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Forschung und Innovation in der Geotechnik
Recherche et innovation en géotechnique

+ Short Course „Numerical Analyses in Geotechnical Engineering"

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Numerical modeling for limit analysis of geotechnical structures

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Numerical modeling for limit analysis of geotechnical structures

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1 Introduction

Verification of ultimate limit states (ULS) of geotechnical structures relies on limit analysis in combination with the use of partial factors of safety. For such verifications, the soil is typically modeled as rigid-plastic with a Mohr-Coulomb or Tesca yield surfaces depending on the scope of the analysis (drained vs undrained / long vs short term failures). Except for very simple situations such as shallow footing where textbook solutions are available, obtaining lower and upper bounds for the failure of geotechnical structures is not a simple task and a large body of the historical geotechnical literature has been devoted to the development of approximated solutions (often from a kinematic / upper bound point of view).
Thanks to the development of numerical tools which combine specific finite element formulation with algorithms developed for the solution of optimization problems with inequality constraints, it is now possible to obtain upper and lower bounds numerically on a laptop computer. In this lecture, we will briefly review the basis of limit analysis and the ingredients required to solve such problems numerically. Classical as well as more complicated examples will illustrate the modeling approach. Moreover, similar algorithms can also be used to verify serviceability limit states with e.g. a Cam-Clay elastoplastic model [5].

1.1 Limit analysis

Let's consider a structure of rigid plastic material which is subjected to a set of body forces (self-weight) as well as a set of tractions on its boundary. For example a soil with a load transmitted via a strip footing, or an excavation supported with a diaphragm wall with anchors. The central question of limit analysis can be posed as: What is the maximum magnitude of the tractions that can be sustained without the structure suffering collapse - or alternatively: what is the minimum magnitude of the tractions that will cause collapse [1].

Typically a multiplier on the load intensity (tractions or gravity) is introduced, and after formulating the upper and lower bounds theorem as numerical optimization problem, an upper & lower bound of the collapse multiplier can be obtained. Another approach is –at constant load – to reduce the strength of the soil in order to obtain a safety factor.

A lower bound of the true collapse load parametrized by a multiplier α can be obtained from the lower bound theorem that states it is the maximum value for which a statically admissible stress field (i.e. satisfying the quasi-static balance of momentum and tractions boundary conditions) satisfying the yield conditions can be constructed.

Alternatively, an upper bound is obtained by finding the minimum value of the multiplier such that the principal of virtual work and the yield criteria is satisfied. This upper bound problem requires consideration of kinematic quantities and provides the possibility to compute an upper bound to the exact collapse multiplier, namely by postulating a compatible velocity field that satisfies the flow rule. This is done in such a manner that the rate of work done by the reference tractions is scaled to unity.

Special finite elements can be devised for lower bound or upper bounds problem [2] allowing to properly reproduce kinematic constraints. Moreover, discontinuities may be introduced by collapsing patches of regular continuum elements to zero thickness. Combining such finite element formulation with a reformulation of the yield criteria as sets of inequality constraints allow to use numerical scheme developed originally for non-linear optimization problem with quadratic inequality constraints. Algorithms for second order conic programming (SOCP) are particularly suited and have been applied to a number of limit analysis as well as elastoplastic problems [3,4,5]. We use here OptumG2, which combines these different algorithms with an easy to use graphical interface geared toward geotechnical problems.

2 Examples

2.1 A simple strip footing

First, we briefly present the classical example of the bearing capacity (see Figure 1) of a shallow footing for a Tresca material (e.g. short-term verification) for which the bearing capacity is known analytically to be equal to $q = (\pi + 2)c_u$. A numerical estimate of the lower bound using a Tresca material with a cohesion of 10kPa is obtained as $q_{LB} = 50.33\ kPa$, while the upper bound is obtained as $q_{UB} = 52.4\ kPa$. We see that these 2 values obtained numerically effectively bound the true solution $q = (\pi + 2)c_u = 51.41 kPa$. Moreover, using mesh adaptivity the failure mechanism is clearly highlighted (see Figure 1).

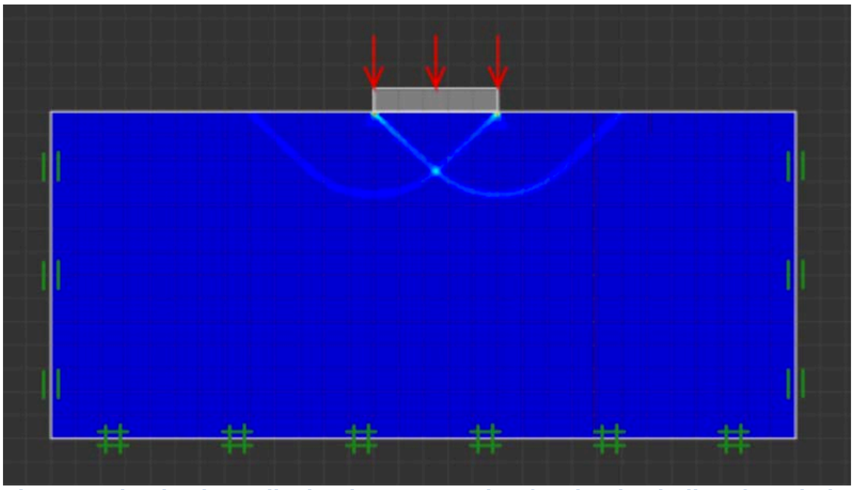

Figure 1 Plastic shear dissipation - Example of a simple shallow foundation ultimate limit state (short term case).

2.2 Stability of an excavation

As a second illustrative example, we verify the general stability of a 14 meters strutted excavation in a two-layer soil consisting of 6 meters of sand (γ_{dry}=20 kN/m^3, c=0 , ϕ=32^o, k_x=k_y=100m/day) overlaying a sandy clay (γ_{dry}=19 kN/m^3, c=10kPa , ϕ=25^o, k_x=k_y=0.001m/day). We focus on the long-term stability (water is lowered to the excavation floor on the excavated side). We use a strength reduction approach on the soil component only in order to get a factor of safety numerically using a static/lower bound approach.
The total length of the retaining wall (e.g. sheet piles / diaphragm walls) is set here to 20 meters, and the wall is modeled here as a flexible plate.

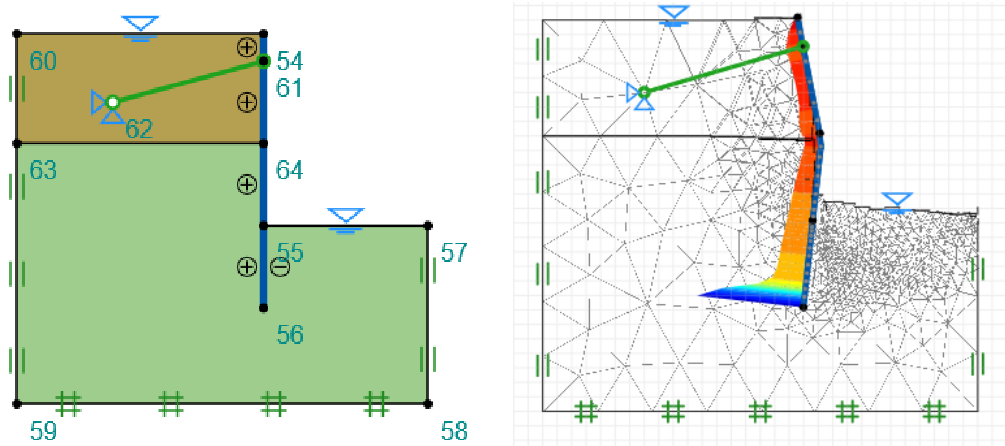

Figure 2 Geometry (left) and failure mechanism together with total horizontal earth pressure acting on the left side of the wall.

First, for illustration, we investigate first the case of a single row of anchor with no pre-stress. As can be guessed for such a configuration (short wall). The obtained strength reduction/safety factor is of 0.54 – clearly below 1 and the maximum bending moment in the plate is of 1424kN.m/m. The horizontal total pressure (earth pressure + water pressure) can be seen on Figure 2 together with the overall failure mechanism. A clear counter-passive zone around the rotation point at the bottom of the wall can be clearly observed.

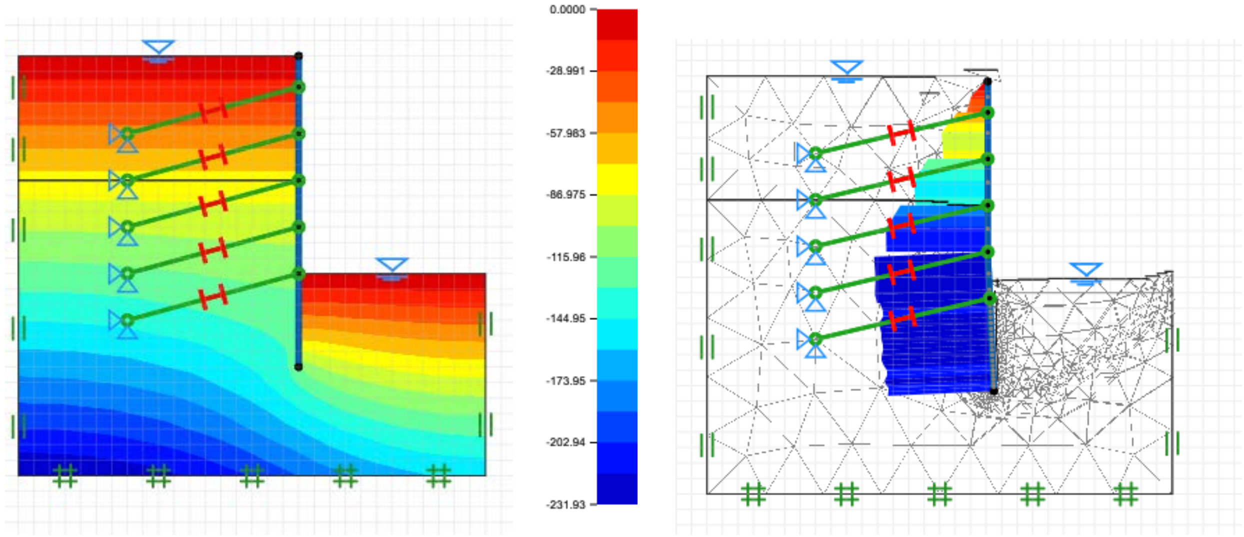

Figure 3 Pore pressure (left) and failure mechanisms together with total horizontal earth pressure acting on the left side of the wall for the case of a row of 5 anchors.

We re-perform the computation with now 5 rows of pre-stressed re-inforcement (to a pre-stress force of 200kN). We thus obtain an overall safety factor of 1.26, therefore stabilizing the excavation. The long=-term pore-pressure due (which is obviously similar for both cases with 1 or 5 row of anchor) is also displayed in Figure 3.
Of course, a short-term analysis can (and should) also be performed. Similarly, such a limit analysis computation should be performed at each excavation stages. It is important to bear in mind that such limit analysis computations do not however depend on the loading/excavation history: they are geared toward the verification of ultimate limit states. In addition, a loading history dependent elasto-plastic analysis would be required to verify serviceability limit states (displacements etc.).

A complete example illustrating how such numerical computational tool can assist in the design of an excavation will be given during the lecture as well as 3D examples if time allow.

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177	MITTEILUNGEN der GEOTECHNIK SCHWEIZ PUBLICATION de la GÉOTECHNIQUE SUISSE AVVISO di GEOTECNICA SVIZZERA
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Experimental determination of behavioural models and their parameters: a critical step for every numerical simulation

Alessio Ferrari

Experimental determination of behavioural models and their parameters: a critical step for every numerical simulation

1 Introduction

Numerical modelling is often a fundamental tool for the analysis and design of geotechnical systems. It offers the possibility to solve demanding problems still considering the relevant physical and mechanical behaviours of the involved materials. Most of the numerical codes used in the daily geotechnical practice, allow dealing with elaborated geometries of the analysis domain, different soil types, sequences of construction/excavation, time dependent mechanical and hydraulic boundary conditions. Each analysis requires assumptions on at least two fundamental components of the numerical modelling: (i) the mechanical constitutive behaviours of the involved geomaterials (i.e. mathematical relationships between changes in effective stresses and changes in strains), and (ii) values for the constitutive parameters. The user of the software assumes the responsibility for these choices himself.

2 Content of the presentation

The presentation discusses selected topics on how laboratory soil testing can guide geotechnical engineers in dealing with these two fundamental aspects of numerical modelling: selection of the constitutive models, and assessment of parameters values.

The following points will be briefly addressed.

2.1 Selection of the constitutive model type on the basis of the observed stress-strain response.

Many models exist which differ for the mathematical framework, the choice of the key variables, the physical phenomena which are reproduced. Some models are widely used so that they are generally available in all numerical codes intended for geotechnical applications (e.g. isotropic linear elasticity, elasto-perfectly plastic Mohr-Coulomb, Modified Cam Clay). Figure 1 depicts the essential types of response that different constitutive frameworks can reproduce in loading/unloading paths (σ and ε being a representative stress and a representative strain component, respectively). Some key characteristics are the occurrence of irreversible deformations upon unloading, the stiffness dependency on the stress level, the role of the stress history. Awareness is needed of the importance of these features in the real response of the materials; of course, this importance must be assessed with reference to the analysed case. The user has to ensure that the adopted constitutive models can reproduce the relevant behaviours. The experimental programme must be designed in order to observe and quantify the important features. The test types and the stress paths have to be chosen in order to quantify the material parameters for the initial stress levels (and their variations) that are expected in the considered applications.

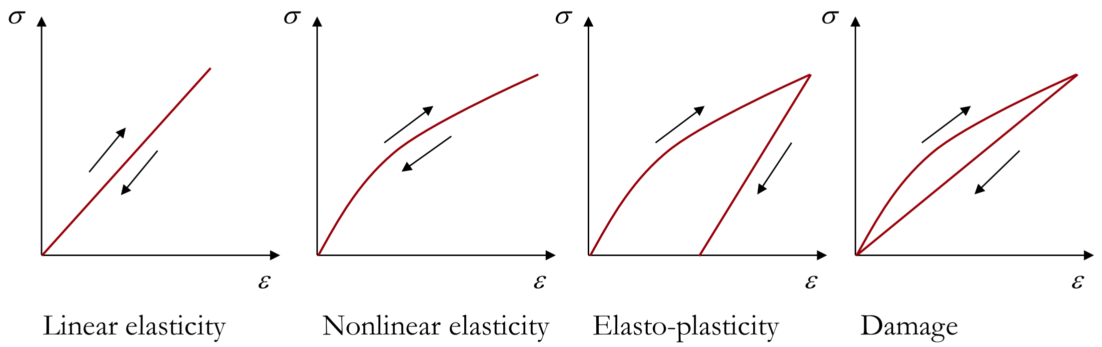

Figure 1: Basic features of different constitutive frameworks for geomaterials.

2.2 Geomechanical parameters from common soil laboratory tests.

The main conventional mechanical laboratory tests are recalled, discussing the set of parameters that can be obtained for the different types of constitutive models previously introduced. Oedometric tests can be planned in order to quantify the stress-dependency of stiffness in both loading and unloading paths. However, oedometric tests do not allow the complete assessment of the stress state, so that – for example - a complete determination of a linear isotropic elastic constitutive matrix is not possible.

When a Mohr-Coulomb type of model is selected, well-planned triaxial tests can provide all the needed constitutive parameters (bulk modulus (K), shear modulus (G), shear strength angle and intercept cohesion, dilatancy). Advantages and limitations for different types of triaxial tests are discussed (Uncosolidated-Undrained, Condolidated-Drained, Consolidated-Undrained). As an example, Figure 2 shows a typical response of a normally consolidated soil in a Consolidated-Undrained triaxial test, in terms of deviatoric stress (q) vs. axial stress (ε_a) and deviatoric stress vs. effective mean stress (p'). TSP is the total stress path; u_0 is the initial back pressure. The interpretation of the test result with a classical elato-plastic Mohr-Coulomb model would allow to identify the strength parameter (M) from the ending point of the effective stress path (ESP). On the contrary, the test would not provide sufficient information to quantify uniquely the elastic parameters of the model: the initial vertical portion of the ESP would correspond to the purely elastic response of the material during the deviatoric loading phase (no volume variation is allowed so that the mean effective stress must be constant); as a result, the bulk modulus cannot be computed and assumption on the Poisson ratio is needed to quantify it.

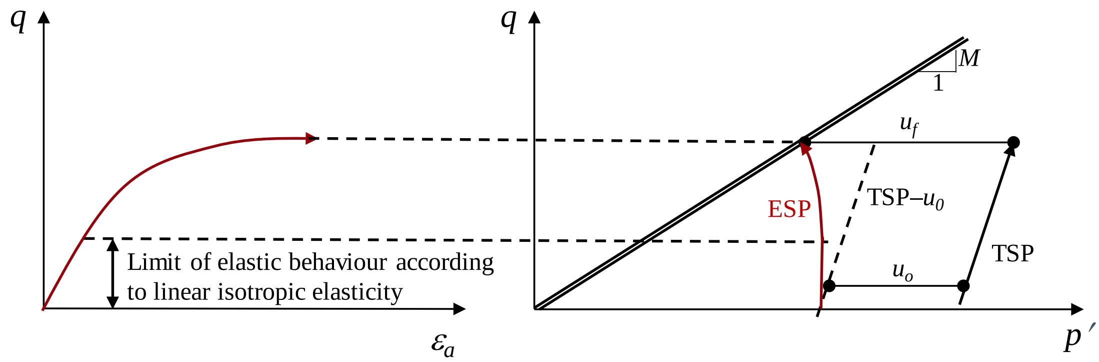

Figure 2: Typical response of a normally consolidated soil in a Consolidated-Undrained triaxial test.

2.3 Design of experimental programs to calibrate more advanced elasto-plastic constitutive models

The analysis of geotechnical systems may require the use of more advanced constitutive laws, which may include behaviours such as non-linear elasticity, accumulation of plastic strains, hardening/softening along deviatoric loading, response at small strains. The increasing complexity of the constitutive models is accompanied by an increasing number of constitutive parameters. Sounded models offer the advantage of assigning to their parameters a clear physical and mechanical meaning. For example, this is the case of the mean effective preconsolidation stress which defines the size of the initial yield surface in a Cam-Clay type of model. In this section, an example is given on how stress paths of triaxial tests can be designed for a complete determination of all parameters for an advanced elasto-plastic constitutive model.

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177	MITTEILUNGEN der GEOTECHNIK SCHWEIZ PUBLICATION de la GÉOTECHNIQUE SUISSE AVVISO di GEOTECNICA SVIZZERA
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In-situ measurements in geotechnical engineering – linking Theory and Practice

Azad Koliji

In-situ measurements in geotechnical engineering - linking Theory and Practice

1 Introduction

During the past three decades, there have been significant developments of in-situ measurements in geotechnical engineering. Continued developments showed that in-situ test can significantly help not only to better understand the results of the laboratory tests, but also, to achieve some geotechnical information which are not accessible by laboratory tests. Overall, the following main reasons can explain the increasing use of in-situ measurements in geotechnical engineering (Mitchel et al. 78, Mayne et al 2009):

(i) Potentially improved efficiency and cost effectiveness compared to sampling and laboratory testing,
(ii) Being more representative data due to larger volume of tested material and larger amount of data,
(iii) Avoiding some difficulties of laboratory testing like sample disturbance and initial condition simulation,
(iv) Possibility of testing some soils in which undisturbed sampling is not easily possible,
(v) Evaluation of both vertical and lateral variability.

However, the main shortcoming of the in-situ measurements is the fact that the test conditions are not very well known and controlled compared to the laboratory tests. Hence, simplification assumptions are usually required for interpretation of in-situ measurements using analytical or empirical approaches. Despite the simplification assumptions and empiricism, in-situ measurements have been widely used and played a key role in geotechnical investigations.

2 In-situ measurements in geotechnics

In-situ measurements are used in different ways and for different purposes in geotechnical engineering. In this regard, three main categories can be considered:

(i) Site characterization and profiling, i.e. classification of materials with similar engineering behavior.
(ii) Measurements of a specific design parameter for empirical, analytical or numerical approaches
(iii) Monitor the ground behavior for construction supervision or back analyses

The first two ones are achieved by in-situ testing; while, the third one is often done by instrumentation and geotechnical monitoring.

Number of in-situ testing methods are used to evaluate the geotechnical design parameters related to stress state, strength, stiffness, and permeability over a wide range of materials. A key point is the choice of an appropriate method in the right material for estimating a specific design parameter.

Table 1 illustrates the applicability of different in-situ tests for estimating geotechnical parameters in different materials. This table is a guideline that combines the appropriateness of each testing method for a specific parameter with its applicability in different types of material ranging from clayey soil to hard rock.

3 Theoretical basis and interpretation

A successful evaluation of geotechnical parameters from in-situ test results requires an appropriate interpretative framework (Mayne et al 2009). In the past years, significant progress has been done in development of rational basis. In general, the results can be

(i) Empirical interpretation: No fundamental theoretical analysis is possible due to uncontrolled condition of loading, drainage and stress state. The interpretation is largely based on empiricism from observation of prior behavior (examples SPT, CPT).
(ii) Semi-empirical solutions: Some analytical relationship can be established between the geotechnical parameters and measurements; however, empirical relations are also used to simplify the variation of in-situ condition such as stress and strain variation (e.g. vane shear test, plate test)
(iii) Sound theoretical solutions: Material parameters are defined in terms of constitutive models and sound theoretical solutions (e.g. self-boring pressuremeter)

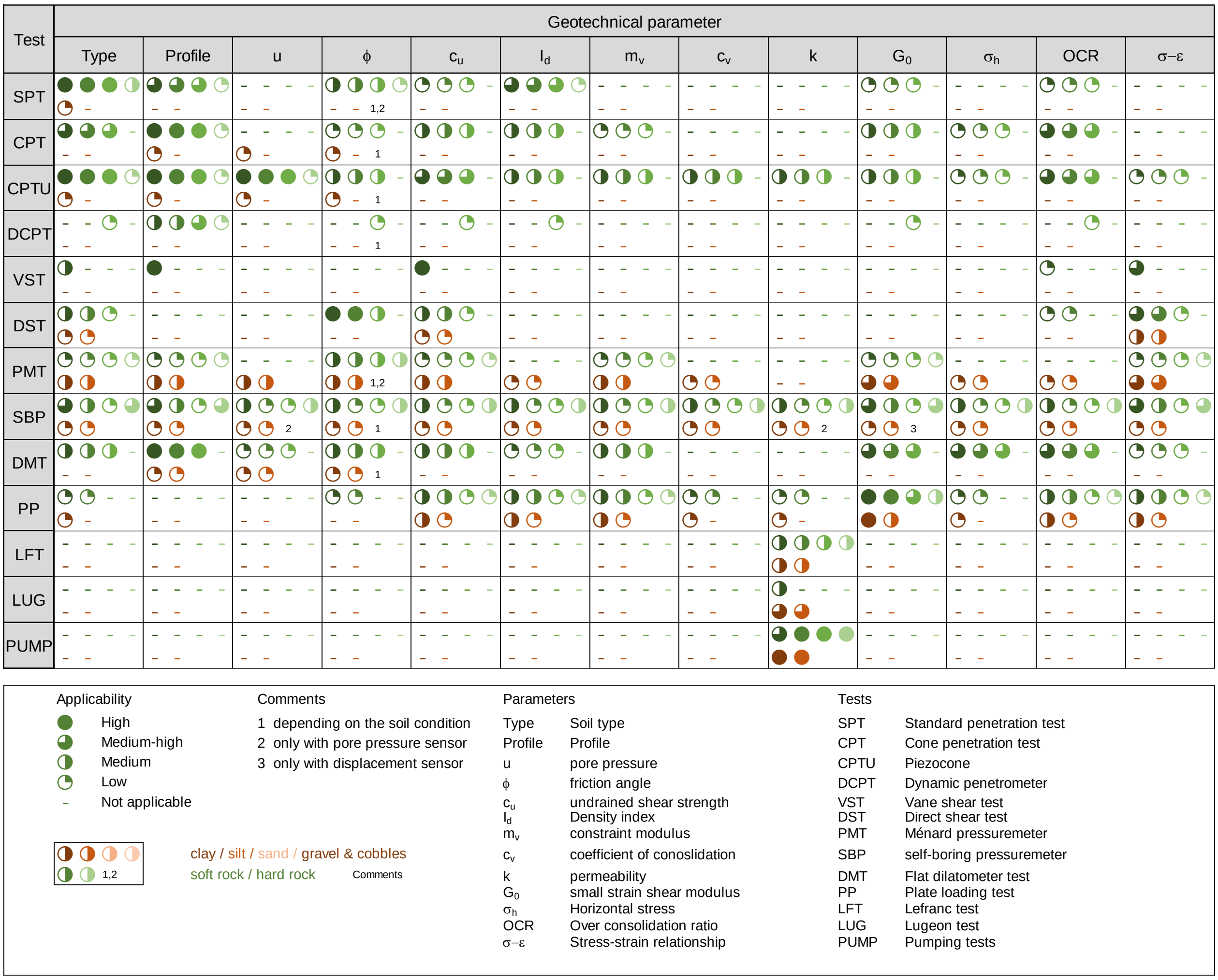

Table 1. Applicability of in-situ tests for estimating geotechnical parameters in different materials (update from Mitchell et al., 1978 and Lunne et al, 1997, Robertson 2012)

4 Practical considerations

Planning of geotechnical investigation depends on the nature and type of project as well as the in-situ condition, and should be in general based on the following criteria:

(i) Available geotechnical information of the site
(ii) Geotechnical risk assessment
(iii) Cost benefit analyses
(iv) Sensitivity of the project
(v) Design requirements
(vi) Appropriateness and availability of in-situ testing methods

The relevant criteria shall be considered when planning geotechnical investigation to make decisions not only about the type of the test but also about the number, location, depth and combination of different methods. In most cases, it is reasonable to foresee an increased investment on the geotechnical investigations and engineering works in return for a potential saving on the construction costs.

5 Conclusion

In-situ measurements plays an essential role in geotechnical engineering. Unlike the laboratory tests, the testing conditions of in-situ measurements are not often fully known neither controlled. Therefore, simplification assumptions are required for an enhanced assessment of parameters within the relevant interpretative framework based on either empirical, semi-empirical and sound theoretical solutions. Despite some simplification, in-situ tests remain advantageous solutions for estimating geotechnical parameters. The paper presents a guideline table for the applicability of in-situ tests for estimating geotechnical parameters in different materials. A successful planning of geotechnical investigation shall consider not only the appropriateness and the applicability of the method but also the project requirements in terms of design and construction.

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177	MITTEILUNGEN der GEOTECHNIK SCHWEIZ PUBLICATION de la GÉOTECHNIQUE SUISSE AVVISO di GEOTECNICA SVIZZERA
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Geophysics for engineers: measurements and modelling

Marie Violay

Federica Sandrone

Geophysics for engineers: measurements and modelling

1 Introduction

La géophysique est l'application de la physique à l'étude de la terre. Les méthodes géophysiques fournissent des données précises sur la structure de la sub-surface - des informations précieuses dans le domaine de l'ingénierie géotechnique. Les données géophysiques sont collectées au moyen de mesures non destructives, le long de la surface terrestre ainsi que dans ou à partir de tunnels et de forages. Ces mesures sont ensuite traitées et transformées en un modèle de sous-sol.
Cet exposé fournit une introduction aux méthodes les plus importantes pour l'exploration et la surveillance des sites géotechniques. Il couvre les principes physiques, les méthodes et les procédures d'interprétation et de modélisation.

2 Géophysiques de sub-surface

Les méthodes géophysiques sont faites pour cartographier les variations spatiales des propriétés physiques du sous-sol ou imager variations temporelles en un site. Elles ne sont pas des substituts au carottage ou aux tests géotechniques en laboratoire, mais permettent dans bien des cas et à moindre cout, de déterminer les propriétés du sol ou de la roche. Par exemple, un gravimètre mesure les variations du champ gravitationnel de la terre et est utilisé pour générer des cartes de densité du sous-sol. Ces mesures améliorent donc la fiabilité, la rapidité et la rentabilité des investigations géotechniques. En particulier elles peuvent servir à :

- Caractériser la sub-surface, i.e. la profondeur du substratum rocheux, le type de roche, la limite de couche, la hauteur et profondeur de la mappe phréatique, les écoulements souterrains, la localisation des zones de fractures, de zones faibles, ou argileuses.
- Connaitre les propriétés des matériaux du sous-sol : rigidité, densité, résistivité, porosité
- Détecter des cavités, des mines abandonnées, etc.
- Localiser des objets enfouis i.e. câble, réservoir de stockage, etc.

