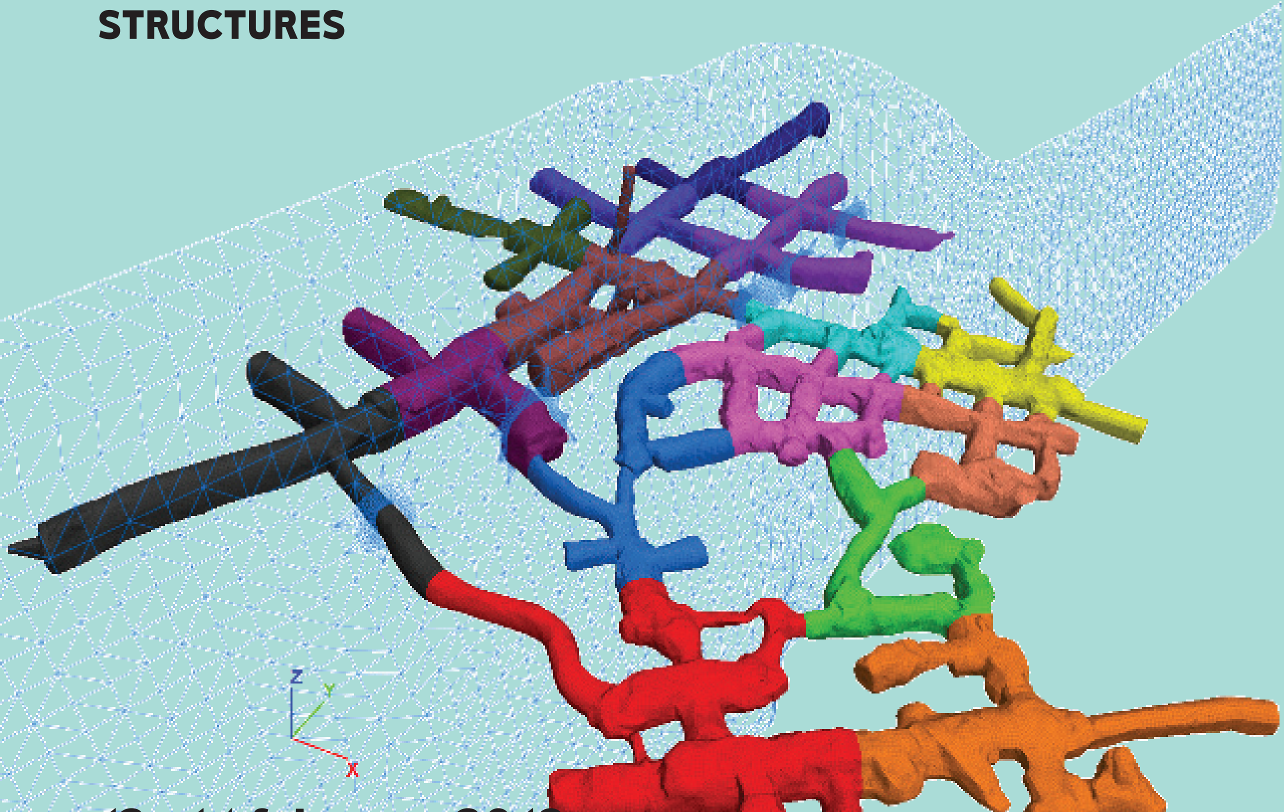
COGGUS²

COMPUTATIONAL & GEOENVIRONMENTAL GEOMECHANICS FOR UNDERGROUND AND SUBSURFACE STRUCTURES



12 - 14 february 2019
ARTEM, Nancy, FRANCE

INERIS
controlling risks
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Design & production: GeoRessources - january 2019

COGGUS^2

Computational & geoenvironmental geomechanics for underground and subsurface structures

Numerical modelling is getting more and more prominent in most geoengineering fields. It offers a unique way to study and better understand complex physical phenomena that must be forecasted facing new industrial projects and uses of the subsurface.

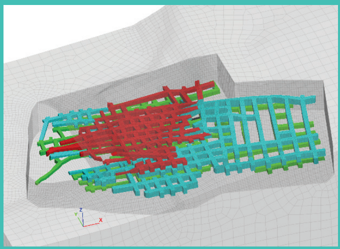
For underground structures, the macroscopic response of geomaterials, especially rocks, are strongly governed by coupled hydro-mechanical and chemical processes occurring at multiple scales. The behaviour of geomaterials involves several physics with different time and length scales. The behaviour at the macroscopic scale is governed by that of the micro scale level (pore scale). Underground structures are subjected to different time-dependent or independent physical loading and initial states: mineralogy, water-gas saturation degree, fluid pressure, temperature, mechanical loading (static, cyclic, dynamic), geochemical reactions, cracking, strains localisation and many other parameters and mechanisms.

One of the goal of Computational Geomechanics is to predict as accurately as possible the behaviour of underground structures in such frameworks.

In order to meet the challenge posed by the complexity of such boundary value problems, increasingly sophisticated numerical models are being developed.

The purpose of this Symposium is multifold. It aims first: to offer a comprehensive stock of the most recent advances in Computational Geomechanics, including both new attributes and current limits; second: to compare modelling methods and approaches; and last: to show the relevance of computation on both real cases and new projects.

The Symposium aims to address a wide range of issues, from the fundamental and theoretical points, to real case studies in many fields of underground operations.



On behalf of Ineris and GeoRessources Laboratory I am very pleased to announce the First International Symposium on " Computational & Geoenvironmental Geomechanics for Underground and subsurfaces Structures " (COGGUS^2) to be held in February 2019 in the beautiful historical town of Nancy, France.

Computational geomechanics is continuously challenged by the development of new ground and underground industrial operations with emerging questions, issues and potential risks. Computational geomodelling is a prominent way to improve understanding, anticipation and prevention.

My sincere hope is that this symposium gathers researchers, experts, engineers and practitioners all together with students from the community, offering a great opportunity to explore and discuss experiences and new directions.

In addition to the program of the symposium, a technical visit of the Andra URL, scholarships for selected young researchers and publication of selected papers in an international journal will be organized.

Note that Nancy offers many interesting visits and excursions among which a wonderful walk through the famous École de Nancy - Art Nouveau style buildings and the extraordinary group of architectural masterpieces inscribed on the UNESCO World Heritage List: Place Stanislas, Place de la Carrière and Place d'Alliance.

The Organizing Committee would be very pleased to help you with planning your visits. So be welcome to Nancy, looking forward to meeting you!

Farid Laouafa,

Symposium Chair

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COMMITEES

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WELCOME TO NANCY!

City of art and culture

The capital city of the dukes of Lorraine, Nancy lays claim to a rich cultural heritage. Due to its fine eighteenth-century architecture, the heart of the town has been declared a UNESCO World Heritage site, and, in the form of Place Stanislas, the city boasts one of the most beautiful town squares in the world. Nancy also played a seminal role in the Art Nouveau movement, and in the École de Nancy museum, visitors can view outstanding works by Gallé, Daum, Majorelle, Prouvé and Vallin.

Live entertainment, terraced bars, renowned and diverse restaurants - Nancy also offers all of the excitement you might associate with a thriving and energetic university town with more than 45,000 students!

The conferences will take place in campus ARTEM: Art, Technology & Management

ARTEM is an alliance between three of Nancy's Grandes Écoles: École Nationale Supérieure d'Art et de Design de Nancy, Mines Nancy and ICN Business School.

The three schools are joined by the city's international centre for research on materials and one of GeoRessources' team also works in campus ARTEM.

ARTEM is an ambitious project for education and research, founded on the opening up of new pathways between the different disciplines.

Tuesday February 12th 2019

8h30 - 9h30	Registration
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10h-11h	Keynote 1 A.P.S. Selvadurai 
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Thursday February 14th 2019

Technical visit of the MHM URL

Publications & authors

Hydro-mechanical study of nuclear waste geological repositories in Callovo-Oxfordian claystone. MAVL excavation and bentonite plugs
Argilaga (1), R. Charlier (1), M.N. Vub (2) and F. Collin (1)
(1) Université de Liège, ArGEnCo Department, Belgium
(2) ANDRA, Agence Nationale pour la gestion des Déchets Radioactifs, France

Coupled hydro-mechanical modeling of advancing tunnel in deep saturated ground
Simon Prassetyo(1) and Marte Gutierrez(2)
(1) Mining Engineering Program, Institut Teknologi Bandung, Bandung, Indonesia
(2) Civil and Environmental Engineering, Colorado, School of Mines, Golden, USA

Depletion-induced permeability loss and its effect on induced seismicity
Fryer (1), G. Siddiqi (2) and L. Laloui (1)
(1) Soil Mechanics Laboratory – Chair "Gaz Naturel"Petrosvibri, EPFL, Lausanne, Switzerland
(2) Swiss Federal Office of Energy, Bern, Switzerland

Deep tunnel fronts in cohesive soils under undrained conditions: application of a new front reinforcements design approach
Flessati and Claudio di Prisco
Politecnico di Milano, Milan, Italy

Hydro-mechanical numerical modelling of the opening and closing of desiccation cracks in claystone
Richard Giot, Stephen Hédan and Philippe Cosenza
UMR 7285 IC2MP, HydrASA, Université de Poitiers, CNRS, ENSI Poitiers, Poitiers, France

Stress evolution in deep tunnel concrete lining
V. Martyniak(2), F. Emeriault(1), R. Plassart(2) and F. Laigle(2)
(1) Univ. Grenoble Alpes, Laboratoire 3SR, 38000 Grenoble, France
(2) EDF-CIH, 73370 Le Bourget du Lac, France

Time-dependent reliability analysis of lined tunnel within the linear viscoelastic Burger rocks using the response surface method
Ngoc-Tuyen Tran(2) , Duc-Phi Do(1) , Dashnor Hoxha(1)
(1) Orleans University, LaMé Laboratory, Orléans, France
(2) Hatinh University, Hatinh, Vietnam

Approximate criterion for porous geomaterials having two populations of pores
Wanqing Shen(1) , Jianfu Shao(1) and Djimédo Kondo(2)
(1)Laboratory of Multiscale and Multiphysics Mechanics, University of Lille, CNRS FRE 2016, LaMcube, France
(2) Institut D'Alembert, UMR 7190 CNRS, UPMC (Paris 6), 75005 Paris, France

Long-term anisotropic hydro-viscoplastic modeling of a drift at the Meuse/Haute-Marne URL
D. Coarita-Tintayaa(1,2), M. Souley(1), M. N. Vu(3) and F. Golfier(2)
(1) Ineris, Nancy, France
(2) GeoRessources Laboratory, Nancy, France
(3) Andra R&D, Châtenay – Malabry, France

3D Discrete Element Modeling of Rock Cutting Experiments Under Confining Pressure
Nicolas Gonze, Fanny Descamps and Jean-Pierre Tshibangu
UMONS – University of Mons, Mons, Belgium

A coupled hydromechanical modelling of internal erosion around shield tunnel
Yang(1,2), Z.Y. Yin(2,3), F. Laouafa(1) and P.Y. Hicher(2)
(1) INERIS, Verneuil en Halatte, France
(2) Research Institute of Civil Engineering and Mechanics (GeM), UMR CNRS 6183, Ecole Centrale de Nantes, Nantes, France
(3) Department of Civil and Environmental Engineering, The Hong Kong Polytechnic University, Hong Kong, China

Cyclic response of Kaolin and Lower Rhine clay: Experimental analysis and Constitutive modeling
Merita Tafili and Theodoros Triantafyllidis
Institute of Soil Mechanics and Rock Mechanics, Karlsruhe, Germany

A fully coupled state-based peridynamic hydromechanical model for geomaterials
Shashank Menon(1) and Xiaoyu Song(2)
(1) University of Florida, Gainesville, Florida, USA
(2) University of Florida, Gainesville, Florida, USA

How artificial intelligence will mitigate natural risks?
F. Darve
Université Grenoble Alpes, Grenoble INP, 3sr lab

Crack propagation study in rock-like materials using coupled PD-FEM method
Yue Tong , Wan-Qing Shen, Jian-Fu Shao
Laboratory of Multiscale and Multiphysics Mechanics, University of Lille, CNRS FRE 2016, LaMcube, Lille, 59000, France

Assessing mechanisms of mechanical deformation to simulate two-phase flow in a swelling geomaterial
E.E. Dagher(1,2), T.S. Nguyen(1,2), J.A. Infante Sedano(2)
(1) Canadian Nuclear Safety Commission, Ottawa, ON, Canada
(2) Department of Civil Engineering, University of Ottawa, Ottawa, ON, Canada

Instability mechanisms of chalk mines in presence of water: feedback from the collapse of the Baulieu mine (France).
Renaud V.*, Cherkaoui A., Watelet J.M., and Gombert P.
INERIS, 60550 Verneuil-en-Halatte – France

Pressurized fluid flow within the mechanical stability domain of fault zones in shale
Frédéric-Victor Donzé(1), Alexandra Tsopela(1), Yves Guglielmi(2), Pierre Henry(3) and Claude Gout(4)
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(3) CEREGE, Aix-Marseille University–CNRS–IRD, Marseille, France
(4) Total S.A., Pau, France.

Modeling of damage with THM coupling by a phase field method and application to disposal of radioactive waste
Zhan YU(1), Jianfu SHAO(1), Darius SEYEDI(2)
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(2) ANDRA, Chatenay Malabry, France

Modeling primary fragmentation in cave mines using BBM (Bonded Block Mode ling)
Ghazal (1), C. Darcel(1), M. Fuenzalida (2) and M. Pierce(3)
(1) Itasca Consultants SAS, France
(2) Itasca Consulting Group, USA
(3) Pierce Engineering, USA

Coupled Hydro-Chemo-Mechanical Model for Fault Activation under Reactive Fluid Injection
POUYA(1) H. TOUNSI (1) and J. ROHMER(2)
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Publications & authors

Constitutive model development and field simulation of excavation damage in bedded argillaceous rocks
S. Nguyen(1) and Z. Li(2)
(1) Canadian Nuclear Safety Commission, Ottawa, Canada
(2) AIRY3D, Montreal, Canada

Strength of anisotropic shale rocks: Empirical nonlinear failure criterion and discrete element modeling analysis
Luc Scholtès(1) and Frédéric-Victor Donzé(2)
(1) Université de Lorraine, GeoRessources, Nancy, France
(2) Université Grenoble-Alpes, ISTerre, Grenoble, France

Tracking the origin of failure in granular materials: going down to the microscale
F. Nicot(1) and F. Darve(2)
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(2)Grenoble Alpes Université, L3SR, Grenoble, France

Hydro-mechanical modeling of granular soils considering internal erosion
Z.Y. Yin(2,3), J. Yang(1,2), F. Laouafa(1) and P.Y. Hicher(2)
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Excavation induced over pore pressure around a drift in Callovo-Oxfordian claystone
M-N. Vu(1), L.M. Guayacán-Carrillo(2) and G. Armand(2)
(1)Andra R&D, 92298 Chatenay-Malabry, France

Numerical modelling of thick-walled hollow cylinder tests on Boom Clay samples cored parallel and perpendicular to bedding
Peguiron and V. Labiouse
School of Engineering and Architecture of Fribourg HEIA-FR, HES-SO, University of Applied Sciences and Arts Western Switzerland

Mechanical and chemical closure of fractures
Blum(1), D. Vogler(2), J. O. Schwarz(3), F. Wendler(4), F. Enzmann(5), F. Amann(6),Pastewka(7) and T. Kling(1)
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Micromechanical and numerical modeling of porous geomaterials with a general plastic matrix
Cheng(1), S. Brach(2) and A. Giraud(1)
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Publications & authors

Propagating Uncertainties to Evaluate Confidence Intervals on the Transmission of Ground Movements
El Kahia,(2), M. Khouri(2), O. Deck(1), P. Rahme(2) and R. Mehdizadeh(1)
(1) GeoRessources Laboratory, Nancy, France
(2) Faculté de Génie, Branche II, Roumieh, Liban

Upscaling Theory for Coupled Hydro Mechanics in 3D Fractured Porous Rock (Application to Damaged Claystone around a Drift)
Rachid Ababou (1), Israel Cañamón (2), Adrien Poutrel (3), Ángel Udías (4)
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Stability assessment of an abandoned underground chalk quarry
Temenuga Georgieva, Fanny Descamps, Nicolas Gonze, Jean-Pierre Tshibangu
UMONS, FPMs, Mining Engineering Unit, Belgium

Thermal performance of an energy tunnel prototype: experimental and numerical investigations”
Marco BARLA, Alessandra INSANA, Fabrizio ZACCO
DISEG , Politecnico di Torino, Italy

Tuesday February 12th 2019

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11:15 - 11:30	M. Tafili et al. Cyclic response of Kaolin and Lower Rhine clay: Experimental analysis and constitutive modeling
11:30 - 11:45	E.E. Dagher et al. Assessing mechanisms of mechanical deformation to simulate two-phase flow in a swelling geomaterial
11:45 - 12:00	L. Cheng et al. Micromechanical and numerical modeling of porous geomaterials with a general plastic matrix
12:00 - 12:15	Y.Tong et al. Crack propagation study in rock-like materials using coupled PD-FEM method
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Tuesday February 12th 2019

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14:45 - 15:00	A. Pouya et al. Coupled hydro-chemo-mechanical model for fault activation under reactive fluid injection
15:00 - 15:15	F. Nicot et al. Tracking the origin of failure in granular materials: going down to the microscale
15:15 - 15:30	B. Fryer et al. Depletion-induced permeability loss and its effect on induced seismicity
15:30 - 15:45	P. Blum et al. Mechanical and chemical closure of fractures
15:45 - 16:00	F. Darve et al. How artificial intelligence will mitigate natural risks?
16:00 - 16:30	Coffee break
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16:45 - 17:00	D. Coarita-Tintayaa et al. Long-term anisotropic hydro-viscoplastic modeling of a drift at the Meuse/Haute-Marne URL
17:00 - 17:15	M.N. Vu et al. Excavation induced over pore pressure around a drift in Callovo-Oxfordian claystone
17:15 - 17:30	R. Ababou et al. Upscaling theory for coupled hydro mechanics in 3D fractured porous rock (Ap-plication to damaged claystone around a drift)
17:30 - 17:45	Z. Yu et al. Modeling of damage with THM coupling by a phase field method and application to disposal of radioactive waste
17:45 - 18:00	S. Nguyen et al. Constitutive model development and field simulation of excavation damage in bedded argillaceous rocks
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09:00 - 10:00	**Keynote 3 - R.I. Borja - Chairmen : A.P.S. Selvadurai & L. Laloui** On the strength of transversely isotropic rocks
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10:15 - 10:30	W. Shen et al. Approximate criterion for porous geomaterials having two populations of pores
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	SESSION 3b - Chairmen: R. Charlier & F. Nicot
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11:15 - 11:30	L. Flessati et al. Deep tunnel fronts in cohesive soils under undrained conditions: application of a new front reinforcements design approach
11:30 - 11:45	V. Martyniak et al. Stress evolution in deep tunnel concrete lining
11:45 - 12:00	Z.Y. Yin et al. Hydro-mechanical modeling of granular soils considering internal erosion
12:00 - 12:15	J. Yang et al. A coupled hydromechanical modelling of internal erosion around shield tunne
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Wednesday February 13th 2019

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14:45 - 15:00	F. Peguiron et al. Numerical modelling of thick-walled hollow cylinder tests on Boom Clay samples cored parallel and perpendicular to bedding
15:00 - 15:15	S. Prassetyo et al. Coupled hydro-mechanical modeling of advancing tunnel in deep saturated ground
15:15 - 15:30	M. Barla et al. Thermal performance of an energy tunnel prototype: experimental and numerical investigations
15:30 - 16:00	Coffee break
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16:15 - 16:30	T. Georgieva et al. Stability assessment of an abandoned underground chalk quarry
16:30 - 16:45	V. Renaud et al. Instability mechanisms of chalk mines in presence of water: feedback from the collapse of the Baulieu mine (France)
16:45 - 17:00	El Kahia et al. Propagating uncertainties to evaluate confidence intervals on the transmission of ground movements
17:00 - 17:15	X.P. Nguyen et al. Investigating the mechanical behavior of a surface repository for low and intermediate-level short-lived radioactive waste
17:15	CIGEO repository project by G. Armand (ANDRA)

TECHNICAL VISIT OF THE MHM URL

7h : Departure by bus from Nancy - Campus ARTEM
8h30 : ANDRA visit
13h15 : Lunch
14h30 : Departure by bus from ANDRA
16h : Back to Nancy

Andra is the French national radioactive waste management agency.
One of the major challenges of the experimental program undertaken by Andra to demonstrate the feasibility of a deep geological radioactive waste repository (500 – 600 meter depth) is to understand and to quantify the hydro-mechanical response of the Callovo-oxfordian claystone (sedimentary rock consisting mainly of argillaceous rocks, carbonates and silts) when excavated.
The objective of this program is to address two closely linked issues:
- the issues of excavation engineering (what kind of reinforcement support is necessary ?)
- safety issue (which impact has the claystone fracturing during excavation on the fluid flow ?).

