2nd International Workshop of Young Doctors in Geomechanics

Ecole Nationale des Ponts et Chaussées
Champs-sur-Marne, France
November 23 – 25, 2005

V. De Gennaro, J.-M. Pereira & P. Delage, editors
Ecole Nationale des Ponts et Chaussées, France
2nd International Workshop of Young Doctors in Geomechanics

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Foreword

The scope of W(H)YDOC 05 (Paris, November 23 - 25, 2005) was to bring together young geotechnical doctors within an informal invited Workshop, so as to allow for the presentation of researches carried out during their PhD thesis. Students completing their last PhD year were also welcomed. Several senior researchers from various European Universities acted as discussion leaders during the sessions. The Workshop aimed at favouring informal and constructive exchanges about recent research results and ideas.

The Universities that agreed to participate to W(H)YDOC 05 are listed below; various of them are already partners within the MUSE, AMGISS, DIGA European networks and ALERT Geomaterials Association: CEDEX Madrid (SP), National Technical University of Athens (GR), University of Durham (UK), Ecole Nationale des Ponts et Chaussées (F), Graz University of Technology (AU), University of Glasgow (UK), Institut National Polytechnique de Grenoble - UJF (F), Ecole Polytechnique Fédérale de Lausanne (CH), Universitat Politècnica de Catalunya (SP), Università di Trento (I), Politecnico di Torino (I), University of Stuttgart (D), Georgia Institute of Technology (USA), Swiss Federal Institut of Technology Zurich (CH), University of Strathclyde (UK), INSA Lyon (F), Pennsylvania State University (USA), Universität di Salerno (I), NTNU Trondheim (NO), Università di Napoli Federico II (I), Ecole Nationale des Travaux Publics de l'Etat Lyon (F), Cukurova University (TR). A total of 19 contributions coming from 10 countries were presented during the Workshop. This book contains the summaries of these contributions, outlined in a short paper provided by the authors. They would be useful for all researchers to have an idea of the ongoing research activities in the field of geomechanics.

A special session of W(H)YDOC 05 was dedicated to the presentation of the European Networks MUSE (unsaturated soils) and AMGISS (soft soils). The PhD students involved in these networks had the opportunity to present their starting projects. Their abstracts are also included in these proceedings.

We are grateful to people who ensured the scientific co-ordination of the Workshop, all the "young doctor contributors" and the PhD students from the RTN networks, who animated the three days meeting. Thanks are due also to their Tutors, who accepted to cover part of their fees of attendance. We acknowledge Prof. Carlos Santamarina (Georgia Tech., USA), Prof. Pieter Vermeer (University of Stuttgart, G) and Prof. Giovanni Barla (Politecnico di Torino, I) for their kind acceptance to hold the keynote lectures presented during the three days of the Workshop.

This Workshop was supported by the sponsorship of: AMGISS (Advanced Modelling of Ground Improvement on Soft Soils) EC Marie Curie RTN, DIGA (Degradation and Instabilities in Geomaterials with Application to Hazard Mitigation) EC Marie Curie RTN, MUSE (Mechanics of Unsaturated Soils for Engineering) EC Marie Curie RTN, ALERT Geomaterials, ENPC (Ecole Nationale des Ponts et Chaussées, F), EDF (Electricité de France), IRSN (Institut de Radioprotection et de Sûreté Nucléaire, F), TOTAL, FNTP (Fédération Nationale des Travaux Publics, F). Their participation is here kindly acknowledged.

Vincenzo De Gennaro
Jean-Michel Pereira
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Champs, November 2005
THM couplings
THE HYDRO-MECHANICAL COUPLING IN SINGLE FRACTURE THROUGH NUMERICAL MODELLING

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ABSTRACT. A mixed BEM-FEM technique has been implemented into a code for the simulation of deformation and pore pressure in fractured rock masses. The hydro-mechanical coupling has been considered with the purpose of evaluating the influence of the fracture deformability on the transient flow response in single fractures.

1. Introduction

In a rock mass induced water-pressure perturbations may influence the state of stress up to promote a significant change of the transmissivity. The use of a numerical technique which accounts for the hydro-mechanical (HM) coupling may result essential for an appropriate prediction of the fluid flow, in particular in the near field. A computer code has been developed aimed at simulating in perspective the fluid flow regime and strain field in a discrete assembly of rock blocks and percolative fractures. In this paper the code is used for a simple system constituted of two impervious blocks and a saturated fracture, where the fracture stiffness can be normal-stress dependent and the transmissivity changes with the fracture closure. The results of the code highlight the importance of the HM coupling in fractured rock masses and are promising in view of the extension to multiple fracture systems.

2. The numerical code outlined

If a discrete model is assumed, for the simulation of the HM coupling a local stress balance equation for both the blocks and fractures is required, in conjunction with a diffusion equation for the fractures only. In the latter, a term associated with the fracture volumetric deformation has been introduced to render the HM coupling effect (Cammarata, 2005; Cammarata et al., 2005). Furthermore, a specific relation linking the normal stress with the normal closure is needed for the fractures.

The local stress balance equations for the blocks are algebraically transformed through the use of a Boundary Element Method (BEM) direct formulation, whereas the Finite Element Method (FEM) and Galerkin weighted residuals are used to reduce the diffusion equation. The time derivatives are written in finite differences. The non-linearities are solved by means of the Picard method. Further details on the numerical derivations and discretization are available elsewhere (Cammarata, 2005).

3. Transient flow in a single fracture

In plane flow and plane strain conditions a fracture 100 m deep having initial aperture \( e = 10^{-4} \) m, confined by two impervious blocks, is subject to an injection pressure increase \( p_i \) of 500 kPa (Figure 1(a)). The fracture terminates over a region of high permeability where the pressure is kept constant (nil pressure increase). The fracture transmissivity \( T_f \) varies with the cube of the aperture \( e \). The blocks have Young modulus \( E_b = 10 \) GPa and Poisson ratio \( \nu_b = 0.25 \).

The results in terms of flow rates \( Q \) at the injection point are shown in Figure 1(b), with reference to normal stiffness of the fracture \( k_{nn} \) equal to 0.1, 1, 10 and 100 GPa/m. When \( k_{nn} = 0.1, 1, 10 \) GPa/m a peak of \( Q \) is experienced. In this particular case the fracture is propped open by the pressure and \( Q \) increases as a consequence of the increased transmissivity and high pressure gradient. The steady-state values of the flow rate increase with the reduction of the \( k_{nn} \) of the fracture (Figure 1(b)). The value of \( T_f \) cannot be considered constant throughout the process but rather changes locally and with time as the pressure inside the fracture modifies...
The pressures at long-term conditions do not conform to the pure steady-state hydrological solution (dashed line in Figure 2(b)). This result is the major implication of the use of the HM coupling.

![Figure 1](image1.png)

**Figure 1.** (a) Scheme of the example. (b) Flow rate $Q$ at the injection point with time $t$ for different $k_{nn}$.  

![Figure 2](image2.png)

**Figure 2.** (a) Transmissivity $T_f$ and (b) pressure $p$ distribution versus time $t$: $k_{nn} = 0.1 \text{ GPa/m}$.  

![Figure 3](image3.png)

**Figure 3.** (a) Saeb-Amadei model and parameters values used in the example. (b) $Q$ at the injection point computed with a stress-dependent $k_{nn}$ compared with the results for two constant $k_{nn}$.  

The Saeb-Amadei (1992) model (Figure 3(a)), is used here to account for a stress-dependent normal stiffness. An initial effective stress $\sigma_{n,ini}$ of 700 kPa is considered at the fracture level. Under the application of $p$, the effective stress after drainage is 200 kPa ($\sigma_{n,end}$). In Figure 3(b) the flow rate is compared to the results for two simulations with constant $k_{nn}$, respectively equal to $k_{nn,sec1}$ and $k_{nn,sec2}$ (secant values to $\sigma_{n,end}$ and $\sigma_{n,50\%} = 450$ kPa, see Figure 3(a)). The results for the stress-dependent $k_{nn}$ and $k_{nn,sec}$ in terms of $Q$ coincide, whereas with $k_{nn,sec1}$ a mild increase is obtained. It is apparent that the use of an appropriate constant value of $k_{nn}$ provides acceptable results when dealing with injection problems, and more refined non-linear models can be eventually disregarded.

4. Conclusion

The prediction of the fluid flow into deformable fractures has been considered in this paper through the use of HM coupling concepts and a suitable numerical technique. The analysis of a simple problem of two blocks and a fracture shows that for “soft” fractures the variation of fracture transmissivity with the fracture closure plays a significant role. This circumstance forces
to consider the HM coupling concept. Instead, the pure hydrological solution (i.e. no coupling and constant transmissivity) holds only for “rigid” fractures. The use of a stress-dependent stiffness for the simulation of injection tests is not mandatory if an appropriate value of constant stiffness is input. In this way an unnecessary source of complexity can be removed from the computational scheme.

5. References


EXPERIMENTAL STUDY OF THE HYDROMECHANICAL COUPLING IN ARGILLACEOUS ROCKS: THE EXAMPLE OF BOOM CLAY

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ABSTRACT. Deep geological formations are increasingly considered as a potential site for nuclear waste disposal. During excavation, a disturbed zone (EDZ) develops in which there can be considerable damage which can significantly modify fluid flow and permeability in the rock mass. In this framework triaxial tests were performed on Boom Clay to characterize its hydromechanical behaviour. A number of technical improvements were needed.

1. Introduction

The study of the feasibility of storing radioactive wastes in deep geological formations includes the assessment of the efficiency of the host rock. Stress relief, due to excavations, may have major consequences on the hydromechanical properties of the host geomaterial. Development and propagation of fractures weaken its mechanical properties and can alter the fluid flow in the rock. Argillaceous formations can be considered as potential sites as they form a low permeability (less than $10^{-18} \text{m}^2$) geological barrier. Nevertheless, argillaceous rocks have a complex hydromechanical behaviour that is governed by numerous coupled processes and by strong interactions between the clay particles and the interstitial fluid. Experimental and numerical investigations of this kind of geomaterials are still challenging. Boom Clay is one example of these argillaceous rocks. Most studies to determine its geological and geotechnical properties and to study its thermo-hydro-mechanical behaviour have been carried out during the last thirty years (e.g. Horseman et al., 1987, Baldi et al., 1991, Sultan, 1997, Delage et al., 2000, etc.). In this framework, isotropic compression tests and triaxial compression tests, including permeability measurements, were performed to establish the constitutive behaviour and to characterize the evolution of permeability with stress (isotropic and deviator).

2. Main characteristics of Boom Clay

Boom Clay is a marine clay deposited in the north of Europe. Intact specimens tested in this study come from the SCK-CEN underground research laboratory at Mol in Belgium, located at a depth of 223 meters from the ground surface. Boom Clay is an overconsolidated argillaceous rock ($OCR \approx 2.2$) of high plasticity, prone to swelling. The very low permeability ($k \approx 3.10^{-19} \text{m}^2$) is contrasting with a high porosity ($n \approx 36 \%$). The natural water content is close to 25% and the uniaxial compressive strength is about 2.5 MPa.

3. Experimental set up

A large experimental campaign including more than 20 tests was carried out. Uniaxial compression tests, isotropic compression tests and axisymmetric triaxial tests (compression and extension) were performed.

The experimental installation used includes a high pressure triaxial cell with two independent drainage lines, a system of internal displacement measurements and a system to stabilize the temperature in order to limit the influence of temperature changes on the hydromechanical behaviour. Internal displacement measurements allow to obtain axial and radial strains in a specimen and to detect the onset of strain localisation.

Special care was taken to prepare cylindrical specimens (40 mm height and 40 mm in diameter) without loosing saturation and damaging the material. Special procedures were also developed to prepare and set up the specimen in the triaxial apparatus in the best conditions.

A permeability measurement procedure suitable for swelling geomaterials was specially developed, based on the classical constant head method (Coll, 2005).
4. Results

Under isotropic loading for a mean (Terzaghi) effective stress less than 3 MPa, an important tendency to swelling is observed. Short term volumetric behaviour is governed by mechanical effects (stress variations), initial water content and physio-chemical effects: consolidation (i.e., volume changes due to variations of effective stress) and swelling processes are both developing simultaneously. In the long term (constant effective stress), physio-chemical effects inducing significant swelling prevail on classical consolidation mechanism which tends to vanish. Interactions between the clay particles and the porous fluid allow the adsorption of water inducing an increase of the aggregates volume. For higher mean effective stresses (greater than 3 MPa) this swelling practically disappears.

Under deviatoric loading Boom clay specimens can exhibit overall strain-hardening or strain-softening with strain localisation, depending on the initial state (mean stress $p'_0$ and water content) as well as on the strain rate. On Figure 1 are plotted the stress-strain relations (Figure 1a) and volumetric strain versus axial strain (Figure 1b) for some consolidated drained triaxial tests, $p'_0$ ranging from 0.4 Mpa to 5 Mpa. For low $p'_0$ (0.4 Mpa), an overall softening behaviour with strain localisation is observed and we showed that the higher the initial water content the lower the shear strength (Figure 1a). This is due to changes at the microstructural level (strength loss of the bonds between clay particles) during the consolidation stage characterized by swelling. For higher $p'_0$ (2.3 and 5 MPa), the behaviour becomes ductile-contractant. The more contractant behaviour is obtained for the highest value of $p'_0$ (5 MPa) (Figure 1b). In addition, we observed that softening-dilatant behaviour and strain localisation are induced by higher strain rate (Figure 1a).

![Figure 1](image)

Figure 1. Shear tests on Boom Clay isotropically consolidated at various mean effective stress. (a) deviator stress-axial strain curves, (b) volumetric strain-axial strain curves.
Permeability was found to decrease when $p'_0$ increases related to the collapse of the largest existing pores. The general trend is more or less linear at least up to $p'_0$ equal to 10 MPa. In addition, an isotropic compression test during which $p'_0$ was increased up to 32 MPa showed the influence of stress history on permeability. On the other hand, no significant evolution of the permeability due to strain localisation was observed. Nevertheless, permeability tests highlighted the question of global versus local measurements when dealing with a few extremely thin zones of localised strain as shown on Figure 2.

*Figure 2. Zones of localised strain observed on a specimen sheared from $p'_0$=0.4 MPa*

Several procedures were developed and improved to study the hydromechanical behaviour of argillaceous rocks. The complex behaviour of Boom Clay was studied and the influence of the microstructure on the macroscopic behaviour (volumetric strain and permeability evolution) was highlighted.

5. Références


A LABORATORY HEATING TEST FOR THERMO-HYDRAULIC IDENTIFICATION OF ROCKS

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ABSTRACT. Within the framework of the theoretical and experimental analysis of coupled Thermo-Hydro-Mechanical (THM) processes developed in geological barrier, a laboratory heating test has been performed in an argillaceous rock known as Opalinus Clay. A heating cell was designed and built in order to measure the rock reaction against a thermal load, under controlled conditions of temperature and pores water pressure.