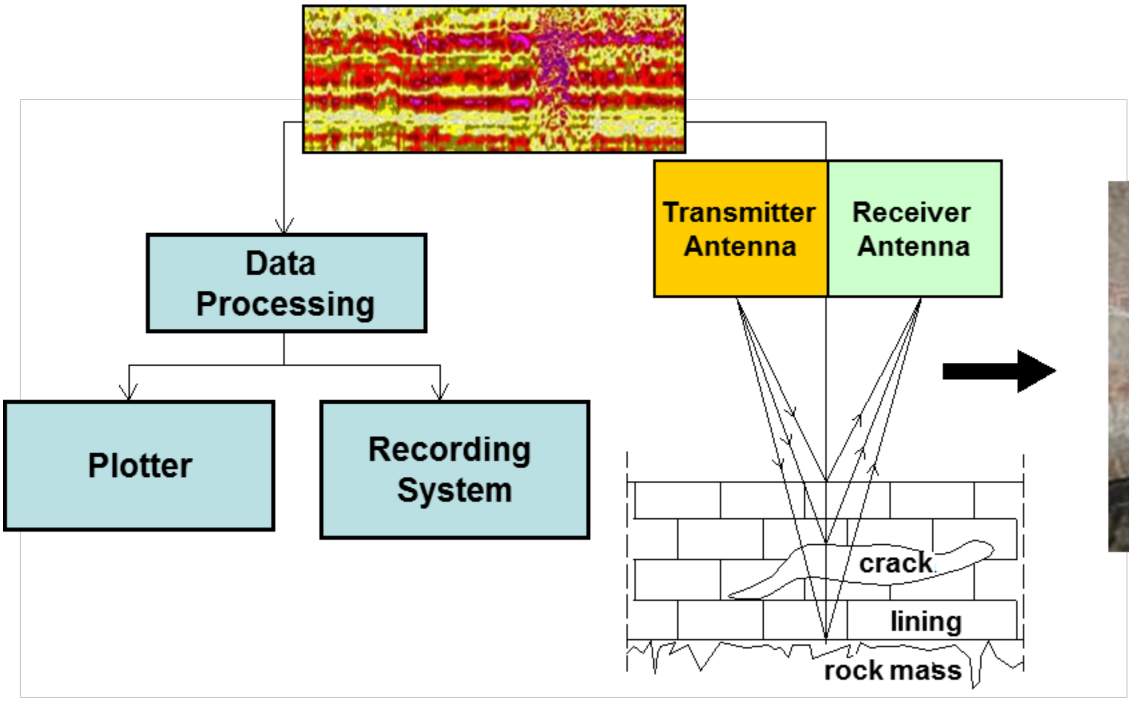
Cependant ces méthodes ne peuvent pas toujours répondre aux exigences et objectifs des ingénieurs. Les cibles d'intérêt peuvent être trop petites, trop profondes pour être résolues ou impossible à imager si les propriétés physique de la cible sont trop similaires à celle de du milieu l'englobant. L'interprétation des mesures peut également être non-unique.
Dans cette première partie de l'exposée, nous présentons les points forts et les points faibles de 5 méthodes géophysiques de sub-surfaces souvent employées en géotechnique : séismique réflexion, séismique réfraction, radar à pénétration de sol, résistivité électrique et potentiel spontané.

3 Géophysique en puits

Afin de corréler les résultats des études géophysiques de sub-surface avec les propriétés géotechniques du sous-sol, l'exploration à partir de puits de forage ou de fosses est nécessaire. Les études géophysiques en forages sont particulièrement utiles car elles permettent un lien direct entre la géophysique et la géotechnique sur site. Dans cette deuxième partie de l'exposée, nous allons discuter les mesures standards en puits, comme les méthodes soniques, de résistivité ou alors les méthodes nucléaires mais également les images de parois de puits, i.e. images optiques, acoustiques et électriques.

4 Cas d'étude Tunnel Torino-Modane : Procédures d'investigation et d'analyse pour la rénovation des tunnels.

Dans cette dernière partie de l'exposée, certaines procédures d'investigation géophysique et d'analyse applicable dans le cas d'investigations des conditions des tunnels seront présentées. En particulier le cas de la rénovation d'un tunnel sur la ligne de chemin de fer entre Torino et Modane sera traité. Ce tunnel présentait des déformations locales des piédroits. Les zones déformées ont été testé grâce aux méthodes de radar à pénétration de sol et de mini-séismique réflexion. Suite à cela, des zones de cavités et de fortes altérations du massif derrière le revêtement ont pu être identifiées et des mesures appropriées de renforcement modélisées. Ceci a permis la rénovation du tunnel tout en évitant le risque d'effondrement de la voûte.



Picture 1: Le Radar à pénétration de sol est une méthode électromagnétique couramment employé pour la surveillance des ouvrages souterrains.

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177	MITTEILUNGEN der GEOTECHNIK SCHWEIZ PUBLICATION de la GÉOTECHNIQUE SUISSE AVVISO di GEOTECNICA SVIZZERA
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Innovation for Safe Slopes and Saving Lives

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Innovation for Safe Slopes and Saving Lives

1 Introduction

Science is often seen as a pure search for truth and knowledge. Engineering is the implementation of science to serve society. But to make science and engineering meaningful, "at some point, you really need to save a life" (trial lawyer Fran Visco at the US Capitol, 2007). In a survey on the state of practice and preparedness for the impact of climate change on landslide risk [1], 21 countries/regions were concerned with four issues: (1) Widespread landslides, including massive landslides, with increased propensity, scale and mobility; (2) Widely affected populations, together with breakdown of infrastructure; (3) Lack of understanding of extreme landslide scenarios, partly due to the scarcity of their earlier occurrence; and (4) Lack of adequate preparedness for extreme landslides, with aggravating consequences. The major needs included: (1) Quantifying the uncertainties; (2) Insights from scenario-based assessments; (3) Application of improved technology and methods; (4) Enhanced communication; (5) Addressing policy gaps, including national and international cooperation; and (6) Formulating strategies for a way forward. This paper looks into the context of landslide risk reduction, and reviews recent technological advances and how they will improve risk mapping, risk assessment and risk management and thereby contribute to the geosciences serving society.

2 Landslide Risk Reduction

Landslide risk reduction requires an understanding of the physical system (the unstable slope) and the context associated with the landslide (i.e. who is endangered, what are the consequences). By identifying needs for information (data), and understanding the methodology required for interpreting the data (models and other tools), there are innovative technologies that can address these needs. In addition to 'engineering', innovative approaches also comprise the 'softer sciences', such as perception and communication. For effective landslide risk reduction (Fig. 1), communication and education are underlying prerequisites for effective risk reduction. Techniques for measuring and sensing variables to provide the data for modelling and forecasting have a variable degree of success. To understand and predict the behaviour of the physical system, theoretical analyses, experience and engineering judgment are key for parameter selection and result interpretation.

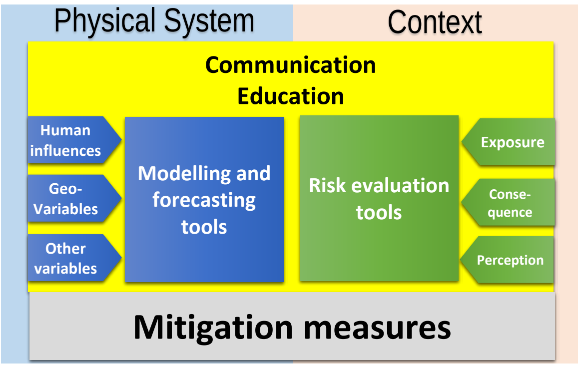


Figure 1: Landslide risk reduction system

2.1 Modelling and forecasting

Tools for modelling and forecasting depend on natural variables and human factors:

- Human (anthropogenic) impact, e.g. land use, construction activities, deforestation, climate change.
- Natural variables, e.g. precipitation, infiltration, run-off, time, soil deformation and strength properties.
- "Non-variables" (invariables), e.g. macro-scale topography, site, geology, base hydrogeology, etc.

2.2 Risk evaluation

Risk evaluation covers a range of issues. Both quantitative and qualitative methods are used, e.g. risk matrices, event trees, indicator-based methods and the quantitative reliability-based methods. Risk evaluation and risk reduction need to consider (1) exposure for population, infrastructure, economic values, coinciding events, changes in land use patterns, development works, demography, as well as political trends; (2) consequences, including loss of life, injury, property and economical losses, disruptions, environmental losses and losses of essential social services; and (3) perception by stakeholders, authorities and the general public.

2.3 Mitigation measures

Mitigation is the implementation of actions to reduce probability of occurrence (hazard) and/or reduce exposure and hence reduce the risk. Mitigation measures include civil works, early warning systems, influencing societal trends, land use planning, political decisions, avoidance, and so on. Both physical and non-physical (e.g. insurance schemes) mitigation measures are important.

2.4 Communication and education

Communication is usually the element that fails in major disasters. Yet, together with education, communication should be the essential and unifying factor of a risk reduction strategy. Technology plays an important role in facilitating education and communication associated with landslide risk. A significant ingredient of successful risk management is the recognition that risk cannot be eliminated completely.

3 Technological advances

3.1 Sensing technology

Sensing technology is deployed to, e.g., warn of danger, follow the evolution of a hazard, monitor the effect of mitigation measures and provide feedback to modify and adjust measures. Safe slope management depends on reliable measurements. The techniques for measuring variables (Fig. 2) can be broadly grouped into two categories: (1) point sensing (measuring one variable in a single location or over a relatively small area) and (2) wide area sensing (measuring parameters over a large area or volume often producing an image), including information gathered from satellites (Fig. 3). The early trends of few sensors for specific purposes, dedicated monitoring systems with high quality expensive electronics and high accuracy measurements have moved to vast quantity of sensors, *ad hoc* data sources, low quality-low cost-low accuracy, but very many, data, sophisticated processing and interpretation tools based on statistics, machine learning and artificial intelligence.

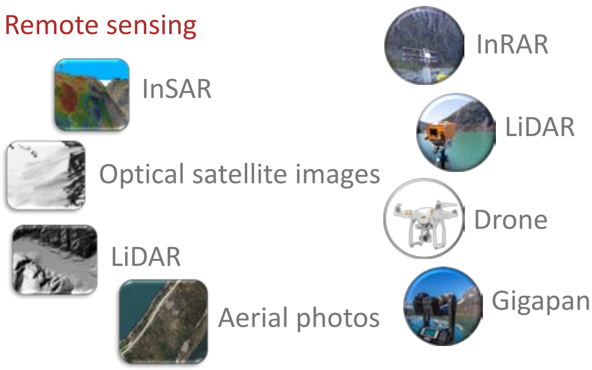

Figure 2: Single point (left) and areal sensing and imaging with remote sensing (radar) technique (right)

Satellites have been operating and collecting data for years. The first Landsat mission was in 1972. Historical data sets are available, and where data in correct format exists it is possible to ‘look back‘ in time at the monitoring locations. The data are collected for many purposes. Using satellite data for slope stability monitoring purposes would provide added value to the large societal investment in the satellite systems. Satellites cover wide areas, repeatedly and at regular intervals. The satellites can access even the most remote areas. Resolution (pixel size) is continuously improving better, and new sensors are constantly being developed. The data are easily accessible, usually at a cost, but often at reduced fee for academic purposes.

Remote sensing techniques (Fig. 4) include drones. The June 11, 2018 issue of the newsmagazine TIME devoted its cover page story to "The Drone Age". The flying technology is transforming the world, from military strategy to life-saving innovations in health care to creating new works of art. In these respect, drones are ideal for reducing landslide risk. When Hurricane Maria hit Puerto Rico in September 2017, it ravaged the island's electrical grid and communications systems, and no communications were possible with mainland U.S. AT&T, with its drone COW ("cell on wheels"), re-established Internet connection and communication.

With any technology, there is an inflection point where it goes from something in the near future to being a part of everyday life. The time has come for drones. Farmers use drones to monitor and spray crops, first responders use drones to coordinate operations and search for missing hikers. Facebook works with drones to beam Internet connectivity to isolated corners of the world, Shell uses drones to keep its network of offshore rigs running smoothly. Artificial intelligence, automation and interconnectivity will enable the simultaneous operation of massive drone fleets, increasing efficiency and greatly expanding their capabilities.

Figure 3: Network of satellites and drone in flight (inset) (Images by NASA and DJI.com)



Picture 4: Newer remote sensing technologies

3.2 Big Data, Machine Learning and IoT

There is a paradigm shift from dedicated, high quality small scale monitoring systems towards large, distributed systems with vast numbers of data. Big Data are "the three Vs": *Volume* (high quantities of data), *Velocity* (often real time), and *Variety* (text, images, audio, video, data fusion), necessitating new tools, new data management and new system architecture for enhanced insight, decision-making and automation. The focus has shifted from designing a system (e.g. specific sensors at specific locations) to observing a response (e.g. streams of data with direct and indirect relationships to the underlying physical processes).

Artificial intelligence (AI) creates technology that allows computers and machines to function in an intelligent manner. Will AI/ML defeat/replace professional engineers in the near future? Machine Learning (ML) works[1] [2], but it will not replace engineering judgment. It helps with added knowledge and may well become standard procedure in a not too distant future. ML is already used to predict several geotechnical aspects, including:

- Landslide displacement from rainfall and dam reservoir water level in Three Gorges area in China.
- Landslide susceptibility and depth to new stratum, e.g. bedrock or sensitive clay layer.
- Cone resistance, pore pressure and side friction from cone penetration tests.
- Soil resistance under installation of suction caisson installation offshore.
- Interpretation of geophysical data and tunnel defect recognition.
- Leak-off pressure (hydraulic fracturing) in rock formations to determine horizontal stress.

Forthcoming interconnectivity will enable using the position, movement and use of mobile phones to help managing disaster response. The position information for mobile phones can help assess the quantity of traffic on

[1] Mastering the game of Go without human knowledge [2]: Machine Learning works, as exemplified in the solution of the game Go. Go is a Chinese abstract strategy board game, in which the aim is to surround more territory than the opponent. Go is very complex, even more so than chess. The machine learning program "AlphaGo" defeated the European Go champion by 5:0 and the second generation "AlphaGo-Master" won all 60 of its completed contests. The 'Machine Learning curve' was as follows:
- Day 0: the machine learning AlphaGo program had no *a priori* knowledge of the game except the basic rules.
- Day 3: Alpha Go Zero surpassed Alpha Go-Master (which had beaten the world champion in 4 out of 5 games (2015).
- Day 21: AlphaGo-Zero surpassed Alpha Go Master and beat the world champion in 3 out of 3 games (2017).
- Day 40: AlphaGo-Zero became the best Go player in the world, solely from self-play without human intervention and historical data.

roadways Trending information on social media and advanced processing using big data and/or machine learning can help predict exposure by identifying where people are likely to be at any given time. However, as data quantity increases, the quality of the data probably decreases, requiring application of big data and machine learning techniques and advanced statistical techniques. The DESR laboratory at the Hong Kong University of Science and Technology [*http://ceyhwang.people.ust.hk/Lab/DESR_lab.htm*] specializes in the applications of the "Geotechnical Internet of Things" (Geo-IoT), Deep Learning and Big Data Analytics for sustainable city development, e.g. critical infrastructure, slope monitoring and open platforms for geotechnical industries to collaborate and share resources.

In a single Internet minute (2013), 360,000 tweets, 3.4 million searches, 500 hours of video, 2.4 million Gigabytes of data, 300,000 photographs, 4 million "shares" and 1.8 billion users have been recorded. In one day, Facebook transfers 25 Terabytes of data. The abundance of data available leads to increased use of statistical methods and machine learning techniques and less use of the "pure science" approach. The IoT allows objects to be sensed remotely across existing networks, resulting in improved efficiency, accuracy and embedded computing system. Experts estimate that the IoT will consist of 30 to 50 billion objects by 2020.

3.3 A Changing Climate

Climate is defined in terms of variations in atmospheric conditions (temperature, precipitation, wind velocity and direction, cloudiness, atmospheric pressure, air humidity and chemistry of the air and contained dust). Landslide and rock slide hazards increase with increased intensity and frequency of rainfall [1]. In a context of (1) more extreme weather and more rainfall, (2) human population increasing at an exponential rate, and thereby increasing vulnerability, (3) human population moving towards coastal areas (*www. Livesciences.com*) and (4) increase in news reporting on natural disasters, especially since the invention of the Internet, the risk associated with climate-driven natural hazards is expected to increase. The survey from 21 countries/regions [1], and especially in permafrost and ice areas, revealed that most of the countries and regions expect increasing landslide hazards and increasing risk to population. The concerns varied greatly, depending on weather patterns, soil types, past landslide history and the nature of the triggers (earthquake, floods, rainfall, etc.). The countries/regions were, however, at different stages of preparedness for climate change impact. Hongkong has come furthest in terms of holistic and systematic assessment of impact of extreme weather events on slope safety through stress testing. All acknowledged the difficulty in preparing realistic scenarios for improved preparedness, but early warning systems and improved communication were seen as the most effective landslide risk reduction measures.

3.4 Example: The BRATT App for Snow Avalanche Warning

The BRATT app ("bratt" is Norwegian for "steep") was designed as a communication/education tool for Norwegian sports enthusiasts planning ski touring in the mountains. The app combines the national digital topographical maps with GIS maps of the terrain. This tool gives colour-coded maps of the areas susceptible to snow avalanche triggering (Fig. 5).

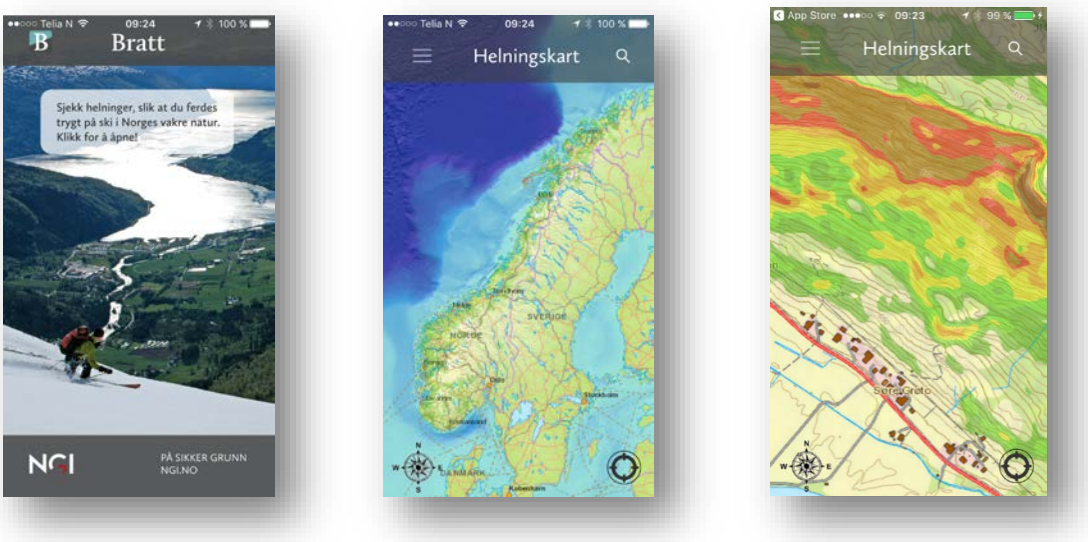


Figure 5: Interface, map of Norway and terrain inclination and dwellings in the BRATT app

The skier can plot a course through the mountainous terrain, while avoiding the worst of the hazards. The app allows downloading the map for offline use. The BRATT app was created to raise awareness of avalanche hazard and risk, communicate areas that were hazardous and educate the population on topography-naked eye observations-avalanche hazard.

4 Landslide Risk Assessment

At the completion of the European SafeLand project, Corominas *et al.* [3] illustrated a scenario-based risk assessment framework (Fig. 6), where the aforementioned components, with enhanced technology, can be included. To modernize hazard and risk mapping and assessment, one should: (1) Improve the methodologies for hazard mapping; (2) Increase the efficiency and quality of data collection; (3) Include the available databases (e.g. drone technology and satellite observations) in the information sources; (4) Facilitate the application of new technology, including AI/ML; and (5) Include climate change/extreme event effects.

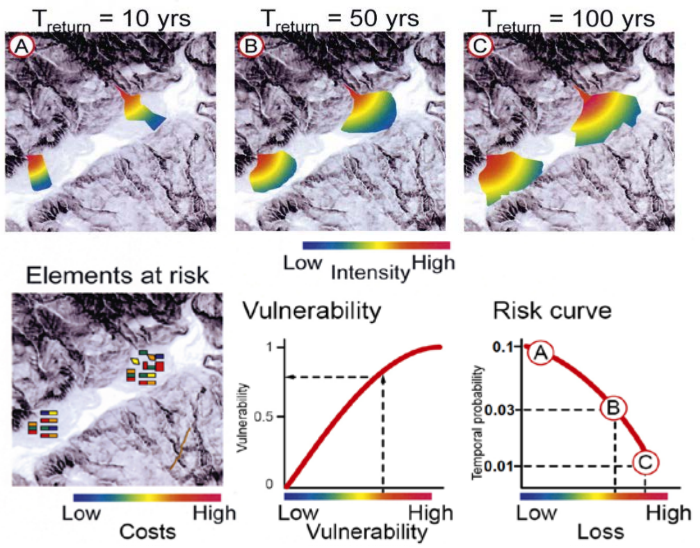


Figure 6: Landslide risk analysis by scenarios [modified [3]

5 Landslide Risk Management

Landslide risk management refers to the coordinated activities to assess, direct and control the risks posed to society by gravity mass flows and landslides. Risk management integrates the recognition of risk and the development of appropriate strategies for the treatment (reduction) of the risk. Risk management needs to be a systematic application of management policies, procedures and practices, including context definition identification, analysis, evaluation, communication, consultation, implementation and monitoring of mitigation measures. A "modern" landslide risk reduction strategy would encompass:

- Avoidance, with land-use planning, warning or alert systems, and public education.
- Prevention, e.g. enforcing slope investigation, design, construction, supervision and maintenance.
- Mitigation, with the implementation of engineering measures to reduce the impact of landslides.
- Preparedness, procedures, human resource management, emergency systems and community training.
- Response, involving humanitarian relief, emergency inspections, safe settlement of evacuated people etc.
- Recovery to bring the affected area back to the normal and do mitigation works.
- Working in a network for climate adaptation.

The newest within landslide risk mitigation is nature-based solutions (NBSs). NBS is an umbrella term for 'ecosystem services', 'green–blue infrastructure' and 'natural capital' [4]. The premise for NBSs is that nature itself is a source of ideas and solutions for mitigating risk from climate-driven natural hazards. Nature's designs are elegant, effective and frugal and adapted to their environment. Water is usually the main culprit in triggering of landslides and debris flows. Planting trees and adapted vegetation to control surface erosion and strengthen the surficial soil layers exposed to excessive amounts of runoff water during extreme precipitation events is one of the few established NBSs for landslide risk mitigation. Several large ongoing research projects in EU's H2020 framework programme, e.g. PHUSICOS [www.phusicos.eu], focus on developing innovative NBSs for landslide control, including comprehensive frameworks for monitoring and verifying the performance of NBSs, i.e. a direct application of Peck's observational method [5]. The monitoring would enable to identify the intensity of extreme events that the NBSs could withstand.

5.1 Cascading Landslides Events

The experience from the events following the 2008 Wenchuan earthquake in China led to a complex hazard-response chain of mass movement processes [6], describing dynamic transitions among six mass movement processes (landslide, rock fall, debris flow, landslide dam, flood and aggraded riverbed) (Fig. 7). The natural geo-phenomena form the outer periphery of the chain. The arrows illustrate the processes and transition that led to increased hazards. A strong earthquake can trigger slides and rock falls simultaneously or consecutively. Under heavy rainfall, the colluvium materials on steep hill slopes can reactivate, move downwards and become channel sediment deposits. The materials in the channels can gradually run out as channelized debris flows under the same or subsequent rainstorms. The materials from a hillslope slide, a rock fall or a debris flow can block a river and form a landslide dam. Flooding due to overtopping occurs if the "natural" dam breaches or fails due to piping. The debris transported by the flood elevates the riverbed. A flood and erosion at the toe of a hillslope can also prompt additional slope failures as the soil materials become saturated or scoured by the flood. The evolution will not stop before the entire system reaches a new balance.

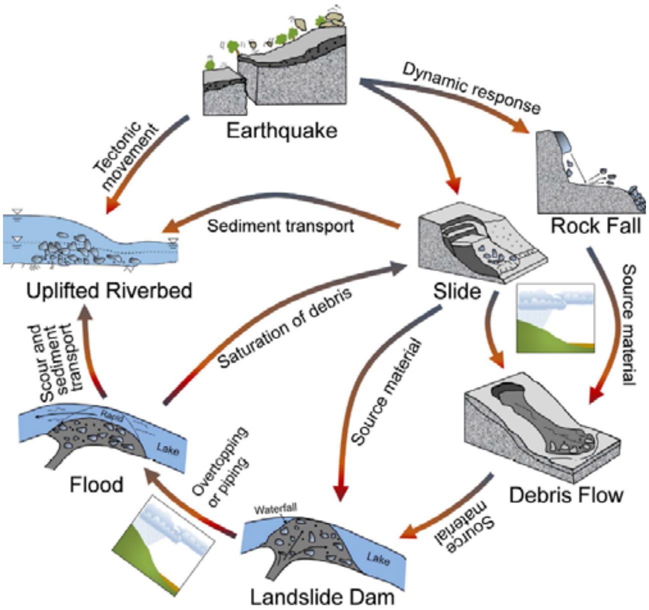


Figure 7: Hazard-response of mass movement processes following a strong earthquake [after 6].

Shortly after the Wenchuan earthquake, slides and rock falls were dominant. With the hillslope deposits evolving into channel sediment deposits, the rain-induced debris flows became dominant 2 to 5 years after the earthquake. As repeated debris flows occurred, the fine particles in the soils eroded due to surface flow. Over time, large particles became exposed, leading to decreased debris flow activity. At present, the debris flow hazards are still active, and debris flows can outbreak in a large storm as a tremendous amount of loose material is suspended on the hill slopes, ready to be eroded and transported. The flood hazards then become dominant. The landform reflects the changes due to the many factors that have occurred over time.

How can one develop a system to keep account of all these events and the cascading and interrelationship among the hazards and the risks posed by each of the events? Stress testing (discussed in the next section) presents an innovative approach for dealing with such complex situations.

5.2 Extreme Events and Stress Testing

Stress testing presents a new solution for complex situations with low probability of occurrence and high consequences. An extreme event may trigger a large number of landslides and debris flows. The consequences can be catastrophic but their occurrence probability is rather low. Stress testing is recognized as an efficient tool for managing the risks of low-frequency high-consequence hazards. The approach was used to evaluate and improve the slope safety system in Hongkong under extreme rainstorms [7]. Stress testing is a targeted reassessment of safety margins of a system under extreme events. It involves testing a system beyond normal operational capacity, often to a breaking point. The stress testing framework (Fig. 8) includes: (1) Identifying future critical rainstorm scenarios under the changing climate; (2) Evaluating the response of the slope safety system to the critical rainstorm scenarios; (3) Assessing the risks posed by the multi-hazard processes; (4) Evaluating the bottlenecks of the slope safety system; and (5) Proposing strategies for improving system performance. The analyses used correlations between landslides and rainfall in Hongkong, layers of digital information (e.g. rainfall intensity, surface geology, slope gradient) and integrated geo-information in a cell-based analysis. A stress test of the response of Hongkong Island to four storms corresponding to 29%, 44%, 65% and 85% of the 24-hour probable maximum precipitation (PMP) was done (Fig. 9, [7]). The number of slope failures and debris flows increased abruptly from 44 to 65% PMP. From 65 to 85% PMP, the impact was less distinct, probably because there was less mobile material left. The testing showed that the extreme storms require an exceptional level of response and new strategies for multi-hazard risk management.

Stress testing is becoming a central tool for the future. In the aftermath of the Tōhoku earthquake and Fukushima Dai-ichi accident, stress tests were imposed by WENRA (West European Nuclear Regulation Association) on all nuclear power plants in Europe in 2011 and 2012.

Stress scenarios	Develop critical rainstorm scenarios under the changing climate (earlier experience)
Impact, system response and risks	Evaluate sizes, locations and impact areas of landslides, debris flows, and flash floods. Evaluate response of «slope safety system».
	Assess consequences of multi-hazard events (No. of people and No. of buildings affected)
Manage-ment strategies	Mitigation: Find “bottlenecks” and develop strategies to improve performance
	Assess effectiveness of proposed strategies Quantify changes in risk profile due to mitigation

Figure 8: Stress testing framework

5.3 The Observational Method

One recurring factor in geotechnical failures is that the construction did not follow the original script. Examples include the Aznalcóllar and the Mount Polley tailings dam failures where, among several factors, the downstream slopes were steeper than intended. These failures reinforce the need for the "Observational Method", a seminal deterministic method in geotechnics [5]. Even in this "Innovation paper", it is recommended that this "old" seminal method be "modernized " with the new technological developments and adapted for statistical analysis with the Bayesian updating method.

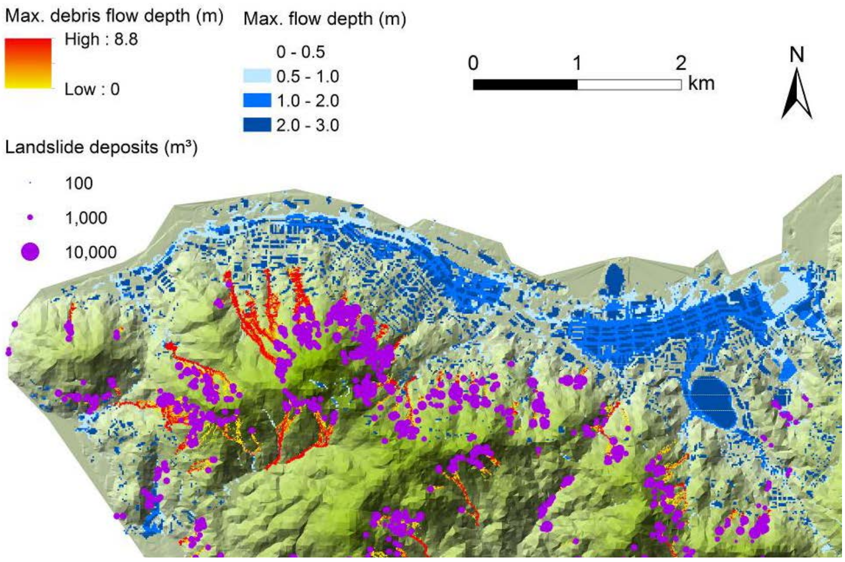


Figure 9: Landslides, debris flows and floods on Hongkong Island, 85% PMP storm

The Observational Method enables monitoring data to adjust a design or early warning system during and after construction (or during observations of a potential landslide). Unnecessary or costly overdesign can be avoided without compromising on safety or the environment. The approach offers a number of advantages for geotechnical projects: (1) the method uses the "most probable" design parameters, as opposed to conservative parameters, and assesses the range of probable behaviours; (2) it sets out the modification in construction or warning to be implemented if the parameters or the behaviour turn out to be less favourable than assumed; (3) it monitors the behaviour of the structure, prescribing the frequency of the monitoring and the trigger levels at which corrective action is required; and (4) it analyses the data and triggers the implementation of contingency plans. The Observational Method could be "modernized" with Bayesian updating. Bayes' theorem allows updating prior estimates with new information. Coupling the Observational Method to risk management, with dynamic updating of the risk and response scenarios would be a viable strategy to make slopes safer, and provide support for risk-informed decision-making. With today's computer and modelling techniques, with real-time internet access to display site data and probabilistic evaluations, the Observational Method reflects the geotechnical engineer's ambition: a coherent approach combining observation, analysis, judgment and risk-informed decision-making, for optimising civil engineering from both cost and safety points of view.

