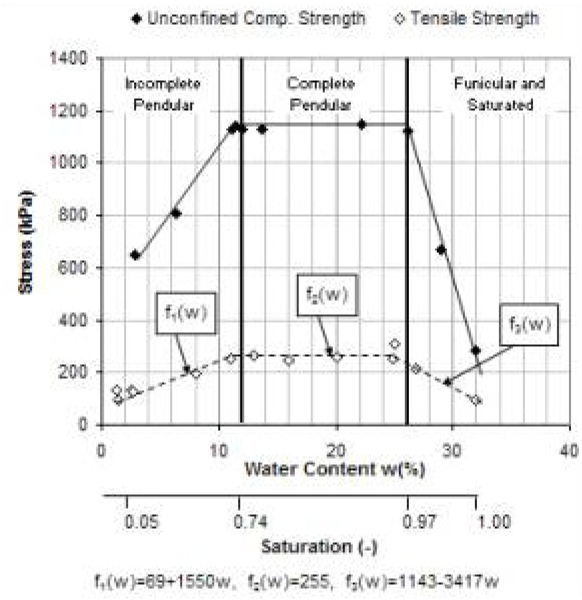

Figure 16: Laboratory results from unconfined compression tests and Barzilian indirect tension tests. The functions f_1, f_2 and f_3 represent the tensile strength as a function of the moisture. (Vesga and Vallejo, 2005).

It is not clear, looking at the data, if the strength parameters reported correspond to drained conditions. Certainly the range of water contents used imply that specimens were unsaturated at the low water content range and fully saturated at the high range of water contents. This situation makes the interpretation of results even more questionable. The author does not seem to be aware of the intensive research performed on the shear strength of unsaturated materials. Suitable references are Escario (1980), Escario and Saez (1986), Fredlund and Rahardjo (1993) and Toll (2000).

2.13 *Romana and Serón "Characterization of swelling materials by the Huder-Amberg oedometric test"*

Huder and Amberg (1970) is a classical reference in Rock Mechanics literature when the expansion of some claystones and clay-anhydrite rocks is examined. As described in Romana and Serón paper, the test is simple: it is performed in an oedometer cell and no control of the specimen water content is exerted other than flooding it at a vertical confining stress equivalent to the "in situ" state. The specimen is first loaded to a target vertical stress, a loading-unloading cycle is applied, it is flooded (swelling strains will presumably develop) and it is finally unloaded in steps, once it is saturated. The impervious nature of the matrix of expansive clay rocks makes it difficult to know in this test the degree of saturation of the successive testing stages. The test is interpreted in a straightforward manner although some of the assumptions (i.e.: the rock swelling pressure may be derived from the reloading and final unloading stress-strain diagram is questionable). Romana and Serón discuss other practical difficulties of test. One of them refers to the testing time, which may be extremely high. They suggest that simple strain-time models may be useful to forecast the final strains in water flooding or unloading steps. In one example given, strain rates depend linearly on time. This model can be extrapolated if some data is available for a limited time interval (say two weeks) and used to predict final deformations. The Huder-Amberg test is perhaps too crude if examined from the perspective of unsaturated soil mechanics.

3 CLAY SOILS. LONG TERM EFFECTS

Strain rate and temperature effects, two topics included in this chapter, have been discussed extensively by Leroueil and Marques (1996).

3.1 *Cola & Simonini "Relevance of secondary compression in Venice lagoon silts"*

The paper presents laboratory and field data on secondary compression of deep soft sediments involved in the design of the protective works of Venice against recurrent flooding. The soil profile investigated is very heterogeneous, although it has a common Pleistocene geological origin. Sand, silt and clay (illite and montmorillonite are the main minerals) fractions are mixed in widely different proportions, although the silt fractions dominates in the upper 40 m. The natural soil appears to be free draining and primary consolidation develops at the same rate of embankment construction (a few months). This behaviour was detected in a preloading test embankment, instrumented by means of high precision extensometers and a variety of additional sensors. Strains recorded in time by the extensometers were interpreted in terms of the common C_c and C_α coefficients and compared with laboratory measurements. This is shown in Figure 17. An asterisk identifies the field values, which include some three-dimensional effect due to the limited dimensions of the loading area (a 40 m diameter circular earth reinforced embankment, 6.7 m high). The figure points out the detailed information provided by extensometer reading a quality that is far from being achieved in conventional investigations. Continuous extensometers provided a detailed identification of the soil variability, in terms of truly relevant parameters, in this case the primary and secondary compression indexes. A similar type of investigation, focused on the effects of OCR on secondary compression has been reported by Alonso et al, (2000). Also given in the paper is the OCR, derived also from compression plots based on the strains recorded during the loading period. The ratio C_α/C_c in field and laboratory data remains in the range 0.015-0.05 and the average is close to 0.028.

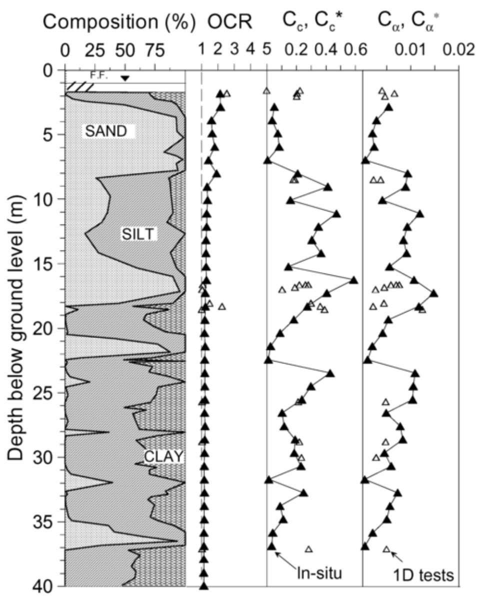

Figure 17: In-situ and laboratory values of primary and secondary compression coefficient. (Cola and Simonini, 2005)

3.2 *Lacasse & Berre "Undrained creep susceptibility of clays"*

The paper reviews some aspects of undrained creep of clays in the presence of deviatoric stresses. Undrained creep is of concern when safety factors against shear failure are small (say less than 1.25). The framework adopted to interpret experimental results was proposed by Singh and Mitchell (1968) and Mitchell (1976). Creep rates are interpreted through the empirical rate process equation:

$$\dot{\varepsilon}_d = Ae^{\alpha D}\left(\frac{t_1}{t}\right)^n \qquad (1)$$

Where t is the current time, t_1 is a reference time and A, α and m are parameters. D describes the ratio of the applied deviatoric stress to the undrained strength. The paper shows that experimental data from triaxial and simple shear testing is described by the above equation. Creep rate increases with the plasticity, organic content and thickness of the clay deposit. High creep rates may evolve towards an acceleration of creep rate and a subsequent failure. Data on several testing programs have been plotted in a creep susceptibility diagram (Fig. 18), which relates strain rate and shear stress ratio D. The position of some clays that exhibit high creep susceptibility is shown in the plot. It may be used as a guide to identify in practice creep-susceptible clay formations.

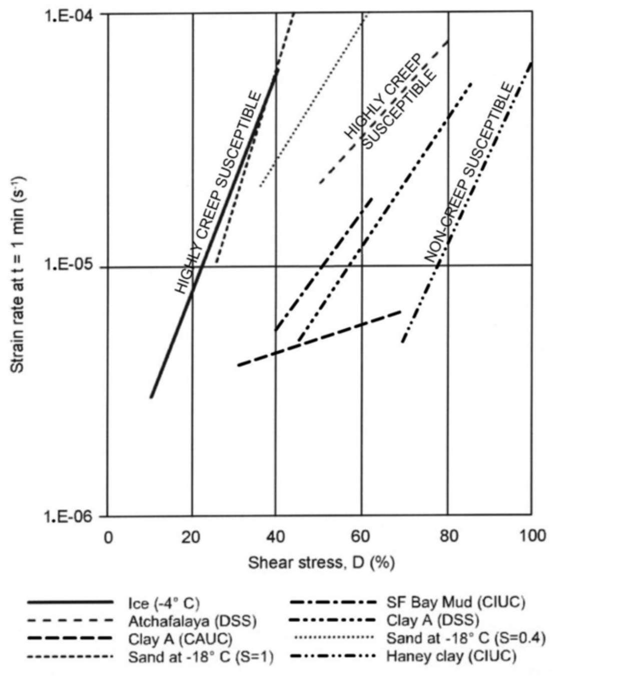

Figure 18: Creep susceptibility diagram. (Lacasse and Berre, 2005)

3.3 *Szymanski, Lechowicz, Drożdż and Sas "Geotechnical characteristics determining consolidation in organic soils"*

The behaviour of soft peats and organic calcareous soils is investigated. In the first part of the paper an interesting field case involving the behaviour of a 3.9 m high embankment on a 7 m deep profile of organic soils is reported. The installation of geodrains under the embankment reduces the lateral deformations outside the loading area (if compared with the normal consolidation case). This behaviour is attributed to the change in boundary conditions for water flow in the foundation soils outside the loaded area. Drained and undrained triaxial tests were also performed. Creep tests for time lengths of 20000 min (two weeks) were interpreted by means of an empirical equation:

$$\varepsilon_s = \eta_o \; q^{\eta} \; t^{\eta_2} \tag{2}$$

where ε_s is the creep strain, q is the deviatoric stress and t, the elapsed time. η_0 η_1 η_2 are constants. This equation for total creep strain is similar to the Mitchell's equation adopted in the paper by Lacasse & Berre to this Session. In fact, the η_2 coefficient was found to be 0.069 and therefore the creep rate is also given by:

$$\varepsilon_s^e \cong \eta \;\; q^{\eta_1} \; \left(\frac{1}{t^{0.93}} \right) \tag{3}$$

Figure 19 shows a plot of secondary strain rate as a function of log time and several deviatoric and mean stresses for one of the soils tested (an organic calcareous soil). The data given for secondary creep plot as straight lines in log $\dot{\varepsilon}$ - log t coordinates. However, the reported values of creep rate for these organic soils (for t = 1 min) seems to be substantially lower than the creep rates reported in the susceptibility diagram of Lacasse and Berre (2005).

