ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

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Συνεργασία Επιχειρήσεων - Πανεπιστημίων

Τον περασμένο μήνα η Πρεσβεία του Ηνωμένου Βασιλείου διοργάνωσε στην Αθήνα εκδήλωση για την παρουσίαση από τον Professor Sir Tim Wilson DL της έκθεσης που συνέταξε, υπό την καθοδήγησή του, ομάδα εργασίας σχετικά με την συνεργασία επιχειρήσεων με τα πανεπιστήμια στο Ηνωμένο Βασίλειο. Στον πρόλογο της έκθεσης αναφέρονται:

The economic and social prosperity of the UK depends upon a healthy knowledge-based economy. In our globally competitive economic environment, never before has there been a greater need for a talented, enterprising workforce, for constant innovation in product and service development, for a thriving culture of entrepreneurship, for dynamic leading-edge scientific and technological development and for world-class research that attracts investment. In collaboration with business, and with the support of government, the UK university sector has the capability to fulfil Lord Dearing's vision: to be the source of strength in the UK's knowledge based economy of the twenty first century.

Universities are an integral part of the skills and innovation supply chain to business. However, this supply chain is not a simple linear supplier-purchaser transaction; it is not the acquisition of a single product or service. This supply chain is multi-dimensional, it has to be sustainable and it has to have quality, strength and resilience. These attributes can only be secured through close collaboration, partnership and understanding between business and universities.

Η Εκτελεστική Επιτροπή της ΕΕΕΕΓΜ, ενστερνιζόμενη την σημα-

(συνέχεια στην σελ. 3)

Π Ε Ρ Ι Ε Χ Ο Μ Ε Ν Α

Και του χρόνου ….

(συνέχεια από την πρώτη σελίδα)

σία της συνεργασίας των Πανεπιστημίων με τις Επιχειρήσεις αποφάσισε την έναρξη μιας σειράς δράσεων που θα συμβάλουν στην βελτίωση της πληροφόρησης των επιχειρήσεων για τις δραστηριότητες των πανεπιστημίων, καθώς και των πανεπιστημίων για τις δραστηριότητες των επιχειρήσεων του τομέα των τεχνικών έργων. Μεταξύ των δράσεων αυτών είναι:

i. Θέσπιση ετήσιου βραβείου για την καλύτερη Διπλωματική Εργασία Εφαρμοσμένου Ενδιαφέροντος.

ii. Θέσπιση βραβείου ανά τετραετία για το καλύτερο Ιστορικό Περιστατικό (Case Study) Κατάλληλο για Διδασκαλία.

iii. Θέσπιση ετήσιου βραβείου για την εκπροσώπηση της ΕΕΕΕΓΜ στο European ή στο International Young Geotechnical Engineer Conference (EYGEC ή IYGEC).

iv. Διεξαγωγή διαλέξεων παρουσίασης ιδιαιτέρου ενδιαφέροντος γεωτεχνικών έργων, καθώς και διαλέξεων παρουσίασης ερευνητικών δραστηριοτήτων των τομέων γεωμηχανικής των πολυτεχνείων και πολυτεχνικών σχολών της χώρας.

v. Διαλέξεις ενημέρωσης σπουδαστών – φοιτητών πολυτεχνείων και πολυτεχνικών σχολών στο αντικείμενο της δουλειάς του γεωμηχανικού.

vi. Διοργάνωση «Ημέρας Σταδιοδρομίας», όπου θα δίνεται η ευκαιρία σε τελειοφοίτους σπουδαστές – φοιτητές των πολυτεχνείων και πολυτεχνικών σχολών να συναντήσουν εκπροσώπους γραφείων μελετών και κατασκευαστικών εταιρείων του κλάδου, αλλά και δημοσίων φορέων που ασχολούνται με δραστηριότητα στην γεωμηχανική.

vii. Δημοσίευση στα ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ περιλήψεων προσφάτων Διδακτορικών Διατριβών.

viii. Δημοσίευση στα ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ, στο τεύχος του Ιανουαρίου κάθε χρόνου αρχής γενομένης από τον Ιανουάριο 2014, καταλόγου των Διδακτορικών Διατρικών και των Διπλωματικών και Μεταπτυχιακών Εργασιών των Τομέων Γεωμηχανικής των Πολυτεχνείων και Πολυτεχνικών Σχολών της χώρας, που εκπονήθηκαν την προηγούμενη χρονιά, με παραπομπή στις ιστοσελίδες των Τομέων για τα πλήρη κείμενα των εργασιών.

ix. Διαλέξεις ενημέρωσης στελεχών δημόσιας διοίκησης, ιδιαίτερα της περιφέρειας, σε θέματα γεωμηχανικής (έρευνες, μελέτες, κατασκευές).

x. Ημερίδες – εσπερίδες ενημέρωσης μηχανικών στην εφαρμογή του Ευρωκώδικα 7 και των σχετικών άρθρων του Ευρωκώδικα 8 με αναλυτικά παραδείγματα.

Πιστεύουμε ότι με τις δράσεις αυτές θα βελτιωθή η αλληλεπίδραση πανεπιστημίων και επιχειρήσεων προς κοινό ώφελος αυτών αλλά και της μαστιζόμενης ελληνικής οικονομίας.

ΕΛΛΗΝΙΚΗ ΕΠΙΤΡΟΠΗ ΣΗΡΑΓΓΩΝ και ΥΠΟΓΕΙΩΝ ΕΡΓΩΝ (Ε.Ε.Σ.Υ.Ε.)

Στις 18 Σεπτεμβρίου 012 στη Βουδαπέστη και στα πλαίσια των εκδηλώσεων του 1st East European Tunnelling Conference 2012, η Ελληνική Επιτροπή Σηράγγων και Υπογείων Έργων (ΕΕΣΥΕ) υπέβαλε υποψηφιότητα για τη διοργάνωση του 2nd East European Tunnelling Conference 2014 (EETC 2014). Το σώμα των αντιπροσώπων των Εθνικών Επιτροπών της International Tunnelling and Underground Space Association (ITA) των χωρών της Ανατολικής Ευρώπης, παρουσία του πρώτου αντιπροέδρου της ITA κ. Markus Thewes και του εκτελεστικού Διευθυντή κ. Olivie Vion ανέθεσε ομόφωνα τη διοργάνωση στην ΕΕΣΥΕ και στην Αθήνα. Οι χώρες που συμμετέχουν στην διοργάνωση και στις οποίες κυρίως, αλλά όχι αποκλειστικά, απευθύνεται είναι: Ελλάδα, Κύπρος, Τουρκία, Αλβανία, FYROM, Βουλγαρία, Ρουμανία, Σερβία, Μαυροβούνιο, Βοσνία, Κροατία, Σλοβενία, Ουγγαρία, Σλοβακία, Τσεχία, Πολωνία, Λευκορωσία, Ουκρανία, Ρωσία, ενώ επιδιώκεται η συμμετοχή και των υπολοίπων Βαλτικών χωρών, ίσως δε και του Ισραήλ.

Οι ημερομινίες που καταρχήν συμφωνήθηκαν για τη διοργάνωση είναι 30-9-2014 για τις πρόδρομες εκδηλώσεις, 1 και 2-10-2014 για τις εργασίες του συνεδρίου και 3-10-2014 τεχνική επίσκεψη. Το κεντρικό θέμα του συνεδρίου και η ειδικότερη προσέγγισή του, όπως προτάθηκε από την ΕΕΣΥΕ είναι:

"Tunnelling in a Challenging Environment. Conditions for the execution of underground projects are becoming Increasingly demanding as new challenges are emerging in every aspect and sector of this multidisciplinary and multifarious business. Geological, geotechnical, structural, operational challenges, that are well known and unavoidable in the tunnelling industry, are now accompanied by greater ones, namely those of a stretched business and financial environment, which require the deployment of even more intelligent and effective tools and solutions".

Σκοπός της ΕΕΣΥΕ με τη διοργάνωση του συνεδρίου είναι να υπάρξει μια περαιτέρω ευκαιρία για προβολή της γνώσης και εμπειρίας που παράχθηκε και σωρεύτηκε στη χώρα, να αναπτύξει τις σχέσεις συνεργασίας με τις άλλες Εθνικές Επιτροπές των χωρών αυτής της γεωγραφικής ενότητας και να δοθεί η ευκαιρία δημιουργίας ενός βήματος επιστημονικής αλλά και επιχειρηματικής καταγραφής και συνεργασίας στην Ανατολική Ευρώπη.

Σύντομα θα γνωστοποιήσουμε τις λεπτομέρειες που αφορούν τα ειδικότερα θέματα της διοργάνωσης.

Γιάννης Μπακογιάννης, Πρόεδρος ΕΕΣΥΕ

Η ΕΕΕΕΓΜ εύχεται στην ΕΕΣΥΕ κάθε επιτυχία στην διοργάνωση του συνεδρίου, την οποία θα στηρίξη πλήρως.

ΑΡΘΡΑ

Τα παρακάτω δύο άρθρα αποτελούν την συμμετοχή των εκπροσώπων της ΕΕΕΕΓΜ στο 22ο Ευρωπαϊκό Συνέδριο Νέων Γεωτεχνικών Μηχανικών (22EYGEC), που διεξήχθη από τις 26 έως τις 29 Αυγούστου στο Gothenburg της Σουηδίας.

Seismic Performance of Caisson Supported Structures

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ABSTRACT

A numerical study of a 3D nonlinear soil-foundation-structure system is performed under the prism of a new "capacity design" principle, in which soil "failure" mechanisms are deliberately mobilized to protect the superstructure. For caisson supported sys-tems this involves material and geometric nonlinearities such as soil inelasticity, separation (gapping) between the caisson and the soil, slippage at the soil–caisson interface, base uplifting, and perhaps even loss of soil strength (e.g. due to development of excess pore water pressures). To investigate the effectiveness of such an approach, simple structures of varying deck mass, simulating heavily or lightly loaded structures founded through similar rigid cubic caissons on a 2-layer soil stratum are used as ex-amples. Two alternatives are compared: one complying with conventional capacity design, with over-designed foundation so that the soil is marginally plastified (the plastic hinge on the column); the second design follows the new approach in which the foundation is under-designed, thereby "inviting" the plastic "hinge" below ground surface. The comparison is performed through Incremental Dynamic Analysis (IDA), where the alternatives are subjected to several ground motion records, each scaled to mul-tiple levels of intensity. IDA curves are produced for a variety of intensity and damage parameters describing both the maximum and the residual response of the system. The results emphasize the beneficial role of foundation nonlinearities on reducing the seismic demands of the superstructure.

Keywords: Caisson foundations, Dynamic soil-structure interaction, Soil and interface nonlinearities, Incremental Dynamic Analysis (IDA), Engineering Demand Parameters (EDP)

1 INTRODUCTION

The seismic design of structures is based on capacity design approaches, where the structural response is presumed ductile. The concept of ductility design for foundation elements is still new in earthquake engineering practice. In essence, the capacity design methodology explicitly considers the problem of determining the failure mechanism of structural members and ensures that regions of inelastic deformation are carefully detailed to provide adequate structural ductility, without transforming the structure into a mechanism. Elastic response of the foundation is usually ensured by increasing the structural strength of the foundation so that plastic hinging occurs in the superstructure instead of the foundation. The possibility of *soil yielding*, denoting the non-linear inelastic soil response without necessarily resulting in bearing-capacity failure, is forbidden in existing regulations, codes and specifications. In simple geotechnical terms, the designer must ensure that the foundation system will not even reach a number of "thresholds" that would conventionally imply failure. For the case of deep caisson foundations, this indicates that passive and shear failure along the sides and the base is prohibited, introducing appropriate "overstrength" factors plus factors of safety larger than 1 against each "failure" mode, as in static design. Although such a restriction may, at first, appear reasonable (the inspection and rehabilitation of foundation damage after a strong earthquake is not a trivial task), it may lead to *unconservative* oversimplifications, especially in strong earthquake loading where geometric non-linearities and soil inelasticity are usually unavoidable (separation (gapping) between the caisson shaft and the soil, slippage at the soil–caisson shaft interface, base uplifting). Therefore neglecting such phenomena prohibits the exploitation of strongly non-linear energy dissipating mechanisms in defense of the superstructure in case of occurrence of ground motions larger than design. In fact, recent research on surface foundations suggests that soil compliance and subsequent soil–foundation plastic yielding may be beneficial, and should be considered in the analysis and perhaps allowed in the design [e.g. 2, 3, 4].

In this framework, the present study aims to investigate the effectiveness of the new capacity design (compared to conventional capacity de-sign) of caisson supported structures, through 3-D *Incremental Dynamic Analysis* (7). IDA is a powerful tool to assess the global and local ca-pacity of structures, by subjecting the soil-structural model to several ground motion records, each scaled to multiple levels of intensity, providing thus useful inputs for applications of performance-based evaluation. The comparison is performed between structures founded on similar caissons, while varying the mass and the height of the superstructure. In total, the response of four (4) idealized configuretions is studied: (a) two corresponding to a conventionally and an un-conventionally designed foundation resulting from a "heavy" superstructure (safety factor for static loading $FS_V = 2.5$), and (b) two corresponding to a conventionally and an *un*-conventionally designed caisson resulting from a "light" superstructure (safety factor for static loading $FS_V = 5$).

2 PROBLEM DEFINITION AND ANALYSIS METHODOLOGY

2.1 Problem definition

The studied problem is portrayed in Figure 1: A mass-and-column structure is founded through a rigid cubic caisson of side h = 10 m in a 20 m thick 2-layer cohesive soil stratum.

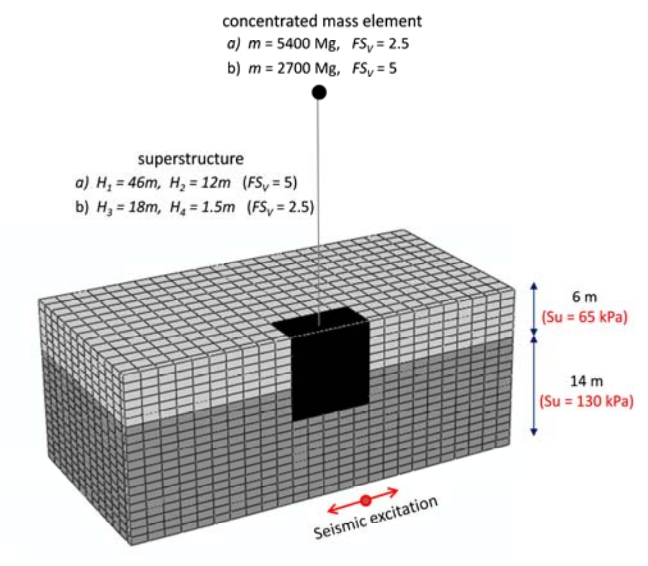

Figure 1. Overview of the 3D Finite Element model used in the analysis.

The soil is considered to be undrained with S_u = 65 kPa at the upper 6 m and S_u = 130 kPa at the lower 14 m. The mass-and-column superstructures are modeled as SDOF oscillators. The mass of the deck, m, is given parametrically the values of 5400 and 2700 Mg, corresponding to a static fac-tor of safety FS_V = 2.5 ("heavy" superstructure) and FS_V = 5 ("light" superstructure) respectively. The alternative design approaches, conventional and *un*-conventional for each case of FS_V, are represented by two different column heights. In total, a set of four structural configurations are analysed.

The height of the superstructure is calculated from static pushover analysis, so that it deliberately matches a "target" critical (yielding) acceleration ($a_{c,f}$) associated with bearing capacity of the foundation. Structural yielding is then either prevented (un-conventional design) or pursued (conven-tional design) by designing the super-structure for a critical acceleration ratio ($a_{c,f}$ / $a_{c,str}$) < 1 or ($a_{c,f}$ / $a_{c,str}$) >1 respec-tively, where $a_{c,str}$ is the critical−at yield−spectral ac-celeration of the superstructure. Since the superstructure is modeled as SDOF oscillator, the horizontal force at the top of the caisson is related to the overturning moment at fail-ure according to: $M = m \cdot a_{c,str} \cdot H$.

To compare the seismic performance of the two alternative design schemes on a 'fair' basis, the critical acceleration ratio, r_{max}, defined as:

$$r_{max} = \frac{\max\{a_{c,f},\ a_{c,str}\}}{\min\{a_{c,f},\ a_{c,str}\}} \tag{1}$$

is kept constant for all cases considered, and deliberately set equal to *2.67*. Assuming $a_{c,f}$ = 0.3 g for the *un*-conventional design, condition (1) leads to $a_{c,\ str}$ = 0.8 g. Likewise, assuming $a_{c,f}$ = 0.8 g for the conventional design, condition (1) leads to $a_{c,\ str}$ = 0.3 g. The four model configu-rations are summarized in Table 1.

2.2 Numerical and constitutive modeling

The problem is analysed with the use of the finite element code ABAQUS. Both caisson and soil are modeled with 3D 8-noded solid elements, assuming elastic behavior for the former and nonlinear for the latter. The superstructure is modeled with 3D nonlinear Timoshenko beam elements. The caisson is connected to the soil with special contact surfaces, allowing for realistic simulation of the possible detachment and sliding at the soil-caisson interfaces. The soil stratum reaches 10 m deeper than the caisson base, thus having a negligible influence on the response. To en-sure uniform stress distribution at the head of the caisson, the nodes of the associated elements are tied through ap-propriate kinematic constrains. For the total stress analysis under undrained conditions, soil behavior is modeled through a nonlinear constitutive model (5) which is a slight modification of a model incorporated incorporated in ABA-QUS. It uses the Von Mises failure criterion with yield stress σ_y related to the undrained shear strength S_u as

$$\sigma_y = \sqrt{3}\, S_u \tag{2}$$

along with a nonlinear kinematic and isotropic hardening law, and an associative plastic flow rule. The model pa-rameters are calibrated to fit published *G−γ* curves of the literature. Rayleigh damping, representing material damp-ing, is taken equal to 5% between the eigenfrequency of the soil deposit and the dominant frequency of the earth-quake ground motion. Appropriate kinematic constraints are imposed to the lateral edges of the model, allowing it to move as the free−field (6).

The nonlinear behavior of the superstructures is described by a simple hyperbolic backbone curve in moment (*M*)−curvature (*k*) space, defined as:

$$k = \frac{M_y}{EI} \cdot \left(\frac{M}{M_y - M} \right) \tag{3}$$

where *EI* the initial structural bending stiffness, calculated from the geometric characteristics of each cross section and the elastic properties of the reinforced concrete, and M_y the ultimate strength associated with the critical acceleration ($a_{c,\ str}$).

Table 1. Summary of the model configurations used in the analyses

	m (Mg)	H (m)	ac : critical spectral acceleration	Design paradigm
model 1	2700	46	0.3g (foundation) 0.8g (superstructure)	**under-** designed foundation
model 2	2700	12	0.8g (foundation) 0.3g (superstructure)	over-designed foundation
model 3	5400	18	0.3g (foundation) 0.8 g (superstructure)	**under-** designed foundation
model 4	5400	1.5	0.8g (foundation) 0.3 g (superstructure)	over-designed foundation

3 INCREMENTAL DYNAMIC ANALYSIS

Incremental Dynamic Analysis (IDA) is a powerful analysis method that offers thorough seismic demand and capacity prediction capabil-ity [7]. It involves a series of nonlinear dynamic analyses time-history analyses under suitable scal-ing, aiming at covering the entire range of re-sponse, from elasticity to collapse, selecting proper Engineering Demand Parameters (EDPs) to characterize the structural response and an In-tensity Measure (IM) to represent the seismic in-tensity. The output of an IDA is an IDA curve, i.e. a plot of a selected IM versus a selected EDP. Similarly, an IDA curve set is a collection of IDA curves of the same struc-tural model under different records that have been param-eterized on the same IM.

3.1 Intensity measure and Earthquake Demand Parameters

Different options are available for the IM to be used in the IDA curves. In this paper, however, a single IM is used, and in particular the PGA calculated at the free-field (top of soil profile). Though the PGA is not the most representative IM, it is preferred among others due to the inherent difficulty in determining a priori the IM at the surface for a given input acceleration time history. Without any doubt, the PGA is the most suitable IM to be approximately estimated through 1-D deconvolution analysis.

Selecting an EDP is application-specific. In this paper two EDPs are considered, which are known to relate well to structural performance and global dynamic instability:

- The maximum ductility demand of the soil–caisson–structure system: max{μ_δ}, defined as the ratio of the maximum dis-placement of the system u_{max}, imposed by an earthquake, to the yield displacement u_y, which is a soil–caisson-structure system property:

$$\mu_\delta = \frac{u_{max}^{mass} - \left[u_{max}^{structure\ base} + \theta \cdot H\right]}{u_y} \tag{4}$$

  where *θ* is the caisson rotation and *H* the structure height. The yield displacement u_y is assessed through static push-over analyses of the alternatives, according to the N2 method of Eurocode 8 [1].

- The maximum caisson rotation, θ_{max}.

3.2 Record suite

An ensemble of 10 records has been chosen as base excitation. The selected records, presented in Figure 2, cover a wide range of seismic motions, ranging from medium intensity (e.g. Kalamata, Aegion) to relatively stronger (e.g. Lefkada-2003, Imperial Valley), and to very strong accelerograms characterized by forward-rupture directivity effects, or large number of significant cycles, or fling-step effects (e.g. Takatori, JMA, TCU).

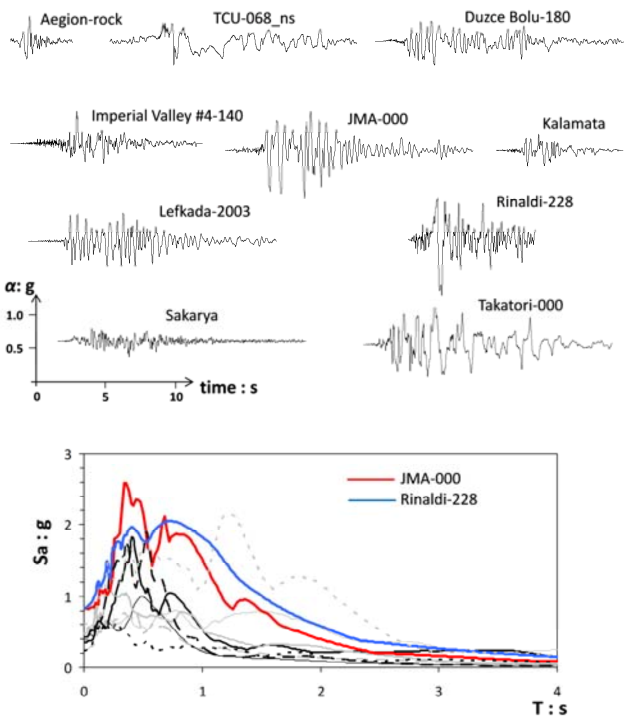

Figure 2. Real earthquake records used for analysis of the design alternatives, along with their elastic spectra.

In this work, the seismic records are appropriately scaled so that 5 ascending target PGA values of free-field motion, namely 0.1 g, 0.2 g, 0.3 g, 0.4 g and 0.6 g, are calculated at the top of the soil profile through 1-D deconvolution analysis. Obviously, since IDA involves nonlinear wave propagation, the actual computed PGA from each scaled record will differ from the targeted one. The reason for examining PGA values smaller than 0.6 g lies in the shear strength of the soil. Stronger signals would produce intense soil yielding which, in turn, would either significantly attenuate the transmitted seismic waves or even leading to bearing capacity failure.

4 ANALYSIS RESULTS AND DISCUSSION

The comparison of the performance of the de-sign alternatives through the IDA curves gener-ated from the analysis results is presented in Figures 3 − 5 for the two most representative cases: *model 1*, lightly loaded *under*-designed founda-tion and *model 4*, heavily loaded over-designed foundation. The IDA curves for the systems' maximum displacement ductility demand (μ), as-sociated with structural distress, are portrayed in Figure 3. Undoubtedly, the most advantageous design alternative concerning structural demand is the *under*-designed lightly loaded *model 1*, exhibiting ductility demands $\mu \leq 0.45$ for most of the seismic motions, while the heavily loaded over-designed *model 4* exhibits $\mu \geq 3.0$: a clear evidence of beneficial effect from mobilizing substantial geometric nonlinearities.

Figure 4 presents the IDA curves for maxi-mum caisson rotation, *θmax*. The performance of the two alternative design schemes does not seem to deviate from any rational intuitive ex-pectation: the *under*-designed system demonstrates substantially larger rotations than the over-designed counterpart, as a result of the in-tense caisson-soil interface separation and gap-ping. Notice the tremendous demand imposed by the large velocity pulse (2.6 m/s) of huge dura-tion (6.3 s) of the TCU-068 record on the *under*-designed alternative, causing global instability and system failure of the lightly loaded model 1 at PGA = 7 m/s2. Nevertheless, it is remarkable that with the exception of the performance under the TCU record at high PGA levels, these alter-native can avoid collapse sustaining rather toler-able rotations and displacements. Furthermore, it should be stated at this point that in the conven-tional design the developed drift is mainly due to flexural distortion, leading to a subsequent in-crease in structural distress, whereas in the *un-conventional* design the drift is mainly due to foundation rotation, causing less seismic loading to the superstructure.

To elaborate on the results from IDA, Figure 5 illustrates the comparison in terms of contours of plastic shear strain magnitude in the soil at the end of the shaking, for the case of JMA-000, scaled at free-field PGA = 0.4 g. Observe the extended soil plastification (material nonlinearities) dominating the response of the convention-ally designed *model 4*. In stark contrast, the un-conventionally designed *model 1* suffers rather extended "plastic hinging" in the form of mobili-zation of passive–type soil failure in front and back of the caisson accompanied by gap formation and sliding in the sides and base (geometric nonlinearities).