6 Summary and Conclusions

New challenges reside in integrating remote sensing, geotechnical engineering and risk assessment and management into innovative and practical risk management tools to reduce landslide risk associated with natural and man-made geo-hazards. The result will be a risk reduction strategy, with robust, documented and practical risk management tools. Remote sensing techniques can identify and quantify geo-hazards and interconnectivity will allow to rapidly update hazard, vulnerability and risk maps. The Observational Method, coupled with

Bayesian updating will focus on risk on the basis of observations and scenarios. Stress testing will provide improved preparedness, especially for extreme events. A risk management strategy should integrate all aspects, with special focus on communication, education and "lessons learned" from earlier events.

Are science and engineering, and landslide risk reduction, candidates for the growing list of failing institutions that often characterise society (e.g. democratic politics, criminal justice, health care, public education)? On the contrary! Science and engineering have innumerable niches and ways to improve their service to society by taking on board technological innovations. In the future, the most valuable "science and engineering" will be linked to the people having urgent problems. This will incentivize scientists to focus on the solutions than the production of abstract knowledge. Research agendas will search for improved technological solutions. The pure science quests ("free play of free intellects") will not be justified because it has lost sight of the better world they are supposed to help create. There is today a cultural shift in landslide risk reduction (Table 1).

Table 1: Cultural shift required for landslide risk management

From	To
Hazard	Consequence
Response	Preparedness and Risk Reduction
Reactive	Proactive
Science-driven	Multi-disciplinary
Response management	Risk Management
Single agencies	"Everyone's business"
Planning for communities	Planning with communities
Decision-making	Risk-informed Decision-making

As a closing perspective, science and engineering helps us to predict hazards. Knowing the hazards and the risk helps us make risk-informed decisions, and the consequences of landslide hazards can be reduced with innovative technology. Human factors can turn unexpected events into catastrophes, and tomorrow’s disasters are exponentially building on the disasters of today. Focus on disaster risk reduction is required because the social costs tend to persist over a person’s lifetime, whereas material costs are a one-off. The time to innovate for improved landslide risk reduction is now. In the future, we need to show that we make «risk-informed» decisions (ISO2394:2015 [8]).

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Recent Thinking on Design for Situations dominated by Water Pressure

Brian Simpson

Recent Thinking on Design for Situations dominated by Water Pressure

1 Introduction

Water is the enemy of geotechnical engineers. It acts as a heavy load actin vertically and horizontally, it has no shear strength and its pressure reduces the strength of ground materials. It is often implicated in serious geotechnical failures and, especially if it is free to flow, its impact can be devastating. So it is extremely important that rational and safe procedures are adopted when considering design situations in which water and water pressure play a significant role. Yet a consistent method of prescribing safety in such situations has proved to be illusive.

The committee responsible for Eurocode 7 (EC7) commissioned a paper on this topic written by three authors from different backgrounds (Simpson, Vogt and van Seters [1]) and subsequently set up an Evolution Group, EG9 [2], to make proposals that have been adopted, in part, in the latest draft of the revision currently being developed.

Three aspirations are critical to the discussion presented here:

a) To maintain compatibility between calculations for the ground and related structures, ensuring that both can accommodate the same design situations and the same assumptions.
b) To carry out geotechnical design using sound principles of soil mechanics, making them explicit and intuitively reasonable as far as possible.
c) Consistently with (b), to retain equilibrium of forces and stresses throughout the geotechnical calculation.

2 Importance of the topic – some illustrations

The author first became aware of the difficulties in prescribing safety in relation to water pressures early in his career when working on the design of the Dubai Dry Dock [3]. Figure 1 shows a cross section through the dock gates, which relied on struts to withstand the water pressure from the sea, the load being transferred into the ground by thrust blocks. During the design there was much discussion about how to guarantee the safety of the struts and thrust blocks, given that the sea level was fairly well known, with only a small tidal variation: how should a factor of safety be calculated and what should its value be? In the event, the struts and thrust blocks did not fail, but sadly, some years later, there was a failure of a gate when the dock was in use, which resulted in the deaths of many workers in the dock as the sea flowed in. The power of water and the importance of safe design were clearly demonstrated. This is particularly important where water is free to flow, as has also been seen in several major dam failures.

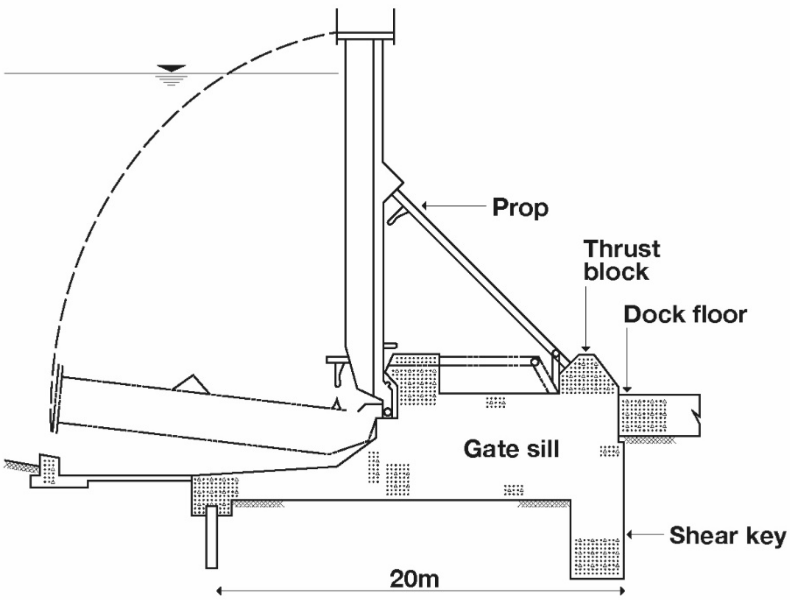


Fig. 1 Dubai Dry Dock – cross section of gate

Figure 2 shows a cross section through a temporary earth cofferdam used to facilitate construction of the dock. Figure 2(a) shows its intended configuration, while Figure 2(b) shows what actually resulted from pumping dredged fill into place. When a small disturbance was made to the unexpected layer of lower permeability material a rapid outflow of water occurred, eroding the cofferdam at an alarming rate. The situation could easily have been catastrophic, leading to fatalities. Fortunately, on this occasion, plant and materials were immediately available to carry out repairs, and disaster was averted.

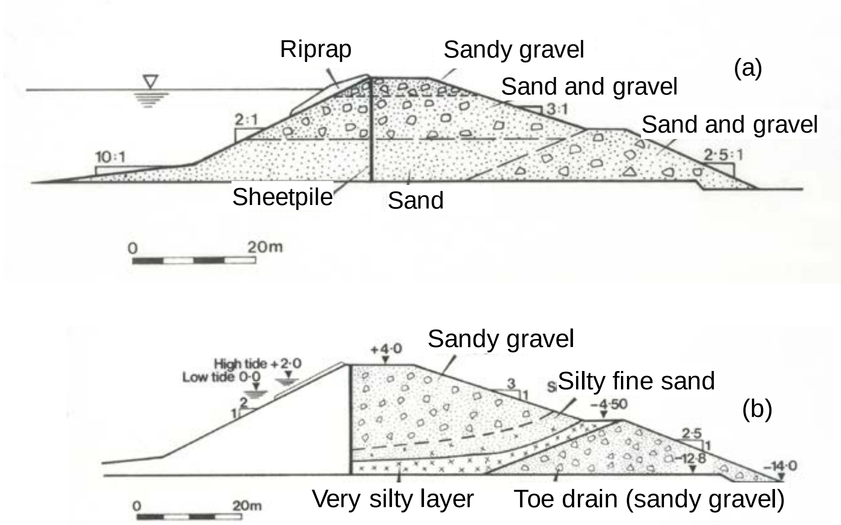


Fig 2. Dubai Dry Dock – earth cofferdam (a) intended section (b) as built

Figure 3 shows an excavation in Singapore reported by Davis [4]. In this case the base of the excavation became a quagmire, unable to support plant or personnel, due to uplifting water pressures beneath the decomposed granite, which became more permeable with depth. Pressure relief was necessary to facilitate completion of the excavation.

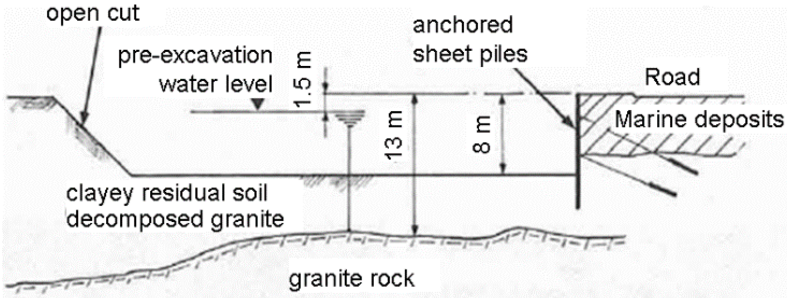


Fig 3. Section through an excavation in decomposed granite (after Davies 1984)

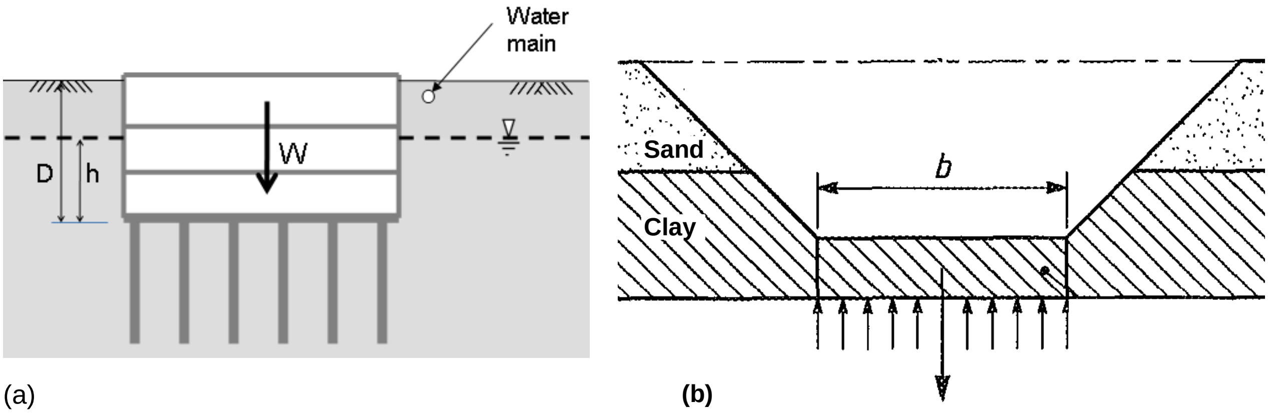
3 Mechanisms

Four main mechanisms can be identified to classify failures that can occur in geotechnical situations owing to water pressure:

- Elevated water pressures lead to reduction of effective stress and thereby loss of frictional shear strength in soils. This is a common cause of sliding failures, including slope stability problems.
- Uplift occurs when a structure or a block of low-permeability soil is lifted by underlying water pressure (Fig. 4).
- Hydraulic heave occurs when water flows through a zone of soil, disturbing it by seepage forces owing to a high upward hydraulic gradient (Fig. 5).
- High hydraulic gradients, in any direction, can cause internal erosion or piping failures. In this case, finer particles are transported through the matrix of coarser particles. Piping is a form of internal erosion in which the displacement of particles becomes concentrated at discrete locations or “pipes” within the soil, allowing a local increase in permeability and concentration of water flow.

Each of these mechanisms can initiate a failure and it is common that once the problem occurs other mechanisms are also involved.

Uplift and hydraulic heave can occur in quite similar circumstances and are sometimes difficult to separate. For example, the failure shown in Figure 3 was probably a combination of the two, and either mechanism could occur in the geological situation shown. The final result, after initial disturbance of the ground, could appear as piping.



(a) (b)

Fig 4. Uplift (a) of a structure (b) of a block of low-permeability soil

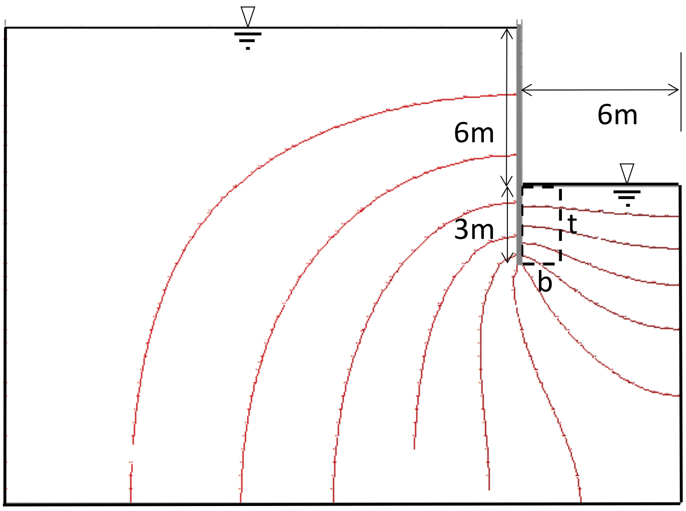


Fig 5. Hydraulic heave

4 Difficulties in prescribing safety conditions for water pressures

4.1 Factoring water pressures leads to logical problems

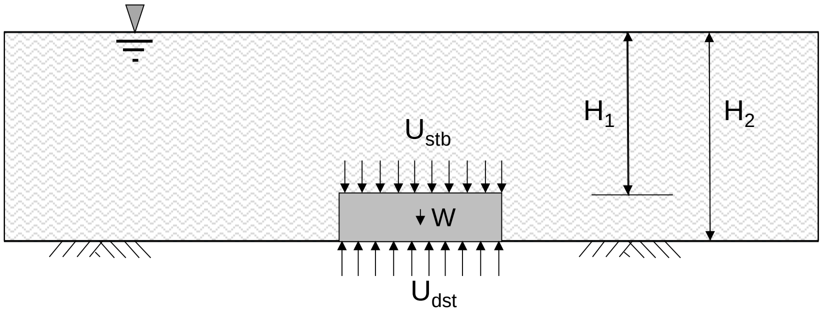
Factoring water pressures leads to logical problems, irrespective of whether they are applied as total or partial factors, as discussed by Simpson et al [1]. For this reason, EG9 proposed that factors of safety should not be applied to water pressures or, generally, to quantities calculated from water pressures. An exception was made for structural action effects (bending moments etc) caused by water pressures, since it was recognised that such factors have a wider role in structural design.

4.1.1 “Brick” subject to buoyancy forces

A simple illustration of one problem can be seen by considering a block of structure – a "brick" – placed at the bottom of a water column and asking what is the factor of safety against the brick moving upwards (Fig. 6, which shows the symbols used here). The factor of safety could be calculated as the ratio of the stabilising (downward) forces to the destabilising (upward) force, as would be normal in conventional geotechnical design: $(W + U_{stb}) / U_{dst}$. Alternatively, it could be calculated as $W / (U_{dst} - U_{stb})$. Figure 6 illustrates the fact that these two formulations give different values for the factor of safety; in the calculations it is assumed that the density of the brick is twice that of water. Furthermore the first formula gives different factors depending on the water depth, whereas the latter is independent of water depth.

In most situations it is reasonable to suppose that the danger of the brick becoming unstable would not be affected by the water depth, so the latter formulation appears to be preferable. This is an illustration of the "single source principle", which states that if disturbing and stabilising forces come from the same source (statistically, if they are strongly correlated) they should be treated together with the same factor applied to both. In this case, the two forces are both water forces, coming from a continuous body of water.

A more detailed review of this problem is given by Simpson et al [1], in which the "brick" is considered as an anchor block restraining a vertical tie force.



	H_1=4m, H_2=5m	H_1=19m, H_2=20m
$F = (U_{stb}+W)/U_{dst} = (H_1\gamma_w+W)/H_2\gamma_w$	F = 1.2	F = 1.05
$F = W/(U_{dst}-U_{stb}) = W/(H_2-H_1)\gamma_w$	F = 2.0	F = 2.0

Fig. 6. Brick under water

4.1.2 "Unreasonable" piezometric levels

Figure 4(a) shows a basement subject to uplift pressures. If the water table is close to ground level, factors of safety applied to water pressure readily imply piezometric levels beneath the basement slabs above ground level, which may be physically very unreasonable. Engineers, particularly geotechnical engineers, generally resist designing for anything that is physically unreasonable, so a factor of safety applied in this way might well be ignored, which could be dangerous if there is a real need for it not obvious to the designer. Alternatively, a factor of safety applied for the purpose of structural design (eg 1.35) might be found to imply that the basement is unstable by buoyancy, requiring more weight than is reasonably necessary.

In other cases, factoring water pressure may have a negligible effect on the design and so add nothing to safety. This would be the case, for example, if the water level in Fig. 4(a) is very close to the slab level.

4.2 Dependency on permeability

In the example shown in Figure 2, and discussed above, a small change in permeability led to a large change in the flow of water through the soil, and consequently to water pressures. Similarly, the tendency for the permeability of the decomposed granite in Figure 3 to decrease upwards towards the excavated surface (as it became more weathered at shallower depth), contributed to the instability on that site.

Simpson and Katsigiannis [5] considered the problem shown in Figure 5, modelling the soil in finite element analysis as a sand with $c'=0$, $\varphi'=35°$. The distributions of permeability shown in Figure 7 were chosen so that the range was no more than 5:1, at least within a depth below excavation of 2t, where t is the penetration of the wall below the excavation. It was considered that such a low ratio might not be noticeable in ground investigation, so the material might well have been modelled as uniform in a seepage computation. To make comparisons, they used the definition of factor of safety proposed by Terzaghi [6]: $F_T = G'/S$¸ where G' is the buoyant weight and S is the upward seepage force on the rectangular block of ground shown in Figure 5, with $b=t/2$. They showed that if the material was incorrectly assumed to be uniform, with a conventional factor of safety F_T = 1.5, the actual factor of safety dropped to 1.17 for the case where the permeability of the soil below the tip of the wall was half that of the soil above. For all the other cases shown in Figure 7, the soil became unstable owing to hydraulic uplift.

These examples show that relatively small variations in permeability can cause very significant changes to the distribution of water pressure in the ground, with major implications for stability and safety. Use of conventional factors of safety might not compensate for this, so it is very important that designers examine the implications of variations that are possible but might not be apparent in ground investigations. Simpson and Katsigiannis noted that a very large range of values is found in the literature for F_T, in most cases with no obvious explanation why different authors have chosen different values (Table 1). This might reflect various authors' appreciation of the ground conditions they had encountered and it tends to indicate that quoting a fixed value for all ground conditions is unwise.

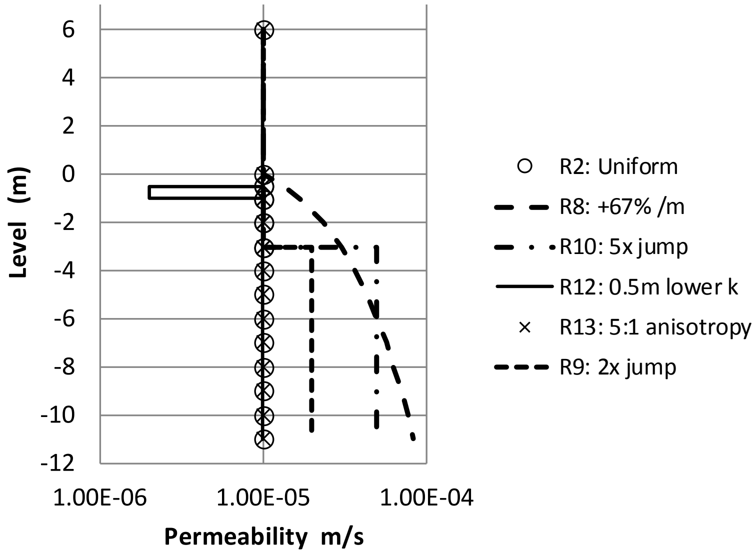


Fig. 7. Distributions of permeability considered by Simpson and Katsigiannis [5]

Table 1. Published values for Terzaghi's factor of safety FT.

Publication and any limitations		Values
Williams B P & Waite D [7] for clean sands		1.5 to 2.0
Kashef, Abdel-Aziz Ismail [8]		4 to 5
Harr, M E. [9]		4 to 5
German practice (DIN 1054/A2 [10])	– unfavourable soils – favourable soils	1.9 1.42
Swedish practice (Ryner et al [11])	– coarse soils – silty material	1.5 2.5
Dutch practice (van Seters [12]		2.8
Das [13], quoting Harr [9]		4 to 5

4.3 Probabilistic approaches

The difficulties in using factors of safety for water pressures, whether partial or total, and the problem of uncertainty of permeability distribution might be thought to suggest that probabilistic approaches will provide an answer. In the author's view, the format of thinking required by probability approaches, recognising uncertainty and considering the range of possible behaviour explicitly in design calculations is, indeed, very valuable. However, there are some issues that mean that numerical probability analysis probably does not provide a panacea for design situations dominated by water pressures.

4.3.1 Lack of statistical data

Engineers working on designs for river, reservoir or marine situations sometimes have extensive databases of water levels as they are affected by weather and tides. They are able to assess return periods for severe events and so make rational predictions of probabilities of adverse water levels. However, in many other situations, the only data available consist, at best, of a few observations of ground water level taken over a short period of time, often not even as much as a full year. These do not, on their own, provide enough information for a useful statistical analysis. Furthermore, in many urban situations future human activity, errors or failure in infrastructure are more likely to lead to severe water pressures than are normal seasonal fluctuations; these are not normally included in a statistical database.

4.3.2 Unwillingness of engineers to make assessments of probability

The author's observation is that many engineers are unwilling to make their own, subjective, assessments of probabilities of severe design conditions. Two main reasons for this are suggested.

Firstly, codes and standards tend to talk of "the probability" of an event in terms that suggest that a reliable statistical database is expected to be available. For example, the current draft of the revision to Eurocode 7 uses the words "design values of groundwater pressures for ultimate limit states shall have a probability of exceedance lower than …". Wording such as this seems to allow no possibility that this might be an assessment made by the designer, or agreed between the various parties to the design – design engineer, regulatory authorities, owner, other specialists, etc.

Secondly, the author considers that engineers tend to underestimate the value and extent of the information they actually have. This is because it probably does not exist in the form of a statistical database, its quality of documentation might be variable; it might be derived from their education or a previous experience. While information of this sort might lack precision, it is nevertheless very valuable in the absence of a more detailed database. In fact, it has been the basis of most engineering design in conventional practice. It is important, however, that every effort is made to have such information documented and transparent so that it can be shared and appraised by the parties involved in design and checking.

Designers' assessments of probabilities, based on diverse and sparse data, will necessarily be imprecise. However, in the author's view it is better to give a clear target for this assessment in numerical form than the alternative of using qualitative words such as "cautious" or "very unlikely", which give the designer an unclear target for the assessment, further adding to the overall uncertainty.

4.3.3 Variables with fixed limits – the issue of robustness

When safety is prescribed by use of partial factors or by probabilistic calculations, it is usual to concentrate attention on a small number of leading variables, taking values that have low probabilities of adverse exceedance. It is then assumed that any other secondary variables will have less effect and so possible adverse values for them are covered by the severe values adopted for the lead variables.

In considering statistical/reliability methods in relation to Eurocodes, a working group of ISSMGE committees TC205 and TC304 discussed the issue of Robustness [14], defined in ISO 2394 [15] as the "ability of a structure to withstand adverse and unforeseen events (like fire, explosion, and impact) or consequences of human errors without being damaged to an extent disproportionate to the original cause". The report of the working group suggests that in order to provide for unforeseen events it is necessary to have safety margins that relate more to the overall magnitude of the construction than to the uncertainty of the identified leading variables. Thus, in cases where there is very little uncertainty in the leading variables, inadequate safety margins might be provided by probabilistic methods that concentrate on their variability.

Structures that retain water often have a maximum possible water level at which they are overtopped, preventing further build-up of pressure. The probability of reaching this maximum might in some cases be quite high, but the probability of it being exceeded is essentially zero. Thus, selecting a design value for water pressure that has a very low probability might imply no change to the design, providing no provision for secondary variables and no increase in robustness in relation to unforeseen events.

5 Possible approaches

The difficulties described in the previous section place some limits on each of the approaches to prescribing safety that have been considered. In this section, the author attempts to suggest some practical ways forward. The concentration here is on design for ultimate limit states (ULS) at which a structure would become dangerous or very badly damaged.

5.1 Use of partial factors

It was argued in 4.1 above that application of partial factors to water pressures is fraught with difficulties and should be avoided. Codes of practice for structural design, however, normally assume that all actions have been factored significantly beyond their likely range, and this influences values adopted for material or resistance factors for structures. As a result, bending moments, direct and shear forces used in structural design should be factored even if they are caused by water pressures. This can be done at the end of the geotechnical calculation, for which it constitutes "action effect factoring".

Application of partial factors to actions (loads) or action effects always defies equilibrium in some way, so it is important to identify a point where this can be done without requiring calculation models that are physically unreasonable. It is submitted that this is achieved by designing structural members for factored action effects. On the other hand, factors applied to water pressures, or to forces derived from water pressures and used in geotechnical calculations often defy equilibrium in a physically unreasonable manner and cause a loss of equilibrium in the middle of the geotechnical calculation.

5.2 Adjustment to water levels

Water pressure, a stress, can be characterised by the corresponding piezometric level, a geometric quantity. Partial factors are usually not appropriate for geometric quantities, but instead additive safety margins are sometimes used. This approach is commonly adopted in design of river walls and embankments, where river authorities often specify a level in excess of the maximum predicted to occur. For example, in the Thames Estuary, east of the Thames Barrier, flood defence levels, allowing for climate change, are planned to be roughly 1m above the level predicted for 2100 with 0.1% annual probability, or roughly 0.5m above the level predicted with 0.01% annual probability [16].

In the review paper prepared for the EC7 committee [1], Simpson et al studied the number of piles needed to hold down the basement slab shown in Figure 4(a). They recommended that a good alternative to factoring water pressures beneath it was to apply an additive margin to the water level. This is particularly important if the expected or characteristic level is near the bottom of the basement, giving very little water pressure; for this case applying a factor to water pressure would provide no safety against an unlikely but adverse higher water level.

The two examples above relate to situations of free or hydrostatic pressure, unaffected by the permeability of the ground. In situations where water pressures (or piezometric levels) are governed by seepage flows, distribution of permeability introduces a further major uncertainty. This typically happens in design for stability of slopes, where water pressures are governed by permeability distributions as well as by external supply from rainfall or other sources. In such cases it is normal for the designer to make an assessment of piezometric levels, which should be intentionally pessimistic, and use these as a basis for design calculations, without application of factors to water pressure. Effectively, a safe margin is being applied to water levels, though these are not specified by external authorities.

5.3 Assessments based on probability

Eurocode 7's EG9 considered all the options discussed above and recommended that the most suitable method of deriving design values of water pressures for ULS design is that they should be directly assessed by the responsible engineers, requiring them to have a very low probability of exceedance. This is the approach favoured by the author. It applies not only to free or hydrostatic water levels, but also to water pressures affected by seepage, requiring the engineer to consider severely adverse distributions of permeability.

It is normal that ULS design values for actions represent situations that are very unlikely to occur. EG9 recommended that ULS design values for water pressures should be specified to have a 1% probability of exceedance within the design life of the structure. If the design life is 100 years, this could be re-stated as a return period of 10000 years, on the basis that the term “return period” is defined as the inverse of the annual probability of occurrence. Some engineers have reacted adversely to this, and the author believes this may be because they are confused by the concept of return period, leading them to contemplate the next 10000 years. What will happen in the next 10000 years cannot be known and is also irrelevant to designs being undertaken. What matters is the probability of a severe event within the lifetime of the structure, and for that a figure of 1% is easier to understand and is thought to be consistent with other types of variable actions.

In situations where an adequate database is available, the required remote probability can be assessed on a purely statistical basis. However, as noted above, when databases are available they are only likely to apply to free or hydrostatic water levels. Where there is little data, or where water pressures are affected by permeability distributions, the engineer’s expertise is required to assess the value with a remote probability, taking account of the effects of geology, hydrology, extreme weather and, in urban situations, infrastructure including the effects of items such as leaking water mains and changes to water extraction regimes.