1. Introduction

The Opalinus Clay formation (Mont Terri, Switzerland) is being considered as a reference host rock for deep geological repositories of nuclear waste. In order to measure the rock reaction against a thermal load under controlled conditions of temperature and pore water pressures, a laboratory heating experiment has been carried out in a heater cell designed and built in the geotechnical laboratory of the UPC. The objective of this test is measure the pore water pressure evolutions and temperature evolutions of the rock, during hydration, heating and cooling phases. Numerical simulations of the experiment have been carried out with CODE_BRIGHT (Olivella et al., 1996), where the hydration, heating and cooling phases are reproduced.

When a heat source such as a canister of radioactive waste is buried in a saturated rock, the heat source will cause a temperature increment in the rock. The rock skeleton and the pore water will expand. However, the thermal dilatation coefficient is larger than the skeleton coefficient. As a result pore water pressure will increase. The magnitude of the water pressure depends on the rate of temperature increase and also on a number of rock parameters (stiffness, permeability, porosity). The increase in water pressure may lead, in extreme conditions to rock fracturing. The risk of fracture is controlled by the mentioned parameters, as well as the boundary conditions of the problem. (Booker & Savvidou, 1985; Ma & Hueckel, 1993) have analyzed these phenomena regarding the response of saturated soil against temperature changes.

2. Description of the cell

The cell designed consists of a stainless steel ring of 75 mm in diameter and 100 mm in height, where the sample is housed. An upper lid and bottom lid are screwed to the ring in order to maintain the sample hermetically sealed. The sample is 70 mm in diameter and 100 mm in height. The annular gap between the sample and the ring is filled with epoxi resin. Figure 1 shows detail of the cell built.

The ring has eight inlets which allow the installation of different type of sensors into the rock sample. The cell has been instrumented with two miniature pore water pressures sensors and two temperature sensors located into the rock. The swelling pressure is registered in the ring by means of two full Wheatstone bridges. The vertical strain of the ring was measured with two strain gages diametrically opposed. The Wheatstone bridge was completed by means of tow calibrated electric resistances. The circumferential strain of the ring was measured with two strain gages diametrically opposed and rotated 90° with respect to the vertical strain gages. The outlet voltage of each Wheatstone bridge was calibrated by means of water pressure. The strain gages were placed in a central band having a reduced thickness. A reduction thickness was also performed in the upper lid in order to measure the bending deformation produced by the inner pressure. A vertical heater, formed by an electrical resistance of cartridge type of 6.0 mm in diameter and 40 mm length, is inserted into the sample.
3. Preliminary results

Figure 2 shows the temporal evolutions of temperature (upper) and pore water pressure reactions (bottom) generated by heating with a power applied of 5 Watt. A sudden pore water pressure increment from 0.6 MPa to 2.2 MPa was measured at the beginning of heating. The dissipation of pore water pressure began when the heating rate decreased. A fall in temperature was produced when the power was removed. As a result a sudden reduction in pore water pressure was produced and negative pore water pressure values of this were recorded.

4. Numerical simulations

A coupled THM numerical model of the heater cell has been performed with CODE_BRIGHT. Figure 3 shows the spatial distribution of temperatures, at the end of the heating phase. An elastoplastic model including damage effects describe the rock behaviour (Vaunat & Gens,
2003). The main controlling parameters of pore water pressure generation and dissipation are permeability, porosity and rock stiffness. They are identified by matching the experimental results with the calculations.

5. References


THERMAL CONDUCTIVITY OF COMPACTED MX80 BENTONITE

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ABSTRACT. The thermal conductivity of MX80 bentonite specimens compacted at different dry densities and water contents was measured. The results evidenced the influence of dry density, water content, saturation degree, mineralogy and volume fraction of soil constituents on the thermal conductivity. A method was proposed for the prediction of the thermal conductivity of compacted bentonite. Its relevance was evaluated.

1. Introduction

Compacted bentonite is often considered as a buffer material for high-level radioactive waste disposal. Its thermal conductivity is one of the key properties for the design of the whole storage system. Farouki (1986) described the thermal properties of soils in an extensive manner by analysing the experimental data on various soils. The author clearly showed the main factors which affect the thermal properties of soil, and made some recommendations as regarding the method or methods to apply to different types of soil: frozen or not, saturated or not. However, as the microstructure of compacted bentonite significantly differs from that of other soils, it appears necessary to deeply examine the thermal conductivity of this particular material. Indeed, early study of Coulon et al. (1987) realised on 18 smectite clays from 14 clay deposits showed that this parameter strongly depends on water content, dry density, mineralogical composition and microstructure of each soil.

In the present work, the thermal conductivity of compacted MX80 bentonite was studied. This bentonite is one of the reference materials for the engineered barriers. A commercial thermal property analyser that is based on the hot wire method was adapted for the measurement.

2. Material and experimental technique

MX80 bentonite clay contains 75–80% montmorillonite and 3–15% quartz among other minerals. Two types of MX80 bentonite were used in the present work: MX80a provided by Cetco France Company in 1999 and MX80b provided by Cetco Europe Company in 2004. 27 samples (50 mm in diameter, 70 mm high) were compacted at 7 water contents ranging from 8.4% to 30% and at dry densities ranging from 1.45 to 1.80 Mg/m³.

The thermal conductivity of compacted specimens was measured by a commercial thermal property analyser. This instrument is based on the hot wire method where the thermal conductivity is determined by monitoring the heat dissipation from a linear heat source at a given voltage. A thermistor is installed in the middle of the metallic heating wire (6 mm long, 1.28 mm in diameter) to monitor the temperature of the probe during test.

3. Experimental results

The thermal conductivity of compacted specimens is plotted in Figure 1 in function of dry density. It is observed that, for a given test series where samples had the same water content, the thermal conductivity is proportional to the dry density. Three regression lines were added for three samples groups, each having close water contents: 8.4-9.0%, 11.7-11.9%, and 17.4-17.9%. In addition, the two points representing the thermal conductivity of the two specimens at 30% water content were also added in the Figure 1. It appears clearly that the thermal conductivity increases with dry density and water content increase. This observation is in agreement with the results found in the literature (Farouki, 1986; Coulon et al. 1987).

The following conclusions can be taken from deeper analysis on the experimental data: (i) the thermal conductivity of MX80a bentonite is intrinsically higher than that of MX80b because of
a probable higher quartz content in the MX80a bentonite; (ii) at constant dry density, the thermal conductivity increases with the increase of saturation degree; (iii) between the three volume fractions of the three soils components (solid, water, air), the air volume fraction is the most pertinent parameter influencing the thermal conductivity of soil.

![Graph showing thermal conductivity as a function of dry density and water content.](image)

**Figure 1.** Thermal conductivity as a function of dry density and water content.

4. **Modelling**

For the predicting of the thermal conductivity, the following formulation was proposed:

\[
K = \alpha \frac{V_a}{V} + K_{sat}
\]

where \(V_a\) and \(V\) are respectively the air-pore volume and the total volume of soil; \(\alpha\) is the slope of \(K\)–\(V_a/V\) plot; \(K_{sat}\) is the thermal conductivity at saturated state.

The determination of the parameters was based on the results of Series 4: \(\alpha = -2.22\) (\(W/mK\)), \(K_{sat} = 1.15\) (\(W/mK\)). The comparison between the experimental results and the values calculated by the proposed model is presented in Figure 2. It appears that the difference between model prediction and the measurements was less than 20% for all the 27 specimens. This confirms the relevance of the proposed model.

![Graph showing comparison between measured and calculated thermal conductivity.](image)

**Figure 2.** Evaluation of the proposed model.
5. References


Clays
THE BEHAVIOUR OF KAOLINITE-CALCIUM CARBONATE MINERAL MIXTURES

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ABSTRACT. The mixture of kaolinite and calcium carbonate is studied using sedimentation, viscosity and liquid limit tests. These macroscale tests represent a wide range of solid volume fractions and strain levels, with emphasis on high water content conditions to magnify the effects of particle interactions. Results show that interparticle interaction forces depend on mineral surface-fluid interactions, particle geometry and relative particle size.

1. Introduction

The study of mineral mixtures requires a detailed understanding of chemical interactions, interparticle forces and ensuing fabric formation. Interparticle interactions in single mineral systems and in mineral mixtures are a function of Coulombian attraction, double layer osmotic repulsion and van der Waals attraction between particles (Schofield & Samson, 1954; Rand & Melton, 1977; van Olphen, 1977; Santamarina et al., 2002; Palomino & Santamarina, 2005). This study assess and interprets the behavior of kaolinite-calcium carbonate mixtures across a range of solid-fluid fractions and applied strain conditions while addressing differences in particle shape, size, mineralogy, and relative mass fraction. The choice of mixture components reflects the distinct pH-dependent surface charge in kaolin and calcium carbonate.

2. Experimental Studies

The selected experiments include sedimentation, viscosity, and liquid limit measurements. The three types of measurements represent a broad range of mass fractions, strain rates and strain levels. Testing is performed on mixtures of kaolinite and calcium carbonate.

3. Materials

Scanning electron micrographs of base materials are shown in Figure 1. Mineral mixtures are prepared with two kaolin clays, Wilklay SA1 and Wilklay RP2, and two calcium carbonates, ground GCC (#12 White) and precipitated PCC (Rhombic). The selection of kaolinite and calcium carbonate reflects their very different particle and surface charge characteristics. The clay particles are platy, while the carbonates are elongated (PCC) or blocky (GCC). Kaolinite particle faces have a net negative charge, while CaCO₃ particles have a net positive charge at neutral pH conditions (Stumm, 1992). The zeta potential for both materials varies with pH. Kaolinite zeta potential ranges from positive values at very low pH (~ +5 mV at pH 1.5) to

Figure 1. Scanning electron micrographs of test materials: (a) Kaolinite SA1, (b) Kaolinite RP2, (c) PCC Rhombic and (d) GCC #12 White.

Scanning electron micrographs of base materials are shown in Figure 1. Mineral mixtures are prepared with two kaolin clays, Wilklay SA1 and Wilklay RP2, and two calcium carbonates, ground GCC (#12 White) and precipitated PCC (Rhombic). The selection of kaolinite and calcium carbonate reflects their very different particle and surface charge characteristics. The clay particles are platy, while the carbonates are elongated (PCC) or blocky (GCC). Kaolinite particle faces have a net negative charge, while CaCO₃ particles have a net positive charge at neutral pH conditions (Stumm, 1992). The zeta potential for both materials varies with pH. Kaolinite zeta potential ranges from positive values at very low pH (~ +5 mV at pH 1.5) to
negative values at moderate to high pH (up to ~ -50mV at pH 10 - Yuan & Pruett, 1998). The isoelectric point of kaolinite typically ranges between 4 and 6 (Sposito, 1989; Drever, 1997; Yuan & Pruett, 1998), while that of CaCO₃ is between 8 and 10.5 (Siffert & Fimbel, 1984). The significantly different self-buffering pH of the individual minerals implies not only mutual attraction at intermediate pH but also mineral dissolution leading to a different stable pore fluid pH.

4. Selected Results

The RP2:GCC and SA1:GCC sedimentation heights 24 hours after test initiation are shown in Figure 2 (no visible particles remain in suspension). Results indicate increasing sediment height with increasing clay mass percentage. For the RP2:GCC case, the comparison between 100% carbonate and 100% clay sediment heights is consistent with the observed flocculation behaviour of the clay suspension and the dispersive behaviour of the GCC. For the SA1:GCC series, the trend is similar to the RP2:GCC mixtures. A relatively low sediment height is expected for the 100% kaolin SA1 suspension due to the dispersive tendency of the clay at its self-buffering pH (as compared to the same suspension at flocculation pH ≈ 7 and ionic concentration c<0.01M NaCl ).

![Figure 2. Sediment heights for RP2:GCC and SA1:GCC mixtures at 24 hours.](image)

5. Conclusions

The main mechanisms for particle flocculation are electrostatic interactions between the positively charged calcium carbonate and the negatively charged kaolin particles at carbonate-induced pH conditions. The fabric is “opened” when platy clay particles bridge rounded or rice-shaped carbonate particles (round cross-section); a voluminous fabric can be achieved with low percentages of carbonate. Liquid limit results show the behavioural dependence on particle associations gives way to specific surface at high solids content, where the shear strength of the mineral mixtures decrease with increasing carbonate content.

6. References


ABSTRACT. We address in this study the mechanisms and the modelling of drying shrinkage and cracking initiation / propagation. In this paper, we focus on the role of boundary constraints and tensile stresses. Desiccation tests are carried out on soil samples in controlled conditions. Tests results allow for conclusions about parameters governing crack initiation and its modelling.

1. Introduction

Desiccation cracks have been investigated for many years (Corte & Higashi, 1960, Abu-Hejleh, 1993, Konrad & Ayad, 1997). Nevertheless, mechanisms which govern their occurrence, and the links between them and soil parameters are still elusive. Desiccation macrocracks are likely to occur if the shrinkage is constrained and/or if tensile stresses are generated, which reach soil tensile strength. Constraints can arise from different causes (Hueckel 1992): (i) any traction or displacement boundary conditions, (ii) eigen-stress concentrations, and (iii) intrinsic factors, such as soil texture or structure. We focus here on the role of boundary constraints and tensile stresses in crack initiation. For this purpose, desiccation tests are carried out on soil samples in controlled conditions.

2. Experimental results

The soil used is a silt from Bioley (Switzerland). The liquid limit 31.8 %, the plastic limit 16.9 %. The soil was prepared as an initial slurry (w = 49 %). Water retention curve was determined, concentrating on mechanical constraint minimization and volume measurement accuracy (Péron et al. 2006a). Strains are large and irreversible (plastic) during the virgin drying of the soil, up to air entry value, which is about 100 kPa. Rewetting of the soil produces only slight swelling.

Strain variation in time and space, mass water content (w), suction (s) and crack form were investigated during the continuous air-drying of a bar-shaped soil cake (Péron et al. 2006b). Two extreme cases with different mechanical boundary conditions were studied: free desiccation tests (no boundary constraints) and constrained desiccation tests (existence of a bottom constraint in the axial direction), both performed with the same humidity and temperature conditions.