The questions are addressed using complementary approaches:
- in situ experiments in the underground laboratory,
- laboratory studies of the mechanisms that may explain the hydro-mechanical behaviour of claystone, conducted at different scales (from nanometre to centimetre)
- numerical simulation.

Abstracts

Hydro-mechanical numerical modelling of the opening and closing of desiccation cracks in claystone

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Introduction

A gallery front at the Tournemire experimental station was monitored by optical metrology during several years, on a 35.5 x 27.5 cm² study area. It exhibited the presence of desiccation cracks sensitive to variations of relative humidity in the gallery: the cracks are closed in summer at 90 <RH <98% and open in winter when the relative humidity is lower. These sub-horizontal (parallel to the rock stratification) and sub-vertical (perpendicular to the stratification) cracks exhibit maximal aperture values ranging from 0.2 to 0.5 mm and different opening kinematics (Hédan et al. 2014; 2018). One possible reason for these differences in surface opening could be the different crack length within the rock mass which the experimental method doesn't allow to measure. In this work, we simultaneously study the extension and opening of desiccation cracks at the front of a gallery within a clay rock mass by numerical technique.

Results and discussion

Coupled hydro-mechanical finite element modellings allowed to reproduce the aperture of predefined cracks at the front of the gallery, for a given hydric loading. The numerical modellings in partially saturated conditions make it possible to impose on the gallery wall a water condition corresponding to the relative humidity measured in situ. To this aim, we converted the measured relative humidity into an imposed capillary pressure at the boundary of the model corresponding to the front of the gallery and the crack lips. The constitutive equations considered for the non-linear poroelastic model for partially saturated porous medium is the one described by Coussy (2004) and already considered for drying test at the laboratory scale (Giraud et al. 2009). In the numerical modellings, the crack penetration in the rock mass is fixed for each simulation. The approach consists in carrying out several numerical modellings with different initially fixed penetration. By fitting experimental and calculated crack aperture at the front of the gallery, it is thus possible to propose a penetration of cracks within the rock mass and to identify some rheological parameters of the clayey rock.
We focused on the aperture of three sub-horizontal cracks (figure 1). From the comparison of the measured and calculated openings, the penetrations of the cracks noted d12, d23 and d34 would be respectively of the order of 12 cm, 7 cm and 4 cm. These estimates are consistent with the results provided by geophysics (Cabrera et al., 1999; Okay et al, 2013). High-resolution seismic refraction and electric tomography surveys at the same site showed that the thickness of the damaged area on the walls of the gallery ranged from 15 to 35 cm.

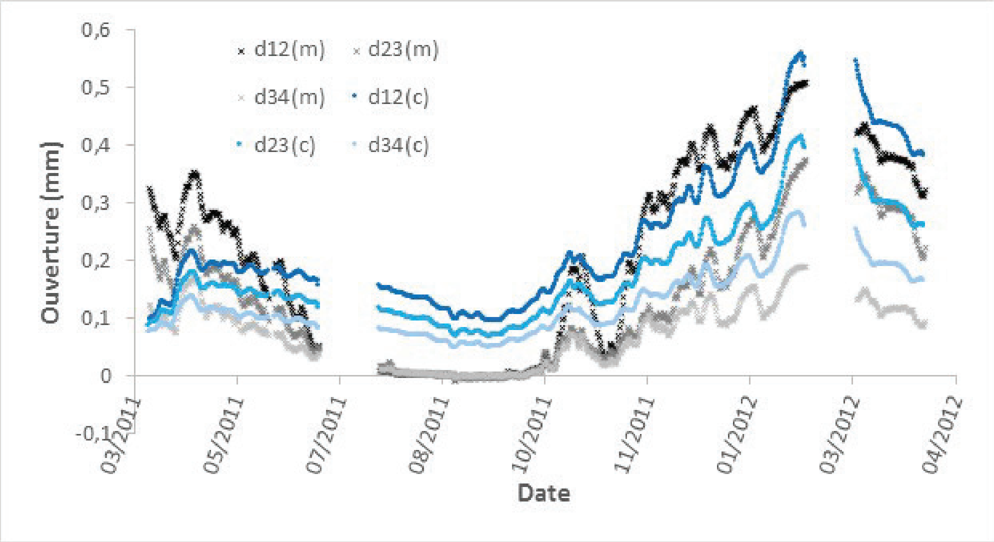

Fig. 1. Comparison of calculated (c) and measured (m) apertures

Conclusion

These first results remain quite qualitative, and many perspectives exist to improve the adjustment of the data measured in situ and calculated by numerical modeling, in order to improve the knowledge of the relationship between aperture and penetration. The current developments concern the extent to 3D numerical modelling accounting for clay anisotropy.

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Cyclic response of Kaolin and Lower Rhine clay: Experimental analysis and Constitutive modeling

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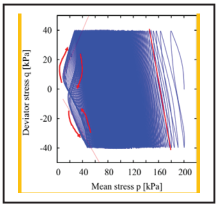

Introduction

Several undrained cyclic test series have been carried out by Wichtmann on a medium plasticity silt, namely Kaolin ($I_p = 12.2\%$), whereby the influences of consolidation pressure, overconsolidation ratio, displacement rate, sample cutting direction, stress amplitude, initial stress ratio and the control (stress vs. strain cycles) up to a failure criterion of axial strain of 10% have been studied. Thereby, various questions arised i.e. what happens with the last stress loops inclination and with the strain accumulation if a failure criterion of e.g. $\varepsilon_1 = 25\%$ is applied or how do the results differ when a high plasticity clay e.g. Lower Rhine clay ($I_p = 34\%$), is used instead. To fill this gap, the present paper collects a database of undrained triaxial cyclic tests on the medium plastiticity Kaolin (K) and the high plasticity Lower Rhine clay (LRC), comparing the different behaviour in terms of inherent and induced anisotropy and overconsolidation ratio (OCR).

For numerical analysis of the behaviour of geotechnical structures (e.g. foundations, damns, embankments) on saturated cohesive soils subjected to cyclic loading (e.g. caused by wind or waves in case of coastal or offshore structures, by traffic in case of infrastructure or by shear wave propagation during earthquakes), the application of enhanced constitutive models for the soil is indispensable. Therefore, the second part of this paper proposes an extension of the model developed by (Fuentes et al. and Tafili&Triantafyllidis) according to the experimental results shown in part one. The model is proposed under the platform of the ISA-plasticity (Fuentes&Triantafyllidis), which has shown good simulation results for the behaviour cyclic loading and captures well the small strain effects for sands. These advantages of the ISA-plasticity have been extended and adjusted to the mechanical behavior of clays. Additionally, various extensions capturing the inherent anisotropy i.e. the rotation of the (hypo)elastic stiffness and evolution of the anisotropic factor and the rate dependency i.e. through a third strain rate mechanism are introduced into the model. The loading surface is revised based on the experimental results shown herein.

The last part of the article presents some FE-simulations, which are carefully analyzed to discuss the models performance with respect to the rate dependency of the strain accumulation.

Citations, tables and figures

Table 1 Calibrated material parameters

Parameters	λ	κ	e_{i0}	ν_h	α	M_c	f_{b0}	I_v	m_r	R	β	χ_0	χ_{max}	C_α
K	0.13	0.05	1.76	0.3	2	0.88	1.5	0.015	5	10^{-4}	0.076	5	40	0.005
LRC	0.27	0.04	2.5	0.25	1	0.95	1.5	0.025	4	10^{-4}	0.1	7	30	0.007

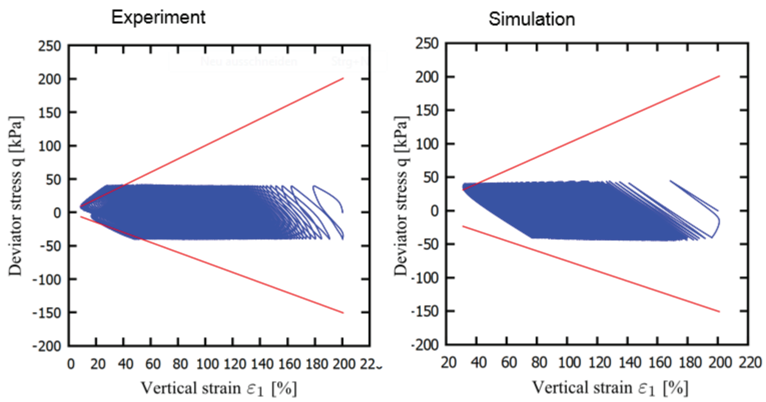

Fig. 1. Experiment and simulation of a cyclic undrained triaxial test on Kaolin with $q_{ampl} = 40$ kPa, $\dot{\varepsilon_1} = 0.1$ mm/min and N=858 cycles

Results and discussion

The Kaolin samples cut out in horizontal direction showed a higher stiffness and could sustain a much larger number of cycles to failure (Wichtmann&Triantafyllidis), whereby no difference have been encountered between horizontal and vertical cutting directions of Lower Rhine clay samples. As expected, the rate of pore pressure accumulation decreases with the OCR.
The proposed model is able to describe the material behavior of viscous and non viscous clays under cyclic as well as monotonic loading, capturing also the present inherent anisotropy of clays. The model covers a wide range of strain amplitudes without restriction on the OCR.

Conclusion

From the point of constitutive modelling, this is very interesting because the proposed model is able to simulate effects such as rate-dependency, creep, relaxation and the behavior of over- and normal consolidated clays under monotonic as well as cyclic loading without changing the model and keeping the same set of material parameters. All data of the present study may serve for the calibration or improvement of constitutive models concentrated on the cyclic behavior of cohesive soils developed by other researchers.

Acknowledgments

The financial support from the German Research Community (DFG TR 218/27-1) is gratefully acknowledged.

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Assessing mechanisms of mechanical deformation to simulate two-phase flow in a swelling geomaterial

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Introduction

The Canadian Nuclear Safety Commission (CNSC), Canada's nuclear regulator, conducts regulatory research in order to build independent knowledge on safety aspects related to the deep geological disposal of radioactive wastes. In a deep geological repository (DGR) for the long-term containment of radioactive waste, gas could be generated through a number of processes including the degradation of organic matter, radioactive decay of the waste, corrosion of metals producing hydrogen gas (H_2), and the radiolysis of water producing H_2. If gas production exceeds the containment capacity of the engineered barriers or host rock, these gases could migrate through the engineered barriers and/or the host rock.

The preferential migration pathway of these radioactive gases, to potentially expose people and the environment to radioactivity, might be through the access and ventilation shafts as these components are typically part of the repository design. Swelling soils, such as bentonite-based materials, are currently the preferred choice of seal materials used for those shafts. Understanding the long-term performance of these seals as barriers against gas migration is an important component in the design and long-term safety assessment of a DGR.

Modelling approach and experimental set-up

This study proposes a mathematical hydro-mechanical (HM) model for migration of gas (two-phase flow) through a low-permeable heterogeneous swelling geomaterial. It is based on a theoretical framework of poromechanics, applies Darcy's Law for both the porewater and poregas, and incorporates a modified Bishop's effective stress principle. The study expands upon the work by Dagher et al. (2017), by assessing two mechanisms of mechanical deformation in an attempt to simulate dilation-controlled gas flow; i) a linear elastic damage model and ii) a non-linear poro-elastoplastic model.

Using the Finite Element Method (FEM), the models were used to simulate 1D and 3D flow through a low-permeable swelling soil. The results were verified against experimental results found in the current literature for a confined cylindrical sample of near-saturated bentonite under a constant volume boundary stress condition (Harrington et al., 2017; Daniels and Harrington, 2017). Fig. 1 depicts an illustration of the experimental set-up for the 1D flow case and the 3D spherical flow case.

For the 1D flow test, gas was injected from one end of the bentonite sample at increasing pressures over time, while maintaining a water backpressure at the other end. For the 3D spherical flow test, gas was injected at the center of the bentonite sample, through an injection rod, while maintaining a specified water pressure at the radial porewater arrays. During the experiment, a number of key features were measured including the gas inflow and outflow, the pore-pressure at the pore-fluid arrays, and the total axial and radial stresses, and were compared to numerical results.

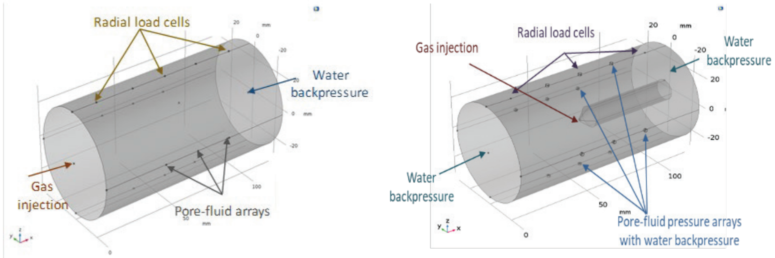

Fig. 1. Experimental set-up for the 1D flow (left) and 3D sperhicial flow (right) tests

Results and Conclusions

Results of the 1D case are presented in Fig. 2. For the 1D flow case, the linear elastic damage model with a swelling strain provides better representation overall of the experimental gas inflow, pore-fluid pressure, and total stress evolution profiles than the non-linear poro-elastoplastic model. Similarly, for the 3D spherical flow case, experimental inflows were better represented by the damage model. However, better representation of the shape and timing of the gas inflow, pore-fluid pressure and total stress evolution profiles were obtained using the poro-elastoplastic model. Neither model could simulate gas outflow. The results provide insight into the use of more complex models to simulate multi-phase flow in swelling geomaterials and provide further understanding of the mechanisms associated with such phenomena. Future studies will evaluate strain localization and multiple simulations of randomly distributed initial porosity to assess evolution of preferential pathways and dilation.

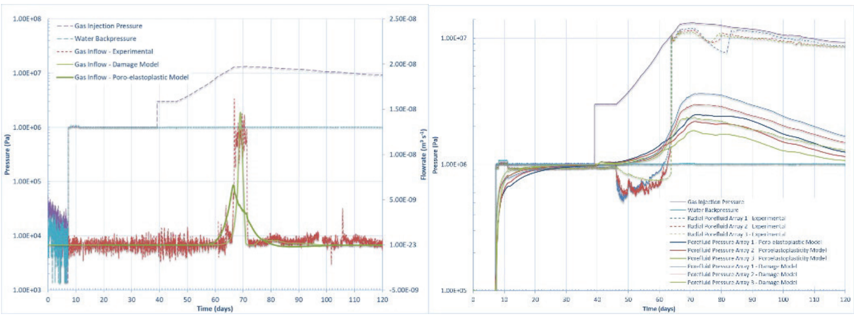

Fig. 2. Results of (a) gas inflow profiles (b) pore-pressure profiles for 1D flow case

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Micromechanical and numerical modeling of porous geomaterials with a general plastic matrix

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Introduction

In his famous paper, Gurson,1977 proposed an upper bound limit analysis approach of a hollow sphere and a hollow cylinder having a von Mises solid matrix. Several extensions of Gurson model have been further developed in literature. Considering the micromechanical modeling for applications to cohesive geomaterials, Guo et al., 2008 and Cheng et al., 2015 have both adopted the theoretical and numerical homogenization to take into account the plastic compressibility of the matrix respectively obeying the associated and non-associated Drucker-Prager yielding laws. However, the aforementioned models have been derived by assuming that the solid matrix obeys a yield criterion either depending on the second stress invariant (i.e. J2 plastic rule) or accounting for the first and the second ones. Nevertheless, the solid matrix of some engineering-relevant porous materials, especially that of geomaterials, may exhibit a more complex plastic behavior that also depends on the third stress invariant, that is on the stress-Lode-angle. Few attempts have been made in literature to include the influence of all the three isotropic stress invariants for describing strength properties of porous media. Mention can be made of the studies by Cheng et al., 2015, Lemarchand et al., 2015, Anoukou et al., 2016, Pastor et al., 2016. Some of these studied have been devoted to the special case of Mohr-Coulomb plastic matrix.

In the present work, we propose a numerical estimate of the macroscopic strength for ductile porous material by considering the local plastic behavior as dependent on all the three isotropic stress invariants and by referring to the case of axisymmetric strain-rate boundary conditions. In this light, the general and flexible yield criterion proposed by Bigoni et al., 2004 and recently particularized by Brach et al., 2018 is considered as a promising candidate to comply with benchmarking indications on local strength properties. This allows to effectively describe the limit behavior of a broad class of pressure-sensitive, frictional, ductile and cohesive bulk materials (e.g. geomaterials). Specifically, a Finite Element -based limit analysis procedure is implemented in order to compute the macroscopic yield surfaces. This allows to assess theoretical predictions and to compare to available numerical upper and lower bounds especially by paying particular attention to the matrix Lode angle effects.

Numerical homogenization

The local yield function considered in the matrix is assumed in the form (Brach et al., 2018):

$$F(\boldsymbol{\sigma}) = m(p) + \frac{q}{g(\theta_\sigma)}$$

with the mean stress, von-Mises equivalent stress, a linear meridian function $m(p)$ and a deviatoric dimensionless function $g(\theta_\sigma)$

$$p\frac{I_1^\sigma}{3}, \quad q = \sqrt{3J_2^\sigma}, \quad m(p) = -3\left(h - \frac{p}{\xi}\right), \quad g(\theta_\sigma) = \frac{1}{\cos[\frac{\pi}{6}\beta - \frac{1}{3}\arccos(\gamma cos3\theta_\sigma)]}$$

Where h, ξ, β, γ are material parameters, I_1^σ and J_2^σ respectively denote the first invariant of the stress tensor $\boldsymbol{\sigma}$ and the second invariant of the stress deviator $\boldsymbol{\sigma_d} = \boldsymbol{\sigma} - p\mathbf{1}$, and θ_σ represents the stress lode angle expressed in function of J_2^σ and the third invariant of $\boldsymbol{\sigma_d}$ denoted J_3^σ through $\cos(3\theta_\sigma) = 3\sqrt{3}J_3^\sigma/[2(J_2^\sigma)^{3/2}]$. It is worthy to note here that the parameter γ induces a smoothing effect on corners in the π plan, resulting in a shape transition of deviatoric profiles, that pass from a multi-sided polygonal shape to a circular one when γ is varied from 1 to 0. And the parameter β controls the tangency of the sharp or smooth vertices on the deviatoric π plan. Additionally, the present local yield function allows to retrieve a class of plastic strength criterions (e.g. Tresca, Mohr-Coulomb yielding, etc.). Readers can be referred to Brach et al. (2018) for more details. This general plastic model is implemented in a subroutine UMAT. Moreover, by adopting the axisymmetric hollow sphere geometry, the numerical homogenization is performed via the ABAQUS/Standard software and a macroscopic third stress invariant J_3^Σ sign depended subroutine MPCs (Multi-Points Constraints), which is devoted to imposing the displacement field on the external boundary of the hollow sphere such that the constraint of constant macroscopic stress triaxiality must be fulfilled for the numerical limit analysis computations.