In principle, this design process could be carried out in a mathematical reliability framework. However, the complexity of this, if thoroughly undertaken, would be beyond the mathematical competence of almost all practising engineers, and the author submits that it is more important to work carefully through all the important influences than to carry out a strict statistical analysis, which will almost certainly be over-simplified.

5.4 Hydraulic heave – what is the limit state?

In a situation such as that shown in Figure 3 or 5, if the ground starts to displace upward either as a block or as a mass of particles flowing in the water the situation is clearly dangerous – an ultimate limit state. However, before this occurs the ground could already be dangerous if it is impossible to walk on the surface, to drive vehicles or to place moderate loads. The author submits that such states are also ultimate limit states, and their avoidance requires that some quantity of positive effective stress is maintained in the ground. A rational criterion for this problem would therefore be a minimum effective stress requirement.

Skempton and Brogan [17] studied the effects of upward flow of water through a range of “stable” and “unstable” sandy gravels. They found that for stable, well graded, material disturbance only occurred when the hydraulic gradient was high enough to reduce the calculated effective stress to zero. However, for unstable, poorly graded materials disturbance occurred at much lower hydraulic gradients. It is arguable that this should not be considered an issue of hydraulic heave but one of internal erosion, governed by material-dependent critical hydraulic gradients. Nevertheless, the author suggests that it is likely that this variation between materials has affected the choices of factors of safety for hydraulic heave calculations shown in Table 1, and this is explicitly recognised by some authorities who have specified different values dependent on the grading of the material.

The author therefore suggests that a good criterion for hydraulic heave would be to require a minimum for the ULS design value of the vertical effective stress, and that this value should be dependent on the type of material. For example, the requirement at a depth z below the free surface could be expressed as:

$$\sigma'_{v;d} \geq \alpha\gamma' z \qquad (1)$$

in which α is a material-dependent factor ensuring a minimum effective stress, not a safety factor γ intended to make provision for uncertainty. The requirement expressed in this form shows the minimum effective stress increasing continuously with depth. This is somewhat arbitrary and might not be necessary at greater depths.

The values shown in Table 2 are suggested as roughly consistent with international practice; equivalent values of F_T are shown for comparison. They are shown as characteristic values (α_k) to which a nationally determined partial safety factor could be applied as a modifier if required.

Table 2. Suggested values for parameter α

	γ kN/m^3	α_k	F_T
Dense sand (Germany)	20	0.18	1.4
Loose sand (Germany)	18	0.36	1.8
Silty, layered sand	18	0.54	2.5

Alternatively, the same result could be achieved by specifying maximum hydraulic gradients or a reduction factor on the buoyant weight density of the soil, though these tend to obscure the primary aim of maintaining a minimum allowable effective stress. In particular, a factor that appears to imply that the problem is uncertainty of density could be confusing.

The requirement in the form of Equation (1) can be used in a variety of ground conditions, including slope stability problems, and can be applied as a post-analysis check on seepage calculations such as finite element runs. Factoring weight density of soil or water makes these analyses more difficult, while use of the factor of safety F_T is limited to the specific situation of vertical seepage towards a horizontal ground surface.

5.5 Internal erosion and piping

Whereas hydraulic uplift and hydraulic heave are related to a balance of vertical forces, internal erosion and piping are caused by excessive hydraulic gradients or seepage velocities. Critical hydraulic gradients are material-dependent. Although these are much studied in the field of dam engineering [eg 18], geotechnical texts and standards often acknowledge the phenomenon but give little guidance. Some methods of determining critical hydraulic gradients are given by BAW [19] and the International Levee Handbook [20].

6 Eurocode 7

The current version of Eurocode 7 was published in 2004, e.g. [21]. A revised version is currently being drafted [22], intended to comply with a revised EN 1990 [23]. These drafts are available to Eurocode 7 Working Groups and so are widely circulated, but they are not formally published.

6.1 Eurocode 7 (2004)

The current version of EC7 says in 2.4.6.1:

> (6)P When dealing with ground-water pressures for limit states with severe consequences (generally ultimate limit states), design values shall represent the most unfavourable values that could occur during the design lifetime of the structure. For limit states with less severe consequences (generally serviceability limit states), design values shall be the most unfavourable values which could occur in normal circumstances.
> (8) Design values of ground-water pressures may be derived either by applying partial factors to characteristic water pressures or by applying a safety margin to the characteristic water level in accordance with 2.4.4(1)P and 2.4.5.3(1)P.

Although less specific in terms of probabilities, paragraph (6) is generally in line with the probabilistic approach described above in 5.3, implicitly relying on engineering assessment where adequate databases are not available. This is stated as the defining Principle, for which paragraph (8) provides an Application Rule – one way of achieving the principle. Paragraph (8) allows the possibility of factors being applied to water pressures or additive margins to piezometric levels.

6.1.1 EC7 Equation 2.9 for hydraulic heave

EC7 represents the requirement for stability against hydraulic heave by Equation 2.9, which is presented in two forms:

$$u_{dst;d} \leq \sigma_{stb;d} \quad (2.9a)$$

$$S_{dst;d} \leq G'_{stb;d} \quad (2.9b)$$

In (2.9a) the requirement is expressed in terms of pore water pressure u and total vertical stress σ, whereas in (2.9b) it is expressed in terms of seepage force S and buoyant weight G' of a rectangle such as that shown in Figure 5. Both forms of the equation use *design values* of parameters (subscript d), already incorporating safety, so no further factors are shown in the requirements. The subscripts *dst* and *stb* refer to destabilising and stabilising effects. For simple cases such as Terzaghi’s block the two forms are mechanically equivalent provided only design values are used.

Annex A of EC7 provides values for partial factors to be used for hydraulic heave, $\gamma_{G;dst} = 1.35$ and $\gamma_{G;stb} = 0.9$. But the code does not state what quantities are to be factored. Unfortunately, some readers of EC7 have interpreted the equations to mean:

$$\gamma_{G;dst}\, u_{dst;k} \leq \gamma_{G;stb}\, \sigma_{stb;k}$$

and

$$\gamma_{G;dst}\, S_{dst;k} \leq \gamma_{G;stb}\, G'_{stb;k}$$

Here, the subscript *k* refers to characteristic values of the parameters. Orr (2005) pointed out that if the two equations are used in this way they can lead to markedly different results for the same values of $\gamma_{G;dst}$ and $\gamma_{G;stb}$. Simpson (2012) argued that this is a misunderstanding of the EC7 requirement, and in particular of the concept of design values, and proposed that if partial factors are to be used in this context they should be applied to *excess* water pressures only, not to the hydrostatic component. This would enable design values to be derived in a compatible way for the quantities in Equations 2.9a and 2.9b, so that the same result is obtained from either formulation.

As noted in 4.2 above, however, application of factors of safety in this situation is problematic, particularly because ground water pressures are strongly affected by distribution of permeability and the criteria should be material dependent. One further problem of this formulation, as with Terzaghi's analysis, is that it is only applicable to one very specific situation of upward flow towards a horizontal surface. In practice, more complex situations are encountered, including flow beneath sloping surfaces in embankments and cuttings.

6.1.2 Hydraulic uplift

For hydraulic uplift (Figs. 3 and 4) EC7 currently places a factor of 0.9 on the stabilising weight and a factor of 1.1 on the destabilising water pressures. The UK National Annex for EC7 [24] recognises that the factor on water pressures, on its own, may be inappropriate or ineffective:

> The partial factor specified for permanent unfavourable actions does not account for uncertainty in the level of ground water or free water. In cases where the verification of the UPL limit state is sensitive to the level of ground water or free water, the design value of uplift due to water pressure may be directly assessed in accordance with 2.4.6.1(2)P and 2.4.6.1(6)P.

As noted in 5.2 above, Simpson et al [b] illustrated the advantages for this type of problem of making an additive adjustment to the piezometric level rather than factoring water pressures. However, since structural design assumes that factors have been placed on characteristic structural loads, it is wise to carry out a second check in which structural action effects (forces and bending moments) caused by unadjusted water pressures are multiplied by normal structural factors (eg 1.35).

6.2 Eurocode 7 (202x)

The revision of EC7 is only partially drafted. Text for Part 1 is in circulation, giving the main principles related to water pressures, but the details of calculation methods to come in Part 3 are not yet available, including how partial factors might be applied. Values of factors to be applied to actions, including water pressures in some cases, are provided in the revised EN1990, which is also under development.

Part 1 will have a special clause on groundwater, considering its chemistry, density and capacity for freezing as well as the definition of water pressures to be used in calculations. It indicates probabilities of exceedance for characteristic and representative values (2% annually), ULS design values (1% within the duration of the design situation, which could be the design life or a shorter period during construction, for example). It also allows ULS design values to be derived by factoring representative values, so it is possible that national annexes will be able to choose between probabilistic assessment (as discussed in 4.3 and 5.3 above) or factoring. If factors are used, their values are to be taken from EN1990.

Calculations for uplift and hydraulic heave are outlined, and in both cases no further factors are applied to water pressures. For uplift, EN 1990 requires a factor of 1.0 on stabilising weight, so it appears that the only safety lies in the values of the assessed or factored destabilising ULS design water pressure; if a factor is used to derive the design uplifting water pressure, its value given by EN1990 is 1.2. All values mentioned here for partial factors are default values which are in principle subject to national variation.

For hydraulic heave, the current draft of EC7 goes beyond EN1990, introducing its own reduction factors, γ_{HYD} = 0.67 for the buoyant weight density of the ground and $\gamma_{pv;stb}$ = 0.9 for the weight of any effective overburden. Using parameters as shown in Figure 8, the complete requirement is:

$$u_d \leq \gamma_{w;k}(z+h_w) + \gamma_{HYD}(\gamma_k - \gamma_{w;k})z + \gamma_{pv;stb}\, p'_{v;k} \qquad (2)$$

The factor γ_{HYD} is effectively an inverse of a Terzaghi factor F_T = 1.5. A note is added: "The purpose of the factor γ_{HYD} is to ensure that the effective stress in the ground remains positive and also to allow for local variations of stresses at a granular scale, relevant to hydraulic failures." This implies that γ_{HYD} is not a safety factor intended to allow for uncertainties, as other γ values are.

It was argued in 5.4 above that this formulation has some disadvantages: it is not readily applied to a wider range of ground conditions such as sloping ground, it makes the use of finite element analysis problematic, and the factor should be material-dependent. Compared with Equation (1), α is equal to $(1-\gamma_{HYD})$, so an enterprising analyst could justify the Eurocode requirement by carrying out calculations with the ground weight unfactored and subsequently checking the results using Equation (1).

Time will tell how this draft will be used in practice. The author hopes that national annexes will avoid the use of factors on water pressure and will consider variation of γ_{HYD} as a function of material types.

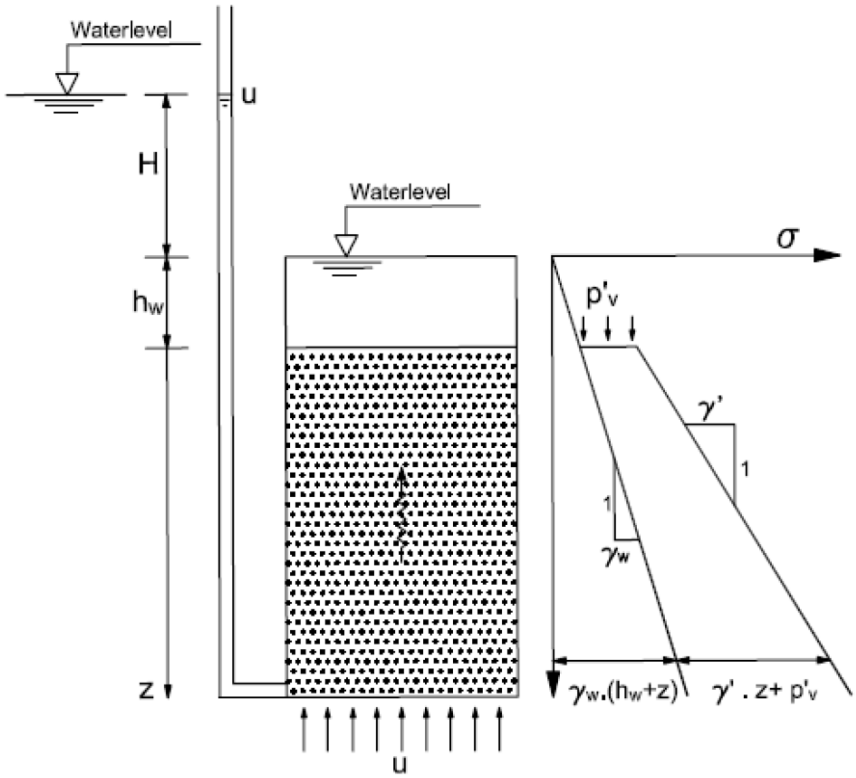


Fig. 8. One dimensional upward flow of water – definition of parameters

7 Concluding remarks

Three aims were stated in the introduction to facilitate sound geotechnical design in situations dominated by water pressures:

a) To maintain compatibility between calculations for the ground and related structures.
b) To carry out geotechnical design using sound principles of soil mechanics.
c) To retain equilibrium of forces and stresses throughout the geotechnical calculation.

It is proposed that in general these aims are best fulfilled by avoiding factoring of water pressures and instead making a probabilistic assessment of design values. Sometimes, there may be a sufficient database available to do this on a strict statistical basis, but in many cases this will not be so. In the latter cases, the geotechnical engineer responsible for the design should make an assessment guided by clear targets of probability of occurrence and careful assessment of the effects of geology, hydrology, extreme weather and, in urban situations, infrastructure including the effects of items such as leaking water mains and changes to water extraction regimes. In cases where water is moving in the ground, a careful and pessimistic assessment of the distribution of permeability is critically important and cannot by substituted by the use of factors.

Structures should also be verified in design for the effects of the same set of extreme water pressures. Factoring forces derived from water pressures inevitably disturbs equilibrium, and this should be avoided within a geotechnical calculation. However, since structural design assumes that all derived internal forces have incorporated factors, structural design should be verified, as a second check, using action effects derived from more likely characteristic values of water pressure together with factors on the internal structural forces (ie the factors are applied to the structural action effects)

Existing Eurocode 7, published in 2004, follows these principles to some extent, giving rather qualified guidance on probability targets for design values. The revised version to be published some years from now will probably be more explicit about these targets. However, both documents allow the option of factoring pore pressures to derive design values and time will tell how the new draft will be used in practice, partly dependent on national annexes.

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An innovative approach for soil compaction testing using microwave technology

Roger Bremen

An innovative approach for soil compaction testing using microwave technology

1. Introduction

The void ratio is one of the most relevant parameters for the geotechnical characterization of soils. In combination with compactness, the void ratio influences significantly the bearing capacity, deformability, shear resistance and finally permeability of soils. The void ratio defined as the volumetric relation between voids and the granular material has a maximum and a minimum value for a given soil to be defined in laboratory (e_{min} and e_{max}).
The ratio of the in situ void ratio to the mentioned limit values, which are determined in the laboratory, is known as relative density, D_R. The relative density is the parameter that indicates the soil compaction level. Amongst many others, this is a determining parameter assessing the risk of liquefaction of loose materials subjected to dynamic loads. The relative density is expressed by the relation:

$$D_R = \frac{e_{\max} - e}{e_{\max} - e_{\min}} \tag{1}$$

where,
e_{max} : void ratio of the soil in its loosest state,
e_{min} : void ratio of the soil in its densest state, and
e : in-situ void ratio.

The relative density, D_r, varies between 0% for a very loose soil and 100% for a very dense soil. A corresponding classification of granular soils based on values of D_r is suggested by Lambe and Whitman (1969).
To solve eq. (1) for determining the relative density of a soil, the three previously mentioned void ratios, i.e. e_{min}, e_{max} and e, have to be known. While the first two parameters, i.e. the densest and loosest possible state are obtained by means of laboratory tests on representative (disturbed) samples of the corresponding soil, the determination of the void ratio of the soil in its natural state on the other hand is often difficult, in particular for deep alluvial and colluvial soil. Even though appropriate equipment might permit obtaining undisturbed samples, such an expense is often not justifiable. If a sufficient budget is available, the in-situ void ratio can also be determined from laboratory analysis of frozen samples, or from neutron or gamma-ray density logging. There are also tables available that provide corresponding soil property correlations. However, these values should be referred to only in exceptional cases, and should be used only with a certain degree of caution.

This is the reason why the natural void ratio is typically estimated indirectly through empirical correlations using standard-penetration-test (SPT) or NSPT and conventional cone-penetration-test (CPT) results.
The SPT is frequently used as a quick, easy and low-cost soil testing solution. Its main drawback is that it is relatively inaccurate, especially when sampling coarse sands, or clays. Furthermore, when gravels are encountered, the SPT results get unreliable and mostly unusable.

The CPT, which provides a result in terms of the tip resistance, is often more reliable than the SPT, since it is less subject to errors. CPT is becoming increasingly popular for site investigations and geotechnical designs. For many construction projects, it is common to use SPT for the preliminary soil investigation, whereas CPT is used for detailed soil investigations and construction quality controls. However, this method requires estimating the content of fines, which presents one of the weaknesses of the method, should no soil samples be available. Furthermore, CPT is not applicable in gravelous material as generally encountered in alluvial soils.

In summary, penetration resistance testing is generally subject to both operator and mechanical errors, which impairs results finding and as a consequence, limits the applicability of these methods to certain soil conditions. As such, they finally affect the reliability of the geotechnical analysis based on these tests. In this context it is emphasized that, especially but not only for the determination of the liquefaction risk, proper knowledge of the relative density is fundamental.

The limitations of the currently available testing methods and the necessity for more reliable measurements arising from actual project experience, have called for the development of a new device. The main requirements are to have a user-friendly apparatus, which provides a quantitative and direct measure of the natural, in situ void ratio.

2. Development of a reliable soil compaction test (SCT)

2.1 Physical background

The basic approach is to determine the water content of a saturated granular soil, and thereby obtain a direct measure for the void ratio. As such, it represents a procedure already known to other industries (i.e. ceramics and cement), where it is used to control the moisture content during manufacturing processes.

The fundamental concept is based on electromagnetic waves of radio frequency, with which the average dielectric constant of the soil can be derived. With this dielectric constant being directly related to the water content of the soil, the void ratio in a saturated soil is obtained immediately. The relative density might then be obtained by determining in laboratory as usual the $e_{min.}$ and $e_{max.}$ values.

To verify the effective applicability of this concept for different soil conditions, as a first step, it had to be proven that the dielectric constant depends only from the water content and is independent from granulometry and mineral composition of the soil. Some of the test results for different soil granulometries in not saturated conditions are shown in (fig. 1). It appears that, despite of the different composition, particle size, grading and mineralogy, the initial dielectric constant is with a value of indicatively 2.5-2-7 very similar for all samples. The same applies for the increase of the dielectric constant in function of the water content, which turns out to be similar for all samples.

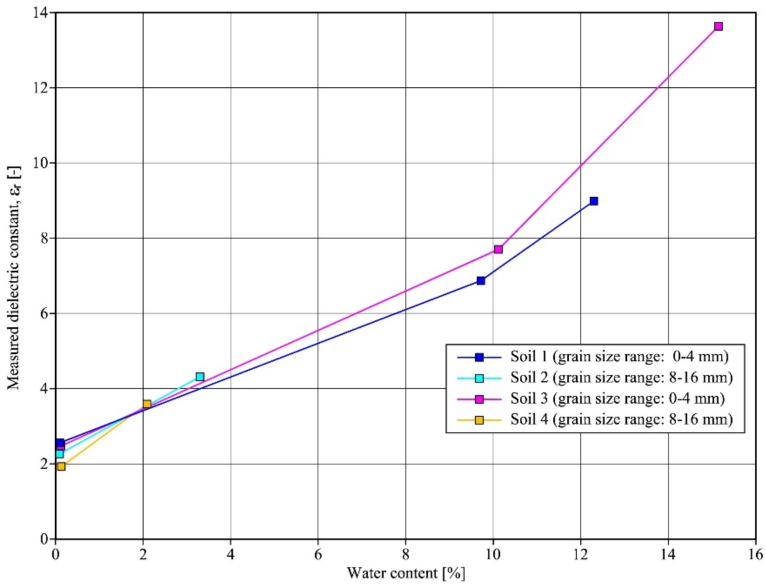

Figure 1: Results of initial laboratory tests. Percentage of water in function of measured dielectric constant for different soil granulometries in not saturated conditions.

Consequently, the average dielectric constant depends essentially on the volumetric ratio between water and soil particles. With the dielectric constant of water being about 30 times greater than that of the soil particles, the soil water content can thus be determined in a good approximation by evaluating the propagation speed, which is related to the dielectric constant by

$$v = \frac{c}{\sqrt{\varepsilon_r}} \qquad (2)$$

where
c: is the speed of light
ε_r: is the relative dielectric constant

The actual wave propagation speed is additionally affected by various delays caused by cables, antennas, and air to be taken into account. Knowing the distance between the sender and receiver, and calibrating the measurements with respect to the mentioned delays, the propagation velocity of the electromagnetic waves can be determined. Provided that the test is performed in saturated soil conditions, the electromagnetic signal propagates through two material types, i.e. water and soil. Thus, the dielectric constant measured by the

SCT device is a combination of these two values and can therefore directly be correlated with the soil-water content. Hereafter, the corresponding analytical conversion is briefly presented.

By knowing the values for the dielectric constant of dry soil, ε_{soil}, that of water, ε_{water} (i.e. ~80) as well as the borehole distance, it is possible to determine the soil-water percentage. The analytical solution for the determination of the water content is:

$$\varepsilon_{measured} = \frac{(x_{soil} \cdot \varepsilon_{soil} + x_{water} \cdot \varepsilon_{water})}{d} \tag{3}$$

with d being the borehole distance, defined by

$$d = x_{soil} + x_{water} \tag{4}$$

and with x being the corresponding content of the soil and water, respectively, defined by

$$x_{soil} = \frac{d \cdot (\varepsilon_{measured} - \varepsilon_{water})}{\varepsilon_{soil} - \varepsilon_{water}} \tag{5}$$

$$x_{water} = d - x_{soil} \tag{6}$$

Consequently, as given by eq. (3) a simple linear relation between the mean dielectric constant, $\varepsilon_{measured}$, and the water content can be considered, according to which the dielectric constant of dry soil can be considered the same for different soil compositions.

In fact, other influences on the dielectric constant have been studied as the water temperature or the water salinity. In the usual temperature range of groundwater temperatures, (+5°÷20°C) this parameter has only a marginal influence on the measurement of the void ratio. The same applies for the water salinity, in order that only for a high salinity groundwater (typically > 4'000 mg/L) a correction factor might be considered.

2.2 Prototype and pilot tests

Following the initial basic studies and tests, laboratory tests were performed using soils of different composition and different granulometries. As shown in (fig.2), the emitter and receiver were placed at different distances to evaluate the measurement range of the equipment. For each soil the water content was determined by oven-drying and the results compared to the records of the new device. These tests showed that the developed instrument is applicable and can be used for the following two measurements:

- Dielectric constant and thus water content for non-saturated soils, respectively,
- Percentage of water content in saturated soils to determine the soil compactness.

With the pilot tests being performed under controlled conditions regarding soil composition and set up, it was also possible to quantify the measurement accuracy. Accordingly, according to these tests, the water content can be determined with a precision of less than 5%.

Figure 2: Laboratory tests. Left: set-up with holes at 67 cm distance in sand. Right: same box filled with gravel and coarse material.

After proving the functionality of the measuring system in laboratory, a first prototype version for in situ tests was prepared. A schematic view of this device is shown in (fig. 3). It consists of a portable main unit and two sensors, i.e. a sender and a receiver, respectively. The equipment can be placed in boreholes down to 75 mm (3") in diameter. For a reasonable and economically feasible device capacity, which in turn determines the signal strength, a testing range distance between the boreholes of up to 20 m can be reached. The tests are carried out in common PVC tubes or similar, as steel casings would impede signal propagation, rendering tests impossible. As mentioned above, the medium to be tested should be a saturated granular soil – only under these conditions the assumption of a two-phase medium might be applied. However, this condition often applies for foundations in difficult geotechnical conditions.

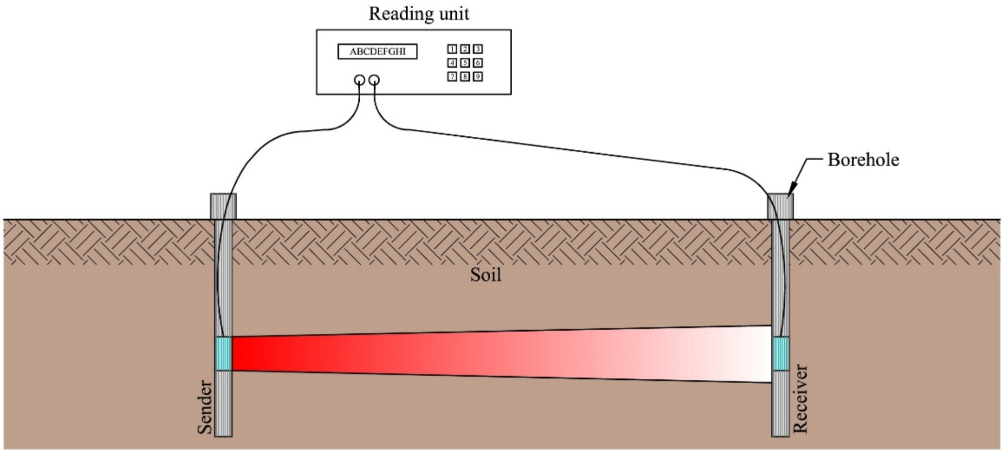

Figure 3: Schematic view of the pilot test device used for in-situ measurements

Before starting with the main measurement, an air-calibration is required. For this purpose, the sensors are located on the surface, right above the two boreholes to be tested, and a single measurement is done. In this way the signal passes exclusively through the air medium - an important measure required for the automatic post-processing calibration. Subsequently, the elevation of the ground water level is determined. The sensors are then stepwise lowered in the two boreholes, while for each depth interval a corresponding measurement is performed. The increment size is selected based on the necessary detail and accuracy required for the project. In most of the cases a range of 0.5 and 1 m appears to be reasonable. The readings are then performed along the entire depth of the borehole.

Following a general check of the measurements, the data is processed and stored. A specially developed PC-tool allows the user to read off the required information directly. A user-friendly interface provides the measurement results, in the form of a dielectric constant profile over the measurement depth and the corresponding water percentage (bar diagram) based on the previously detailed approach. A screen shot of the user interface with an example measurement is shown in (fig.4).

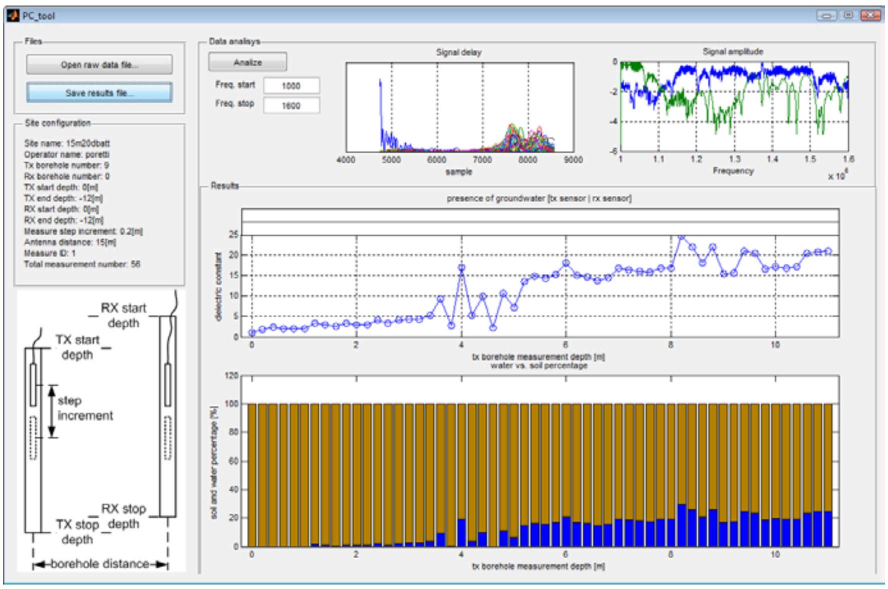

Figure 4: Screen capture with example of measurement results of a 12 m deep borehole.

For a consistency check, the measure is repeated uphole and downhole and by switching the sender and receiver in the holes. These different measures give an indication on the reliability of the performed measurements. Further parameters, such as the signal delay are also illustrated. They give the user the possibility to perform verifications and to confirm the correctness, consistency and repeatability of the measurements.

3. Use of the SCT test device

After various laboratory tests as well as on specially prepared field sites, the general functionality of the SCT device was verified on real sites. The main purposes of these tests were to verify the practical on-site functionality of the device, and to have a comparison with other measuring devices. The intention was thus to implement into the prototype all the needs arising from the practical use of the equipment.

With the purpose to compare the results with other measuring techniques, a site was selected for which SPT and seismic investigations (cross-hole) have been performed and the boreholes were still available.The Nenskra dam construction site in northwestern Georgia was finally chosen fulfilling the requirements.

To ensure the safety of the 130 m high AFRD dam, an evaluation of the foundation characteristics in particular as regards the liquefaction risk was necessary. The valley of glacial origin is filled at the dam site with glacial, colluvial and alluvial soils, for a maximum depth of up to 170 m. These soils include a wide range of grain sizes, with silts, sands, as well as gravels up to several decimeters in diameter, and finally boulders. In (fig.5), an example of a typical borehole log indicating lithology, permeabilities as well as photos of the core boxes obtained from the dam foundation are presented.