In free desiccation tests, below the liquid limit, strains evolved linearly, and water content was found to be rather homogeneous throughout the cake. Cakes never experienced cracking. Examples of crack patterns obtained during constrained desiccation tests are shown in Figure 1. The first crack appeared on average at w = 24 %, and the last crack never appeared below w = 22 %. In addition, local water contents, measured in the vicinity of each newly formed crack, were always close to 22 %. The local value of suction measured by a tensiometer, in the vicinity of the cracks, was equal to 90 kPa/100 kPa, so close to air entry value. Axial strains were noticeably decreased compared to unconstrained case. Opening of all the cracks was also measured once each crack was fully developed. A marked tendency for successive crack openings to decrease was observed.
3. Analysis of the results

**Origin of cracks**

It is assumed that the desiccation induced stress increment \( d\sigma_{ij} \) may be expressed by:

\[
d\sigma_{ij} = E_{ijkl}d\varepsilon_{ij}^m = E_{ijkl}(d\varepsilon_{ij}^m - d\varepsilon_{ij}^h)
\]

where \( E_{ijkl} \) is an incremental stiffness tensor, \( d\varepsilon_{ij}^m \) refers to the mechanical strain responsible for stress generation, \( d\varepsilon_{ij}^h \) is the observed total strain rate, \( d\varepsilon_{ij}^h \) is a free desiccation strain rate (the only one occurring if desiccation is not constrained). Thus, tensile stress in a soil undergoing desiccation depends on two factors: constraint induced strains \( (d\varepsilon_{ij}^m - d\varepsilon_{ij}^h) \), and change in incremental soil stiffness \( (E_{ijkl}) \). For the discussed experimental setup, during constrained desiccation tests, constraint induced stresses arise due to boundary constraints, as possible water content gradients, which do not satisfy compatibility conditions are negligible, and given that cracks are perpendicular to the axial bottom constraint.

**Parameters controlling desiccation induced stresses and crack initiation**

Mechanical strains \( d\varepsilon_{ij}^m \) (assumed to be equal to the difference between observed constrained desiccation strain and free desiccation strain) have been calculated up to first crack appearance, at \( w = 24\% \): as the sample loses water, it experiences a measurable mechanical strain (in tension) leading to the building up of tensile stresses.

The building up of calculated mechanical strains \( d\varepsilon_{ij}^m \) is governed by the nature of the constraint, and by the amount of free desiccation strains experienced by the cake (see Equation 1). In consequence, desiccation stress building up can be seen as being largely determined by the nature of suction induced strains. Large plastic strains experienced in a saturated state, are believed to be one of the cause of stress generation and subsequent cracking. Crack onset occurs at the end of this phase, close to air entry value and shrinkage limit.

Finally, using linear elasticity, a first crack opening value has been calculated, which is significantly higher than the measured value. This could mean that the stress release is not total when the first crack is created, and that subsequent cracks release some residual stresses. These cracks, as they are appearing at lower total water content, should also release a subsequent rebuild-up of stresses due to subsequent partially constrained desiccation.

4. Ongoing and future works

Together with current investigations on tensile stress and strength behavior quantitatively with respect to hydration state, this desiccation experimental characterization allows for the validation of a crack initiation model. The comprehensiveness of the study will be achieved by adding a crack propagation model.
5. References


SMALL STRAIN PLASTICITY IN LACUSTRINE CLAY

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ABSTRACT. The anisotropy, non-linearity and small strain plasticity of lightly overconsolidated Swiss lacustrine clay was investigated with drained triaxial stress path tests. The stiffness response, as well as the elastic anisotropy inside the bounding surface, was analysed.

1. Introduction

Distinct horizontal stratification of clay and silt layers, due to the annual fluctuation in the deposition mode in lakes, is typical for post-glacial lacustrine deposits. This inherent anisotropy of the subsoil influences its failure behaviour significantly (e.g.: Plötze et al., 2005). The influence of the layered soil composition on the mechanical behaviour before failure is investigated in this research project.

2. Laboratory investigations

The investigations performed included drained triaxial stress path tests on specimens from block samples of natural Swiss lacustrine clay. Local axial LVDT’s mounted directly on the sample were used for accurate axial displacement measurement. For the local radial displacement measurements, a laser scan device was developed (Messerklinger et al., 2004; Messerklinger, 2006). The samples were one-dimensionally reconsolidated up to a mean effective stress of 300 kPa, then one-dimensional swelling was applied to an overconsolidation ratio of 2 and finally each sample was loaded along a different stress ratio under drained conditions. These stress paths are shown in Fig. 1 (a). Consequently the stress-strain response for loading from an overconsolidation stress state towards failure was investigated within the triaxial stress space.

3. Analysis

The data analysis was performed, as described in (Smith et al., 1992), based on the stiffness zone framework proposed by Jardine (1992). Yield points on the history surface (Stallebrass, 1990) and at the bounding surface were detected for the Swiss lacustrine clay investigated. The results are presented in the invariant stress space $q = \sigma_3 - \sigma_3'$ and $p' = (\sigma_1' + 2\sigma_3')/3$, Fig. 1 (b).

The points evaluated for the Y2 history surface indicate a horizontally orientated ellipse, while the points describing the Y3 bounding surface suggest an inclined ellipse in the $q – p'$ space. With the application of the anisotropic elastic stiffness matrix for the triaxial stress space, proposed by Houlsby & Graham (1983) in which the 5 anisotropic elastic parameters are reduced to three by introducing the anisotropy factor $\alpha$, the ratio of the axial to the radial stiffness was determined as 2.7. With the use of bender elements, the anisotropic linear elastic stiffness matrix, which describes the stress-strain response inside the stiffness zone 1 (Jardine, 1992), was established as:

\[
\begin{bmatrix}
\delta \sigma_a \\
\delta \sigma_r \\
\end{bmatrix} = 
\begin{bmatrix}
1 & -2\nu_{ar} & 0 \\
\nu_{ar} & 1 & 0 \\
\end{bmatrix}
\begin{bmatrix}
\delta \sigma_a \\
\delta \sigma_r \\
\end{bmatrix}
\]

with

\[
\begin{align*}
E_a &= 497 \text{ MPa} \\
\nu_{ar} &= 0.52 \\
\nu_{rr} &= 0.31 \\
E_r &= 179 \text{ MPa}
\end{align*}
\]
The plastic strain vector at the yield points was determined from the strain measurement by subtracting the elastic strain component, Eq. (1) and is given in Fig. 2 (a) for the Y3 yield points and in Fig. 2 (b) for the Y2 yield points. Additionally, a line perpendicular to the plastic strain vector at the yield points is given (thin dotted line), which represents the tangent to the plastic potential at this point. Although the fluctuation in the data is significant, the results indicate that an associated flow rule to the previously suggested yield surfaces may simulate the results well.

4. Acknowledgement

The author is grateful to her supervisors Profs. Sarah M. Springman and Helmut Schweiger for their guidance throughout this study. The author would also like express her thanks to the workshop group A. Zweidler, E. Bleiker, H. Buschor, F. Ehrbar and the laboratory team T. Ramholt, D. Bystricky, M. Sperl, R. Rohr for their assistance with the laboratory tests. The author acknowledges gratefully the support provided by grant No. 200021-100 362/1 from the Swiss National Science Foundation.
5. References

Constitutive modelling
A CHEMO-MECHANICAL MODEL FOR BONDED GEOMATERIALS

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ABSTRACT. A simplified chemo-mechanical model for bonded geomaterials is presented. In order to calibrate it, some experimental tests on silica cemented sand are carried out. The aim of the experimental setup is the measurement of the weathering effects of a contaminant compound migrating within a bonded soil. The chemo-mechanical coupling is performed combining the contaminant transport equations with a F.E. code in which an apt model for bonded soils (Nova et al., 2003; Castellanza et al., 2002) is implemented.

1. Introduction

Rock weathering has a great relevance in the study of several engineering problems such as slope stability, evaluation of the settlements of foundations (or subsidence phenomena) and estimation of the safety factor for underground geostructures (i.e. mines).

A new numerical tool is performed linking a F.E. code, GeHoMadrid (Fernandez Merodo et al., 1999), and a Finite Difference code, RT3D (Clement, 1998); the former is able to reproduce the debonding process of a soft rock by means of a strain-hardening constitutive model (Nova et al., 2003); the latter is able to solve the transport equations in a saturated porous media for different chemical species including advection, diffusion and chemical reaction processes.

2. Problem position

The link between GeHoMadrid and Rt3D is performed by introducing a single parameter (X_d(t)) which represents, at time t, the amount of remaining bonds, calculated by RT3D, with respect to the initial one. With this information the mechanical model can update the hardening variables and, in turn, it can calculate the new stress-strain state and the modified permeability of the porous medium.

\[
\text{CONC}_1, \text{CONC}_2, X_d(t) \quad \text{GeHoMadrid} \quad \text{RT3D} \quad \text{Update of mechanical and hydraulical parameter}
\]

\[
\begin{align*}
\text{CONC}_1, \text{CONC}_2 & \quad \text{GeHoMadrid} \\
X_d(t) & = f(X_d(0))
\end{align*}
\]

\[
\begin{align*}
\theta \frac{\partial C}{\partial t} &= \nabla \cdot \left( \frac{D}{\theta} \nabla C \right) - \frac{\partial}{\partial x_i} \left( \frac{\partial C}{\partial x_i} \right) + q_i C \sum_i R_i
\end{align*}
\]

\[
X_d(t) = 1 - \frac{[\text{CaCO}_3(0)]}{[\text{CaCO}_3(t)]}
\]

\[\text{Figure 1. Schematic description of the link and governing equations for the transport model.}\]

3. Experimental setup

The transport model was calibrated with some laboratory tests on specimen of cemented silica sand in 1D seepage conditions. The scope of the tests is the determination of the rate constants which rule the chemical interaction among the different transported compounds. Moreover, for the validation of this model, a particular oedometer apparatus was employed (Castellanza, 2002).
4. Applications

The presented model could be applied in order to obtain a qualitative prediction of the settlements of a shallow foundation due to the debonding of the cemented soil beneath. In particular, different types of boundary conditions can be considered in order to perform 2-Dimensional simulations:

A series of experimental tests are presently performed in Milan on a model foundation resting on an artificially cemented soil, to have a benchmark against which checking the validity of the proposed theory.
5. References


MODELLING SMALL-STRAIN STIFFNESS IN A DOUBLE HARDENING MULTILAMINATE MODEL FOR SOILS

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ABSTRACT. The presented constitutive model accounts for higher stiffness of soils at small-strain level using a central feature of the multilaminate framework. Based on a micromechanical approach for the determination of the contact stiffness of soil grains, the global elastic stiffness matrix is formulated from the response of each independent acting integration plane. The degradation of stiffness is governed by two additional parameters and the material's strain history.

1. Introduction

Due to advances of experimental techniques, it is possible to measure the stiffness of soils at small-strain level with reliable accuracy (e.g. Jardine et al., 1984; Soga et al., 1995). Another major conclusion is the strong dependence of the soil stiffness on the loading history. Hence, a change of the loading direction causes an abrupt stiffness increase.

Advanced constitutive models can be extended to consider the behaviour of soils at small-strain levels resulting in improved prediction capabilities for practical boundary value problems.

2. Multilaminate Model for Clay

The Multilaminate Model for Clay (MMC) was developed and implemented into the commercial finite element code PLAXIS (Brinkgreve, 2002) by Wiltafsky (2003). It is based on the multilaminate framework, introduced for clays by Pande & Sharma (1983) and incorporates shear and volumetric hardening on independently acting integration planes. As the employed integration rule is a fully symmetric 66 point formula according to Bazant & Oh (1986), only 33 different planes are analysed while the weight factors are doubled. It is one of the most important features of this model that the development of the yield surface on each sampling plane is based on the evolution of plastic strains on the respective plane. Only under hydrostatic loading the same preconsolidation pressures are obtained on each integration plane. When any deviatoric stress is applied, the state variables related to the state of the yield surface vary over the planes. An initially isotropic soil becomes anisotropic after loading when analysed within the multilaminate framework, thus it captures plastic flow induced anisotropy intrinsically.

Within the model published in 2003 only plastic strains are calculated from the various integration planes. The stress dependent elastic stiffness matrix and the resulting elastic strains are determined globally. With the following formulation, based on a micromechanics model, the MMC is extended to take small-strain stiffness into account.

3. Computation of the elastic stiffness matrix

By considering the interactions among the discrete soil particles in the assembly, the stress-strain behaviour at small strains can be expressed at each contact of soil grains. With the assumption of perfectly rounded, smooth-surface spheres, the normal contact stiffness $K_{nn}$ (Mindlin & Deresiewicz, 1953) and the tangential contact stiffness $K_{tt}$ (Johnson, 1985) are derived as follows:

$$K_{nn} = \left[\frac{3 \cdot r_{contact} \cdot G_{grain}^2 \cdot \sigma_{n,0}}{(1 - \nu_{grain})^2 \cdot N_c}\right]^{1/3}$$

$$K_{tt} = \frac{2 \cdot (1 - \nu_{grain}) \cdot K_{nn}}{2 - \nu_{grain}} \cdot K_{nn} \left[1 - \frac{\tau_0}{\sigma_{n,0} \cdot \tan(\varphi_{grain})}\right]^{1/3}$$

(1)
G\text{\textsubscript{grain}} and ν\text{\textsubscript{grain}} are the shear modulus and Poisson’s ratio of the solid particles, θ\text{\textsubscript{grain}} is the inter-particle friction angle, σ\text{\textsubscript{n,0}} and τ\text{\textsubscript{0}} denote the stress state on the contact plane and N\text{\textsubscript{c}} represents the number of contacts per unit area. Employing the number of planes N\text{\textsubscript{p}} per unit length perpendicular to the contact plane, the local compliance matrix C\text{\textsubscript{cp}} for each integration plane is calculated. Weighted summation over all planes according to Equation 2 allows the transformation to global level (C\textsubscript{gl}).

\[ C_{gl} = \sum_{n=1}^{cp} T_j \cdot C_{cp} \cdot T_i^T \cdot W_n \]  

Equation 2

T\textsubscript{j} and T\textsubscript{i} characterise the 6x3 transformation matrices according to Gerrard & Pande (1985) and W\textsubscript{n} are the weight factors related to the direction of the integration plane.

4. Stiffness degradation at small-strain level

Numerous stiffness degradation curves from laboratory tests, describing the material behaviour during shearing at small-strain level can be found in the literature. Plotting the shear modulus over the shear strains in log scale results in a S-shaped curve, which can be fit by various functions to match the test results.

Within the Multilaminate Model for Clay, the reduction of stiffness is governed by the change of contact conditions between the soil grains. Particularly, the contact radius r\text{\textsubscript{contact}} varies with the rearrangement of grains during shearing according to Equation 3. The two material parameters a and b determine the reduction of the contact radius with the accumulated shear strain γ\text{\textsubscript{cp}} on each contact plane and influence the global stiffness degradation curve plotted in Figure 1.

\[ r_{\text{contact}} = \frac{r_{\text{contact,max}}}{1 + a \cdot \gamma_{cp}^b} \]  

Equation 3

5. Conclusions

With the here proposed extension of the Multilaminate Model for Clay, the increased material stiffness at small strains can be taken into account in a straightforward way. Forming the stiffness matrix for each integration plane allows the use of a simple formulation for the contact stiffness of the grains while the global stiffness matrix is calculated by summarising the results from every single plane.
6. References

CONSTITUTIVE MODELS EXTENSION TO UNSATURATED STATES: A GENERAL FRAMEWORK AND ITS APPLICATION TO CJS MODEL.