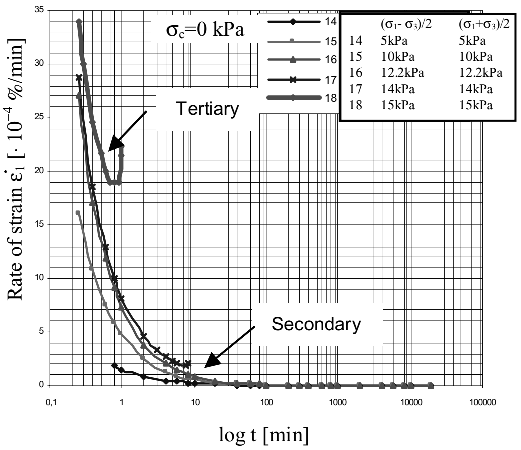

Figure 19: Rate of strain versus log time for *in situ* calcareous soil. (Szymansky et al, 2005)

3.4 *Taïbi, Ghembaza & Fleureau "Comportement thermo-hydro-mécanique d'une argile plastique saturée"*

The effect of temperature on the mechanical behaviour of clayey soils has received considerable attention in recent years. (Hueckel and Borsetto, 1990; Laloui and Cekerevac, 2003). A significant proportion of published results is associated with the behaviour of engineered clay barriers in the context of nuclear waste disposal. Results are not always consistent among them but a good starting point to understand the effect of temperature is to accept as a framework the evolution of yield conditions sketched in Figure 20. Following this graph, temperature reduces the yield stress and the size of the elastic domain.

The authors present a set of triaxial tests performed on specimens of kaolinite mixed with sand particles (10%). This soil is described as a sandy clay. It is a low plasticity material (w_L = 38%; PI = 19%; k_{sat} = 10^{-9} m/s), which is probably not very reactive against changes in temperature, due to the low internal specific surface. Tests were performed in a temperature controlled triaxial cell on reconstituted as well as compacted samples. Temperature contributes to accelerate primary consolidation became of its effect on water viscosity, which in turns controls permeability. This effect is well-captured by the tests performed. However, the set of deviatoric tests, show a very limited effect of temperature (in the range 22º-80ºC). This is shown, for instance, in Figure 21, which corresponds to clay tested in normally consolidated conditions. It was found that the increase in temperature seemed to erase the previous overconsolidation history, a result that is consistent with the framework outlined. The effect of temperature on strength (parameter M in Fig. 21) was not clearly established although changes in M were found to be small, as indicated also in Fig. 21 for the normally consolidation case.

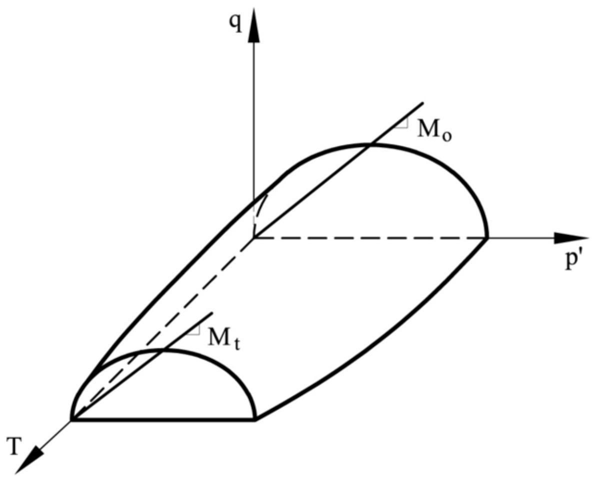

Figure 20: Yielding for increasing temperature

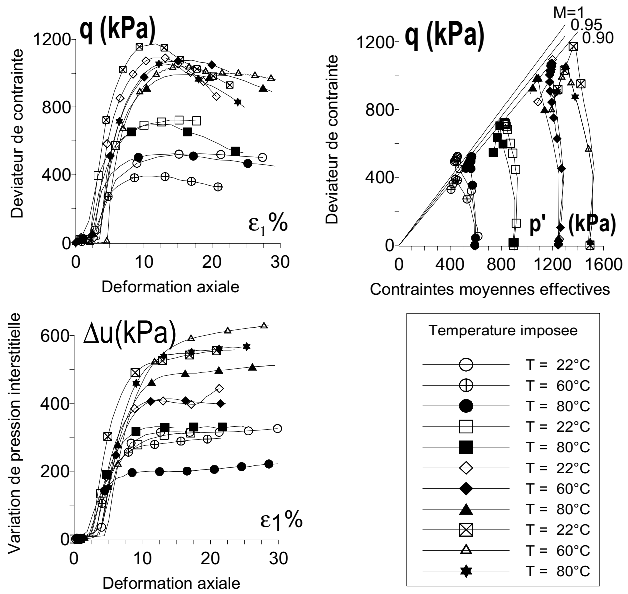

Figure 21: Chemins triaxiaux saturés normalement consolidés à différentes températures imposées (T=22 et 80°C). (Taibi et al, 2005)

3.5 *Rodríguez "Yielding and stress-strain relationship for Bogotá clays"*

The city of Bogotá is founded on very soft and highly plastic clays, which may reach a very high thickness (300 m are mentioned in the paper). Void ratios are in the range 2.70-3.0 and a liquid limit of 144 and a plasticity index of 96 are reported. The paper presents the results of a few drained and undrained triaxial tests, which are interpreted within the framework of critical state. Yield loci for peak conditions are given in Figure 22. It is interesting to note the substantial loss of strength from peak conditions (28º) to critical state conditions (13º). A marked brittleness is, therefore, expected in this soil although the deviatoric stress-strain curves reported do not show clearly this major change in strength. Bogotá clays exhibit pronounced structural and probably cementation effects and this makes it difficult to apply a modified Cam-clay model for computational purposes as stated in the paper, even if the K_0 value "in situ" is close to one.

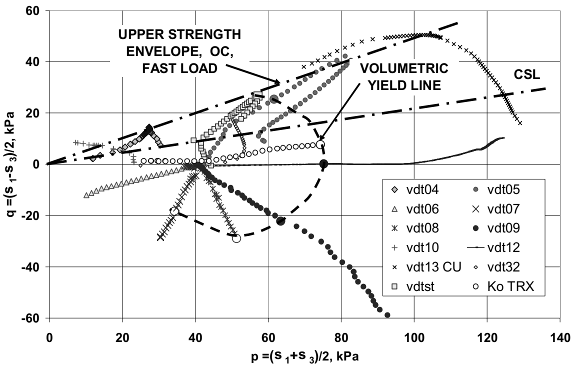

Figure 22: Triaxial test stress paths. (Rodríguez, 2005)

3.6 *Elias & Titi "Effect of sample size on resilient modulus of cohesive soils"*

The resilient modulus characterises the elastic behaviour of grade and subgrade soils for repeated loading. The measuring technique involves the application of repeated cycles at varying confining and deviatoric stresses. The modulus, M_r, is identified as an elastic modulus once a number of conditioning cycles (500 mentioned in the paper) have been applied. The paper reports an experimental investigation of the effect of sample size on M_R.

Tests were performed on statically compacted low plasticity clays following AASHTO standards. Figure 23 shows typical results for one of the clays tested under a confining stress of 27.6 kPa. The effect of sample size is perhaps too marked. The authors doubt, however, of the accuracy if the loading cell used in the tests reported, especially for the smaller specimens.

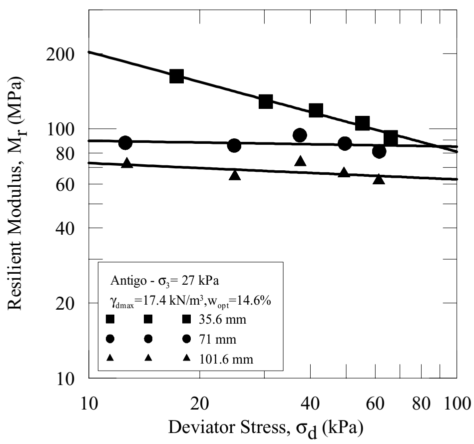

Figure 23: Results of repeated load triaxial test on Antigo clay under different specimen sizes and confining stress of 27.6 kPa. (Elias and Titi, 2005)

4 GRANULAR SOILS

4.1 *Naughton & O'Kelly "Yield behaviour of sands under generalised stress conditions"*

The validity of two general yield criteria (Matsuoka & Nakai, 1985; Lade & Duncan, 1974) in the presence of stress rotation is experimentally investigated. Specimens of well-graded fine to medium Leighton-Buzzard sand were prepared at a relative density of 72-78% and then tested in a hollow cylindrical apparatus. The experimental technique was aimed at determining segments of the yield surface by varying the effective stress ratio (σ'_1/σ'_3) and the intermediate principal stress parameter. Specimens were first anisotropically consolidated and a given orientation of the major principal stress (α) was initially applied. The technique to find portions of the yield surface involved unloading-reloading paths and a suitable procedure to identify yield points in stress-strain plots. Once a segment of the yield surface could be identified, the normalised constants of the two mentioned criteria could be found and plotted in terms of α and b. This is shown in Figure 24 for the Matsuoka-Nakai criteria. It can be shown that the variation of the M-N constant with α (and b) is small. A similar result is found for the Lade criterion. It is concluded that the rotation of principal stresses during the consolidation stages does not affect the two yield criteria selected. It may be added that the authors probably tested a very isotropic sand fabric in this case.

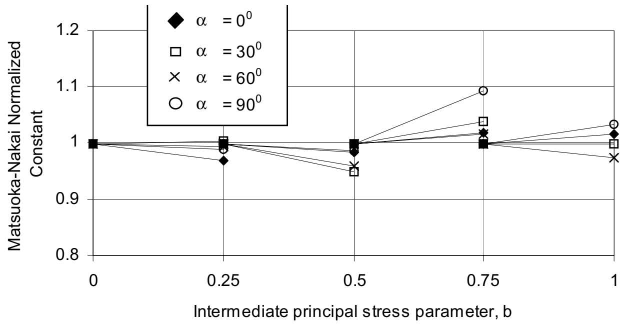

Figure 24: Experimental and theoretical yield surfaces expressed in terms of the Matsuoka-Nakai yield criteria. (Naughton and O'Kelly, 2005)

4.2 *Kuwano & Nahadi "Effect of shear stress history on yielding of Toyoura sand in p'-constant shear plane"*

The behaviour of soils inside the current yield surface has received considerable attention in the last decade. The paper presents a detailed experimental program, performed in a hollow cylinder apparatus, on 80% relative density specimens of sand. The technique to investigate the behaviour inside the general yield surface (Y_3) was to conduct radial probing tests from a given stress point. Small strains were measured in order to capture the transition from linear elastic behaviour to the initiation of yield (yield surface Y_1). They identified a third yield locus (Y_2), which marked the beginning of a rapid decrease in shear modulus. A similar proposal was already made by Jardine et al (1991). Figure 25 shows the set of stress paths applied in several test series. In all cases, the mean effective stress was kept constant. One example of the yield loci identified is given in Figure 26. Yield surfaces $Y_1$and Y_2 accompany the stress point. Y_1 has a very small and apparently constant dimension. Both Y_1 and Y_2 are elongated ellipses oriented in the direction of plastic strain increment vectors, which are non associated with the yield locus. The macro yield surface Y_3 exhibits an isotropic hardening behaviour and an associated plastic flow.