With these soil conditions, special attention was given to the assessment of expected settlements, as well as the degree of compactness in general and liquation risk. The purpose of the investigation is as commonly the definition of the required excavation depth.

It turns out that due to the presence of coarser material and boulders, the performed penetration tests (SPT) proved to be unusable for any further elaboration. The SPT revealed mostly blow counts indicating a withdrawal already at a shallow depth once coarser material was encountered. Since SPT results are no giving any reliable result some cross-hole tests were also performed. Finally, measurements by means of the new SCT device were carried in order to provide a better and complete data basis as required to evaluate the in-situ soil compactness. Considering the importance of accurate information especially in the central foundation part of the future dam, the SCT campaign focused on the available, accessible boreholes in this part.

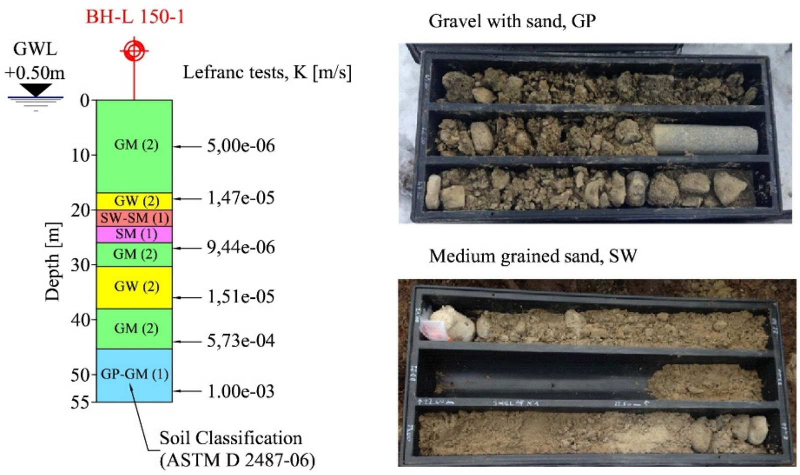

Figure 5: Left: typical borehole log at the Nenskra dam site indicating lithology and permeabilities. Right: photos of the core boxes obtained from the same part of the dam foundation.

The exemplary boreholes presented here BH-L-150-1, BH-L-150-1a and BH-L-150b have a distance of between 8 and 15 m. The foundation in this part is characterized by artesian ground water conditions, so that generally saturated soil conditions could be assumed already at shallow depth. With these conditions, it was therefore possible to perform the measurements basically from the borehole mouth down to a depth of 35 to 40 m below the surface. In view of the required accuracy and resolution of the void ratio profile with depth, the sensors were lowered in increments of 1 m. For consistency, every borehole couple was measured in both directions, exchanging both antennas in the holes. The general test set up is shown in **figure 6**.

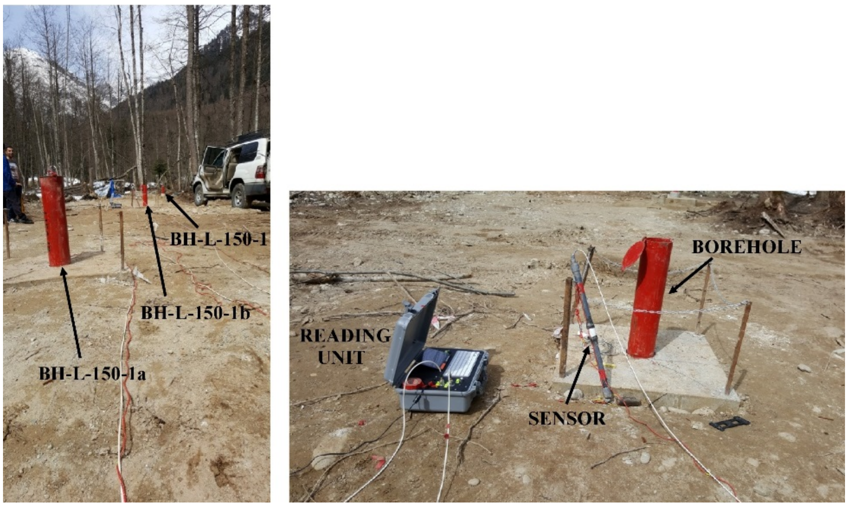

Figure 6: Arrangement of the SCT test equipment for the Nenskra dam site.

For the present case, special interest was in the relative density values of the foundation soil. This information was essential to evaluate as accurately as possible the foundation behavior once being exposed to the project loads. To create a corresponding relative density profile, the available information from the core logs was used to get the void ratio of the foundation soil in its loosest state, e_{max} on one hand, and the void ratio of the soil in its densest state, e_{min}, on the other hand. One typical relative density profile is shown in **figure 7**.

In the previous figure SPT values which, due to the presence of boulders, were out of range (i.e. blows > 50) are not shown. Due to the large scattering of the SPT results it was not possible to determine a reasonable range of the relative density. It has to be noticed that this strong limit of the SPT test is rather common in coarse soils. On the same site also cross hole tests have been performed.

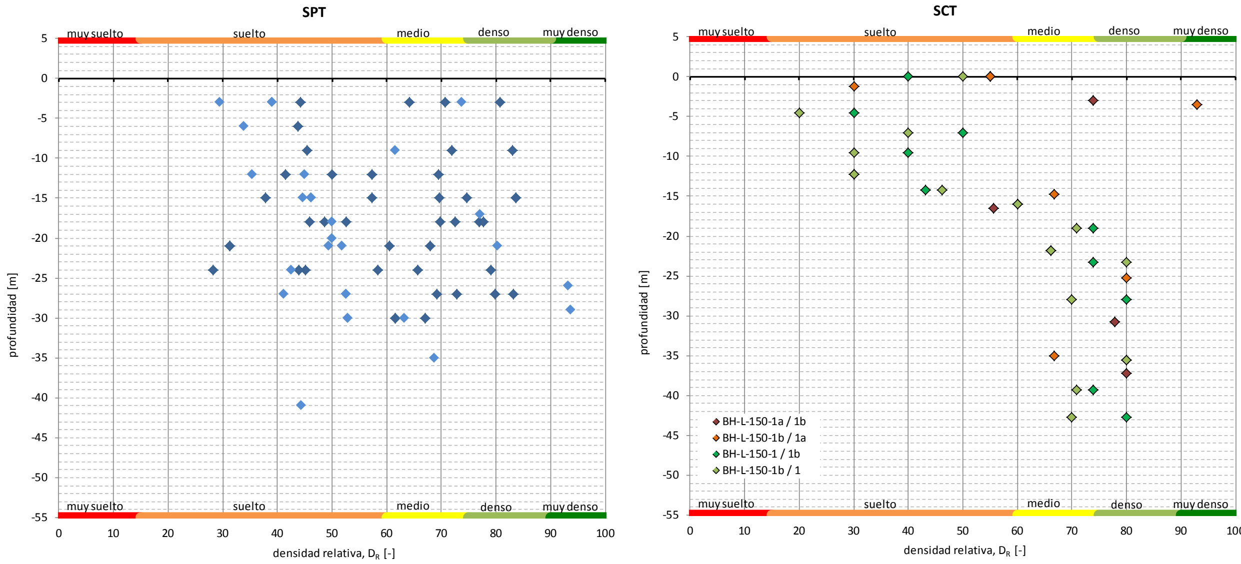

Figure 7: Comparison of SPT and SCT test results on boreholes BH-L-150 1a/1b and BH-L-150-1/1b at the Nenskra dam site.

. Due to the large scattering of the SPT results it was not possible to determine a reasonable range of the relative density. It has to be noticed that this strong limit of the SPT test is rather common in coarse soils. On the same site also cross hole tests have been performed. **Figure 8** shows some of the obtained profiles.

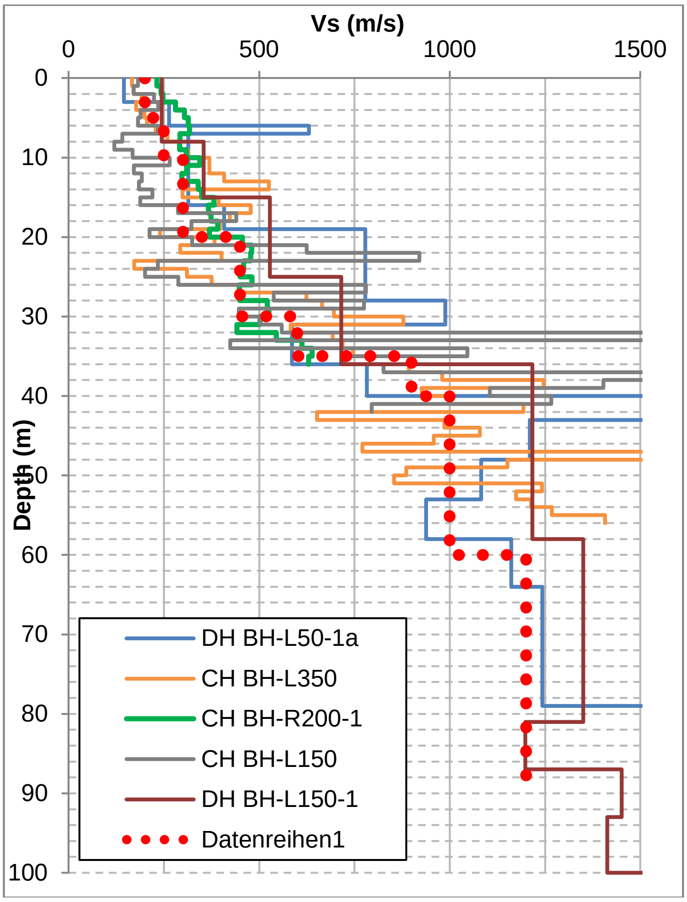

Figure 8: Typical shear velocity profiles obtained on boreholes of the Nenskra site.

According to these results it is quite interesting to notice the correspondence between the SCT tests and the shear velocity profiles obtained with of the cross hole tests. The common limit of these tests is that no generally valid relation exists between the shear velocity and the relative density in order that different calibration relations have been developed by different authors according to their specific investigation results.

The present results show that the SCT device is able to accurately determine the water content in soils and thus the relative density provided e_{min} and e_{max} have been established in laboratory. In the present case the upper 10 to 15 m are generally loose to medium dense, while, as expected, the compactness increases with depth. The locally notable decrease is due to the presence of coarser material. The rather high density observed between 3 and 4 m depth can be explained by the presence of very fine grained lacustrine deposits. These findings are in line with the information of the core boxes. The developed SCT device permitted to obtain a complete and accurate relative density profile on which design considerations can be based on.

4. Present status and further developments

Based on the tests performed on project sites, the prototype has been improved and a first series of 3 industrial devices have been developed and are currently in use. In comparison to the first units, the equipment has been more robustly designed to meet transport and in-situ requirements, The cable length has been increased in order to increase the measuring dethts and more recent electronic components have been used. The commercial use of the device is currently initiated and in Switzerland the equipment is promoted and used by the MSP company based in Luzern.

5. Conclusions

In order to assess the quality and performance of any soil foundation, reliable knowledge of the void ratio and compactness is essential. Due to the difficulties associated with obtaining undisturbed soil samples for direct measurements, indirect methods such as SPT, NSPT or CPT are usually used. However, these methods have their limitations and often fail, especially with heterogeneous soils also containing coarser materials. To overcome this limitation, a new device based on microwave technology, referred to as SCT (Soil Compaction Testing) has been developed and tested. By means of the dielectric constant, this method permits to determine both the water content percentage of unsaturated soils (water-soil ratio), and also the void ratio and compactness of saturated soils.

Measurements can be performed between boreholes with a distance of up to 20 to 25 m. It is possible to determine the water content with an accuracy of about 5%. The measuring device itself is user-friendly, including a user-friendly interface Various tests on actual construction sites performed so far have already proven, not only the general functionality of the SCT device, but also its higher reliability and increased feasibility with respect to common friction based testing methods. This paper presents one application example i.e. at the foundation of a 130 m high rock fill dam. By means of the SCT, it was possible to easily and rapidly obtain a reliable basis for further geotechnical assessments such as settlements, and liquation risk.

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„Seeing the big picture“, Innovative Radar Remote Sensing for Slope Stability Monitoring

Andrew Kos

„Seeing the big picture“, Innovative Radar Remote Sensing for Slope Stability Monitoring

1 Introduction

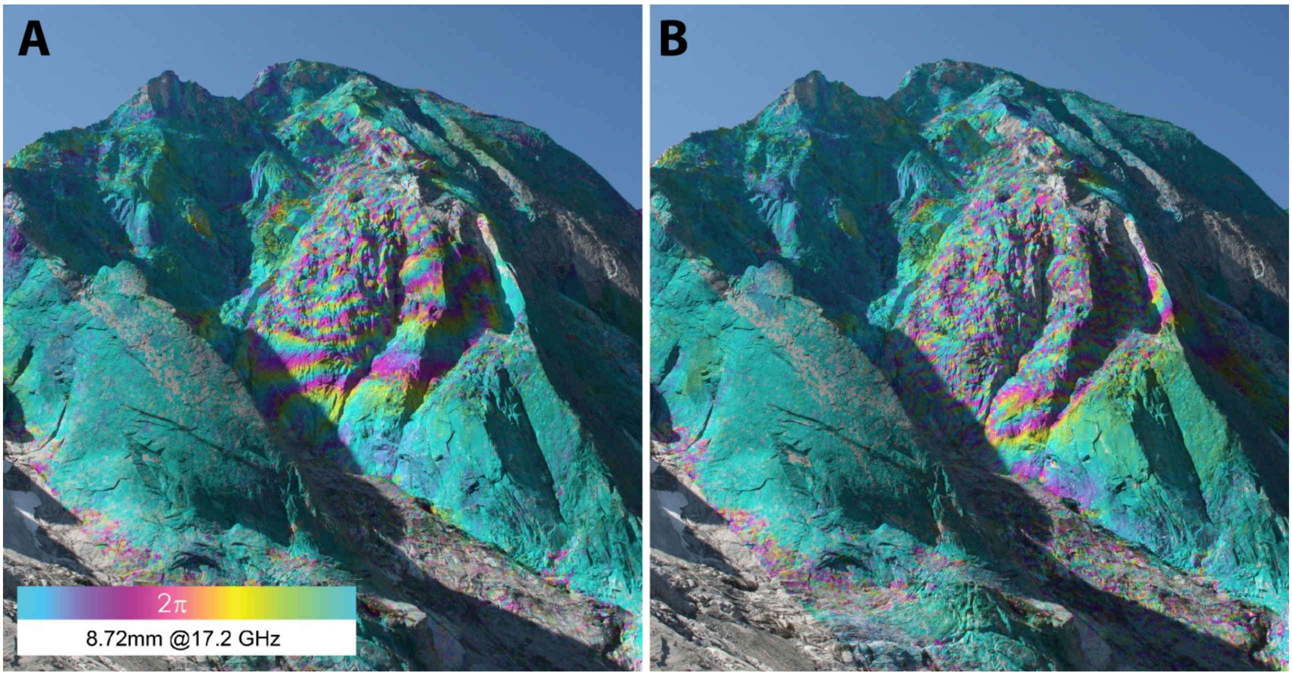
Rock slope deformation monitoring for early warning requires a view of the big picture. This enables experts to distinguish between behavior of the entire rock slope, indicating signs of immanent catastrophic failure versus symptomatic small-scale local failures in response to slope deformations. Ground-based radar remote sensing [1] has emerged as an alternative to traditional techniques for diverse slope monitoring applications offering detailed insights at a wide range of spatial and temporal scales [2, 3, 4, 5, 6].

Over the last decade, Terrasense Switzerland Ltd, a recognized ETH Spin off, has played a lead role in pioneering slope deformation monitoring services for decision makers in the public safety and private sector. The challenges of innovation in a previously non-existent market will be briefly discussed. A service application of the technology and services will be illustrating using the 2017 Piz Cengalo Bergsturz case study, where Terrasense played a key role in providing early warning to Canton authorities.

2 Portable Radar Interferometry

2.1 Early Warning issued in 2017

On 23 August 2017 a rare 3.15×10^6 m^3 rock mass failed on the Pizzo Cengalo located in the Bondasca valley in Switzerland. The rock slope failure resulted in human casualties and significant damage to local infrastructure. Together with the Department of Forests and Natural Hazards (Canton Graubünden) and partners from ETH Zurich, SLF Davos, and Bonanomi AG, Terrasense undertook radar interferometric monitoring of slope deformations between 2012 and 2017 (Arge-Alp project Pizzo Cengalo).



Picture 1: Interferograms showing dramatic changes in the magnitude of large scale slope deformations on the Pizzo Cengalo, which were used to interpret the stability situation. (A) Interferogram 2015-2016, (B) Interferogram 2016-2017.

The radar interferometric monitoring utilized techniques developed as part of the Terrasense start-up mission of transferring research know-how to practice. An example of the radar technology application and method development for the slope monitoring service market is concretely illustrated in picture 1. In this example, long term periodic monitoring, combined with geotechnical/geological investigations revealed important behaviour of the rock slope instability. Building upon previous experience [3], changes in the critical slope stability state was recognised resulting in a robust early warning.

2.2 Radar Interferometry at Pizzo Cengalo

A portable radar interferometer was used to measure rock slope deformations on a periodic basis between 2012 and 2017 from a fixed position on the Bondasca moraine located ~1 km from the Pizzo Cengalo rock slope instability (Picture 1). The instrument is a real aperture FMCW system with a central frequency of 17.2 GHz, (Ku band). The radar utilizes a fan-beam antenna array that rotates around a central axis with sampling rates of 10 deg s^{-1}, ensuring very high phase coherence during image acquisition. Movement is measured along the radar line of sight (LOS), with a precision of < 2.0 mm [7, 8, 9]. The field of view is programmable to 360 degrees and rotation velocity is 10 degree/sec. Once a radar image is acquired, repeat measurements can be made within ~30 seconds. In practical terms, rapid scene acquisition allows for (1) highly coherent radar images (a pre-requisite for all types of radar interferometer) resulting in high data quality, (2) a high degree of data stacking to reduce atmospheric effects, (3) coherent tracking of fast moving objects (e.g. ~0.3mm/sec), relevant for near real-time monitoring involving critical failure, and (4) reduced field setup and measurement times, particularly for periodic control measurements. A range of data processing strategies have been developed by Terrasense, which requires a combined specialized knowledge of radar remote sensing and the geotechnical/geological context of the target rock slope, in order to reveal detailed spatial and temporal characteristics of the radar signals representing rock slope deformation.

Picture 2: Ground-based radar interferometer measuring slope deformations on the Pizzo Cengalo rockslope instability in July 2013.

3 Service Market for Radar Interferometric Monitoring

Developing a new-service market for civil applications is a risky undertaking. In building the market, Terrasense has sought to initially demonstrate the technology and applications in slope monitoring, through applied research projects and pilot projects since ca. 2007.

Challenges to market development include: 1) business scalability, 2) scalability of monitoring solutions with respect to cost/benefit, 3) opportunistic characteristic of the market e.g. Off-the-shelf simplified technologies being applied by competitors with little contextual experience, 4) client expectations, and 5) high level of expertise required for achieving high quality results.

Despite the pioneering efforts of Terrasense to develop services within a virtually non-existent market, an example of our success is the fact that competitors have been attracted into the market. An unresolved question for the service market in ground-based radar monitoring is whether simplified user-operated solutions are more efficient, robust and cost effective than services offered by highly experienced practitioners.

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DNA-based tracers to track subsurface fluids and assess reservoir properties

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DNA-based tracers to track subsurface fluids and assess reservoir properties

1 Tracing in underground reservoirs

Tracers are becoming a mature technology to characterize fluid transport and reservoir properties groundwater, geothermal, and oil/gas reservoirs. A typical interwell tracer test is performed as depicted in Figure 1: a set of reservoir rock- and fluid-compatible tracers are designed and unique distinguishable tracers are injected together with the fluid (usually water) in various locations across the reservoir. Ideal passive tracers blindly follow the fluid phase in which they have been injected. The produced fluid in production wells distributed across the reservoir is then sampled over time, followed by the sample analysis to detect and to quantify the tracers. The concentration profile over time is then obtained, and used (e.g. by tomographic inversion) to characterize the field.

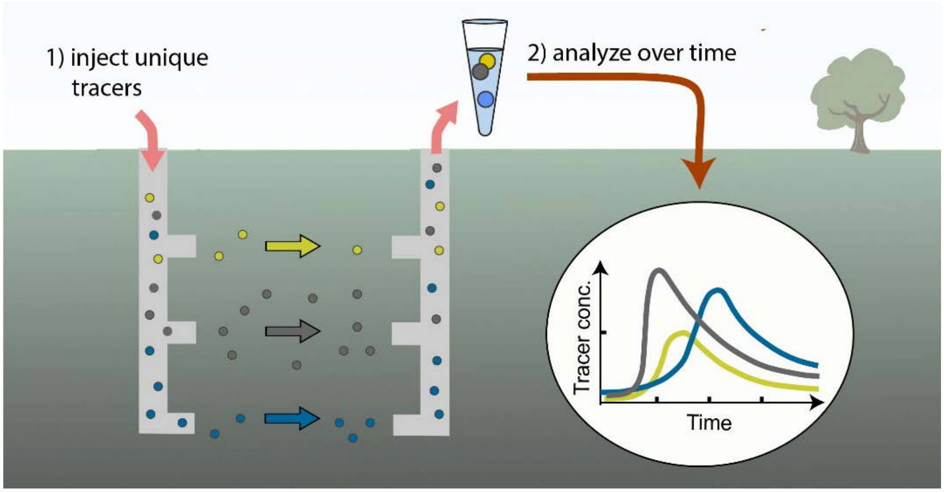

Figure 1: A general overview of a tracer application: unique tracers are injected at multiple injection wells, and water samples are collected in another set of producing wells over time. The collected samples are analyzed for the individual tracers to obtain the tracer concentration development over time (breakthrough curves). Such breakthrough curves are used to establish reservoir and fluid transport parameters.

By performing tracer tests, reservoir engineers obtain information about well connectivity; determine the flow paths, velocities, characterize convection-diffusion processes or simulate contaminant transport. By performing multitracer tests, and translating the tracer breakthrough times into tomographic maps, key parameters such as a reservoir's porosity and permeability field may be obtained Such information allows optimizing the resource (e.g. water, oil, heat) extraction and improving the field development.[1]

2 Limitation of the current tracing technologies

2.1 Requirements for a good tracer

To qualify for injection into the formation, a tracer has to fulfill a list of requirements: the material has to be stable in the operational conditions (to not degrade thermally, chemically, physically and biologically); it has to follow the fluid of interest, and not be absorbed to rock or partition into other phases; it should be easy to detect at low concentrations, and be absent from reservoir to avoid background noise; finally it should be non-toxic and environmentally friendly.[2] Furthermore, because tracer operations today are often performed in fields with a large number of wells, a high number of simultaneously distinguishable tracers with identical characteristics are needed to perform a high-resolution reservoir assessment.

2.2 Current technologies

The most common classes of tracers used in underground reservoir assessment are presented in Table 1. Fluorescent dyes are the most frequently aplied tracers in aquifer characterization, but only a few different dyes can be distinguished simultaneously, limiting their broader application. Furthermore, many of the dyes interact with the solid phase inside reservoirs, which may in turn result in the tracer loss. Fluorobenzoic acids, on the other hand, are very common in oilfield characterization, because although more difficult to detect (no online measurement is possible), around 40 unique tracers can be quantified in low ppb or even high ppt concentrations. Other tracer classes include salts, radioactive tracers (barely used today) as well as evolving technologies (colloidal tracers, DNA, etc). Many tracer operations today require more than 40 tracers to be injected in the same field, and therefore new tracer designs capable of fulfilling the above requirements are needed.

Table 1: An overview of the currently available tracing technologies. Based on the work by Serres-Piole et al.[2]

Tracer class	Examples	# unique tracers	Limit of detection	Lack of toxicity/ environmental regulation	Fast, portable on-site detection	Stability at 80°C and 100 bar
Fluorescent dyes	Uranine, naphthionate	<10	1-60 ppt	+/-	+	+
Salt tracers	NaCl, KCl, NaBr	<10	3-50 ppb	+	+/-	+
Radioactive tracers	^{3}H, ^{51}Cr, ^{114}In, ^{84}Br	<10	Single molecules	-	+	+
Fluorobenzoic acids	2-FBA, trifluoro-p-toluic acid	~40 [d]	10-50 ppt[3]	+/-	-	+
Free DNA	Single and double stranded oligonucleotides	unlimited	<0.1 ppt	+	+/-	-
Haelixa DNA tracers	DNA particles	unlimited	<1 ppt	+	+	+

DNA has often been proposed as an alternative to the current tracer classes, in particular because a virtually unlimited number of unique DNA sequences can be generated to trace a large number of wells. The main drawback of DNA as a tracer is its insufficient thermal stability. This allows applying DNA only in small-scale surface water tracing tests.[4] Free DNA has not been shown to survive oil and geothermal reservoir conditions.

3 Haelixa DNA tracers

3.1 Design

To overcome the limited DNA stability, Haelixa encapsulates DNA inside inorganic oxide particles (Figure 2).[5] Such particles protect DNA from thermal and chemical stress, yet keeping the advantages of DNA (uncountable unique tracers, lack of toxicity, low limits of detection). The technology has been developed at ETH Zürich, and has found applications not only in underground traceability, but also material marking applications to avoid counterfeiting and ensure supply chain integrity.

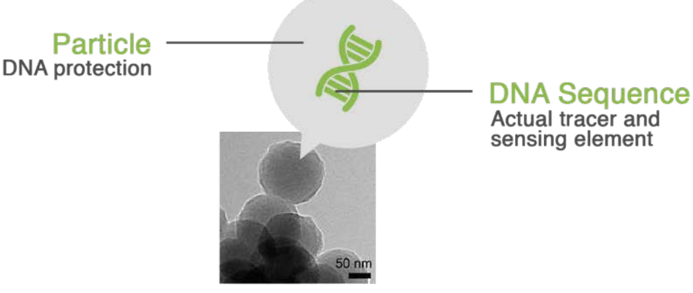

Figure 2: Haelixa's DNA tracer design: short DNA sequences are stabilized by encapsulation inside inorganic oxide to protect them from harsh underground conditions

Despite different design, the data generated from DNA-based tracers is the same as for any conventional solute tracer. To quantify the extent of DNA particles in samples collected over time in different locations (Figure 3), the particles are first dissolved using a proprietary particle dissolution buffer. The specific DNA sequences are then quantified by quantitative polymerase chain reaction (qPCR) and related to a standard tracer calibration curve. When plotted over time, DNA tracer concentration results in a breakthrough comparable to any other passive tracer.[6] Such data can then be interpreted qualitatively (e.g. well connectivity, existence of faults, etc), or quantitatively (e.g. tomographic inversion algorithms to obtain 3D reservoir permeability maps).

Figure 3: To produced DNA tracer breakthrough curves, DNA particles in collected samples are dissolved, and the resulting free DNA is analyzed by qPCR to obtain the tracer concentration in the sample, which is then plotted over time to obtain a breakthrough curve.

3.2 Added features: temperature, pH and porosity measurement

Since reactivity of DNA sequences inside particles can be tuned, tracers with different reactivity towards environment can be designed. If a DNA sequence inside a particle is designed to degrade under controlled kinetics at elevated temperatures, the DNA degradation pattern can be correlated to the cumulative temperature that a tracer has experienced between the injection and sampling wells.[7-8] Analogous designs can be applied to produce tracers capable of measuring pH, light, oxidative stress, or pore size distribution.

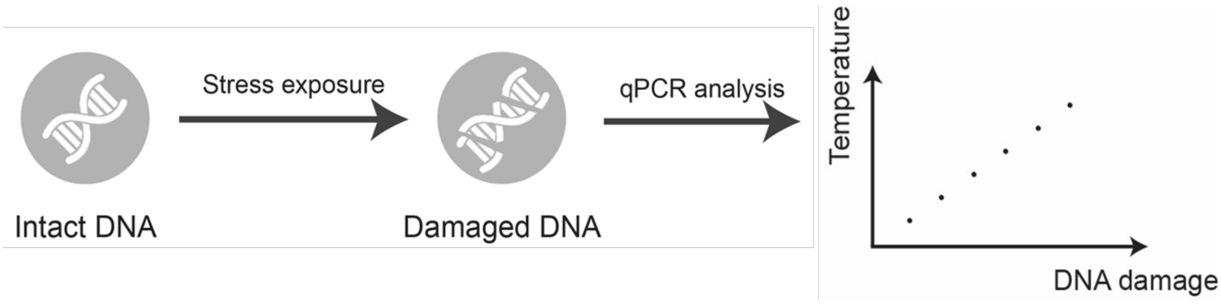

Figure 4: Specific DNA modifications and decay may be exploited to provide additional information about the reservoir including temperature and effective porosity

3.3 Tracer performance

Although considered a passive tracer that travels together with the fluid front, the colloidal DNA tracer is transported slightly faster than small molecules tracers (Figure 5A). The reason for this is that colloidal DNA travels through larger pores, whereas the dye tracer uranine enters much smaller pores, resulting in slower average transport velocity. Furthermore, column tests in sand showed that the recovery of Haelixa's DNA tracers with a size of ~150 nm is higher than that of uranine, because negatively-charged DNA particles interact less with the rock than the dye.

Non-encapsulated DNA has a half-life of less than 10 h in the environment at 20° C,[9] and is not suitable for most large scale tracer tests. Haelixa's DNA tracers are stable at elevated for extended periods of time (Figure 5B). Therefore, they have potential not only in long term groundwater tracing tests, but also in oil, and geothermal reservoir characterization.