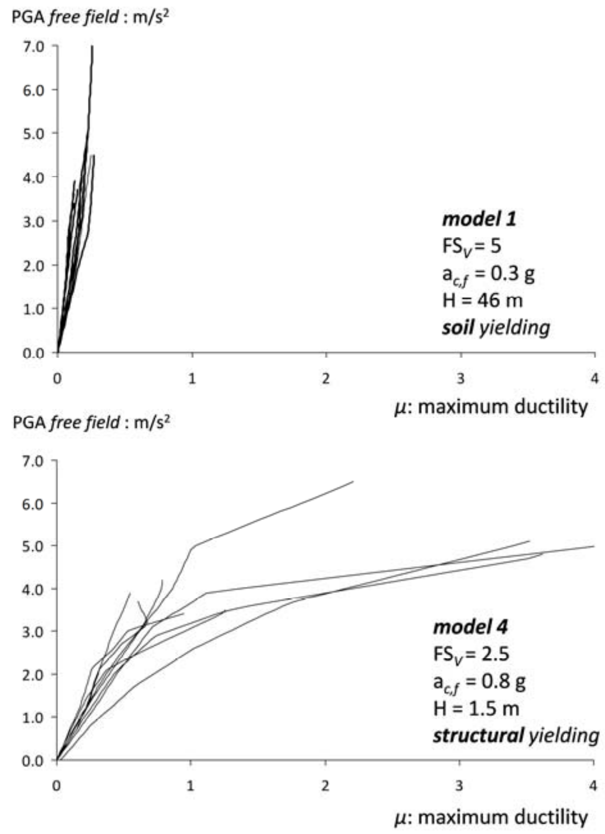

Figure 3. IDA curves of EDP: μ (maximum ductility) for *model 1* and *model 4*.

The stress−strain response of the soil for the two representative cases (*model 1*, lightly loaded *under*-designed foundation and *model 4*, heavily loaded over-designed foundation) is illustrated in Figure 6, in terms of shear stress−strain loops calculated at the caisson−soil interface near the surface.

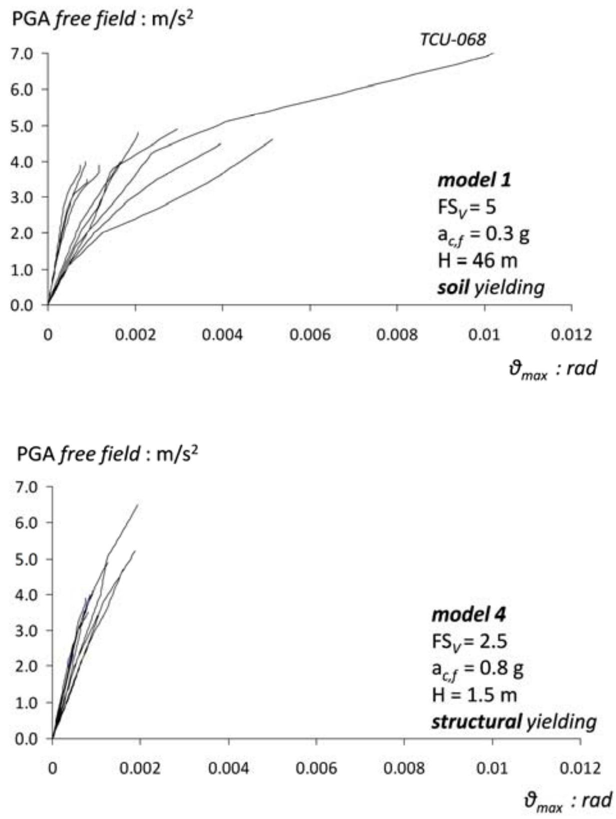

Figure 4. IDA curves of EDP: θ_{max} (maximum caisson rotation) for *model 1* and *model 4*.

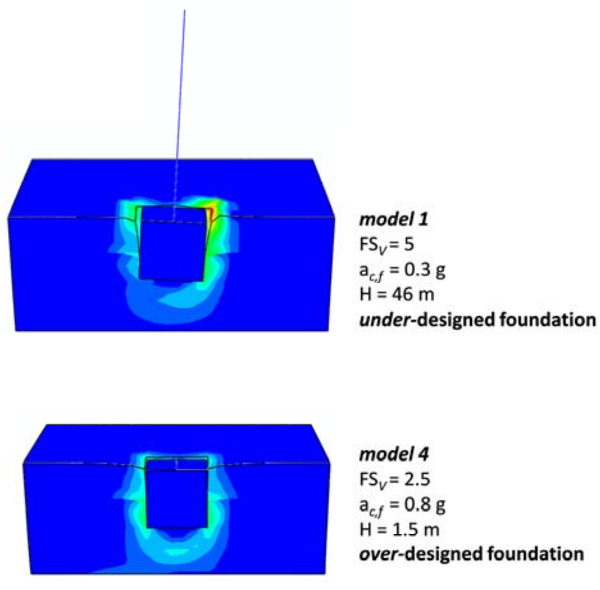

Figure 5. Contours of plastic shear strain magnitude in the soil (PEMAG) at the end of shaking for *model 1* and *model 4*. Record: JMA-000, scaled at free-field PGA = 0.4 g (deformation scale factor = 20).

The larger shear strains computed in *model 1* reflect the mobilization of extensive geometric nonlinearities (gapping between the caisson and the surrounding soil) as compared to the strong material nonlinearities developed in the soil by the heavily loaded *model 4*. Observe that the ultimate shear strength does not exceed the undrained shear strength (S_u = 65 kPa) in both cases.

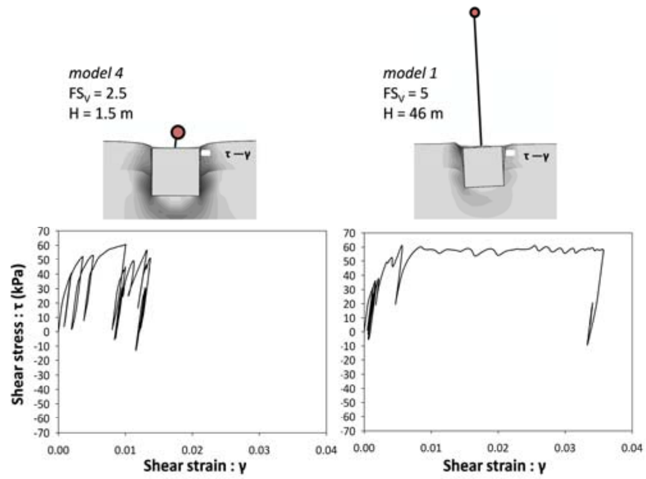

Figure 6. Shear stress-strain loops calculated at the caisson-soil interface near the surface, for the heavily loaded *model 4* and the lightly loaded *model 1*. Record: JMA-000, scaled at free-field PGA = 0.4 g.

5 CONCLUSIONS

The present study is the first attempt to explore the nonlinear soil-foundation-structure interaction (SFSI) effects during the earthquake loading of caisson foundations. Within this framework, the efficacy of the capacity design "philosophies", the conventional (without allowing for nonlinear SFSI effects) and the *unconventional* (allowing for SFSI effects), in reducing the seismic structural demand of the supported structures was compared. SDOF structures of varying deck mass, simulating heavily or lightly loaded structures founded through similar rigid cubic caissons on a 2-layer soil stratum are used as examples. The investigation is performed considering soil and structural inelasticity through 3D finite element incremental dynamic analysis (IDA).

From the numerical results, it was observed that the response of "heavy" structures is deter-mined by excessive material (soil) inelasticity, whereas intense caisson−soil interface separation and gapping prevails in the response of "light" structures. Furthermore, the results highlight the effectiveness of interface nonlinearities in dissi-pating the seismic energy and the favorable per-formance of the *under*-designed founda-tion−structure systems with high static safety factor compared to the conventionally designed heavy structures in both static and dynamic terms, providing a low-cost solution with high seismic isolation potential.

ACKNOWLEDGEMENT

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Investigation of long-term creep deformations on soil strength

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ABSTRACT

Clays under high shear stress levels can exhibit significant loss of long-term shear strength which can even cause delayed creep induced failure. While volumetric creep (often called secondary compression) tends to increase the shear strength, deviatoric creep has deleterious effects on shear strength which often prevail in the overall soil behaviour. The paper presents an incremental elastoplastic-viscoplastic constitutive model based on the overstress theory for natural clays possessing structure. The model combines structure/strength degradation due to plastic shear strains and strength envelope degradation due to creep shear strains. The elastoplastic-viscoplastic constitutive model incorporates a Structure Strength Envelope along with an Intrinsic Compressibility Framework, to account for structural degradation associated with plastic strain evolution. On the other hand, strength envelope degradation is realized through the evolution of the inclination of the critical state line in strength space to a residual state. Model predictions tested in triaxial shearing mode reveals that the model can predict tertiary creep behaviour, i.e., long-term creep-induced failure under high shear stress levels. The model is implemented in the commercial F.E. Code ABAQUS and used in the bearing capacity of surface foundations.

Keywords: constitutive relations, elastoplasticity-viscoplasticity, tertiary creep, soil structure, strength degradation, clays, finite elements

1 INTRODUCTION

The mechanical behaviour of clayey soils is inhibited by the rate of deformation either induced or imposed [1]. In landslides, pile penetration and offshore foundation wave loading the rate governs the overall mechanical behaviour associated with strength and deformation characteristics. Constitutive relations have been widely employed to represent the viscoplastic characteristics associated with creep. The elastoviscoplastic constitutive models are based on (1) the overstress theory [2, 3], (2) the non-stationary flow surface theory [4, 5] and (3) other general time dependent formulations that elude the purpose of this paper [6].

The focal point of the present paper is the development and application of a generalized time-dependent constitutive model for cohesive soils able to predict the rheological transformations undertaken during tertiary creep. The model is founded on the concept of bounding surface plasticity, in the generalized stress space, and the critical state soil mechanics principles implemented within the framework of a generalized creep theory. The general overstress theory employed is not limited to boundary and loading conditions, inherently associated with rheological models, but can describe all possible stress paths and boundary conditions.

2 CREEP CHARACTERISTICS

Creep is a term employed to account for the time evolving deformations under constant loading conditions i.e. in oedometer or triaxial tests. The modes of creep however, activated in the two experiments are entirely different. In the oedometer tests the volumetric deformation measured is the keystone leading to a decrease in void ratio and increase of soil strength (**Fig1**). On the other hand, undrained triaxial specimens imposed to relatively high stress levels tend to fail in tertiary creep portrayed in **Fig2**. It is evident that in the triaxial tests the deviatoric term of creep reveals its deleterious effect on the overall mechanical behaviour.

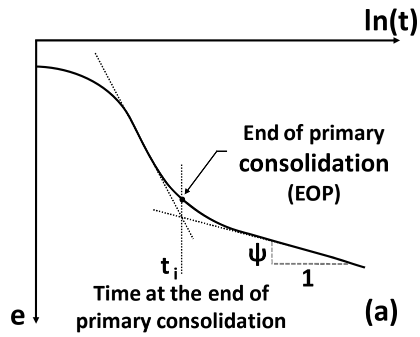

Figure 1. Definition of the secondary compression coefficient ψ for an oedometer test.

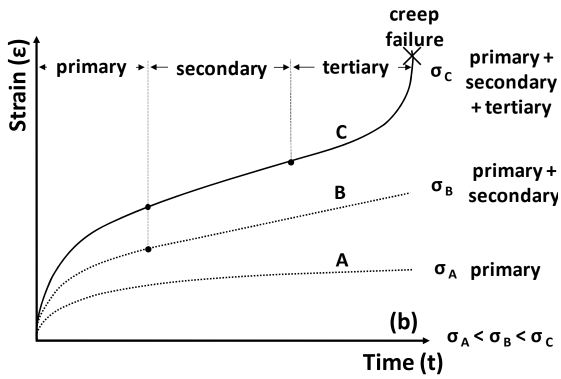

Figure 2. Definition of creep stages for triaxial creep tests at different stress levels.

The general creep theory based on overstress needs to account for instantaneous alterations of the volumetric and deviatoric creep components of strain. Thus, the overstress formulation employed herein has undertaken modification to allow for simultaneous evolution of both components.

The standard logarithm creep rate was selected for the prediction of the volumetric creep component while the Singh & Mitchell expression [7] was employed to express the evolution of the deviatoric creep strain rate.

The elastoplasticity-viscoplasticity theory however, requires the incremental form of the rate formulated in terms of stress and strain rather than stress and time. Since, all mechanical parameters already defined in conventional elastoplasticity are described fully by the state of stress and strain characteristics it stands to reasoning that the time-dependent formulations should be defined in similar trend. The volumetric can be expressed as follows:

$$\dot{\varepsilon}^t = \frac{\psi}{(1+e)t_0}\exp\left[-\frac{(1+e)}{\psi}\varepsilon^t\right] \quad \Big| \quad t \geq t_0 \tag{1}$$

$$\dot{\varepsilon}_q^t = \frac{B}{\left[1+\frac{(1-m)}{B\cdot t_0}\varepsilon_q^t\right]^{\frac{m}{1-m}}} \quad \left| \begin{array}{l} m \neq 1 \\ t \geq t_0 \end{array} \right. \tag{2}$$

The secondary compression coefficient ψ, void ratio e and reference time t_0 are employed in the expression above. In the definition of the deviatoric creep strain rate above the shear stress level $\overline{D} = \frac{q}{q_F}$ is employed in the formation of another quantity $B = 2A\cdot\sinh\left(\overline{\alpha}\overline{D}\right)$. Where $A, \overline{\alpha}, \overline{D}, m$ are the Singh-Mitchell coefficients.

Although the volumetric portion of creep is evident even in the elastic region, the deviatoric component takes place only at the plastic state. Thus, the viscous nucleus in the case of volumetric creep coincides with the isotropic axis, since the envelopes employed are oriented along the octahedral axis (**Fig3**). Conversely, the viscous nucleus associated with the deviatoric component of creep is identical to the structure strength envelope (*SSE*).

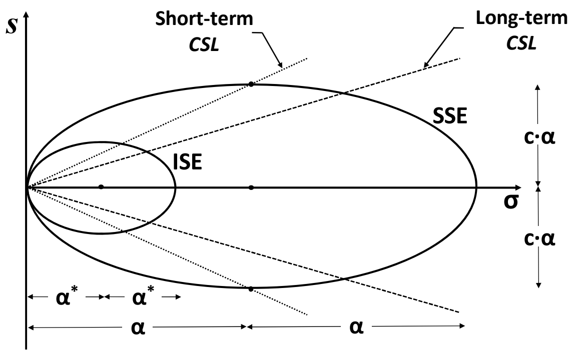

Figure 3. Characteristic surfaces of the proposed model.

In order for expressions (1) and (2) to be employed in the elastoplasticity-viscoplasticity the creep strain rate components need to be redefined in the generalized stress space.

3 THE PROPOSED MODEL

In this section, the author shall attempt to elaborate on the bounding two-surface elastoplasticviscoelastoplastic formulation which incorporates the Critical State concepts into the Structured Soils behavioural framework (**Fig3**). Assuming that the infinitesimal strain increments $d\boldsymbol{\varepsilon}$ can be decomposed in an elastic $d\boldsymbol{\varepsilon}^e$ and an inelastic portion $d\boldsymbol{\varepsilon}^i$, consisted of the plastic $d\boldsymbol{\varepsilon}^p$ and rheological components:

$$d\boldsymbol{\varepsilon} = d\boldsymbol{\varepsilon}^e + d\boldsymbol{\varepsilon}^i = d\boldsymbol{\varepsilon}^e + d\boldsymbol{\varepsilon}^p + \dot{\boldsymbol{\varepsilon}}^i \cdot dt \qquad (3)$$

3.1 Characteristic Surfaces

The proposed model adopts the Structure Strength Envelope (SSE) and one reference surface corresponding to the Intrinsic Strength Envelope (ISE) [8, 9]. The ISE corresponds to the intrinsic soil properties denoted with a star symbol:

$$F^* = \frac{1}{c^2}\boldsymbol{s}^* : \boldsymbol{s}^* + \left(\sigma^* - \alpha^*\right)^2 - \left(\alpha^*\right)^2 \qquad (4)$$

The size of the *SSE* denoted *α* is characteristic of the magnitude of the soil structure and is described by an isotropic hardening rule. The *SSE*'s mathematical formulation is described below:

$$F = \frac{1}{c^2}\boldsymbol{s} : \boldsymbol{s} + \left(\sigma - \alpha\right)^2 - \alpha^2 \qquad (5)$$

where the stress tensor has been decomposed in the isotropic component denoted $\sigma \equiv p$ and the stress tensor deviator $\boldsymbol{s}$. In this version of the model the Plastic Yield Envelope is assumed to coincide with the *SSE*.

In the general case an incrementally linear non-associative flow rule is employed for the computation of the plastic strains [8, 9].

3.2 Isotropic Hardening

An isotropic hardening rule has been employed in the constitutive mathematical formulation as to control the size of the *SSE*:

$$d\boldsymbol{\varepsilon}^p = \left(d\Lambda\right)\cdot \boldsymbol{P} \ \ \& \ \ \left(d\Lambda\right) = \frac{1}{H}\cdot\left(\boldsymbol{Q} : d\boldsymbol{\sigma}\right) \qquad (6)$$

Thus, the size of the *SSE* is inextricably associated to the intrinsic characteristic surface *ISE*. It is evident that in the expression above there is no component for the volumetric component due to structure/strength degradation. The measured value of secondary compression coefficient comprises the net effect of the structure/strength degradation, due to volumetric plastic strain, and volumetric creep. In other words, the effect of volumetric plastic strain on the degradation of the *SSE* has already been incorporated in the definition of the volumetric creep component.

The parameter A_q^p employed is associated to the deviatoric creep strain modulus as follows:

$$A_q^p = \zeta_q^p \cdot \exp\left(-\eta_q^p \cdot \varepsilon_q^p\right) + \theta_q^p \qquad (7)$$

The constants $\zeta_q^p, \eta_q^p, \theta_q^p$ comprise the parameters associated to the destructuring process [8, 9]. Aiming to capture the rheological transformations undertaken in the tertiary creep process we initially attempted to capture the delayed failure through structure/strength degradation. However, keeping in mind that the deviatoric component of creep accumulates only once the plastic state has been achieved and the volumetric component continuously increases the size of the *SSE* it stands to reasoning that the state of stress returns to the elastic region with evolving time.

In order to resolve the aforementioned issue and ultimately capture the failure in tertiary creep we employed the inclination of the critical state line (*CSL*) is the stress space denoted *c* in expression (5) as a hardening variable. The inclination of the *CSL* is representative of rapid rates of induced strain. On the other hand, ageing is a rheological phenomenon evolving through time. In this end, the need to employ a limiting value of the *CSL* for quasi-static loading c_{cs}. Incrementally the rate of inclination of the *CSL* transitioning from rapid to long-term loading conditions can be defined as:

$$\dot{c} = -\left(c - c_{cs}\right) a \cdot \dot{\varepsilon}_q^t \qquad (8)$$

Instead of the creep strain to produce plastic stress we here state that considerable plastic deformation needs to take place for the soil to fail in tertiary creep. This can be justified by the fact that deviatoric creep strain can only be accumulated once the plastic state has been achieved. Parametear controls the tempo at which failure in tertiary creep is attained. Assuming that creep experimental data leading to tertiary failure are available parametear can be calibrated via trial and error analyses.

4 FINITE ELEMENT RESULTS

In this section, results will be presented on triaxial drained specimens along with a bearing capacity problem of a 2m wide footing. Since creep is a time dependent phenomenon revealing its deleterious effects on shear strength after the consolidation equilibrium has been attained, it stands to reasoning that the results to be presented here below should assume drained rather than undrained boundary conditions.

4.1 Triaxial Test Results

In **Fig4** the effect of structure is evident on shear strength.

Regardless that the overall behaviour might portray significant alterations from the elasoplastic response predicted by the Modified Cam-Clay (MCC) the specimen tends to fail at the same level of shear stress.

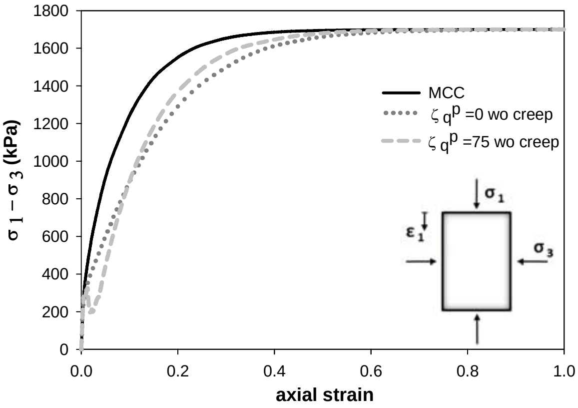

Figure 4. Effect of structure on shear strength at drained triaxial specimens without creep.

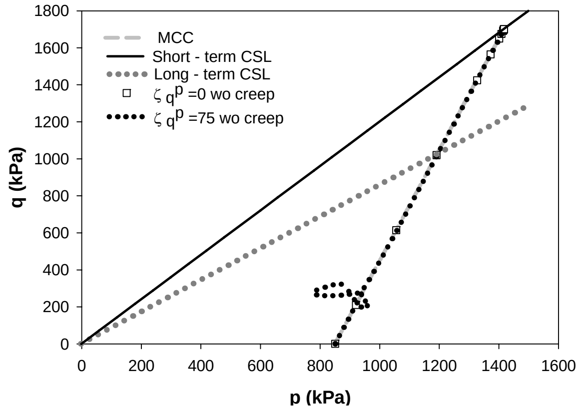

Figure 5. Triaxial stress paths at drained conditions without creep.

Although the **Fig4** is informative in terms of shear strength it fails to provide a clear portrait of the stress paths. **Fig5** provides a visual representation of the triaxial stress path at drained conditions in the *p-q* space.

Numerical analyses have been conducted for two different shear stress levels. In one case the specimen is stressed at a point lying above the quasi-static *CSL*. Assuming that the inclination of the *CSL* is allowed to evolve then eventually the specimen will fail in tertiary creep (**Fig6**).

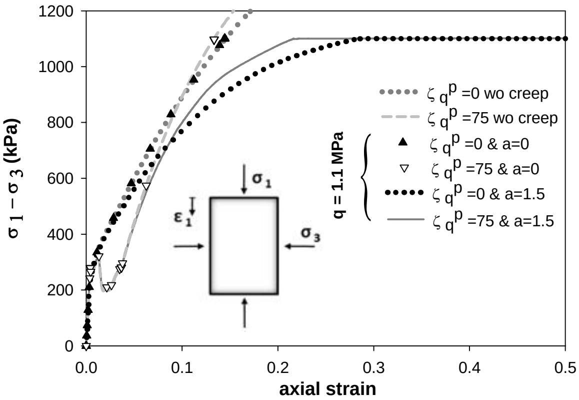

Figure 6. Shear strength experienced at high levels of shear stress with creep.

On the other hand, if the stress point lays underneath the threshold of the short-term *CSL* (**Fig7**) the specimen will not fail.

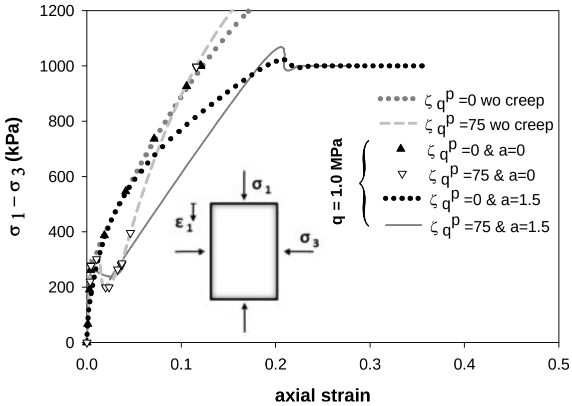

Figure 7. Shear strength experienced at low levels of shear stress with creep.

It is evident that even for high levels of loading the effect of structure degradation does not produce failure.

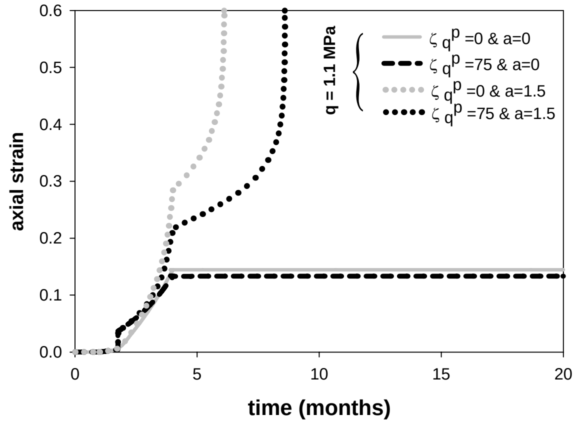

Figure 8. Axial strain as a function of time at high shear stress levels with creep.

On the other hand, strength envelope degradation towards its residual state has revealed the deleterious effect on shear strength. Regardless whether structure/strength degradation is evident or not, the evolution of the *SSE* inclination tends to cause creep-induced failure. In **Fig8** the axial strain is portrayed as a function of time.

4.2 Bearing Capacity of Surface Foundations

The bearing capacity of a 2m wide footing overlaying a clayey deposit under drained conditions has been examined. The soil medium portrays a punching failure mechanism rather than distinct failure lines which become evident only at undrained conditions, as can be seen from **Fig9.**

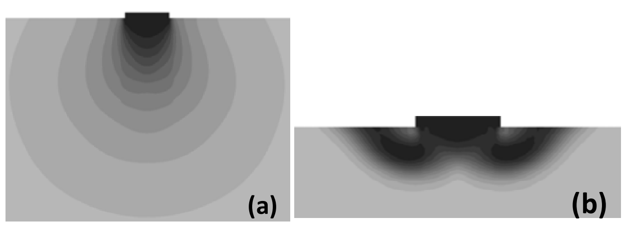

Figure 9. Displacement contour plots for (a) drained and (b) undrained boundary conditions without creep.