4.3 *Lade, Yamamuro & Bopp "Relative density effects on drained and undrained strengths of sand at high pressures"*

The persistence of the initial sand packing and the dilatancy observed in shear at relatively low confining stresses complicate the acceptance of a critical state framework to describe the behaviour of sands in compression and shear. However, as the confining stress is increased, the dilatant behaviour decreases and the progressive breakage of particles make the behaviour of sand qualitatively similar to the behaviour of normally consolidated clay. The authors present results of a set of isotropic, compression and extension triaxial tests performed at high stresses (up to 60 MPa in isotropic compression). They found (figure 27) that for confining stresses in excess of 15 MPa the effect of initial density (specimens of coarse uniform sand were tested at 30%, 60% and 90% relative density) disappears

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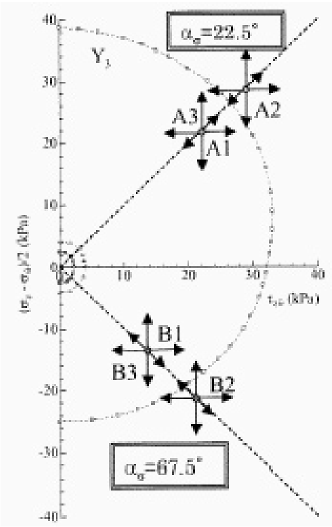


Figure 25: Shear stress paths for the test series 2-7. (Kuwano and Nahadi, 2005)

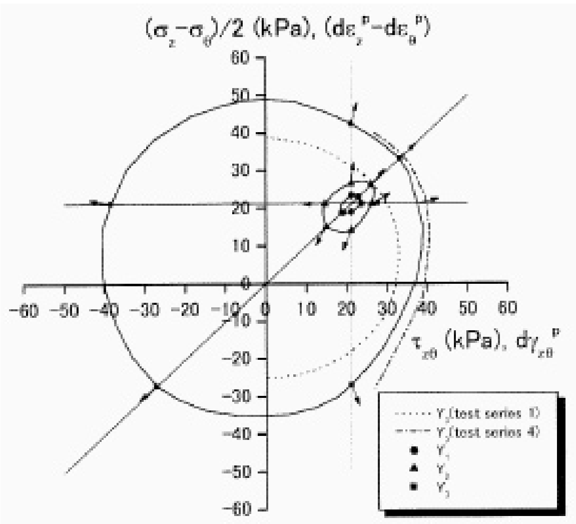


Figure 26: y_1-y_2 surfaces in series 6. (Kuwano and Nahadi, 2005)

Friction secant angles in compression decrease with increasing confining stress but they level beyond a stress (which changes with the initial relative density). Effective friction angles in extension also decrease with confining stress; however, they preserve the memory of the initial state for the full range of confining stresses investigated (from 0.2 MPa to 50 MPa). A summary of results, in terms of void ratio vs. log (effective confining stress) is shown in Figure 27. At "low" stresses, the void ratio at failure tends to be higher than the equilibrium void ratio in compression (1D or isotropic). This is especially the case of dense sand because of dilatancy effects. At higher stresses, sand becomes contractant, due, in part, to particle crushing and void ratio at failure remains below the compression curves (which is the regular behaviour of normally consolidated clay). The authors stress that this behaviour is useful to predict sand strength at high confining pressures since strength values depends on the attained void ratio.

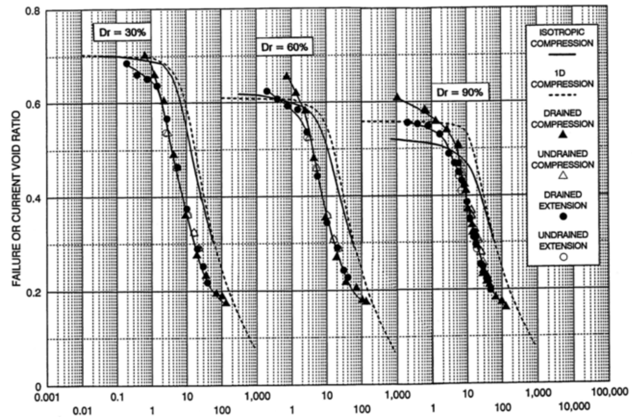


Figure 27: Void Ratio vs. Log (Stress) for Isotropic and One-Dimensional Compression Tests and Maximum Deviator Stress for Drained and Undrained Triaxial Compression and Extension Tests on Loose, Medium Dense, and Dense Cambria Sand. (Lade et al, 2005)

4.4 *Bobei & Lo "Reverse behaviour and critical state of sand with small amounts of fines"*

In a reference paper for the work reported by Bobei & Lo (2005), Yamamuro & Lade (1998) described the "reverse" behaviour of silty sands against liquefaction. They found that the addition of a small amount of fines enhanced the compressive behaviour of silty sands when sheared, especially at low to medium confining stresses. The term "reverse" refers to the relative position of the critical state (alternatively, the steady state line, if reference is made to undrained tests) and the normal compression line. In sands with no fines, even at relatively low densities, the material is dilatant. However, the addition of small quantities of silt or clay induces a contractive behaviour, similar to the behaviour found in non-consolidation clays. Yamamuro & Lade (1998) explain this behaviour as a consequence of the unstable structure created by bridges of silt or clay particles located near the sand grain contacts. When the structure is compressed and sand grains become closer or in contact, the behaviour becomes dilatant and therefore more stable in undrained loading. The undrained triaxial tests reported by Bobei & Lo (Fig. 28) exemplify this behaviour and show full liquefaction at low confining stresses and low quasi-steady state response after peak at higher confining stresses. This is consistent with the relative position of the CSL and ICL implied by the "reverse" behaviour. The authors propose a state parameter to identify this behaviour, which is a function of the distance (in void ratio terms) between the ICL and CSL lines (the parameter of Been & Jefferies, 1985). But they modify it including a stress factor and the current void ratio to account for the influence of confining stress outlined above. A similar dependence was introduced by Bolton (1986) to describe the dilatancy of sands. The proposed state parameter offers a better discriminating capability for sandy soils with a small amount of fines.

4.5 *Wijewickreme & Sanin "Some observations on the cyclic loading response of a natural silt"*

Low-density silts exhibiting a non-negligible plasticity are susceptible to cyclic mobility and full liquefaction in the extreme case. Cyclic simple shear testing of undisturbed specimens is an established procedure to investigate the cyclic response of natural materials. The paper presents simple shear data on the undrained cyclic response of a low-density (e = 0.88) plastic silt (w_L = 30.5%; w_p = 27.3%) classified as ML. Tests were conducted to investigate the effect of the previous application of a history of cyclic loading on subsequent cyclic behaviour. An

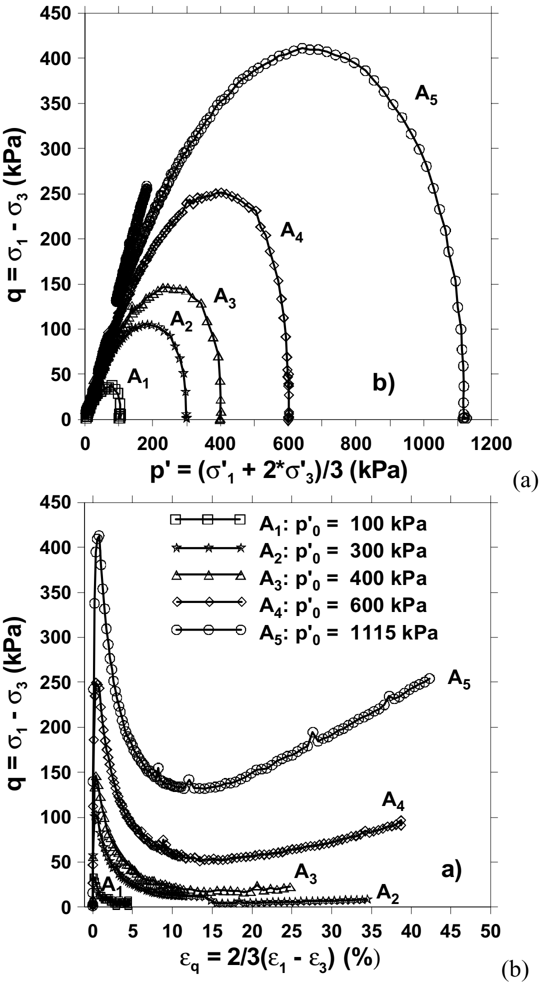


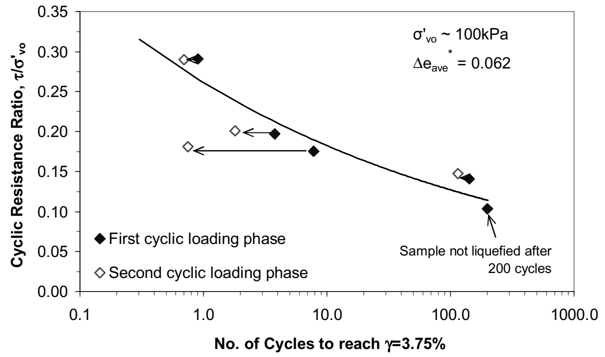
Figure 28: Response of sand with fines in undrained shearing. a) Stress-strain relationships b) Effective stress paths. (Bobei and Lo, 2005)

intermediate consolidation to the original confining stress (which densified the specimen) was applied.

The plot in Figure 29 summarizes the result. The normalized cyclic shear stress ratio to reach a target shear deformation of 3.75% decreases with the number of applied cycles. An interesting result was the observed increase in cyclic susceptibility during the second loading phase, even if the specimen had densified during the applied consolidation. This is attributed to fabric changes during the first loading period. In applied studies of the stability of marine dykes subjected to cyclic loading, the reporter (Alonso & Gens, 1999) found useful to express the cyclic mobility diagrams in the form suggested by McCarron et al. (1995). The data offered in Figure 29 addresses partially the question of damage induced by successive storms on the design stability of these structures.

4.6 *Lipinski & Wolski "Parameters describing flow liquefaction of soils"*

The purpose of the paper is to show that the undrained behaviour of contractive sands presents a series of common features, which may help to find principles of general applicability to identify liquefaction potential. The results of undrained triaxial tests on sands with different proportions of fines are discussed from this perspective. In qualitative terms the undrained response of contractive sand is illustrated in Figure 30. It shows a first stage of rapid development of positive pore water pressure until a peak shear stress is reached. Further straining results in a decrease of shear stress until the ultimate strength line is hit.