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ABSTRACT. The present work aims to provide a general framework allowing extending in a straightforward way existing elastoplastic models to unsaturated states. The generality of the proposed framework and the easiness of the methodology introduced are illustrated by extending the CJS model, a multi-mechanism elastoplastic model for granular materials. The validation of the extended model shows its ability to reproduce complex volumetric responses of unsaturated soils.

1. Introduction

A precise description of unsaturated soils behaviour concerns many civil engineering applications like slope stability, earth structures, nuclear waste disposal, etc. The particularly rich activity observed in this research field in the recent years has led to the formulation of several constitutive models (e.g. Alonso et al., 1990; Kohgo et al., 1993; Abou-Bekr, 1995; Wheeler & Sivakumar, 1995; Loret & Khalili, 2000; Laloui et al., 2001).

The choice of suitable stress variables for modelling purposes constitutes a key point. First attempts of unsaturated soils modelling have naturally extended Terzaghi’s effective stress. The most famous proposition is due to Bishop (Bishop & Blight, 1963). It however appears that such model definitions, strictly applying the effective stress concept as a single state variable, are unable to predict some typical features of unsaturated soils behaviour.

Later attempts have then used two distinct stress variables. It has been shown (Fredlund & Rahardjo, 1993) that any couple among the triplet $\{s, \sigma - p\}$ could be selected $(\sigma$ and $\mathbf{1}_F$ stand for the total stress and unit tensors, $p_s$ and $p_l$ are the pressures of the gas and liquid phases).

Some recent approaches have tried to define a particular effective stress used in parallel with suction as two state variables. The purpose of such models is to simplify the transition between saturated and unsaturated states and to reuse existing behaviour models and computational codes.

2. General framework formulation

In front of the diversity of approaches, a general framework is proposed. It aims to provide a simple methodology allowing the extension of existing elastoplastic models to unsaturated states. This framework lies in a two stress state variables approach, one of them being a complex combination of total stress and suction (often called effective stress) used in parallel with suction. It appears to be able to encompass several recent unsaturated models (Pereira et al., 2005).

To keep all its generality, our proposition does not directly define an effective stress. Indeed, once an equivalent pore pressure has been introduced, the influence of the effective stress is usually treated similarly. This pressure will therefore be given later, during the model extension stage.
3. Extension of CJS model and validation

Most of recent propositions extend a simple constitutive model (often a Modified Cam-Clay like model). An objective of the present study was to determine the benefits that appear from the extension of a more complex model. This work has put the focus on the CJS model (Cambou & Jafari, 1988), an elastoplastic model defined for unsaturated granular materials (sands or silts). It counts three mechanisms, one non-linear elastic and two plastic with both isotropic and kinematic strain-hardening. It includes a characteristic state surface allowing accounting for complex volumetric responses (contractancy-dilatancy can be predicted depending upon stress state).

The extension of this model is processed using the proposed methodology. The equivalent pore pressure \( \pi \) (Eq. 1) derives from the work of Coussy & Dangla (2002). It takes into account the energy of interfaces separating the different phases. This formulation requires the knowledge of the sorption curve. As a first approximation, the Brooks and Corey model is used. The effective stress is then completely defined by \( \sigma' = \sigma + \pi' \). Apart from effective stress definition, the influence of suction is also accounted for in the yield surfaces definition, flow rules and hardening laws.

\[
\pi(s) = \begin{cases} 
-P_e + s & \text{for } s \leq s_c \\
-P_e + sS_f + \frac{2}{3} \int_{s_f}^{s_c} s(S) dS & \text{for } s > s_c 
\end{cases}
\]

(1)

Figure 1 presents model predictions during triaxial tests at constant suction under a confining pressure of 100 kPa. Experimental data used to fit the model concerns an unsaturated compacted silt from Jossigny, France. A good agreement between numerical and experimental data can be observed.

4. Conclusions

A general framework allowing the extension of existing elastoplastic models to unsaturated states has been introduced. The adaptation of a complex model (CJS) demonstrates the generality of the proposed framework and the easiness of the methodology that it provides. The validation of the extended model shows the benefits furnished by the use of such refined models (particularly in terms of volumetric description).
5. References


ABSTRACT. Phenomenological rate dependent behaviour of soft and sensitive clay is studied with the help of plastic Visco-plastic constitutive model implemented in finite element code BIFURC.

1. Introduction

The effect of time on the strength and deformation characteristics is well recognized for variety of geo-mechanics application. Several efforts have already been made, in order to incorporate elasto-Visco-plastic model and capture localization in normally consolidated and in over consolidated clays (Oka et al, 1994; Plaxis soft soil creep model, 1999; Loret et al, 1990). Most of these constitutive models are based on over stress type theory proposed by Perzyna (1963).

In present study, a mobilized friction theory proposed by Nordal (1999) and Mitchell’s time dependent creep theory is utilized, which is independent of over stress type theory. Shear induced pore pressure, \( u \), develops due to inelastic strains while reversible strain i.e. elastic strain is neglected, in this study. Time dependent change of mobilization is observed due to the contribution from rate dependent visco plastic strain, \( \gamma_{vp} \). Also, to avoid controversial “reference time”, an update scheme is chosen where reference time is computed from previous equilibrium state at “reference curve”. Reference curve and reference time depicts the state when Visco plastic strain actually starts developing.

2. Model Description and Implementation in BIFURC (Finite Element Code)

Followings are the one dimensional constitutive relations used to find time dependent stress strain relationship for a column subjected to pure shear.

\[
d\gamma = d\gamma^p + d\gamma^{vp} \\
F = \frac{\tau}{\sigma'_{\gamma}} - \tan \rho = 0 \\
f = \frac{\tan \rho}{\tan \varphi_p} \\
\frac{d\gamma^{vp}}{dt} = \frac{\left(\alpha_{\varphi_{\text{peak}}} - \alpha \varphi_{\text{peak}} \right) \left(\frac{t_{\text{ref}}}{t + dt}\right)^m}{\bar{A}} \\
f' = \frac{2 \times \sqrt{\frac{\gamma^p_{\gamma_{\text{ref}}}}{\gamma^p_{\gamma_{\text{ref}}}}} \gamma^p_{\gamma_{\text{ref}}}}{1 + \frac{\gamma^p_{\gamma_{\text{ref}}}}{\gamma^p_{\gamma_{\text{ref}}}}}, \\
du = \frac{2U_{50}d\gamma}{\left(1 + \frac{U_{50}}{U_{DA}}d\gamma\right)}
\]

where \( \alpha = \alpha_{\varphi_{\text{peak}}} \), \( \bar{A} = \frac{1}{A} \) are laboratory parameters, \( t_{\text{ref}} = \) reference time; \( t = \) total time; \( dt = \) time increment, \( U_{50} \), pore pressure ratio corresponding to 50% of peak shear strain and \( U_{DA} \) is maximum pore pressure in any hyperbolic type function. In present case \( m \) is equal to unity.

Equations (1-6) have been implemented in BIFURC, finite element code indigenously developed at Norwegian Geotechnical Institute, and analysed for one dimensional case. Shear bar is tested under varying strain rate from 0.001%/min to 1%/min. Obtained results are reported in Figure 1. Input data are shown in Table 1.
\[ \tan \phi_p = 0.5 \quad A = 0.006 \% / \text{min} \quad \sigma_v' = 100 \text{ kPa} \quad \text{UDA} = 70 \text{ kPa} \quad \alpha = 4.2 \quad \gamma_p \text{ ref} = 5\% \quad U50 = 20 \text{ kPa/\%} \]

**Table 1. Input Parameters for BIFURC plastic Visco plastic model**

![Figure 1. Results from plastic visco plastic model](image)

Readers are suggested to follow (Mitchell, 1992) and (Nordal, 1999) for more detailed parametric study of the model.

### 3. Discussions and conclusions

Rate dependent peak shear strength and shear induced pore pressure is reported in Figure 1. Fast test does give enough time to develop visco plastic strain and shear induced pore pressure, which means higher effective stress at the peak and so higher shear strength. To overcome from the limitation associated with Mitchell’s creep theory, is valid only for a constant stress level, mobilized friction model coupled in constitutive formulation. Change in mobilization due to creep strain is also responsible for strain softening as it can be noted from the Figure that for slow tests visco plastic strain is developing significantly i.e. 30 to 40\% of total strain. Present model is also able to capture secondary creep; however creep rupture can not be obtain due to unit value of \( m \) parameter. This model is further being used to capture strain localization in clays.
4. References

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Miscellaneous
A DGT-METHOD FOR THE NUMERICAL ANALYSIS OF WAVE PROPAGATION IN FULLY-SATURATED POROUS MEDIA

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ABSTRACT. In the present contribution we investigate a multifield space-time finite element approach, i.e. a coupled Time-Discontinuous Galerkin method (DGT) for the study of the dynamical response in porous media. The physical model is based on a four field formulation within the framework of the Theory of Porous Media (TPM). Numerical experiments demonstrate the improved behaviour of the proposed numerical solution scheme.

1. Introduction

The numerical analysis of wave propagation within fluid-saturated porous materials like soils or granulates is of relevant importance in geology, seismologic engineering, petroleum engineering and geotechnical engineering. Besides the well investigated theory of Biot (1956), the mechanical behaviour of multi-phase media can also be described by the more rational and thermodynamically consistent Theory of Porous Media (TPM), (c.f. Bowen, 1980; de Boer & Ehlers, 1986). As we want to restrict ourselves to the numerical treatment of the transient behaviour of multiphase materials, we keep the underlying model as simple as possible. Thus, we investigate a two-phase material incompressible model proposed by Diebels & Ehlers (1996).

The Discontinuous Galerkin (DG) method in space as well as in time is of increasing interesting in our days. Nevertheless, the study of Time-Discontinuous Galerkin methods started already in late 60s. However, in the last 10 years, a remarkable influence to the spatial DG method was made by Oden & Baumann (Baumann, 1997), in which a new method without artificial penalty parameter was proposed. Contemporarily, various DGT methods have been proposed for different applications. However, almost all of these numerical schemes can not abandon inherent penalty parameters. Due to the difficulty in determining these highly boundary-value-dependent parameters, the application of such methods are restrict to very specific cases.

In this work, we present a new DGT method which do not contain any penalty terms. The governing finite element discretization covers the space and time domain simultaneously. Instead of introducing penalty terms, the continuity condition in the temporal domain is weakly enforced by an upwind flux term, which is a technology already successfully applied in spatial DG methods.

2. DGT approach of a two-phase model based on the Theory of Porous Media

As already mentioned, the physical model is based on the TPM. Furthermore, we restrict ourselves to the geometrical and material linear case. Let \( \Omega \subset \mathbb{R}^d \), \( (d=1,2,3) \) denotes the spatial domain, and let \( I = [t_0,T] \) be the temporal domain. In addition, we introduce the constituents \( \varphi^\alpha (\alpha \in \{S,F\}, S:\text{ solid, } F:\text{ fluid}) \) with the constant effective densities \( \rho^{\alpha r} = \text{const} \). The resulting four field formulation (Diebels & Ehlers, 1996) leads to the following set of primary variables: The solid displacement \( u^S \), the solid velocity \( v^S \), the seepage velocity \( w_F = v_F - v^S \), and the pore pressure \( p \). Inserting appropriate constitutive assumptions, we obtain the coupled set of field equations:

\[
\begin{align*}
\mathbf{u}^S & = \mathbf{v}^S & \text{on } \Omega \times I \quad (1) \\
(n^S \rho^{SR} + n^F \rho^{FR}) \mathbf{v}^S &= \nabla \cdot (\mathbf{T}^S - p \mathbf{I}) - (n^S \rho^{SR} + n^F \rho^{FR}) \mathbf{b} & \text{on } \Omega \times I \quad (2) \\
\rho^{FR} (v_{\mathbf{F}}^S + w_F) &= \frac{n^F}{K_F} w_F + \nabla p = \rho^{FR} \mathbf{b} & \text{on } \Omega \times I \quad (3) \\
\nabla \cdot (u^S + n^F w_F) &= 0. & \text{on } \Omega \times I \quad (4)
\end{align*}
\]
The partial derivative of the motion function is introduced as \((\cdot)^\prime\). Convective terms are neglected according to the assumption of a geometrically linear theory. Eq. (2) represents the balance of momentum of the mixture, in which \(T_E^S\) is the Cauchy extra stress tensor of the solid constituent. The balance of momentum of the fluid is described in Eq. (3). Notice, that we neglect the viscous fluid extra stresses \((T_E^F \approx 0)\). The influence of internal friction between the solid and the fluid constituent is modeled by the momentum interaction force in the form \(\mathbf{F}^F \propto \mathbf{w}_F\). This results in a transient Darcy-type equation Eq. (3). Eq. (4) devotes to the balance of volume of the mixture, i.e. the mixture’s continuity equation. Eq. (1) was introduced to reduce the order of temporal derivative to one.

Due to the concept of superimposed continua, the boundary conditions are described as follows

\[
\begin{align*}
\mathbf{u}_S &= \mathbf{u}_S & \text{on} & & \Gamma_D^S \times I, \\
(T_E^S - p\mathbf{I}) \cdot \mathbf{n} &= \mathbf{t} & \text{on} & & \Gamma_N^S \times I, \\
\mathbf{w}_F &= \mathbf{w}_F & \text{on} & & \Gamma_w^F \times I, \\
p &= \bar{p} & \text{on} & & \Gamma_p^F \times I,
\end{align*}
\]

where \(\partial \Omega = \Gamma_D^S \cup \Gamma_N^S = \Gamma_w^F \cup \Gamma_p^F\) with \(\Gamma_D^S \cap \Gamma_N^S = \emptyset\) and \(\Gamma_w^F \cap \Gamma_p^F = \emptyset\). The initial conditions are prescribed for \(\mathbf{u}_S(x,t_0) = \mathbf{u}_{S,0}(x)\), \(\mathbf{v}_S(x,t_0) = \mathbf{v}_{S,0}(x)\) and \(\mathbf{w}_F(x,t_0) = \mathbf{w}_{F,0}(x)\). We denote that since in Eq. (1) \(\sim (4)\) there exists no time derivative of the pore pressure \(p\), its value is determined in a consistent way within the system of differential algebraic equations.