The effect of structure and structure/strength degradation is evident on shear strength in **Fig10**.

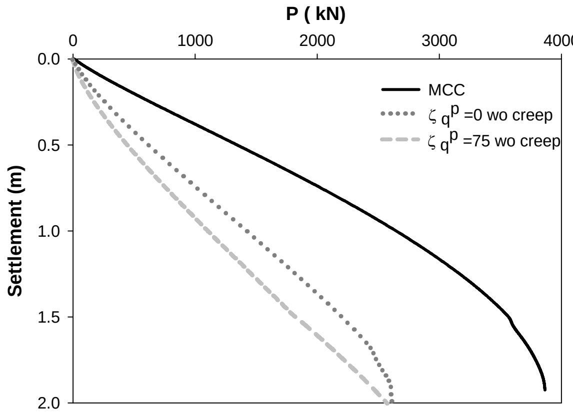

Figure 10. Effect of structure on shear strength at drained triaxial specimens without creep.

For imposed displacement equal to the width of the footing the soil reaction is significantly less than the one obtained from an elastoplastic analysis without an *ISE* surface.

Numerical analyses have been conducted for two different levels of imposed loading. The loads selected (0.5 & 1MN) appear to be nowhere near the bearing capacity. In **Fig11** settlement is given as a function of soil reaction for the case of *P*=0.5MN.

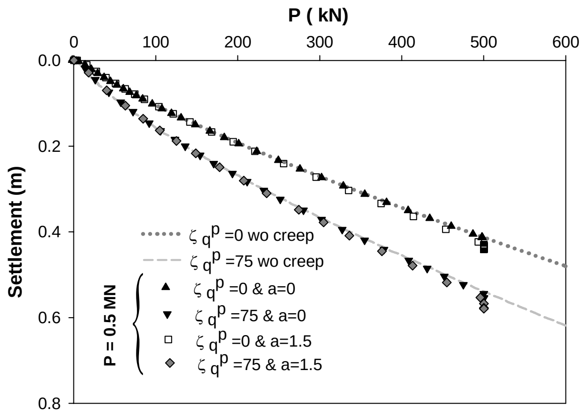

Figure 11. Soil reaction as a function of settlement, *P*=½ MN.

On the other hand, once the level of imposed loading doubles in value the footing tends to fail in creep as can be seen in **Fig12**.

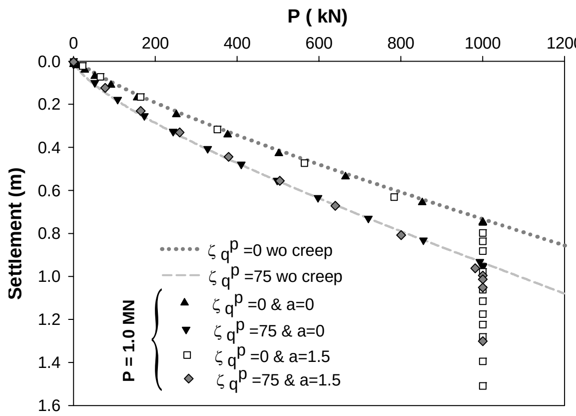

Figure 12. Soil reaction as a function of settlement, *P*=1MN.

Even for high levels of loading the effect of structure / strength degradation does not produce failure. On the other hand, degradation of the strength envelope revealed the deleterious effect of creep on shear strength, as can be seen from **Fig13**.

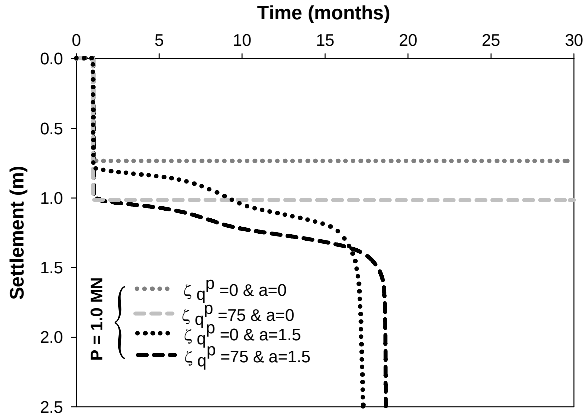

Figure 13. Settlement as a function of time, *P*=1MN.

5 CONCLUSIONS

A new elastoplastic-viscoplastic constitutive model has been developed based on the concepts of bounding surface plasticity and critical state soil mechanics principles incorporated within the framework of the overstress theory. Although structure/strength degradation has been shown not to result in tertiary creep, delayed failure can be realized through degradation of the *CSL* in the stress space to a residual state. Model predictions tested in triaxial shearing mode revealed that the model can predict tertiary creep behaviour at high shear stress levels. The model is implemented in the commercial F.E. Code ABAQUS and was further employed in the bearing capacity of a surface footing reproducing similar results.

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Αναμνηστική φωτογραφία των συμμετασχόντων στο 22nd EYGEC

Tunnelling in overstressed rock

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ABSTRACT: Overstressing of the rock surrounding a tunnel can result in either brittle fracture of the intact rock or shear failure along pre-existing discontinuities such as joints or shear zones. These two types of failure can co-exist and the extent to which the failures propagate depends upon the characteristics of the rock mass, the magnitude and directions of the in situ stresses, the shape of the tunnel and the intensity and orientation of the discontinuities. Numerical analysis plays an increasingly important role in the assessment of tunnel stability and design of reinforcement and support. Rapid evolution of computer software and hardware offers the potential for the calibration or possible elimination of some of the empirical techniques upon which tunnel designers have to rely.

1 INTRODUCTION

The rock mass surrounding a tunnel can be overstressed when either the intact rock or the discontinuities fail as a result of the stresses induced by the excavation of the tunnel. There are a variety of conditions under which such overstressing can occur and three examples will be discussed in this paper. These are the failure of massive intact rock, sparsely jointed anisotropic rock masses and heavily jointed rock masses. These three cases serve to illustrate the basic principles of assessing the type and extent of failure and of designing reinforcement or support to stabilise the tunnel.

When the conditions for overstressing exist it is seldom possible to prevent failure initiating. When such failure occurs the aim of the design of reinforcement or support is to control the propagation of the failure and to retain the profile of the tunnel. Stability becomes increasingly difficult to control if ravelling of the near surface rock pieces is allowed to occur and the interlocking or arching of the rock mass is destroyed by progressive deformation. Support must be chosen to match the deformation characteristics of the rock mass surrounding the tunnel.

2 IN SITU STRESSES

Of all of the quantities that the geotechnical engineer is required to estimate or to measure, the in situ stress field in a rock mass is one of the most difficult. The vertical stress can be approximated, to an acceptable level of accuracy, by the product of the depth below surface and the unit weight of the rock mass. However, in complex tectonic environments the vertical stresses may be lower (Mayer and Fabbre, 1999) or higher (Stille and Palmström, 2008) than the overburden stress. Horizontal stresses of interest to civil and mining engineers are influenced by global factors such as plate tectonics and also by local topographic features.

Zoback (1992) described the World Stress Map project that was designed to create a global database of contemporary tectonic stress data. The data included in this map were derived mainly from geological observations on earthquake focal mechanisms, volcanic alignments and fault slip interpretations. The results included in this map, available at www.world-stress-map.org, are very interesting to geologists involved with regional or continental scale problems. However, other than providing a first estimate of stress directions, they are of limited value to engineers concerned with the upper few hundred metres of the earth's crust. The local variations in the in situ stress field are simply too small to show up on the global scale.

A more useful basis for estimating near-surface horizontal in situ stresses was proposed by Sheorey (1994). He developed an elasto-static thermal stress model of the earth. This model considers curvature of the crust and variations of elastic constants, density and thermal expansion coefficients through the crust and mantle. A plot of the ratio of horizontal to vertical stress predicted by Sheorey's analysis, for a range of horizontal rock mass deformation moduli, is given in Figure 1. This plot is similar in appearance to that derived by Brown and Hoek (1978) from measured in situ stresses around the world (data points included in Figure 1). While this similarity does not constitute a proof of the correctness of Sheorey's solution, it is at least comforting to find this correlation between theory and observations.

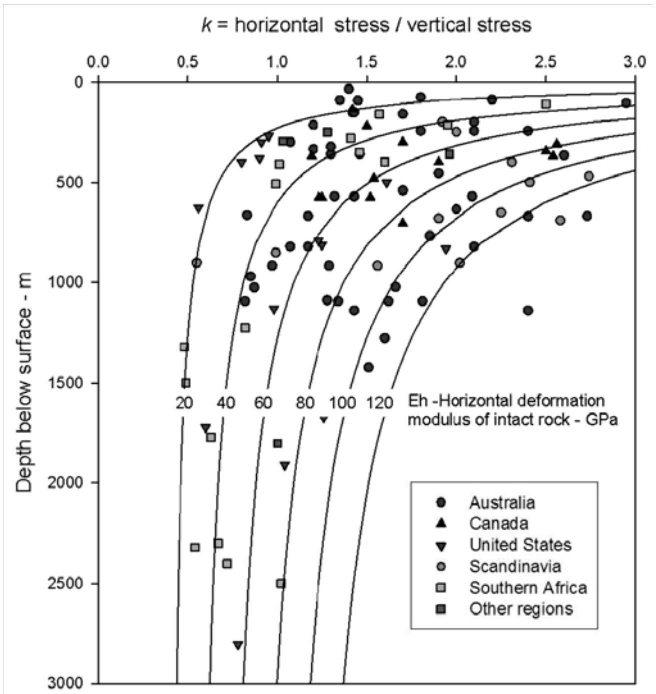

Figure 1. Measured horizontal to vertical stress ratio for various regions around the world compared with theoretical relationships derived by Sheorey (1994).

Note that neither Sheorey's equation nor the trends established by Brown and Hoek account for local topographic influences on the stress field. Hence, when making estimates of the in situ stress field in a mountainous area, adjustments must be made to account for these topographic factors. In carrying out an analysis of the stresses induced by the excavation of a tunnel, it is prudent to consider a range of possible in situ stresses. For example, consider a tunnel located in a steep-sided valley where the regional horizontal in situ stress is estimated to be twice the vertical stress. The horizontal stress at right angles to the valley axis could be varied from one half the vertical stress to twice the vertical stress. The stress parallel to the valley could be varied from a minimum value equal to the vertical stress to a maximum value of three times the vertical stress. An exploration of the effects of all possible combinations of these stress values would give a good indication of whether or not these in situ stresses would be critical to the design of the underground excavations. In cases where a preliminary analysis indicates that the design is very sensitive to the in situ stresses, measurement of the in situ stresses has to be considered a priority in the ongoing site investigation and design process.

3 FAILURE OF MASSIVE UNJOINTED ROCK

In massive unjointed rock tensile failure, originating at flaws and defects such as grain boundaries, can occur when the maximum stress on the tunnel boundary exceeds about 40% of the uniaxial compressive strength of the material.

These failures propagate along maximum principal stress trajectories and form thin plates parallel to the tunnel boundary as shown in Figure 2. In hard brittle rock, failure of these plates can be associated with significant energy release, which is known as "popping rock", "strainbursts" or, in extreme cases, "rockbursts".

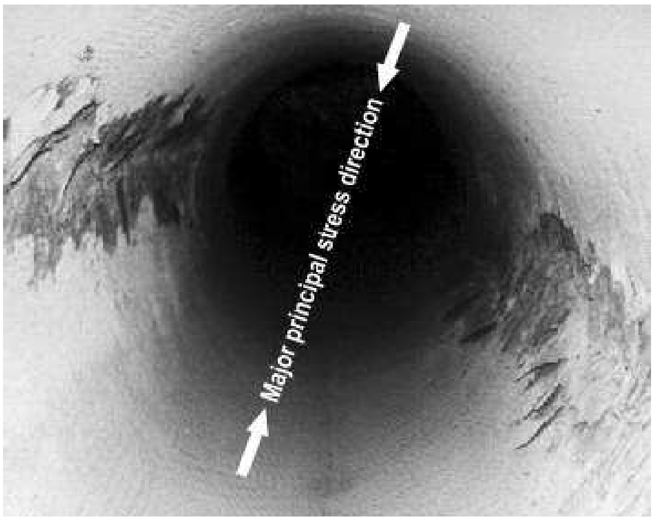

Figure 2. Brittle failure in the walls of a bored vertical shaft in a hard rock deep mine.

Underground hard rock miners have been familiar with this type of failure for many years and there has been considerable research devoted to this problem (Kaiser et al, 1996, Blake and Hedley, 2004). Civil engineers involved in the design of shallow tunnels have paid little attention to this type of failure since it seldom caused major problems. However, with the development of deep level tunnels in mountainous terrain, particularly those driven by tunnel boring machines, there have been an increasing number of brittle failure problems which have raised awareness amongst civil tunnel engineers.

Martin (2008) has shown that tensile cracks initiate at 40 to 50% of the uniaxial compressive strength of most massive rocks, including massive sedimentary rocks. Figure 3 is a plot of crack initiation stresses, determined by strain and acoustic emission measurements, for a wide range of rocks. In general, brittle failure tends to be self stabilising when the stresses at the tip of the notch formed by the failure no longer satisfy the conditions for the failure to propagate. This is shown in Figure 4, based on field observations, in which the depth of brittle failure is plotted as a function of the ratio of maximum boundary stress to uniaxial compressive strength of the intact rock. This plot is useful in that it gives an indication of the volume of rock that has to be supported once the failure has stabilized. As the depth of failure increases, the excavation becomes increasingly difficult to support and the energy release associated with the failure increases.

Figure 5, compiled by Martin et al (1999) from field observations collected by Hoek and Brown (1980) shows the increasing difficulty of stabilising the tunnel as the ratio of maximum boundary stress to uniaxial compressive strength increases. The start of brittle failure in the bored vertical shaft shown in Figure 2 would correspond to $\sigma_{max}/\sigma_c \approx 0.45$ while the rockburst conditions illustrated in Figure 6 suggest $\sigma_{max}/\sigma_c \approx 1.6$.

Considerable progress has been made in understanding and predicting the onset of brittle fracture (Martin and Christiansson, 2008, Diederichs et al 2004, Martin 1997) and in the development of empirical depth of failure relationships such as that given in Figure 4 (Martin et al, 1996, Martin et al, 1999).

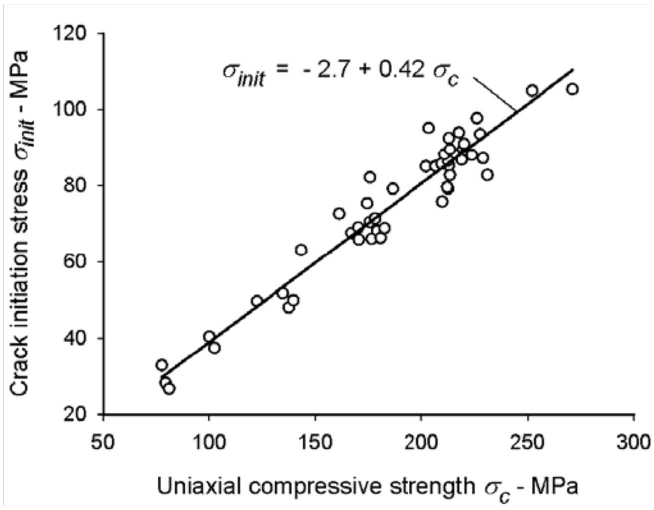

Figure 3. Relationship between brittle crack initiation stress and uniaxial compressive strength of massive rock. After Martin (2008)

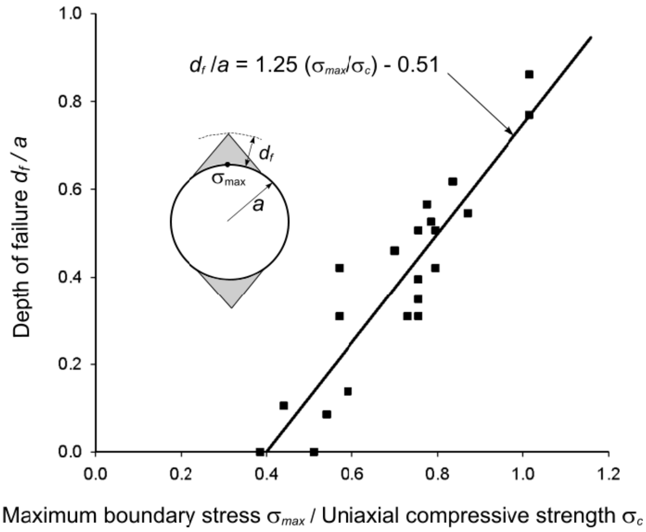

Figure 4: Empirical observations for normalized radius of failure. The standard excavation radius R is calculated from a circle circumscribing a square or horseshoe shaped excavation. The best fit equation is presented in terms of the average uniaxial compressive strength σ_c from laboratory tests. (After Martin et al., 1996).

Reliable numerical models have been developed for predicting the depth, shape and surface extent of brittle failure (Diederichs, 2007). However, these numerical models do not give meaningful predictions of the dilation (bulking) of the fracture zone nor do they explain why very small support pressures can suppress the propagation of failure. With improvements in computational efficiency and the development of efficient discrete element models it is probable that these limitations will be overcome in time.

The best tools that we have at present are empirical relationships developed by the mining industry (Kaiser et al, 1996). These show that support of tunnels in massive rock, in conditions under which brittle failure can occur, range from light wire mesh and rockbolts, for very minor failure, to heavy support such as the cable lacing illustrated in Figure 7. In such cases early installation of the support, in order to maintain the excavation profile, is essential and the support has to have sufficient ductility to accommodate the volume changes associated with the failure.

Figure 8 shows steel sets being installed inside the finger shield, immediately behind the roof shield of an open face hard rock TBM in the Olmos tunnel in Peru. This tunnel is being excavated at depths of up to 2000 m below surface and overstressing in andesitic rock resulting in strainbursting has been controlled by means of the support illustrated (Guevara, 2008). Because of space limitations and the time requirements, it is not practical to install a regular rockbolt pattern immediately behind the TBM and steel sets provide an effective support system.

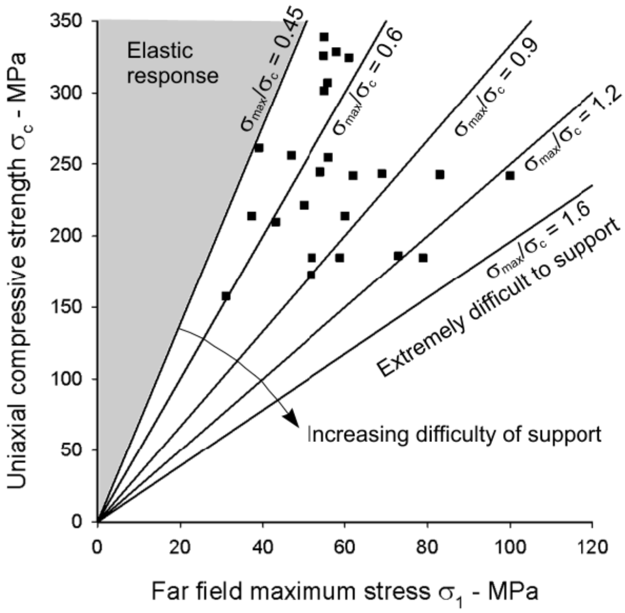

Figure 5. Increasing difficulty of supporting underground excavations with increasing ratio of maximum boundary stress to uniaxial compressive strength (Martin et al, 1999, based on field observations collected by Hoek and Brown, 1980).

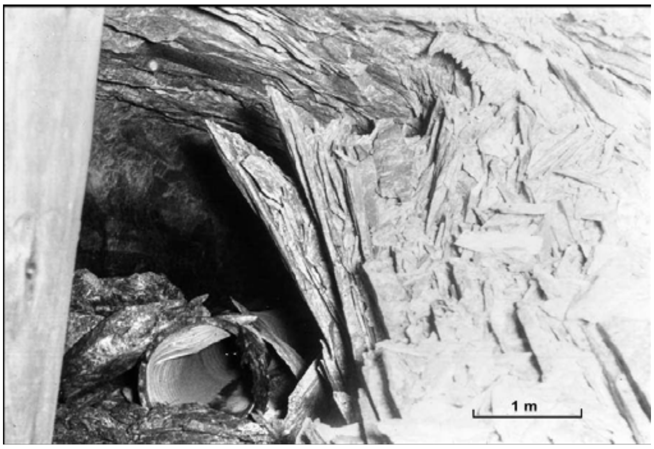

Figure 6. Results of a rockburst in a deep level (≈ 3000 m) gold mine in South Africa.

4 INFLUENCE OF DISCONTINUITIES

The presence of discontinuities (joints, bedding planes, schistosity planes, shear zones etc.) in the rock mass surrounding a tunnel introduces the potential for shear failure along these discontinuities. This depends upon the number, spacing, continuity, orientation and inclination of each discontinuity set as well as the shear strength of the surfaces and the stiffness of the intact rock. For convenience in the following discussion the term "jointed rock mass" will be used to cover all of these discontinuity types.

Apart from those fractures induced by the tunnelling process, all the joints in a rock mass are the result of the rock genesis and tectonic deformations during the geological history of the area. Before embarking upon any form of analysis of the behaviour of a jointed rock mass it is necessary to develop a sound geological model and an understanding of the genesis of the joints and of the sequence of their formation (Fookes et al., 2000, Harries and Brown, 2001). This understanding involves input from structural and/or engineering geologists who are familiar with the regional geology and the tectonic history of the area under investigation. Engineers should avoid the temptation to start assigning numbers to the joints or to the rock mass properties until an adequate geological model has been developed.

Figure 7. Cable lacing in a deep level hard rock mine to control damage from brittle failure. Tensioned cables are attached to grouted anchors installed on a regular pattern.

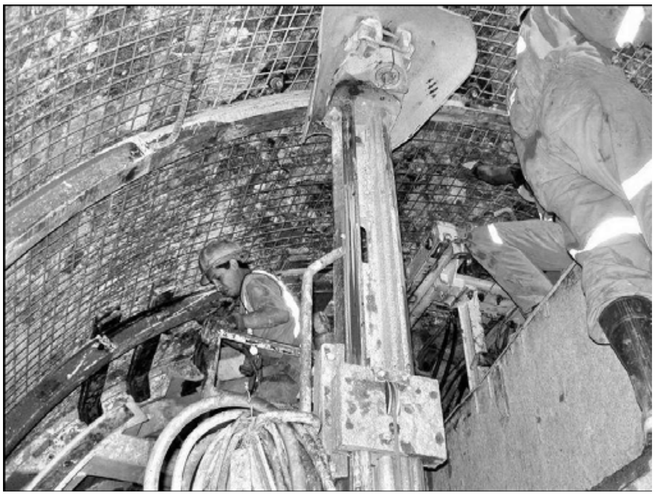

Figure 8. Installation of steel sets immediately behind a TBM shield in the Olmos tunnel in Peru. Photograph provided by R. Guevara (2008).

A full discussion on jointing in rock masses is clearly beyond the scope of this paper. However, in order to provide some guidance, the authors have compiled a matrix of joint characteristics together with other features for typical rocks and this is presented in Appendix 1.

4.1 *Anisotropic failure of sparsely jointed rock*

A sparsely jointed rock mass can be defined as one in which relatively few joint sets occur and where there is frequently a strongly preferred inclination and orientation of the dominant joint set. A typical example would be a bedded sedimentary deposit which has not been subjected to significant postdepositional deformation (a molasse). An example of such a situation is illustrated in Figure 9.

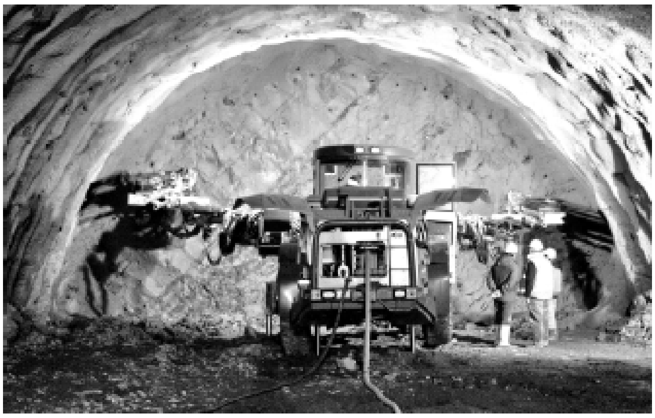

Figure 9: Sandstone and siltstone molasse exposed in the face of a 12 m span tunnel top heading in Greece.

During preliminary site investigations classification systems can be useful in establishing the general characteristics of the rock mass but these classifications, including the GSI system and the associated Hoek-Brown failure criterion (Hoek et al, 2005), are of limited value when highly anisotropic stability problems are anticipated. Under these circumstances there are no short cuts and comprehensive site investigation programs, including establishing the orientation of fractures encountered in diamond drill holes, are required.

Once the geological model has been established and a reasonable understanding of the mechanics of potential failures has been arrived at, a numerical model can be created. This model should incorporate the sequence of excavation and support installation as well as the intact rock and discontinuity characteristics and in situ stresses. Groundwater should be included in the model where this is considered to be significant (Hoek et al, 2008). Fortunately there are a number of commercially available programs which enable users to model all of these features to a level of detail that is acceptable for engineering design.