*: Average change in void ratio after first cyclic loading phase

Figure 29: Cyclic resistance ratio versus number of cycles to reach γ=3.75% on first and second cyclic loading phases from constant volume DSS tests on Fraser River silt. (Wijewickreme and Sanin, 2005)

Then a "phase transformation" may take place. The sand dilates and the strength increases until steady state conditions are eventually found. The paper describes some relationships between the initial confining stress, the mean stress at peak shear strength, the mean stress at phase transformation and the mean stress at steady state. Liquefaction parameters (brittleness index, undrained shear strength) are also related to some state parameters of the sand. Figure 31 is one example: it provides the dimensionless stress ratio s_u/σ'_{vc} (σ'_{vc} is the vertical effective consolidation stress) as a function of the state parameter ψ (difference between the initial void ratio and the void ratio at steady state conditions). It is shown that reasonably good unique correlations could be found for the different sands tested, for both isotropic and anisotropic consolidation conditions.

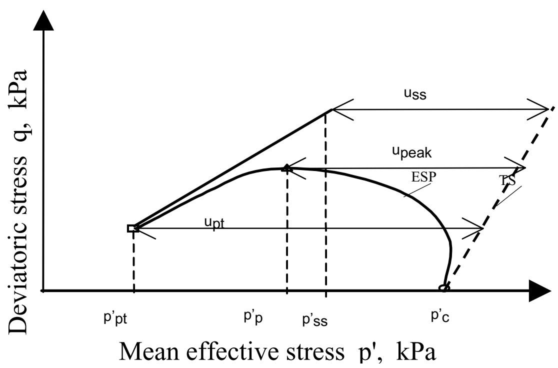


Figure 30: Identification of terms used in a description of the undrained behaviour of contractive soil. (Lipinski and Wolski, 2005)

4.7 *Zhang & Zhang "Test study on behaviour of interface between structure and coarse grained soil"*

The paper presents the results of tests on gravel-steel interfaces in a large shear machine (the interface area is 50 by 36 cm). As expected, increasing roughness of the steel surface leads to an increase in the measured friction (which does not change with the number of applied cycles). It is also observed that the accumulation of cycles leads to a progressive volumetric contraction (in terms of measured vertical strain) for a constant confining stress (Figure 32). The accumulation of vertical strains is probably associated with a progressive breakage of particles in the intermediate vicinity of the interface, although no particle breakage data is reported. The authors stress also the non-symmetric behaviour of the normal displacement when subjected to a symmetric cycle.

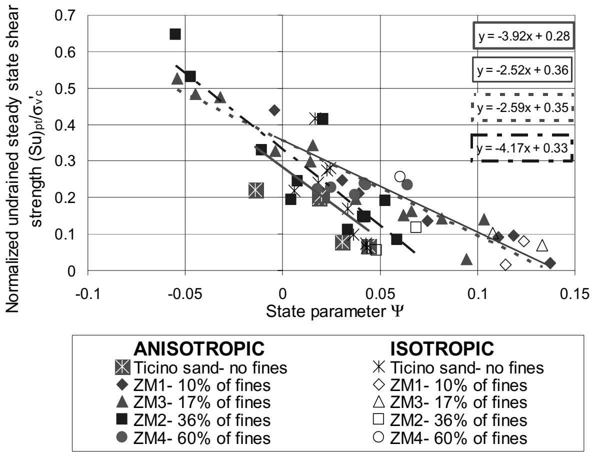


Figure 31: Some parameters characterising flow liquefaction shown against state parameter ψ. (Lipinski and Wolski, 2005)

This is presumably an effect of the oriented geometry of particles at the interface. In any case, this is a minor effect compared with the observed accumulation of irreversible compressive deformations. No reference shear tests on the gravels used are given in the paper, in order to better appreciate interface effects.

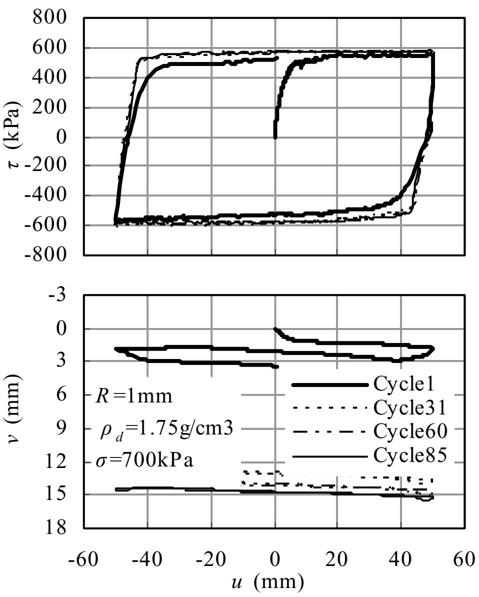


Figure 32: Cyclic stress-displacement relationship under constant normal stress boundary condition.(Zhang and Zhang, 2005)

4.8 *Rankine & Sivakugan "Drainage characteristics and behaviour of hydraulically placed mine fill and fill barricades"*

The paper reports on some hydraulic properties of Australian tailings and the geotechnical problems associated with its disposal in previously excavated "stopes". The hydraulic fills (silty sands and sandy silts with a negligible clay fraction) may reach relatively low values of permeability (in the approximate range of 1 x 10^{-6} cm/s to 3 x 10^{-5} cm/s), which makes it difficult a free draining process through the outlets (porous brick barricades). Tailings from mining operations exhibit often angular particle geometry and this leads to high friction angles. The paper suggests an empirical rule to estimate the angle of friction:

$$\phi = 19\, D_r^2 + 33 \tag{4}$$

where D_r is the relative density (hydraulic fills reach D_r values in the range 0.5 to 0.8).

Undisturbed specimens of pyrite tailings involved in the failure of Aznalcóllar dam were subjected to consolidated undrained triaxial testing (Alonso & Gens, 2005). Finer (pyritic) and coarser (pyroclastic) specimens were tested. Particle shape was also angular and the solid specific weight was high (G_s = 3.5-4). Measured friction angles varied in the range 37º-42º, although no correlations with grain size (D_{50}) or void ratio could be found. These friction angles are, however, consistent with the empirical rule suggested for the Australian tailings.

5 SOIL MIXTURES AND SOIL "DESIGN"

5.1 *Reiffsteck & Nguyen Pham "Influence of particle size distribution on mechanical behaviour of soil"*

There is a maintained interest in geotechnical engineering in relating soil identification properties with engineering properties, both mechanical and hydraulic. The paper reports on some triaxial tests performed on mixtures of granular aggregates (either glass balls or sand) and a fine filler (either fine sand or kaolin). Information is also given on the minimum density of the mixture, which is found for a 20-30% of fine particles. Triaxial tests were performed undrained, on dry specimens (glass balls-sand mixture) and on compacted specimens (sand-kaolin mixture). The second case brings the issue of the interpretation of the tests within the more general context of unsaturated soils since water suction is changing during the test. The authors do not mention this aspect, however. Figure 33 shows a typical result: the decrease of the measured friction angle with the increase in the content of finer particles. The experiments reported in the paper are compared with other published data. The authors distinguish two critical values for the percentage of finer particles (C). Below C_r = 0.3 friction is dominated by the skeleton of coarser particles. Above C_r= 0.70 friction is dominated by the finer particles. In those two regions φ' variations are small. In the central zone, a transition controlled by C is observed. The paper presents also some data on the compaction density achieved in natural mixtures of sands and fine soil. The shape of the grain size distribution curve controls the attained density at the compaction optimum. When the accumulated distribution of particles (the classical grain size distribution curve) presents a concave shape, maximum densities are achieved. In fact, in this case the curves are close to the theoretical Fuller distribution. On the other hand, convex shapes lead to low compaction densities. It is finally stressed that the classical coefficients of curvature and uniformity are of little help to characterise mixtures of components with widely different grain sizes.

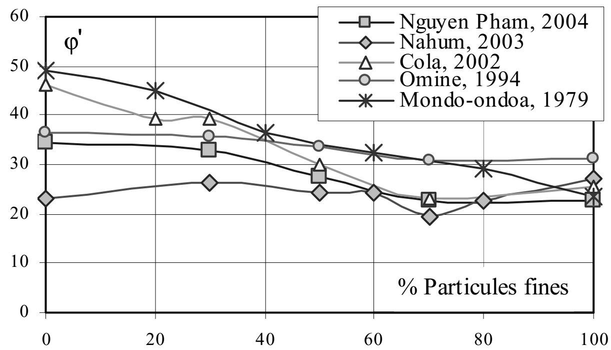


Figure 33: Variation de l'angle de frottement avec le pourcentage des particules fines. (Reiffsteck and Pham, 2005)

5.2 *Pedro, Canou, Dupla, Dormieux & Kazan "Caractéristiques de rupture d'un sol hétérogène de référence"*

Gravelly soils with a significant proportion of a "fine" matrix (sand, silt and clay) are common in nature. They are difficult to test and the determination of its bulk mechanical properties is an unresolved problem. Very often the larger particles are removed before testing. This practice leads to errors for at least two reasons: samples tested are disturbed and, in addition, they have a modified grain size distribution.

The experimental investigation described in the paper addresses these difficulties. Patterns of behaviour are sought by testing in triaxial cells mixtures of sand and a variable proportion of gravel (defined as a volume fraction of gravel). In all cases the specimens tested had a common relative density (70%) (equivalent to a total specific weight of 1.58 g/cm^3), and a common consolidation stress (100 kPa). As expected, an increase in the volume fraction of gravel (sizes: 8-10 mm) lead to a progressive increase in strength and in the associated friction angle. The most interesting result, however, is reproduced in Figure 34. It shows the deviatoric stress strain curves obtained for a fixed proportion of the gravelly component and variable sizes of the coarse grains. The recorded compression plots seem to be independent of the gravel size. This result opens the possibility of substituting the coarse gravel by finer particles in testing coarse natural materials. It should be added that the behaviour observed in Figure 34 cannot be maintained for finer particles (and probably not either for increasing size of the coarse component). Practical limits to the described finding should be given. Certainly other aspects: shape of gravels, strength of gravel materials, overall grain size distribution, etc. should enter in a more comprehensive treatment of the problem.