We construct the space-time domain \(Q = \Omega \times I\) by adding the time axis orthogonal to the spatial domain. Hence, the applied finite element must contain an extra dimension with respect to the time domain, which means that we employ quadrilaterals or hexahedral for a one-dimensional or two-dimensional spatial problem, respectively. Let us denote the time interval \(I^n = [t_n, t_{n+1}]\) \((t_0 < t_n < t_{n+1} < T)\) and the time-slab \(Q^n = \Omega \times I^n\). Hereby, a similar technology usually applied in spatial DG methods dealing with convective terms (first-order derivatives in space) is employed in the temporal domain. Suppose \(\Psi\) is a field quantity in the space-time domain \(Q\), we define the upwind flux \(\bar{\Psi}\) on the time-slab border as

\[
\bar{\Psi}(x,t_n) = \begin{cases} \\
\Psi^-(x,t_n), & \text{if } n > 1 \\
\Psi_0, & \text{otherwise}
\end{cases}
\]

where \(\Psi^-(x,t_n) = \lim_{\varepsilon \to 0^+} \Psi(x,t_n - \varepsilon)\) (5).

Notice, that \(\Psi_0\) denotes the prescribed initial condition. Let \(\delta \Psi\) be a proper test function in \(Q^n\). Applying integration by parts in time, we obtain the weak form on the time-slab as

\[
\begin{align*}
\int_{\Omega^n} \Psi' \delta \Psi \, dQ &= -\int_{\Omega^n} \Psi \delta \Psi' \, dQ + \int_{\Gamma_n} \left[ \Psi \delta \Psi \right]_{t_n}^{t_{n+1}} \, d\Gamma + \int_{\Gamma_n} \left[ \delta \Psi \right] \, d\Gamma = -\int_{\Omega^n} \Psi \delta \Psi' \, dQ + \int_{\Gamma_n} \left[ \delta \Psi \right] \, d\Gamma
\end{align*}
\]

We remark here, that for the calculation on the current time-slab \(Q^n\) \(n > 1\), \(\Psi_{n+1}^-\) represents the concerned quantity at the time level \(t_{n+1}\), while \(\bar{\Psi}_n\) takes the value computed from the previous time-slab \(Q^{n-1}\) at \(t_n\). In the case \(n = 1\), no previous time-slab is available, according to the flux definition in definition (5), \(\bar{\Psi}_n\) equals the initial value. Here it can be seen, that the computed results from the previous time-step are naturally included in the current time-slab by applying integration by parts and by introducing an upwind flux term at the border of the time-slab. So far, the analogy of the DGT method to the standard Methods Of Line (MOL) is obvious as the concerned quantity is computed subsequently on every time-slab. This also restricts the total DOFs in the resulting algebraic equations to those on a certain time-slab. The governing weak form on the time-slab \(Q^n\) is given exemplarily for Eq. (1)

\[
\int_{\Omega^n} (\mathbf{u}_S \delta \mathbf{u}_S - \mathbf{v}_S \delta \mathbf{u}_S) \, dQ = \int_{\Omega^n} \mathbf{u}_S \delta \mathbf{u}_S - \bar{\mathbf{u}}_S \delta \mathbf{u}_S \, d\Omega.
\]

For more details we would like to refer to (Chen et al., 2005).
3. Examples & Conclusions

In this example, the wave propagation in the given material incompressible two-phase model is investigated numerically. Biot (Biot, 1956) has demonstrated that within a compressible two-phase mixture, two compressible waves and one shear wave exist. However, considering the incompressible model investigated in our work, the second compressible wave (Biot’s slow wave) caused by the dilatancy effect of the compressible constituents is not covered. A specimen of the size 20mX10m as depicted in Figure 1(left) with impervious side walls and undrained conditions at the bottom is investigated. Drained conditions at the top of the specimen are assumed. An impulse load as

\[
F(t) = \begin{cases} 
100 \sin(78.54 \cdot t) \text{ [kN]} & \text{if } t \leq 0.04 \text{s}, \\
0 & \text{if } t > 0.04 \text{s}, 
\end{cases}
\]

is applied on the impervious middle part of the upper surface. This example was first proposed by Beuer (1999). The material parameters therein are cited in Table 1.

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>(\mu^I)</td>
<td>5583 \text{ kN/m}^2</td>
</tr>
<tr>
<td>(\lambda^I)</td>
<td>8375 \text{ kN/m}^2</td>
</tr>
<tr>
<td>(n_{S0})</td>
<td>0.67</td>
</tr>
<tr>
<td>(\rho^{SR})</td>
<td>2000 \text{ kg/m}^3</td>
</tr>
<tr>
<td>(\rho^{FR})</td>
<td>1000 \text{ kN/m}^3</td>
</tr>
<tr>
<td>(k^F)</td>
<td>0.01 \text{ m/s}</td>
</tr>
</tbody>
</table>

Table 1. Material parameters

In this investigation, the propagation of waves in the mixture is of interest; therefore we recorded the particle motion at point A, c.f. Figure 1. The problem is calculated by 21 X 10 elements with an extended Taylor-Hood-type formulation in space combined with linear Lagrangean polynomials in time. A numerical comparison is given between the DGT method and the implicit Euler (EU) scheme. In Figure 1 (right) we present the numerical results obtained by both methods with different time-step lengths. It is observed that qualitative differences occur within the results obtained by both approaches. A relatively coarse temporal discretization, \(\Delta t = 0.01\text{s}\), leads to jumps (numerical errors) at every time interval within the numerical results of the DGT method. However, from the numerical simulation point of view, such unphysical jumps render more accurate results. Furthermore, such jumps are well cured by a refined discretization, which also set the stage for a jump-based error indicator (Jump-Indicator) for the DGT method. On the other hand, it is observed clearly, that the implicit Euler scheme leads to unphysical damping due to its inherent numerical dissipation. A reliable result could only be obtained with an extremely refined temporal discretization. In spite of the lower quality of the results obtained with the implicit Euler scheme, the resulting quantities are always smooth and therefore well disguised as good ones. Adaptive methods designed for such cases are usually much more expensive than the simple Jump-Indicator of the DGT method.

![Figure 1. Boundary value problem, geometry, discretization and particle motion of point A at the surface.](image)
4. References


PROBABILISTIC METHODS APPLIED TO GEOTECHNICAL ENGINEERING

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ABSTRACT. Geotechnical problems are often dominated by uncertainty, such as inherent spatial variability of soil properties or scarcity of representative data. Engineers try to solve these problems by deterministic calculation, but there maybe so many uncertainties that even advanced deterministic methods become useless. Therefore probabilistic methods are applied. This paper describes the probabilistic Point Estimate Method and shows its application in the field of geotechnical engineering. The bearing capacity study of a shallow foundation is considered.

1. Introduction

Geotechnical variability results from different sources of uncertainties. The three primary sources are inherent variability, measurement error and model uncertainty. Inherent variability results primarily from natural geologic processes that created in situ soil layers. Measurement error is caused by sampling and lab testing. This error is increased by statistical uncertainty that arises from limited amount of information. Finally the model uncertainty is introduced when field or laboratory measurements are transformed into input parameters for design models involving simplifications and idealisations.

For the already mentioned geotechnical problem homogeneous soil layers are considered, whose shear strength parameters are described by given probability distribution functions. The Gaussian (normal) function is the simplest and best known distribution function (see figure 1a). However in geotechnical engineering the distribution of parameters around the mean value would not always seem to be symmetrical as suggested by the Gaussian distribution. Especially the soil cohesion seems to be better approximated by the asymmetrical lognormal function, as indicated in figure 1b.

The normal and lognormal distributions are characterised by the mean value \( \mu_x \), standard deviation \( \sigma_x \) and skewness coefficient \( \nu_x \). Considering a certain random variable \( x \), as the soil cohesion or friction angle, and its corresponding distribution function \( f(x) \), then it yields:

\[
\mu_x = \int_{-\infty}^{+\infty} x \cdot f(x) \, dx \quad \sigma_x = \int_{-\infty}^{+\infty} (x - \mu_x)^2 \cdot f(x) \, dx \quad \nu_x = \int_{-\infty}^{+\infty} \frac{(x - \mu_x)^3}{\sigma_x^3} \cdot f(x) \, dx
\]

Instead of specifying the standard deviation, one often gives the coefficient of variation, which is the ratio of the standard deviation over the mean value, i.e. \( \text{COV} = \frac{\sigma_x}{\mu_x} \). Phoon and Kulhawy (1999) presented data for a sand and a clay layer to find values between 5-15% for the effective friction angle. This range was also put forward by Harr (1987). Occasionally higher COV-values can be found, as in figure 1a, where its value is about 30%. Considering the effective cohesion, Harr suggests a value of about 20%, Li and Lumb (1987) report a particular clay layer with 40% and Moormann and Katzenbach (2000) find 50% for Frankfurt clay as indicated in figure 1b. It can thus be concluded that the available data on the effective cohesion show much more variation then the data of the effective friction angle.

The skewness coefficient describes the degree of asymmetry of a distribution function. When the skewness coefficient is equal to zero then a function is symmetric, as in the case of the Gaussian approximation of the friction angle of Frankfurt clay, as shown in figure 1a. Figure 1b shows the case of a positively skewed lognormal distribution which is steep for low values of the cohesion and flat for large values. A negatively skewed distribution will be flat for low values of the soil parameter and steep for large values. However negative skewnesses would seem to be unrealistic for the distribution of soil parameters. Considering data as reported by Moormann & Katzenbach and El Ramly et al. (2005), it would seem that the skewness coefficient of the effective friction angle could be disregarded and the choice of a Gaussian distribution would
seem suitable. On the other hand, the above authors find skewness of \( \nu_c \approx 0.2 \) for effective cohesion, thus a lognormal distribution would seem to be appropriate. Anyway very little data about the skewness coefficient are available in literature and the value of \( \nu_c \approx 0.2 \) would need further testing.

![Figure 1. Probability density functions of strength parameters of Frankfurt Clay (Moormann & Katzenbach, 2000)](image)

Some authors have based their probabilistic studies on the assumption that soil cohesion and friction angle are uncorrelated, but this is often done to simplify calculations. Lumb (1969) was probably the first to study the correlation between soil cohesion and friction angle on the basis of experimental data. He found a negative correlation implying that low values of cohesion are associated with high values of the tangent of friction angle, and vice versa. However in some cases the correlation was found to be insignificant. He concluded that the assumption of independence of the strength parameters simplifies strength interpretation considerably, and also leads to conservative results if the correlation is in fact negative. In contrast the results of Cherubini (1998) indicate a significant negative correlation between effective cohesion and friction angle. Similarly Schad (1985) reported a strong negative correlation. In order to incorporate a dependence between the strength parameters one needs input data on the so-called correlation coefficient, denoted as \( \rho \). Both Cherubini and Schad find \( \rho = -0.6 \), whereas Lump reported values in the range \(-0.3 < \rho < -0.7\). Hence it would seem that a value of about -0.6 is realistic. In this presentation the strength parameters are assumed to be negatively correlated. It will be shown that negative correlation coefficient decreases the standard deviation of computational results, such as bearing capacity of a footing or safety factor of a slope. It can be concluded that it seems to be interesting to incorporate a negative correlation for strength parameters in the analysis.

2. Bearing capacity of a shallow foundation

The bearing capacity problem refers to a shallow foundation with a width of 2m on top of a homogeneous soil layer. The unit soil weight is 15 kN/m\(^3\) and an initial surcharge 10 kPa is considered. The effective friction angle and cohesion are assumed respectively as normally and lognormally distributed variables with their corresponding mean value, standard deviation and skewness coefficient, i.e.: \( \mu_\phi = 4 \text{ kPa}, \sigma_\phi = 3.3 \text{ kPa}, \nu_\phi = 3.1 \text{ and } \mu_c = 25^\circ, \sigma_c = 2.6^\circ, \nu_c = 0 \). The bearing capacity, denoted as \( q_{f1} \), from Brinch-Hansen’s formula gives a deterministic value of 287.6 kPa.

Initially independent input parameters are considered and Monte Carlo simulations are applied. In Monte Carlo simulations the accuracy of the computed function depends on the number of realizations as carried out for each input parameters. 1000 realizations were carried out in order to obtain a nearly exact solution. Table 1 shows the results of the Monte Carlo method with a mean bearing capacity of 304.72 kPa, which differs only 6% from the deterministic value. For practice Monte Carlo simulations are computationally too time consuming, therefore alternative methods have to be considered, such as the Point Estimate Method (Rosenblueth, 1981).

With the Point Estimate Method the distribution function of the shear strength parameters are discretised by choosing some sampling points and the mean value, standard deviation and
The skewness of the bearing capacity can be computed. The determination of these values is realized through the weights sum of every discrete realizations (sampling points). The results of the Point Estimate Method considering uncorrelated strength parameters are listed in Table 1. When these results are compared to those of Monte Carlo simulations, it can be observed that the mean values and standard deviation are very similar; on the other hand there is a significant difference between the skewness coefficients. The Point Estimate Method doesn’t give the entirely probability density function of the bearing capacity as the Monte Carlo method does. Thus assuming a lognormal distribution, the probability density function can be plotted, as Figure 2 shows. Despite the different skewness, the curves are practically coincident. Hence the Point Estimate Method apparently seems to be insensitive to the skewness and virtually gives the same curve as the computationally no-attractive Monte Carlo simulations.

Additionally in Table 1 the results from the Point Estimate Method are listed for a correlation coefficient of –0.6. For this case the mean value of the bearing capacity hardly changes, but the standard deviation strongly decreases, down to 69.97 kPa. Thus reducing the uncertainty in qf.

<table>
<thead>
<tr>
<th>Method</th>
<th>$\mu_{qf}$ (kPa)</th>
<th>$\sigma_{qf}$ (kPa)</th>
<th>$\nu_{qf}$</th>
<th>COV$qf$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Monte Carlo</td>
<td>304.72</td>
<td>115.94</td>
<td>1.9</td>
<td>0.38</td>
</tr>
<tr>
<td>PEM $\rho = 0$</td>
<td>305.39</td>
<td>110.48</td>
<td>1.2</td>
<td>0.36</td>
</tr>
<tr>
<td>PEM $\rho = -0.6$</td>
<td>298.79</td>
<td>69.97</td>
<td>0.59</td>
<td>0.23</td>
</tr>
</tbody>
</table>

Table 1. Bearing capacity results predicted by the Monte Carlo and the Point Estimate methods.

As a consequence also the skewness coefficient and the COV decrease. Hence the bearing capacity density function becomes narrower, as shown in Figure 2.

In many countries a safety factor, denoted as FS, of two on the bearing capacity used to be applied. In Figure 2 the line for FS=2 is also plotted. On neglecting the correlation coefficient for the strength parameters the probabilistic analysis, i.e. for $\rho=0$, the results show that a safety factor of two still involves a considerable risk of failure. However on introducing a realistic negative correlation of –0.6 the variation of the bearing capacity decreases considerably and this matches well with the use of a safety factor equal two.