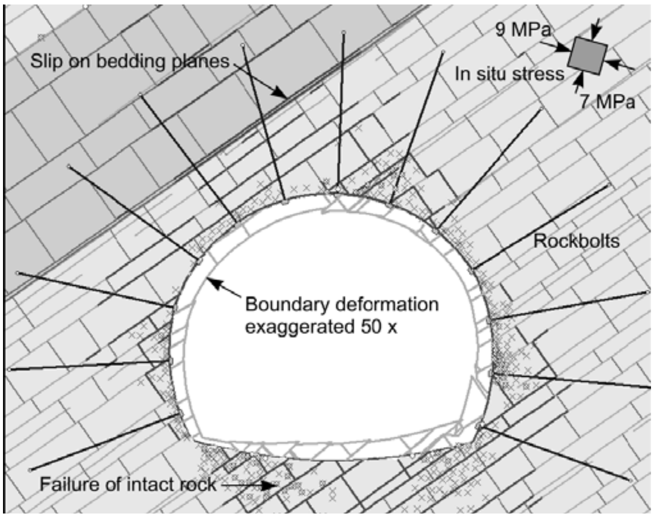

Figure 10. Finite element model of a 12 m span tunnel excavated in interbedded sandstone and siltstone with presheared bedding surfaces.

An example of this type of analysis is shown in Figure 10 in which a 12 m span tunnel is excavated in an interbedded series of sandstone and siltstone layers similar to those illustrated in Figure 9. In this analysis a pattern of 6 m long, 32 mm diameter untensioned grouted rockbolts and wire mesh have been installed 2 m behind the face. A 200 mm thick layer of shotcrete has been installed 5 m behind the face. The procedure for sequencing the installation of reinforcement and support in a two dimensional numerical model is described by Hoek et al (2008).

There is no method for calculating the factor of safety of a tunnel, with a combination of reinforcement and support, such as that shown in Figure 10. As stated earlier, once conditions for failure of either the discontinuities or the intact rock are satisfied it is not possible to prevent this failure. However, the extent of the failure and the tunnel boundary deformations can be controlled by the installation of reinforcement or support. The aim of the designer should be to retain the tunnel profile as far as possible and to prevent or minimise small rockfalls from the surface.

In order to achieve this goal it may be necessary to install a combination of reinforcement and support and to vary the rockbolt length, spacing and inclination to capture specific instability zones. Similarly, the initial and final lining thickness and reinforcement may have to be varied to deal with anisotropic deformation patterns. These changes can only be optimised by iterative analyses with checks to ensure that the factor of safety of each individual support component is within acceptable limits.

4.2 *Failure of heavily jointed rock masses*

Figures 11 and 12 show examples of heavily jointed rock masses. In Figure 11 the structural pattern of the many joints has been retained while in Figure 12 the fabric has been completely destroyed by tectonic deformation. In either case the joints are sufficiently closely spaced that the rock mass, on the scale of a tunnel, can be treated as isotropic and homogeneous. This greatly simplifies numerical analyses and permits the use of homogeneous models and of rock mass classification schemes to provide input data for rock mass properties.

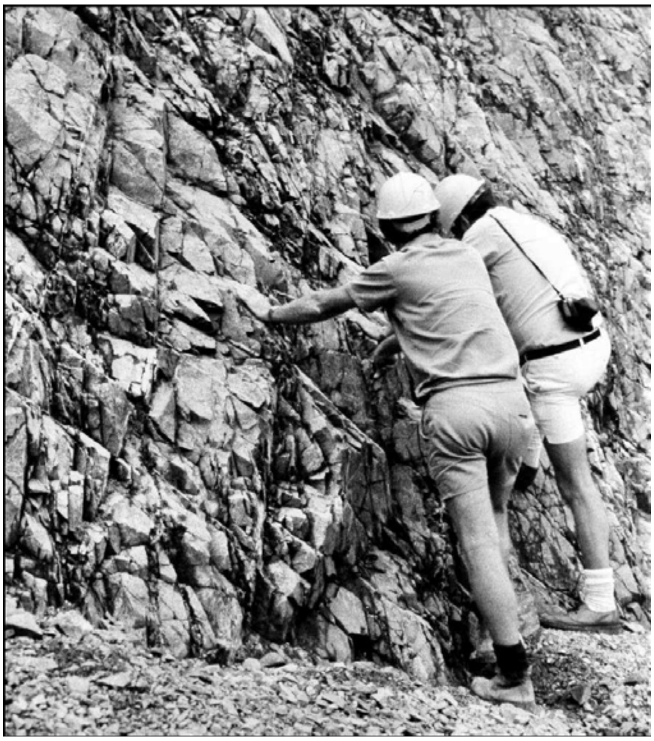

Figure 11. Heavily jointed andesite in an excavated slope face in Papua New Guinea.

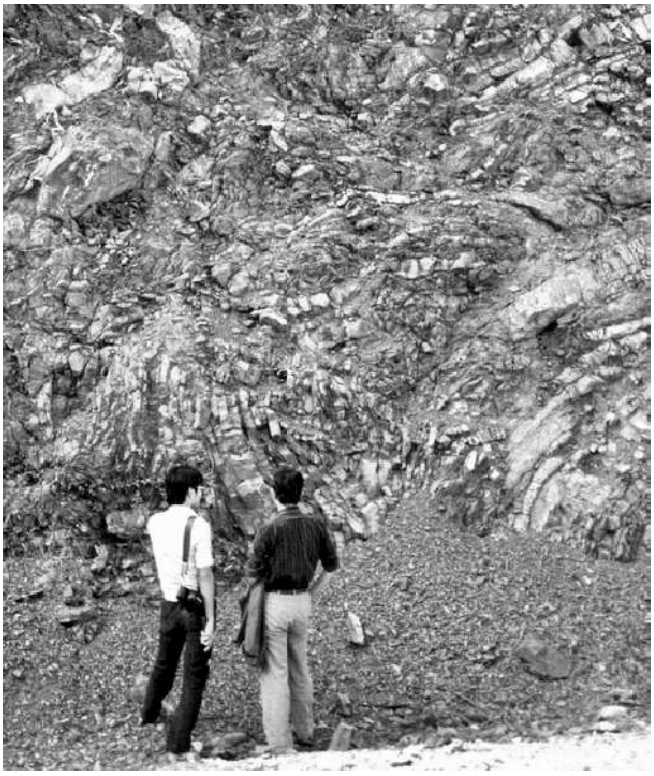

Figure 12: Destruction of structural fabric as a result of tectonic deformation of an interbedded sedimentary rock mass in southern Taiwan.

Another feature of rock masses in which the discontinuities have been pre-sheared by tectonic deformation is that they have a limited capacity to sustain shear stresses. Consequently, over geological time, the in situ stresses tend to equalise and a reasonable first approximation of the in situ stress field is that horizontal and vertical stresses are equal. This conclusion is generally impossible to confirm by direct measurement but back analysis of the behaviour of tunnels excavated in such materials tends to confirm this equalisation of the in situ stresses.

Figure 13 gives a comparison between finite element analyses of a tunnel excavation using a heavily jointed model (on the left) and an equivalent homogeneous model (on the right). The jointed model has the advantage that structural data on jointing obtained in the field and laboratory data on rock mass properties can be used directly in the model. However, the computational demands of such models limit their current use to relatively simple problems. On the other hand, the equivalent homogeneous model allows for very efficient numerical modelling but it imposes significant demands on the user to estimate realistic material properties.

4.3 *Inclusion of joints in numerical models*

Several commercially available continuum models permit the insertion of individual joints or joint networks (Rocscience, 2008, Lorig, 2007). These networks may include parallel or cross-jointed arrays, joints with a statistically distributed finite trace length (Baecher et al 1978), Veneziano or Voronoi joints patterns (Dershowitz, 1985). These models make it possible to study the behaviour of a wide variety of heavily jointed rock masses and they have proved to be very useful for practical tunnel designs.

In using these continuum models it has to be kept in mind that they do not permit separation of joint planes or rotation of blocks and that their use should be restricted to small deformation problems. However, since most tunnel designers are interested in limiting the deformation of tunnels and in maintaining the opening profile, this restriction is not a serious limitation in tunnel design.

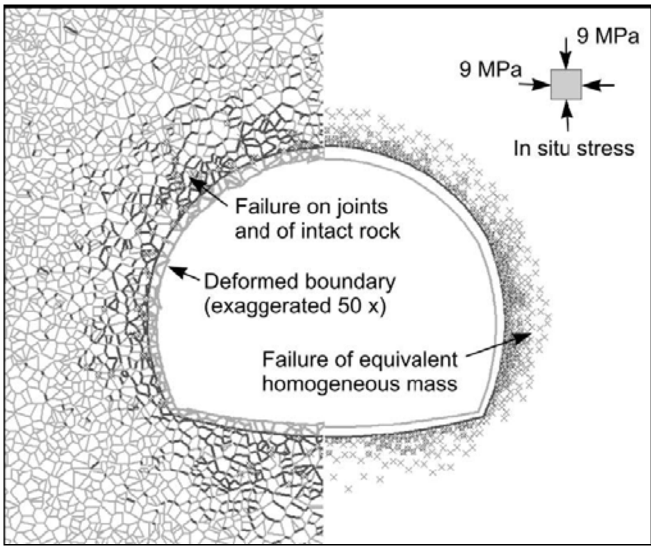

Figure 13. Comparison between results of analyses using a two dimensional continuum model with closely spaced Voronoi joints (left) and equivalent rock mass properties (right). Note that intact properties are used for the rock pieces between joints in the jointed model. Phase2 v7 model (www.rocscience.com).

For larger deformation problems where joint separation and block failure and rotation are likely to occur, discrete element (Lorig, 2007) or combined finite-discrete element models (Munjiza, 2004, Crook et al, 2003) need to be used. Some of these codes were developed for applications other than rock engineering and need some adaptation before they can be used efficiently for tunnel design.

These are sophisticated and powerful codes and the potential user should not under-estimate the investment in time and resources required in order to learn to utilise them correctly. For research or consulting groups interested in remaining in the forefront of tunnel design this investment is well worth making.

4.4 *Use of rock mass classifications*

The use of rock mass classification systems goes back more than 60 years when authors like Terzaghi (1946) attempted to describe the characteristics of rock masses and their response to tunnelling. Since that time numerous rock mass classifications have been developed and probably the best known are those of Barton et al (1974) and Bieniawski (1976). These classification systems played an important role in tunnel design before the development of the numerical models discussed above. They continue to play an important role in providing initial estimates of the range of problems likely to be encountered and of solutions that can be considered and also in estimating rock mass properties for input into numerical models.

Hoek and Brown (1980) considered that more detailed rock mass property information would be required as numerical modelling became more readily available and more widely used in design. They set out to develop a failure criterion (the Hoek-Brown criterion) and a classification system (The Geological Strength Index, GSI) specifically for this purpose (Hoek and Marinos, 2007). This classification differs from those described above in that it has no end use other than to provide input for the Hoek-Brown criterion. It was not intended for and should not be used for estimating tunnel support requirements, excavation advance rates or tunnel costs. Furthermore, it assumes a homogeneous isotropic rock mass and should not be used for the analysis of anisotropic or strongly structurally controlled rock mass behaviour.

An example of the application of the GSI classification and the Hoek-Brown criterion has been published by Hoek and Guevara (2009). This deals with the Yacambú-Quibor tunnel in Venezuela in which severe squeezing was encountered in graphitic phyllite at depths of up to 1200 m below surface. The analysis utilises a relationship developed by Hoek and Marinos (2000) for the prediction of the extent of squeezing on the basis of the ratio of rock mass strength σ_{cm} to overburden stress po, shown in Figure 14.

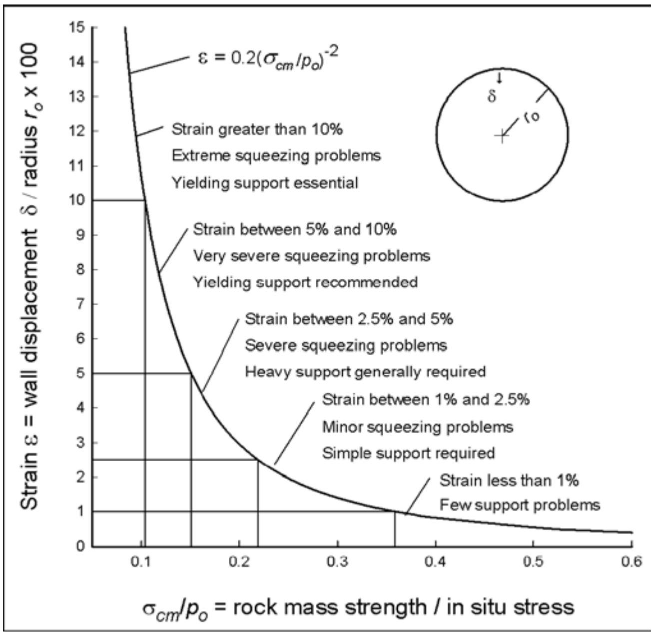

Figure 14: Relationship between tunnel strain and ratio of rock mass strength to in situ stress (Hoek and Marinos, 2000).

Figure 15 shows that there are a significant number of locations along the tunnel where the strain approaches or exceeds 10% which, according to Figure 14, represents a severe squeezing condition.

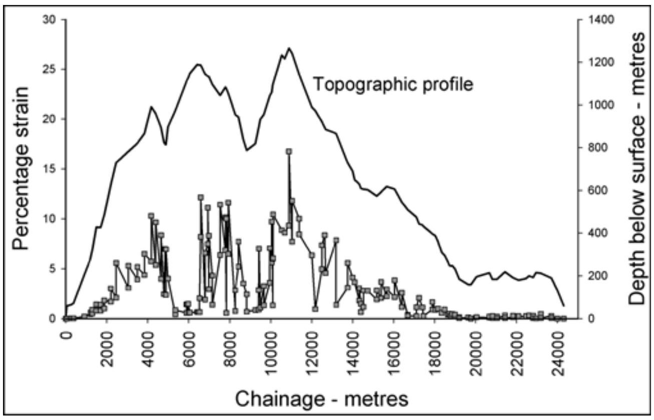

Figure 15: Strain along the Yacambú-Quibor tunnel predicted from the ratio of rock mass strength to in situ stress. (Hoek and Guevara, 2009).

During excavation of the tunnel severe squeezing was encountered in a number of the locations where inadequate support had been installed. Re-mining and rehabilitation was required in these locations as illustrated in Figure 16. Lessons learned in such cases were used to develop a support design procedure based on the use of sliding joints in circular steel sets. These sets were installed as close as possible to the excavated face and the 5 m diameter tunnel was permitted to converge about 50 cm before the sliding joints locked and the sets developed their full capacity. At a distance of about 15 m behind the face the shotcrete windows, that had been left to permit the joints to slide, were filled and additional shotcrete was applied to build up a final lining of up to 70 cm thickness, depending on the rock mass properties and in situ stress levels.

Figure 16: Failure of a tunnel section due to very severe squeezing (background) and re-mined and re-supported tunnel (foreground).

5 PREDICTIONS FOR THE FUTURE

Rock engineers have to work within the limitations of available technology and, without doubt, some of the most severe limitations are associated with the estimation of rock mass properties. The strength and deformation characteristics of the rock and the discontinuities play a major role in determining stability as well as the reinforcement and support requirements in tunnelling,

Efforts to overcome these limitations have resulted in tools such as the GSI classification (Hoek and Marinos, 2008) which, at best, can only be regarded as crude interim solutions. It is simply not possible, within the constraints of a classification system based on a limited number of estimated input parameters, to capture the actual behaviour of heterogeneous rock masses. This is not to say that these efforts have not been useful since, when they were developed, there very few practical alternatives available.

Fortunately, with developments in computer hardware and software technology, there is now a reasonable expectation that some of mysteries of rock mass property estimation may be dispelled over the next decade. This expectation is centred on our rapidly improving ability to incorporate laboratory determined intact rock and discontinuity properties into numerical models. As discussed in Section 4.3 above, several commercially available codes permit this type of analysis and some of them are capable of producing credible results in the analysis of failure initiation and post failure behaviour of complex rock mass and applied stress conditions (Lorig, 2007).

Discrete element and combined finite - discrete element analyses are currently fashionable research topics and geotechnical journals and conferences abound with papers with spectacular demonstrations of fractured rock masses falling apart. Very few of these techniques are available as robust, validated and user-friendly tools that the average tunnel designer could use. As with the development of other numerical tools, several years will be required to sort the wheat from the chaff and to allow a consolidation of the technology. Eventually these methods will be integrated into existing software or they will form the basis of a new family of powerful two- and three-dimensional design tools.

The authors look forward to the time when these numerical tools will allow us to at least calibrate if not replace completely some of the empirical methods, such as the GSI classification and the Hoek – Brown criterion that we use today.

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7 APPENDIX 1 - ROCK MASS PROPERTIES

The authors of this paper have always advocated that users of the GSI classification should rely on the application of sound engineering geology principles rather attempt to be too quantitative in the estimation of rock mass properties. In the introductory box of the GSI chart, a typical example of which is given in Table 1, they warn: “Do not attempt to be too precise. Quoting a range of GSI of 33 to 37 is more realistic than giving GSI = 35.”

One of the typical problems faced by inexperienced users is to judge the properties of the rock and the discontinuities in a specific rock mass. Even geologists will sometimes fail to recognise the engineering significance of their interpretation of the rock mass characteristics. A discussion on the applications of GSI can be found in Marinos et al (2007).

In an attempt to provide some ideas on the choice of the properties that should be used in working with the GSI charts, a new pair of tables are presented in this appendix. Table 2 give a number of possible combinations of properties that may occur in different rock types. Table 3 defines these properties for intact rock, joints and for rock masses. The properties considered include intact strength, alteration, weathering, solution potential, anisotropy, joint characteristics and permeability.

For example, serpentinite would be categorised as follows:

Dominant factors

1B-C: UCS ranges from 15 to 100 MPa
3D: High weathering potential
8C-D: Planar to very planar joints
9D: Possibility of clay filling in joints
10D: Potential for heavy slickensided joints
11A-C: Generally low joint controlled permeability

Significant factors

2B-C: Slight to moderate alteration potential
7B: Slight anisotropy
13B: Slight persistent schistosity when tectonised
4B: Slight swelling potential

Factors that can be ignored

5A: No solution potential
6A: No potential for void formation
12A: No persistent thin bedding planes
14A: No heterogeneity due to alternating layers

Note: The combinations included in the Table 2 are those found most frequently in rock and rock masses in situ depending from their petrographic nature and tectonic history. Other combinations may occur.

Keynote address presented at EUROCK2009 in Dubrovnik, Croatia in October 2009 (http://www.eurock2009.hr).

Table 1- Geological Strength Index Table for graphitic phyllite

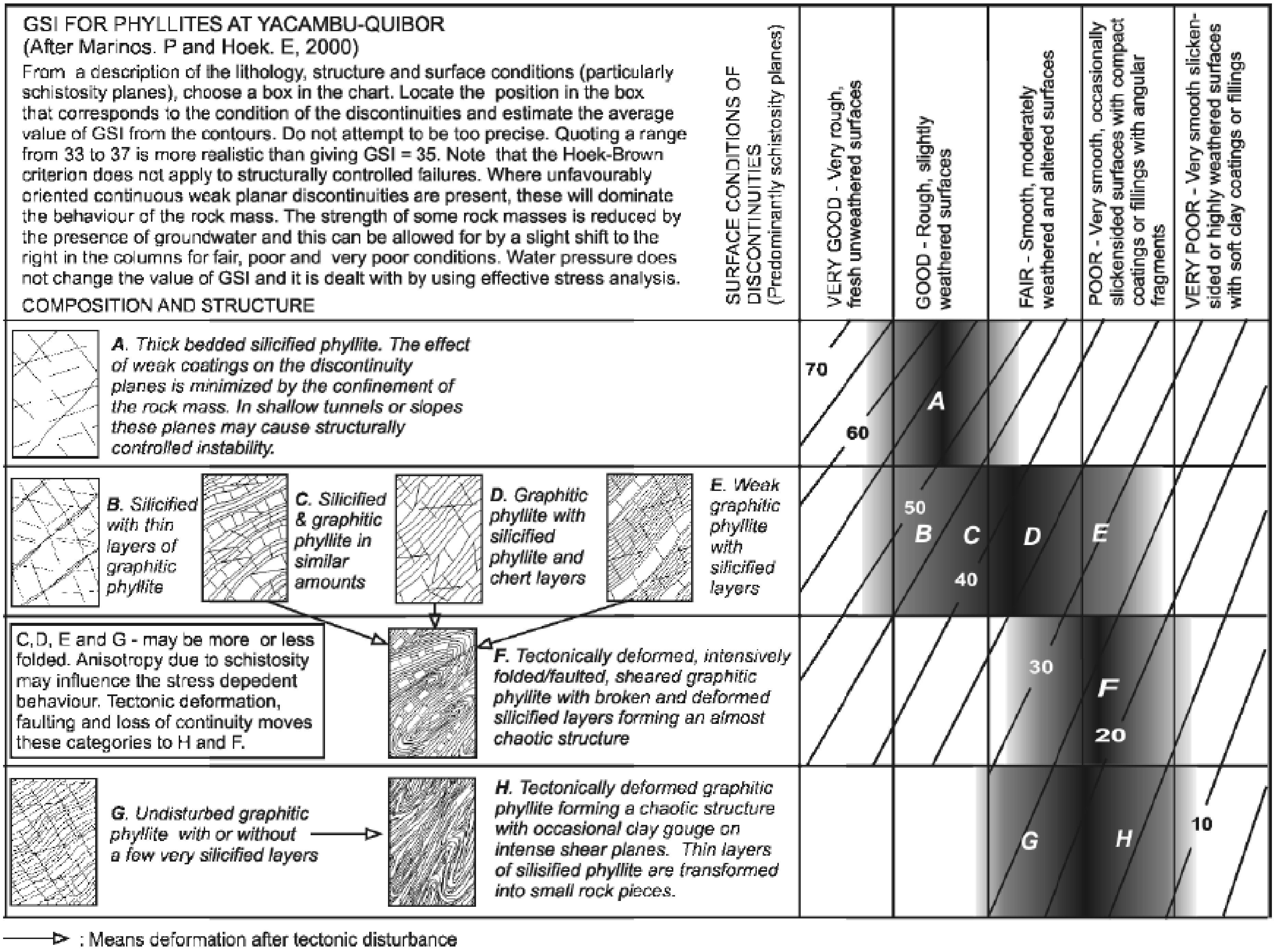


Table 2 – Possible features for different rock types

Rock type	Class	Group	Texture			
			Coarse	Medium	Fine	Very fine
SEDIMENTARY	Clastic		Conglomerates Breccias 1B-D, 2A, 3B-C, 4A, 5B, 6B, 7A, 8B, 9BC, 10B, 11A, 12A-B, 13A, 14C	Sandstones 1A-B, 2A, 3B-C, 4A, 5A, 6A, 7A, 8A, 9B, 10B, 11A, 12B, 13A, 14D	Siltstones 1C-D, 2A, 3B, 4A-B, 5A, 6A, 7A, 8C, 9D, 10D, 11D, 12C, 13A, 14D	Claystones - Marls 1D, 2B, 3B-C, 4B-C, 5A, 6A, 7A, 8D, 9D, 10D, 11D, 12B, 13A, 14D
						Shales 1D, 2B, 3B-C, 4B-C, 5A, 6A, 7C-D, 8D, 9D, 10D, 11D, 12D, 13D, 14A
	Non- Clastic	Carbonates	Limestones – Dolomites 1B-C, 2A, 3A, 4A, 5A, 6D, 7A, 8B, 9A, 10B, 11A, 12A, 13A, 14C			
		Evaporites	Gypsum 1C, 2A, 3B, 4B, 5D, 6D, 7A, 8C, 9?, 10C, 11A, 12A, 13A, 14C		Anhydrite 1C, 2A, 3D, 4D, 5B, 6B, 7A, 8C, 9?, 10C, 11D, 12A, 13A, 14A	
METAMORPHIC	Non Foliated		Marble 1B, 2A, 3A, 4A, 5A, 6D, 7B, 8B, 9A, 10B, 11A, 12B, 13A, 14C	Hornfels (or cherts in sedimentary rocks) 1A, 2A, 3A, 4A, 5A, 6A, 7A, 8B, 9A, 10B, 11A, 12C, 13A, 14D		Quartzite 1A, 2A, 3B, 4A, 5A, 6A, 7B, 8A, 9B, 10B, 11A, 12B, 13C, 14D
	Slightly Foliated		Amphibolites – Gneiss 1A, 2B, 3C-D, 4A, 5A, 6A, 7C 8A, 9C-D, 10C, 11A, 12A, 13A, 14A			
	Foliated		Micaschists 1A, 2B, 3B-C, 4A, 5A, 6A, 7C, 8B, 9C-D, 10C, 11A, 12B, 13C, 14C		Phyllites 1C, 2B-C, 3C-D, 4B, 5A, 6A, 7D, 8D, 9C-D, 10D, 11C, 12C, 13D, 14D	
IGNEOUS	Plutonic	Light	Granite - Diorite – Granodiorite 1A, 2C, 3D, 4A, 5A, 6A, 7A, 8A, 9C, 10B-C, 11A, 12A, 13A, 14A			
		Dark	Gabbro - Norite 1A, 2C, 3C, 4A, 5A, 6A, 7A, 8A, 9C, 10C, 11A, 12A, 13A, 14A			
	Hypabyssal		Peridotite 1A, 2C-D, 3D, 4A, 5A, 6A, 7A, 8B, 9C-D, 10C, 11A, 12A, 13A, 14A		Serpetinites 1B-C, 2B-C, 3D, 4B, 5A, 6A, 7B, 8C-D, 9D, 10D, 11A-C, 12A, 13B, 14A	
	Volcanic	Lava	Rhyolite – Dacite – Andesite 1A, 2C, 3B-C, 4A, 5A, 6B, 7A, 8A, 9C, 10C, 11A, 12A, 13A, 14A		Basalt 1A, 2A-B, 3A-B, 4A, 5A, 6A, 7A, 8B, 9B-C, 10B, 11A, 12A, 13A, 14A	
		Pyroclastic	Agglomerate – Volcanic Breccia 1B-C, 2C, 3C, 4B-D, 5A, 6B, 7A, 8A, 9C, 10B, 11A, 12A, 13A, 14C		Tuff 1C-D, 2D, 3D, 4B-C, 5A, 6B, 7A, 8B, 9C, 10C, 11A, 12A, 13A, 14D	

For definitions of 1,2,..., A, B,..., refer to table 3.