It is shown in Fig.1 an example of the computational results realized in the case of porosity f = 10% and its comparison with the analytical model proposed by Anoukou et al., 2016 and the numerical bounds obtained in Pastor et al., 2016. The material parameters h = 0.5323, ξ = 5.1620, β = 0.6277 and γ = 1 are calculated accounting for the physical Mohr-Coulomb ones: friction angle ϕ = 20° and the cohesion $c = 1MPa$. Note that Σ_m is the macroscopic mean stress and $\Sigma_{gps} = \Sigma_\rho - \Sigma_z$ represents the scalar macroscopic stress deviator in cylindrical coordinates (ρ, θ, z). Excellent agreement can be observed especially between the numerical homogenization results and the upper and lower bounds. It is also worthy to note that the slight difference between them may be due to the approximation adopted in the treatment of the yield singularities in the π plan.

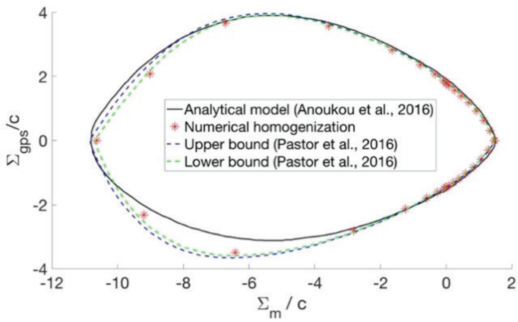

Fig. 1. Comparison of the numerical homogenization computations with the analytical macroscopic model (Anoukou et al., 2016) and the rigorous numerical bounds (Pastor et al., 2016)

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Crack propagation study in rock-like materials using coupled PD-FEM method

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Abstract

Peridynamics (PD) is introduced by Silling S A (2010) as a nonlocal theory to deal with discontinuous problems. It is formulated by integral equations instead of derivatives of displacement components. Hence, the basic mathematical structure of the formulation still makes sense whenever a discontinuity appears. This feature highlights crack initiation and propagation with arbitrary paths inside the material without special crack growth criteria. However, the PD method is computationally more demanding compared with methods based on classical continuum mechanics, such as Finite Element Method (FEM). On the other hand, it suffers some difficulties in dealing with boundary conditions. FEM, as a widely used numerical method, is effective for calculation without localized discontinuities. Therefore, it is of great significance to couple PD with FEM (PD-FEM) in a way that the potential region for crack propagation is modeled by PD theory. Based on the coupling strategy proposed by Galvanetto et al. (2016) and Zaccariotto et al (2018), this PD-FEM coupling method is implemented and its efficiency is proved by comparing with phase-field theory results (Molnár & Gravouil, 2017) with crack propagation problems in quasi brittle solids by several 2D examples. Furthermore, considering rock-like materials specific characteristics and corresponding loading conditions, this PD-FEM coupling method is extended to describe the elastoplastic damage mechanical behavior of geomaterials taking into account the crack initiation and propagation, as well as hydro-mechanical coupling. A series of numerical calculations are presented in this paper. It is found that the coupled PD-FEM method can be helpful to study crack propagation mechanisms in civil engineering structures.

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Pressurized fluid flow within the mechanical stability domain of fault zones in shale

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Introduction

Recent field studies suggest that taking into account subcritical mechanical response may be required to explain why the permeability of a fault zone in shales can increase by a factor of 10-100 (reaching the milli-Darcy range) when increasing the fluid pressure during injection tests, while strain records do exhibit generally very low plastic deformation [Guglielmi et al., 2015; Guglielmi et al., 2017]. This has been observed during fluid injection tests performed inside the clay-rich Tournemire fault zone. These injection tests were part of the "Fluids and Faults" project which was undertaken to constrain and better understand the relationship relating permeability, pressure, stress and strain in fault zones in shale for application to basin and reservoir modeling [Henry et al., 2016]. Based on the results obtained from this highly monitored project, we have set up a numerical model to investigate the hydromechanical conditions in which the flow rate and permeability increase dramatically depending on the fluid pressure while the mechanical response of the hydraulically loaded zones, exhibits a relatively minor irreversible strain response.

Tournemire in-situ experimental tests

A series of experiments using the SIMFIP Probe [Guglielmi et al., 2015] were performed within a fault zone in very low permeability (less than 10-19 m^2), in the lower Jurassic shale formations at the Tournemire Underground Research Laboratory (URL), France. The apparatus used for the fluid pressurization was composed of a probe used to monitor 3D displacement between two points anchored to the borehole walls at the same time as fluid pressure and flow rate. During the tests, the water was injected at a low pressure for a given duration. The injection pressure was incremented and held constant, typically for the same duration. This was repeated while monitoring the deformations across the fault with the 3D deformation probe. After reaching a threshold pressure, a large increase in flow rate at constant pressure could be observed. Several series of tests were carried out and in all of them the transmissivity-pressure dependency exhibited a substantial increase (more than 100 mL/min) only above a pressure threshold which has been called the Fault Opening Pressure (FOP) [Henry et al., 2016]

Numerical modeling

To reproduce this fluid propagation process from a geo-hydromechanical point of view, a discrete element model has been set up using the three-dimensional software 3DEC [Itasca C. G., 2015]. Models assuming homogeneous slipping plane could well reproduce data during the first steps of the pressurization tests associated to the hydraulic opening of the Tournermire tests (Figure 1), but they remain limited to explain the observed FOP associated to a low plastic response. The FOP concept associated to this limited mechanical response might be a consequence of the heterogeneous nature of the solicited rock volume. Taking into account the in-plane heterogeneities can explain the observed FOP with its increase of permeability and its limited slip motion while explaining explicitly the channelling process of the fluid.

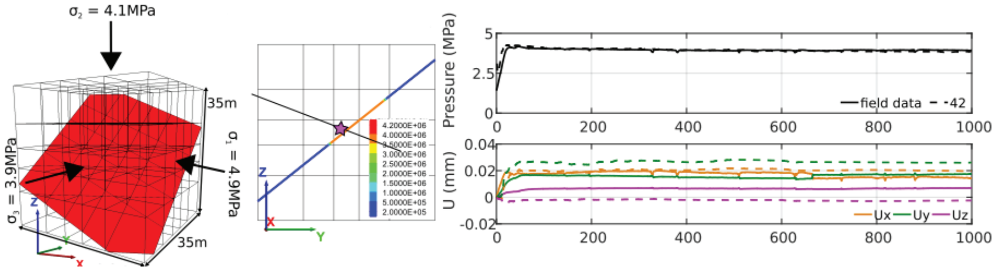

Figure 1: Left: 3D simple model, center: fluid pressure distribution along the fracture(s) in the z-y plane for t = 1000 seconds, right: fluid pressure and displacement variation versus time of the injection for the different configurations studied.

Conclusion

Several field observations suggest the existence of these flow paths in fault zones that transmit fluids rapidly over large distances and some of them have been observed to line up with regularly spaced pockmarks on the seabed [Gay et al., 2007]. These observations highlight the importance of weak zones along the faults, as suggested in the present model, thereby stressing the importance of focusing fault seal research based on geomechanical models good enough to provide accurate stress and pressure values.

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Coupled Hydro-Chemo-Mechanical Model for Fault Activation under Reactive Fluid Injection

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Introduction

The migration of fluid rich in CO2 in fractured rocks can cause processes such as mineral dissolution and precipitation and chemically induced weakening which affect the long-term mechanical stability as well as transport properties of fault systems. Some numerical approaches are already available in the literature for modelling of the dissolution/precipitation phenomena in fractures and modelling of subcritical crack propagation applied for instance to wellbore stability. Also the effect of chemical processes on the mechanical stability has been studied and modelled numerically in the framework of continuum materials and the context, in particular, of weathering in underground galleries (Ghabezloo & Pouya 2006).

From a practical point of view, the safety study of a CO_2 storage site consists, among others, in the evaluation of the sustainable injection pressure. A fault zone generally has a different permeability comparing to the neighboring rock matrix. If the fault zone runs across the reservoir and the cap rock, it may create a preference leakage path leading to the contamination of the aquifers located above the cap rock. Moreover, the decrease of effective stress in the fault, due to the fluid pressure, combined to chemical degradation of the fault materials may lead to the failure of the fault and induced seismicity.

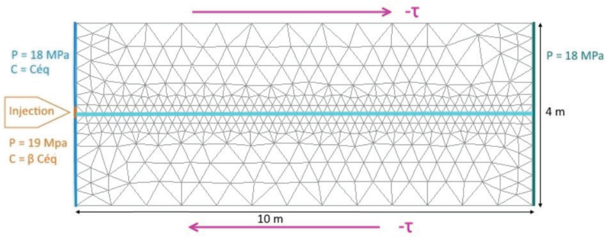
The effect of chemically active fluids flow has been studied in the context of long-term evolution of faulted regions of the crust (Pili et al. 1998). In the particular case of CO_2 geological storage, the chemical interactions between the CO_2 or CO_2 enriched brine and the fault zone increase the complexity of the problem. Although there is a lack of experimental data for the effects of pore fluid rich in CO_2 on rock properties, generally the dissolution due to reactive flow is supposed to increase the rock porosity and so decrease the rock strength. Some experimental data are available for the variation of rock strength and stiffness parameters with the porosity as a consequence of dissolution process (Bemer et al. 2004). However, in the context of fault systems, a complete numerical modelling of the stability evolution with the flow of a reactive fluid was not yet been done.

Results and discussion

In this paper a simplified but complete set of equations for a whole system of coupled hydro-chemo-mechanical process of reactive fluid flow inside a fault is presented. The chemical equation of calcite dissolution by a fluid which is unsaturated in Ca^{++} is written. This chemical reaction is coupled with fluid flow and the Fick law for diffusion. These equations have been implemented in Porofis, a FEM research code derived from the commercial code DISROC (2016) specially conceived for coupled processes in porous fractured media.

To model the chemically active fluid flow, unsaturated fluid in Ca^{++} is injected in a fault system. The concentration in $[Ca^{++}]$ increases in the fluid when flowing along the fault line due to dissolution of the fault filling material. At the same time, the porosity of this material increases and so its cohesion decreases.

To model the effect on the mechanical stability, a constant shear stress is applied on the fault. The shear stress value is chosen smaller than the initial cohesion of the filling material but greater than the degraded cohesion of this material. The simulation results show that after some duration of the injection process, ensuring some advance in the chemical processes, the shear displacement of the fault starts to be accelerated and tends to infinite values after a finite time. This shows the fault instability due to chemical processes.



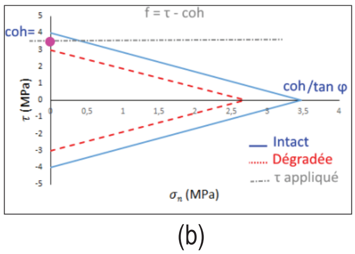


Fig. 1. (a) Fault model submitted to a constant shear and to the injection of an unsaturated reactive fluid, (b) Strength domain for the fault material before (blue line) and after chemical degradation (red)

Conclusion

To model the evolution of the mechanical stability in fault systems in presence of reactive fluid flow coupled mechanical, hydraulic and chemical processes were modelled numerically. Although the model was very simplified, it showed that the weakening of rock strength under dissolution processes can lead to fault activation after a finit time of fluid injection into the fault system.

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Tracking the origin of failure in granular materials: going down to the microscale

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Roughly speaking, granular media exhibit three basic scales: the specimen scale, the contact scale, and an intermediate scale made up of a set of adjoining particles. In this lecture, we will discuss this latter scale, in a two dimensional context. More specifically, the granular assembly can be regarded as a two phases medium. Grain column like patterns (force chains) develop within the medium, participating actively to its mechanical strength. These columns are surrounded by grain loops, made up of 3, 4, 5, or 6 grains (larger grain loops are much less frequent). According to the number of constituting grains, the mechanical properties of grain loops are very different [1]. In particular, 6 grains loops are prone to deform, contributing locally to a change in the void ratio. On the contrary, 3 grains loops deform just a little, but resist quite well to a deviatoric stress. According to the initial porosity of the assembly, and depending upon the loading path considered, the nature of grain loops surrounding force chains is versatile, with continuous transition mainly from 3 grains loops to 6 grains loops (or vice versa). This is a new route to investigate from a microstructural point of view why a granular assembly may be destabilized, leading to a localized or diffuse failure pattern [2-5]. In addition, these ingredients are shown to give rise to a microstructural interpretation of the so-called critical state, according to the failure mode taking place.

In 2D granular material, column-like structures, called force-chains (FCs), are formed as force transmission tunnels that carry a major external loading. The movement of FCs is highly correlated to the strength and the volumetric behavior of the granular matter. Since FC stability is ensured by the confining structures surrounding them, these confining structures must be quantitatively characterized to investigate the moving pattern of FCs and significant mechanical behaviors of granular materials. This study investigates two aspects: how the FC leads to heterogeneity on the originally relatively homogeneous meso-structures; and, the way the meso-structures affect the FC's behavior. The results show that the material is highly heterogeneous on the meso-scale, depending on the distance to the FC. The granular assembly area is split into two parts, with one consisting of adjacent meso-loops around FCs, called the force-chain loop (FCL), whereas the other loops are called not force-chain loops (NFCLs) (Fig. 1). The FC's surrounding area is observed to have greater dilatability than other areas, because it is better able to generate loose structures on the meso-scale [6-7]. Hence, the FC's adjacent areas can be seen as the main source of the global dilatancy (Fig. 2). In contrast, FC movability, a concept closely related to FC instability, is observed to depend to a large extent on the nature of its surrounding meso-structures. As a result, depending on the initial meso-structures, stress boundary conditions will direct dilatancy or contractancy. Finally, the life cycle of the FCs is presented in relation with their confining meso-structures.

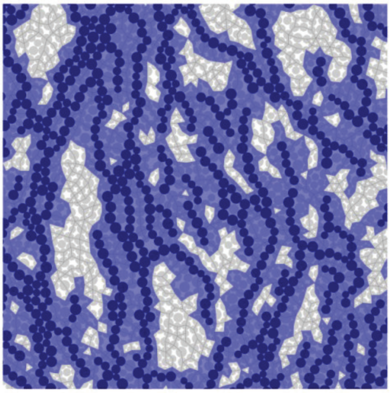

Fig. 1: An illustration of the FC loop (FCs are in dark blue; the remaining particles are grey; FCLs are highlighted by the transparent shadow).

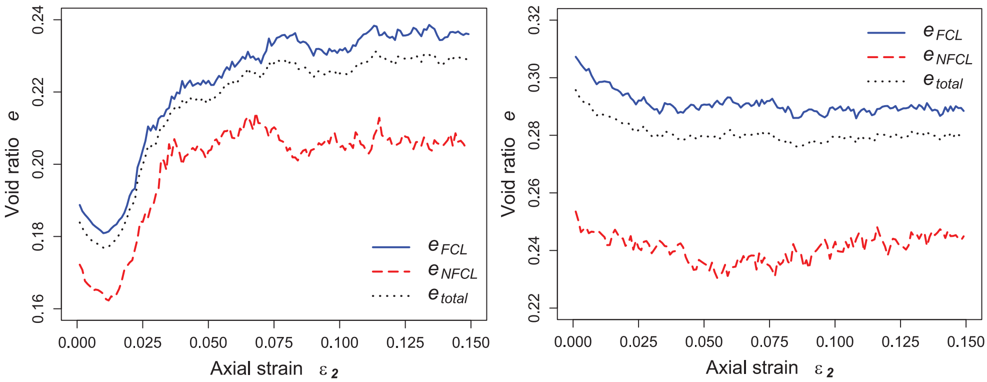

Fig. 2: The void ratio e of FCL, NFCL, and total area in (a) dense and (b) loose specimens as it evolves with respect to the axial strain ε_2

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Depletion-induced permeability loss and its effect on induced seismicity

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Introduction

It has been shown that production from subsurface reservoirs causes stress changes that can lead to induced seismicity (Segall 1989). In the fully-coupled theory of poroelasticity, these stress changes are caused by the gradient of pore pressure which acts as an internal force in the momentum balance equation. This implies that, if a larger pore pressure gradient is required to produce the fluid, larger stresses will be induced, likely leading to a higher seismicity rate.

Darcy's Law predicts that the gradient of pore pressure required to achieve a certain fluid production rate is inversely proportional to the permeability. The smaller the permeability, the larger the pore pressure gradient required. Therefore, the permeability reductions that are seen due to compaction during fluid production (Schutjens et al., 2004) may be significant in terms of induced seismicity because compaction will increase the pore pressure gradient required to produce a certain amount of fluid. For this reason, in this work, the effect of depletion-induced permeability loss on induced seismicity is investigated.

Methodology

Pore pressure and stress changes are modelled using a fully coupled Finite Volume flow and Finite Element mechanical model. Within the flow model, compaction-induced changes are accounted for by assuming a linear relationship between permeability and mean effective stress, an assumption which is valid for the near-elastic range of mean effective stresses present in the model (Schutjens et al., 2004). These pore pressures and stresses are then used in a seismicity model based on the model introduced by Dieterich 1994 and extended by Segall & Lu 2015. In this way, one can compare the seismicity rate with and without compaction-induced permeability loss in the reservoir and the surrounding formation.

Results and Discussion

The numerical experiment suggests that compaction-induced permeability loss does have the effect of increasing the stress changes associated with the production of a certain amount of fluid and thereby indirectly increases the seismicity rate. This effect is especially prevalent in strike-slip and reverse faulting stress regimes but also applies to normal faulting stress regimes. These results have implications for reservoir management during production (such as avoiding large drawdowns or utilizing water flooding for pressure management). It also highlights the importance of understanding how reservoir rocks compact as inelastic compaction may, in some cases, lead to permeability loss of an order of magnitude.

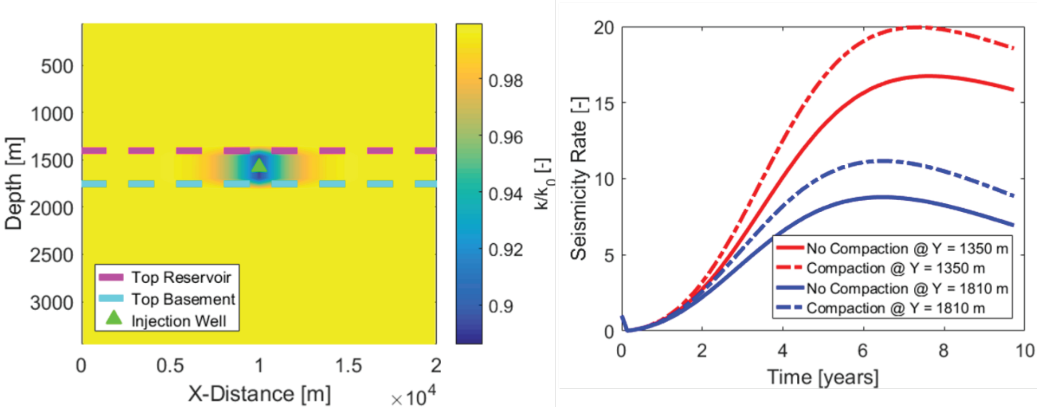

Fig. 1. Reverse faulting stress regime: (a) the permeability loss due to compaction after ten years of production from a horizontal well and (b) the resulting seismicity rates when this permeability loss is and is not accounted for. The seismicity rates are taken at various depths (Y) vertically in-line with the producing well. The simulation is plane strain with injection occurring via a horizontal well.