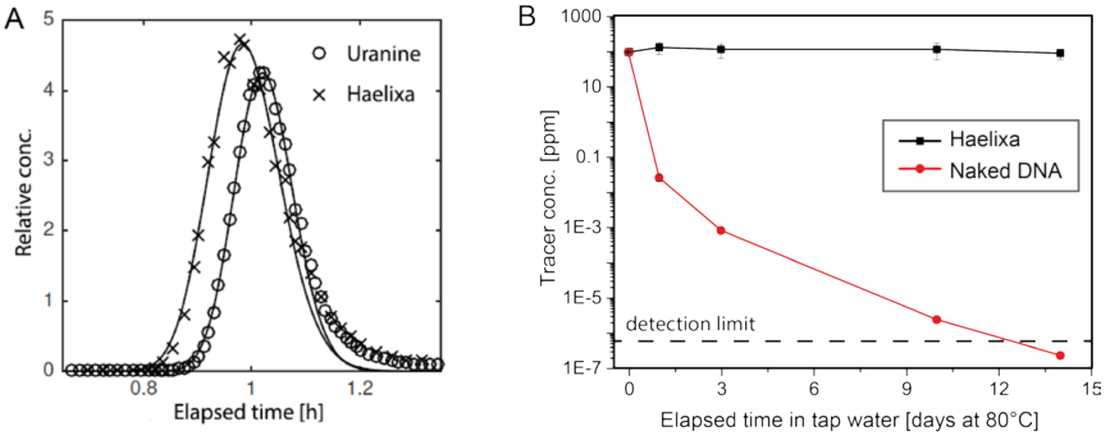

Figure 5: Encapsulated DNA tracer breakthrough in a sand column in comparison to fluorescent dye uranine (A); A comparison of thermal stability of encapsulated and naked DNA at elevated temperature.

The tracers have been validated in sand column experiments, small scale (<20 m) porous and fractured media tests. Tracer validation in more challenging conditions (higher temperature, different rocks, higher salt concentration) is ongoing.

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Soil bio-reinforcement in a series of geotechnical applications

Dimitrios Terzis

Lyesse Laloui

Soil bio-reinforcement in a series of geotechnical applications

1 Abstract

We introduce a nature-based, soil stabilization agent to obtain bio-cemented soils through fast, reactive mineral formation which lasts. We apply a liquid solution, ready-to-mix with soils, which stimulates the enzymatic activity of the soil bacterium *Sporosarcina Pasteurii* and is composed of environmentally-friendly chemical components. Further, by coupling the agent responsible for soil stabilization with conventional geo-textiles we suggest a mechanism which ultimately enhances the efficiency and reproducibility of this nature-based, mineral bio-cementation solution for traditional geotechnical engineering problems. Results show that the execution of bio-mediated $CaCO_3$ precipitation results in strong mineral links within soils, in a "cell-free" environment, upon complete breakdown of bacterial cell clusters. Further, strength and stiffness parameters of bio-cemented sands are determined. Bio-cemented sands yield compressive strengths up to 12 MPa. We further look inside the bio-cemented material to understand its micro-structural characteristics. Overall, we contribute to the debate on the importance of factors affecting: (i) soil bio-cementation efficiency, (ii) the mechanical response of bio-cemented sand and (iii) its peculiar micro-architecture.

2 Introduction

The study puts the focus on bio-mediated soil cementation, an emerging technology which alters substantially the structure of geo-materials and endows the subsurface with improved overall mechanical behaviour. The technique integrates bacterial metabolic activity in soil-permeation solutions, to ultimately induce the formation of calcite ($CaCO_3$) mineral crystals which act as binders among particles of granular soils (Figure 1). Quite unlike most applications targeting the artificial cementation of soils for improving their overall mechanical properties and bearing capacity, microbial-induced calcite precipitation (MICP) [1]-[3] via the ureolytic soil bacterium *S. Pasteurii* is applied via non-erosive, low-pressure propagation of environmentally-friendly bio-chemical components dissolved in water (Figure 1). More precisely the relatively cheap combination of urea and calcium is used, with the former resulting in the release of bicarbonate (Equation 1) which under the presence of the latter precipitates in Calcium Carbonate mineral crystals (Equation 2). Details on sample preparation can be found in [4] and [5]. In a previous study by the authors [5], it was shown that the reactive mechanism can be activated even when the bacterial cells are not alive and their cell cluster has been degraded. This finding confirms that the desired reaction takes place in a "cell-free" environment with the active enzyme being responsible for the transformation of urea into bicarbonate which subsequently reacts with the provided dissolved calcium.

Figure 1: Calcite mineral crystals (arrows; light grey) acting as bridges among soil grains (dark grey) observed through scanning electron microscopy (left); liquid agent reacts with urea- and calcium-rich medium to form mineral crystals of calcite (middle); calcite mineral precipitates, formed after activation of the bio-reinforcement agent, accumulate at the bottom of the flask (right).

The reactive solution can be directly mixed with soils under low pressure injection or even under gravity flow. Alternatively, the liquid can be coupled with conventional geo-textiles and geo-membranes to enhance the performance of geo-synthetic reinforced earth structures such as embankments. Despite the wide application of conventional geosynthetics in myriads of geo-technical and geo-environmental problems, the associated materials and fundamentals of installation processes remain, to a large extent, unchanged. By introducing the novel, combined use of traditional geosynthetics with fast, reactive, bio-reinforcement we ultimately engineer earth-geosynthetic structures of improved resistance and durability. We integrate the mechanism of natural mineral formation within the layers of conventional geosynthetics and control the diffusion of the reactive, environmentally-friendly, solution to the surrounding geo-material. The newly formed mineral is calcite, and acts as binder between the geosynthetic layer and its surrounding soil. An example application is demonstrated which refers to a one-meter geosynthetic-reinforced sand column.

Regarding the durability of the soil bio-cementation mechanism, the soil stabilization agent is stress-tested under different environmental conditions of varying temperature and water salinity. This series of tests reveals the performance of the reactive mechanism for various environments.

$$\mathrm{CH_4N_2O} + 2H_2O \rightarrow CO_3^{2-} + 2NH_4^{+} \quad (1)$$

$$CO_3^{2-} + Ca^{2+} \rightarrow CaCO_3 \downarrow \quad (2)$$

3 Results

Figure 2 illustrates a schematic representation of the reaction, initiated by hydration of the enzymatic agent, and subsequent mineral formation of calcite, which ultimately acts as binder between the geosynthetic layer and surrounding soil, as well as among grains of soil in the vicinity of the geosynthetic element.

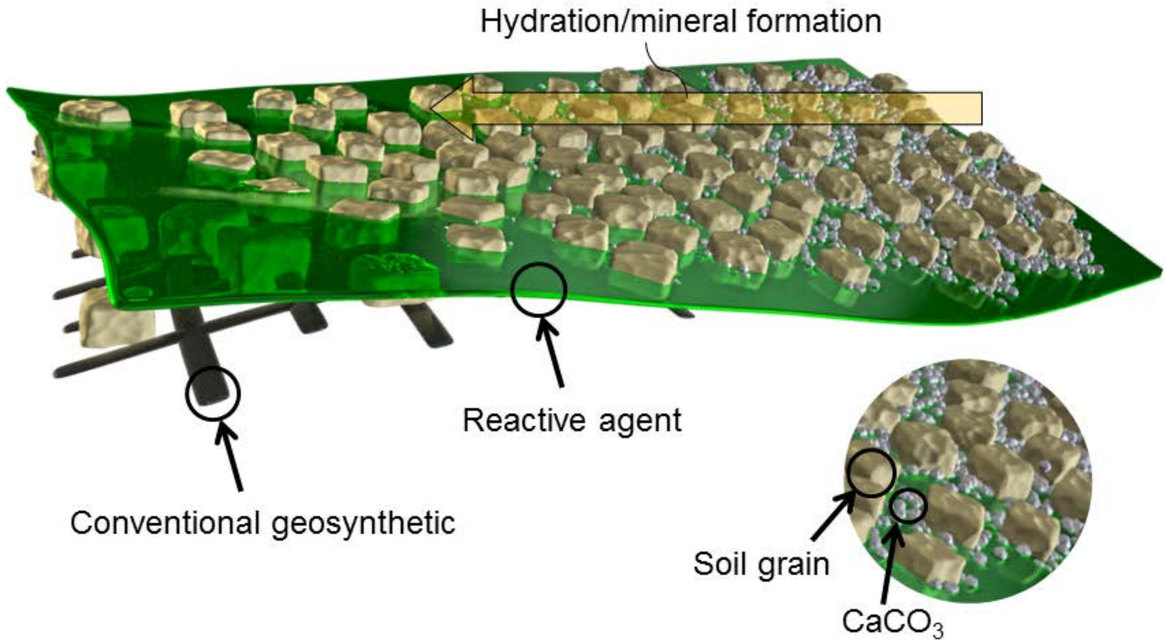

Figure 2: Schematic representation of a typical geo-grid, permeated with the enzymatic bio-reinforcement agent.

The produced geosynthetic-reinforced column is shown in Figure 3. Interestingly, removing completely the geosynthetic was impossible. Its plastic 3D matrix remained integrated into the column after completion of bio-cementation while the fabric part was completely separated. The column retains its integrity in the absence of confinement and remains highly permeable after flushing 100 L of water. Samples were collected across the height of the column to estimate the final calcite content which ranges between 4 - 6 %. Bio-cementation is observed across the height of the column which suggests that reactant agents diffused through gravity flow, despite adopting a single infiltration source at the top of the column.

A better understanding of microstructural properties is further provided. Figure 4 shows the process of volume reconstruction through combination of 2D X-Ray scans, where sand particles (brown) and calcite bonds (green) can be distinguished and analyzed individually. We postulate that higher initial porosity (medium-grained sand) favors precipitation of bond particles which grow their sizes upon continuous infiltration of reactive media throughout the porous network. The mean minimum and maximum diameters of bonds precipitated in medium-grained sand are found to range between 80-370 micrometers for calcite volumetric contents in the range between 3-7.5%. Contrary, MICP treatment in fine-grained sand yields bonds with mean diameters in the range between 100-200 micrometers for calcite contents between 9-13% [5].

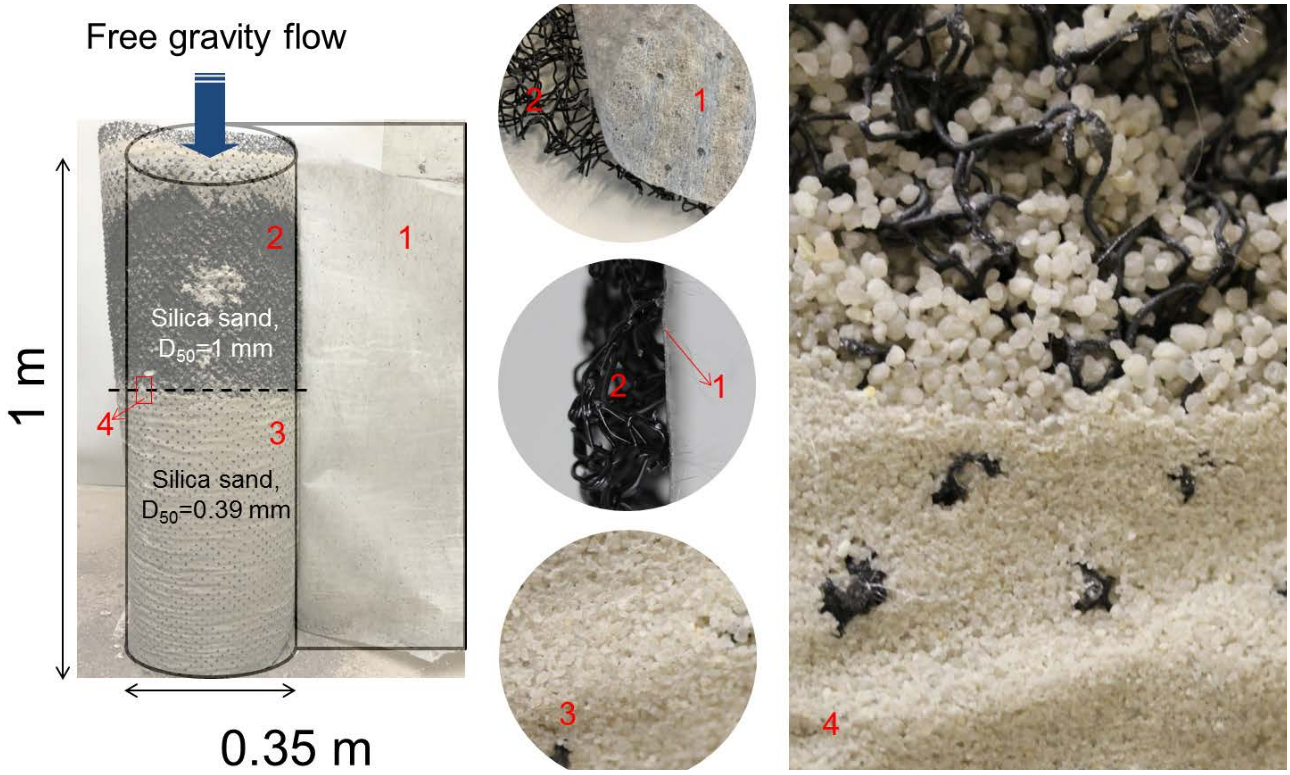

Figure 3: Geosynthetic-reinforced sand column (left); fabric cloth detached from the plastic 3D skeleton after bio cementation (1); 3D plastic matrix of the utilized geosynthetic (2); bio-cemented sand (3); zone between the two different types of sand which were bio-cemented through gravity flow (4).

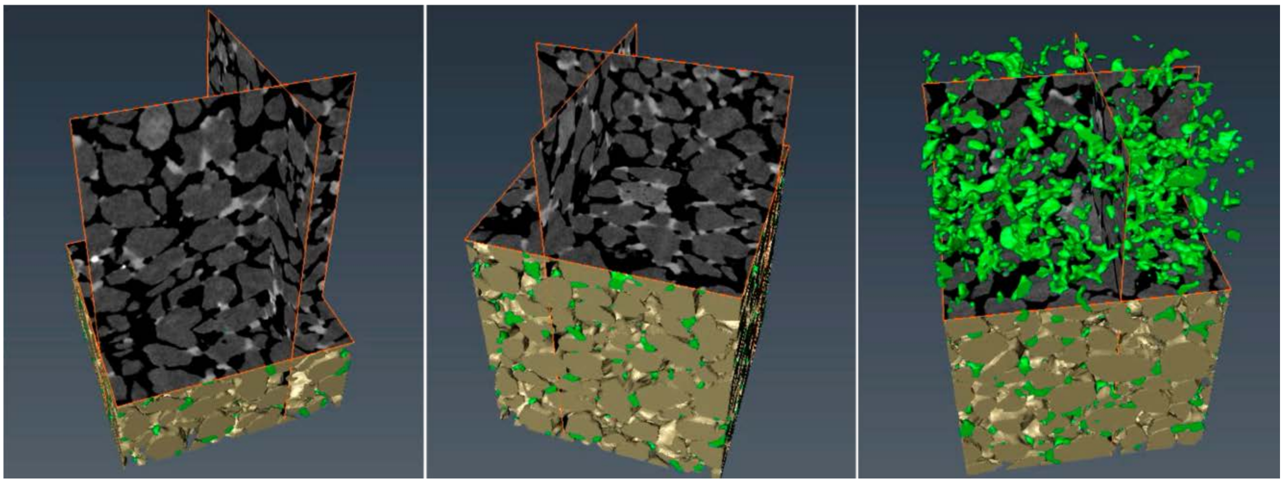

Figure 4: 3D volume reconstruction of bio-cemented sand through combination of micro-CT scans; brown particles represent sand grains and green particles represent calcite bonds in rectangular subvolume with horizontal section equal to 1.5 mm^2.

Further, the agent was tested under a variety of temperatures at low bacterial cell concentration, expressed in optical density (OD_{600}), equal to OD_{600}=0.06. Results in Figure 5 demonstrate a linear evolution of the reaction rate. For comparison, at 5 °C, 1 Mol of calcite would mineralize in a total of 13 days while at 20 °C the same mass of calcite would be produced in less than 3.5 days. One should consider, however, that the above reaction rates refer to very low concentrations of the reactive agent. The overall agent concentration remains to be determined with respect to the nature of the foreseen engineering application and the desired mass of calcite which is necessary to improve the engineering properties of the material.

Finally, the strength and stiffness of bio-cemented fine- and medium-grained sands is presented in Figure 6. The unconfined compressive strength (UCS) of medium-grained sand is found to exhibit higher values for similar range of $CaCO_3$ contents. More precisely, biocemented columns of medium sand reach 11.3 MPa of UCS while samples of fine-grained sand, prepared under identical external conditions, yield UCS values shy of 2.5 MPa.

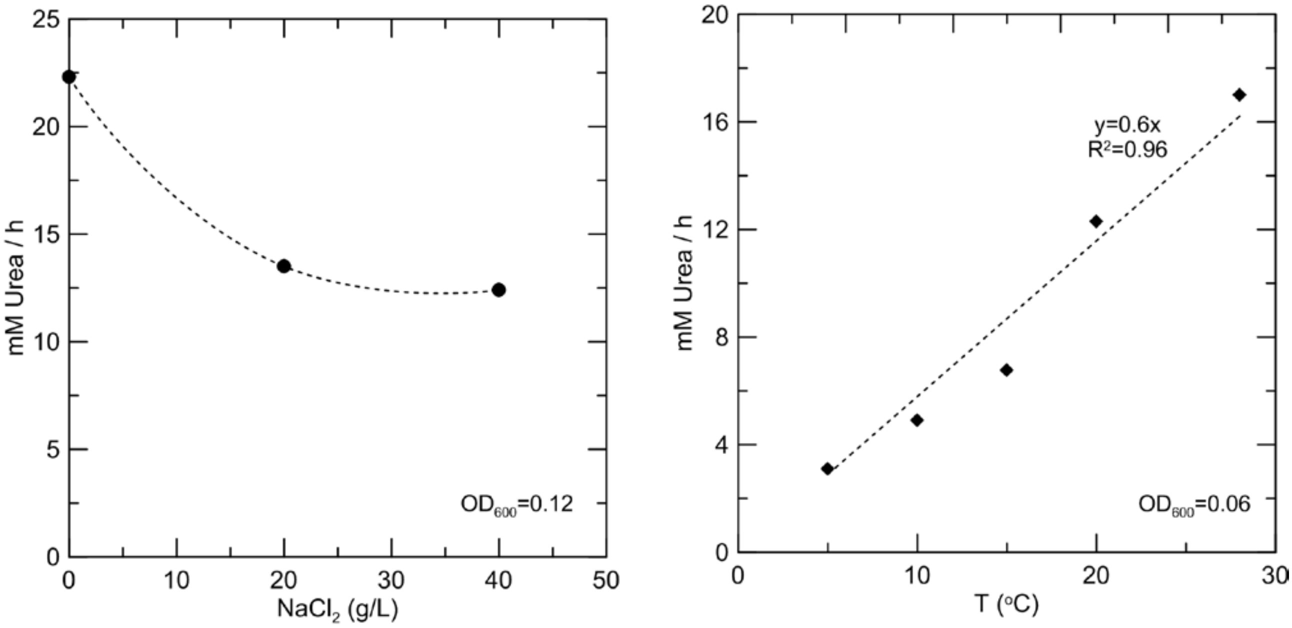

Figure 5: Soil-stabilization agent performance under various environmental conditions expressed in terms of reaction rate (mM of Urea hydrolyzed per hour); (left) increased water salinity and (right) varying temperature.

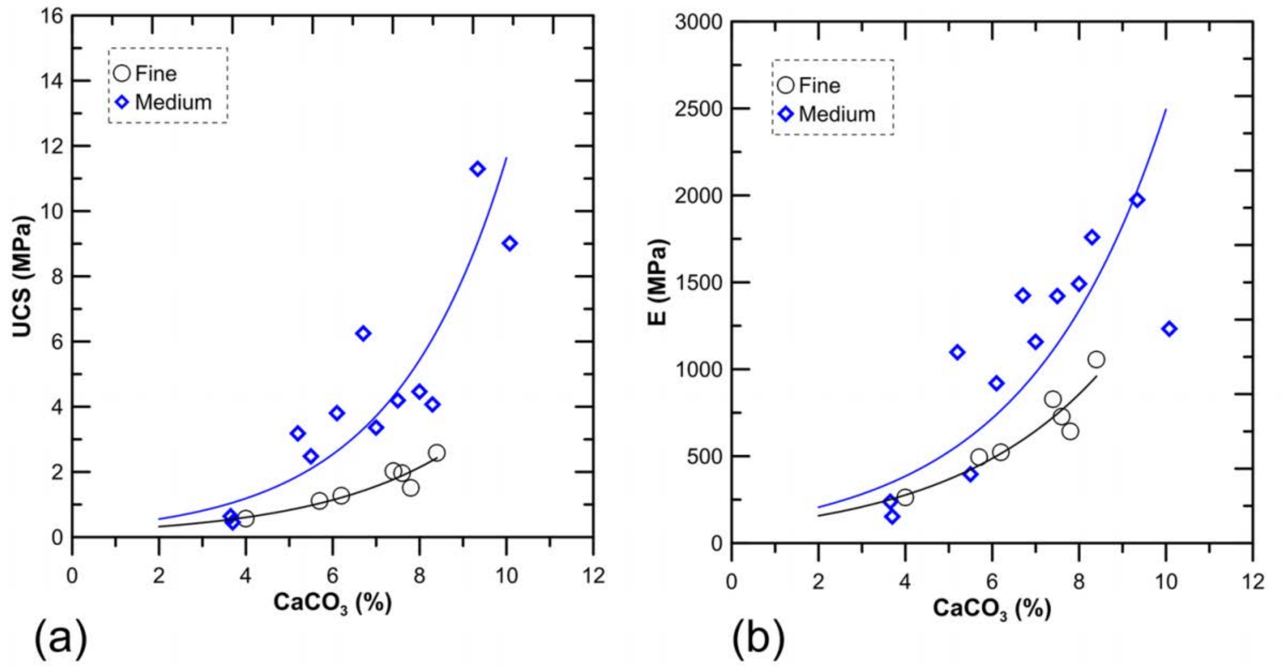

Figure 6: (a) Unconfined compressive strength for fine- and medium-grained bio-cemented specimens; (b) evolution of the Young's modulus for fine- and medium-grained bio-improved sand with respect to increasing bond content.

4 Conclusions

Overall, the technology of soil bio-cementation is introduced as a new approach to earth reinforcement applications which passes through efficient, nature-based and environmentally friendly soil stabilization. A new stabilization agent is developed and employed. In its core we find a reactive enzyme which stimulates a natural process of mineral formation in a cell-free environment. The agent can be either introduced directly into soils or used coupled with geo-textiles, with the latter representing mainstream materials in the geotechnical field.

A 1-meter tall column of geosynthetic-reinforced sand was produced in just 5 days by simple, gravity flow. In addition, mechanical results reveal the improved performance of sand columns stabilized through infiltration with solutions which were rich in soil stabilization agent. Finally, the agent is tested under various environmental conditions, with results suggesting an all-round efficiency under varying water salinity and temperatures.

5 Acnkwoledgments

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JANSEN shark for geothermal systems: efficient, environmentally friendly and long term saving in operating costs

Benjamin Pernter

JANSEN shark for geothermal systems: efficient, environmentally friendly and long term saving in operating costs

1 Widerstandsvermindernde Oberflächen in der Natur

1.1 Die Haifischhaut

Der Hai gehört zu den schnellsten Schwimmern der Tierwelt. Vor allem seine Schnelligkeit macht ihn zu einem überlegenen Jäger im Ozean. Maximale Geschwindigkeit bei minimalem Wasserwiderstand sind für ihn lebenswichtig. Ein sehr besonderes Phänomen verhilft ihm zu einmaliger Energieeffizienz.

Die Schuppen der Haifischhaut tragen scharfe Rippen, genannt Riblets, welche die Strömung des Wassers entlang des Körpers gezielt lenken. (Picture 1) Die feinen Längsrillen auf den Haifischschuppen vermeiden Querbewegungen in der Wasserströmung. Kombiniert mit seiner hydrodynamischen Körperform gleitet der Hai daher pfeilschnell durchs Wasser – beinahe ohne Widerstand.

Bei schnell schwimmenden Haifischen ist an verschiedenen Körperzonen eine an die strömungstechnischen Gegebenheiten angepasste Struktur vorhanden. Wären die Strukturen zu gross, könnten Wirbel zwischen die Riblets wandern, welche dann einen grösseren Strömungswiderstand zur Folge haben. Zu kleine Strukturen wiederum blieben ohne spürbaren Effekt.

Picture 1: Schuppen auf der Haifischhaut weisen eine der Strömung ideal angepasste Struktur auf

1.2 Existierende Technologien

Seit vielen Jahren wird in diesem Bereich der Bionik geforscht und es werden immer wieder neue Produkte lanciert. Beispielsweise wurden Oberflächen für Flugzeuge und Schiffe entwickelt, um deren Geschwindigkeit oder Treibstoffeffizienz zu verbessern. [1] Verbesserte Schwimmanzüge erlauben es Hochleistungssportlern, Kräfte zu sparen und dabei neue Rekorde zu erzielen.

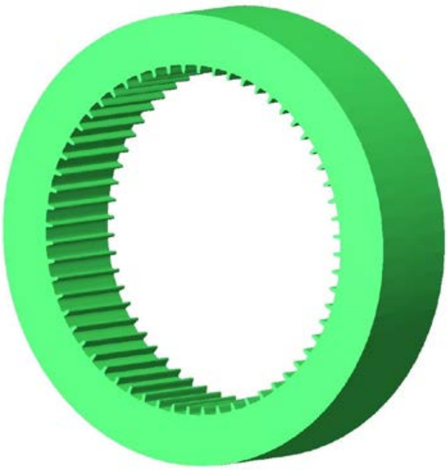
Auch Windräder können von verbesserten Oberflächen profitieren. Eine Verminderung von unerwünschten Luftturbulenzen an den Rotorblättern führt zu einer besseren Nutzung des Energiepotentials im Wind, und dies gänzlich ohne zusätzliche Lasten für die Konstruktion der Windenergieanlage. [2]

2 Adaptierung für Geothermiesysteme

2.1 Entwicklung

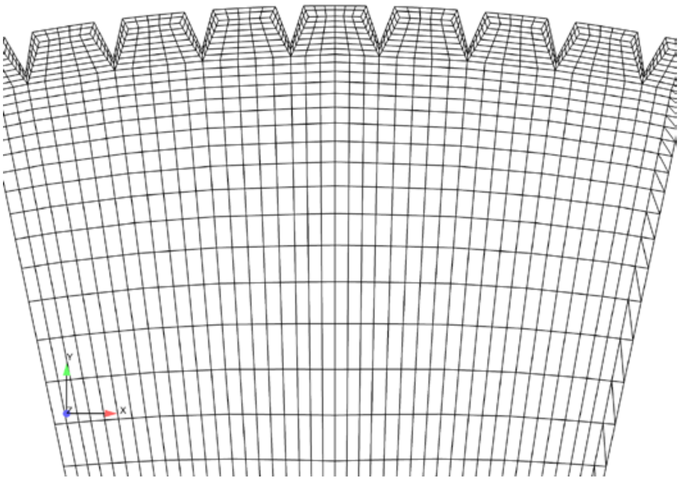
Die Entwicklungsingenieure der Jansen AG wollten diese Effekte auch für Erdwärmesysteme nutzbar machen. In geschlossenen Erdwärmetauschern werden laufend grosse Mengen an Sole umgewälzt. Eine Reduktion des hydraulischen Widerstandes wirkt sich somit unmittelbar auf den Strombedarf und somit die Energieeffizienz des gesamten Wärmepumpensystems aus.

In Kooperation mit dem Institut für Energietechnik der Hochschule Rapperswil «IET/HSR» (Schweiz) und den erfahrenen Forschern der bionic surface technologies GmbH (Österreich) hat Jansen ein Kunststoffrohr entwickelt, das den widerstandsvermindernden Effekt der Hautoberfläche des Haifisches nachahmt: die JANSEN shark Technologie. Die Technologie wird in den Herstellungsprozess der extrudierten Rohre integriert, sodass für den Installateur oder Anwender keine weitere Bearbeitung notwendig ist. (Picture 2) Die fertigen Bunde in gängigen Rohrdimensionen zwischen üblicherweise 32 und 50 mm können dann im Erdreich installiert werden.



Picture 2: Die Herstellung der Struktur im Rohrinneren wurde erfolgreich in den Produktionsprozess integriert.