Note: The combinations included in the table are those found most frequently in rock and rock masses in situ depending from their petrographic nature and tectonic history. Other combinations may occur in some cases except if not applicable (N/A) is noted (see table 3). This table is for guidance only and it should not be used as a substitute for site observations and data acquisition from a site investigation program.

Table 3 – Definition of rock type features

Property		A	B	C	D	
UCS of sound intact material	**1**	Very high > 100 MPa	High 50 -100 MPa	Medium 15 - 50 MPa	Low < 15 MPa	
Alteration potential	**2**	None	slight	moderate	high	
Weathering potential	**3**	None	slight	moderate	high	**Intact Rock**
Swelling potential	**4**	N/A	slight	moderate	high	
Solution potential	**5**	N/A	slight	moderate	high	
Voids – potential for formation	**6**	N/A	possible		Yes	
Anisotropy of intact rock	**7**	None	slight	moderate	high	
Joint surface characteristics (excluding schistosity)	**8**	very rough	waviness	planar with slight waviness	very planar	
Joint infilling from crushed rock – excluding excavation damage	**9**	None	Sandy	Clay with sandy particles	Clay	**Joints**
Slickensided joints – potential in sheared rock	**10**	None	minimal	moderate	heavy	
Permeability or rock mass	**11**	Depending on jointing or karstification	-	Low	Very Low	
Persistent thin bedding planes	**12**	None	possible	frequent	In most cases	
Persistent schistosity	**13**	N/A	slight	moderate	high	**Rock mass**
Heterogeneity - Possibility of alternating weak and strong rock layers on the scale of the engineering structure	**14**	No	-	Rare	Yes	

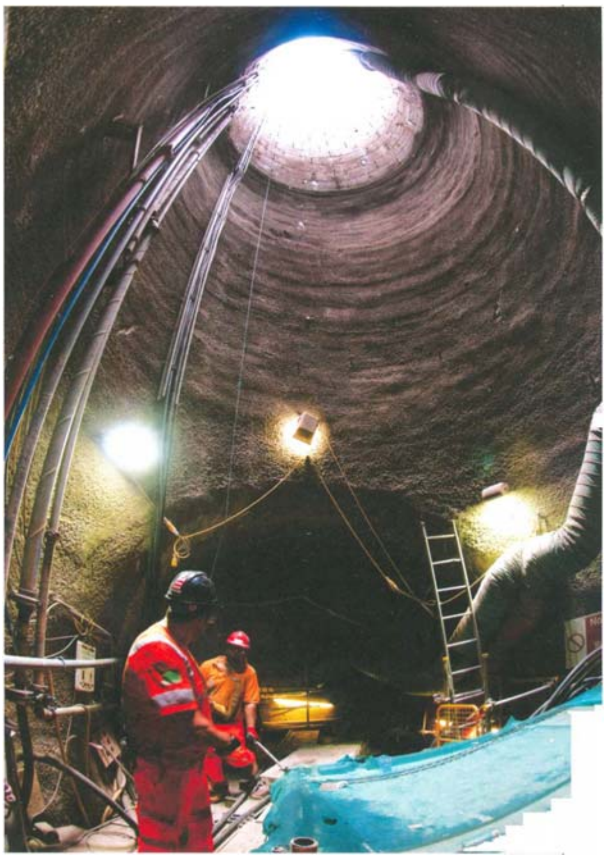
Can someone please explain exactly what is SCL?

David Hindle

A speedy and uncomplicated method has fallen prey to over-design and endless codes. David Hindle, partner of OTB Engineering speaks out on the present state of SCL design in the United Kingdom.

These days I despair for our tunnelling industry in the UK, which appears to have been hijacked by armchair experts, computer geeks and regulation fanatics, no more so than in sprayed concrete lined tunnels. Whilst I can see the need for some input from such specialists, when did we lose sight of the simplicity, elegance and practicality of the sprayed concrete method and replace it with the monster of their devising we now call SCL?

In any debate about the subject we must inevitably refer back to the New Austrian Tunnelling Method, which was not originally devised for soft ground but was adapted from its hard rock, high stress version by a collaboration between eminent Austrian and German engineers in the 1960s for use in constructing the Frankfurt Metro in a similar clay to that found in London. Perhaps it was a mistake to retain the NATM acronym for the soft ground version but the rather catchy name stuck and the method quickly caught on in Europe and then Worldwide for the very good reasons that it was simple, adaptive, cost effective and above all it worked.



SCL shaft construction on Tottenham Court Road station, London

Unfortunately, it met with massive resistance in the UK throughout the 1970s and 80s on several fronts, especially contractual, as it was difficult to incorporate the flexibility of the method within the rigid confines of the forms of contract in use at the time. Also, the traditionalist tunnellers vigorously expressed their scepticism by asserting that shotcrete was poor quality concrete that would not stick to clay or chalk, which was subsequently demonstrated to be erroneous on all counts. Perhaps the biggest anti-NATM cry was that it was neither new nor Austrian, which may well have been true, certainly German engineers had a big input and shotcrete had been used very successfully elsewhere, notably in Norway. However, I suspect that the main reason was that it had the word Austrian in it and not British. Incidentally, the acronym was originally adopted simply to distinguish it from the old Austrian tunnelling method and not for any overt nationalistic motives.

Eventually sanity prevailed and NATM began in the UK, not in civil engineering but in mining. However the inadequacy of traditional tunnelling methods to construct the massive underground complexes required for London's new transport infrastructure forced the UK tunnelling industry to adopt NATM and all went along swimmingly until the Heathrow debacle in 1994. What cannot be denied is that it was a monumental cock-up and the companies responsible were duly prosecuted. What then followed was a lengthy process of collective navel gazing from which emerged the SCL method

What seems to have been forgotten was that all tunnelling methods can potentially fail given a particular circumstance or combination of circumstances such as poor design, inferior materials, substandard construction, poor supervision or unforeseen ground, but above all there is only one mechanism that causes a tunnel to collapse and that is gravity. Tunnelling in whatever context is in the end a contest against the force of gravity with gravity holding most of the aces.

Pick and mix

The mistake was that NATM was turned into cookbook of the engineering codes and standards beloved of the British engineer and has since evolved into a methodology that would make Delia Smith [a British cooking author] proud. The fundamental but true criticism of the NATM is that it was never clearly defined as to what it is and is attempting to achieve. Unfortunately the same can be equally applied to SCL. When this question is raised, one commonly held belief is that NATM is essentially an observational method whereas SCL is not; i.e. SCL is designed to cope with all the anticipated ground conditions and performance requirements. If this is so then what is the purpose of monitoring it? One would reasonably assume that it is to check whether the design is working correctly. If the monitoring shows that it is not, what then, change the design? If so then it is, by definition, an observational method. By way of confirmation an SCL design often incorporates some 'pick-and-mix toolbox' support supplements (rather like NATM).

The SCL acronym itself is somewhat misleading as it refers only to the finished product, when the fundamental key to its successful application is the sequential nature of the excavation and support and above all the most important design consideration is to close the invert as quickly as possible otherwise gravity (remember that irritating component) will have its evil way. So the best approach must surely be the simplest and quickest method in achieving ring closure and, to their credit, British contractors have looked to new technology to achieve this, such as semi-automated laser controlled excavation and robotic shotcreting. I would contend that this was the true and fundamental definition and aim of NATM that was developed using the best technology of the day to achieve rapid ring closure and should be the basis of SCL.

Spurious focus

Unfortunately, SCL development appears to have concentrated mainly on the sprayed concrete lining itself. For whatever reason there has been a fundamental shift towards very high strength mix designs and the exclusive use of steel and/or plastic fibres. On largely spurious health and safety grounds the use of lattice girders and steel mesh has been ditched and as a result we now have SCL linings double the thickness and twice the concrete strength, totally reliant on chemical additives and quality control of the mix and application for holding it up in the roof. Consequently, in a modern SCL tunnel you are at more risk of injury from falling shotcrete than collapsing ground (that damned gravity again) and I await with trepidation the first casualty in the UK - there has already been a fatality in the USA. Let me state my heretical view that fibres and high-strength shotcrete, whilst being very beneficial from a durability point of view, are no effective structural substitute for steel reinforcement both in the temporary and permanent condition and is no quicker to install, given the increased lining thickness and excavation volume required.

I recently had sight of an SCL design of 4m diameter, 12m deep shaft in London Clay. It has a lining thickness of up to 575mm applied in four specified layers. Not only that, on top of an under-reamed and domed, heavily reinforced base is placed a 0.5m thick reinforced concrete base slab below a further 1.5m thick mass concrete plug (you can't be too careful). From the shaft bottom a 3.5m wide cable tunnel is driven, which has a 425mm SCL lining applied again in four layers. The tunnel is (for no apparent reason) ovoid in profile requiring no less than seven setting out points and, just to be sure, has an 85mm internal profile tolerance, oh, and a further 75mm allowance for a drip-shield (you really can't be too careful). Amazingly this 'heroic' design has been constructed and I expect the contractor enjoyed himself hugely spraying up the world's most expensive textured rendering job.

Finally, let me air another heretical view that London Clay and the underlying Lambeth Group is arguably some of the best tunnelling ground in the World and, rather than a soil, can often be classified as a weak rock. Perhaps it would do our armchair SCL designers some good to turn off the finite element program for an hour or two and venture underground to view it in its native habitat, then decide whether the computer output reflects reality or not.

Send your views to: editor@tunnelsandtunnelling.com *or* T&TI Editor, John Carpenter House, 7 Carmelite Street, London, EC4Y0BS, UK

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Successful Tunnelling in Challenging Mountainous Conditions

Trevor G. Carter

Dealing with adverse geology can be problematic, leading to significant tunnelling delays if not adequately foreseen. In mountainous regions, adverse conditions can prove disastrous depending on stress state, rock competence and groundwater inflows. Mitigating delays associated with bad ground at significant depth requires foresight and advanced planning.

From the tunnelling perspective, the Himalayas arguably pose the most challenging ground conditions almost anywhere in the world. One of the prime reasons is that they are the youngest of the mountain chains and are demonstrably rising faster than anywhere else. Based on their "active" stress state alone, similar-length deep tunnel excavations under the Himalayas likely will pose significantly more challenges than an equal-length, equal-cover drive almost anywhere else in the world. These difficulties of tunnelling at depth through high mountainous terrain pose major challenges not just for tunnel boring machines (TBM) but also for use of drill and blast (D&B) and New Austrian Tunneling Method (NATM) approaches.

The more challenging the ground, the greater the preplanning that is required before tunnelling. This challenge is not just one of tackling adverse ground by modifying excavation and support processes to deal with the specific problem zone, stress state and/or groundwater conditions. It is also often about logistics, as for deep tunnels in mountainous regions problem geologic zones often are at significant distance from the nearest portal and at such significant depth that surface pre-treatment is generally impractical.

Traversing faulted and disturbed ground at significant depth requires that tunnelling procedures are able to cope with a huge range of difficult geological conditions. Investigating, evaluating and assessing anticipated geology ahead of tunnelling and dealing with encountered difficult ground conditions requires that better understanding be gained of the interaction between complex geology and stress conditions when excavating at such depths.

Extremes of ground conditions present major contrasts to tunnelling, so much so that they inevitably demand use of flexible rock engineering solutions for the tunnel to progress. The fact that conditions within the Himalayas can be expected to be as bad as has ever been encountered elsewhere means there has to be the ability while tunnelling to allow changes in excavation procedures and in pre- and post-excavation support approaches. This need to adopt flexible solutions is often seen as being at variance with the constraints imposed by the rigidity of design elements incorporated into the fabrication of a typical TBM.

As a result, traditionally there has been a reluctance to use machines in these conditions, mainly due to the perceived extremely adverse consequences of entrapping or damaging the TBM. In some part this is due to the perception that there is more difficulty dealing with adverse ground conditions in the confined working area of a TBM, in comparison to dealing with the same problem in the larger working space of a D&B/NATM heading. Machine designers are attempting to combat some of these problems by making machines more robust and at the same time flexible enough to be capable of safely and successfully excavating through extremely bad ground.

Improving tunnelling effectiveness

Two issues essentially control our ability to improve tunnelling effectiveness for traversing through the characteristically complicated ground conditions found beneath the mountainous regions of the world. First is the influence of adverse geotechnics, i.e., dealing with difficult ground conditions and, second is the limitations of current tunnelling technology.

Tunnelling in adverse ground is significantly less forgiving of the limitations of the tunnelling approach than tunnelling in good ground. Generally, the more difficult the ground, the more flexibility is also needed. Tunnelling in the Himalayas, the Andes and until recently the Alps has shied away from TBM use due to perceived inflexibility and the likelihood of the machines getting trapped by adverse ground conditions, either as a result of squeezing or spalling/bursting conditions or because of ground collapses associated with rockfalls or with running or flowing ground within faults. Any of these situations can lead to problematic tunnelling at best and collapses and abandonment at worst.

Inflow of high-pressure groundwater can occur during construction of tunnels in areas with faults or tectonized zones where unusually low stress states can exist.

Dealing with such problems is always challenging but is many times worse when the tunnel heading is, say 10 km from the nearest portal, as is the case in many TBM drives. The fact that such conditions pose almost as many challenges for conventional D&B/NATM methods as for a modern machine drive often gets ignored. When similar conditions are met and encountered in D&B headings, sometimes it can take as long or longer to negotiate the problem zone than it might have taken with a properly configured, well-operated machine with an experienced, well-trained crew.

Three main geotechnical elements control our ability to execute trouble-free tunnels at significant depth — stress state, groundwater conditions and the rock itself. Adverse characteristics of any of these three elements can, on its own, compromise D&B or TBM tunnelling, but it usually takes a combination of all three being adverse to trap a machine or halt a D&B drive to the extent that a bypass becomes necessary.

Analytical approaches

Detailed analyses are usually not warranted at early project stages but may be necessary if significant segment lengths of the tunnel are of concern and a TBM is being contemplated. Most of these approaches rely on numerical modelling, and estimating appropriate parameters may be difficult, given the usually limited laboratory data at early project stages.

While vast strides have been made with numerical analyses to better understand the behavior of difficult rock masses at the two ends of the rock competency scale, application of these methods as a predictive tool rather than for back-analysis of existing or ongoing tunnelling conditions is generally not justified at this stage, unless work has been previously done on the site or on similar materials. This is because, typically, current analytical and numerical assessment capability far outweighs early project ability to properly define input parameters.

Unfortunately, it is always early in a project that decisions about TBM usage are needed. Almost always there is also inadequate definition of stress state, rock competence and groundwater for most of the tunnel, so estimating conditions in the zones geologically considered most problematic becomes the focus for minimizing risk and maximizing objectivity for decision-making.

Decisions on whether to utilize a TBM remain therefore a matter of judgment. The key issues include: How much of the tunnel length is problematic, and how much of this problematic length is of critical concern? Alone, no amount of analysis can yield the necessary answers. It requires a combination of information — yielded by the best possible geological assessment of likely conditions along a planned alignment, coupled with application of numerical and analytical techniques to back-analyze similar conditions and assess applicability. Such analyses need to be credible and done in sufficient detail that reasonable estimates can be made of critical yield extent and probable closure magnitudes. Only by such definition can difficult decisions be made on TBM applicability and the suitability of different design types.

Once some appreciation of the extent of problem sections is gained, estimates of cover and rock type can be made for typical zones within these segments and then numerical modelling can be undertaken of representative critical sections. This in turn helps to identify controlling indicators that are diagnostic for evaluation of problem conditions and aids prioritization of which parameters need to be assessed or better still measured for each anticipated problem zone along any deep tunnel route.

Reducing risk

In mountainous terrain, when considering a decision on whether or not to use a TBM, and which type of TBM to use for a deep tunnel, it must be appreciated that, historically, three types of ground conditions have proved the most problematic from the viewpoint of halting tunnel advance. In order of severity, case records suggest bad faults, heavy water and major stress, individually and/or in combination, constitute the most problematic ground conditions. These are almost irrespective of tunnelling method.

Squeezing ground conditions have been cited as the prime reason for the failure of tunnel boring machines used in certain areas of the world.

For deep mountain tunnels, with few exceptions, major disturbance zones associated with faulting have posed the most problems to tunnelling advance, often historically requiring bypass drifts and significant ground treatment before being able to be traversed. While squeezing conditions associated with the weak phyllites of the Yacambu drive in Venezuela are frequently cited as the prime reason for the failure of the TBMs used for mining along this tunnel, it should be appreciated that one of the prime reasons the phyllites encountered were so contorted and stressed is that they occur within the 2 km-wide Bocono Fault zone, one of the main plate margin faults of the Andes.

Similarly, several of the faults on the Nathpa Jhakri scheme in India, including the Sungra Fault, where extremely poor ground associated with bursting and mudrush events was encountered, when viewed on a continent-wide scale can be seen to constitute a sliver off the Main Central Thrust (MCT), which slices across the southern boundary of the Himalayas. It is therefore to be expected that stress states locally to these features, when encountered in a deep tunnel, might be anomalous, with magnitudes and directions totally at variance to conditions expected to be "normal" for that depth.

The TBM inundation experience at Parbati, which also comes into close proximity with the MCT, and the D&B drivage experience at Nathpa Jhakri with respect to the Sungra Fault are diagnostic of an extreme stress riser situation adjacent to a zone of low to non-existent stress. Tunnelling behaviour in both cases was almost identical — a zone of heavy spalling and bursting being encountered just preceding a zone of major mudflow inrush.

This points to the need to carefully look not just at the basic geotechnics of deep tunnel alignments but also at regional structural geology domains. In particular, three key geological factors need consideration over and above straightforward definition of rock mass quality, cover depth and groundwater conditions. Although these three geotechnical control indicators give an initial clue to degree of adversity, alone they do not provide the extra insight needed to assess the possible degree of adversity posed by different types of faults likely to be encountered at depth along deep mountain tunnels. The three additional factors to be considered are structural geological regime, current regional tectonic state and likely palaeostress history.

In mountain zones, understanding these factors can help route planning and alignment definition, as they provide clues to probable stress regime variability associated with specific styles of geological faulting.

Improving decision making

The lack of foresight of where adverse conditions can occur is central to many of the problems encountered in deep tunnel execution. It frequently clouds understanding to the extent that errors and unnecessary uncertainties are introduced into the decision-making process related to D&B versus TBM selection and even more so related to selection of machine type, if a machine option is favored.

Further complications in the decision-making process relate to the timing when making this key decision, as it needs to be made 12-18 months in advance of actually starting tunnelling, so that sufficient lead time is available for building the machine. However, often detailed project site investigations are incomplete, still ongoing or not even started when this key decision is to be made. Furthermore, once the contract is awarded to the contractor, generally after a long and arduous tendering process, almost always insufficient time and/or funds have been allocated to allow the contractor any opportunity for additional customized exploration to support his own excavation technology selection procedures before initiating equipment procurement.

In nearly all projects, the tender exploration data, which can be exceedingly variable in quality, is the only basis on which to make equipment selection.

Furthermore, unlike many civil geotechnical tunnel works, for subways and such like, because of the depth and length of many deep mountain tunnels in mountainous terrain, such as the Himalayas, investigating tunnel alignment is a challenge all its own. In urban areas, tunnel investigations frequently end up with boreholes on 50 meter centers or closer along the alignment. This is impractical if not cost prohibitive for deep mountain tunnels. As a result, heavy reliance needs to be placed on gaining as best as possible an appreciation of geological conditions at depth and along the alignment.

On the scale of typical project risk reduction, even for the most heavily investigated of deep tunnels, probably only 20-30% understanding of what was finally known was available at the time decisions were already to be made about D&B versus TBM and with respect to TBM type. While this possible 70% lack of understanding arguably led to many of the delays and cost overruns ultimately experienced in Himalayan tunneling, it is important to note that the extent of actual ground problem zones, given the length of the tunnels, was quite minor, affecting less than 5% of the length. And, if forewarned of such zones, their treatment would only amount to a small fraction of the total cost of the project.

Identifying the likely location and character of adverse geological structure is of paramount importance to early decision-making. The importance of good focused site investigation cannot be over-emphasized as it is upon the data acquired from early investigations that the decision must be made between D&B and TBM or about what type of TBM. It is clear that excavation of deep rock tunnels poses several unique challenges that can be daunting, but all must be addressed in the best possible manner when considering potential TBM applications.

The key risks include:

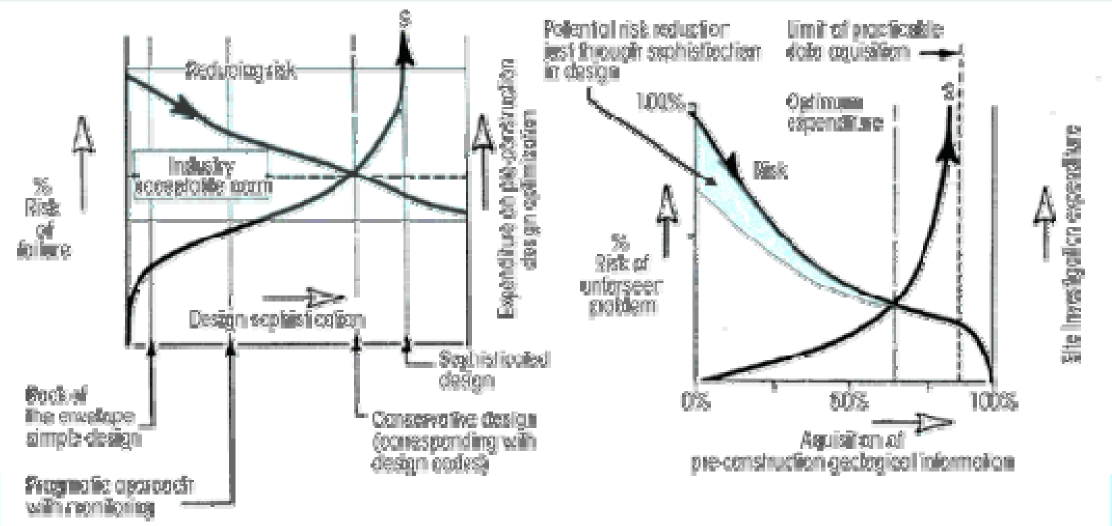
— High rock stress leading to spalling or squeezing. Predicting whether any given tunnelling situation will result in conditions that will entrap a machine requires careful evaluation of the likely alignment geology, geotechnical properties and stress regime through which the tunnel will be driven;

— High temperature. Temperature at the tunnel level depends on the geothermal gradient at the project site. For deep tunnels in young mountain belts, temperatures of the rock and groundwater frequently can exceed 40°C;

— High-pressure groundwater. While generally deep tunnels are well below the groundwater table and thus can be at high pressure, due to the depth and high *in-situ* stress, most rock fractures are tight, thus inflows are generally not a problem. The exception is faults and tectonized zones where unusually low stress states can exist. Groundwater inflows from such features can yield significant water under appreciable pressure; and

— Access and logistics. Not only does this issue affect schedule and costs, it can adversely affect tunnelling effectiveness and in rare situations even stability due to lack of rock support/segments when a crucial fault zone is encountered. Where use of a TBM is being contemplated, component sizing (for transport along tortuous mountain roads) and machine assembly must be considered, Ancillary equipment supply (for conveyors, shotcreting and grouting gear) and just the continuous maintenance of material that must be routinely supplied can be significant issues. The progress of a state-of-the-art TBM has in a number of cases been reduced severely by lack of support supplies, such as segments, mesh and rockbolts.

FIGURE 1 Approaches to Reduce Risk



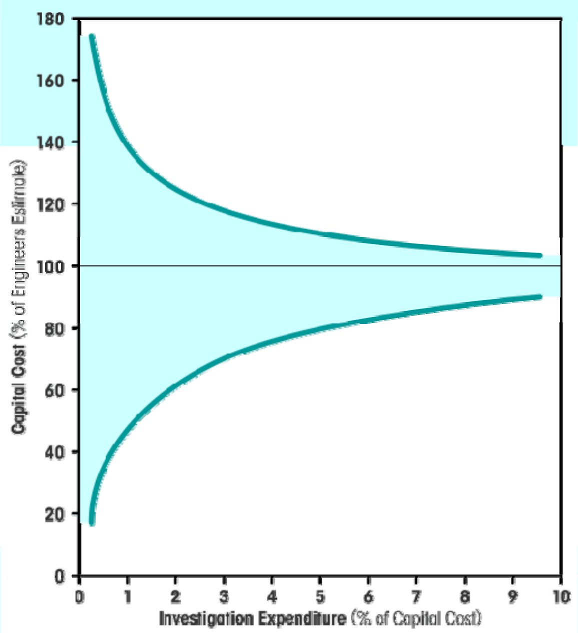
Risk reduction can only be achieved through more investigative effort, not through refinement of the design.

Many case records of difficult tunnelling through complex ground conditions in mountainous areas — particularly associated with faults and often with mud and debris flows into the tunnel — have been reported through the history of mountain tunnelling. Case records dating to the turn of the 20th century indicate the extensive use of bypass tunnels to navigate around the most difficult fault zones. For example, in July 1908 in the 14.6 km-long old Lotschberg railway tunnel (now called the apex tunnel to distinguish it from the recently completed base tunnel), during construction through faulted ground a section collapsed, killing 25. The collapsed section was reportedly beyond repair, so a bypass tunnel was driven around the site of the disaster.

Few collapses these days end with fatalities, especially where TBMs are employed, but the magnitude of today's inrushes and collapses are, nonetheless, just as dramatic, and the need for bypass tunnels remains, with several recent examples evident within both D&B and TBM-driven tunnels. One recent classic case occurred on the Gibe II project in Ethiopia. In this case, significant multiple mud-flow events occurred, initially burying a state-of-the-art Universal Double Shield machine until it could be extricated and refurbished.