Acknowledgements

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Mechanical and chemical closure of fractures

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Abstract

In most geothermal or hydrocarbon reservoirs fractures are an important feature, which act as conduits or barriers and significantly co-determine if the reservoir is productive or not. Often reservoir models are based on discrete fracture network (DFN) models, whose performance however relies on reliable input parameters representing actual fracture properties. This study summarizes two state-of-the-art methods, which are able to reproduce mechanical and chemical fracture closure geometries and provide the basis to find representative fracture properties such as hydraulic apertures and permeabilities (Kling et al., 2017, 2018). A novel contact mechanical approach, which is implemented in a free web-app and accounts for pure elastic as well as elastic-plastic contact deformations, is introduced and validated for a tensile granodiorite fracture in order to simulate normal fracture closure. Furthermore, a phase-field model (PFM) for hydrothermally induced quartz growth is used to understand the effect of sealing structures on fracture hydraulics. It is found that, depending on the (mechanical or chemical) fracture problem, these methods can be used in different ways to estimate hydraulic properties of rock fractures. The results of the contact model indicate that particularly the elastic-plastic module of the web-app is able to reproduce experimentally derived normal closure with a root-mean-square error of 15 μm and provides a useful basis for subsequent fluid flow simulations. The PFM-based models show that fluid flow and hydraulic properties in sealing fractures significantly depend on the evolving crystal geometries. Consequently, a new equation to estimate hydraulic apertures is introduced, which factors in a geometry factor α for different crystal geometries ($\alpha = 2.5$ for needle quartz and $\alpha = 1.0$ for compact quartz). Thus, the three suggested methods are able to improve future DFN simulations.

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How artificial intelligence will mitigate natural risks?

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Introduction

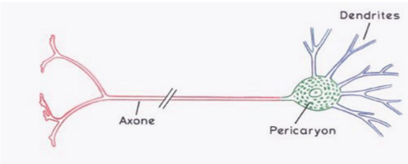
The new generation of in situ sensors (less expensive, more resistant, energetically more easily manageable) allows to disseminate them largely in structures, engineering works and natural media. For example, hundreds of kilometers of dykes can be equipped by optic fibers, landslides and cliffs by local sensors providing local displacements, interstitial water pressures, stresses , temperatures, … For a given period of time, thousands of measurements are obtained and constitute the so-called "big data". These enormous sets of values are impossible to be analysed by an human brain or even by usual numerical treatments. Clearly they contain a lot of discriminant informations with respect to the settlements of structures, the local and global failures of dams, dykes, slopes, cliffs, excavations, underground structures.

So a basic question emerges: how to analyse – if possible in real time – these big data coming from in situ measurements? An answer is today available thanks to the recent explosion of artificial intelligence capacities.

What is Artificial Intelligence?

Artificial intelligence is basically the construction of an artificial neuronal network and the implementation of a learning process from the given fact of a certain number of known solutions.

The initial idea of an artificial neuron is based on the reproduction of the biological neuron in the brain of living beings, schematically constituted by thousands of dendrites converging upon a nucleus out of which a single axon emerges (Fig.1).



Le neurone

Fig.1: schematic representation of a biological neuron of a living being

An artificial neuronal network is thus constituted by the nodes, disposed in layers and interconnected through links.

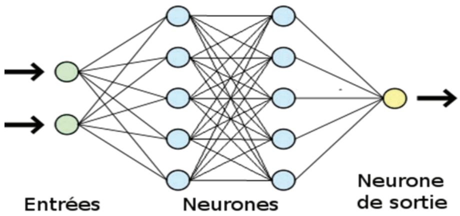


Fig.2: artificial neuronal network

At the origin of the development of artificial intelligence, the neuronal network was constituted by some dozens of nodes situated in several layers disposed on a plane. This method could have been considered as a simple method of interpolation between known data. Today, the "massic" neuronal networks consist of millions of nodes distributed in 3D and their creative extrapolation capacities have been established [1].
As far as the learning process of a neuronal network is concerned, it is necessary to introduce from the outset (see Fig.2) data for which the solutions at the outcome are known. The calibration of the network thus comes down to giving to different links the weights determined through the learning process by the mathematical method of retro-gradients.
This calibration ends up in an implicit definition of the hyper-surface passing through a set of points, of which the coordinates correspond to entries and exits. This hyper-surface then provides for the new data a unique determined exit if the surface is sufficiently regular [2].

Why Artificial Intelligence represents such a profound revolution?

Until the 1960's, the methodology applied to solve a "well-posed" problem consisted of formulations by a system of mathematical equations. Then, this system developed analytical methods to come up with a solution, specified by some analytical expressions.
The numerical revolution, which has become so important in engineering sciences from the end of the 1960's onward, has permitted extending in a remarkable manner the scale of the problems treated. Indeed, practically all the problems which can be formalized today by a system of equations can be solved numerically.
But, with artificial intelligence, a problem can be simply characterised by a discrete set of data with their associated responses; and this set can today be constituted by millions of data (as the "big data" evoked in the introduction).
Obviously, these algorithms of artificial intelligence encounter their limits; for example, when the hyper-surface, evoked previously, is not sufficiently regular which will lead to the inexistence of solution or to multiple solutions (bifurcation points of various types, …), or because the system studied is pathologically sensitive to initial conditions (meteorology, …)

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Hydro-mechanical study of nuclear waste geological repositories in Callovo-Oxfordian claystone. MAVL excavation and bentonite plugs

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Abstract

The disposal and storage of nuclear waste in geological repositories has been studied for some decades (OECD/NEA, 1995). Argillite and other argillaceous rocks are considered as suitable host rocks because of their ability to absorb radionuclides and low fluid and gas permeability. In France, the Callovo-Oxfordian argillite (COX) fulfils the requirements to host the geological repositories and it is the chosen geological stratum by the National Agency ANDRA.

The study of the COX as a host rock has been performed in Liège University since several years. The problem, involving strain localization, is not well posed when modelled using phenomenological theories combined with classical FEM codes; a microstructured model is proposed: local Second Gradient (Collin et al. 2006), in this approach the microstructure regularizes the problem with the introduction of an internal length, this allows to obtain an objective solution with respect to the mesh size.

At the moment, the model takes into account several anisotropies, e.g. cohesion (Pietruszczak et al. 2002), a visco-plastic model is retained to model the long term convergence of the excavation, a permeability evolution model allows to correlate strain localization and permeability increase (Pardoen et al. 2016), and finally a concrete hydration law allows to take into account the concrete hardening in the different structure layers.

The goal of this study is to model the long term behaviour of the geological repositories with a special focus on the EDZ extent, effect of ventilation and sustaining structure design. Results of the effective vertical stress of a quarter of the MAVL gallery (Fig. 1) show the stress level in the different parts of the sustaining structure and rock mass.

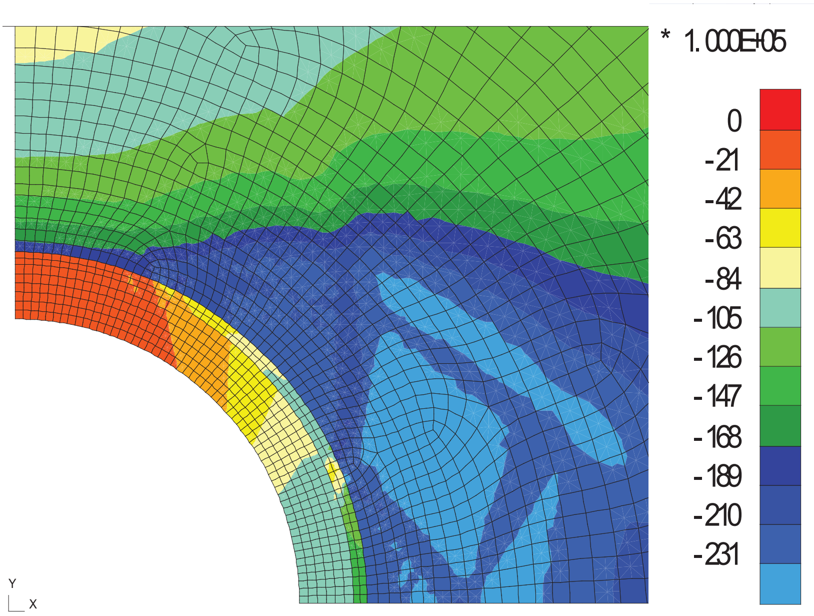

Figure 1: Vertical effective stress in the MAVL gallery (Pa).

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Long-term anisotropic hydro-viscoplastic modeling of a drift at the Meuse/Haute-Marne URL

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Introduction

In the context of radioactive waste management, one of the options currently being considered is to store it in deep geological formations. Clay formations, in particular, show very favorable confining conditions as repositories for long-term safety of nuclear waste due to their low hydraulic conductivity and significant retention capacity for radionuclides. In France, the National Radioactive Waste Management Agency (Andra) began to build the Meuse/Haute-Marne underground research laboratory (MHM-URL) in the Callovo-Oxfordian (COx) claystone formation, lying between depths of 420 m and 550 m. With regards to the main objectives of feasibility and safety assessment of a potential deep geological repository (Cigéo), a comprehensive scientific research programme of specific in situ experiments, laboratory tests and numerical modelling have been carried out to characterize and understand the behavior of the COx claystone formation in the framework of hydro-mechanical (HM) processes (Armand *et al.* 2017). In addition to the HM couplings and the associated parameters, it appears necessary to consider at least the inherent anisotropies (mechanical and hydraulic) of such rock, but also its time-dependent behavior (Armand *et al.* 2017). In this paper, we consider the transient creep mechanism determined from laboratory tests in order to study its effect on anisotropy and hydromechanical behavior of COx clay. The hydromechanical couplings are then evaluated around the GCS drift of MHM-URL (drift excavated following the major principal stress and without rigid lining) under satured conditions using Comsol Multiphysics code.

The long-term behavior of the COx claystone has been studied through laboratory creep tests on samples and in-situ experiments in MHM-URL (Armand *et al.* 2017). The laboratory tests did not demonstrate the existence of a viscoplastic threshold from which the creep mechanism occurs. Moreover, the measurements campaigns of the natural stress state performed at the MHM-URL show an anisotropic natural stress state. The absence of a measurable creep threshold stress at the laboratory scale means that the creep deformation of COx claystone starts even before excavation, which is unlikely from a physical point of view (Armand *et al.* 2017). Based on these observations, the viscoplastic strain rate that consider a threshold stress is assumed and expressed according to the power law:

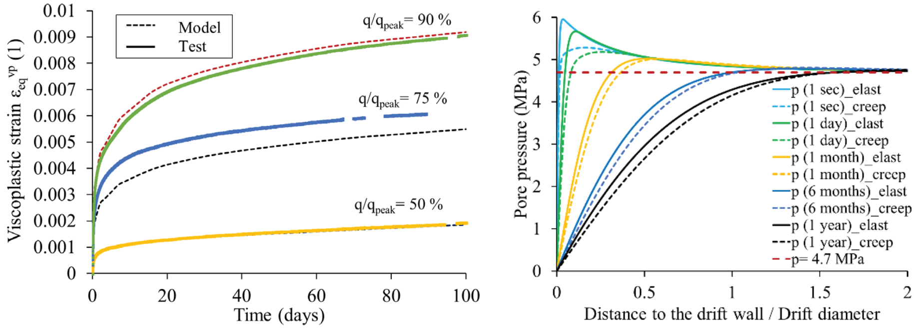
$$\dot{\varepsilon}_{ij}^{vp} = A_c \left(\frac{q - q_o}{\sigma_{ref}} \right)^{n_c} \frac{3}{2} \frac{\partial q}{\partial \sigma'_{ij}} \tag{1}$$

where A_c and n_c are coefficients of the power law, σ_{ref} is the reference stress, $q = \sqrt{3J_2}$ is the deviatoric stress, q_o is the initial deviatoric stress prior to any disturbance and J_2 is the second deviatoric stress invariant. The original implementation of power law in Comsol Multyphisics® does not assume a creep threshold (q_0 = 0).

The expression (1) reflects the stationary creep, i.e. that observed in the long term on in situ convergence measurements. In order to take account for the transient creep observed both on samples and in situ scales, the transient curve is then adjusted in pieces by using (1). Parameters relating to the long-term behavior of COx claystone are identified under the following assumptions: (a) the creep threshold is considered as null (q_o = 0), which is in agreement with most of laboratory tests, even under low stresses, (b) the parameters A_c and n_c of the power law are constants for $t \in [t_i\ ;\ t_e]$, where t_i is the initial time and t_e is the end time in the time interval considered and (c) the reference stress σ_{ref} = 1.0 MPa.
The equivalent viscoplastic strain for triaxial creep tests at a confining pressure of 12 MPa and three levels of stress at 50, 75 and 90% of the peak strength (36.5 MPa) were used to identify the parameters A_c and n_c. For the creep test (q_0 = 0), the deformation is expressed as:

$$\varepsilon_{eq}^{vp} = A_c \left(\frac{q}{\sigma_{ref}}\right)^{n_c} t \qquad (2)$$

In order to verify and validate the model implemented in Comsol Multiphysics®, three creep tests were simulated. A comparison between the tests and simulations (E= 4000 MPa and v=0.3) is shown in Fig. 1.a, with good agreement for 50 and 90% of peak strength, while for 75% it has a good trend.



(a) (b)

Fig. 1. (a) Tests and simulations of triaxial creep tests at stress levels of 50, 75 and 90% of peak strength. (b) Pore pressure profiles in horizontal direction (continue = poroelastic, dashed = poroviscoplastic).

The proposed model was then used to model the excavation of the GCS drift. The model parameters are found in Coarita-Tintaya *et al.* (2018). Comparison between poro-elasticity and poro-visco-plasticity modeling was made. Pore pressure evolution in the horizontal direction is particularly considered. As shown in Fig. 1.b, the creep behavior reduces the pore pressure due to the stress relaxation. The consideration of instantaneous plastic behavior is an ongoing work.

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Excavation induced over pore pressure around a drift in Callovo-Oxfordian claystone

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Introduction

Continuous monitoring around drifts excavated in the Meuse/Haute Marne Underground Research Laboratory (URL) of the French National Radioactive Waste Management Agency (Andra) has been performed in order to study the hydro mechanical behavior of the Callovo Oxfordian claystone. In such very low permeable rock mass ($k \approx 10^{-13}$ m/s) small deformation implies pore pressure change. During drift excavation an overpressure is observed before and after the passage of the face, followed by a drop of the pore pressure. The amplitudes of the pore pressure change in the ground depend on the drift orientation, the excavation rate, upon the distance to the face and to the drift wall. Moreover, influence of the face advance on pore pressure measurements is observed at a distance which is twice the distance of the influence observed in a purely mechanical measurement such as extensometer (Armand et al. 2017). This difference is linked with the fact that in a nearly undrained condition (due to the permeability of the rock) very small deformations which are difficult to measure through extensometer, induce pore pressure change which are easily measured. Moreover, Armand et al. 2014 show that excavation implies an induced fracture network around the opening, with shear and unloading fractures. The area with unloading fractures exhibits the highest hydraulic conductivity ($k>10^{-10}$ m/s, due to fracture transmissivity and not to the permeability of the rock matrix). Farther from the wall, the shear fractures exhibit low transmissivity which not affect a lot the average hydraulic conductivity.

Interpretation of the pore pressure variations observed during excavation is a challenging task. As shown recently by Guayacán-Carrillo et al. (2017), drift excavation in an anisotropic elastic rock with horizontal Young's modulus higher than the vertical one ($E_h>E_v$) shows an overpressure in the horizontal direction due to the volumetric strain in compression. However, the overpressure is localized very close to the wall and an anisotropy ratio $E_h/E_v > 2$ is required to reproduce the absolute value of measured overpressure. A similar observation is made when taking into account an anisotropic elastic and isotropic plastic behavior (Coarita-Tintaya et al. 2018).

In an attempt to better understand the processes explaining such pore pressure change during the drift excavation, new approach is proposed in this study. The main idea is to check the impact of the extension and shape of fracture zone around the drift (induced by drift excavation), as well as the anisotropic elasticity on the pore pressure evolution observed in-situ. An analysis is performed by means of a fully coupled hydro-mechanical finite element simulation, taking into account an anisotropic poro-elastic behavior for intact rock and a perfect poro-elasto-plastic behavior for the induced fractured zones.

Results and discussion

An analysis is performed taking into account the measurements performed in a drift excavated following the direction of major horizontal stress. In-situ measurements show that for drifts following this direction, even if the initial total stress state is quasi-isotropic in the plane of the drift section, the pore pressure evolution and the mechanical response are anisotropic. These observations indicate that the intrinsic anisotropy of the material plays a key role in the response of the rock formation. The influence of the induced fractured zone, observed in situ, on the pore pressure field around drifts is analyzed by introducing two predefined fractured zones (ZFC, ZFD) with different permeability in the model (Fig. 1). ZFC and ZFD are described by a Mohr-Coulomb perfect elasto-plastic behavior, whose parameters were calibrated based on in-situ experiments. The rock mass formation outside these zones is defined by an anisotropic elastic behavior. It should be noted that such elliptical damage zone conduct to an increase of the orthoradial stress at the ellipse major axis. Main results showed that the combination between the increase of fracture zone extension during excavation and the anisotropic elasticity of the intact rock allows a better description of the pore pressure evolution (Fig. 2). Those results give an insight that overpressure measured in situ are due to the combination of the elastic anisotropy of the rock mass and a “structural effect” induced by the extend of fracture zone.

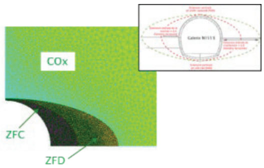
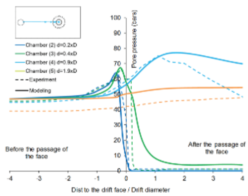


Fig. 1. Two-dimensional numerical model mesh. ZFC: connected fracture zone (mixed extensional and shear fractures); ZFD: discrete fracture zone (unconnected shear fractures)

Fig. 2. Pore pressure evolution in the horizontal direction of the drift cross section as a function of the distance to the face. Comparison between in-situ measurements and numerical results



Conclusion

The influence of the induced fracture zone by tunnel excavation, on the pore pressure evolution, has been analyzed by means of a coupled poro-elastic and poro-elasto-plastic model. It is shown that upon the determination of extension and shape of fractured zone, overpressures induced by the excavation can be well reproduced.