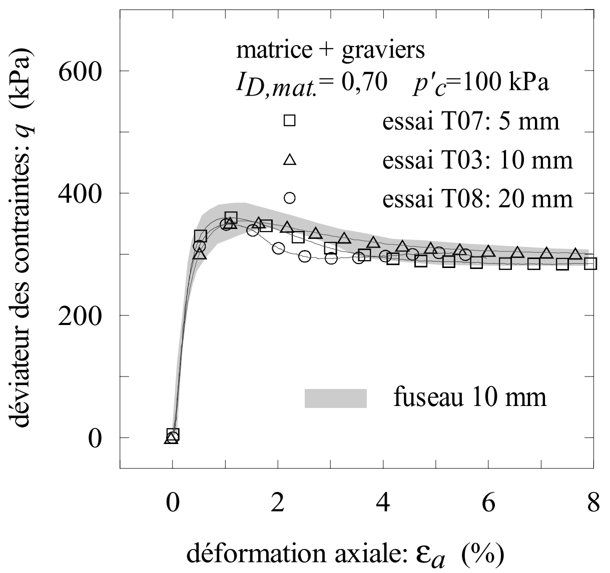


Figure 34: Influence de la taille des graviers sur la résistance au cisaillement (f_v=20%). (Pedro et al, 2005)

5.3 *Watabe & Saitoh "Influence of sand fraction on compressibility and hydraulic conductivity of clayey soils"*

A high plasticity clay (Nagoya clay: w_L = 62.6%; w_P = 30.7%) and a fine sand having rounded particles (Niigata sand: e_{max} = 1.096; e_{min} = 0.66) have been mixed in different proportions and the compressibility and permeability were determined through conventional oedometer testing. It was found that the C_c index decreased linearly with the increase in sand fraction but reached a constant value beyond sand contents in excess of 65%. This percentage marks the development of a sand skeleton, which dominates mechanical behaviour. This is consistent with the findings of Reiffsteck & Pham in a paper in this session. Permeability, however, (Fig. 35) remains low and constant for most of the mixtures tested, provided the clay content is higher than 50%. Beyond this limiting value permeability increases fast indicating again the development of a granular network within the mixture. The authors report also data on pore size distributions and SEM observations. Especially useful to interpret the macroscopic results are the pore size distributions. For instance, the rapid increase in permeability beyond a 50:50 mixture is associated with the appearance of a significant proportion of large diameter pores (0.008 mm) in the soil. These results provide a guide for the properties of mixtures in the context of barrier design for waste containment.

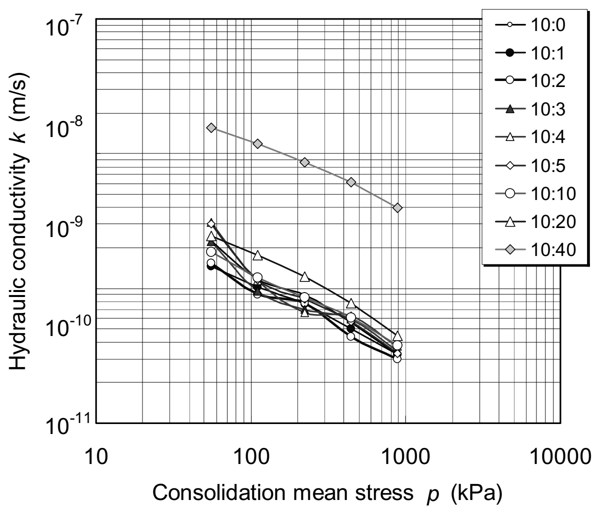


Figure 35: log k—log p relationships. (Watabe and Saitoh, 2005)

5.4 *Watanabe, Tateyama, Yonezawa & Aoki "Strength characteristics and construction management of cement-mixed gravel"*

The construction problem behind the research presented in the paper is the proper design of cement-mixed gravel bridge abutments. Two gravels having different particle specific weights and grain size distributions were mixed with cement in a 2.5% cement/gravel (by weight) ratio and compacted to the optimum density. Specimens were then tested in a large diameter cell (diameter of specimens: 150-300 mm). After isotropic consolidation specimens were loaded in compression. A marked peak strength was recorded followed by a sharp decrease in strength. At high vertical deformations in excess of 6% no "residual" strength differences were recorded for different initial dry densities. Peak strength data (for σ'_3 = 20 kPa and 2.5% cement ratio) is plotted in Figure 36 as a function of dry density. Data for treated and untreated gravel is shown in the figure. The effect of increasing the density of the compacted specimens is well observed in the cement-treated gravels. The authors also found a noticeable effect of the compaction water content. Best results were obtained at optimum. The paper provides also a construction standard for these materials. Cores recovered from a built abutment showed peak strengths well above the required minimum specifications although a substantial heterogeneity in results was found. This fact was attributed to a non-uniform distribution of cement within the gravel fill.

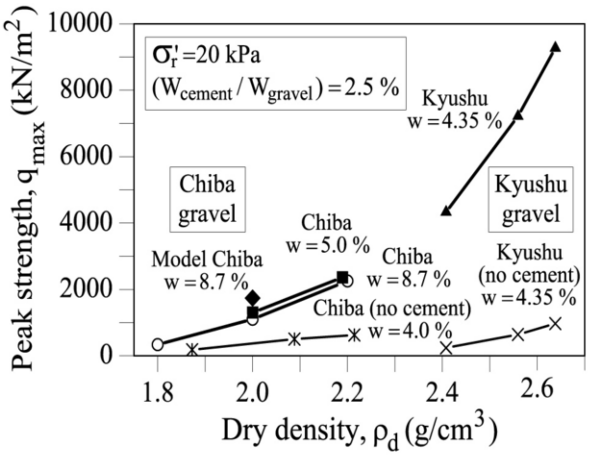


Figure 36: Relations between peak strength and dry density. (Watanabe et al, 2005)

6 CHEMO-MECHANICAL INTERACTIONS

6.1 *Graham, Man, Alfaro, Blatz & Van Gulck "Gypsum cementation and yielding in plastic clay"*

The failure of dykes founded on soft montmorillonitic clay was not easily explained by conventional limit equilibrium analysis using, as reference strengths, the post-peak and the residual strength friction angles (in the vicinity of 13º and 11º respectively). The dykes and their foundations have been subjected to a long-term filtration, which may have led to a change in the pore water chemistry of the clay and as a side-result of the clay strength. The paper provides triaxial strength data and consolidation properties of both undisturbed and reconstituted specimens. The main purpose of the reported experimental investigation is to establish the effect of gypsum cementation on clay strength for different pore water chemical compositions. Triaxial tests on specimens taken from the foundation of an unstable section showed an increase in brittleness and softening properties (Fig. 37), which are difficult to explain if gypsum is considered only as a cementing agent. In fact, determinations of pore water salt concentrations show a significant decrease in calcium and gypsum concentration in the unstable section if compared with the background site, not affected by water leaching. Consistently, however, the Na/Ca ratio of the foundation clay increases in dyke sections, especially in the unstable ones. This index seems to play a significant role in the problem although its effect on strength is not discussed in the paper. Reconstituted specimens rich in gypsum show, however, an increased straining to peak conditions and a stiffer behaviour in consolidation, a finding which may offer a partial explanation to the origin of the instabilities of the dyke sections. This interesting case, which is case of chemo-mechanical interaction of smectitic clays, is under investigation and more data will presumably be available in the future.

6.2 *Foncea, Acevedo & Olquin "Geotechnical characterization of saline soils"*

The authors describe the origin, identification and some properties of saline soils of Northern Chile. Soil behaviour is dominated by salt dissolution processes. Their rate is a function of the type of salt and degree of crystallization and also on the possibility of the water to circulate through the soil microstructure. In this regard, they point out the dominant effect of the soil structure and distinguish two extreme cases: A "macro-porous" structure, in which particles of primary minerals are bonded by salts, leading to a continuous network of relatively large pores, which allows the circulation of water, and a "micro-porous" structure in which a salt matrix dominates and voids are filled by fine soil particles. This latter structure leads to a very low permeability.

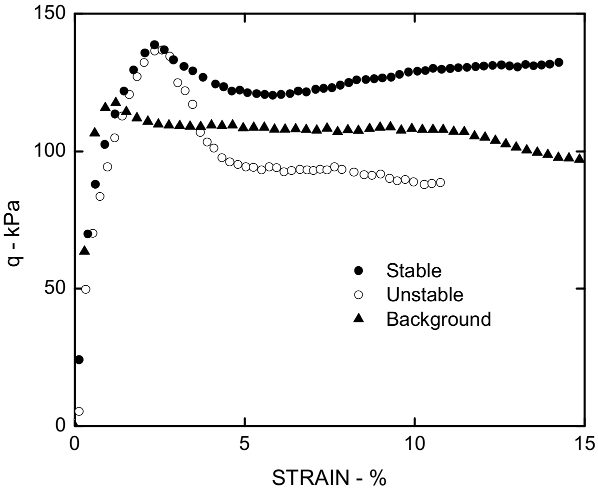


Figure 37: Stress-strain curves and post-peak strain softening from TXC tests. (Graham et al, 2005)

The most salient geotechnical feature of these soils discussed in the paper is the observed settlement induced by water flooding. It is formally similar to the collapse observed in unsaturated (and cemented) soils of open structure. In the case of saline soils, the observed settlement under constant confining stress is attributed to the dissolution of salt bonds. Being more pervious and offering an increased internal specific surface to water dissolution, "collapse" deformations are larger for macro-porous structures, than for micro-porous structures, for the same salt content. This is shown in Figure 38, which summarizes the results of several oedemeter tests in which flooding was applied at a given confining stress.

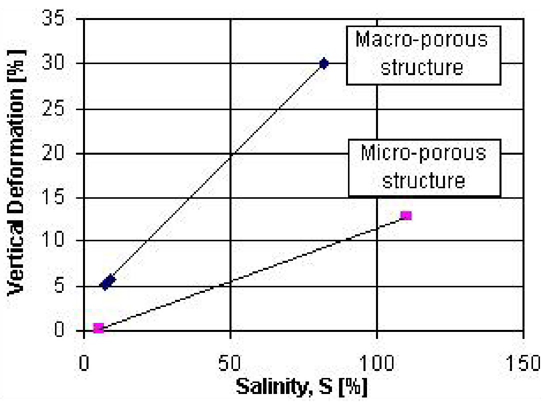


Figure 38: Influence of structure on deformation after saturation in oedometer tests. (Foncea et al., 2005)

6.3 *Westerberg, Albing & Lawson "Research on strength and deformation properties of Swedish fine-grained sulphide soils"*

Sulphide bearing soils are not very common but they may give rise to some undesired effects. The soils described in the paper have a low strength, high compressibility, high secondary compression, if oxidized they release low pH compounds and the precipitation of hydroxides may lead to clogging of conduits. The Swedish sulphide soils are described as fine grained (clay content: 0-40%; silt content: 60-90%) containing iron, sulphur and organic matter (< 10%). They present an anisotropic varved and porous structure (γ_{nat} = 12-18 kN/m^3). Pores are filled with water, organic matter and iron sulphide and this leads to low permeabilities.