The scope of the problem when it was encountered was completely <. More than 30 long exploratory drillholes and significant exploratory and bypass drifting was completed to develop a scheme that could successfully advance the TBM though this difficult zone. Arguably this again poses the same typical question — could the problems have been foreseen using regular probe drilling and, if so, what could have been done differently?

FIGURE 2 **Expenditures on Investigation**



For complex ground conditions, about 5% of the engineer's estimate of capital expenditure should be spent on investigating ground conditions.

In more difficult ground conditions, such as those encountered in the Himalayas, with minimal investigation comes the wider risk of the TBM getting trapped by adverse ground conditions — either as a result of squeezing or spalling/bursting conditions or because of ground collapses associated with rockfalls or with running or flowing ground within faults, always in these cases complicated by heavy water inflows. To reduce these risks to an acceptable level, considerably more investment must be made in the hydropower design process in these complex mountainous regions. Significant reduction of real risk can only be achieved through more investigative effort, not through design refinement (see Figure 1). Cost and schedule analysis of past case records suggests that for complex ground conditions, some 5% of the engineer's estimate of capital expenditure is required to be expended on investigating ground conditions to push the process in the right direction (see Figure 2).

Dr. Trevor Carter is principal - Rock Engineerin g Division of Golder Associates in Toronto, Ontario, Canada.

http://www.hydroworld.com/index/display/article-display/7984412578/articles/hydro-review-worldwide/vol-19/issue-3/articles/construction/successful-tunnelling-in-challenging.html

ΠΡΟΣΚΛΗΣΗ ΥΠΟΒΟΛΗΣ ΥΠΟΨΗΦΙΟΤΗΤΩΝ ΓΙΑ ΤΗΝ ΣΥΜΜΕΤΟΧΗ ΣΤΟ 5iYGEC’13

Dear Colleagues of Member Societies,

Following on my invitation letter of March 9, 2012, the financial situation has been improved for the organization of our Fifth International Young Geotechnical Engineers’ Conference (5iYGEC’13). This allows the organizing committee to modify the previous offer for both the nominated (two per Member Society) and non-nominated participants, as follows:

(1) Nominated participants

We will be able to provide (in addition to participation at the Conference):

- lunch during the conference time
- banquet
- two-day travel pass in Paris
- two-day participation at the main Conference Paris 2013
- hotel for five days (from 30 August to 4 September 2013)

The corresponding registration fee is 320 Euros.

The registration fee for participants organising their own accommodation will be 200 Euros.

(2) Non-nominated participants

We will be able to provide (in addition to participation at the Conference):

- lunch during the conference time
- banquet
- two-day travel pass in Paris

The corresponding registration fee is 320 Euros.

In addition, for a better organization of our 5iYGEC’13, the organizing committee has decided to slightly modify the deadlines for papers, as follows:

- Abstract: **November 30, 2012**
- Full-length paper: **February 28, 2013**
- Notification of acceptance: **April 30, 2013**

All submission will be done through the conference website:

http://www.lepublicsystemepco.com/events.php?IDManif=696

Sincerely yours,

Prof. Yu-Jun Cui, September 21, 2012
Chair

Με βάση τα παραπάνω καλούνται οι ενδιαφερόμενοι να υποβάλουν στην γραμματεία της ΕΕΕΕΓΜ (secretariat@hssmge.gr) μέχρι την 15^{η} Νοεμβρίου 2012 περιλήψεις των άρθρων τους. Η Εκτελεστική Επιτροπή της ΕΕΕΕΓΜ θα καλέση την αρμόδια επιτροπή κρίσης των περιλήψεων να επιλέξη τις επικρατέστερες τέσσερις (4), από τις οποίες θα επιλεγούν οι δύο (2) που θα αποτελούν την επίσημη εκπροσώπηση της ΕΕΕΕΓΜ.

Η υποβολή των περιλήψεων στην ιστοσελίδα του συνεδρίου **θα γίνη μόνον μέσω της ΕΕΕΕΓΜ**.

ΘΕΣΕΙΣ ΕΡΓΑΣΙΑΣ ΓΙΑ ΓΕΩΜΗΧΑΝΙΚΟΥΣ

Amida

Senior Tunnel Engineer – Singapore ($80-100k SGD)

A major new Cable Tunnel project is underway in Singapore with designers sought to work with a major MNC here as the works get going.

Key Responsibilities:

a. to familiarise with BTS requirements

b. design of segment lining design : sacrificial and permanent

c. designing, coordination, design support of tunnel cross passage using mining methods.

d. coordination and preparation of design of ground treatment in consultation with contractor

e. underpinning of structures due to tunnelling works

f. coordination of tunnelling works submission

g. instrumentation and damage assessment due to tunnelling works

h. attending meetings and drafting replies to contractors request

i. adhoc site visit to trouble shoot site issues.

Technical:

- Relevant tertiary qualifications in Civil Engineering
- Chartered qualified preferred
- Circa 10 years of relevant experience in the design of tunnel projects (Singapore experience would be a distinct advantage)
- Track record in tunnel design and up to date experience of tunnel projects and procedures

Key Competencies:

Knowledge, Experience & Skills

- Ability to consult with others to develop the level of knowledge required for the role
- Identifies own training and development needs, ways to meet these needs and shares knowledge with others

Contact:

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Amida Global Pte. Ltd.

ΑΝΑΣΚΟΠΗΣΗ ΓΕΓΟΝΟΤΩΝ ΓΕΩΤΕΧΝΙΚΟΥ ΕΝΔΙΑΦΕΡΟΝΤΟΣ

Shaking the Foundations of Geo-engineering Education

Διεθνές συνέδριο για την Εκπαίδευση του Γεωτεχνικού Μηχανικού

4-6 Ιουλίου 2012, NUI Galway, Galway, Ireland

Ολοκληρώθηκαν με επιτυχία οι εργασίες του διεθνούς συνεδρίου για την Εκπαίδευση του Γεωτεχνικού Μηχανικού "Shaking the Foundations of Geoengineering Education", που έγινε στην Γκόλγουει (Galway) της Ιρλανδίας στις αρχές Ιουλίου 2012. Είχαν προηγηθεί άλλα δύο διεθνή συνέδρια με παρεμφερή θεματολογία, το 2000 στην Σινάια της Ρουμανίας και το 2008 στην Κωνστάντζα, πάλι στην Ρουμανία. Το τρίτο διεθνές συνέδριο για πρώτη φορά συνδύασε στη θεματολογία του καλές πρακτικές στην εκπαίδευση του γεωτεχνικού μηχανικού με ερευνητικά αποτελέσματα σε θέματα διδασκαλίας και μάθησης (teaching and learning). Τα συμπεράσματα από το συνέδριο θα συμπεριληφθούν στην τελική έκθεση για την τετραετία 2009-2013 της Τεχνικής Επιτροπής για την Εκπαίδευση της ISSMGE, TC306, τα κύρια μέλη της οποίας απετέλεσαν την οργανωτική επιτροπή του συνεδρίου. Τα πρακτικά του συνεδρίου έχουν κυκλοφορήσει από τον εκδοτικό οίκο CRC Press (McCabe, B., M. Pantazidou and D. Phillips (Eds.), 2012, Shaking the Foundations of Geoengineering Education, ISBN 978-0-415-62127-4 και σε e-book: ISBN 978-0-203-08306-2), και θα είναι ελευθέρως διαθέσιμα στο διαδίκτυο μετά το 2014. Ανάμεσα στις δημοσιεύσεις των πρακτικών, ξεχωρίζουν οι προσκεκλημένες ομιλίες των Brian Simpson (Equilibrium, strength, strain and superposition) και Richard Felder (Engineering education: A tale of two paradigms), που έδωσαν την οπτική γωνία της πράξης και της έρευνας στην εκπαίδευση του μηχανικού, αντίστοιχα. Την Ελλάδα εκπροσώπησαν στο συνέδριο οι Μαρίνα Πανταζίδου, ΕΜΠ (μέλος της οργανωτικής επιτροπής), Κατερίνα Χαραλαμπίδου, ΟΤΜ ΑΕ και Γιώργος Αναγνωστόπουλος, ΟΤΜ, ΑΕ.

Οι συμμετέχοντες στο συνέδριο "Shaking the Foundations of Geoengineering Education", Galway, Ιρλανδία, 4 Ιουλίου 2012.

Μαρίνα Πανταζίδου

ΠΡΟΣΕΧΕΙΣ ΕΚΔΗΛΩΣΕΙΣ ΓΕΩΤΕΧΝΙΚΟΥ ΕΝΔΙΑΦΕΡΟΝΤΟΣ ΣΤΗΝ ΕΛΛΑΔΑ

ΕΛΛΗΝΙΚΗ ΕΠΙΤΡΟΠΗ ΜΕΓΑΛΩΝ ΦΡΑΓΜΑΤΩΝ
ΑΞΟΝΟΣΥΜΜΕΤΡΙΚΑ ΦΡΑΓΜΑΤΑ ΣΚΛΗΡΟΥ ΕΠΙΧΩΜΑΤΟΣ

6 Νοεμβρίου 2012, ΕΜΠ

Η ΕΕΜΦ σας προσκαλεί στην Ημερίδα που διοργανώνει με θέμα **«ΑΞΟΝΟΣΥΜΜΕΤΡΙΚΑ ΦΡΑΓΜΑΤΑ ΣΚΛΗΡΟΥ Ε-ΠΙΧΩΜΑΤΟΣ»**, η οποία θα λάβει χώρα στην αίθουσα τελετών του Κτιρίου Διοίκησης του ΕΜΠ, την Τρίτη 6 Νοεμβρίου 2012, από τις 5μμ έως τις 8.30 μ.μ.

Στόχος της Ημερίδας είναι να παρουσιασθούν οι βασικές αρχές σχεδιασμού των αξονομετρικών φραγμάτων σκληρού επιχώματος καθώς και εμπειρίες από την κατασκευή τους.

Η είσοδος στην Ημερίδα είναι ελεύθερη και η συμμετοχή σας κρίνεται ιδιαίτερα σημαντική και αμοιβαία χρήσιμη, αφού θα έχετε την ευκαιρία να ενημερωθείτε αλλά και να εκφράσετε τις παρατηρήσεις και απόψεις σας.

Ο Πρόεδρος
Γιώργος Ντουνιάς

Η Γενική Γραμματέας
Χαρά - Ρέα Παπαχατζάκη

ΠΡΟΓΡΑΜΜΑ

ΧΡΟΝΟΣ	ΟΜΙΛΗΤΗΣ	ΘΕΜΑ
17:00 – 17:10	**Γιώργος Ντουνιάς** *Πρόεδρος ΕΕΜΦ*	Καλωσόρισμα
17:10 – 17:40	**Νίκος Μουτάφης** *Λέκτορας ΕΜΠ – Τεχνικός Σύμβουλος*	Αξονομετρικά Φράγματα σκληρού επιχώματος – Γενική παρουσίαση - αρχές σχεδιασμού
17:40 – 18:20	**Γιάννης Δαούτης** **Βαγγέλης Γκίκας** *Μελετητές Υδραυλικών Έργων*	Η μελέτη των αξονομετρικών φραγμάτων σκληρού επιχώματος και οι εμπειρίες από την εφαρμογή στα φράγματα του Ελληνικού χώρου.
18:20 – 18:35	**Οδυσσέας Καρασαχινίδης** *Προϊστάμενος Κατασκευών Εγγειοβελτιωτικών Έργων (ΥΠΑΑΤ)*	Αξονομετρικά Φράγματα σκληρού επιχώματος – Το πρόγραμμα του ΥΠΑΑΤ
18:35 – 18:50	**Marco Bacchielli** *Carpi*	Εναλλακτική μέθοδος στεγανοποίησης αξονομετρικών φραγμάτων σκληρού επιχώματος με χρήση μεμβράνης
18:50 – 19:10	**Διάλειμμα**	
19:10 – 19:25	**Δημήτρης Κούμουλος** *Τεχνικός Σύμβουλος Ποιοτικού Ελέγχου*	Εμπειρίες από την κατασκευή των φραγμάτων της Μυκόνου Μαράθι και Άνω Μερά
19:25 – 19:40	**Στέλιος Φελέκος** *Τεχνικός Σύμβουλος Ποιοτικού Ελέγχου*	Εμπειρίες από την κατασκευή των φραγμάτων Στενού Σερίφου και Λιθαίου Τρικάλων
19:40 – 19:55	**Ευάγγελος Μαμαγκάκης** *Επιβλέπων Κατασκευής* **Μιχ. Στεφαδούρος** *Τεχνικός Σύμβουλος Ποιοτικού Ελέγχου* **Γεώργιος Χαραράς** *Εργοταξιάρχης*	Εμπειρίες από την κατασκευή του φράγματος Βαλσαμιώτη, Κρήτης
19:55 – 20:10	**Παναγιώτης Αναγνωστόπουλος** *Εργοταξιάρχης* **Χαράλαμπος Γκούβας** *Τεχνικός Σύμβουλος Ποιοτικού Ελέγχου*	Εμπειρίες από την κατασκευή του φράγματος Φιλιατρινού, Πελοπόννησος
20:10 – 20:30	**Συζήτηση**	

ΠΡΟΣΚΛΗΣΗ ΣΤΗΝ ΗΜΕΡΙΔΑ ΝΕΩΝ ΕΡΕΥΝΗΤΩΝ ΤΟΥ Ε.Τ.Α.Μ.

Θεσσαλονίκη, 7 Δεκεμβρίου 2012
www.etam.gr

Αξιότιμα Μέλη/ Φίλοι του ΕΤΑΜ,

Το Ελληνικό Τμήμα Αντισεισμικής Μηχανικής (Ε.Τ.Α.Μ.) διοργανώνει, υπό την αιγίδα του Τμήματος Πολιτικών Μηχανικών της Πολυτεχνικής Σχολής του Αριστοτελείου Πανεπιστημίου Θεσσαλονίκης, ημερίδα που απευθύνεται αποκλειστικά και μόνον σε νέους ερευνητές, με τίτλο:

Η ΑΝΤΙΣΕΙΣΜΙΚΗ ΜΗΧΑΝΙΚΗ ΜΕΣΑ ΑΠΟ ΤΗΝ ΕΠΙΣΤΗΜΟΝΙΚΗ ΜΑΤΙΑ ΝΕΩΝ ΕΡΕΥΝΗΤΩΝ ΚΑΙ ΜΗΧΑΝΙΚΩΝ

Σκοπός της ημερίδας είναι η ανάδειξη και η διάχυση του έργου της νέας γενιάς ερευνητών και μηχανικών (μελών ή μη του ΕΤΑΜ), η γνωριμία καθώς και η ανάπτυξη σχέσεων συνεργασίας μεταξύ τους. Η ημερίδα απευθύνεται σε σπουδαστές (μεταπτυχιακούς, υποψήφιους διδάκτορες), σε διδάκτορες (έως και 6 χρόνια μετά την απόκτηση του τίτλου), καθώς και σε ερευνητές ή επαγγελματίες μηχανικούς κάτω των 38 ετών.

Η ημερίδα θα πραγματοποιηθεί στην Θεσσαλονίκη την 7η Δεκεμβρίου 2012 στο Κέντρο Διάδοσης Ερευνητικών Αποτελεσμάτων (ΚΕ.Δ.Ε.Α) του Αριστοτελείου Πανεπιστημίου Θεσσαλονίκης.

Λεπτομέρειες για την ημερίδα παρατίθενται στην ιστοσελίδα του ΕΤΑΜ και στο τεύχος 45, Μάιος 2012 των «ΝΕΩΝ ΤΗΣ ΕΕΕΕΓΜ».

☙ ❧

ΠΑΝΕΛΛΗΝΙΟ ΣΥΝΕΔΡΙΟ
ΙΣΤΟΡΙΑΣ ΤΩΝ ΕΠΙΣΤΗΜΩΝ ΚΑΙ ΤΗΣ ΤΕΧΝΟΛΟΓΙΑΣ

ΠΡΟΣΚΛΗΣΗ ΕΚΔΗΛΩΣΗΣ ΕΝΔΙΑΦΕΡΟΝΤΟΣ

Η Εταιρεία Μελέτης και Διάδοσης της Ιστορίας των Επιστημών και της Τεχνολογίας (ΕΜΔΙΕΤ) και το Τμήμα Μεθοδολογίας, Ιστορίας και Θεωρίας της Επιστήμης (ΜΙΘΕ) του Πανεπιστημίου Αθηνών διοργανώνουν Πανελλήνιο Συνέδριο Ιστορίας των Επιστημών και της Τεχνολογίας στις 28, 29 και 30 Μαρτίου 2013 στην Αθήνα.

Μετά το πρώτο επιτυχημένο συνέδριο νέων ερευνητών το 2011 στην Αθήνα, η ΕΜΔΙΕΤ σε συνεργασία με το ΜΙΘΕ αποφάσισε να καθιερώσει τη διοργάνωση συνεδρίου κάθε δύο χρόνια, αποσκοπώντας στην καλλιέργεια συνθηκών που θα φέρουν σε επαφή μεταξύ τους Έλληνες ερευνητές από τον χώρο της ιστορίας των επιστημών και της τεχνολογίας, καθώς και συγγενών κλάδων, που εργάζονται στην Ελλάδα ή το εξωτερικό.

Η κεντρική θεματική του φετινού συνεδρίου είναι «Ευρώπη - Επιστήμη - Τεχνολογία».

Όσοι ενδιαφέρονται καλούνται να υποβάλουν σχετική πρόταση για ανακοίνωση. Προτάσεις που δεν εμπίπτουν στη θεματική αυτή μπορούν επίσης να υποβληθούν. Σημειώνεται ότι ενθαρρύνονται ιδιαίτερα οι προτάσεις για την οργάνωση συνεδριών· για όλες προβλέπεται η ύπαρξη σχολιαστή. Τέλος, προγραμματίζεται η δημοσίευση των ανακοινώσεων μετά από διαδικασία αξιολόγησης.

Οι ενδιαφερόμενοι μπορούν να υποβάλουν τις προτάσεις τους ηλεκτρονικά με τα επισυναπτόμενα δελτία μέχρι τις 30 Οκτωβρίου 2012 στη διεύθυνση info@emdiet.gr.

☙ ❧

2ο ΠΑΝΕΛΛΗΝΙΟ ΣΥΝΕΔΡΙΟ ΦΡΑΓΜΑΤΩΝ ΚΑΙ ΤΑΜΙΕΥΤΗΡΩΝ
Σχεδιασμός – Διαχείριση – Περιβάλλον
Αθήνα, 6 - 8 Νοεμβρίου 2013
www.eemf.gr

Η Ελληνική Επιτροπή Μεγάλων Φραγμάτων (ΕΕΜΦ) διοργανώνει το **2ο Πανελλήνιο Συνέδριο Φραγμάτων και Ταμιευτήρων στις 6, 7 & 8 Νοεμβρίου του 2013 στην Αθήνα.**

Η απαίτηση για ορθολογική διαχείριση του υδατικού δυναμικού είναι μεγαλύτερη παρά ποτέ. Στις αυξανόμενες ανάγκες για ύδρευση, άρδευση, ενέργεια και πλημμυρική προστασία προστίθεται ολοένα και πιο επιτακτικά η ανάγκη για προστασία και επανατροφοδότηση των υπόγειων υδροφορέων και η αναβάθμιση και προστασία των ποτάμιων και λιμναίων οικοσυστημάτων.

Ο ρόλος των φραγμάτων και ταμιευτήρων είναι κομβικός για την αντιμετώπιση των ανωτέρω. Η χώρα μας, αν και καθυστέρησε σημαντικά στην εκμετάλλευση του υδάτινου δυναμικού, έχει κατασκευάσει τις τελευταίες μεγάλο αριθμό φραγμάτων, και ταμιευτήρων ενώ ένας μεγάλος αριθμός νέων έργων είναι τώρα σε φάση μελέτης ή υλοποίησης.

Τα φράγματα και οι ταμιευτήρες είναι πολύπλοκα έργα με πολλές συνιστώσες που δημιουργούν αυξημένες απαιτήσεις κατά το σχεδιασμό, την υλοποίηση και τη λειτουργία τους. Ο σεβασμός στο περιβάλλον, η ολοκληρωμένη διαχείριση των υδατικών πόρων, η χρήση νέων τεχνολογιών, η μακροχρόνια συμπεριφορά και ασφάλεια, η ευθύνη του κυρίου του έργου ή του διαχειριστή για την ασφαλή λειτουργία των έργων, είναι θέματα στα οποία θα επικεντρωθούν οι εργασίες του συνεδρίου.

Σας καλούμε να συμμετάσχετε στο 2ο πανελλήνιο συνέδριο φραγμάτων και ταμιευτήρων και να αποστείλετε εργασίες σχετικές με το θεματολόγιο που παρουσιάζεται παρακάτω.

Θεματολόγιο

1. Φράγματα, Ταμιευτήρες και Περιβάλλον

- Φιλικές προς το περιβάλλον κατασκευές φραγμάτων και ταμιευτήρων
- Κοινωνικά αποδεκτός σχεδιασμός φραγμάτων και ταμιευτήρων
- Περιβαλλοντικοί όροι, επιπτώσεις και κοινωνικά οφέλη
- Περιορισμός υδρομορφολογικών αλλοιώσεων και αισθητική αποκατάσταση περιβάλλοντος
- Εμπλουτισμός – αποκατάσταση υπόγειων υδροφορέων, δημιουργία υγροβιότοπων αντιπλημμυρική προστασία κ.λπ.
- Διατήρηση και βελτίωση ποιότητας υδατικών πόρων
- Φερτές ύλες

2. Φράγματα και ολοκληρωμένη διαχείριση υδατικών πόρων

- Διαχείριση υδατικών πόρων σε επίπεδο λεκάνης απορροής
- Οι ταμιευτήρες ως έργα διαχείρισης υδατικών πόρων πολλαπλού σκοπού
- Τεχνικο-οικονομικά κριτήρια υλοποίησης νέων φραγμάτων
- Ο ρόλος των φραγμάτων στον ενεργειακό σχεδιασμό - Σύγχρονες τάσεις και τεχνολογικές εξελίξεις
- Ταμιευτήρες και υβριδικά συστήματα παραγωγής ενέργειας

3. Ασφάλεια φραγμάτων και ταμιευτήρων

- Κανονισμοί μελέτης, κατασκευής και λειτουργίας φραγμάτων
- Η πρόταση της ΕΕΜΦ για την σύνταξη εθνικού κανονισμού ασφάλειας φραγμάτων.
- Κίνδυνοι σχετιζόμενοι με προβλήματα οργάνωσης του κυρίου - διαχειριστή του έργου
- Απαιτήσεις παρακολούθησης συμπεριφοράς
- Αναλύσεις θραύσης φράγματος και επιπτώσεις
- Μακροχρόνια συμπεριφορά και κίνδυνοι οφειλόμενοι στη γήρανση των φραγμάτων
- Κίνδυνοι οφειλόμενοι σε αστοχίες Η/Μ εξοπλισμού
- Παρουσίαση πρόσφατων συμβάντων ή περιστατικών
- Φράγματα , ταμιευτήρες και δημόσια ασφάλεια
- Ασφαλής παροχέτευση εκτάκτων πλημμυρικών παροχών κατάντη – απαιτήσεις οριοθέτησης της κοίτης

4. Εξελίξεις στις μεθόδους σχεδιασμού & κατασκευής

- Υλικά κατασκευής Φραγμάτων, μέθοδοι κατασκευής, νέες τεχνικές

- Εκτίμηση, επιλογή και αναθεώρηση πλημμυρών σχεδιασμού
- Σχεδιασμός υπερχειλιστών – αναβάθμιση υπερχειλιστών
- Η επιρροή των Γεωλογικών συνθηκών στο σχεδιασμό.
- Εξελίξεις στο Γεωτεχνικό σχεδιασμό
- Εξελίξεις στον αντισεισμικό σχεδιασμό
- Εξελίξεις στον Η/Μ εξοπλισμό

5. Παρουσίαση έργων

Κρίσιμες ημερομηνίες για την αποστολή εργασιών:

- Υποβολή περιλήψεων: **15 Οκτωβρίου 2012**
- Αποδοχή περιλήψεων: **30 Νοεμβρίου 2012**
- Υποβολή πλήρους κειμένου: **30 Απριλίου 2013**
- Αποδοχή πλήρους κειμένου: **30 Ιουνίου 2013**

Οδηγίες για την αποστολή των περιλήψεων θα βρείτε στη ιστοσελίδα της ΕΕΜΦ www.eemf.gr.

Οι περιλήψεις θα αποστέλλονται ηλεκτρονικά στην διεύθυνση της ΕΕΜΦ eemf@eemf.gr.

ΠΡΟΣΕΧΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΚΔΗΛΩΣΕΙΣ

Για τις παλαιότερες καταχωρήσεις περισσότερες πληροφορίες μπορούν να αναζητηθούν στα προηγούμενα τεύχη του «περιοδικού» και στις παρατιθέμενες ιστοσελίδες.

VOLSAM 2012 - Volcanism of the Southern Aegean in the frame of the broader Mediterranean area, 10-12 October 2012, Santorini island, Greece, http://volsam2012.conferences.gr

61. Geomechanics Colloquy "50 Years NATM", October 11th and 12th, 2012, Salzburg, Austria, salzburg@oegg.at

The 4th International Forum on Opto-electronic Sensor-based Monitoring in Geo-engineering (4th OSMG-2012)
October 11-13, 2012, Nanjing University High-tech Institute at Suzhou, China
http://osmg2012.acei.cn

With the rapid development in the construction of various infrastructure engineering in China, the construction quality of these structures and their safety operation are the main concerns. To ensure this and prevent various potential disasters, the improvement of the level of engineering monitoring technology is a necessity. Recently, the distributed monitoring technologies, such as Brillouin Optical Time Domain Reflectometer (BOTDR), Brillouin Optical Time Domain Analysis (BOTDA), Raman Optical Time Domain Reflectometer (ROTDR), and Fiber Bragg Grating (FBG), have been successively applied to geo-engineering monitoring. Due to their apparent advantages, the development and application of distributed monitoring technologies have become a research focus and a key research topic in some developed countries.