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Upscaling Theory for Coupled Hydro-Mechanics in 3D Fractured Porous Rock (Application to Damaged Claystone around a Drift)

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Introduction and summary

A methodology for upscaling 3D coupled Hydro-Mechanical (HM) properties of fractured porous rocks is developed theoretically, then tested on synthetic fractured rock samples, and finally, applied to a damaged and fractured claystone around a drift (the GMR gallery located in the 400 m to 600 m deep layer of Callovo-Oxfordian claystone at the Meuse/Haute-Marne Underground Research Laboratory operated by ANDRA). We focus on the Mechanical and coupled Hydro-Mechanical (HM) properties of fractured porous rock (Darcy permeability was upscaled separately in *Ababou, Cañamón Valera et al. 2011, JPCE*). In the application, the geometric structure of the EDZ (Excavation Damaged Zone) around the GMR gallery is described by a hybrid model comprising **(i)** a set of 10 000 statistical fissures with radially inhomogeneous statistics (size, thickness and density increase near the wall), and **(ii)** a deterministic set of large curves fractures ("chevrons"), periodically spaced along the axis of the gallery according to a 3D chevron pattern. The upscaled HM coefficients are obtained theoretically and calculated for different cases (synthetic samples, and application to the GMR drift). They include scalars, 2nd rank, and 4th rank tensors. The "3D" and "2D transverse" distributions of the upscaled coefficients are analyzed (displayed using spheres or ellipsoïds). Global values are obtained by upscaling over the entire damaged/fractured annular zone around the drift. Equivalent isotropic coefficients are also extracted from these tensors: Young modulus E, bulk modulus K, Lamé shear modulus μ, Poisson ratio ν, and HM coupling coefficients: Biot coefficient B (stress/pressure) and Biot modulus M (pressure / fluid production). Theoretical expressions are provided in the tensorial case, and in the simplified case of statistically isotropic cracks. The isotropic expressions are tested by running 3D "upscaling experiments" on synthetic Poissonian crack systems with isotropic orientations. The impact of fissuring/fracturing on equivalent stiffnesses and HM couplings is analyzed. References: *Cañamón, Ababou et al. 2019 (to appear in IJRMMS); Ababou, Cañamón et al. 2014 (Geol. Soc. London, Spec. Publi., v.400).*

Theory

The equivalent (upscaled) system of continuum equations involves the following macroscale tensorial laws and coefficients. The first law is the hydro-mechanical Hooke / Biot stress-strain-pressure law, called here "first Biot constitutive equation": $\sigma_{ij} = R_{ijkl}\Delta\varepsilon_{kl} - B_{ij}^{(I)}\Delta p$ where R_{ijkl} [Pa] is the stiffness tensor; $B_{ij}^{(I)}$ is the tensorial "Biot" coupling coefficient. Its reciprocal law expresses strain vs. stress and water pressure: $\varepsilon_{ij} = C_{ijkl}\Delta\sigma_{kl} + \overline{B}_{ij}\Delta p$, where C_{ijkl} [Pa^{-1}] is the compliance tensor, and $\bar{B}_{ij}$ [Pa^{-1}] is a "reciprocal" Biot coefficient representing strain/pressure coupling (also known as the poro-elastic expansion coefficient). The second law is the hydro-mechanical coupling (called here "2nd Biot constitutive equation"), which relates water production $\Delta\xi$ (increment of fluid content [m^3 of water / m^3 of clay rock]) to water pressure variation Δp and to global strain ε_{ij}, as follows: $\Delta\xi = B_{ij}^{(II)}\varepsilon_{ij} + \Delta p/M^{(II)}$, where $B_{ij}^{(II)}$ is a dimensionless tensorial "Biot" coupling coefficient (called here "2nd" Biot coefficient), and $M^{(II)}$ is the dimensionless Biot modulus (called here "2nd" Biot modulus. Concerning water transport ('Hydraulics'), the equivalent conductivity tensor K_{ij} was obtained earlier by upscaling based on flux superposition under fixed hydraulic gradient (*Ababou, Cañamón Valera et al. 2011, JPCE*). Thus, we may focus now strictly on M/HM processes.

The above-stated M/HM macroscale laws were obtained theoretically, in this work, by a method that accounts explicitly for the 3D geometric/statistical structure of the fractured porous rock. Briefly, the method is based on the superposition of matrix strain and crack displacements under fixed total stress, assuming a fluid-filled isotropic porous matrix, and fluid-filled cracks obeying Terzaghi or Biot elasticity. This theory generates in particular two Biot coefficients and two Biot moduli, due to the "dual" nature of the porous/fractured medium (*Cañamón, Ababou et al. 2019, to appear in IJRMMS).*

Sample results and discussion

Fig. 1 presents frontal views of locally upscaled 3D coefficients around the drift: $Bij^{(0)}$ (ellipsoïds) and $1/M^{(0)}$ (spheres) are shown in the first transverse layer (Y,Z) at axial position $X \in [-10.0$ m, -9.5 m$]$. The upscaling subdomains are cubes of side 0.5 m. Superscript (0) indicates that the plotted coefficients are calculated *relatively* to the coupling effects of fissures and fractures. The global coefficients of the EDZ around the drift were also isotropically upscaled. Comparing them with the intact rock matrix, it was concluded that the global Biot coefficient (B) increases with damage (while staying below unity as it should), and the global Biot modulus (M) decreases with damage. This is as expected physically, since damaging and fracturing should weaken the rock.

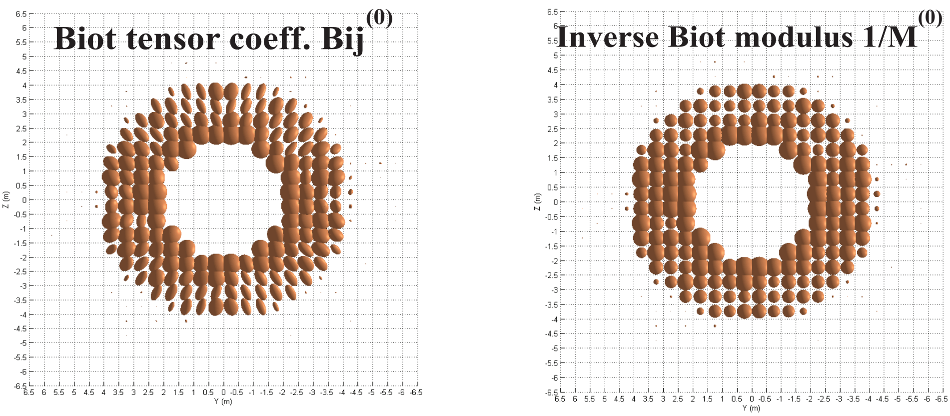

Fig. 1. Frontal views of 3D upscaled coefficients $Bij^{(0)}$ (ellipsoïds) and $1/M^{(0)}$ (spheres) around the drift.

Acknowledgements. This work has been supported by ANDRA, the French national agency for the management of radioactive waste, who provided the initial framework for the theory & application.

Modeling of damage with THM coupling by a phase field method and application to disposal of radioactive waste

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Abstract

Study of induced damage zone and its evolution is an important issue for the feasibility study of geological disposal of radioactive waste. In this context, coupled thermos-hydro-mechanical (THM) solicitations should be considered. In the paper, we present a new numerical method for studying damage field induced by excavation and heating and its application to Callovo-Oxfordian claystone.
Localized damage process is one of main physical processes leading to failure of quasi-brittle materials such as claystone. Various models have been developed for modeling of localized damage such as non-local damage models. In the present study, we propose to use a phase field method to capture localized damage process. As a difference with classical approaches, the damage field is described by independent governing equations and solved by a variational approach in the context of finite element method. The damage localization process is regularized with the help of a characteristic length. This method was initially developed for pure mechanical problems under tension dominated stresses. It is here extended to THM coupled problem. Further, for underground structures, compressive stresses constitute main loading paths. We have proposed a new damage evolution law considering respectively tension and shear induced micro-cracking. The damage growth is further influenced by fluid pressure evolution which is coupled with temperature change.as another important extension, the coupling between plastic deformation and damage process is also taken into account. Finally, the time-dependent deformation of claystone is considered.
The proposed method is first applied to laboratory tests under various loading conditions. Tested samples are studied as small structures subjected to initial and boundary conditions. The nucleation and growth of localized damage and plastic strain bands are properly investigated with the help of phase-field method. The transition from diffuse damage to localized failure is captured. The whole response before and after localization are reproduced.
The proposed method is then applied to study THM responses around an underground borehole subjected to heating process in the context of geological disposal of radioactive waste. The spatial and temporal fields of temperature, fluid pressure, displacement and damage are determined and compared with in situ measurements. Interactions between thermal, hydraulic and mechanical processes are discussed. The efficiency of the proposed numerical method is illustrated.

Constitutive model development and field simulation of excavation damage in bedded argillaceous rocks

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Introduction

Argillaceous rocks are candidate host and/or cap formations for the geological disposal of nuclear wastes in many countries, including Canada, France and Switzerland. The understanding of the long-term mechanical behaviour of such rocks is an essential requirement for the assessment of their performance as barriers against radionuclide migration. Using the laboratory test data on Opalinus Clay (Mont Terri, Switzerland) and Tournemire shale (France), we developed an anisotropic visco-elastoplastic constitutive model for these rocks and implemented it in finite element models to simulate the shape and extent of the excavation damaged zone (EDZ) around tunnels excavated in both rock types.

Constitutive model development

The constitutive model for the stress-strain behaviour of both the Tournemire Shale and the Opalinus Clay was developed within the framework of visco-elasto-plasticity (Nguyen and Le, 2015; Li et al. 2016). The presence of bedding in both rock formations results in three principal material directions, one perpendicular and two parallel to the bedding orientation. This inherent anisotropy was taken into account in the constitutive model, using the microstructure tensor approach (Pietruszczak and Mroz, 2000).The model was able to simulate the stress-strain behaviour found in laboratory triaxial and true triaxial tests. Modelling results for cyclic triaxial tests on Tournemire shale are shown in Figure 1 and show good agreement with the experimental data for all loading directions.

Simulation of EDZ at Mont Terri and Tournemire URLs

The EDZ and pore pressure evolutions around a micro-tunnel at the Mont Terri Underground Research Laboratory (URL) in Opalinus Clay, and a century old-tunnel at the Tournemire URL in Tournemire shale, are simulated by finite element models that coupled hydro-mechanical processes. The constitutive model, using the same parameters as those determined from the laboratory tests was implemented in the finite element models. It was found that the shape and extent of the EDZ at Mont Terri was well simulated (Fig. 2.a) and reflects the effects of stress re-orientation and material anisotropy. For the century-old tunnel at Tournemire, the effects of the fault and dessication due to seasonal temperature and relative humidity variations have to be additionally taken into account. This resulted in a simulated arrow-head shaped EDZ which closely resembles the one characterized from the field data (Fig. 2.b).

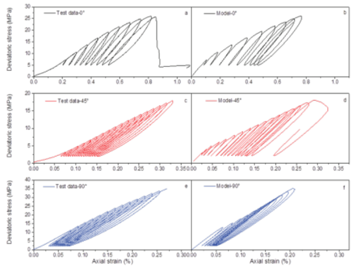
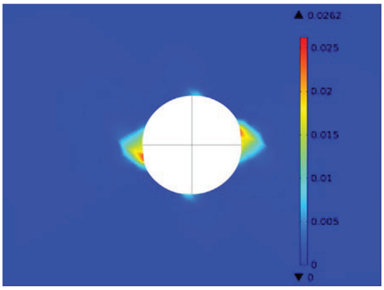
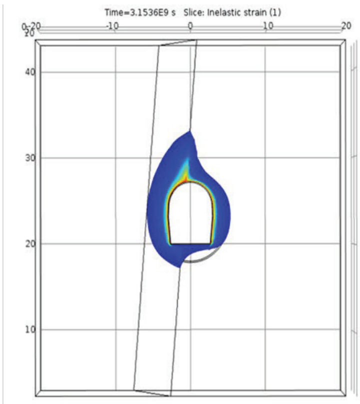


Figure 1. Comparison of modelled and experimental stress-strain behaviours of Tournemire shale under cyclic axial loading at different bedding orientations:



a) Microtunnel at Mont-Terri



b) Century-old tunnel at Tournemire

Figure 2. Modelled EDZ at Mont Terri and Tournemire URLs

Conclusion

EDZ around openings in deep geological repositories (DGR) for radioactive waste might constitute preferential pathways for radionuclide migration. The extent and characteristics of the EDZ must therefore be assessed. For argillaceous rocks, the EDZ assessment should take into account inherent anisotropy due to the sedimentation process, weathering and dessication which can occur during the operational period of a DGR.

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Strength of anisotropic shale rocks: Empirical nonlinear failure criterion and discrete element modeling analysis

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Introduction

Predicting the strength of shale rocks is a basic but nonetheless critical requirement when designing CO2 or nuclear wastes storages. Even though geologic structures usually need to be considered above the meter scale when evaluating the strength of such systems, the intact rock behavior still has to be thoroughly characterized before any further analysis. A particular feature of shale rocks is that they present nonlinear failure envelopes which tend to saturate at high confining pressures and, interestingly, despite this unanimously observed behavior, the linear Mohr-Coulomb criterion is still widely used in engineering practice. Empirical nonlinear formulations like the ones proposed by Hoek and Brown or by Singh (2015) exist and should thus be considered more systematically when dealing with shale rocks. Nonetheless, as practical as they are to assess the integrity of geologic structures, these formulations do not explain the underlying mechanisms involved in the failure processes. For instance, shale rocks generally present anisotropic structures, characterized by laminations, parallel layering or bedding features. These fabric properties induce a strong anisotropic behavior which directly controls the strength of shale rocks. It is thus crucial to consider such characteristics when formulating a failure criterion so as to better understand the associated failure mechanisms.

In this work, we propose to study the strength properties of Tournemire shale based on comprehensive experimental laboratory test results, so as to emphasize the relevance of Singh's failure criterion. Then, a transversely isotropic discrete element model is utilized to discuss how the fabric of such material contributes to the strength variation as a function of the material orientation with respect to the loading direction.

Results and discussion

Experimental triaxial tests carried out by Bonnelye et al. (2017) on Tournemire shale have shown that the material orientation has a strong impact on its strength. A nonlinear failure envelope with saturation at high confining pressures has also been observed. As illustrated on Figure 1, a nonlinear failure criterion based on the critical stress concept such as the one proposed by Singh appears to be more relevant than the conventional linear Mohr–Coulomb criterion to describe the strength of Tournemire shale. Experimental post-mortem sample analyses showed that the way fracture develops in Tournemire shale depends on the orientation of the bedding with respect to the loading direction. When the bedding is perpendicular to the loading (β = 90°), strain localizes across the bedding planes at all confining pressures. When the bedding is parallel to the loading (β = 0°), fractures propagate along the bedding planes at low confining pressures while strain localizes through the bedding planes when the confining pressure increases. Finally, when the bedding is inclined at 45°with respect to the loading direction, deformation localizes along the bedding as it is aligned with the maximum shear stress. Numerical simulations performed with a transversely isotropic discrete element model show exactly the same trends, supporting therefore the assumption that in such materials, the lamination scale anisotropy and heterogeneity cannot be ignored when dealing with the integrity of the corresponding geologic systems

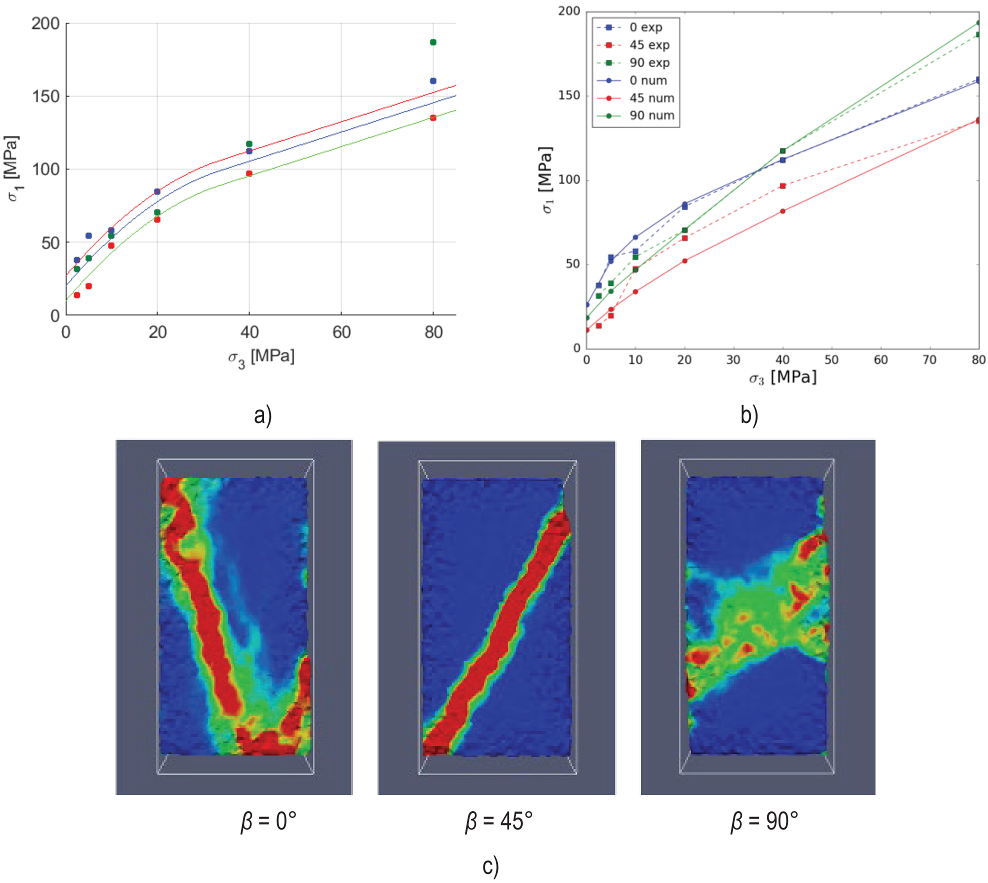

Fig. 1. a) Singh criterion applied to Tournemire Shale rock fitting the experimental dataset from Bonnelye et al. (2017) b) Numerical (calibrated discrete element model) and experimental failure envelopes. c) Failure patterns predicted by the discrete element model for a confining pressure of 20 MPa (shear strain distribution maps).

Acknowledgments

The experimental results from Bonnelye et al. (2017) were obtained as part of the "Fluids and Faults" project funded by TOTAL.

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Approximate criterion for porous geomaterials having two populations of pores

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Abstract:

Pore is one of the common characteristics of geomaterials, which affects strongly the strength of the studied porous materials. Since the pioneer work of Gurson (1977) in the framework of limit analysis approach, the derivation of the macroscopic criterion of ductile porous materials has been widely investigated, for example, extending the Gurson's model to geomaterials by considering a Drucker-Prager or Mises–Schleicher type matrix; taking into account the influence of pore shapes on the macroscopic mechanical behavior by replacing the spherical pore by spheroidal one; accounting the pore size in the macroscopic criterion, and so on. This study focuses on a class of porous geomaterials (like chalks, clayey rocks, cement-based materials) having two populations of pores which are well separated by two different scales. The pores at the mesoscale are spheroidal which are randomly oriented and distributed and the ones at the microscale are spherical. The solid phase at the microscale is assumed to obey the Drucker-Prager type criterion to account for the pressure sensitivity. By using a kinematics limit analysis theory and in the framework of a two-step homogenization procedure, a closed-form expression of the macroscopic criterion for the studied double porosity material are derived which combines the effects of solid phase plastic volumetric strain and the influences of pore shape and distribution. The special case of penny-shaped cracks has also been included in the analysis. For validation purpose, the new derived criterion is assessed and validated by comparing its predictions to available upper bounds and numerical data from literature. Then, the work is extended to saturated double porous saturated media with different pore pressures at the microscopic pores and at the mesoscopic ones.

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Time-dependent reliability analysis of lined tunnel within the linear viscoelastic Burger rocks using the response surface method

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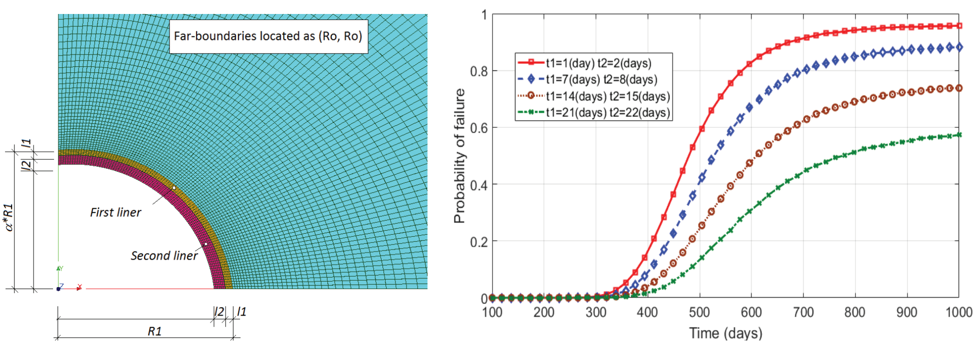
Abstract

This work concerns with the stability analysis of the deep tunnel excavated in the linear viscoelastic Burger rocks. For this purpose, the time-dependent reliability based on the response surface method (RSM) is chosen. The sequential excavation, and the installation in the sequence of the two liners of the tunnel, is taken into account in these simulations. The validation of the numerical simulation is conducted in the first step by comparing with our new closed-form solution in a particular case of the circular tunnel excavated the viscoelastic Burger rock and in the isotropic initial stress state. In the second step, through a series of numerical simulations conducted in Code_Aster, we highlight the influence of different parameters on the time-dependent stability of tunnels and their supports. Follow that, the probability of failure versus the service lifetime of the tunnel depends on the uncertainty of the viscoelastic properties of the rock mass, the sequential installation time of the two liners as well as their thickness. With respect to the particular case that the Monte Carlo simulation is conducted by using the closed form solution, this numerical reliability analysis based on the RSM offers the possibility to consider some other parameters like the anisotropic initial stress state and the elliptical geometry of the tunnel. The results presented in this study confirm the applicability of the RSM in the time-dependent reliability analysis of tunnel as well as in the supports design.