The research reported is preliminary and only some data on field strength is given in the paper (Fig. 39). In the case shown in figure 39 the three field investigation methods provided a rather consistent profile of c_u (which is roughly equal to 0.2-0.25 σ'_v). The authors stress, however, that in other sites different field tests lead to significant discrepancies in c_u, an issue that requires further research.

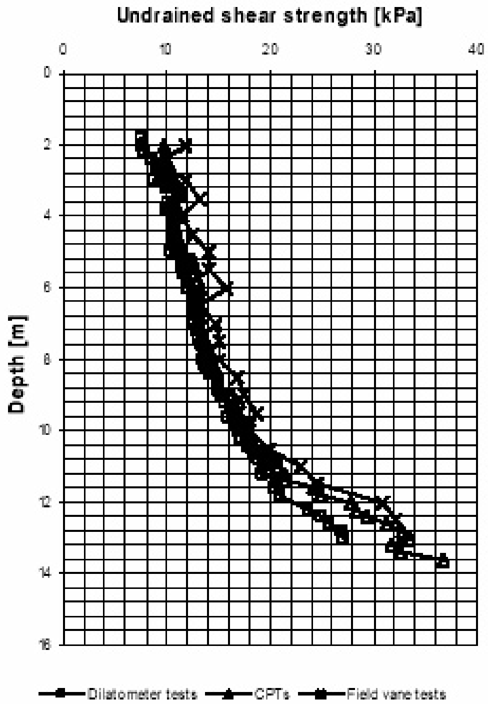


Figure 39: Undrained shear strength versus depth for sulphide soil at Gammelgården evaluated from dilatometer tests, CPTs and field vane tests. (Westerberg et al, 2005)

7 EXPERIMENTAL TECHNIQUES

7.1 *Yamamuro & Liu "Effects of necking and its suppression in axisymmetric extension tests on clay"*

Necking is unavoidable in extension tests, but its effects could be minimised by a proper geometry of the specimen. The paper presents some consolidated undrained triaxial extension tests on kaolinite using lubricated end plates. Radial consolidation was allowed by means of filter paper connected with the edges of the end plates. The objective was to evaluate the severity of necking for three heights (H) to diameter ratio (H/D). Illustrative results are reproduced in Figure 40. It provides the change in area ratio (cross-sectional area at the center of the neck divided by the top or bottom area of the specimen) with accumulated axial strain. Section dimensions were derived from photographs of the test. Necking increases with H/D but even for H/D = 0.5 some necking is observed. Furthermore necking starts at the beginning of the test. The effect of necking on the derived stress-strain behaviour of the soil is further discussed in the paper. Effective stress-strain curves, as well as effective stress paths, are affected by necking. Some qualitative effects may even be possible since the response of the neck region (dilation, for instance) may be compensated by a contractive behaviour of specimen regions close to the caps. The final recomendation is to perform extension tests with small H/D ratios (H/D = 0.5 leads to an acceptable response).

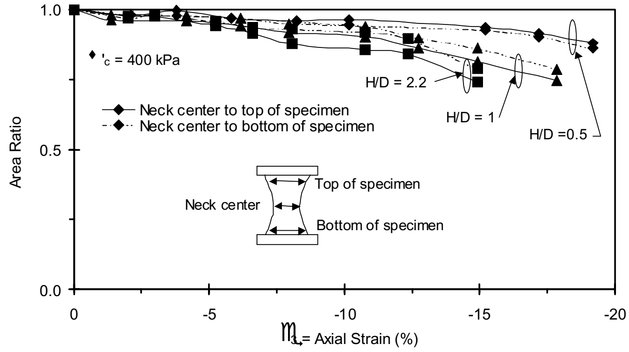


Figure 40: Comparison of the extent of necking in clay specimens during extension tests with H/D ratios of 2.2, 1.0 and 0.5. (Yamamuro and Liu, 2005)

7.2 *Perić & Su "Influence of the end friction on the response of triaxial and plane strain clay samples"*

The paper presents a numerical simulation of laboratory specimens subjected to drained triaxial and plane strain conditions for non-consolidated and overconsolidated states. The declared objective is to investigate local inhomogeneities in the sample induced by the end restrictions imposed by rough upper and lower platens. The behaviour of four representative points: two in contact with the platens (C, D) and two in the middle plane (A, B) is investigated (Fig. 41). No data is given, however, for point B, which is located in the mid plane, at the external specimen surface where local measuring instruments are generally placed. The simulations are taken to large strains (25%) in the average although local points, such as the central A point may reach vertical deformations of 50%. These strains are well beyond what could be considered a small strain analysis, which is apparently the underlying assumption made in the model (although it is not specifically mentioned). The "soil" selected for the analysis is a weald clay previously identified, in terms of cam-clay parameters, in a published paper. Figure 42 is taken as representative of some of the results obtained in the paper. It provides the calculated stress paths for the normally consolidated analysis. Central points (A) follow approximately the imposed nominal loading path. Points close to the rigid and rough platen deviate notoriously as it could be expected. At the platen edge, the soil does not reach critical state and it is heavily loaded. On the contrary, at the center of the platen (Point D) deviatoric stresses are reduced. Points in the soil near the platens follow stress paths, which differ markedly from the nominal average path. Some unloading is even computed at some stages. Similar results were calculated for the overconsolidated specimens. One of the concerns raised by this analysis is the implications that the heterogeneous distribution of specimen strains may have on localization. Additional work is announced by the authors.

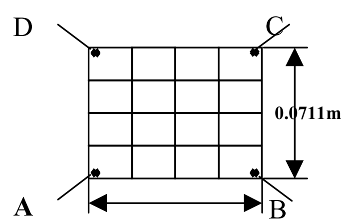


Figure 41: A schematic of the quarter-sample mesh for a sample with 2:1 aspect ratio.

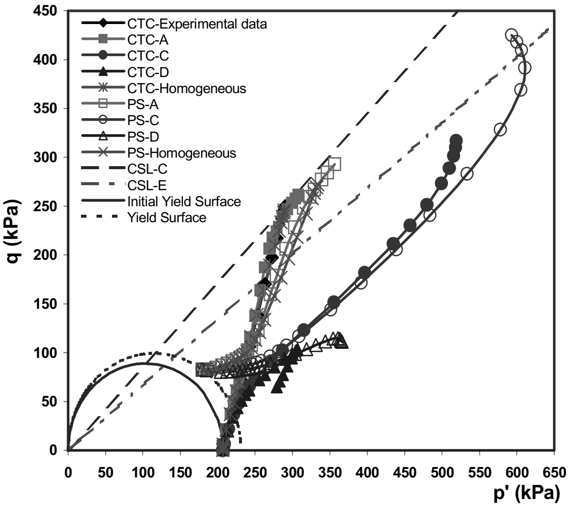


Figure 42: Deviator stress versus mean effective stress for normally consolidated samples.

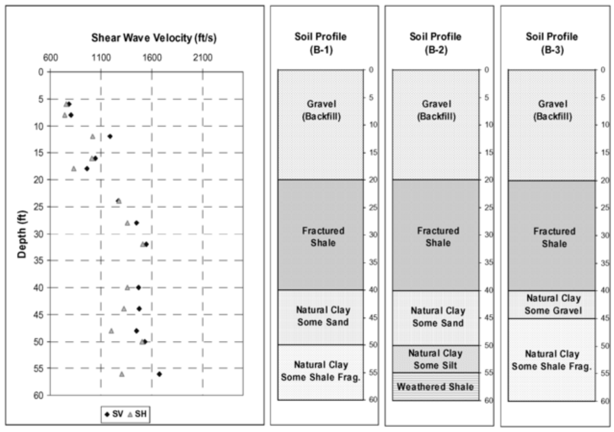


Figure 43: Composite Material Profile: Structure 203. (Hiltunen and Choi, 2005)

7.3 *Grammatikopoulos & Anagnostopoulos "Difference between the values of friction angle Φ derived from the theoretical fracture plane and the reliable one obtained from triaxial tests"*

Based on elementary soil mechanics, the authors accept that the orientation of a failure plane in a triaxial test is a well-known function of the slope of the failure line. They performed consolidated drained triaxial tests on artificially prepared medium plastic clay and clay-sand-cement mixtures. They state that water was removed by drying the specimen in an oven, for 24 hours at 105ºC. Presumably they had produced a hard (and unsaturated) material. The comparison made suggest that the friction angle derived from failure plane inclinations is 1-6º higher than the "direct" friction angle determined from Mohr-Coulomb circles. Although not discussed in the paper, the theoretical problem implied by the comparison made is of some magnitude. The observed failure plane is the result of the localization of shear strains at a given point within the specimen and then its propagation to the entire specimen. The first phenomenon has received considerable attention in the soil mechanics literature and it can be said that it is controlled by the full constitutive behaviour of the soil. The propagation of the failure surface responds to different conditions. In short, the crude relationship relating both angles is only a crude approximation and the term used ("reliable") to describe the angle derived from failure plane orientations is not justified in view of the complexity of the problem.

7.4 *Hiltunen & Choi "Cross anisotropic stiffness properties of soils via crosshole seismic wave measurement"*

The paper discusses several references, which provide the background information to determine the elastic constants of a cross-anisotropic body, on the basis of shear wave measurements particularly using "in situ" cross-hole techniques. Travel times for compression waves and two polarised shear waves (SH, SV) along two pathrays (one horizontal and another inclined) are apparently necessary to determine the five elastic constants. The paper describes an equipment for generating horizontally polarised shear waves and presents two soil profiles where velocities of SH and SV waves were determined (Fig. 43). The difference in SH and SV velocities is interpreted as an indication of soil anisotropy, but the actual soil elastic constants are not determined.

7.5 *Piriyakul & Haegeman "Automated K_0 consolidation in stress path cell"*

The authors describe a standard procedure to perform K_0 consolidation in an automated stress-path cell. Procedures are commercially available and it is standard practice in many laboratories. The drained friction angles for reconstituted Boom clay and kaoline specimens "determined" from the measured K_0 values during consolidation and Jaky's formula is probably an attempt to stretch empirical expressions beyond its intended use.