As the fourth forum following the 1st, 2nd and 3rd International Forums on Opto-electronic Sensor-based Monitoring in Geo-engineering held by Nanjing University, China, in 2005, 2007 and 2010, respectively, this forum will focus on the subject of distributed monitoring technologies and engineering applications, about which related topics will be discussed and communicated. During the forum, some well-known foreign and domestic scholars and experts will be invited to give keynote lectures on up-to-date research findings, hotspots and difficult subjects in the geo-engineering monitoring field. An information release and communication platform for new technologies and products of relevant suppliers will be provided, as well.

MAIN TOPICS

- Recent development of distributed monitoring technologies
- Demodulation technologies for distributed opto-electronic sensing network
- Data acquisition and wireless transmission technologies for distributed opto-electronic sensing network
- Temperature compensation and abnormality recognition for distributed opto-electronic sensing network
- Distributed monitoring technologies of geo-materials with large deformation
- Development of special distributed sensing fibers and cables
- Designing, installation and protection of distributed sensing network in geo-engineering monitoring field
- Safety monitoring and diagnosis system based on distributed monitoring technologies
- Integration of distributed opto-electronic sensing network in geo-engineering monitoring field
- Case study of engineering applications of distributed monitoring technologies
- Key technologies of temperature monitoring using ROTDR
- Technical solutions of field installation and wireless data transmission of FBG sensors

CONTACT DETAILS

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Email: osmg2012@nju.edu.cn
Tel: +86-25-83597888 +86-25-83596220
Fax: +86-25-83596220

2nd International Symposium on Constitutive Modeling of Geomaterials: Advances and New Applications (IS-Model 2012), October 15 and 16, 2012 Beijing, China, www.csrme.com/ISMODEL/index.html

SAHC 2011, 8th International Conference on Structural Analysis of Historical Constructions, October 15 – 17, 2012, Wroclaw, Poland, www.sahc2012.org

7th Asian Rock Mechanics Symposium, 15-19 October 2012, Seoul, Korea, www.arms7.com

37th Annual Conference on Deep Foundations, October 16-19, 2012, Houston, TX, USA, www.dfi.org/conferencedetail.asp?id=193

10th International Congress on Advances in Civil Engineering, 17-19 October 2012, 17-19 October, Ankara, Turkey www.ace2012.metu.edu.tr

Montreal TAC 2012 - Tunnels and Underground Spaces: Sustainability and Innovations, 17 - 20 October 2012, Montreal, Canada, www.tac2012.ca

6th International Symposium on Roller Compacted Concrete (RCC) Dams October 23 to 25, 2012, Zaragoza, Spain, www.meetandforum.net/RCC2012

HYDRO 2012 Innovative Approaches to Global Challenges, 29 to 31 October 2012, Bilbao, Spain, www.hydropower-dams.com

International Conference on Ground Improvement and Ground Control: Transport Infrastructure Development and Natural Hazards Mitigation, 29 Oct - 2 Nov 2012, Wollongong, Australia www.icgiwollongong.com

Tangible Risks, Intangible Opportunities: Long-Term Risk Preparedness and Responses for Threats to Cultural Heritage – 2012 Theme: Reducing Risks to Cultural Heritage from Natural and Human-Caused Disasters, 31 October 2012, Beijing, China, pamela.jerome@icomos.org)

IV Panamerican Landslides Symposium, 31 October – 2 November 2012, Boyacá, Colombia, www.scg.org.co/web%20IVSPD/img/IV-SPD-BOLETIN-JULIO.pdf

6th Congress on Forensic Engineering, October 31 – November 3, 2012, San Francisco, USA http://content.asce.org/conferences/forensics2012/index.html

ACUUS 2012 13th World Conference of the Associated Research Centers for the Urban Underground Space Underground Space Development – Opportunities and Challenges, 7 – 9 November 2012, Singapore, www.acuus2012.com

International Symposium on Earthquake-induced Landslides November 7-9, 2012, Kiryu, Japan http://geotech.ce.gunma-u.ac.jp/~isel/index.html

GEOMAT2012-KL, MALAYSIA Second International Conference on Geotechnique, Construction Materials and Environment, November 14-16, 2012, Kuala Lumpur, Malaysia, http://geomat2012.webs.com

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http://www.middleeasttunnelling.com/homepage.asp

The 2nd Annual **MEED** event is focussed on opportunities, developments and challenges for tunnelling projects in the Middle East – though the event will include speakers based outside of the region. Due to the nature of the industry of a number of the companies with particular expertise are from outside of the Middle East – though because of the huge volume of work being undertaken in places like Qatar, Saudi Arabia and the UAE the region is arguably the most attractive projects market in the world.

The event will focus on these main themes:

- Scene setting overviews (data heavy) of global tunnelling industries and market value in the Middle East
- Project reports from key Middle East Tunnelling projects (Qatar Metro, Doha Bay Crossing, Inner Doha Re-Sewerage Integrated System IDRIS, Riyadh Metro, Cairo Metro, Abu Dhabi Strategic Tunneling Enhancement Program STEP)
- Examining specific challenges and lessons learnt from different tunnelling techniques in the region (cut cover, immersed tube, boring, microtunneling)
- Geotechnical challenges attached to tunnelling projects in the Middle East (Groundwater, Ground topography, materials / equipment)
- Effective risk management strategies for tunnelling projects in the Middle East (contracting)
- Best practice for on-going maintenance of long tunnelling projects (degradation, corrosion, technologies for extending life / lower costs, ventilation, drainage systems etc)

For further information please contact Jon Connell, Project Leader, Tel: +971 4 3755010 Email: jon.connell@meed.com

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20 November 2012, London, UK
www.ncetunnelling.co.uk

New Civil Engineer and *Ground Engineering* are proud to announce details for their leading **Annual Tunnelling conference**. An essential one-day event showcasing the latest innovative technology and project opportunities in the tunnelling sector.

- Tunnelling sector forecasting which includes such key projects as **Fehmarnbelt, Thames Tideway and HS2.**
- In-depth information and operational experiences from two major UK tunnelling projects: **Crossrail and Lee Tunnel**
- About the successful application of the new and innovative **URUP (Ultra Rapid Under Pass)** shield tunnelling technology in Japan
- Debate whether **SCL tunnel systems** are better delivered exclusively through Design and Build rather then through Engineer's design
- How the **Monitoring and Tunnel Boring Machine** experts are responding to the ever complex demands of the tunnelling industry – what are the increased capabilities and the latest developments?
- **...and many more...**

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32. Baugrundtagung with exhibition "Geotechnik", Mainz, Germany, 26 – 29 November 2012, www.baugrundtagung.com

GEOSYNTHETICS ASIA 2012 (GA2012) 5th Asian Regional Conference on Geosynthetics, Bangkok, Thailand, 10 - 14 December 2012, www.set.ait.ac.th/acsig/igs-thailand

First International Congress FedIGS, 12 – 15 November 2012, Hong Kong – China, www.fedigs.org/HongKong2012

2012 Forum on Urban Geoenvironment & Sustainable Development, 4-7 December 2012, Hong Kong, CHINA, www.civil.hku.hk/ugsd2012/en/

GA2012 - Geosynthetics Asia 2012 5th Asian Regional Conference on Geosynthetics, 13 - 16 December 2012, Bangkok, Thailand, www.set.ait.ac.th/acsig/GA2012

Forensic geotechnical engineering
www.editorialmanager.com/feng

Fourth International Seminar on FORENSIC GEOTECHNICAL ENGINEERING, January, 10-12, 2013, Bengaluru, India, Prof. G L Sivakumar Babu, isfge2013@gmail.com

Geotechnical Special Publication, ASCE "Foundation Engineering in the Face of Uncertainty". Abstracts to Mohamad H. Hussein at: MHussein@pile.com.

Geotechnical Special Publication, ASCE "SOUND GEOTECHNICAL RESEARCH TO PRACTICE", http://web.engr.oregonstate.edu/~armin/index_files/Holtz GSP

Themed Issue on Geotechnical Challenges for Renewable Energy Developments, Geotechnical Engineering 2013, ben.ramster@icepublishing.com

Pam-Am UNSAT 2013 First Pan-American Conference on Unsaturated Soils, 19-22 February 2013, Cartagena de Indias, Colombia, panamunsat2013.uniandes.edu.co

ICGE'13 3rd International Conference on Geotechnical Engineering New Developments in Analysis, Modeling, and Design, 21-23 February 2013, Hammamet, Tunisia www.icge13.com

International Conference on«Landslide Risk»
ICLR13
14 – 16 March 2013, Draham, Tunisia
http://www.iclr13.com/#HOME_PAGE.A

Characterization of soil and rock slopes has improved considerably over recent years with the continuous development of prediction methods, monitoring and mitigation techniques. However, seismic effects and dramatic climatic changes, in part due to industrial human activities, make the risk of landslides and their impact on the economical and social activities a serious problem. Also, the population concentration in risky zones increases the potential of critical consequences.

For these reasons important attention is given across the word to landslide risk. Practical and research issues are investigated, which range from prediction and modeling to mitigation and efficient monitoring techniques.

In Tunisia landslides are present in numerous regions. They strongly affect the traffic in many national roads. Recently, on December 2011, in the northwest of Tunisia (Ain-Draham, Beja, Tabarka, Mateur…) large-scale slope failures, triggered by the rainfall, developed suddenly (around 210 cases). Many slopes collapsed catastrophically without prior signs of warning. The mitigation of slope instability and its associated damage remains an essential question for the geotechnical profession in Tunisia. At present, few investigations have been engaged to study the landslide initiation and propagation or to study the rainfall infiltration. Specific attention should be given to the "in situ" hydraulic and environmental conditions, namely the rainfall intensity and evaporation flux as well as its cyclic nature.

The International Conference will promote the debate of relevant issues among engineers and researchers in soil and rock mechanics and in environmental fields. An important challenge for scientists is not only to advance in the knowledge on technical matters but also to promote the teaching of these aspects in a wide sense and to increase the awareness of public and local authorities, as well as civil protection institutions, with regards to landslide risks.

OBJECTIVE

A main objective is to identify emerging developments in the analysis, mitigation and monitoring technologies capable of generating successful methodologies to deal with landslide risk and to contribute to the development of international standards.

The Conference will hopefully provide engineers and decision-makers with improved procedures for the quantitative assessment of Landslide risk.

Special focus is given to:

1) Climatic change effects on slope behavior and the effects on the human activities.

2) Efficient prediction and modeling and related issues.

3) Optimization of tools to monitor and mitigate landslides in different environments, especially in mountainous zones.

The envisaged technical program will be developed around three Workshop Sessions.

A) First session: Field Characterization and Mapping

- Advances in field surveying and mapping technologies
- Geomorphological survey
- Relevant soil properties. Field and laboratory characterization
- Screening and macro-zonation methodologies
- Risk criteria and its evaluation

B) Second session: Prediction and Modeling

The session is dedicated to the development of theoretical and numerical modeling applied to soil and rock slopes. These methods should help engineers understand the physical processes of landslides and improve the quality of the prediction tools. The session includes the following topics:

- Stability in rock masses and hard soils/soft rocks
- Progressive failure
- Soil-atmosphere interaction. Extreme events
- Slopes in unsaturated soils
- Earthquake triggering of landslides
- Back analysis of field cases

C) Third session: Mitigation and monitoring techniques

1) Practical mitigation solutions for landslides/slope stability problems:

   - Drainage
   - Passive works
   - Anchoring techniques and piles

2) Advances in monitoring techniques:

   - Field monitoring
   - Interferometric radar technologies
   - Satellite platforms
   - LIDAR techniques (or Laser Scanner Techniques)
   - Cases studies

GUIDED TOUR

The following visits are planned during the Conference:

- Visit to the sites of important landslide cases in the "Ain-Draham" region.

- Visit to the Roman cities of Bellarejia, Chemtou and Dogga

CORRESPONDENCES

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TU-SEOUL 2013 International Symposium on Tunnelling and Underground Space Construction for Sustainable Development, March 18-20, 2013, Seoul, Korea www.tu-seoul2013.org

International Conference on Installation Effects in Geotechnical Engineering, 24-27 March 2013, Rotterdam, The Netherlands, http://geo-install.co.uk

EURO:TUN 2013 Computational Methods in Tunneling and Subsurface Engineering, 17-19 April 2013, Bochum, Germany, www.eurotun2013.rub.de

From geological conditions to numerical modeling of underground excavations

Στα πλαίσια του 3rd International Conference on Computational Methods in Tunneling and Subsurface Engineering (EURO:TUN 2013), που θα διεξαχθή στο Ruhr-University Bochum, Germany, την περίοδο 17-19 Απριλίου 2013, διοργανώνεται από τον το μέλος της ΕΕΕΕΓΜ και καθηγητή στην Σχολή Μηχανικών Ορυκτών Πόρων του Πολυτεχνείου Κρήτης Δρ. Γιώργο Εξαδάκτυλο ένα mini συμπόσιο με τίτλο «From geological conditions to numerical modeling of underground excavations».

In this mini-symposium we shall discuss the topics of rock mass and soil characterization, parameter identification and upscaling as well as homogenization techniques, for subsequent numerical modeling of cutting with TBM or other means, as well as for the assessment of the mechanical behavior of underground excavations (deformability & stability). The classical FEM and BEM methods or more sophisticated numerical methods like coupled DEM-FEM etc cannot be effectively applied for predicting the behavior of openings and excavation machines in "opaque-heterogeneous-size dependent-nonlinear geological environments" unless proper constitutive models and spatially varying/uncertain material parameters are identified and effectively implemented in the final 3D ground model ready to run in the computer (in almost real-time?). Size effects of jointed rocks, as well as of soils, models of heterogeneous & uncertain grounds and material parameter identification as well as constitutive testing of soils and rocks, almost real-time rock-soil parameters identification and prediction during excavation, real-life successful and unsuccessful case studies (e.g. tunnel failures like the recent Sao Paulo collapse among many others) may be also discussed. Also we may discuss the effectiveness of geostatistical methods or other stochastic methods on catching the main trends in rock/soil behavior behind the tunnel face and their coupling to 3D numerical models. Emphasis will be also placed on coupled analytical-numerical models for fast predictions and back-analysis procedures.

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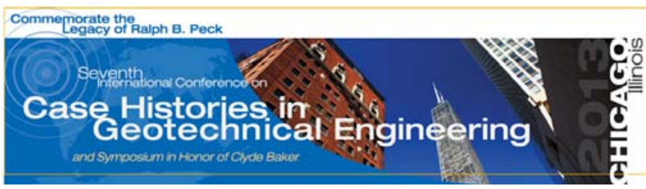
12th International Conference Underground Construction Prague 2013, 22-24 April 2013, Prague, Czech Republic, www.ita-aites.cz/en/conference_underg_constr/conference-uc-2013



Conference to Commemorate the Legacy of Ralph B. Peck, 7th International Conference on Case Histories in Geotechnical Engineering & Soil Dynamics and Symposium in Honor of Clyde Baker, Chicago, USA, 29 April – 4 May, 2013, http://7icchge.mst.edu

IGS-Incheon 2013 - 5th International Symposium on Geotechnical Engineering, Disaster Prevention and Reduction, and Environmentally Sustainable Development, May 15-17 May 2013, Incheon, South Korea, www.geochina-cces.cn/download/2013_5th_Dsiaster_prevention_Bulletin_1.pdf

HF2013 Effective and Sustainable Hydraulic Fracturing - an ISRM Specialized Conference, 20-22 May 2013, Brisbane, Queensland, Australia, http://www.csiro.au/events/HF2013

Experimental Micromechanics for Geomaterials Joint workshop of the ISSMGE TC101-TC105, 23 - 24 May 2013, Hong Kong, owlam@hku.hk

18th SouthEast Asian Geotechnical & Inaugural AGSSEA Conference, 29 – 31 May 2013, Singapore, www.18seagc.com

Second International Symposium on Geotechnical Engineering for the Preservation of Monuments and Historic Sites
30 -31 May 2013, Napoli, Italy
www.tc301-napoli.org

The conservation of monuments and historic sites is one of the most challenging problems facing modern civilization. It involves a number of factors belonging to different fields (cultural, humanistic, social, technical, economical, administrative), intertwining in inextricable patterns. In particular, the requirements of safety and use appear (and often actually are) in conflict with the respect of the integrity of the monuments. In almost all countries of the world the conservation is looked after by an official trained in Art History or Archaeology. He has generally the control of any action to be undertaken, and imposes constraints and limitations that sometimes appear unreasonable to the engineer. The engineer, in turn, tends to achieve safety by means of solutions which appear unacceptable to the official in charge of conservation, sometimes mechanically applying procedures and regulations conceived for new structures. It is evident that some equilibrium has to be found between the safe fruition of a monument and the respect of its integrity. The former task belongs to the know-how of any well trained and experienced engineer, while the latter one is more difficult, being the same concept of integrity rather elusive.

The difficulty of the problem is increased by the lack of a general theory, universally accepted and guiding the behaviour of the actors involved as the Mechanics does with the structural engineer. The possibility of finding in practice an acceptable equilibrium is linked to the development of a shared culture. The International Society of Soil Mechanics and Geotechnical Engineering contributed to this development by an ad hoc Committee (TC 19 – Conservation of Monuments and Historic Sites), that has been promoted over 25 years ago by French and Italian engineers (Jean Kerisel, Arrigo Croce). A number of international and regional symposia have been organised, always with large audience and lively discussions. A Lecture dedicated to Jean Kerisel will be given for the first time at the next International Conference on Soil Mechanics and Geotechnical Engineering to be held in 2013 in Paris. In this framework, the Technical Committee (now TC301) is organising the 2nd International Symposium on Geotechnical Engineering for the Preservation of Monuments and Historic Sites, which will be held in Napoli on May 2013. Its aim is that of comparing experiences, presenting important achievements and new ideas, establishing fruitful links.

The contributions to the Conference should focus on the following main themes:

1. Geotechnical aspects of historic sites, monuments and cities;
2. Past design criteria and traditional construction methods;
3. Techniques to preserve ancient sites and constructions;
4. Rehabilitation of heritage;
5. Role of geotechnical engineering in preservation of cultural and historical integrity.

Scientific secretariat

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WTC 2013 ITA-AITES World Tunnel Congress and 39th General Assembly "Underground – the way to the future", Geneva, Switzerland, May 31 to June 7, 2013. www.wtc2013.ch

First International Conference on Rock Dynamics and Applications (RocDyn-1), 6-8 June 2013, Lausanne, Switzerland, www.rocdyn.org

Strait Crossing Norway 2013 : Extreme Crossings and New Technologies, 16-19 June 2013, Bergen, Norway www.sc2013.no

SINOROCK2013
中國岩石2013

SINOROCK 2013
Rock Characterization, Modelling and Engineering Design Methods
an ISRM Specialized Conference
18-20 June 2013, Shanghai, China
www.sinorock2013.org

This is the third SINOROCK symposium: the first was held at the Three Gorges Dam Project site in 2004; the second was held at the University of Hong Kong in 2009. The Symposium will concentrate on developing the rock engineering design process through the linkage of rock characterisation, modelling and design methods, including feedback from the construction process itself. The Symposium is being held in association with the 2011-2015 ISRM Commission on Rock Engineering Design.

The Themes

- Rock Modelling
- Rock Characterisation
- Engineering Design Methods
- Others Aspects of Rock Mechanics & Engineering

For inquiries please contact

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STREMAH 2013 13th International Conference on Studies, Repairs and Maintenance of Heritage Architecture, 25 – 27 June 2013, New Forest, UK, carlos@wessex.ac.uk

TC215 ISSMGE - International Symposium on Coupled Phenomena in Environmental Geotechnics (CPEG) - "From theoretical and experimental research to practical applications", 1 - 3 July 2013, Torino, Italy, www.tc215-cpeg-torino.org

The 6th International Symposium on Rock Stress, 20-22 August 2013, Sendai, Japan, http://www2.kankyo.tohoku.ac.jp/rs2013

18th International Conference on Soil Mechanics and Geotechnical Engineering "Challenges and Innovations in Geotechnics", 1 – 5 September 2013, Paris, France www.paris2013-icsmge.org

13th International Conference of the Geological Society of Greece, September 5-8 2013, Chania, Greece, www.ege13.gr

Géotechnique Symposium in Print on Bio- and Chemo-Mechanical Processes in Geotechnical Engineering, www.elabs10.com/content/2010001471/SIP%202013.pdf

EUROCK 2013 ISRM European Regional Symposium "Rock Mechanics for Resources, Energy and Environment", 21-26 September 2013, Wroclaw, Poland www.eurock2013.pwr.wroc.pl

VAJONT 2013 - International Conference Vajont, 1963 – 2013 Thoughts and Analyses after 50 years since the catastrophic landslide, 8-10 October, 2013, Padova, Italy, http://www.vajont2013.info/vajont-pd

International Symposium on Design and Practice of Geosynthetic-Reinforced Soil Structures, 14-16 October, 2013, Bologna, Italy, www.civil.columbia.edu/bologna2013

ANDORRA 2014 14th International Winter Road Congress 2014, 4-7 February 2014, Andorra la Vella (Andorra), www.aipcrandorra2014.org

World Tunnel Congress 2014 and 40th ITA General Assembly "Tunnels for a better living", 9 - 15 May 2014, Iguassu Falls, Brazil, www.wtc2014.com.br

EUROCK 2014
ISRM European Regional Symposium
Rock Engineering and Rock Mechanics: Structures in and on Rock Masses
26-28 May 2014, Vigo, Spain

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8th European Conference "Numerical Methods in Geotechnical Engineering", Delft, The Netherlands, 17-20 juni 2014, www.numge2014.org

Second European Conference on Earthquake Engineering and Seismology, 24-29 August 2014, Istanbul, Turkey www.2eceesistanbul.org

IAEG XII CONGRESS Torino 2014 Engineering Geology for Society and Territory, IAEG 50th Anniversary, September 15-18, 2014, Torino, Italy, www.iaeg2014.com

10th International Conference on Geosynthetics – 10ICG, Berlin, Germany, 21 – 25 September 2014 www.10icg-berlin.com

ARMS 8 - 8th ISRM Rock Mechanics Symposium, 15-17 October 2014, Sapporo, Japan www.rocknet-japan.org/ARMS8/index.htm

13th ISRM International Congress on Rock Mechanics
Innovations in Applied and Theoretical Rock Mechanics
10 – 13 May 2015, Montreal, Canada

The Congress of the ISRM "Innovations in Applied and Theoretical Rock Mechanics" will take place on 29 April to 6 May 2015 and will be chaired by Prof. Ferri Hassani.

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World Tunnel Congress 2015
and 41st ITA General Assembly
Promoting Tunnelling in South East European (SEE) Region
22 - 28 May 2015, Dubrovnik, Croatia
http://wtc15.com

Contact
ITA Croatia - Croatian Association for Tunnels and Underground Structures
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XVI ECSMGE 2015

16th European Conference on Soil Mechanics and Geotechnical Engineering
"Geotechnical Engineering for Infrastructure and Development"
13 - 17 September 2015, Edinburgh, UK
www.xvi-ecsmge-2015.org.uk

The British Geotechnical Association (BGA) is pleased to announce that it will be hosting the 16th European Conference on Soil Mechanics and Geotechnical Engineering at the Edinburgh International Conference Centre from 13th to 17th September 2015. The conference was awarded by a meeting of the European Member Societies on 13th September 2011 at the 15th European Conference on Soil Mechanics and Geotechnical Engineering in Athens, Greece.

You can view the BGA bid document at the following link: http://files.marketingedinburgh.org/bid/ECSMGEELECTRONICBID.pdf

The conference website will be updated regularly as arrangements for the conference progress. Please bookmark it and visit regularly.

We look forward to welcoming you all in Edinburgh, one of Europe's truly great cities, in September 2015.

Dr Mike Winter
Chair of the Organising Committee
mwinter@trl.co.uk

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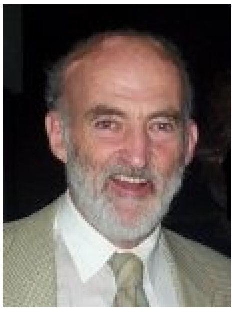
EUROCK 2015
ISRM European Regional Symposium
64th Geomechanics Colloquy
7 – 9 October 2015, Salzburg, Austria

ΝΕΑ ΑΠΟ ΤΙΣ ΔΙΕΘΝΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΝΩΣΕΙΣ

International Society for Rock Mechanics

www.isrm.net

A Tribute to John Franklin, 1940-2012



Professor John Franklin passed away on 6 July, after a prolonged illness that he fought with unparalleled strength and courage. Those who attended the ISRM Congress in Beijing, last October, where John received his Fellowship of the ISRM, can confirm his perseverance and dedication to our Society until the limit of his capacities.

For the Geo-engineering community and for the ISRM in particular this is a very sad moment. Geo-engineering at large is poorer. John Franklin was a top scientist, a successful practitioner, a strong leader and a marvelous person. Those who had the privilege to know him will never forget his dedication, his capacity to motivate people, his brilliant mind and his insurmountable capacity to discover new research themes, or new working methods. He was and will remain a guide for younger generations.

Of his lifetime accomplishments, he was most proud of his association with the ISRM. He served as ISRM President (1987-1991) and Chairman of the ISRM Commissions on Testing Methods (1975-1987) and Education (1991-1995). Among his uncountable contributions to our Society, John has organised and directed the preparation of most of the ISRM "Suggested Methods" for rock testing. On our website you can find his lectures on Geological Engineering which are still up to date after more than twenty years passed. Click here to go to the video.