(a) (b)

Numerical model of elliptical tunnel supported by two liners in the viscoelastic Burger rock with anisotropic initial stress state (a), time-dependent probability of failure with respect to installation time of two liners (b)

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Deep tunnel fronts in cohesive soils under undrained conditions: application of a new front reinforcements design approach

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Tunnel fronts excavated under difficult ground conditions are commonly supported either by improving the soil mechanical properties or by introducing linear inclusions. This last technique is particularly popular in tunnel engineering since is the reinforcement distribution is simply to be adapted to the different conditions encountered during the excavation.
In this paper, the authors present a practical application of a new displacement-based approach to design the front reinforcements.

Introduction

In conventional tunnelling, a technique commonly adopted to reinforce the front consists in inserting in the advance core a series of linear inclusions grouted in drilled boreholes. In the current engineering practice, this expensive and time-consuming reinforcement technique is designed by employing simplified approaches (e.g. Grasso (1989), Anagnostou & Perazzelli (2015) and Perazzelli & Anagnostou (2017)) disregarding the front displacements. Recently, di Prisco et al. (2018b) proposed a new approach, based on a non-dimensional front characteristic curve, explicitly taking into account the displacements as a design variable. In this paper, this approach will be briefly presented and a practical application will be discussed.

Non-dimensional reinforced front characteristic curve

According to di Prisco et al. (2018a), the employment of two non-dimensional variables, one for the displacements (q_f) and one for the stress (Q_f), is particularly convenient for the description of the front characteristic curve, relating this latter the average front displacements (u_f) and the average horizontal stress applied on the front (σ_f). These two variables are respectively defined as:

$$q_f = \frac{u_f}{u_{fr,el}} \frac{\sigma_{f0}}{S_u}, \qquad 1.$$

$$Q_f = \left(1 - \frac{\sigma_f}{\sigma_{f0}}\right) \frac{\sigma_{f0}}{S_u}, \qquad 2.$$

where σ_{f0} is the initial (geostatic) value of σ_f, S_u the undrained strength and $u_{fr,el}$ is the residual elastic displacement (corresponding to σ_f=0). In the Q_f-q_f plane, the front characteristic curve for an unreinforced front does not depend on the system geometry, soil mechanical properties and initial state of stress (di Prisco et al. (2018a)). Its shape is only a function of the initial total stress anisotropy factor $\bar{k}$.

In the case of reinforced fronts, the same authors (di Prisco et al. (2018b)) have shown in the light of the results of a very wide non-linear 3D FEM numerical campaign, that the characteristic curve can be written as:

$$q_f = \begin{cases} \dfrac{Q_f}{1+\Delta R} & Q_f < a_f \\ \dfrac{a_f}{1+\Delta R} e^{\frac{Q_f}{a_f}-1} & Q_f \geq a_f \end{cases}, \qquad 3.$$

where a_f is defined as:

$$a_f = \frac{a_{fu}+\Delta Q_f}{1+\Delta R}. \qquad 4.$$

a_{fu} is related to $\bar{k}$ (Figure 1), whereas both ΔQ_f and ΔR are related with the reinforcement length (L), number (n), diameter (d) and stiffness (E_r):

$$\Delta Q_f = \eta \frac{ndL}{D^2}, \qquad 5a.$$

$$\Delta R = \left(r_1 \frac{L}{D}\right)\left(1 - e^{-r_2 \frac{nd}{D}}\right)\left(\frac{1}{2} + \frac{1}{2}\,\frac{r_3 \frac{E_r d^2}{E_u L^2} - 1}{r_3 \frac{E_r d^2}{E_u L^2} + 1}\right), \qquad 5b.$$

being D and E_u the tunnel diameter and the soil elastic undrained Young modulus, respectively. The dimensionless parameters η and r_i (i=1,3) values are reported in Table 1

Table 1 parameters of ΔQ_f and ΔR

η	r_1	r_2	r_3
0.8	0.47	2	80

Practical employment

In this section, by considering a reference case (D=12m, tunnel cover H=72m, soil saturated unit weight 20kN/m^3, $\bar{k}$=0.8, S_u=250kPa, E_u=25MPa), the use of Equations 3-5 is exemplified.

The designer defines initially the target front displacement (here supposed to be of 30cm). Once L, d and E_r are assigned (these variables are imposed by the manufacturer) the unique design variable is n. By substituting for L, d and E_r the following values: 18m, 12cm and 30GPa, respectively, and by employing Equations 3-5, the dependence of the residual front displacement u_{fr} (the front displacement corresponding to σ_f=0) on n plotted in Figure 2 is obtained. As is evident, by employing n>105, the front displacement is lower than the target value (dashed line).

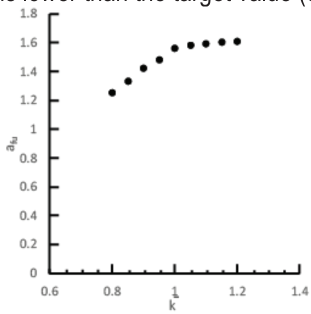

Fig. 1. relationship between a_{fu} and $\bar{k}$

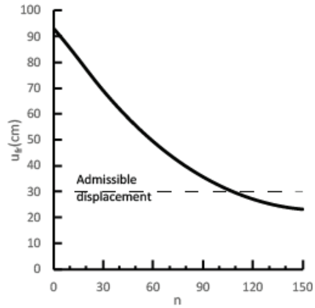

Fig. 2 variation of u_{fr} with n

Concluding remarks

In this paper, the authors presented a displacement based approach to design the front reinforcements. This approach is applied to a reference case to estimate the minimum reinforcement number necessary to obtain an admissible front displacement.

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STRESS EVOLUTION IN DEEP TUNNEL CONCRETE LINING

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Introduction

Estimating the stress evolution in deep underground structure lining remains challenging. Even if the behavior of the ground is accounted for with complex viscous models calibrated on laboratory samples, the stress in the concrete lining after a long period of time (more than 20 years) can be overestimated by a factor of 2 to 3. The main source of error lies in the actual combination of both complex behaviors of ground and concrete with time, leading to strong soil-structure interaction phenomenon. The influence of several aspects of the behavior of concrete on the evolution of stress in the lining have been analyzed, trying to determine which one has the most prominent effect depending on the structure geometry and construction methodology. The paper presents the effect of creep which appears as having the most significant effect.

Methodology

The effect of creep in concrete is not the same depending on the construction method, two different reference cases have therefore been considered, representing the most common situations in deep underground structures. The surrounding rock mass is also assumed to exhibit time effect. The first case represents a horseshoe shaped tunnel excavated with a sequential method. The second one is a circular tunnel excavated with a TBM. In order to estimate the influence of the concrete creep on the stress evolution, three levels of complexity have been considered. A preliminary semi-analytical method based on convergence-confinement [1] has been first developed. A Singh-Mitchell [2] model has been introduced to take into account the creep of the ground and a Burger model for the concrete creep based on the concepts of Eurocode 2. Then, 2D numerical models (FLAC2D) with a visco-plastic model (L&K [3]) have been defined to study more precisely the behavior of the lining in the two reference configurations. In this approach, the two different cases still represent the behavior of the lining with a Burger model. Finally Code_Aster with LKR [4] model for the ground has been used to take into account more complex creep mechanisms.

Results and discussion

The stress evolution rates measured in various deep galleries (St-Martin-de-la-Porte, Bure Laboratory…) always seems to evolve with a t^{-m} form, and the Singh-Mitchell [2] model proposes a visco-elastic strain following this function. The results of the semi-analytical method (convergence-confinement) using this model shows that creep in concrete can reduce significantly the stress level (around 20% after 3 years in Figure.1). This result depends on the distance between the excavation face and the installation of lining, the type of concrete, and geometrical characteristics of the gallery. The evolution of the stress rates fit quite well the measured data for various deep galleries.
Figure 2 shows the results of simulation using L&K model for the ground and Burger model for the concrete in FLAC 2D simulations. Creep reduces the average stress in the lining by 15% also for 3 years, with a great effect on the local stress concentration that occurs in the corners of the section.

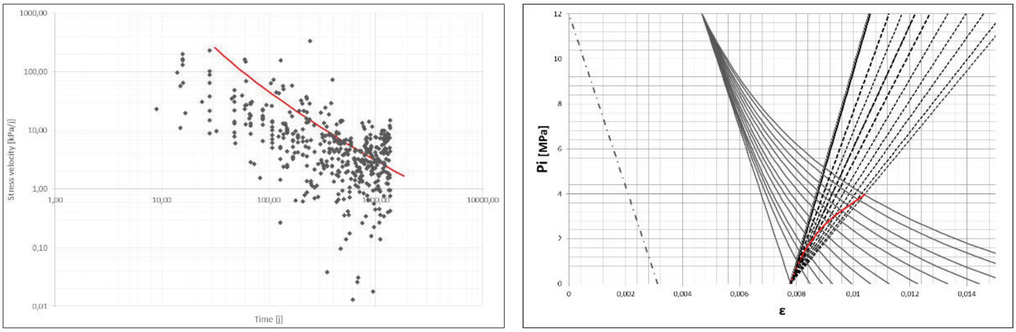

Fig. 1. Evolution of stress rate measured in the lining for different underground structures [5] [6] (left) convergence-confinement method using Singh Mitchell and concrete creep (right)

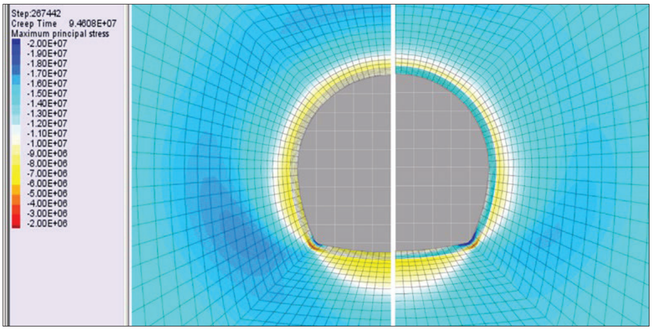

Fig. 2. Stress distribution in gallery in clay rock with creep in lining (left) compared to the case of an elastic lining (right) after 3 years

Conclusion

Accounting for creep in concrete lining in numerical estimations of the evolution of stress will reduce the stress in the lining. The earlier the lining is placed, the more significant the effect of the creep lining concrete will be. This also depends on the age of concrete when it starts receiving the pressure from the surrounding ground.

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Hydro-mechanical modeling of granular soils considering internal erosion

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Introduction

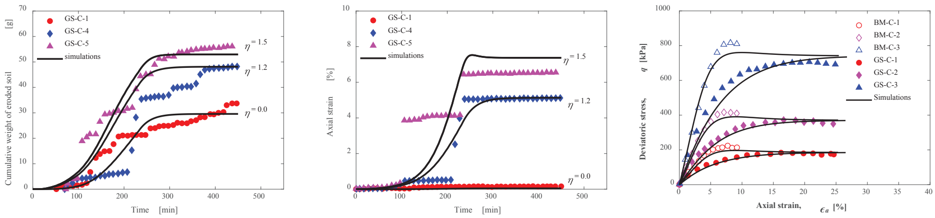
Internal erosion occurs when fine particles are plucked off by hydraulic forces and transported through the coarse matrix. The known causes are either a concentration of leak erosion, backward erosion, soil contact erosion, or suffusion (Bonelli and Marot 2008). This study attempts to formulate a coupled numerical model within the framework of continuum mechanics to investigate the phenomenon of internal erosion and its consequences on the behavior of granular soils. First, a four-constituent model has been developed to describe the detachment and transport of fine particles induced by the fluid flow. This process has been coupled with an enhanced critical state based mechanical model considering the effect of the fines content. Then, the predictive ability of the approach has been examined by simulating internal erosion tests on HK-CDG mixtures. Finally, the influence of the stress state, the initial density, and the initial fines content of the soil have been investigated by the developed numerical model.

Formulation of the time-dependent physical problem

According to Schaufler et al. (2013), it is possible to consider the saturated porous medium as a material system composed of 4 constituents in 2 phases: the stable fabric of the solid skeleton, the erodible fines, the fluidized particles and the pure fluid. The fines can behave either as a fluid-like (described as fluidized particles) or as a solid-like (described as erodible fines) material. Thus, a liquid-solid phase transition process is considered by a mass production term in the corresponding mass balances. The mechanical behaviours of the solid skeleton are reproduced by a non-linear incremental constitutive model including the critical state concept has been adopted (Yin et al. 2018). The model was enhanced by formulating the influence of the fines content on the critical state line in the *e-p’* plane based on experimental results in order to take into account the impact of the amount of eroded fines on soil deformability and strength.

Simulations of laboratory tests on HK-CDG mixture

A series of hydraulic-gradient controlled downward erosion tests on gap-graded HK-CDG mixtures, performed by Chang and Zhang (2011) with a newly developed stress-controlled erosion apparatus, were selected for simulations to investigate the initiation and the development of internal erosion in soils subjected to multi-step fluid flow under complex stress states, as well as the stress-strain behavior of soils subjected to internal erosion.



(a) (b) (c)

Fig. 1. Comparison between experimental results and simulations for HK CDG mixture under different stress states: (a) Cumulative eroded soil mass; (b) axial strain during erosion tests and (c) stress-strain vurves of drained triaxial tests before and after erosion

Fig.1 compared the experimental results and simulations for HK CDG mixture. GS-C-1, GS-C-2, and GS-C-3 were erosion tests under isotropic stress states, GS-C-4 and GS-C-5 were erosion tests under anisotropic stress states, BM-C-1, BM-C-2, and BM-C-3 were conventional drained triaxial tests on isotropically consolidated samples without erosion. The proposed model was able to reproduce the general trend of the cumulative weights of the eroded soil under different stress states. It is also able to reproduce the mechanical behaviors before, during and after erosion. Furthermore, the simulation results confirmed that the deformation is linked to the stress ratio under which the erosion process is active. Interestingly, besides the stress ratio, the amount of loss of fines, which indicates the yielding from a stable to an unstable mechanical response, appeared to be related to the initial density as well as to the initial fines content of the soil mixture.

Conclusion

This study provides a novel contribution to the numerical approach of quantifying the impact of internal erosion on the mechanical behavior of granular soils. A good agreement was obtained between the model predictions and the experimental results on HK-CDG mixtures. The results confirmed that the deformation is linked to the stress ratio under which the erosion process is active, the amount of loss of fines, and the initial fines content of the soil mixture.

Acknowledgements

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A coupled hydromechanical modelling of internal erosion around shield tunnel

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Introduction

Tunnel leakage is one of the most important factors of the ground movement (Zhang et al. 2017). However, the effect of the loss of fines has not been deeply discussed in the previous study. The leakage of underground water through the cracks and joints of lining commonly carries fines in the form of silt and clay from the surrounding ground, i.e. internal erosion (Bonelli and Marot 2011). The change of porosity due to loss of fines affects the mechanical behaviour of the surrounding soil (Yin et al. 2014; Yin et al. 2016). Conversely, the change of porosity influences the permeability of the sand and, therefore, its hydraulic conductivity. In this study, the effect of leakage on the tunnel in sand layer was investigated numerically with a coupled hydro-mechanical continuum approach, considering the internal erosion phenomenon induced by the local flows of underground water.

Formulation of the time-dependent physical problem

The mass balance equations are based on the porous media theory (de Boer 2000). The saturated porous medium is modelled as a system of 4 constituents: the solid skeleton, the erodible fines, the fluidized particles and the fluid. The fines can either behave as a fluid-like (described as fluidized particles) or a solid-like (described as erodible fines) material. Thus, a liquid-solid phase transition process is considered by a mass production term in the corresponding mass balances. Furthermore, the flow in the porous medium is governed by Darcy's law, in which the permeability of the medium depends on porosity. The mechanical behaviours of the solid skeleton are reproduced by a critical state based constitutive model (Jin et al. 2016; Yin et al. 2013, 2016), which is able to take into account the influence of the change of porosity on the mechanical behaviours of the soil during the process of internal erosion.

Numerical analysis of internal erosion around shield tunnel

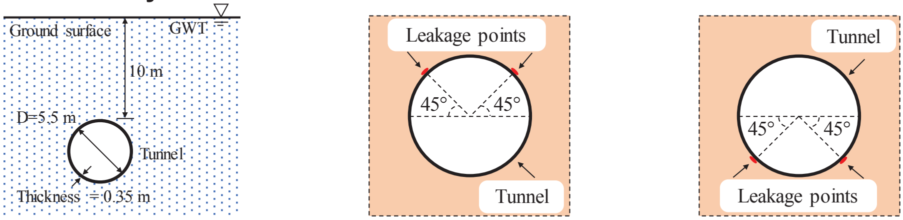

Fig. 1. Problem geometry and leakage location at tunnel lining

The suggested numerical model was conducted to investigate the hydro-mechanical response of an existing tunnel installed in sand layer and experiencing a local internal erosion due to the leakage of underground water from the damaged joints or cracks. The tunnel was assumed to have a circular shape with an inner diameter of 5.5m and to be constructed at a depth of 10m below the ground surface. The locations of the leakage points are shown in Fig. 1.

Fig. 2(a) shows the evolution of the fines during the leakage process. Internal erosion took place, leading to the decrease of the fines which was initially uniform in the whole soil volume. The decrease was critical in the vicinity of the cavity, which created a highly eroded zone. Furthermore, it is observed that for two-dimensional simulation, the lower leakage location results in a larger ground surface settlement, as shown in Fig. 2(b). Moreover, about 50% of the total settlement for 300 days is produced at very early stage of leakage. This is due to the change of effective stress induced by the change of boundary condition of pore pressure at the leakage point before the development of eroded area. After that the settlement keeps increasing and tend towards stabilizing. Significant increase of bending moment of linings was observed as well.

(a) (b)

Fig. 2. (a)Spatial distribution of fines for the two cases after 300 days of leakage; (b) Time history of maximum ground surface settlement for different leakage locations

Conclusion

In this study, the effect of leakage on the tunnel in sand layer was investigated numerically with a coupled hydro-mechanical approach, considering the internal erosion induced by the local flows of underground water. The investigations indicated that with the development of ground water leakage and internal erosion, there is an obvious increase in the maximum bending moment in the tunnel lining and the ground sur-face settlement.