7.6 *Briaud & Chen "The EFA, Erosion Function Apparatus: An overview"*

The erosion rate ($\dot{Z}$) of the soil-water interface exposed to a given velocity of the current, v is addressed in the paper. The function describing $\dot{Z}$ is a necessary input for the calculation of scour depth in soils. The paper describes the equipment necessary to measure $\dot{Z}$ as a function of the velocity of flowing water, following the scheme of Figure 44. The erosion rate depends on the boundary shear τ, induced by the water that, in turn, may be approximated if the friction coefficient could be calculated. It depends on the Reynold's number of the channel eroding the soil and the pipe roughness, which is a function of particle size (in sands) or of the depth of depressions when erosion proceeds at a larger scale. The apparatus described uses standard soil sampling equipment. A 1 mm thick soil protrusion is exposed to flow of varying velocity. The erosion rate is then directly observed. The magnitude of the shear stresses imposed by the water on the soil is very small in the order of N/m2, therefore many times lower than what can be conceived as the soil shear strength. Yet water flow erodes most soils and destroys bridges The reason is that the micro shear strength which resists erosion is not comparable to the "macro" shear strength as we know it in Soil Mechanics.

8 CONCLUSIONS AND TOPICS FOR DISCUSSION

Unsaturated soils

A classic subject in unsaturated soil research is the determination of the water retention law. In its simplest description the water retention curve provides a relationship between water content and applied suction. The original ideas were developed in Soil Science research and in most cases the tested soils had a granular composition and they were fairly incompressible. In silty and clayey soils the application of suction not only reduces the degree of saturation but it deforms volumetrically the soil.

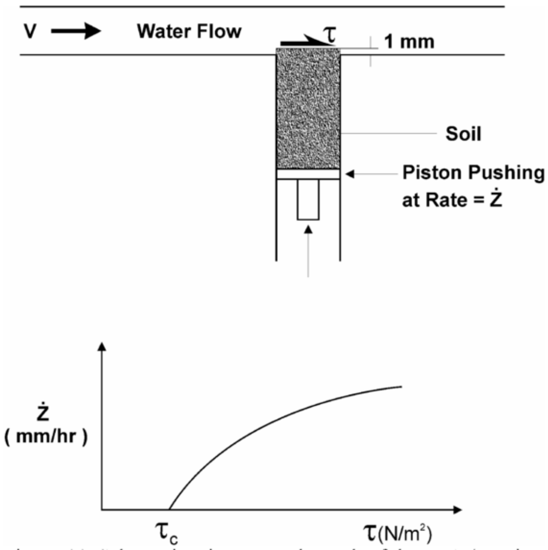


Figure 44: Schematic Diagram and Result of the EFA (Erosion Function Apparatus). (Briaud and Chen, 2005)

In modern times the water retention "curve" is actually interpreted as the response of the soil to changes in suction, when changes in stress are also known. The paper by Cottecchia and Cafaro (2005) illustrates the effects of loading history on the subsequent water retention properties. It also raises the issue of the differences in suction loading and mean stress loading, a matter which requires further clarification.

Having practical applications in mind a substantial research has been performed on the relationship of water retention curves (wrc) and some identification parameters of the soil, typically its grain size distribution but other properties are also included to improve predictive capabilities. Two papers address this issue: Sung et al. (2005) and Lee et al. (2005). Sung et al (2005) explore the possibility of using a reference state (the liquid limit) to derive the wrc for more compacted states. The idea is interesting and more research is needed to extend the obtained results for a more comprehensive description of the wrc. In the second paper Lee et al (2005) propose a neural network approach to predict the drying wrc of a particular class of soils. The paper shows also the difficulties of a number of empirical expressions to predict the actual laboratory data.

Shear strength is another classical subject of research in unsaturated soil mechanics. Suction controlled testing techniques are today common in many laboratories around the world. The accumulation of testing data in the last decade has provided a fairly consistent picture of the effect of suction on strength (Toll, 2000). Four papers included in the Session deal with strength determinations from different perspectives: Vanapalli and Nishimura (2005), Chae et al (2005), Vesga and Vallejo (2005) and Jimoh (2005). The limited variation of friction angle with suction and the more relevant effect of suction on apparent cohesion have been described in some of the papers mentioned. A question often raised in the review, however, is the necessary control of the suction prevailing during shear in the failure plane. This in turn requires suction controlled testing and a sufficiently low shearing rate, an aspect insufficiently addressed in some of the work discussed.

Zhang et al (2005) discuss soil compaction and relate strength with the energy of compaction. It has been argued that suction is a necessary additional variable for a more consistent description of soil strength (and other properties).

Volumetric strains upon moisture (or suction) changes are a key aspect of the behaviour of unsaturated soils which has received considerable attention for many decades. A practical approach is to relate potential volume changes to basic identification variables. Cui et al. (2005) found that the available empirical procedures to identify collapse potential are quite unreliable. Procedures to identify the expansive potential of clayey soils are discussed by Skutnik and Garbulewski (2005). It has been stressed in the discussion of the paper that for a given soil mineralogy, the expansion is essentially controlled by three variables: the initial packing (void ratio), the initial suction and the applied confining stress.

Unsaturated soil modeling is addressed in two papers: Mrad et al (2005) which use an elastoplastic model to describe suction controlled oedometer tests on an exapansive clay and Ng and Zhou (2005) which report dilatancy data measured in suction controlled direct shear tests. There is a lack of comprehensive testing programs in expansive soils due to the very high equilibration times. Mrad et al (2005) provide an interesting set of data in this regard, although limited to oedometric stress paths. Dilatancy data is required to formulate the flow potential of constitutive models. It was found (Ng and Zhou, 2005) that suction enhances dilatancy of a compacted residual granitic soil but pure suction effects are difficult to separate in this case from density effects, since the application of increasing suction leads to a progressively more compacted soil.

In a final paper summarized in this Report, Romana and Serón (2005) found that swelling strain rates of an expansive clay-stone depend linearly on time. This approximation may be used to reduce testing times if the protocol suggested by Huder-Amberg is followed to identify swelling pressures and swelling strains of expansive clay-stones.

Questions for discussion:
Inspired in part by the reviewed papers the following questions are suggested for discussion:
- In compacted soils, what is the effect of the initial structure and its subsequent modifications induced by suction changes (drying or wetting after compaction) on the soil response during the operational stage of the compacted structure?
- Are there alternative variables to the classic ones of water content and dry density to describe more properly a compacted soil?
- Effect of previous stress and suction path on the water retention characteristics of a given soil
- What is the reliability of current empirical approaches to determine the wrc on the basis of basic identification data?. How can it be improved?
- What is the reliability of current empirical approaches to determine collapse and swelling potential of natural soils?

Clay soils. Long term effects
Continuous strain records under a preloading embankment are probably the best procedure available to investigate the heterogeneity of natural clay formations and, at the same time, to derive reliable parameters of primary and secondary compression behaviour. The case history described in Cola and Simonini (2005)'s paper is a good illustration of this comment. On the basis of the recorded "in situ" data the authors provided continuous plots of primary and secondary compression indices as well as the OCR.

Lacasse and Berre (2005) and Szymanski et al (2005) discuss creep effects under deviatoric stress. This is relevant in practice when safety factors against shear strength failure are low. Lacasse and Berre propose a procedure to identify the creep susceptibility of a given soil, based on a "creep susceptibility diagram" in which data for the creep rate observed one minute after the application of a deviatoric load is compared with the deviatoric stress ratio. Szymanski et al (2005) perform a similar analysis and use some empirical equations for creep rate which are similar to the equations reported by Lacasse and Berre. Interestingly, the strain rates measured by Szymanski et al (2005) at t = 1min are apparently one to two orders of magnitude lower than the values reported in the Creep susceptibility diagram of Lacasse and Berre. Yet, they indicate the occurrence of tertiary creep, leading to failure, in one of the examples given.

Taibi et al (2005) provide some triaxial test results on temperature effects on the stress-strain behaviour of a low plasticity

soil. The soil selected is not very sensitive to temperature effects but the results shown are consistent with a general framework which predicts a reduction of the elastic domain as the temperature increases. Rodríguez (2005) interprets triaxial data on the natural high plasticity Bogotá clay within the framework of a Critical State model, although no model simulations of the experimental data is given. In the paper by Elias and Titi (2005) it was found that sample size may have a significant effect on the measured resilient module of some statically compacted low plasticity cohesive soils, in the sense that the smaller the specimen diameter the larger the measured stiffness. No physical explanation for this effect is however offered in the paper.
Questions for discussion:

- In heterogeneous soft sediments (such as the soils in the Venezia area, described by Cola and Simonini) is there a chance for a conventional soil investigation (samples + laboratory tests) to lead to a reliable model for soil compression behaviour in the short and long term?
- Are the creep susceptibility diagrams relating creep rate and deviatoric stress ratio (Lacasse ad Berre's paper) sufficiently general to provide a guide for creep susceptibility?.

Granular soils
Principal stress rotation may lead to significant effects when the soil has an anisotropic fabric. However, the sand tested by Naughton and Kelly (2005) seemed to be very isotropic and only a very limited effect of stress rotation on the "gross "or general yield envelope could be detected. Results were obtained in a comprehensive series of hollow cylinder tests, interpreted with the help o three alternative yield criteria.
Another hollow cylinder experimental research is reported by Kuwano and Nahadi (2005). The interest in this case is to investigate in detail the yield behaviour inside the gross yield surface. Following previous research they find two inner yield loci: one which establishes the limit of true elastic behaviour and an intermediate one which marks the rapid degradation of soil moduli. Both inner surfaces exhibit strain induced anisotropy and non-associated behaviour. On the contrary, the general yield surface reacts isotropically and it seems to be associated in the deviatoric plane.

The behaviour of sand under very high stresses (up to 60 MPa) is investigated by Lade et al (2005). They found, in compression tests, that the memory of the initial sand packing disappears beyond confining stress of 15 MPa. Void ratio at failure in compression and extension tests follow also a regular "clay" behaviour at high stresses. This change is attributed to particle crushing. For the sand tested, friction angles in compression seem to be independent of initial density for confining stresses in excess of 15 MPa but this is not the case for extension tests.

The addition of a small amount of fines to a granular soil creates a contractive response for low confining stresses. Bobei and Lo describe this behaviour already identified in previous research and propose a new state parameter in terms of the distance between the NCL and the CSL, ψ, the current void ratio and a stress factor. This research is relevant in connection with the undrained behaviour of saturated sand and with the risk of static or dynamic liquefaction. The paper by Lipinski and Wolski (2005) addresses the same problem from a similar perspective. They found, for granular soils with different amount of fines, that some relevant parameters, such as the undrained shear strength ratio could be related to parameter ψ. It would be interesting to check if the new state parameter proposed by Bobei and Lo improves the correlations given by Lipinski and Wolski.

The undrained response of low density natural silts against cyclic loading is further investigated by Wijewickreme and Sanin (2005), again with the purpose of finding conditions for cyclic mobility and static liquefaction. As expected, the cyclic stress ratio to reach a target shear deformation (3.75%) decreases with the amount of applied cycles. An unexpected result is that an intermediate period of consolidation (which densifies the soil) results in a decrease of the number of loading cycles necessary to reach the target shear strain. This result is explained as a change in soil structure but no further data on this assumed change is given in the paper.