3. Conclusions

From the probabilistic analysis of the bearing capacity problem it is possible to compare advantages and disadvantages of well-known probabilistic methods. In spite of some limitations, the Point Estimate Method shows to be a simple, but powerful technique as it matches Monte Carlo simulations extremely well. The Point Estimate Method will be used for a further study as it is much faster than Monte Carlo simulations. As an alternative to the Point Estimate Method, the First Order Second Moment method was applied, which gave results similar to those of the Point
Estimate Method. However it doesn’t allow the evaluation of the skewness, thus being substantially less accurate.

From figure 2 an enormous variation for the bearing capacity can be observed, when neglecting the correlation between the strength parameters. These results would seem representative for many soil problems as typical input data for the strength parameters, including the coefficients of variation, have been applied. It would seem very important to do probabilistic analysis and to use a negative correlation. A slope stability analysis was also executed, which confirmed the advantages of the Point Estimate Method. However additional case studies would be required for a more definitive judgement.

4. References


CONSOLIDATION OR CREEP OF A MULTIPHASE POROUS CHALK

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ABSTRACT. Related to the subsidence of North Sea Ekofisk oilfields, this study focuses on the analysis of some oedometer tests performed on a porous chalk under various conditions of saturation. The application of the general findings of unsaturated soils mechanics for the interpretation of the subsidence phenomenon is first demonstrated. The reliability of the consolidation theory for the interpretation of the oedometric compression tests for such a material is then discussed.

1. Introduction

In the North Sea Ekofisk oilfield, oil is located in a 150 m thick layer of porous chalk (n = 40-50%) at a 3000 m depth. After the depletion phase, an enhanced oil recovery procedure has been carried out by injecting sea water (waterflooding). An unexpected consequence of this waterflooding has been the occurrence of a seafloor subsidence of about 10 m. The subsidence due to waterflooding in reservoir chalks is a coupled problem typical of a multiphase geomaterial, i.e. the chalk full of water and of oil. Delage et al. (1996) showed how this problem could be considered within a framework taken from the mechanics of unsaturated soils, considering oil as the non-wetting fluid and water as the wetting fluid. The influence of the fluids on the mechanical behaviour of chalk has been deeply investigated during the Pasachalk EU projects (1997-2003). An extensive experimental programme, in which the oil-water suction \( s_o = u_o - u_w \) (where \( u_o \) and \( u_w \) are the oil and water pressures, respectively) was controlled and considered as an independent stress variable, like in unsaturated soils, was undertaken. The results showed that chalk strength decreases when the water saturation degree increases and that compaction of chalk when water saturated (i.e. pore collapse) can be interpreted as a classical phenomenon of collapse under wetting, typical of a number of loose fine grained soils (De Gennaro et al., 2004). The results also confirmed that \( s_o \) is a viable independent stress variable enabling to account for the intermediate states of oil and water partial saturation. Recently, time-dependent behaviour (viscous behaviour), that is supposed to be significant in chalks, has also been considered (Pasachalk 2, 2004; De Gennaro et al., 2003; Priol, 2005). Laboratory tests under controlled oil-water suction conditions were then carried out on chalk samples containing two fluids (oil and water). Some of these tests were performed using a suction controlled oedometer (Delage et al., 1992). The analysis of these tests raised some questions about the reliability of the classical consolidation theory and the well known methods of Casagrande (1936) and Taylor (1948) for the interpretation of the consolidation curves. These tests results are here discussed.

2. Material and methods

All the oedometric test results presented in this paper have been carried out on an outcrop chalk coming from a quarry near Lixhe (Belgium). Lixhe chalk is an almost pure chalk (99% calcite), which originates geologically from the same formation of the petroleum chalk of the North Sea deposits. The average porosity is quite high, about 43%. Lixhe chalk is characterised by a slight cementation due to precipitated calcite. Previous investigations on this material allowed to quantify values of the intrinsic permeability of about \( 1 \times 10^{-14} \) m\(^2\) (\( k_{water} \approx 1 \times 10^{-9} \) ms\(^{-1}\), \( k_{oil} \approx 7 \times 10^{-9} \) ms\(^{-1}\)). Water retention curves giving the relationship between oil-water suction and water saturation degree following both drying (i.e. oil injection) and wetting (i.e. water injection) paths were deduced adopting various techniques (De Gennaro et al., 2005), namely : osmotic
technique, axis translation technique and vapour phase control. Results indicated that Lixhe chalk is water wettable rather than oil wettable.

The material was retrieved by block sampling from the quarry. The cylindrical samples for the oedometric tests were trimmed from the block and then reshaped on a lathe. Final dimensions of the samples were 50mm in diameter and height to diameter ratio (H/D) of 2.5.

Oedometric tests were performed on samples water saturated, oil saturated, dry and at intermediate state of water and oil saturation. In the latter case partial saturation (50% water – 50% oil) was obtained imposing 200 kPa of oil-water suction according to the retention properties of the material.

3. Stress-Strain curves

Results presented in Figure 1 suggest that the stress-strain relationship depends on the nature of the saturating fluid. The sample partially saturated with water and oil (i.e. $s_o = 200$ kPa) exhibits a behaviour between the two extreme saturations in oil and water. Chalk compressibility increases when the water saturation increases, or alternatively when suction decreases, in a very similar way to what is generally observed in partially saturated soils. The yield stress can then be considered as a function of the oil-water suction $s_o$, supporting the concept of LC curve in the plan suction vs. mean net stress, originally introduced in the Barcelona Basic Model (Alonso et al., 1990).

The overall effect of oil-water suction on chalk compressibility permits also to explain the increasing subsidence observed in the North Sea oilfields just after the sea water injection (water flooding). Two mechanisms are likely to be part of the cause of subsidence. The first mechanism corresponds to the collapse phenomenon produced when chalk is wetted under constant vertical load. This phenomenon is shown in Figure 2. During the oedometric tests on the oil saturated sample, water infiltration in the sample was performed under a constant vertical load of 19.8 MPa. Inspecting the consolidation curves in Figure 2, an instantaneous collapse was observed.

However, it is evident from Figure 2 that consolidation curves look like creep curves, with an instantaneous compaction and a delayed deformation from the early stage of the loading, with no apparent effects of excess pore pressure dissipation. Moreover the delayed deformation depends on the nature of the saturating fluid. Specifically, the void ratio decreases much more sharply once that chalk is water saturated. This is likely to translate a second mechanism of deformation at the origin of subsidence, which is time-dependent (creep), due to chalk viscous behaviour.

![Figure 1: Stress-strain curves for loading stages tests](image1)

![Figure 2: Water infiltration test](image2)
4. Creep curves

The use of an incremental step loading procedure during oedometric tests is relevant for the study of time dependent behaviour of chalk. Nonetheless, it was not possible to use the usual graphical methods (Casagrande, 1936; Taylor, 1948) to identify the end of primary consolidation and complete pore pressure dissipation. Actually, due to cementation, chalk doesn’t really follow the classical theory of consolidation, and no significant pore pressure has been detected during the ‘consolidation’ process. Also, the relatively high permeability ($10^{-8}$ m/s) is sufficient to admit full drained conditions. The delayed settlements during the load stages have been interpreted as creep. Based on this hypothesis, a model has been introduced in order to provide some parameters (Priol et al., 2004). The evolution law for the void ratio during creep is chosen as follows:

$$\frac{e}{e_{oi}} = t^{-\alpha_i} + \beta_i$$  \hspace{1cm} (1)

Where $e_{oi}$ is the initial void ratio, $\alpha_i$ and $\beta_i$ two parameters controlling shape and position of the creep curve. Taking into account the hardening effect of suction (and fluids), we have compared in Figure 3 the creep evolution (parameter $\alpha$) vs. the overstress ratio (applied stress divided by yield stress, $\sigma/\sigma_\text{y}$). A unique relationship seems to exist between $\alpha$ and $\sigma/\sigma_\text{y}$, as shown in Fig. 3. In Figure 4 creep curves are plotted for different saturation configurations and same value of overstress ratio.

![](Figure 3: Creep rate evolution against the overstress ratio)

![](Figure 4: Creep at the same overstress ratio (1.1))

5. Conclusions

The experimental data presented in this paper were aimed at showing some effect of multiphase fluids on the compression properties and time dependent behaviour of a petroleum chalk. As in unsaturated soils, suction decrease and the related increase in water degree of saturation has a significant weakening effect on the chalk. Water weakening effect in chalks has been extended to intermediate states under suction controlled conditions. Under similar stress states, creep is increased by suction decrease.

These new features are obviously important to better address the problems of oilfield subsidence and of the long term stability of chalk.
6. Acknowledgements

The authors want to acknowledge the AMGISS and MUSE EU RTN networks.

7. References


MECHANICAL BEHAVIOUR OF CLAY-SHALES (ARGILLE SCAGLIOSE) AND IMPLICATIONS ON THE DESIGN OF TUNNELS

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ABSTRACT. This document describes the studies carried out for the characterisation of time-dependent behaviour of Italian clay shales. The interest arises in connection with tunnelling works in Italy through structurally complex formations. Attention is mainly paid to the results of the laboratory tests and their effects on the mechanical behaviour. Concluding remarks and future developments are discussed.

1. Characterisation studies

A brief discussion of the most important topics concerning the characterisation of Chaotic Complex Tectonised Clay Shales formation (CCTCS) is given in the following. More details may be found in (Bonini et al., 2003; Barla G. et al., 2004a). Laboratory tests were performed in order to determine the parameters needed for simulating the tunnel behaviour in short- and long-term conditions. To this end, a number of cubic samples were taken from the Raticosa tunnel and from the Osteria access adit. From North to South (Figure 1 left), the Raticosa tunnel crosses respectively the CCTCS (through a 5.5 km length), marly and arenaceous formations. A landslide, involves approximately 500 m of the tunnel length from the northern portal. Here the formation has undergone softening and/or weathering processes which have modified both its structure and properties. The recent geological history and the variable structure of the material are likely to have originated a very different mechanical behaviour in terms of time dependence during excavation.

The following tests were carried out: physical properties; mineralogical contents; oedometer tests (conventional tests on both natural and reconstituted materials and Huder-Amberg tests on natural materials); triaxial tests in closely controlled stress-path conditions. The CCTCS are “inorganic clays of medium to average plasticity”. Index properties and grain size distribution are in the same range as shown by similar Apennines clay shales which involved tunnelling under difficult conditions.

From a qualitative point of view (X-ray diffraction analyses), the CCTCS show an average-high swelling potential due to the presence of illite and smectite. The swelling potential has also been investigated by means of the Huder-Amberg modified oedometer tests. The compressibility characteristics of clay shales were inferred by comparing oedometer tests on natural and reconstituted samples (Figure 1 right). To this end, the testing program was designed in order to obtain both the intrinsic properties and the sensitivity of the double diffuse layer to the pore fluid composition (NaCl saturated solution). The only oedometer test performed on natural material gave little information on both the compressibility and the original state of stress. On the other hand, the results of the oedometer tests performed on reconstituted specimens with different pore fluid composition show the CCTSC (i.e. the void index, the consolidation coefficient, the consolidation time $T_{100}$ and the permeability) to depend upon the cations concentration (e.g. Na$^+$), according to the smectite content. Chemical consolidation and swelling under constant loading conditions can be induced by changing the ionic concentration.

The natural water content of CCTCS is very low and ranges from 5% to 22%, increasing with the overburden. The permeability $k$ resulting from the interpretation of oedometer tests is $1 \times 10^{-9}\div 1 \times 10^{-12}$ m/s, while Lugeon tests performed in situ give values in the range $3 \times 10^{-7}\div 2 \times 10^{-9}$ m/s.
The most important characteristics of the CCTCS are obtained from triaxial tests in closely controlled stress-path conditions. The specimens are subjected to a test procedure purposely designed for swelling soft rocks. In particular, the $s = 0.5 \cdot (\sigma_v + \sigma_h) = \text{constant}$ tests performed to study the mechanical behaviour in undrained conditions show the CCTCS to exhibit a ductile stress-strain behaviour, with axial failure strain reaching about 5%. Both the stress-strain behaviour and stress path prove to be influenced by the applied strain rate and saturation degree (Figure 2 left).

Finally, significant time-dependent strains are observed in drained and undrained creep tests, even for relatively low stress levels (50% of failure deviator). Time-dependent strains prove to be increasing with strain level and decreasing with elapsing time (also at incipient failure).

2. Implications on the mechanical and recent developments

Based upon the results of triaxial and creep tests it is concluded that swelling strains cannot develop in the “short term”, while the time-dependent strains are likely to represent a significant amount of the total deformation of the CCTCS. For the purpose of representing the mechanical time-dependent behaviour of the clay shales tested, two constitutive models were adopted: a visco-elastic plastic model and an elastic visco-plastic potential model (Barla G. et al., 2004b). In both cases, the necessary model parameters have been obtained successfully. In the case of the visco-elastic plastic model it has been shown that, in general, the laboratory data cannot be used directly to assess the material behaviour and that the parameters need to be scaled up significantly in order to obtain an appropriate in situ prediction. In the case of the elastic visco-plastic potential model it has been shown that the material response can be described in detail with parameters based directly on laboratory testing (Bonini et al., 2005).
Further developments are needed and some open questions remain to be addressed for the study of the time-dependent behaviour of clay shales in relation to tunnel excavation. To this end, difficulties encountered in testing clay shale specimens need yet to be resolved in relation to the degree of saturation, the influence of disturbance and the initial state of stress. Finally, the interest arises to study the time-dependent behaviour of clay shales when tunnelling at depth.

3. References


Numerical modelling
PHYSICAL AND NUMERICAL MODELLING OF SOFT SOIL IMPROVEMENT BY VERTICAL RIGID PILES

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ABSTRACT. The improvement by vertical rigid piles consists of a pile grid and a granular earth platform, in which partial load transfer onto the piles occurs due to arching, permitting surface settlement reduction and homogenization. The system presents complex soil-structure interactions. A two-dimensional physical modelling is developed to constitute an experimental database, which is used as reference for a 2D numerical modelling using a continuum approach.

1. Introduction

Rarefaction of good quality soil necessitates the use of techniques to improve soft soils. The improvement by vertical rigid piles consists of a pile grid and a granular earth platform, in which partial load transfer onto the piles occurs due to arching, permitting surface settlement reduction and homogenization. Figure 1 illustrates the improvement principle. The main application areas are road- and railway embankments and industrial pavements. Various design methods exist to determine the load transfer onto the piles, but they often lead to different results (Briançon et al., 2004). There is a need to understand more precisely the system behaviour. The thesis work focused more precisely on the modelling of the mechanisms occurring in the granular earth platform. The proceeding consists in performing physical experiments to constitute an experimental database, in order to validate a numerical modelling. The aim of this modelling is to be extended to real three-dimensional cases.