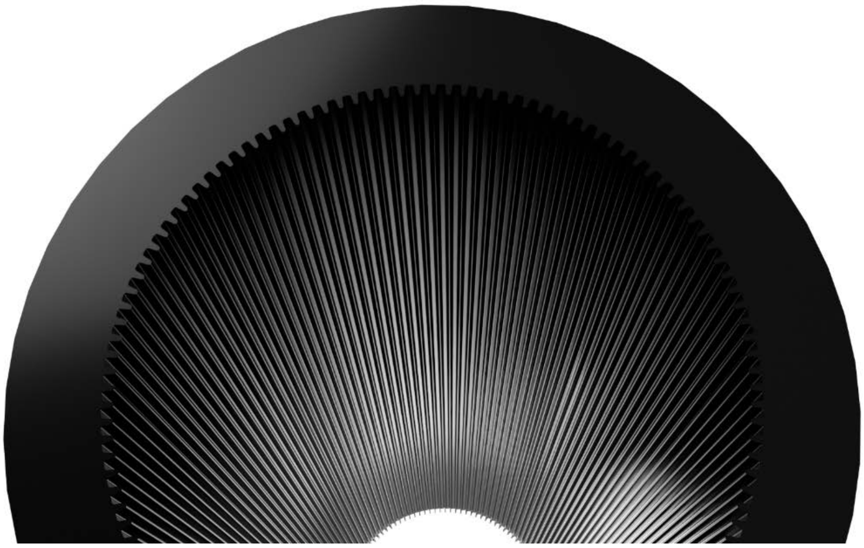
Herausforderung hierbei war unter anderem, die für die Anwendung in Erdwärmesystemen ideale Oberflächenstruktur zu finden, die den bestmöglichen Effekt liefert, ohne ins Nachteilige abzudriften. (Picture 3) Hierzu war es nötig, verschiedenste Strukturen zu simulieren, zu produzieren und auch in der Praxis zu testen. Die finale Struktur im Rohrinneren ist nun optimal an das Strömungsverhalten des zirkulierenden Solemediums angepasst und bewirkt so eine deutliche Verringerung des hydraulischen Widerstandes. Die Oberflächenstruktur wurde zum Patent angemeldet.



Picture 3: Nach mehreren Simulationen und Entwicklungsschritten wurden Strukturen ermittelt, die ideal an die Strömung in Erdwärmesystemen angepasst sind.

2.2 Vorteile

Mit über 60 Jahren Erfahrung in der Entwicklung und Herstellung innovativer Kunststoffsysteme steht Jansen als Schweizer Industrieunternehmen sowohl für höchste Qualität und Präzision als auch für wegweisende Hightech-Lösungen. Mit der JANSEN shark Rohrtechnologie (Picture 4) ist es gelungen, die Effizienz von Erdwärmesystemen auf ein neues Niveau anzuheben. Dabei ergeben sich noch zusätzliche Vorteile.



Picture 4: Neben verbesserter Pumpeffizienz bietet die JANSEN shark Technologie weitere Vorteile.

2.2.1 Reduzierter Strombedarf

Gegenüber herkömmlichen Glattrohren kann der Druckverlust um 7% reduziert werden. (Chart 1) Das wirkt sich sofort positiv auf die benötigte Pumpleistung und damit den Stromverbrauch der Umwälzpumpe aus. Systeme mit der JANSEN shark Technologie können je nach Wirkungsgradkurve der Solepumpe deutlich Strom einsparen. Dank der angepassten Oberflächenstrukturen ist der Normalbetrieb von Erdwärmesystem gleichzeitig in den allermeisten Fällen auch der optimale Betrieb.

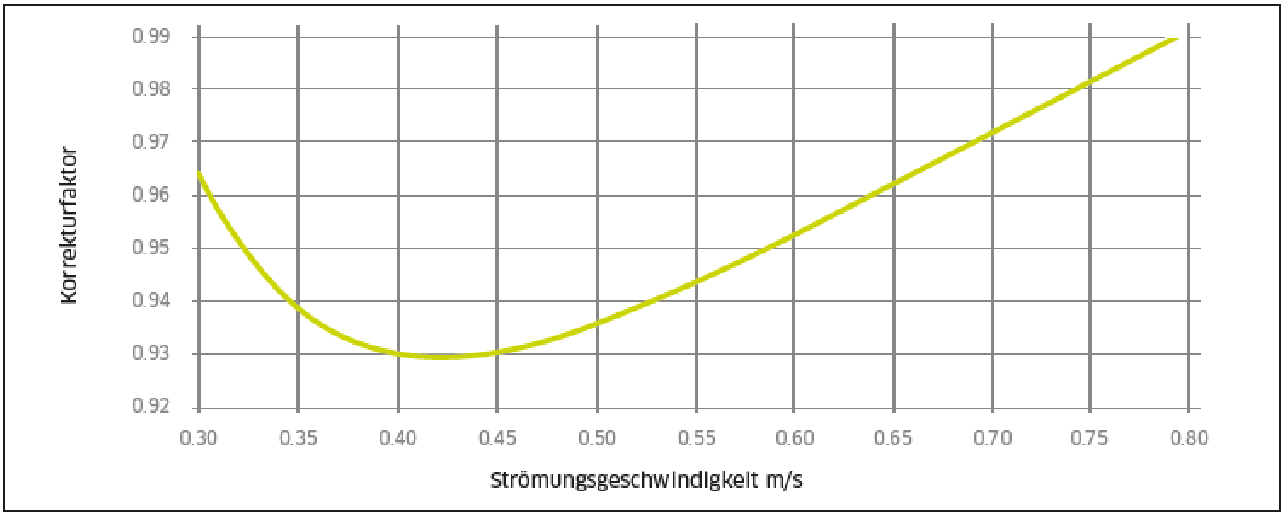


Chart 1: Gegenüber herkömmlichen Glattrohren kann der Druckverlust um 7% reduziert werden.

2.2.2 Erweiterte Einsatzmöglichkeiten

Die JANSEN shark Technologie erlaubt es, bei tiefen Erdwärmesonden tendenziell kleinere Rohrdimensionen einzusetzen, da der ansonsten mit zunehmender Sondenlänge zu stark steigende Druckverlust unmittelbar kompensiert wird. Dies wirkt sich nebst einer erhöhten Effizienz der Umwälzpumpe, geringeren Betriebskosten und einem vereinfachten Einbau auch positiv auf die gesamten Investitionskosten aus.

2.2.3 Günstigere und schnellere Befüllung

Die veränderte Innenoberfläche sorgt für ein geringeres Füllvolumen. Deshalb wird insgesamt weniger Wärmeträgermedium benötigt. Frostschutz ist teuer; bei JANSEN shark Erdwärmesystemen ist somit auch das Befüllen um bis zu 4% günstiger als bei herkömmlichen Glattrohren.

2.2.4 Vertrautes Handling

Die Dimensionen der JANSEN shark Erdwärmerohre entsprechen den gängigen Rohrtypen. Es ist keine weitere Bearbeitung notwendig. Der Einbau der Rohre erfolgt wie gewohnt einfach und sicher mit herkömmlichen Gerätschaften, Elektroschweissfittings und Materialien.

2.2.5 Gewohnte thermische Auslegung

Die vergrösserte Wärmetauscheroberfläche im Rohrinnern führt selbst bei Rohren mit höherer Druckstufe und damit dickerer Rohrwand zu einem gewohnt ausgezeichneten Wärmefluss. Somit erfolgt die thermische Planungs- und Auslegungsphase sicher und reibungslos mit herkömmlichen Kennzahlen.

2.3 Anwendungen

Die JANSEN shark Technologie profitieren Erdwärmesysteme jeglicher Art, sowie verwandte Systeme in der Haustechnik und in ähnlichen Branchen. (Picture 5)

Picture 5: Erdwärme-Anwendungen der JANSEN shark Technologie

2.3.1 Erdwärmesonden

Mit der JANSEN shark Technologie profitieren Erdwärmesonden jeder Tiefe und Ausführung von einem geringeren Druckverlust. Bei sehr tiefen Erdwärmesonden ist neben einem möglichst geringen hydraulischen Widerstand auch die Druckstabilität des eingesetzten Rohres ausschlaggebend. Die ebenfalls von Jansen entwickelte JANSEN hipress Tiefen-Erdwärmesonde mit ihrem diffusionsdichten Metallmehrschichtaufbau und dem mit einem Metallmantel ausgestatteten Hochdruck-Sondenfuss hält höchsten Druckbelastungen von PN35 und rauen Baustellenbedingungen stand. (Picture 6) Die JANSEN hipress Tiefen-Erdwärmesonde kann zusätzlich mit der JANSEN shark Technologie ausgestattet werden. So werden neue Dimensionen in der Nutzung von Erdwärme erreicht.

Picture 6: Die JANSEN hipress Tiefen-Erdwärmesonde profitiert ebenfalls von einem geringeren Druckverlust mit der JANSEN shark Technologie

2.3.2 Anbindeleitungen

Die Leitungen zur Verbindung der Erdwärmesonden mit der Wärmepumpe können bei Verwendung von normalen Glattrohren bis zu einem Drittel des gesamten Druckverlustes ausmachen. Auch diese Rohre können mit der JANSEN shark Technologie ausgeführt werden, um die Gesamteffizienz der Anlage markant zu verbessern. Die Betriebskosten werden minimiert, die Amortisationszeit der Gesamtanlage verkürzt.

2.3.3 Erdwärmekollektoren

Auch Flächenkollektoren profitieren von einem geringeren Druckverlust. Mit den JANSEN shark Kollektorrohren sind längere Solekreise einwandfrei möglich. Das führt zu einem vereinfachten Einbau und einem kleineren und damit kostengünstigeren Verteilersystem.

2.3.4 Ausblick

Ein Blick in die Zukunft zeigt, dass die Technologie noch ausbaufähig ist und auch in anderen Anwendungen zum Einsatz kommen kann; beispielsweise in Fussbodenheizungen, Anergienetzen und weiteren Systemen, die von einer hohen Effizienz bei einem geringen Rohrdurchmesser profitieren.

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Cased CFA – Combining the Advantages of Safe and Fast Pile Installation

Franz-Werner Gerressen

Cased CFA – Combining the Advantages of Safe and Fast Pile Installation

1 Introduction

The first double rotary drilling method was developed around 40 years ago for the installation of deep excavations close to existing structures in inner cities. The main advantage was to minimize the distance between the existing building and the required retaining wall. The "Front of Wall" (FOW) method used two separate rotary heads for the auger and the casing, which are rotating opposite directions simultaneously. Initially the system was limited to relatively small diameters. The development of larger and more powerful drilling rigs has allowed the double rotary system to install cased piles with a diameter of up to 1,200 mm for various applications, namely King Post, contiguous and secant pile walls and foundation piles.

2 Construction principle

2.1 Installation procedure

The basic principle of a CCFA pile is that a continuous flight auger in combination with an outer casing is drilled simultaneously, but in opposite directions into the ground to the required depth (Figure 1, Step 2). These characteristics led to the common term Cased Continuous Flight Auger Pile (CCFA). The spoil is transported upwards by the auger flights surrounded by the casing and exits through openings at the top of the casing. Once the final depth is reached (Figure 1, Step 3) the concrete pump is activated and concrete is pumped through the hollow stem of the auger. After completion of concreting the drill string has to be cleaned (Figure 1, Step 4). The cleaning is done by rotating the casing and the auger in the opposite direction than during drilling. In some geology it can be beneficial to postpone the cleaning after reinforcement installation (Figure 1, Step 5).

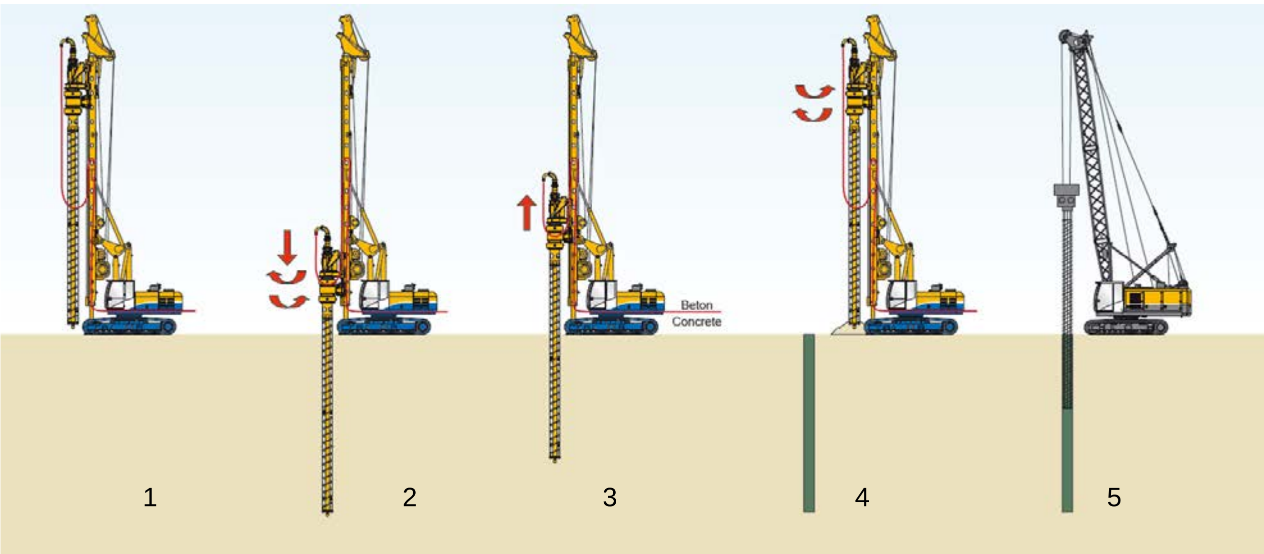

Figure 1: CCFA Installation Process

The CCFA method is suitable for cohesive soils and granular soils with sufficient cohesive content to allow the soil transport on the auger, were the undrained shear strength “cu” should be above 15 kN/m² and no boulder should be present. Rock socketing is possible to UCS values of approx. 20 MPa.

2.2 Equipment

This procedure requires double rotary drive systems to run auger and casing at the same time. Actually, different setups of double rotary drive systems exist. One option is the use of two rotary drives, either working independent or dependent to each other (Figure 2, left). With a specific geometry this system is also known as Front of Wall (FoW) system (Figure 2, centre). Another option is to use one rotary drive in combination with a torque multiplier (Figure 2, right). The torque multiplier is a mechanical unit attached to the cardanic joint below the rotary drive. The ratio is fixed at 1:2, so that the torque multiplier doubles the incoming torque, halves the speed and inverts the direction of rotation of the casing.

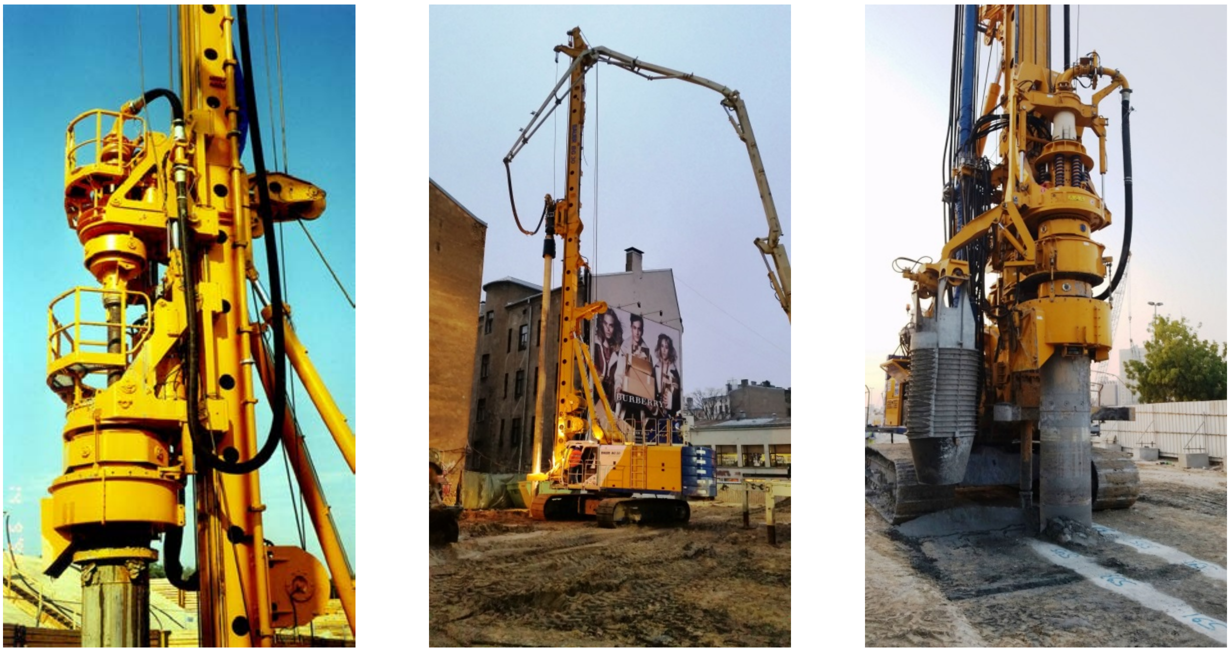
Figure 2: CCFA configurations, two rotary drives (left), FoW (centre), rotary drive with torque multiplier (right)

2.3 Quality control system of CCFA method for secant pile walls

The CCFA method is mainly used for the installation of secant pile walls. The order of installation reduces many of the sequence issues experienced with traditional methods. The quality standard for pile walls is necessarily very high as the costs in case of repairs would be significant and repair work time consuming. It is therefore essential to employ an accurate quality surveillance system. As primary requirements, a stable working platform with less than 3% inclination and a guide wall as shown in Figure 3 are highly recommended.


Fig. 3. Typical concrete cast guide wall.

The guide wall ensures the correct starting location of every pile and facilitates the set-up of auger and casing. The verticality of the casing is checked manually with a water level. However, the verticality of the mast is both measured and recorded by electronic sensors. Verticality corrections are easily made in the x & y directions either manually or automatically, ensuring that the mast and tools are kept vertical at all times. For quality control all drilling data and data relative to the verticality of the mast are available in real time.

Drilling assistant CCFA

Many factors contribute to the efficient drilling and subsequent concreting of CCFA piles. To assist and allow the operator to monitor all the various parameters, drill and extraction assistant programs have been developed (Figure 4). During the drilling phase the optimal performance is affected by the applied torque and crowd forces and rotation speed. These will need to be varied for each particular soil conditions encountered. In addition to these rig-related factors the geometry of the casing and auger type also has an influence. All these factors are used in the drill assistant program to achieve the desired optimal performance, which is measured as the penetration per revolution. Control of this measure is widely accepted as a major factor in avoiding problem in CFA and CCFA piling. In particular there are the following benefits:

- avoids over-flighting (crowd speed too slow in combination with too many flight rotations, therefore loosening of surrounding soil)
- avoids corkscrew of auger (Crowd speed too quick in combination with too little flight rotations, cork screwing effect)
- optimizes the filling grade of the auger
- assists the operator.

Extraction assistant for CCFA

An efficient concreting process is achieved by monitoring a combination of the concrete volume, concrete pressure and the extraction rate. There are two basic methods of measuring the volume of concrete supplied by a concrete pump. The first is the direct method where the volume is measured by a flow meter placed in the concrete pipe work. Another method is to measure each stroke of the concrete pump. A pressure gauge is installed in the concrete delivery pipe at the drilling rig. The advantage of this system is its simplicity. Any considerable mistake can be encountered easily on site. Each stroke that is not counted or counted too much is shown as an abrupt discontinuity of the concrete flow. Therefore it is important to maintain the speed of the concrete pump.
Now the extraction assistant for the concreting process has sufficient information to guide the equipment for the casting of the pile. The retraction speed is calculated by volume formulas. A significant over consumption of concrete depending on ground conditions, is adjusted in the program. The working screen (Figure 4) of the equipment displays all important information of the production process in real-time.

Figure 4: B-Tronic screenshots – left: Drilling Assistant – right: Pulling Assistant

Verticality

The verticality of the pile depends on an accurate alignment of the casing, the casing guide and the stiffness of the drilling tools. The relative flexural rigidity of a casing is min. 100 times higher than the rigidity of a CFA auger. The opposed drilling direction of casing and auger increases the stability additionally as any deflection is compensated and the drill string is stabilized. The high grade of verticality and pile quality could be seen impressively on a jobsite where standard CFA and CCFA piles were used next to each other.
During the execution of the pile wall by the CFA method, the operator noticed already a deflection in the verticality of the piles. Especially when drilling a secondary pile, the auger drifted out of direction. Therefore the piling method was changed to CCFA. The result was an imposing improvement of pile quality in the same soil conditions, which could be seen after the excavation of the pit. With the CCFA method a 1 in 200 verticality tolerance could be achieved, whereas the secondary CFA piles show significantly higher deviations

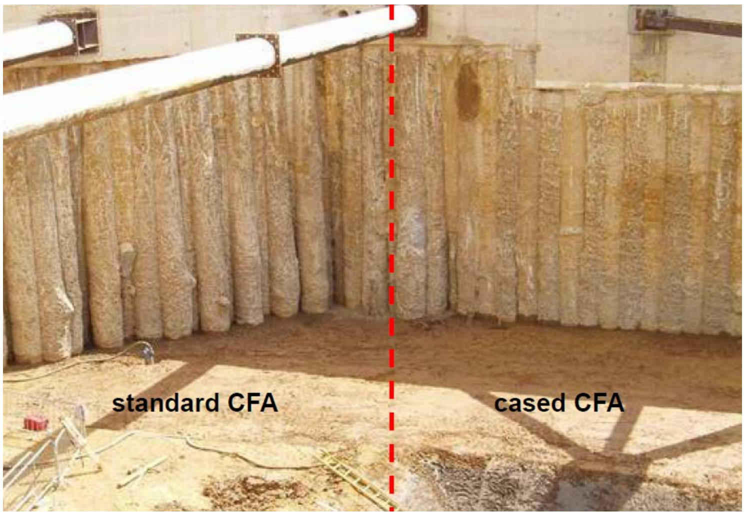

Fig. 5. Secant pile wall, diameter 750 mm.

3 Conclusion

The major application for the CCFA system is in the construction of secant pile walls. Compared to the traditional systems with fully cased bored piles in Kelly mode or standard CFA piling, the CCFA system has advantages in costs and time:

- improved installation times
- improved verticality tolerance
- pile is protected from influence of surrounding ground (e.g. ground water)
- reduced concrete consumption
- consistent performance of drilling and concreting processes using the Assistant Instrumentation Programs
- clean jobsite due to absence of water
- straight forward conversion of the drilling rig
- penetration through hard layers possible
- reduced noise disturbance compared to cased piles in Kelly mode

Especially at the beginning of projects the team needs to gain some experience to adjust the method to the soil conditions. The drilling rig must have sufficient torque, pull down and extraction force to keep the drilling process steady at all times, even in difficult soil conditions. A consistent and reliable concrete supply must be organized as part of the work preparation to ensure a constant operating sequence. Good communication and coordination among the crew on site is important and changes in staff should be avoided.

These improvements and advantages contribute to a better quality product. However, these will only be successful if used properly. Therefore the importance of training, understanding and briefing to all personnel involved is vital to its success. Considering that, the CCFA system can be seen as a significant improvement for the installation of secant pile walls.

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Application and capabilities of energy geostructures in present and future smart built environments

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Application and capabilities of energy geostructures in present and future smart built environments

1 Introduction

The global energy market has been historically dominated by the use of non-renewable energy sources, which by definition cannot renew themselves at a sufficient rate in human time frames. In this context, the combustion of fossil fuels has played a central role to meet the world energy consumption, with an influence on the emission of greenhouse gases, the pollution of the environment and the contribution to the phenomenon of global warming. Fossil fuels have met at least 60% of the final energy consumption (since the mid-XX century) up to recent years. Fossil fuels have also generally met at least 80% of the total primary energy supply (since the mid-XX century) up to recent years. At the same time, 40% of the final energy consumption may be associated to the building sector, both in developed and developing countries [1]. In Switzerland, for example, from approximately 60 to 85% of the final energy consumption may be associated to the building sector for space conditioning (heating and cooling) and the production of hot water [2].
A development that meets human activity, needs and progress goals with a limited impact on the environment undoubtedly contributes to restraining environmental pollution. In the construction sector, national and international directives and regulations are increasingly requiring, or promoting, to resort to so-called "environmentally friendly" technologies that involve a limited impact on the environment by harvesting renewable energy sources. In many countries, all new buildings and infrastructures will soon need to be constructed with such technologies, whereby the harvested renewable energy sources will need to be present on-site. The ultimate purpose is to establish so-called "low-carbon built environments".
Shallow geothermal energy, i.e., the thermal energy present in the underground within the first 400 m of depth, is a renewable energy source available continuously, everywhere and irrespective of the weather. It represents one of the man's oldest methods to achieve heating and cooling applications as well as the storage of thermal energy in the subsurface by resorting to soils and rocks as heat exchanger, storage or extraction media. Along with the ancient use of soils and rocks as heat exchanger, storage or extraction media, the employment of geostructures (e.g., piles, walls, tunnels, etc.) for structural support purposes represents an effective historical means to meet human activity. Energy geostructures are an innovative, environmentally friendly technology that couples the structural support role of the ground structures with the role of the geothermal heat exchangers, harvesting renewable energy for the heating and cooling of the built environment.
This work addresses the technology of energy geostructures, with the aim to present the features of this technologies as well as its capabilities in contributing to a sustainable development via the establishment of low-carbon buildings and infrastructures.

2 The energy geostructure technology

2.1 Roles of energy geostructures

Energy geostructures, more properly defined in a theoretical sense as thermo-active geostructures, are an innovative technology that couples the structural support role of conventional ground structures with that of the ground heat exchanger of conventional shallow geothermal systems. This technology includes all ground-embedded structures that can be used as structural supports while exchanging heat with the ground. Similar to other shallow geothermal systems, energy geostructures deal with low enthalpy and take advantage of the ground as a heat exchanger and storage medium, rather than as a heat source. This fact follows the relatively constant temperature field in the shallow subsurface throughout the year involving a warmer temperature than the ambient temperature in a cool weather or during cold seasons, and a cooler temperature in a warm weather or during hot seasons [3].
Energy geostructures can involve deep foundations (e.g., piles, piers, barrettes), earth retaining structures (e.g., diaphragm walls and sheet pile walls), shallow foundations (e.g., footings, base slabs), tunnels linings and anchors as well as pavements. The resulting geostructures are so-called energy piles, energy walls, energy slabs, energy tunnels, etc. (cf., Figure 1).
Various are the purposes of the heat exchange that can be established with energy geostructures [4]. These can consist in (i) heating and cooling superstructures to reach comfort levels in the built environment, (ii) contributing to the production of hot water for anthropogenic, agricultural or tank-farming uses, (iii) providing heat to prevent the icing of pavements and decks of infrastructures such as roads, bridges, station platforms and airport runways, and (iv) storing heat in the subsurface for a successive use.

Picture 1: Examples of energy piles constructed at the Swiss Federal Institute of Technology in Lausanne, EPFL, Switzerland.

2.2 Materials and technology

Energy geostructures are typically made of reinforced concrete. From a technological perspective, they differ from conventional geostructures only because pipes are fixed along their reinforcing cage or are placed within the filling material (cf., Figure 2). The former application is more frequent especially when dealing with energy walls or tunnels, whereby potential issues due to maintenance of the geostructure or adjacent environment are avoided by placing the pipes along the reinforcing cage on the groundside.

Inside these pipes, a fluid is pumped via electrically driven machines and is used as a thermal energy carrier for the operation of the energy geostructures as in most shallow, closed-loop geothermal systems. Energy geostructures are closed-loop, shallow geothermal systems too.

The pipes mounted along the reinforcement of energy geostructures are usually made of high-density polyethylene and are characterised by a diameter of 20 to 40 mm with a wall thickness of 2 to 4 mm. Two or more pipe loops can be installed in series or in parallel. Typical configurations are the U-shape, double U-shape, W-shape and spiral shape. Thermal insulation of the pipes can be considered for the first meters of the inlet and outlet to limit the influence of the climatic condition on the heat exchange process, aiming at optimising the energy efficiency [3] (cf., Figure 2).

Fixing the pipes to the reinforcing cage of energy geostructures can be performed either in a plant or on site. The latter is more common [5], whereby the piping is delivered to site on reels and a special working area is used. At the inflow and outflow of the pipework of each energy geostructure, a locking valve and a manometer are fixed [5]. These instruments allow the pipe circuit to be pressurised within a range of 5 to 8 bar for integrity check. In most applications, the locking valves and manometers are also used upon concreting to resist the head of the wet concrete without collapsing. Pressure testing for 24 hours after concreting is good practice. The pressure in the pipes is again applied before the working phase involving the construction of the superstructure starts [5].

The heat carrier fluid (i.e., the heat transfer medium) usually consists of water, water with antifreeze (e.g., glycol), or a saline solution. Glycol–water mixtures containing additives to prevent corrosion are also a well performing and durable solution.

Picture 2: Typical pipes and thermal insulation in an energy pile constructed at the Swiss Federal Institute of Technology in Lausanne, EPFL, Switzerland.

2.3 Advantages involved with energy geostructures

Similar to other technologies harvesting renewable energy, such as conventional geothermal systems, energy geostructures are an environmentally friendly technology that reduces the need of fossil energy sources and hence the greenhouse gas emissions. For this reason, the use of energy geostructures promotes and complies with national and international initiatives, policies, regulations and agreements such as those highlighted above. Furthermore, energy geostructures may be applied with other technologies harvesting renewable energies to form highly efficient systems.

In contrast to conventional shallow geothermal systems, the earth-contact elements that characterise energy geostructures and serve as heat exchangers are already required for structural reasons and need not to be constructed separately [5]. This fact involves savings related to the construction process that should be undertaken in a separate realisation of geostructures and geothermal heat exchangers.

Another key difference between energy geostructures and other conventional, closed-loop geothermal systems is that concrete has more favourable thermal properties than the filling materials (e.g., bentonite) of the other geothermal technologies. This feature makes the heat exchange more favourable in the former case compared to the latter. One final difference is that usually the bending radius of the pipes in energy geostructures in greater compared to that characterising the pipes in conventional geothermal heat exchangers. This fact involves a lower flow resistance of the fluid circulating in the pipes, which results in a lower pumping power and, thus, in a lower operation cost.