The cyclic behaviour of steel-gravel interfaces is described by Zhang and Zhang (2005) on the basis of tests performed in a large direct shear machine. A significant result is the accumulation of normal deformations presumably as a consequence of particle breakage, not measured in the tests.

The Rankine and Sivakugan (2005) paper examines the permeability and strength properties of dense, hydraulically placed mine fills. They were found to be more impervious than expected. The high friction angle is found to be correlated with the attained dry density.

Questions for discussion

- Sand looses the memory of the initial density as the confining stress increases. This behaviour is attributed to the breakage of particles. Size effects make these phenomena increasingly relevant as the grain size increases. But, other factors such as grain petrology, grain shape and grain size distribution control particle breakage. With this background, are there rules to predict the confining stress which marks the transition to "clay like" behaviour other than expensive laboratory testing?
- Since particle breakage is the physical phenomena controlling sand static behaviour at high confining stress but also cyclic behaviour of other granular materials such as gravel, shouldn't it be explicitly or implicitly incorporated into frameworks for the behaviour of granular aggregates?
- Is the state parameter ψ enough to describe for practical purposes, the static undrained behaviour of sands with variables amounts of fines?.
- Can the previous history of loading be incorporated in criteria for undrained cyclic mobility in a simple way?

Soil mixtures and soil design
The title of this section requires a minimum explanation. Soil mixtures of widely different grain size are often found in practice. The determination of their properties is not an easy task as the paper of Pedro et al (2005) state. From a different perspective artificial soil mixtures are often used in a variety of applications. Examples are the subgrade materials used in pavement design or the aggregates of highly compacted clay pellets, which behave as granular materials when dry or as a highly swelling material when wet. One may wonder if artificial soils can be designed in order to meet a set of specifications, including hydraulic, mechanical, thermal or chemical target values. The relationship between shape of particles and a number of engineering properties of the resulting soil has been discussed recently by Santamarina and Cho (2004). The Session includes four papers, having different motivations but all of them have in common the investigation of the properties of mixtures of diffrents constituents: glass balls and sands and sand and kaolin (Reiffsteck and Pham, 2005); gravel and sand (Pedro et al, 2005); sand and high plasticity clay (Watabe and Saitoh, 2005) and gravel cement mixtures (Watanabe et al 2005). This "direct" approach is probably the first necessary step before a full design exercise could be performed.

In three of the papers mentioned triaxial tests were performed to investigate strength and stress-strain properties of the mixtures. The development of a granular network within the soil when the percentage of the coarse particles exceeds a given threshold value marks a transition in properties. On the opposite end, the dominant effect of the fine fraction is identified when the coarser particles loose contact among them. This is well illustrated in the work of Reiffsteck and Pham (2005). Pedro et al (2005) found that the triaxial stress-strain response is independent of the size of the coarser component, for a given range of sizes and for a fixed proportion of coarse/fine materials. It would be interesting to know the limits f this result for other proportions and ranges of coarser particles. The attained pore size distribution, as discussed in Watabe and Saitoh (2005), is a good information to interpret hydraulic and compressibility properties. The addition of cement (Watanabe et al, 2005) increases strength but makes the material brittle. Peak strengths depend not only on the attained dry density of the compacted base material but also on the compaction water content, a result probably associated with the availability of water for cement hydration and with the prevailing soil suction at the time of testing.

Questions for discussion

- What are the most informative identification properties of components in order to predict the geotechnical properties of a soil mixture?
- Are there suggested procedures to predict the resulting properties other than direct testing?

Chemo-mechanical interactions

This a wide subject and only a few aspects are covered in the papers presented to the Session. A good review of chemo-mechanical effects in clays may be found in Di Maio et al (2002).

In Graham et al (2005) one of the most interesting findings is the increase in brittleness and the reduction of strength at large deformations in montmorillonitic clay specimens having a high Na/Ca ratio in its pore water. The case highlights the effect of pore water chemistry on clay strength. The cementing effect of salts is illustrated in Foncea et al (2005) paper which investigates the collapsing properties of saline soils. However, the actual collapse potential is also governed by the permeability of the soil. Open, more pervious structures are more susceptible to bond dissolution and therefore to experience severe settlements upon flooding.

The "in situ" undrained strength of Swedish soils with a significant proportion of sulphur and organic content is described in Westerberg et al (2005). No conclusive evidence on the effect of sulphide content on strength is given but the discrepancies between alternative methods to determine c_u are not generally found in more common clay soils.

Questions for discussion

- Is there a simple consistent framework to interpret the effect of pore water chemistry on the mechanical behaviour of clay soils?

Experimental techniques

Only a few isolated aspects of this vast subject are covered in the Session. The effect of necking in triaxial extension tests is discussed by Yamamuro and Liu (2005) who report the results of a number of tests on samples with different aspect ratios. Necking effects are very much reduced for small H/D ratios (H/D = 0.5 leads already to an acceptable response). End friction effect on the overall specimen response against triaxial loading is investigated in some detail by Perić and Su (2005) by means of a series of finite element simulations. The clay was described by means of a cam-clay model. They found significant deviations from the expected overall behaviour of the sample, although the stress paths of points located in the central part of the specimen are much closer to the theoretical response. The results demonstrate also the advantage of local measurements when end friction is not avoided.

Grammatikopoulos and Anagnostopoulos (2005) derive friction angles from the slope of the observed failure planes in triaxial tests. The approach is difficult to recommend in view of the theoretical complications of the phenomenon. Hiltunen and Choi (2005) discuss the possibility of deriving the cross-anisotropic elastic constants from compression and polarized shear wave velocities determined in cross-hole tests. The approach is certainly promising but only preliminary results are reported, as well as references to the theoretical formulation of the problem Piriyakul and Haegeman (2005) describe the procedure to run K_0 tests in stress path triaxial equipment. Finally, Briaud and Chen (2005) describe a specialized test very uncommon in Soil Mechanics Laboratories. They describe the basis of soil erosion by flowing water and the equipment developed, which simply requires tube samples recovered in standard boring operations.

ACKNOWLEDGEMENTS

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Papers submitted to Osaka'05:

Bobei, D.C., Lo, S.R. Reverse behaviour and critical state of sand with small amounts of fines (AS-09)

Briaud, J.-L., Chen, H.-C. The EFA, Erosion Function Apparatus: An overview (US-03)

Cafaro, F., Cotecchia, F. A structure-based approach to the estimate of the water retention curve of soils (IT-07)

Chae, Y.-S., Kim, C.-K., Lee, K.-I., Kim, T.-H. Shear characteristics of an unsaturated compacted g `ranite soil (KO-11)

Cola, S., Simonini, P. Relevance of secondary compression in Venice lagoon silts (IT-12)

Cui, Y.J., Delage, P., Marcial, D., Terpereau, J.-M., Marchadier, G. On the collapse susceptibility of the loess of Northern France (FR-11)

Elias, M., Titi, H.H. Effect of sample size on resilient modulus of cohesive soils (US-65)

Foncea, C., Acevedo, P., Olguín, R. Geotechnical characterization of saline soils (CH-2)

Graham, J., Man, A., Alfaro, M., Blatz, J.A., Van Gulck, J. Gypsum cementation and yielding in plastic clay (CA-04)

Grammatikopoulos, I.N., Anagnostopoulos, C.A. Difference between the values of friction angle Φ derived from the theoretical fracture plane and the reliable one obtained from triaxial tests (GR-6)

Hiltunen, D.R., Choi, Y. Cross anisotropic stiffness properties of soils via crosshole seismic wave measurements (US-19)

Jimoh, Y.A. Shear strength/moisture content models for a laterite soil in Ilorin, Kwara State, Nigeria (NI-3)

Kuwano, J., Nakada, T. Effects of shear stress history on yielding of dense Toyoura sand in p'-constant shear plane (JA-08)

Lacasse, S., Berre, T. Undrained creep susceptibility of clays (NO-15)

Lade, P.V., Yamamuro, J.A., Bopp, P.A. Relative density effects on drained and undrained strengths of sand at high pressures (S-26)

Lee, S.R., Kim, Y.K., Lee, S.J. A method to estimate soil-water characteristic curve for weathered granite soil (KO-12)

Lipiński, M.J., Wolski, W. Parameters describing flow liquefaction of soils (PL-05)

Mrad, M., Cuisinier, O., Abdallah, A., Masrouri, F. Modelling of the hydromechanical behaviour of unsaturated swelling soils under high suctions (FR-33)

Naughton, P.J., O'Kelly, B.C. Yield behavior of sand under generalized stress conditions (IL-1)

Ng, C.W.W., Zhou, R.Z.B. Effects of soil suction on dilatancy of an unsaturated soil (HO-1)

Nishimura, T., Vanapalli, S.K. Volume change and shear sstrength behavior of an unsaturated soil with high soil suction (CA-05)

Perić, D., Su, S. Influence of the end friction on the response of triaxial and plane strain clay samples (US-37)

Piriyakul, K., Haegeman, W. Automated K0 consolidation in stress path cell (BE-02)

Rankine, K.J., Sivakugan, N. Drainage characteristics and behaviour of hydraulically placed mine fill and fill barricades (AS-06)

Reiffsteck, Ph., Nguyen Pham, P.T. Influence of particle size distribution on mechanical behavior of a soil (FR-28)

Rodriguez, J.A. Yielding and stress-strain relationships for Bogotá Clays (CO-4)

Romana, M., Seron, J.B. Characterization of swelling materials by Huder-Amberg oedometric test (SP-14)

Skutnik, Z., Garbulewski, K. Suction-swelling relation for Warsaw clays (PL-07)

Sung, S.-G., Lee, I.-M., Cho, G.-C., Reddi, L.N. Soil-water characteristic curve assessment using a reference sate concept (KO-05)

Szymański, A., Lechowicz, Z., Drożdż, A., Sas, W. Geotechnical characteristics determining consolidation in organic soils (PL-09)
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Watabe, Y., Saitoh, K. Influence of sand fraction on compressibility and hydraulic conductivity of clayey soils (JA-09)
Watanabe, K., Tateyama, M., Yonezawa, T., Aoki, H. Strength characteristics and construction management of cement-mixed gravel (JA-10)
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Wijewickreme, D., C. Sanin. Some observations on the cyclic loading response of a natural silt
Yamamuro, J.A., Liu, Y. Effects of necking and its suppression in axisymmetric extension tests on clay (US-56)
Zhang, G., Zhang, J.-M. Test study on behavior of interface between structure and coarse grained soil (CN-2)
Zhang, Z., Tao, M., Tumay, M.M. Energy concept and soil compression (US-53)

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