2. Physical modelling

The developed physical model is two-dimensional and composed of analogical materials. It permits to perform parametric studies on geometrical parameters and on soft soil compressibility. It also permits to investigate the contribution of basal geosynthetic reinforcement.

A diagram of the test apparatus is given on figure 2. The granular earth platform is modelled by a Schneebeli analogical soil assembly, which is a mix of 3, 4 and 5mm diameter rods. This analogical soil behaves as a dense granular soil: it is frictional, cohesionless, dilative and the deformation modulus depends on the stress level. The soft soil is modelled by foam and the rigid piles by steel element. The instrumentation of the model by load cells permits the determination of the load distribution at the platform base. The system is loaded by successive rod layers and photographs are taken at each step. An image correlation method (Mguil-Touchal et al., 1996) then gives the displacement field in the model. The multiplication of the parametric studies, the precise and reproducible results in term of both loads and displacements, permit to create a complete experimental database which constitutes a reference for various numerical modelling approaches and for a confrontation to design methods (Jenck et al., 2005).
3. 2D numerical modelling in a continuum

The finite difference software “Flac” is used in plane strain. Due to symmetry conditions, only a quarter of the physical model is represented, as shown on figure 4. The half rigid pile is simulated by fixing nodes. The foam behaviour is determined on an experimental loading test and is simulated by an elastic non linear model. The analogical soil behaviour is first simulated by the Mohr-Coulomb model, and then by the CJS2 model (Maleki et al., 2000), in order to investigate the model complexity influence. Biaxial test performed on analogical soil samples (Dolzhenko, 2002) are used to calibrate the parameters. The friction angle is 24°, the cohesion is null, the dilation angle is 4°, the Poisson ratio is 0.48 and the Young’s modulus varies according to the confinement pressure. Using Mohr-Coulomb, this last parameter is not easy to determine because of linear elasticity, in opposition to the hardening elastoplastic model CJS2, in which the non linear behaviour is directly taken into account.

![Figure 4. Two-dimensionnal numerical modelling in a continuum](image)

Figure 5 compares the numerical and the experimental results for 3 values of a/s (15, 22 and 31%). The “efficacy” is the ratio of the load applied on the piles to the total platform weight. It increases with the platform height. The numerical results are almost the same using Mohr-Coulomb and CJS2. This figure shows that the numerical simulation overestimates the load transfer onto the piles and reproduces the observed maximum settlement at platform base. The confrontation was also performed on the displacement field in the model and on the surface total and differential settlements. It was shown that the proposed model is able to represent the observed phenomena. The parametric study could then be extended using the numerical modelling.

![Figure 5. Confrontation between numerical and experimental results](image)

4. Conclusions

The developed small scale model permits better understanding of the system behaviour and multiplication of the experiments with good reproducibility conditions, in order to perform parametric studies. A precise and complete experimental database is constituted, used as a reference for a numerical modelling approach in a continuum. The numerical modelling will be extended to real 3D cases, validated on real scale experiment results.
5. References


ABSTRACT. The Haarajoki test embankment improved with vertical drains installed in soft clay is analysed with the finite element method using three different constitutive models. The constitutive models are the two recently proposed advanced constitutive models for soft clays, namely SCLAY1 and S-CLAY1S. For comparison, the problem is also analysed with the isotropic Modified Cam Clay model. Finally, the results of the numerical analyses are compared with the observed field measurements.

1. Introduction

Vertical drains are used to improve soft soils under embankments in order to reduce the settlements, accelerate consolidation and increase stability. Soft soils tend to be highly anisotropic due to their deposition history. Neglecting the anisotropy of soil behaviour during loading can lead to inaccurate predictions of the settlements of embankments. In addition to anisotropy, the structure of soft soils includes interparticle bonding. When plastic straining occurs, the bonding degrades, referred to as destructuration.

The construction and consolidation of the Haarajoki embankment is simulated with the 2D finite element code PLAXIS. The soft soil is modelled with three different constitutive models, the isotropic Modified Cam Clay (MCC), The S-CLAY1 model, which accounts for plastic anisotropy and its extension the S-CLAY1S that accounts in additions for interparticle bonding and the degradation.

2. Haarajoki Test Embankment

The Finnish National Road Administration organised an international competition to calculate the behaviour of the Haarajoki Test Embankment (FinnRA, 1997). The 3.0 m high and 100 m long Haarajoki test embankment, has been built as a noise barrier. The crest of the embankment is 8 m and the slopes have a gradient of 1:2. Half of the embankment is constructed on area improved with vertical drains and the other half on natural deposits without any additional ground improvement. The embankment itself was a constructed 0.5 m layer whereas each of the layers was applied and compacted within 2 days. In the improved area the vertical drains are installed in a regular pattern with 1 m spacing. Numerous monitoring devices (settlement plates, piezometers, inclinometers, extensometers, pressure cells) are installed for monitoring the vertical and lateral displacements and the pore pressures. The Haarajoki deposits can be characterised as a sensitive anisotropic soft soil with an organic content of 1.4 to 2.2%.

3. Constitutive Models

The anisotropic elasto-plastic model S-CLAY1 is an extension of conventional critical state models, with anisotropy of plastic behaviour represented through an inclined yield surface and a rotational component of hardening to model the development or erasure of fabric anisotropy during plastic straining. An initial version of the model was proposed by Wheeler (1997) and this was subsequently modified to its current form by Näätänen et al. (1999) and Wheeler et al.
(2003). For the simplified conditions of a triaxial test on a cross-anisotropic sample, with the horizontal plane in the triaxial sample coinciding with the plane of isotropy of the sample, the S-CLAY1 yield curve can be expressed in terms of the mean effective stress $p'$ and deviator $q$:

$$f = (q - \alpha p')^2 - (M^2 - \alpha^2)(p'_m - p')p' = 0$$

(1)

where $M$ is the value of the stress ratio $\eta = q/p'$ at critical states, $p'_m$ defines the size of the yield curve and $\alpha$ defines the orientation of the yield curve. The S-CLAY1S model is an extension of the S-CLAY1 model that additionally accounts for bonding between clay particles and degradation of bonding. The additional strength of soil given by bonding is expressed by using the concept of an intrinsic yield curve, which is the theoretical yield curve for the same soil with the same void ratio and fabric but without bonding. The shape and inclination of the intrinsic yield curve are the same as those of the yield curve for the natural soil (Eq. 1). However, the size of the intrinsic yield curve $p'_m$ is smaller than $p'_m$ for natural soil.

$$p'_m = (1 + x)p'_m$$

(2)

where $x$ is a measure for the amount of bonding.

4. Numerical Analyses and Comparisons with Observations

The construction and consolidation of Haarajoki test embankment has been modelled with the two dimensional (2D) finite element code PLAXIS V8. Due to symmetry of the embankment just half of it was modelled in a plane strain analysis. The construction was simulated in 0.5 m stages using a coupled consolidation analysis to account for the construction schedule. An example of the comparison between the results of the finite element simulations and the field observations at the centre line of the embankment are presented in Fig. 1. The finite element predictions of the vertical displacements by the two anisotropic models (S-CLAY1 and S-CLAY1S) are in good agreement with field observations. The isotropic MCC model predicts notably smaller settlements than the two anisotropic models. The simulations demonstrate that for this type of problem it is important to account for the anisotropy and destructuration. It has also been found that the inclusion of the smear effect improves the accuracy of settlement predictions based on the equivalent plain strain model, whereas the analysis results using “perfect drains” significantly overestimates the settlements.

![Figure 1. Surface settlements at the centerline for Haarajoki test embankment.](image-url)
5. References

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MODELLING WITH FINITE ELEMENT STONE COLUMNS IN SOFT CLAY

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ABSTRACT. Soil reinforcement by means of stone columns can be analyzed by two dimensional finite element method. The influence of factors that play a major role on the design of stone columns projects has been determined so as to conclude some practical results.

1. Introduction

The vibrated stone column technique is an economical and environmentally friendly process that treats weak ground to enable it to withstand low to moderate loading conditions (Dhouib & Blondeau, 2005). Large-sized columns of coarse backfill material are installed in the soil by means of special depth vibrators (Priebe, 1995). Reinforcement of the soil with stone columns provides basically reduction of foundation settlement, improvement of the bearing capacity of the soil, reduction of the risk of liquefaction due to seismic activity and accelerates the consolidation process (Barksdale & Bachus, 1983).

The main objective of this study is to examine the influence of different factors and to conclude some practical results on the design of stone columns projects. The effects of column diameter (D) and spacing (s), initial stresses in the ground, gravel and soil material properties and applied load of the stone column on the deformation in the ground are examined, showing the beneficial effect of this ground improvement technique on settlement reduction.
in the upper part of the stone column. The ultimate stress that can be applied to the stone column (Soyez, 1985) is:

\[
\sigma_{c,\text{lim}}' = K_{p,c} \cdot \sigma_r'
\]

where \( \sigma_r' \), the ultimate lateral stress of the soil and \( K_{p,c} \), the coefficient of passive pressure of the stone column.

The conversion of axisymmetric stone column unit cell model with finite element analysis is used in this study. The stone column and the surrounding soil are assumed to behave as elasto-plastic Mohr-Coulomb materials. Several factors, such as column spacing and depth, initial stresses in the ground, stone and soil material properties, applied load and dilatancy of stone column material influence the prediction of the settlements of foundations supported by end-bearing stone columns. After determining the influence of each parameter the proposed model is validated by comparisons with field test results.

2. References


GEOMECHANICAL MODELLING OF TRIGGERING MECHANISMS FOR FLOW-LIKE MASS MOVEMENTS IN PYROCLASTIC SOILS

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ABSTRACT. Flow-like mass movements involving volcanic soils are widespread all over the world and often result on catastrophic consequences. Modelling of triggering mechanisms represents a fundamental step towards an adequate hazard assessment. Moving in the field of environmental geomechanics, integrated approaches and multidisciplinary studies appear as fundamental aspects for a better understanding, modelling and classification of such complex phenomena.

1. Introduction

Flow-like mass movements involving volcanic soils are among the most dangerous natural hazards because of their weak warning signals, long travel distances, high velocities and often huge involved volumes (Legros, 2002). They are widespread all over the world and can be triggered by several causes as rainfall, earthquake, weathering, human activities or their combination, frequently producing catastrophic consequences. Notwithstanding their similar consequences, these phenomena can present different first-failure and post-failure mechanisms, according to regional, seasonal and local features, triggering factors and mechanisms, soil properties and boundary conditions.

When facing such a complex problem, analyses are generally based on both classifications and models aiming to capture, under different assumptions, the essential features of the first-failure, post-failure and propagation stages. However, available classifications do not always allow a unique conceptual framework for the instability phenomena and a sector-based modelling can disregard some relevant factors characterizing the first and post-failure stages. This topic is in the following discussed with reference to the first-failure stage induced by rainfall.

2. Modelling of rainfall induced triggering mechanisms

Referring to flow-like mass movements induced by rainfall, a variety of triggering mechanisms are discussed in literature. They are mainly related to the soil shear strength reduction caused by the pore water pressures increase as a result of several phenomena: surface run-off, raising of the water table, groundwater supplies provided by artesian conditions or hidden springs, particular groundwater flow patterns related to the stratigraphic setting, increase of saturation degree in unsaturated soils (Leroueil, 2004). The appropriate modelling of triggering mechanisms, that can arise from one or more of the above conditions, represents a powerful tool for landslides hazard assessment. To this aim, the different available approaches can be classified into the following main groups: the black-box models, the geological models, the physically based models, the geomechanical models and physical models.

The use of each model, often allows satisfactory back-analyses of several phenomena at regional, local and site scale. However, with reference to hazard assessment and forecasting, the output of each model can be strongly conditioned and/or limited by several approximations. More reliable results could be achieved by joining the potentialities of some available models, in order to provide an adequate recognition, at different scales, of the relevant key factors, so allowing a significant improvement of both the understanding and classification of the studied phenomena. An example of the usefulness of a multidisciplinary approach is hereafter discussed.
3. A case study from Southern Italy

The case study refers to a sample area involved, on May 1998, by a catastrophic event that threatened five little towns of Southern Italy causing 160 victims (Cascini, 2004). To study this event several investigations were carried out. The in-situ activities included topographical surveys, stratigraphic investigations and soils suction monitoring using portable and in-place tensiometers (Cascini & Sorbino, 2002). Moreover, an extensive laboratory test program was performed on undisturbed and remoulded specimens by means of Suction Controlled Oedometer, Volumetric Pressure Plate Extractor, Richard Pressure Plate and Suction Controlled Triaxial Apparatus, allowing the collection of a noticeable data set of physical and mechanical properties of the involved ashy and pumice soils (Bilotta et al., 2005).

By adopting a geological approach, rainfall-triggered mechanisms were found to be as strictly related to the bedrock morphological and hydro-geological features, as well as to the past and actual processes involving pyroclastic covers. From the analysis of all the available elements, six typical triggering mechanisms were recognised and mapped all over the sample area. They are characterised by different intensity in terms of mobilised volume and travel distance and they are not casually distributed on the massif.

In order to validate these triggering mechanisms, simulations on the mechanical processes, able to reproduce in-situ evidences, were performed at massif, site and Representative Elementary Volume (REV) scales. The preliminary analyses took account of: simple constitutive models with a non-associated flow rule; a saturated-unsaturated groundwater modelling by means of a commercial finite element code (Geo-Slope, 2005); limit equilibrium method, analysing the slope stability conditions, based on the previously computed pore water pressures. The obtained results were then used to assess the instability scenarios and to point out the most relevant triggering factors at massif scale (Geo-Slope, 2005). Successively, uncoupled finite element analyses were addressed to deepen the understanding of the in-situ instability conditions, by adopting an elasto-plastic model with a Mohr-Coulomb criterion extended to unsaturated conditions (Fredlund et al., 1978). Finally, fully coupled analyses were performed by means of the GeHoMadrid finite element program (Pastor, 2002), assuming plane strain and 3D conditions and using respectively eight-node and twenty-node elements for the calculations.

The obtained results strongly encourage towards an integrated multidisciplinary approach, as highlighted in Figure 1 that refers to a typical triggering mechanism, producing elongated source areas, induced by both rainfall and water supplies from the bedrock. As a matter of fact, the results of both geological and geomechanical analyses are capable to match the in-situ evidence, while similar results are not obtained by any of the sector-based approaches used to face the problem, as it can be observed referring to the wide scientific literature on the topic (Cascini et al., in press).