With reference to the purposes of the heat exchange that can be established with energy geostructures, various are the advantages included with energy geostructures compared to other technological systems [4]. The use of energy geostructures for heating and cooling superstructures to reach comfort levels in the built environment reduces the environmental impact of any construction and can be exploited to get incentives for the design project and construction of the superstructure. The use of energy geostructures for contributing to the production of hot water for anthropogenic purposes reduces the costs compared to systems entirely resorting to more conventional technologies and is again characterised by a reduced environmental impact. When energy geostructures are employed for contributing to the production of hot water for agricultural or tank-farming uses, cost savings can be achieved via lower operational costs and environmental impacts. The use of energy geostructures for providing heat to prevent the icing of pavements and decks of infrastructures such as roads, bridges, station platforms and airport runways involves reducing the environmental impacts of these applications, because the use of salts or grits in not necessary, and achieving cost savings for the infrastructure owners. The use of energy geostructures for storing heat in the underground allows harvesting waste heat that would be lost otherwise.

3 Energy geostructure operation modes

3.1 Possible operations

Two main operation modes of energy geostructures involving a markedly different conceptual purpose can be employed [4]: the heat exchange operation and the heat storage operation. Depending on whether energy geostructures are used for heat exchange or storage purposes through the respective operations, so-called “Ground Source Heat Pump Systems” (G.S.H.P.S.) and “Underground Thermal Energy Storage Systems” (U.T.E.S.) are employed, respectively. Figure 7 presents a schematic of typical energy geostructures operation modes.

3.2 Heat exchange operation

In this operation, the primary purpose of energy geostructures is to use the ground as a heat exchanger medium. The heat present in the ground is typically extracted and transferred to the superstructure in cool climates or cold seasons. On the contrary, the heat is typically extracted from the superstructure and injected in the ground in warm climates or during hot seasons.
Two possible uses of the energy geostructures are possible for the heat exchange operation mode [4]:

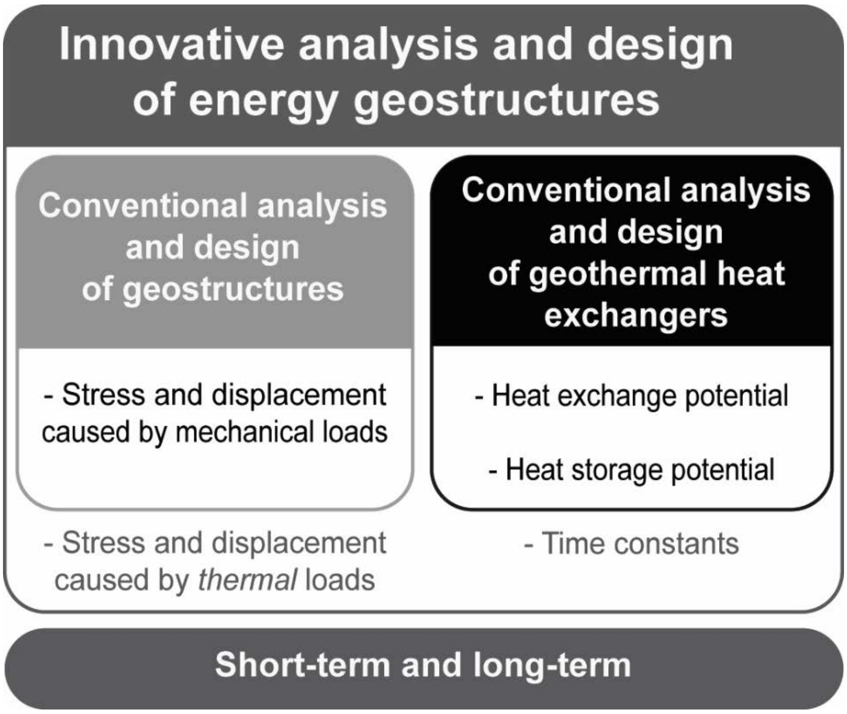
a. *Pure heating and/or cooling only* can be employed when the natural thermal recharge occurring in the ground during non-operating periods of the energy geostructure system is sufficiently high to keep the shallow temperature field in the subsurface undisturbed (except for the influence of climatic conditions) over time. This situation generally characterises energy geostructures in permeable soil with significant groundwater flow. A seasonal heating-cooling operation of energy geostructures can be considered the most economical and environmentally friendly.
b. *Heating and/or cooling combined with heat storage* has to be employed when the natural thermal recharge occurring in the ground during non-operating periods of the energy geostructure system is insufficient to keep the shallow temperature field in the subsurface undisturbed (except for the influence of climatic conditions) over time. This situation generally characterises energy geostructures in low permeable soil with no groundwater flow.

3.3 Heat storage operation

In this operation, the primary purpose of energy geostructures is to use the ground as a storage medium. Waste heat and solar heat is typically injected in the ground. While solar heat is usually injected in warm climates during hot seasons for a successive heating use in cold seasons, waste heat can be stored for a successive use in the ground for both heating and cooling purposes in cool climates or cold seasons and in warm climates or hot seasons, respectively. Heat storage is often required when heating or cooling needs do not match the heating or cooling productions.

4 Energy geostructures application: analysis and design

The multifunctional operation of energy geostructures as structural supports and geothermal heat exchangers involves various challenges for the application of this technology. These challenges concern planning, analysis, design (e.g., geotechnical, structural and energy), construction and maintenance aspects in the design project of buildings and infrastructures employing energy geostructures. The focus is given here to the challenges belonging to the analysis and design of energy geostructures themselves, because these activities crucially contribute to the suitable conceptual development and operation of such technology in the broad picture of the design project of constructions.
The multifunctional role of energy geostructures involves mechanical and thermal loads applied to such elements. These loads cause variations of the temperature, stress, deformation and displacement in the subsurface that need to be considered in analysis and design (cf., Figure 3).



Picture 2: Typical pipes and thermal insulation in an energy pile.

The temperature changes caused in the energy geostructures and the subsurface crucially characterise the thermal response, as well as the energy behaviour and performance of such technology. A sound analysis of this problem, specific for any site and energy geostructure, is essential to ensure an adequate energy behaviour and performance, and justify the use of geostructures as geothermal heat exchangers. This is true for both short- (e.g., hourly, daily) and long-term (e.g., weeks, years) periods.
The stress, deformation and displacement field in the subsurface are not only caused by the conventionally applied mechanical loads to the geostructures, but in the case of energy geostructures by the unprecedented thermal loads applied to such technology as well. Thermal loads are responsible for a dual interplay of stress and strain development in the geostructures and the subsurface. This interplay depends on whether the expansion or contraction caused by the application of the thermal loads to energy geostructures and the surrounding ground is restrained or allowed. The stress, deformation and displacement caused in the energy geostructures and the subsurface crucially characterise the mechanical response, as well as the geotechnical and structural behaviour and performance of such technology. A comprehensive analysis of this problem, specific for any site and energy geostructure, is essential to ensure an adequate geotechnical and structural behaviour and performance, and justify the use of geostructures as structural supports. This is true for both short- and long-term periods.
A substantial amount of knowledge can be considered for addressing the previous challenges. This knowledge includes, for example, (i) the results of full-scale field tests on energy geostructures [6-9] that can be employed to understand the fundamental behaviour of such technology, (ii) modelling tools [10-14] that can be employed to predict the response of energy geostructures, and (iii) data from experimental laboratory tests on soils and soil-concrete interfaces [15-17] that can provide further insights into these multidisciplinary and multiphysical problems.

5 Concluding remarks

This paper expanded on the technology of energy geostructures, highlighting the key features of this revolutionary technology for the construction sector. The capability of energy geostructures in contributing to achieve and improve human progress goals in an environmentally friendly way through the establishment of low carbon buildings and infrastructures was discussed. The roles and possible operation modes of energy geostructures were described and the unique capabilities of energy geostructures in serving for heating and cooling, hot water production, de-icing and storage applications were discussed. Challenges involved with the application of the energy geostructure technology were also analysed and references were proposed to serve as examples of the currently available knowledge that may be employed to address the previous challenges. Currently, the available competences allow covering the essence of the interdisciplinary and integrated aspects required in the geotechnical and structural analysis and design of energy geostructures that civil engineers, architects and urban project managers have to face when addressing such innovative technology. Much improvement can still be targeted in the analysis and design of energy geostructures based on new knowledge in this scope, but excellent results both in terms of economic investment and performance of the adopted technological solution can already be achieved.

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Seismic retrofit of existing bridges taking advantage of nonlinear soil response

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Seismic retrofit of existing bridges taking advantage of nonlinear soil response

1 Introduction

The extension of motorway infrastructure, to adapt with increasing mobility demands, represents an important challenge worldwide. Most commonly, the reduction of bottlenecks in critical areas is achieved by the construction of additional traffic lanes, which often involves the widening of existing bridges. The existing foundations need to be retrofitted in order to support the increased dead- and live-loads. Furthermore, the seismic actions are also increased due to the increased mass of the system but also because often the widened bridge has to be designed for larger acceleration levels compared to the initial design.

A pivotal element for the retrofit is the estimation of the conventionally-defined moment capacity of the existing pile groups (moment at which the capacity of the edge piles is reached). The methods used for this assessment are based on analytical [1], empirical [2], or numerical approaches [3]. However, depending on the method used, the conventional moment capacity of a pile group may be strongly under-estimated. Therefore, the existing foundations need to be critically evaluated after the most modern guidelines for the new loading conditions.

The second pivotal element related to the retrofit is the design concept itself. Current elastic design prevents the mobilization of the full moment capacity of the pile group. Capacity design makes sure that the foundation capacity is larger than that of the bridge pier, thus guiding plastic hinging to the superstructure. An alternative concept has been proposed by Anastasopoulos et al. [4], according to which soil failure can be used for seismic protection of structures. The concept of ductility (generally accepted and used in structural design) could be also introduced in the design of piles. Such a plastic design approach would mobilize the full moment capacity by allowing load redistribution to the inner rows after the edge piles have fully mobilized their bearing capacities.

2 Problem definition

2.1 Case study

A typical overpass bridge is examined, inspired from widening of the Aare bridges at Ruppoldingen in Switzerland. The structural and geotechnical details are modified corresponding to a wider range of overpass bridges. The examined system (Fig. 1) is a 3-span bridge with a continuous concrete box-girder deck, supported on two reinforced concrete (RC) cylindrical piers of diameter $d = 1.9\,m$ and height $h = 13\,m$. The reinforcement of the RC piers is computed according to the Swiss norms [5] for columns with large ductility demands. The piers are monolithically connected to the deck, which is supported by 4 elastomeric bearings at each abutment. The deck of the bridge is widened from $8.8\,m$ to $13.6\,m$ in order to serve two additional lanes, resulting in about 50% additional dead loads but also to increased traffic loads. The RC piers are retrofitted accordingly for the new loading conditions (Fig.1). The bridge is founded on an idealized (yet realistic for Switzerland) soft clay layer described by linearly increasing undrained shear strength, S_u, profile:

$$S_u = (15 + 3.5z)\,kPa \tag{1}$$

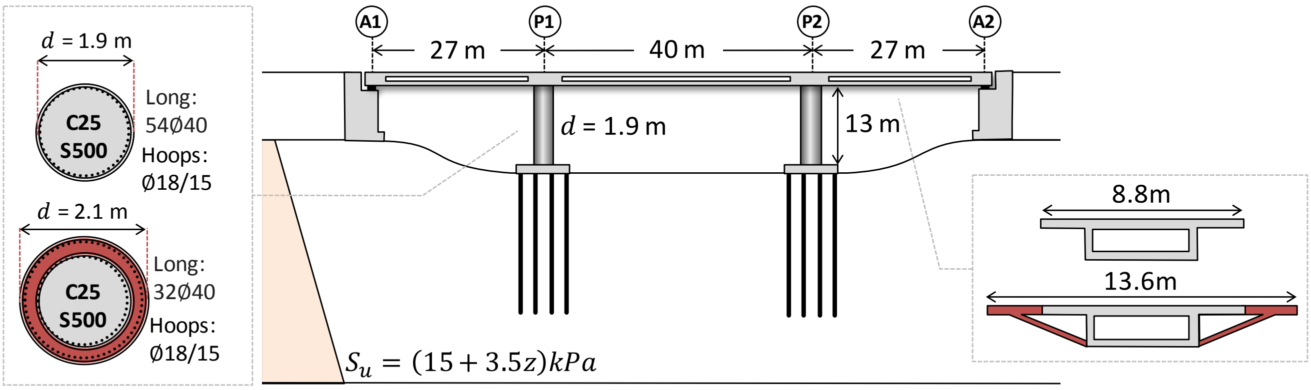

Figure 1: Key attributes of the bridge used prior to and after the deck widening.

2.2 Design considerations

The majority of Swiss bridges were constructed before 1975, and therefore the initial bridge (prior to widening) assumes the design practice of previous decades, not accounting for seismic actions. Furthermore, another difference to the current practice is the use of global safety factors. On the contrary, the evaluation of the initial design and the retrofit needs are determined based on the provisions of the current Swiss codes [6, 7].

The design of the pile group follows the analytical approaches described by Lang et al. [8] (total stress methods) which are still commonly applied in Switzerland, targeting to a global safety factor $FS_{global} = 3$ (common practice of 60s – 70s). The tip resistance of a single pile of diameter D (pile tip area: A), is estimated based on the undrained shear strength of the clay, S_u, the bearing capacity factor, N_c [9] and the depth factor X [10]:

$$Q_S = A \cdot S_u \cdot N_c \cdot X \quad (2)$$

Similarly the shaft resistance of the pile (perimeter: U) assumes an average Su,av for each layer of depth h, and is calculated:

$$Q_M = \Sigma(U \cdot h \cdot 0.6 \cdot S_{u,av}) \quad (3)$$

This is similar to the a-method [12] assuming a-coefficient a=0.6, for bored piles in soft clay. The selected foundation is a 4x4 pilegroup (Fig. 2) (diameter $D = 1m$, total length $L = 21\,m$). The distance between the piles is $s = 2.8 \approx 3D$, therefore the pile group effects are considered negligible [11]. The characteristic axial resistance of a single pile, $V_{s,k}$, is determined adding the shaft and tip resistance and is compared to the characteristic static loads, $V_{e,k}$, resulting in a global safety factor $FS = 3.1$.

The deck widening and the consequent retrofit of the piers increases 40% the dead- and 20% the live-loads. The existing foundation is evaluated according to the current Swiss code using partial safety factors (design values), considering both the increased static loads (Ve,d) and the seismic actions (Me,d). The design moment capacity, Mel,d, is determined assuming an elastic approach (moment at which the edge piles reach their design axial resistance). The acting loads and the pile resistance are collected in Table 1 for both the initial and the widened bridge. The design acting moment is higher than the elastic moment capacity calling for the retrofit of the existing foundation.

Table 1: Synopsis of static and dynamic safety factors for each design consideration

Scenario	Initial pile group – Initial bridge	Initial pile group – Widened bridge	Retrofitted Pile group – Widened Bridge
	Global Safety Factor	Partial Safety Factors	
Static	$V_{s,k} / V_{e,k} = 3$	$V_{s,d} / V_{e,d} = 1.1$	$V_{s,d} / V_{e,d} = 1.6$
Seismic	–	$M_{el,d} / M_{e,d} = 0.8$	$M_{el,d} / M_{e,d} = 1.02$

The common practice of attaching additional pile rows is chosen for the case examined. Two rows, each consisting of three piles, have been added in the transverse direction of seismic loading. The new piles have the same diameter but a smaller total length. This retrofit solution increases both the static axial resistance, $V_{s,d}$, and the conventionally-defined moment capacity of the pile group, $M_{el,d}$, which reaches the required safety level for the critical case of seismic loading ($M_{el,d}$ / M_{Ed} $= 1.02$).

An overview of the retrofitted foundation is illustrated in Fig. 2, presenting a number of technical challenges. Considering the heavy machinery needed for the installation of the new piles, a partial excavation in the vicinity of the existing pile group is unavoidable. A new pile-cap has to be constructed and further connected to the existing one by means of shear dowels. Accounting also for potential space limitations, the retrofit of an existing pile group is a major operation. To that end it is important to investigate the necessity of such operation and quantify its benefits in the seismic performance of the bridge compared to the initial non-retrofitted foundation.

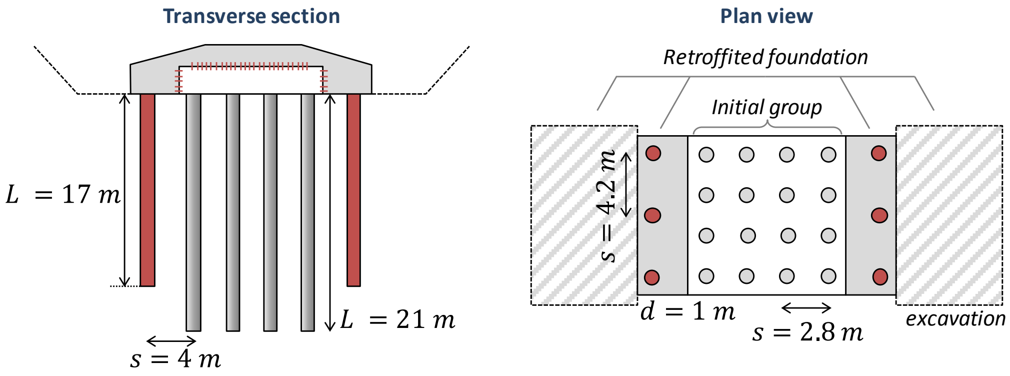

Figure 2: Configuration of the initial 4 x 4 and the retrofitted pile group with two additional pile rows in the transverse direction of the bridge.

3 Analysis methodology

3.1 FE modelling

The seismic performance of the widened bridge assuming: (a) the initial and (b) the retrofitted foundation, is analysed employing the FE method. Figure 3 gives an overview of the FE model developed in ABAQUS, including the structural components and the soil-foundation system. The deck and the piers of the bridge are modelled with elastic and inelastic beam elements, respectively. The inelastic behaviour of the piers is simulated with a nonlinear model, calibrated against the results of RC section analysis using the KSU_RC software [12]. Linear elastic springs and dashpots are used to model the compression and shear stiffness and the damping of the abutment bearings.

The piles are simulated using a hybrid modelling technique [13]. Inelastic beam elements are used to simulate the structural properties of the piles (nonlinear model, calibrated as previously, considering minimum 1% reinforcement). The beam is positioned in the center of the pile and is circumscribed by eight-node hexahedral continuum elements of nearly zero stiffness. The nodes of the beam are connected to the solid-element nodes at the same level with kinematic-constraints (each pile section behaves as a rigid disk). This way, the 3D geometry effects are captured accurately. The top-nodes of the piles are fixed to the pile cap which is modelled with elastic brick elements, assuming the properties of RC ($E = 30\,GPa$). To eliminate boundary effects the distance of lateral boundaries to the edge pile is 10 m ($10\ d$) and to the bottom boundaries is 6 m ($6\ d$) (Fig. 3).

The selected clayey stratum is modeled with eight-node hexahedral continuum elements and contact elements are introduced at the soil–piles-cap interfaces to model possible separation and sliding. Appropriate "free–field" boundaries are used at the lateral edges of the model, while dashpots are installed at model base to simulate the half-space underneath. Nonlinear soil behavior is modeled with a kinematic hardening model, with a Von Mises failure criterion and associated flow rule [14]

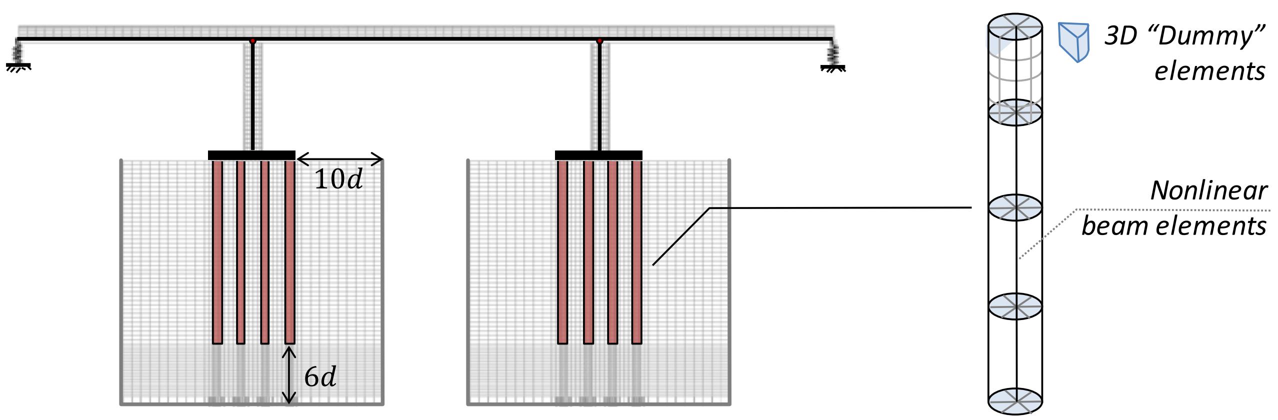

Figure 3: Key attributes of the 3D FE model of the widened bridge including the structural members, the pile foundation and the subsoil

3.2 Moment – rotation response

The non-retrofitted pile group is subjected to a displacement-controlled horizontal pushover analysis. A total horizontal displacement $\delta = 3\,m$ is applied at the centroid of the deck. The pier is modelled as rigid, thus the analysis focuses at the foundation response. The results are presented in Fig. 4 in terms of moment-rotation (M–θ) response.

The grey-shaded area shows the increased design acting moment due to the widening of the bridge. The conventionally defined moment capacity (green point) lies between the design moments prior to and after the widening of the bridge. This point corresponds to the applied rotation at which the edge piles reach their design axial capacity ($M_{el,d} = 29.3\,MNm$). With further rotation, the edge piles mobilise their characteristic ultimate axial resistance and the pile group mobilise its entire ¨elastic¨ moment resistance. ($M_{el,k} = 80.7\,MNm$).

Further increase results in a progressive redistribution of loads from the edge piles to the inner rows. When all piles have fully mobilized their axial resistance in compression and tension respectively the moment resistance ($M_{pl,k} = 120\,MNm$) exceeds substantially the design moment. The structural integrity of the piles is still maintained and the increased resistance is associated only with additional rotations. The final point corresponds to the rotation at which the first pile yields. This is considered as the ultimate resistance that can be mobilised avoiding any structural damage at the foundation level. Preventing seismic structural damage of piles is definitely reasonable, considering the difficulties to identify and remediate such damage below the ground surface. Overall, the non-retrofitted foundation is capable of undertaking the increased loads provided the redistribution of axial loads from the edge piles to the inner rows.

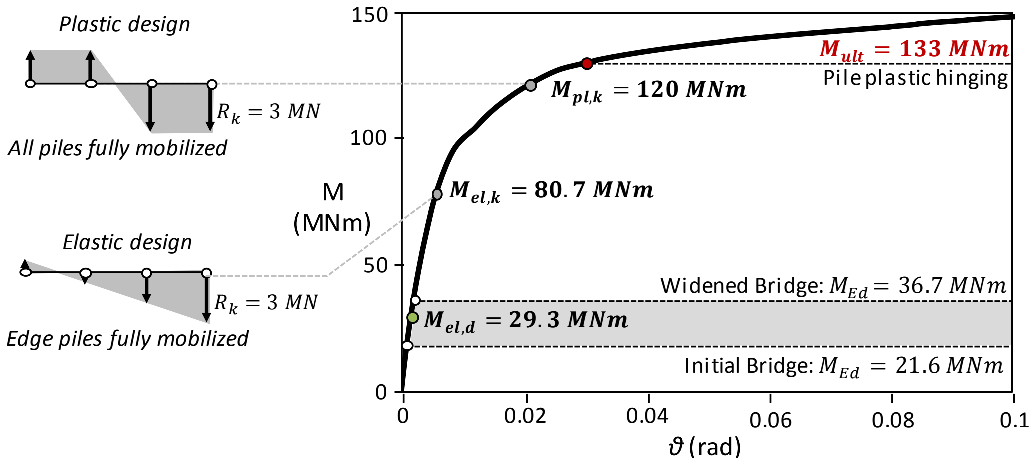

Figure 4: Moment-rotation curve of the non-retrofitted pile group subjected to a horizontal pushover test: Points of interest and comparison to design considerations

4 Dynamic analysis

The previous analysis showed that the retrofit of the initial foundation may be avoided exploiting nonlinear soil–foundation response. However, the increased moment resistance is associated with additional rotations and settlements. To that end, a nonlinear dynamic time history analysis is conducted to examine the benefits of the retrofit in the seismic performance of the entire bridge. The Rinaldi–228 record of the 1994, 6.7 Northridge earthquake was selected in order to compare the two alternatives under very strong shaking, substantially exceeding the design considerations. The results are collected in Fig. 5 and the comparison is conducted in terms of deck drift time histories, δ, pier section moment–curvature response, $M - c$, and time histories of foundation settlements, w.

Interestingly, although the bridge with the non-retrofitted pilegroup experiences almost double total deck drift, δ, it shows a slightly better response in terms of structural deflection (Fig. 5 a, b). The retrofitted pilegroup even under such severe seismic shaking acts almost as fixity experiencing limited nonlinearities. Thus, ‘’plastic hinging’’ is guided to the pier. On the contrary, the non-retrofitted pilegroup experiences highly nonlinear response and over 50% of the total deck drift is due to foundation rotation. The latter acts as a ‘’fuse’’ for the superstructure, thanks to the activated energy dissipation mechanisms during soil yielding.

Consistently to the drift time histories, the bridge with the non-retrofitted pile group experiences lower pier section moments and therefore less structural damage (Fig. 5c). In both cases the piers exceed their yielding moment which may be expected for such a severe motion, substantially larger than the design considerations. This slightly better response of the bridge with the non-retrofitted foundations, in terms of $M - c$ curve, is however associated as expected with larger total settlements (Fig. 5d).

Overall, even under an extreme seismic scenario, the benefits of the pile group retrofit are limited and mainly reflected to the deterioration of the residual settlements and rotations. Considering the technical and financial challenges, the actual need for such a major operation may be questioned.

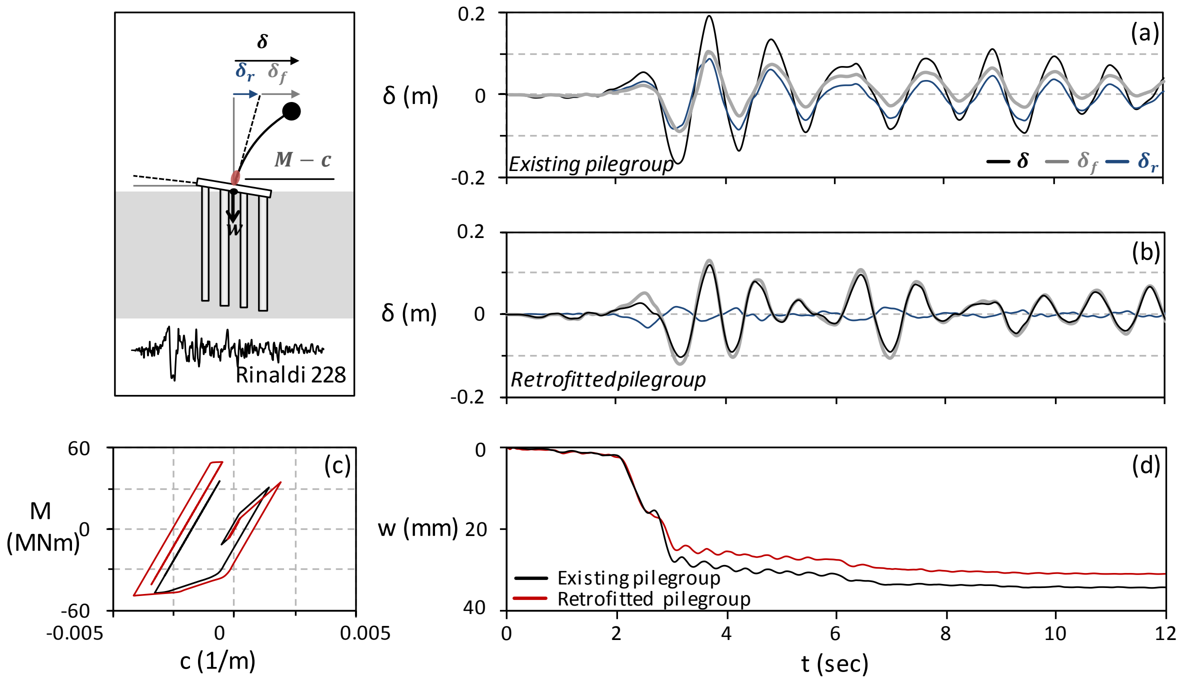

Figure 5: Seismic performance of the two alternatives during Northridge Rinaldi-228 record: (a, b) deck drift; (c) moment curvature response of the pier; and (d) foundation settlement

5 Conclusions

In this paper two different approaches on the retrofit design of pile foundation were comparatively assessed. An illustrative case study of a widened bridge was used to compare the conventional retrofit design (elastic design approach) of pile foundations to a new design scheme exploiting nonlinear soil–foundation response (ductility concept – non-retrofitted pile group).

The non-retrofitted pile group can accommodate the increased design loads of deck widening, provided the mobilization of its full moment capacity (plastic design). Furthermore, under severe shaking, substantially exceeding the design limits, the performance of the bridge with the non-retrofitted pile group was shown equally efficient to the retrofitted one. The energy dissipation mechanisms mobilized act as a “fuse” protecting the superstructure, in the cost of limited additional settlements.

Although these conclusions are the outcome of limited numerical investigations and cannot yet be considered of general validity, exploiting nonlinear foundation response constitutes a promising potential alternative to conventional retrofit design of pile groups which is worth further exploration.

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