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A fully coupled state-based peridynamic hydromechanical model for geomaterials

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Introduction

Strain localization or shear banding is a typical phenomenon observed in a wide range of materials and is a ubiquitous failure mode in multiphase geomaterials such as soils. Typically, when using standard finite elements without special treatments, localization of strains in a narrow band occurs and extreme dependence on discretization inevitably follows. The finite element solutions try to capture the localization zone of zero thickness, which results in their mesh dependence. This result is against experimental or field observations that the width of a shear band is directly connected to the microstructural length scale or the problem scale. The state-based peridynamics is a promising method for modeling shear banding and the scale-dependent thickness of shear bands. State-based peridynamics is a nonlocal reformulation of continuum mechanics using integral equations rather than the partial differential equations used in the classical theory of mechanics. This formulation allows handling the initiation and propagation of discontinuities like shear bands in solid without any special numerical treatment. Moreover, the peridynamic horizon, the influence neighborhood of a peridynamic material points, allows for an intrinsic length scale in the mathematical formulation in a straightforward manner. In this article, we formulate and implement a state-based peridynamic hydromechanical model for geomaterials. The conservation of mass in state-based peridynamics can be written as

$$\frac{\partial}{\partial t}(\rho[x]\phi[x]) = \int_{H_x} \underline{Q}[\mathbf{x}]\langle\boldsymbol{\xi}\rangle - \underline{Q}[\mathbf{x}']\langle-\boldsymbol{\xi}\rangle dV_{x'} + R[\mathbf{x}] - I[\mathbf{x}]$$

where ρ is the fluid density, Φ is the porosity of the solid skelton, and Q[x]<ξ> is the contribution to the mass flux scalar state at point x due to the bond ξ. The terms R and I are the source and sink terms at the point x. The balance of linear momentum under the quasi-static condition in state-based peridynamics can be expressed as

$$\int_{H_x} \left(\underline{T}[x]\langle\xi\rangle - \underline{T}[x']\langle-\xi\rangle\right) dV_{x'} + \phi^s\rho_s \cdot g + \phi^f\rho_f \cdot g = 0$$

where ρ_s and ρf are the solid and fluid densities respectively, g is the acceleration due to gravity, ϕ^f and ϕ^s are the fluid and solid volume fractions respectively. The term $\underline{T}[x]\langle\xi\rangle$ represents the total vector force state at point x from the bond ξ. In the numerical code, both equations are discretized using the meshfree method in the spatial space and the backward Euler method (implicit) is used for the temporal discretization.

Results and discussion

We run a simple two-dimensional problem under plane strain condition. The problem domain is 5cm x 10 cm in the x-y Cartesian coordinate system. The soil density is 2710 kg/m³. The bulk and shear moduli are K = 83 MPa and G = 18 MPa, respectively. The fluid density is 1000 kg/m³, the permeability k = 1×10^{-11} m/s. The initial pressure in the problem domain is assumed zero. The problem domain is discretized into 40 x 80 = 3200 equal material points (i.e., cells) with dx = dy = 0.25 cm for each cell. For the numerical simulation, the horizon δ = 0.75 cm. The soil is assumed to be fully saturated with zero excess pore water pressure. An initial confining pressure of 100 KPa is applied to the lateral boundaries of the solid skeleton. Zero flux boundaries are defined on the fluid phase. As a loading protocol, the vertical and horizontal displacements are applied on the top boundary layer. The displacement increments are 0.001 cm/s in the downward direction and 5×10-4 cm/s in lateral (toward the right corner) direction respectively for 100 load steps. Figure 1 shows the contours of dilation (left), plastic shear strain (middle) and fluid pressure (right) superimposed on the deformed configuration at t = 100 seconds. The results in Figure 1 show the localized zone formed in the sample. The fluid pressure is shown to be smaller in the localized zone that the value outside the localized zone. This lower fluid pressure in the localized deformation zone can be caused by the dilation of the skeleton in the shear banded zone. For instance, the change in fluid pressure is partially driven by the change in local porosity which in turn is driven by the dilation of the solid skeleton.

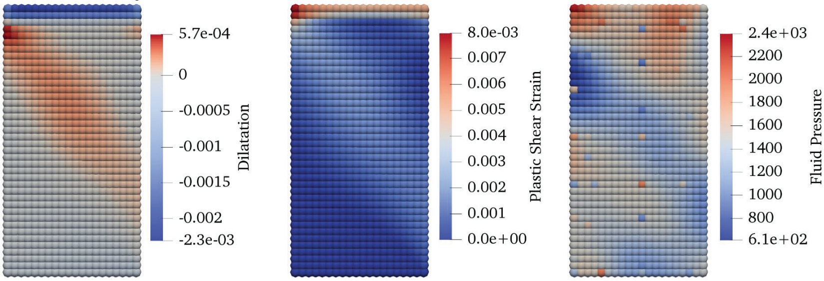

Fig. 1. Contours of dilation of the solid skeleton (left), plastic shear strain (middle) and fluid pressure (right) superimposed on the undeformed configuration at t = 100 seconds.

Conclusion

We have formulated and implemented a state-based peridynamic hydromechanical model for fluid-saturated porous media. The material model is implemented through the correspondence principle of the state-based peridynamics theory. As demonstrated by the numerical example, the formulated numerical model is suitable for modeling the shear band failure of geomaterials in that the formulation is based on the integral equations which allow for discontinuities in both displacement and fluid pressure.

Acknowledgements

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Numerical modelling of thick-walled hollow cylinder tests on Boom Clay samples cored parallel and perpendicular to bedding

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Introduction

The disposal of high-level radioactive waste is a complex issue due to the very strict requirements to achieve a long-term security. Their storage in deep geological formations is considered as a highly promising solution. Due to its low permeability and self-sealing capability, Boom Clay is a particularly adequate formation. The stress release occurring during the excavation of galleries in Boom Clay induces an Excavation Damaged Zone (EDZ). During the excavation of the underground laboratory HADES at Mol (Belgium), a higher convergence was measured horizontally (i.e. parallel to the bedding planes) and the damaged zone was as well larger in the horizontal direction (Bastiaens and al. 2003). This could be related to mechanical anisotropy (strength and/or deformation) as well as to hydraulic anisotropy. The initial anisotropic stress field in the clay formation could also play a role.

Hollow cylinder tests

Thick-walled hollow cylinder experiments have been carried out in the framework of TIMODAZ project (Labiouse and al. 2014). After recovery of the in situ stress conditions, the inner confining pressure was decreased in 70 minutes to reproduce tunnel excavation under laboratory conditions. Medical scans have been carried out before and a few hours after the unloading to observe the displacements occurring in the sample. Tests conducted on Boom Clay samples cored parallel to bedding show an anisotropy of convergences similar to the one observed in situ, while an isotropic behavior is observed for samples cored perpendicular to bedding. Further experiments are underway at HEIA-FR. The deformation of the clay over time is observed by means of several high-resolution scans taken after mechanical unloading. The effect of the unloading rate is also investigated.

Numerical modelling

In parallel to the laboratory work, numerical simulations are performed in order to assess the capability of existing constitutive models to adequately reproduce the experiments and to gain further understanding. Hollow cylinder tests on samples cored perpendicular to the bedding planes are first simulated with an isotropic linear elastic perfectly plastic constitutive law. Then, tests on samples cored parallel to the bedding planes are modeled considering the inherent anisotropy of Boom Clay. For this purpose, among the various constitutive models taking into account the mechanical anisotropy of geomaterials (e.g. François and al. 2014), a rock mechanics oriented failure criteria is adopted using ZSoil Multilaminate model. In addition to the matrix which is modelled as an isotropic linear elastic perfectly plastic continuum with a Mohr-Coulomb failure criteria, weak planes with reduced Mohr-Coulomb strength parameters are considered in the specific orientation of the bedding. The model however does not consider elastic anisotropy.

Results and discussion

Results provided by the isotropic model are in accordance with the measurements on samples cored perpendicular to bedding. Computations were performed to investigate the influence of the inner confining pressure unloading rate on the radial displacement and on the extent of the plastic zone. Due to Boom Clay low permeability, a strong hydro-mechanical coupling occurs during the unloading. The pore water pressure redistribution that follows produces additional displacements after the unloading. (Fig. 1).

With respect to the samples cored parallel to the bedding planes, both experiments and modelling (when the anisotropy is considered) show an average larger convergence of the clay around the central hole. However, numerical models considering strength and/or hydraulic anisotropy predict a larger displacement and extent of the plastic zone in the direction perpendicular to the experimental and in situ observations (Fig. 2). This statement is not software dependent as similar results were obtained using the finite difference program FLAC with both strength (i.e. Ubiquitous Joint model) and hydraulic anisotropies (Kivell 2015).

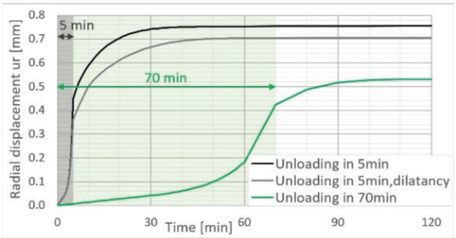

Fig. 1.Influence of the inner pressure unloading rate on the evolution of central hole convergence

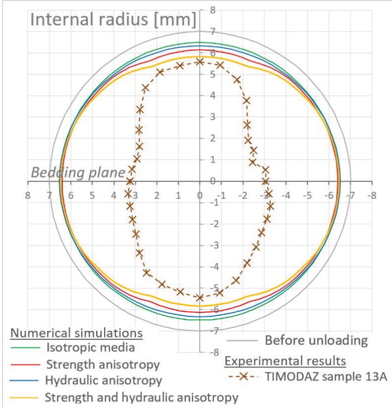

Fig. 2. Measured and predicted hole convergence for samples cored parallel to bedding

Conclusion

Hollow cylinder tests performed in the framework of the TIMODAZ project and in-situ observations at Mol URL have underlined a significant influence of bedding on the development and the orientation of the damaged zone. Numerical simulations with standard constitutive models considering strength and/or hydraulic anisotropy show a significant discrepancy with these observations. Upcoming work will be carried out with a model including as well elastic anisotropy (Souley and al. 2017).

Acknowledgements

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Coupled hydro-mechanical modeling of advancing tunnel in deep saturated ground

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Introduction

Computational geomechanics has gained wide prominence in simulating coupled hydro−mechanical (H−M) interactions of fluid flow and deformation in many geotechnical engineering applications. For coupled problems, numerical methods may be the only approach to obtain solutions on the stress/displacement and pore pressure distributions in complex boundary-value problems. This paper highlights the use and importance of computational geomechanics in the H-M analysis of an advancing tunnel in deep saturated ground. The paper focuses on two aspects routinely encountered in tunnel analysis and design: (1) the transient H-M interactions of an advancing tunnel in deep saturated ground, and (2) the effects of H-M interactions on the convergence-confinement (CC) response and longitudinal displacement profile (LDP) of an advancing tunnel. The tunnel is represented by a 2-D axisymmetric coupled model built using the computer program Fast Lagrangian Analysis of Continua (FLAC) developed by Itasca. The tunnel is advanced in a step-wise coupled manner featuring alternating stages of undrained excavation and drained consolidation until the final face is reached, and the ground is left to consolidate towards the steady-state condition. Two cases were analyzed. In Case 1, simulation results show that in the short term, tunnel excavation causes a non-monotonic variation of pore pressure that would stiffen the advance tunnel core and provide temporary confinement to the tunnel face. It is shown in Fig. 1 that significant differences are observed in the predicted response of the ground from the coupled vs. uncoupled simulations particularly during early stages during of tunnel advance. In fact, the predicted for the short-term pore pressure from coupled modeling can increase above the initial value. In the long term, the excess pore pressure dissipated nonlinearly with time, accompanied by transient deformation of the ground and load transfer toward the advance core. In Case 2, a nonlinear unloading factor is proposed for estimating excavation-induced pore pressure changes in the CC. New LDP equations are also proposed that account for transient radial displacements along the axis of the tunnel (Fig. 2). The new CC and LDP equations, which improve on the predictions of ground response to an advancing tunnel by accounting for the coupled-interactions between the effects of water seepage and ground deformation, is envisioned to improve analysis and design of tunnels.

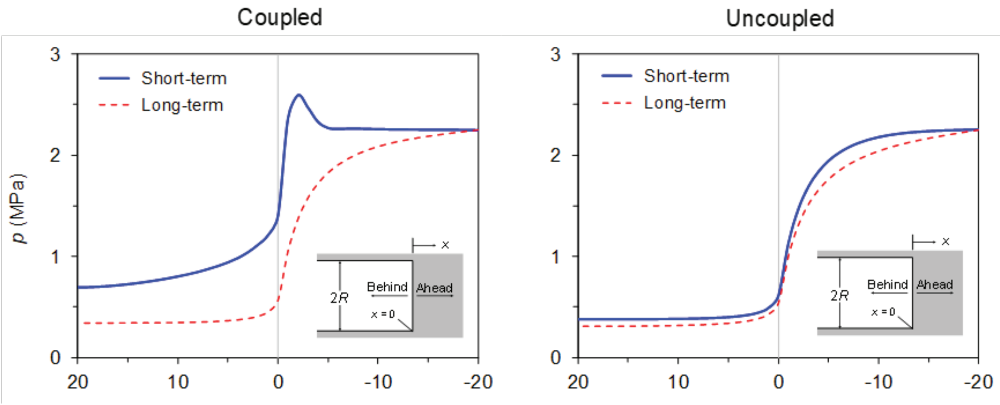

Fig. 1 - Longitudinal pore pressure profiles in an advancing tunnel from coupled and uncoupled simulations.

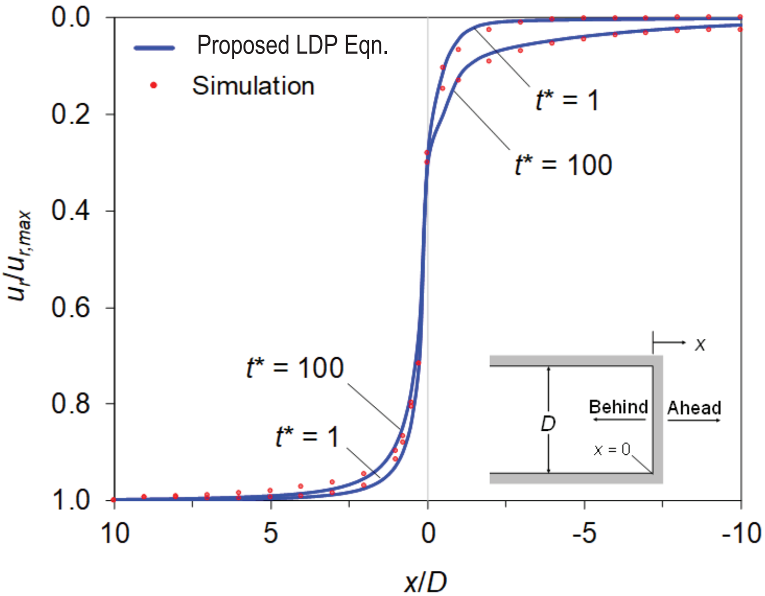

Fig. 2. Plots of normalized transient LDPs accounting for H-M coupling effects.

Modeling primary fragmentation in cave mines using BBM (Bonded Block Modeling)

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Introduction

The Bonded Block Modeling (BBM) is a novel modelling technique implemented in Distinct Element Code *3DEC* (Itasca, 2016), where rock mass is simulated as an assembly of small scale interlocked tetrahedra that can break at their contacts. This method naturally follows crack initiation and the formation of fragments (Garza-Cruz and Pierce, 2014) as well as bulking and spalling around deep underground excavations (Garza-Cruz et al., 2014).

In the context of cave mining, the BBM can model primary fragmentation in the cave back vicinity due to gravity and induced stresses. Primary fragmentation impacts the width of draw, drawpoint spacing, the probability of hang-ups and associated production losses. Low persistent veins and pre-existing fractures are critical for the initiation and process of fragmentation. A large parametric study investigates their role using the BBM.

Modeling approach

A numerical model is built in a *3DEC* environment where the veined rock mass is represented by an assembly of Bonded Blocks. Veins are simulated by Coulomb contacts with dilation in shear, and a tension limit. Tensile strengths are derived from a wide Weibull distribution, so the contacts represent both weak veins and strong intact rock bridges. The cohesion to tensile strength ratio is assumed constant and equal to 2.5, to produce a macro UCS to tensile strength ratio in the order of 10 to 20 which is consistent with typical hard rock observations.

The BBM is cut by a network of persistent and cohesionless joints introduced as a Discrete Fracture Network (DFN, Fig1 – here, three sets of orthogonal joints). Block caving is simulated by excavating below the sample and progressively relaxing stresses at the surface of the excavation.

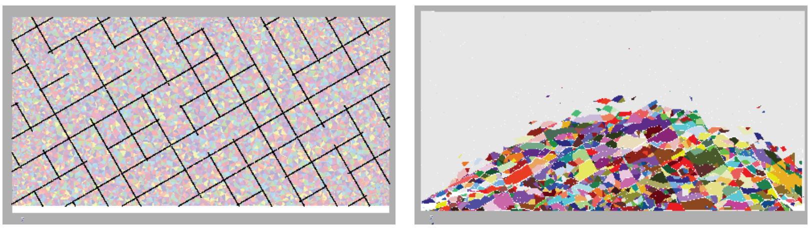

Fig. 1. Cross section of a BBM sample with an embedded DFN (left) and resulting primary fragmentation (right).

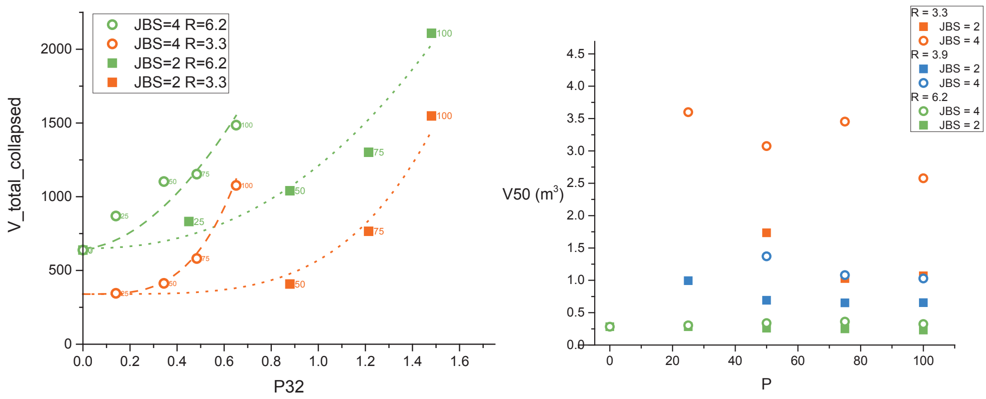

Fig. 2. Total fragmented volume (m³) versus DFN density P_{32} (left) and 50th percentile of fragments volume (V50) versus DFN persistency *P* (right), for various R and JBS.

Results and discussion

The parametric study considered cave back stress, veins and DFN mechanical properties (friction angle, cohesion and tensile strength) and DFN geometry (orientation, joint spacing and persistence). Key parameters affecting primary fragmentation were identified: $R = \sigma_1/coh_{vein}$ (ratio of cave back stress to vein cohesion), *JBS* (Joint Block Spacing) and *P* (DFN Persistency).

The total fragments volume is correlated to an increase of *R* and fracture density P_{32} (Fig 2). However, for a given P_{32}, different combinations of *P* and *JBS* result in fairly different collapsed volumes.

The impact of DFN properties on fragment sizes depends on ratio *R* (Fig 2). For high values of *R* (high cave back stresses or weak veins), *V50* is almost insensitive to DFN properties and veins control the fragment size distribution. When the ratio decreases, *V50* is larger for larger *JBS* and smaller *P*.

Conclusion

The BBM technique was found to be an effective tool to study the effect of the DFN and veined rock mass properties on primary fragmentation in cave mining using fairly simple DFNs. Future work will investigate the use of more realistic DFNs, and the effect of varying cohesion to tensile strength ratio of the veins.