Figure 1. A typical triggering mechanism: a) in-situ evidence, b) geological model, c) limit equilibrium analysis, d) displacement contours for 2D uncoupled FEM analysis, e) plastic strains for 3D coupled FEM analysis.
4. References


NUMERICAL MODELLING OF DEEP MIXED COLUMNS

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ABSTRACT. Numerical simulations of embankment fills constructed on deep mixed columns using an axisymmetric unit cell approach are studied. The constitutive models used to model the soft soil are the recently proposed anisotropic S-CLAY1 model and its extension, called the S-CLAY1S. The S-CLAY1S model accounts, additionally, for interparticle bonding and degradation of bonds. Further, a volume averaging technique is presented, that allows modelling the global response of deep mixed columns in plane strain simulations.

1. Introduction

Deep mixed columns installed in a periodic manner are a common and very effective method, extensively used in Scandinavian countries to improve the characteristic and behaviour of soft soils under embankment fills. Currently the design is based on rigid-plastic solution to predict ultimate loads and empiric (elastic) solutions to estimate settlement of the periodic system. Numerical simulations can provide a powerful alternative design strategy by accounting for the real stress-strain behaviour of the soft soil and the column material. Two numerical models are introduced, one is a simple unit cell and the second model uses a volume averaging technique to model deep mixed columns installed in a periodic manner under an embankment fill. In all simulations advanced soil models are applied to the soft ground to account for realistic stress-strain behaviour of the soil deposits.

2. Modelling soft soils

Natural soil deposits, due their geological history and mainly one dimensional deposition tend to have a significant anisotropy of fabric. Fabric anisotropy can influence both elastic and plastic behaviour. For normally consolidated or lightly overconsolidated soft clays, plastic deformations are likely to dominate for many engineering problems. In addition to anisotropy, natural soils exhibit bonding between particles. The bonding can be progressively destroyed during plastic straining and this often referred as destructuration (Burland 1990). The S-CLAY1S model developed at the University of Glasgow accounts for plastic anisotropy, bonding and destructuration. S-CLAY1S is developed within the standard elasto plastic framework. It uses an inclined yield surface with a rotational hardening law (Wheeler et al. 2003) to represent anisotropy and the evolution of anisotropy during plastic straining. The effect of bonding is considered following the idea of Gens and Nova (1993) by introducing a second yield surface, the “intrinsic yield surface” to represent the unbonded soil. The difference in size between these yield surfaces is a measure for the bonding effect and can be written as

\[ p_m = (1 + x) p_{ui} \]  

where \( p_{m} \) specifies the size of the intrinsic yield surface, \( x \) defining the degree of bonding and \( p_{m} \) the size of the yield surface of the natural soil.

In three-dimensional stress space the yield surface of S-CLAY1S model is a sheared ellipsoid given by

\[ F = \frac{3}{2} \left[ (\sigma_d - p' \mathbf{a}_d) (\sigma_d - p' \mathbf{a}_d)^T \right] - \left[ M^2 - \frac{3}{2} \left( \mathbf{a}_d (\mathbf{a}_d)^T \right) \right] (p_m' - p) = 0 \]  

where \( \sigma_d = \) deviatoric stress tensor; \( p' = \) mean effective stress; and \( \mathbf{a}_d = \) dimensionless second order tensor describing the fabric anisotropy, called the fabric tensor. \( M = \) critical state value of the stress ratio in triaxial space and the state parameter \( p_m' \) defines the size of the natural yield.
surface. S-CLAY1S incorporates three hardening laws (Karstunen et al. 2005). The first, one similar to that of the Modified Cam Clay model, describes the changes in the size of the intrinsic yield surface. The second and third hardening laws describe, respectively, the rotation of the yield surfaces and the degradation of bonds with plastic strains.

![Figure 1. S-CLAY1S yield surface (a) General stress space and (b) Triaxial stress space.](image)

3. 2D numerical modelling

The system is idealised as an axisymmetric problem, by using a two-dimensional unit cell. With such a model one can represent a single column and the surrounding soil. The radius of the unit cells depends on the column spacing. The parameter values chosen for these simulations to represent the soft soil and the columns are based on laboratory tests on natural and deep mixed Vanttila clay. The simulation investigates how fabric anisotropy, interparticle bonding and the degradation of the bonds in the soft soil (between) the column influence the stress-strain behaviour of the system. Based on the results it can be shown that anisotropy and destructuration have a minor effect on the vertical stress distribution but increase the magnitude of the total displacements of the system.

4. 2D modelling using the volume averaging technique

A powerful alternative to the unit cell approach is the volume averaging technique. With this technique it is possible to model the global behaviour of deep mixed columns installed in a regular pattern. Equivalent material properties are established for the in-situ soil and the column composite. The difficulties encountered in carrying out elastoplastic analyses of composite materials are overcome by adopting separate yield functions for each constituent material and a sub-iteration procedure. In the proposed procedure, equilibrium as well as kinematic conditions implied in the homogenization procedure are satisfied for both elastic as well as elastoplastic stress states.

Averaging rules, equilibrium and kinematic conditions at the local interface between soil and column material can be described (Lee & Pande, 1998). By combining the constitutive equations of soil and column with the equilibrium and kinematic conditions an equivalent constitutive matrix \( D^{eq} \) can be determined. \( D^{eq} \) is a function of the volume fractions and the constitutive matrixes for the soil and column. After the determination of an equivalent material constitutive matrix \( D^{eq} \) the strain increments for each individual constituent is calculated from the equivalent strain. Then the yield criterion for each constituent is checked individually and possibly return mapping procedures are applied. Thereafter the stresses have to be redistributed to satisfy the local equilibrium conditions. In the current implementation the stress differences are added to the column material. After a recheck of the column yielding and a possible further iteration the equivalent stresses for the averaged material can be calculated using \( D^{eq} \). For soil and column in the presentation the S-CLAY1S and ideal elastic models are used respectively.
5. References


CENTRIFUGE MODELLING OF THE BEHAVIOUR OF DOUBLE POROSITY SOILS

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ABSTRACT. The geotechnical centrifuge is presented as a tool for modelling consolidation and compression behaviour of double porosity soils, and for simulation of these processes for a case history of an embankment constructed on such deposits in North-Western Bohemia.

1. Introduction

Double porosity soil is a material with complex mechanical behaviour and detailed investigation of its properties is required before it can be constructed on. In the Northern part of the Czech Republic, an area of ca. 100 km$^2$ is occupied by double porosity material composed of clay lumps. Overburden soil from mines was dumped in the neighbouring areas and formed landfills with depths typically of tens of metres. It is now planned that a new motorway between Prague and Dresden will cross an area of landfills 20-30 years old. Two trial embankments were built on the projected route and the subsoil was instrumented by hydrostatic levelling profiles, reference points to measure vertical movements, standard inclinometers and pore water pressure gauges. Results of the measurements have been published (Boháč & Škopek, 2000; Škopek & Boháč, 2004) and the data are available for the projected simulation of the site response in the ETH Zürich geotechnical centrifuge (Springman et al., 2001).

2. Preliminary investigation

The total porosity of lumpy clay is composed of the inner porosity of the clay lumps (intrgranular porosity) and the spaces between clay fragments (intergranular porosity). The consolidation behaviour of lumpy clay is difficult to predict and is influenced by the degree of saturation ($S_r$, both total and intergranular) and the dimensions and shape of the lumps.

Preliminary tests in a mini-centrifuge are in progress to model the consolidation behaviour of this soil. This mini-centrifuge has a diameter of 37cm with 1D consolidation performance measured in 14cm deep and 8cm diameter pots. The aim of these tests is to compare lumps of different shapes and dimensions to choose the optimal material for large drum centrifuge testing. The first results of mini-centrifuge tests (200g at 2/3 model depth) are presented in Figs. 1 & 2. Fig. 1 shows the consolidation of fully saturated samples with different lump dimensions and initial heights. Settlement was measured at 7 intervals up to a model time of 392 min., roughly corresponding to a 30 year-old landfill (using a scaling law based on consolidation theory). Difference in final settlement is a result of the different initial intergranular porosities. After consolidation, water content was measured at 8 depths in each sample. Fig. 2 shows that in the upper two thirds of both samples, some reduction of intergranular porosity has taken place, while in the lower third, intergranular voids have probably closed.

These preliminary tests will shortly be broadened and extended by a parametric study with varying $S_r$ and different g-levels. Oedometric tests are planned to confirm the stress history and measure the apparent preconsolidation stress and the variation in compressibility of soil taken from different levels of the samples after consolidation.
Material preparation

The complex interaction of mechanisms in a double-porosity material means that the material properties can greatly affect the deformation behaviour and must be carefully controlled. It is planned for all models to be constructed with soil from the test embankment site in Bohemia. Consolidation mechanisms are strongly influenced by $S_r$, so it is crucial to control the initial water content of the lumps and intergranular water level.

The main constraints on material preparation methods are the lump size grading and maximum lump dimensions. The preparation method must be repeatable and mimic as closely as possible that in real life, so remoulding into balls is not considered an appropriate technique (due to loss of stress history and non-angular lumps). Mechanical crushing methods have proved successful, with the drawback that the clay must first be dried, and the smaller lumps produced are flake-shaped with dust that must be sieved out.

3. Large drum centrifuge modelling

A series of drum centrifuge model tests will simulate the consolidation behaviour of this double porosity soil, and assess how construction-dependent settlements might be predicted from field data and hence performance improved. Ground improvement techniques to be trialled will include preloading on geotextile layers, and sand compaction piles, with the aim of minimising post-construction and secondary settlements. Recent projects using the ETH Zürich drum centrifuge have been successful in modelling various consolidation deformation geometries in clay and the in-flight installation of sand compaction piles (Weber, 2004; Weber et al., 2005).

Axisymmetric tubs will initially be used to reproduce the simplest 1-dimensional deformation geometries of double porosity soil models. Model dimensions will correspond to the size of the trial embankments previously mentioned. This will necessitate a complex test procedure, since constraints on the lump dimensions and g-level of the centrifuge will require the model to be deeper than usual for this centrifuge. Double-stacked tubs will be used, with the upper one removed after the initial stages of consolidation to allow space for the tool plate. Instrumentation will consist of pore pressure transducers and laser measurement of the displacement of the surface of the soil. After modelling self-weight consolidation behaviour of the fully saturated soil, tests will be repeated with a simulated embankment and ground improvement through sand compaction piles and surcharge preloading. Cone penetration and T-bar tests will be assessed for their capacity to predict the consolidation behaviour of the soil.

2-dimensional consolidation geometries will be modelled in square tubs fitted with a transparent window in one face. This will allow measurement of lump deformation throughout the depth of the sample using sequential in-flight images and particle imaging velocimetry adapted for geotechnical models (White et al., 2005).
A full-drum centrifuge model is planned as a final stage of testing, in which 3-dimensional deformation geometries can be studied. A large number of sand columns will be installed and their interaction behaviour, and the disturbance and stress change in the soil will be measured.

4. References


ABSTRACT. Presented are the calculations of a 3-dimensional FE-analysis of a slab on a soft soil supported by stone columns. They are put in contrast to calculations under plane-strain and axi-symmetric conditions. Both are carried out using a higher-order material model with stress-dependent stiffness and shear and compression hardening.
RESPONSE OF A CEMENTED GEOMATERIAL UNDER EXCAVATION AND CYCLIC CHANGES OF RELATIVE HUMIDITY

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ABSTRACT. In this paper, aspects of modelling geotechnical problems for a class of hard soils and soft rocks, soft argillaceous rocks, are addressed. The model developed by Vaunat and Gens (2003), dealing with typical features of cemented geomaterials, is introduced first. Afterwards, additional developments are proposed to tackle suction induced effects. Finally a simulation of the excavation and ventilation of a 6m diameter shaft in a Callovo-Oxfordian mudstone has been performed using a computer code, Code Bright, in which the new model has been implemented. The presented model appears to offer a possibility to simulate the time dependent damage and fracturation induced by suction cycles often observed in this type of geomaterials.

Cemented geological materials normally occupy the transition zone between soils and rocks (Kavvadas & Amorosi, 2000; Nova et al., 2003; Baudet & Stallebrass, 2004). They are involved in many geotechnical projects: mining excavations, road construction, tunnelling, landslide remediation, etc. The response of these materials is often characterised by features such as high spatial variability, significant carbonate calcium content, marked anisotropy, elastic stiffness degradation, brittle behaviour during shearing, crack opening during unloading and high water sensitivity. The modelling of these materials requires therefore the development of constitutive laws able to tackle such complex response as well as the proposal of robust integration schemes for the solution of Boundary Value Problems. Softening response associated with material degradation, in particular, is one of the central questions associated to the numerical modelling of these materials. Furthermore, suction can have significant additional effects on mechanical behaviour. In addition to the usual stiffening and strengthening effects, there is a direct influence on the cementation effects that generally increase when the material becomes unsaturated (Leroueil & Barbosa, 2000).

References

PERFORMANCE OF THE DURHAM UNIVERSITY – WYKEHAM FARRANCE Tensiometer

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ABSTRACT. A tensiometer able to measure soil suctions in excess of 100 kPa is currently being investigated at Durham University in association with Wykeham Farrance Limited. Recent studies have revealed that the range and stability of suction measurements depend on the achievement of full saturation inside the tensiometer’s filter and reservoir. Any undissolved small air bubbles or dust particles represent potential nuclei where water under tension can fracture and cavitate.

The presentation focuses on two aspects, (1) the maximum suction measured and, (2) the accuracy of its measurement. The maximum suction measured will depend on the tensiometer structure (including materials, dimensions, and transducer characteristics) and saturation procedure. The tensiometer has been calibrated in the positive range and the extrapolation of such calibration to the negative range has been investigated. In the positive range it was calibrated against a standard transducer. In the negative range, its accuracy has been studied by 2 different techniques: (1) axis translation technique, by releasing the air pressure from a positive value and measuring the corresponding decrease of pore water pressure with the tensiometer. (2) Isotropic unloading of a saturated sample in undrained conditions in a triaxial cell, by decreasing the cell pressure and measuring the decrease in water pressure with the tensiometer. For both techniques, the decrease of the air pressure or cell pressure should lead to an equal decrease of pore water pressure. Details on the post-cavitation response and other aspects of the tensiometer will also be addressed.
COUPLING OF WATER RETENTION BEHAVIOUR AND MECHANICAL BEHAVIOUR IN UNSATURATED SOILS

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ABSTRACT. It has been found that degree of saturation in addition to the suction plays an important role in the stress-strain behaviour in unsaturated soils. This is important, because the occurrence of hydraulic hysteresis in the water retention behaviour means that degree of saturation and suction are not uniquely related. As a consequence, mechanical behaviour and water retention behaviour are coupled at a constitutive level. In order to incorporate this coupling, an elasto-plastic framework has been proposed by Wheeler, Sharma and Buisson (2003). These authors presented a single illustrative constitutive model for isotropic stress states, to determine, at a qualitative level, that key features of unsaturated soil behaviour can be represented by the new framework. Further work is however required, in order to fully validate the proposed framework and to refine the proposed constitutive model and extend it to non-isotropic stress states.
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