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Stability assessment of an abandoned underground chalk quarry

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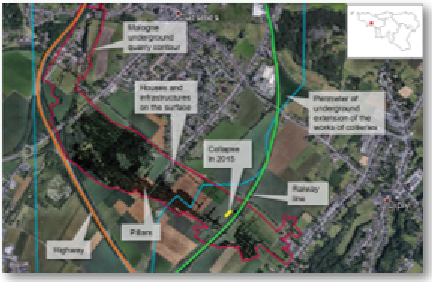
Introduction

Scope of work

- Rock and rock mass characterization (case study in Malogne underground quarry)
- Understanding of failure mechanisms of underground openings

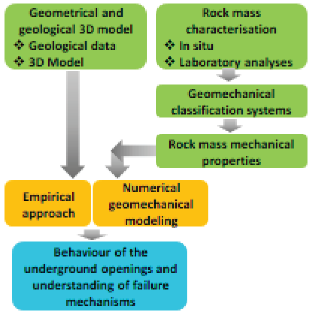
Working method

- Data collection
- In situ and laboratory analyses
- Numerical modeling for simulation of different configurations for understanding the failure phenomena



Map of underground Malogne phoshatic chalk quarry with surface infrastructure

Methodology



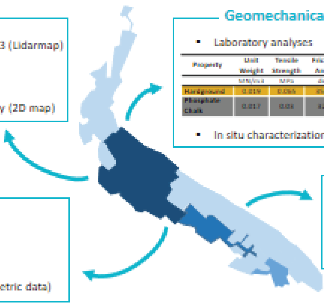
Data collection

Geometry

- Topography: 1865, 2013 (Lidarmap)
- Quarry: hangingwall
  footwall
  pillars geometry (2D map)

Geology

- Geological map
- Drillholes
- Structural analysis
- Hydrogeology (piezometric data)



Geomechanical properties

- Laboratory analyses

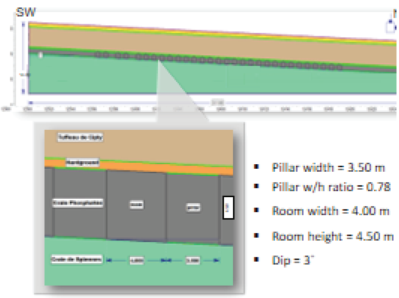
Property	Unit Weight	Tensile Strength	Friction Angle	Cohesion	Young's Modulus	Poisson's ratio
	MN/m3	MPa	deg.	MPa	MPa	-
Hardground	0.019	0.065	45.87	0.39	1 429	0.15
Phosphate Chalk	0.017	0.03	32.5	0.31	431	0.35

- In situ characterization

Land management

- Land ownership and access permition
- Roads, railway, buildings

Building a preliminary numerical model

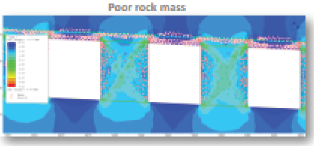


- Pillar width = 3.50 m
- Pillar w/h ratio = 0.78
- Room width = 4.00 m
- Room height = 4.50 m
- Dip = 3°

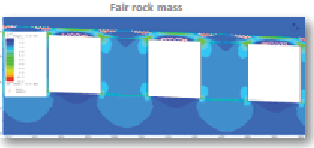
Numerical model

Parametric study

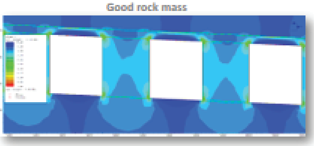
Poor rock mass



Fair rock mass



Good rock mass

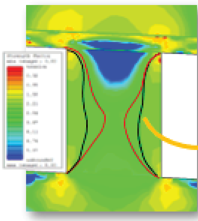


Poor rock mass:
- Laboratory data for the rock mass properties
- Yielded rock mass

Fair rock mass:
- Slightly increased rock mass properties
- Several yielded elements

Good rock mass:
- Increased rock mass properties
- Sporadic yielded elements

Focus on a pillar



Numerical modeling of pillar (vertical section) with data about the Strength Factor

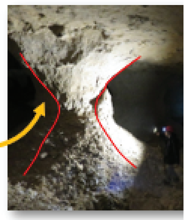


Photo of spalling pillar from Malogne underground quarry

Conclusions and perspectives

- Multi approach assessment for rock mass characterization and numerical modeling are instruments to understand the failure mechanism
- Parametric study allow better understanding the sensitivity of the rock mass properties and their influence on the rock mass behaviour.
- Preliminary results are in agreement with the underground observations

Future work:
- Rock mass characterization/ in situ and additional laboratory analysis
- Weakening effect of water
- 3D numerical modeling: BEM
- Study the influence of old deep mining activities on underground cavities
- Understanding the failure mechanisms
- Develop risk management solutions

Instability mechanisms of chalk mines in presence of water: feedback from the collapse of the Baulieu mine (France).

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Introduction :

The weakening of chalk mechanical characteristics under the influence of ageing and flooding/unflooding cycles may be at the origin of dramatic collapses. For illustration, in January 1910, the collapse of the underground chalk mine known as "Beaulieu" (Chateau-Landon, France) caused a large landslide, leading to the death of seven people and destroying several houses. The sudden nature of the event and its concomitance with the flooding of the mine by the nearby “Loing” river have raised the hypothesis that the collapse was caused by the degradation of the mechanical properties of the chalk induced by the increase of saturation degree.

To investigate this hypothesis and more generally, to better understand the mechanisms at the origin of instabilities in chalk mines, a large program including a laboratory characterization, in situ instrumentation and numerical modelling, was set up in 2011. Three underground chalk mines of the Paris Basin (France), including Chateau-Landon, were selected to study the geomechanical behavior of chalks subjected to changes in water saturation degree. Various series of laboratory tests have been undertaken, which allowed to advance in the understanding of the physical and mechanical water-chalk interactions. However, the extent of the phenomenon observed in Chateau-Landon could not be explained only by the reduction of the mechanical properties of chalk in the presence of water.

In this context, it was decided to study the behavior of a neighboring and still accessible mine known as "Royer", located less than 500 m from the Beaulieu mine. This mine appeared as the ideal site to get further insights into the mechanisms responsible for this collapse. Indeed, this mine has extracted a similar material, and showed signs of instability in 2016 following exceptional climatic events for this region (177 mm of precipitation for the month of May and important flood of the Loing).

Modelling approach

A first phase of study confirmed similarities between the mines Royer and Beaulieu and the interest to proceed to numerical modelling and instrumentation of the mine Royer in order to better understand the behavior of the works and the mechanisms that led to the sudden collapse in the mine Baulieu.

Given the particularly complex geometric configuration of the site (surface topography with hillsides and valleys, heterogeneous galleries), geometric modeling would not have been possible without prior digitization of mine galleries by 3D laser scanner. Then, the numerical model was built with a 3D numerical code integrating a fault and the geological layers with their geomechanical properties from the laboratory characterization.

The numerical model thus created has 3,500,000 zones spread over 476 m x 387 m horizontally and on a height varying between 57 m and 83 m. The overburden height varies between 1 m and 27 m. The mesh is more precise around the mine (mesh width = 0.2 m compared to 3 m at the model

boundaries) which extends horizontally about 110 m in both directions. A phasing with 15 excavations was made to represent the history of the exploitation. The aspects taken into account by the model are mechanical (excavations + fault) and indirectly hydraulic (rise of groundwater simulated by a reduction of the properties of the chalk and the fault).
Different scenarios were carried out: current state (reference case), flooding with rising water in 11 steps, probable scenario (rise of groundwater without flooding). The aim was to focus on the mechanisms potentially responsible for a collapse after an extremely rainy period: increased lubrication of the fault and rise of the groundwater which rapidly modifies the geomechanical properties of the chalk pillars.

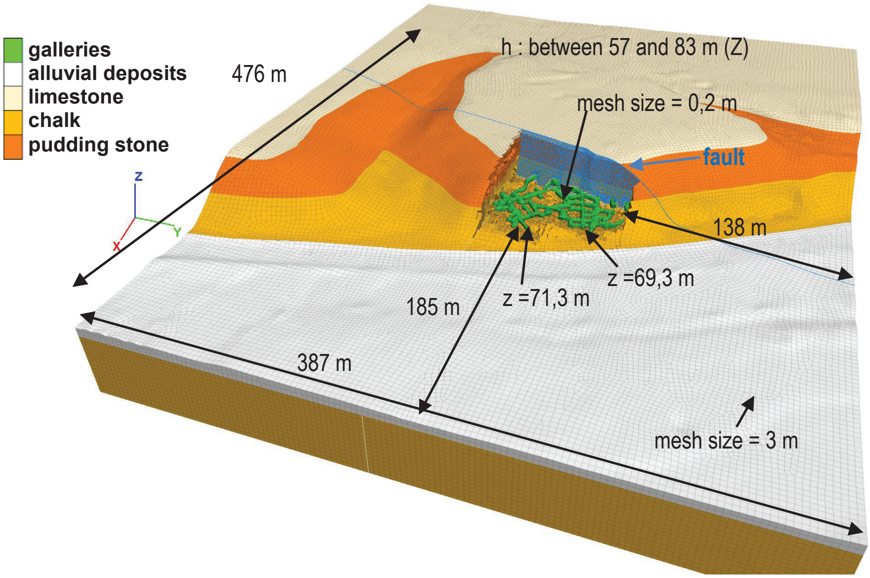

Fig. 1. Mesh of the Royer mine (Chateau-Landon, France)

Results and discussion

Various scenarios have been proposed to understand the primary mechanisms that participated in the collapse of the mine Beaulieu. In this paper the results associated with these scenarios are presented and discussed in light of the observations made in the Beaulieu and Royer mines. Moreover, the construction of a differential state (difference between a probable scenario and the reference state) enabled to identify the zones of maximum variations of displacements, strains and stresses. That information will be used to design an adequate monitoring instrumentation.

Conclusion

This innovative multidisciplinary approach, i.e. feedback from the 1910 collapse, 3D digitization, 3D modeling, instrumentation, opens new roads for the understanding and prevention of collapse risks that may concern other chalk mines in Paris Basin and elsewhere.

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Propagating Uncertainties to Evaluate Confidence Intervals on the Transmission of Ground Movements

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Introduction

Recent developments in civil engineering present multiple interaction problems between the soil and the structure. One of the major problems is the impact of ground movements on buildings. Consequently, managing risks associated with these movements requires a determination of various influencing factors and a specific knowledge of their variability/uncertainty (ElKahi, et al., 2018). The main purpose of this article is to develop a sensitivity analysis study through an analytical approach to investigate the behavior of structures subjected to differential settlement in order to assess their vulnerability taking into account important model and knowledge sources of uncertainties in addition to the natural variability of the Soil-Structure Interaction (SSI) geometrical and geotechnical factors. Results reveal the effect of taking these uncertainties into consideration via confidence intervals manifesting their influence on the transmission of the ground movement to the structures.

The great majority of reliability studies represent uncertainty by a probability theory, thus assuming that ignorance of soil-structure properties is of a random nature (Piegay, 2015), one of the most useful tools is the development of a meta-model in order to quantify the influence of uncertainties related to the variability of these parameters on the transmission of the ground movement.
Consequently, this study will develop a meta-model representing the transmission of the ground movement as a function of the relative stiffness. The “basic random variables theory” is used to model the uncertainty of the SSI properties and particularly the Monte-Carlo simulations: This method is compelling for propagating the uncertainties on the input data of a model, so as to determine the uncertainty on a result given by this model. To use the Monte- Carlo method, it is usually necessary to have probability distributions functions with corresponding characteristics (normal, lognormal distribution, etc.) for the input data (Hawchar, et al., 2016). Consequently, in this study, the developed approach for estimating the SSI parameters uncertainties is the use of estimates based on published values, which are most conveniently expressed in terms of the coefficient of variation (COV).
Taking into consideration the SSI parameters variability combined with the soil profile deformation form uncertainties, this study will develop possible confidence intervals that contain the true values of the unknown transmission ratio versus the relative stiffness. The desired level of confidence is set between 70 and 95%. Consequently, in practice, engineers can directly refer to these intervals in their design considering these uncertainties affecting the SSI.
Based on the previous analytical results for the polynomial free-field deflection, a meta-model was developed to check the variability of the deflection transmission ratio Δ/Δ_0 upon the relative stiffness ρ^*, consequently upon the structure stiffness EI, the soil stiffness K and the structure length L (ElKahi, et al., 2018). According to this meta-model, the deflection transmission ratio curve is assumed to respect the following equation:

$$\frac{\Delta}{\Delta_0}(\rho *) = 0.5 - 0.5 \text{ Tanh } [\, 2.98 + 0.47 \text{ Log } (\rho *)]$$

$$\frac{\Delta}{\Delta_0}(EI, KB, L) = 0.5 - 0.5 \text{ Tanh } [\, 2.98 + 0.47 \text{ Log } (\frac{EI}{KBL^4})\,] \qquad (1)$$

Considering numerous particular values of the relative stiffness, and developing 50,000 iterations for every parameter (EI, K and L) with the corresponding probability distribution function and the corresponding COV, a confidence interval can be associated to the curve of Δ/Δ_0 versus ρ*. Fig. 1 represents the confidence interval with 30% risk, since it considers 1 standard- deviation difference with the COV (EI) = COV (KB) = 20%; COV (L) = 5%. To reduce the risk to 5%, 2 standard-deviations are considered.

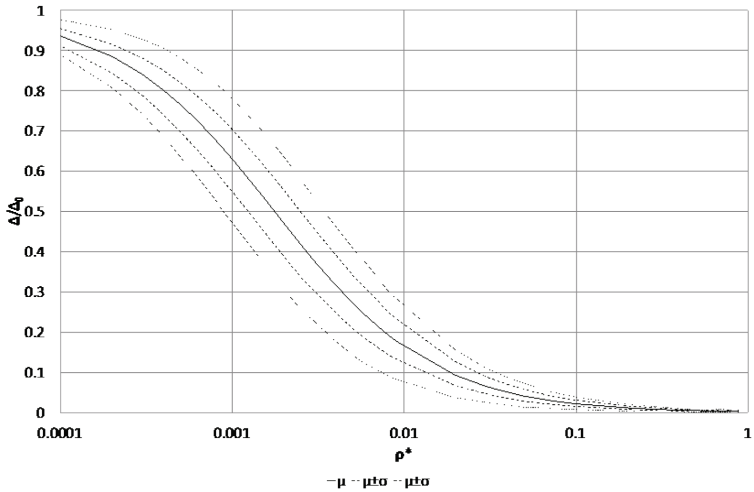

Fig. 1. Confidence Interval to the curve of the transmission ratio Δ/Δ_0 versus the relative stiffness ρ*. (COV (EI) = COV (KB) = 20%; COV (L) = 5%).

As perspectives, this study raises many questions related to the uncertainties that can affect the evaluation of the transmission of the ground movement. One of the main questions is the validity and the accuracy of the analytical model used to model the SSI phenomenon, a simple model that does not take into consideration the vertical shear stress, the plasticity that can occur in the ground, the displacement of the soil outside the building, etc. Consequently, validation of analytical results by numerical and experimental models is necessary to evaluate and propagate uncertainties affecting the transmission of ground movements to structures.

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Investigating the mechanical behavior of a surface repository for low and intermediate-level short-lived radioactive waste

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Introduction

Andra, the French national radioactive waste management agency, operates a surface repository in Aube ("CSA", North-Eastern France), where some of the concrete canisters containing low and intermediate-level short-lived radioactive waste are stacked in concrete structures, then filled with gravel (Fig. 1). ITASCA Consultants SAS, together with EGIS Industries, has studied the mechanical behaviour of the assemblies, in the framework of design and safety demonstration programs.

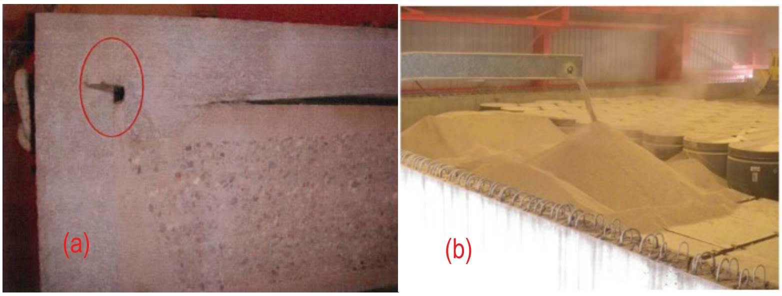

Fig. 1. Cross-section through a concrete canister (a) and gravel filling around piles of canister in a large concrete structure (b)

Modelling approach and some results

Making use of progressive upscaling (individual canister, then pile of stacked canisters and gravel, and finally assembly of gravelled piles into concrete structure) and downscaling (load on piles deduced from stress transferred to homogenised content in structure simulations), the study aimed at understanding and quantifying the mechanical behaviour of these assemblies at different scales.

At the individual canister scale, to better represent the heterogeneous load on the lid surface, 2 superposed canisters were simulated. The top canister, with an elastic upper part, was loaded uniformly. More than 40 cases were carried out, considering four container types (one cube and three cylinders), several waste mixes, possible lid defects (Fig. 2a, b), and raising gas pressures due to oxidoreduction reactions in packages containing metal.

At the pile of canisters scale, in addition to variations in individual canisters, pile eccentricity was also considered (Fig. 2c), resulting in more than 30 cases. Effective properties of piles without the overlying gravel were estimated.

Finally, representative volumes were evaluated for various pile arrangements in a large concrete structure, using previously estimated pile properties (Fig. 3). The most critical efforts transferred onto piles were identified, then used to re-evaluate these corresponding piles.

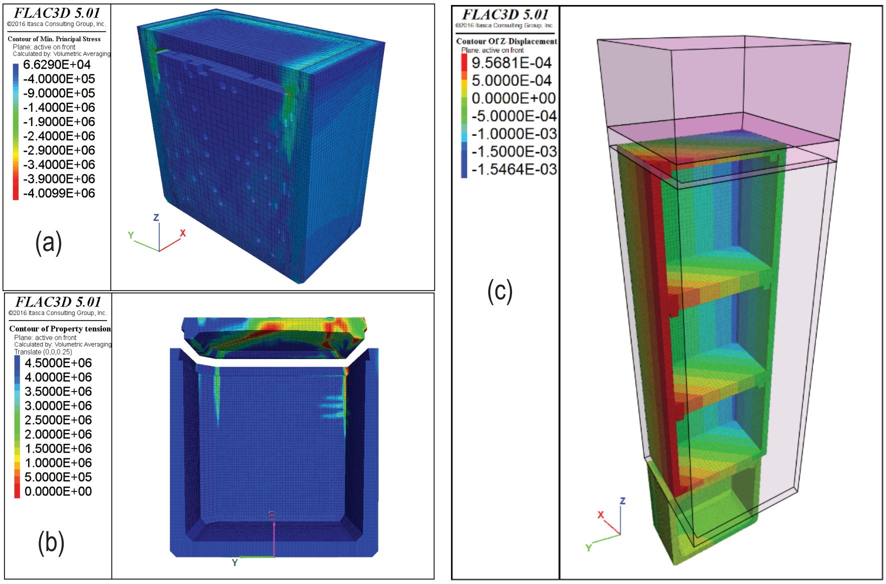

Fig. 2. Minor principal stress in a canister without defect and with a hard waste mixture (a), and tensile strength reduction in a canister with multiple defects and soft waste mixture (b). Vertical displacement in a pile of canisters with eccentricity, the transparent parts represent gravel (c)

Conclusion

The mechanical behaviour of many canister types, waste mixes, canister defects, canister packing eccentricities and arrangements of piles of canisters in large concrete structures were investigated. To cover these variations as well as their combinations, a progressive upscaling and downscaling approach was adopted, identifying representative and envelop scenarios.

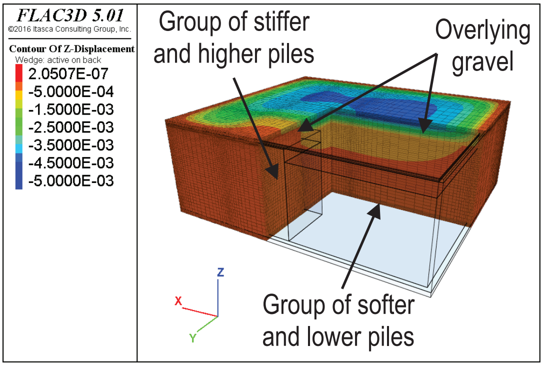

Fig. 3. Vertical displacement in a concrete structure filled with 2 groups of piles of canister and gravel

The most critical elements and phenomena at each scale, namely (i) individual canister, (ii) pile of canister, and (iii) representative volume of piles of canister, were identified and evaluated. Some limiting values, such as the maximum acceptable internal gas pressure, and unfavourable situations were recommended to Andra.

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