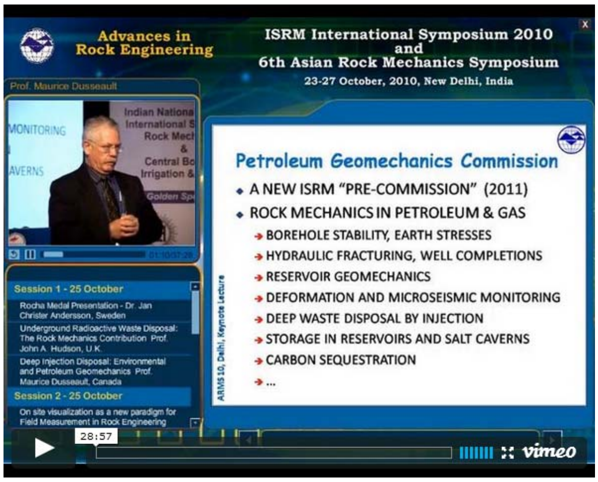
Xia-Ting Feng
ISRM President

New Delhi Asian Rock Mechanics Symposium Keynote lectures now online

The videos and presentations of the keynote lectures of the 2010 ARMS held in New Delhi, India are now online. ISRM members can watch the full length lectures, which are accompanied by the powerpoint presentations:

- Jan Christer Andersson, Rocha Medal Winner - Äspö Pillar Stability Experiment
- John A. Hudson - Underground Radioactive Waste Disposal: The Rock Mechanics Contribution
- Maurice Dusseault - Deep Injection Disposal: Environmental and Petroleum Geomechanics
- Shinichi Akutagawa - On Site Visualization as a New Paradigm for Field Measurement in Rock Engineering
- Herb Wang - Deep Underground Instrumentation and Monitoring
- Giovanni Barla - Progress in the Understanding of Deep-Seated Landslides from Massive Rock Slope Failure
- Yossef H. Hatzor - Modelling Dynamic Deformation in Natural Rock Slopes and Underground Openings with Numerical DDA Method
- Claus Erichsen - Challenges in the Design and Construction of Tunnels in Jointed Rock
- Guowei Ma and Yingxin Zhou - Rock Dynamics Research in Singapore: Fundamentals and Practices
- Xia-Ting Feng - Application of Intelligent Rock Mechanics Methodology to Rock Engineering
- John Read - The Large Open Pit Project

Click here to go to the videos page (login required).



ΕΝΔΙΑΦΕΡΟΝΤΑ ΓΕΩΤΕΧΝΙΚΑ ΝΕΑ

Researchers develop field device for bridge scour testing

Scour is the most common cause of bridge failure in the United States and contributes greatly to bridge construction and maintenance costs. Monitoring and mitigation is essential, but information to evaluate potential scour in erosion-resistant soils is scarce. Because of time and cost constraints, most scour prediction methodologies do not account for the wide range of naturally occurring soils and their resistance to erosion and scour.

To address the need for more reliable and practical methodologies, researchers at the Federal Highway Administration's (FHWA) J. Sterling Jones Hydraulics Research Laboratory at the Turner-Fairbank Highway Research Center (TFHRC) in McLean, Virginia are developing a scour testing field device. The project is made possible through an FHWA Pooled-Fund solicitation with contributions from California, Colorado, Kansas, North Carolina, New York, Texas, Utah, Wisconsin, and the FHWA Federal Lands Highway.

"The device will have a confined column of continuously flowing water directed downward and then horizontally across the soils that are to support pier foundations," explains Kornel Kerenyi, a senior hydraulics research engineer in FHWA's Office of Infrastructure Research and Development. "To determine the erosion rate, the shear strength of the flow is reduced with the depth of advancing scour to reflect the natural decay of the scouring mechanism."

Through physical model testing, the device will be calibrated to identify the input energy needed to produce scour depths predicted by equations for sand-bed channels. According to Bart Bergendahl, a senior hydraulic engineer in the Central Lands Federal Highway Division, "the input energy will be scaled up for a prototype device and field tests will run until equilibrium conditions are reached in the resulting scour hole, or until some maximum period of time has elapsed."

One possibility for duration is the expected cumulative time the foundation will be exposed to design discharge over bridge life. In-situ soils will be exposed to energy necessary to develop scour depth predicted by equations. Equilibrium or maximum scour depth resulting from a field test that is less than the predicted depth for a sand-bed channel will be attributable to the erosion-resistant characteristics of in-situ soils.

"The full-scale field device is envisioned to be a closed, re-circulation and filtering system that will operate in both wet and dry conditions while minimizing environmental impacts," says Kerenyi. "The column would be suspended vertically from an overhead crane."

Attached to the top of the column will be a weight sufficient to propel the column into the soil, incrementally upon release, as in-situ soils are scoured away by the cutting head. The progression of the cutting head and reductions in flow rate and shear will be coordinated by sensors in the head and computerized controls.

"The field device will be used for foundation analysis and design in a manner similar to present-day soil borings," adds Bergendahl. "Testing will be conducted at proposed foundation locations across the channel and floodplain area at the site of a new bridge or replacement bridge."

To adjust design scour depth predicted by equations for sand-bed channels and to reflect actual erodibility of in-situ soils at the bridge site, scour depth information resulting from field tests will be used in conjunction with subsurface soil boring information.

Currently a second generation, lab-scale device consisting of an outer circular pipe column with a concentric cutting head centered within the column is being tested.

"To contain incoming flow and minimize soil disturbance, the outer pipe column will advance slightly ahead of the cutting head," Kerenyi explains. "The inflow enters the cutting head-soil interface from around the perimeter of the head, flows horizontally inward across the soil, and exits vertically upward through an outlet in the center of the cutting head, carrying the eroded material away with it."

In initial testing, the cutting head has performed well. Designed to ensure a uniform horizontal shear and symmetrical pressure distribution, it was created with 3-dimensional, computational fluid dynamics modeling performed by the super-computer at the Transportation Research Analysis Computing Center at Argonne National Laboratories in Illinois.

Looking ahead, Kerenyi and Bergendahl plan to pressurize the lab-scale system to allow accurate control of flow and shear at the cutting head to implement the shear decay function. "This will define how the flow should be reduced with depth," says Bergendahl. "If we're successful, we can begin on a prototype device and auxiliary components."

Initially, the prototype will be used for field calibration. If results are favorable, it can be used to test the erosion and scour potential of a wide range of soils types to depths of 20 meters. More accurate estimates of scour in erosion resistant cohesive, cementatious, and rock-like soils could make foundation construction less difficult and expensive.

"This research will determine if a practical field testing device can be developed to optimize expensive bridge foundation construction," observes Kerenyi. "Preliminary results are very encouraging. If successful, the payoff of such a device will be limitless."

For more information about the project, visit www.pooledfund.org/Details/Study/438.

(Federal Highway Administration, Rebuilding America's Infrastructure, September 14, 2012, http://www.rebuildingamericasinfrastructure.com/article-comments-research-ers_develop_field_device_for_bridge_scour_testing-1050.html)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΣΕΙΣΜΟΙ

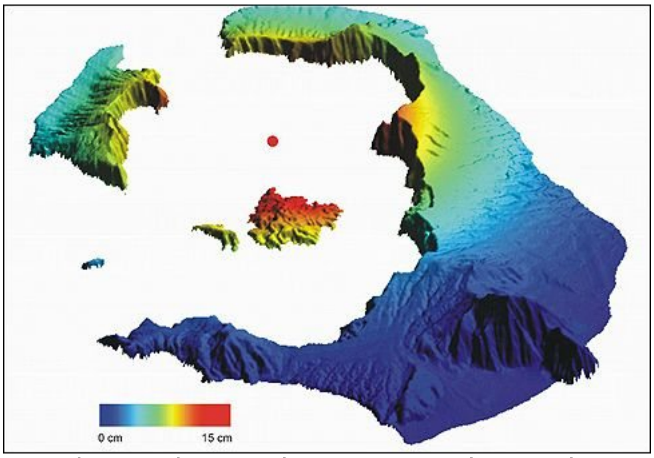
Το νησί ανυψώθηκε
Σημαντική συσσώρευση μάγματος κάτω από το ηφαίστειο της Σαντορίνης

Η τελευταία έκρηξη του ηφαιστείου σημειώθηκε το 1950 και συνοδεύτηκε από ισχυρό σεισμό

Ο θάλαμος μάγματος που κρύβεται κάτω από την καλδέρα της Σαντορίνης διογκώθηκε κατά 10 με 20 εκατομμύρια κυβικά μέτρα σε διάστημα 15 μηνών -μια ανησυχητική τάση, η οποία όμως φαίνεται να έχει πλέον σταματήσει, αναφέρουν ερευνητές της Οξφόρδης στην επιθεώρηση Nature Geoscience.

Όπως δείχνουν μετρήσεις δορυφορικών ραντάρ και δεκτών GPS στην επιφάνεια του εδάφους, η διόγκωση του ηφαιστείου συνεχιζόταν από τον Ιανουάριο του 2011, όταν καταγράφηκε η πρώτη σεισμική δραστηριότητα εδώ και 25 χρόνια, μέχρι τον Απρίλιο του 2012, οπότε ολόκληρο το νησί της Σαντορίνης είχε πλέον ανυψωθεί κατά 8 έως 14 εκατοστά.

«Αν αυτός ο ρυθμός διόγκωσης συνεχιζόταν για λίγα ακόμα χρόνια, ο όγκος [του μάγματος] θα ήταν αντίστοιχος με τον όγκο προηγούμενων εκρήξεων του ηφαιστείου» γράφουν οι ερευνητές.



Μοντέλο της κάθετης ανύψωσης της καλδέρας. Η κόκκινη περιοχή στο κέντρο (το νησί της Νέας Καμμένης) ανυψώθηκε κατά 15 εκατοστά (Πηγή: Michelle Parks / University of Oxford)

Η αύξηση, υπολογίζουν, αντιστοιχεί περίπου στο μισό της ποσότητας λάβας που απελευθέρωσε το ηφαίστειο στη διάρκεια προηγούμενων μικρών εκρήξεων, όπως αυτή που αναστάτωσε το νησί το 1950.

Τα νέα δεδομένα «αποτελούν ισχυρή ένδειξη ότι το νέο επεισόδιο ηφαιστειακής διαστολής είναι το μόνο σημαντικό που έχει καταγραφεί από την έκρηξη του ηφαιστείου το διάστημα 1939-1941» επισημαίνουν οι ερευνητές.

Προειδοποιούν μάλιστα ότι «θα ήταν λάθος να υποθέσει κανείς ότι η σημερινή κατάσταση ανησυχίας δεν θα οδηγήσει σε έκρηξη».

Πιο καθησυχαστικό είναι ωστόσο το σχετικό δελτίο Τύπου που αναρτήθηκε στο δικτυακό τόπο του Πανεπιστημίου της Οξφόρδης: Τα δεδομένα «δεν σημαίνουν ότι επίκειται έκρηξη. Η αλήθεια είναι ότι η ηφαιστειακή δραστηριότητα έχει εξασθενίσει τους τελευταίους μήνες».

Την ίδια επισήμανση έκανε στο πρακτορείο Bloomberg ο Γιώργος Βουγιουκλάκης, ηφαιστειολόγος του Ινστιτούτου Γεωλογικών και Μεταλλευτικών Ερευνών, ο οποίος δεν συμμετείχε στη νέα έρευνα: «Όλη η δραστηριότητα που καταγράφεται τώρα αντιστοιχεί στην κανονική, λανθάνουσα κατάσταση του ηφαιστείου» είπε.

Γεγονός είναι πάντως ότι η σεισμική δραστηριότητα, η εκπομπή αερίων και η παραμόρφωση του εδάφους προκάλεσαν πέρυσι το ενδιαφέρον των γεωλόγων και δεν έμειναν απαρατήρητες από τους ντόπιους.

«Στη διάρκεια των επισκέψεων πεδίου στη Σαντορίνη το 2011, έγινε εμφανές ότι πολλοί κάτοικοι είχαν αντιληφθεί μια αλλαγή στη συμπεριφορά του ηφαιστείου τους. Οι ξεναγοί, οι οποίοι επισκέπτονται το ηφαίστειο αρκετές φορές την ημέρα, με ενημέρωναν για αλλαγές στην ποσότητα των αερίων με έντονη οσμή που απελευθερώνονταν στην κορυφή, ή για αλλαγές στο χρώμα του νερού σε ορισμένους κόλπους γύρω από τα νησιά» αναφέρει η Μισέλ Παρκς, μέλος της ερευνητικής ομάδας στην Οξφόρδη που ταξίδεψε στη Σαντορίνη. Προσθέτει ότι ορισμένοι από τους μικροσεισμούς που προκαλούσε το ηφαίστειο γίνονταν αντιληπτοί στα εστιατόρια από το κούνημα των πιάτων και των ποτηριών στα τραπέζια.

Μεγάλες εκρήξεις του ηφαιστείου της Θήρας πιστεύεται ότι συμβαίνουν κάθε 10 με 30 χιλιάδες χρόνια, ενώ μικρότερες εκρήξεις σημειώνονταν στο παρελθόν ακόμα και κάθε 14 χρόνια.

Το ηφαίστειο είναι παγκοσμίως γνωστό για την έκρηξη του 17ου αιώνα π.Χ, μια από τις ισχυρότερες εκρήξεις στην ιστορία της Γης.

Η ενεργοποίηση του ηφαιστείου προκάλεσε την κατάρρευση του κρατήρα και το σχηματισμό καλδέρας, της οποίας το όριο αντιστοιχεί στο νησί της Σαντορίνης όπως τη γνωρίζουμε σήμερα.

Το τσουνάμι που προκάλεσε η έκρηξη έχει ενοχοποιηθεί και για την εξαφάνιση του μινωικού πολιτισμού στη γειτονική Κρήτη. Το σεισμικό κύμα δεν αποκλείεται να έφτασε μέχρι την έρημο του Σινά, τέσσερα χιλιόμετρα από τις μεσογειακές ακτές.

(Newsroom ΔΟΛ, 10 Σεπ. 2012, http://news.in.gr/science-technology/article/?aid=1231212477)

Μεγάλο «φούσκωμα» του μάγματος κάτω από το ηφαίστειο της Σαντορίνης, σύμφωνα με Βρετανούς και 'Ελληνες επιστήμονες

Ο υπόγειος θάλαμος του μάγματος (των λιωμένων πετρωμάτων) κάτω από το ηφαίστειο της Σαντορίνης επεκτάθηκε σημαντικά, κατά περίπου 10 έως 20 εκατομμύρια κυβικά μετρα, μεταξύ Ιανουαρίου 2011 και Απριλίου 2012, σύμφωνα με Βρετανούς επιστήμονες των πανεπιστημίων της Οξφόρδης και του Μπρίστολ, σε συνεργασία με 'Ελληνες συναδέλφους τους από το πανεπιστήμιο Αθηνών (Τμήμα Γεωλογίας & Γεωπεριβάλλοντος) και το Εθνικό Μετσόβιο Πολυτεχνείο (Εργαστήριο Ανώτερης Γεωδαισίας).

Οι επιστήμονες εκτιμούν ότι το «φούσκωμα» του μάγματος έχει όγκο έως 15 φορές μεγαλύτερο από το Ολυμπιακό Στάδιο του Λονδίνου. Η προσθήκη αυτού του μάγματος, από άποψη όγκου, θεωρείται η σημαντικότερη που έχει λάβει χώρα από το 1955, λίγο μετά την τελευταία έκρηξη του ηφαιστείου.

Οι ερευνητές με επικεφαλής τον καθηγητή ηφαιστειολογίας Ντέηβιντ Πάιλ και την ηφαιστειολόγο Μισέλ Παρκς του Τμήματος Γεωεπιστημών της Οξφόρδης, καθώς και την Τζούλιετ Μπιγκς του Τμήματος Γεωεπιστημών του Μπρίστολ, που έκαναν τη σχετική δημοσίευση στο περιοδικό "Nature Geoscience", υπολόγισαν ότι η διόγκωση του μάγματος οδήγησε σε ανύψωση της επιφάνειας του νησιού κατά 8 έως 14 εκατοστά στο ίδιο χρονικό διάστημα. Οι επιτόπιοι υπολογισμοί στη Σαντορίνη έγιναν με τη βοήθεια εικόνων από δορυφορικά ραντάρ που παρείχαν ο Ευρωπαϊκός Οργανισμός Διαστήματος (ESA) και η Γερμανική Υπηρεσία Διαστήματος (DLR), καθώς και μετρήσεων GPS από το έδαφος.

Αν και τα νέα στοιχεία ρίχνουν περισσότερο φως στη λειτουργία του ηφαιστείου (το οποίο γύρω στα 1600 π.Χ. σημείωσε μία από τις μεγαλύτερες εκρήξεις στην παγκόσμια ιστορία, οδηγώντας πιθανώς στην καταστροφή του Μινωικού πολιτισμού), σύμφωνα με τους ερευνητές, δεν διαφωτίζουν για το μείζον ερώτημα: πότε το ηφαίστειο θα εκραγεί ξανά.

Το «ζωντάνεμα» του ηφαιστείου καταγράφηκε στην αρχή του 2011 με μικρούς σεισμούς, που τράβηξαν την προσοχή των Ελλήνων και ξένων επιστημόνων, καθώς για περίπου 25 χρόνια επικρατούσε σχετική ησυχία στην καλδέρα. Οι μικροσεισμοί συνοδεύτηκαν από ορισμένες άλλες περιστασιακές ενδείξεις, όπως αλλαγές στο χρώμα του νερού σε ορισμένα σημεία και έκλυση αερίων με χαρακτηριστική οσμή. Οι ερευνητές εκτιμούν ότι η ποσότητα λιωμένων πετρωμάτων (μάγματος) που έχει σωρευτεί κάτω από το ηφαίστειο κατά το περασμένο έτος, ισοδυναμεί με περίπου 10 έως 20 έτη ανάπτυξης του ηφαιστείου. Επισημαίνουν όμως πως αυτό δεν σημαίνει ότι επίκειται κάποια έκρηξη, αντίθετα, όπως τονίζουν, η σεισμική δραστηριότητα έχει σαφώς υποχωρήσει κατά τους τελευταίους μήνες.

Οι ερευνητές σημειώνουν ότι το ηφαίστειο της Σαντορίνης εμφανίζει δύο διαφορετικών ειδών εκρήξεις στο πέρασμα του χρόνου: αφενός μικρές εκρήξεις που συμβαίνουν σχετικά συχνά και εκλύουν λάβα και, αφετέρου, μεγάλες εκρήξεις που συμβαίνουν σπάνια, ανά περίπου 10.000 έως 30.000 χρόνια. Και οι δύο τύποι εκρήξεων θεωρείτο ότι «εκκολάπτονται» σε ένα ρηχό θάλαμο μάγματος, ο οποίος τροφοδοτείται σε συνεχή βάση από μικρές ποσότητες λιωμένων πετρωμάτων, οι οποίοι ανεβαίνουν από κάτω προς τα πάνω.

Όμως, όπως αναφέρει η νέα μελέτη, γεωλογικές (ορυκτολογικές) μελέτες δείχνουν ότι τουλάχιστον το 15% του υλικού που εκτινάχθηκε κατά την Μινωική έκρηξη του ηφαιστείου, έφθασε στον θάλαμο του μάγματος λιγότερο από 100 χρόνια πριν την έκρηξη, δηλαδή όχι σταδιακά και σε βάθος χρόνου, αλλά σε πολύ πιο σύντομο χρόνο.

Σύμφωνα με τους ερευνητές, είτε το ηφαίστειο της Σαντορίνης βρίσκεται στην πιο συνηθισμένη (αργή) φάση συσσώρευσης λάβας, είτε στην πιο σπάνια (και γρήγορη) εκρηκτική φάση του, ο ρηχός θάλαμος του μάγματος τροφοδοτείται περιστασιακά με ταχείας ροής ποσότητες μάγματος. Η χρονική διάρκεια αυτών των περιόδων τροφοδότησης του μάγματος είναι σύντομη σε σχέση με τις ενδιάμεσες περιόδους ηρεμίας, ενώ το πότε θα συμβούν αυτές οι περιστασιακές ανατροφοδοτήσεις, δηλαδή ο χρονισμός τους, εξαρτάται από την υπόγεια δυναμική που αναπτύσσεται στο ακόμα βαθύτερο τμήμα του μάγματος κάτω από τη Σαντορίνη.

Κατά τις πιο πρόσφατες εκρήξεις του ηφαιστείου της Σαντορίνης, στην επιφάνεια έχουν ανέλθει συνήθως δύο διαφορετικά είδη μάγματος, κατά κύριο λόγο δακίτης (λάβα πλούσια σε πυριτία) και δευτερευόντως ανδεσίτης (πιο καυτή λάβα με λιγότερη πυριτία). Προηγούμενες έρευνες έχουν δείξει ότι οι εκρήξεις εμφανίζονται να πυροδοτούνται από την άνοδο προς την επιφάνεια του ανδεσίτη, ο οποίος «αναμοχλεύει» τον πιο άφθονο δακίτη, δίνοντας έτσι το έναυσμα για την έκρηξη, πιθανώς μέσα σε διάστημα λίγων εβδομάδων.

Επειδή όμως, σύμφωνα με τους Βρετανούς γεωεπιστήμονες (στην έρευνα των οποίων συνέβαλαν οι έλληνες ερευνητές Παρασκευή Νομικού, Ξάνθος Παπανικολάου, Δημήτρης Παραδείσης, Κώστας Ραπτάκης και Βαγγέλης Ζαχάρης), η πρόσφατη ηφαιστειακή δραστηριότητα έχει διαρκέσει πολύ περισσότερο χρόνο, γίνεται η υπόθεση ότι τα λιωμένα πετρώματα που συσσωρεύονται τώρα κάτω από τη Σαντορίνη, είναι δακίτες και όχι ανδεσίτες.

Link: Για την πρωτότυπη επιστημονική εργασία (με συνδρομή) στη διεύθυνση:
http://www.nature.com/ngeo/journal/vaop/ncurrent/abs/ngeo1562.html

(News Room «Κέρδος» με πληροφόρηση από το ΑΠΕ – ΑΜΠ, 10/9/2012,
http://www.kerdos.gr/default.aspx?id=1796078&nt=103&utm_source=KerdosNLetterApp&utm_medium=email&utm_campaign=html_newsletter)

Giant 'balloon of magma' inflates under Santorini

A new survey suggests that the chamber of molten rock beneath Santorini's volcano expanded 10-20 million cubic metres – up to 15 times the size of London's Olympic Stadium – between January 2011 and April 2012.

The growth of this 'balloon' of magma has seen the surface of the island rise 8-14 centimetres during this period, a team led by Oxford University scientists has found. The results come from an expedition, funded by the UK's Natural Environment Research Council, which used satellite radar images and Global Positioning System receivers (GPS) that can detect movements of the Earth's surface of just a few millimetres.

The findings are helping scientists to understand more about the inner workings of the volcano which had its last major explosive eruption 3,600 years ago, burying the islands of Santorini under metres of pumice. However, it still does not provide an answer to the biggest question of all: when will the volcano next erupt?

Michelle Parks makes GPS measurements on Santorini.

A report of the research appears in this week's *Nature Geoscience*.

In January 2011, a series of small earthquakes began beneath the islands of Santorini. Most were so small they could only be detected with sensitive seismometers but it was the first sign of activity beneath the volcano to be detected for 25 years.

Following the earthquakes Michelle Parks, an Oxford University DPhil student, spotted signs of movement of the Earth's surface on Santorini in satellite radar images. Oxford University undergraduate students then helped researchers complete a new survey of the island.

Parks, who is based in Oxford University's Department of Earth Sciences and is an author of the paper, said: 'During my field visits to Santorini in 2011, it became apparent that many of the locals were aware of a change in the behaviour of their volcano. The tour guides, who visit the volcano several times a day, would update me on changes in the amount of strong smelling gas being released from the summit, or changes in the colour of the water in some of the bays around the islands.

'On one particular day in April 2011, two guides told me they had felt an earthquake while they were on the volcano and that the motion of the ground had actually made them jump. Locals working in restaurants on the main island of Thera became aware of the increase in earthquake activity due to the vibration and clinking of glasses in their bars.'

Dr Juliet Biggs of Bristol University, also an author of the paper, said: 'People were obviously aware that something was happening to the volcano, but it wasn't until we saw the changes in the GPS, and the uplift on the radar images that we really knew that molten rock was being injected at such a shallow level beneath the volcano.

'Many volcanologists study the rocks produced by old eruptions to understand what happened in the past, so it's exciting to use cutting-edge satellite technology to link that to what's going on in the volcanic plumbing system right now.'

Professor David Pyle of Oxford University's Department of Earth Sciences, an author of the paper, said: 'For me, the challenge of this project is to understand how the information on how the volcano is behaving right now can be squared with what we thought we knew about the volcano, based on the studies of both recent and ancient eruptions. There are very few volcanoes where we have such detailed information about their past history.'

The team calculate that the amount of molten rock that has arrived beneath Santorini in the past year is the equivalent of about 10-20 years growth of the volcano. But this does not mean that an eruption is about to happen: in fact the rate of earthquake activity has dropped off in the past few months.

(University of Oxford, 10 Sep 12, http://www.ox.ac.uk/media/news_stories/2012/120910.html)

Evolution of Santorini Volcano dominated by episodic and rapid fluxes of melt from depth

Michelle M. Parks,[1] Juliet Biggs,[2] Philip England,[1] Tamsin A. Mather,[1] Paraskevi Nomikou,[3] Kirill Palamartchouk,[1,4] Xanthos Papanikolaou,[5] Demitris Paradissis,[5] Barry Parsons,[1] David M. Pyle,[1] Costas Raptakis[5] & Vangelis Zacharis[5]

Abstract

Santorini Volcano, the site of the catastrophic Minoan eruption in Greece, exhibits two distinct eruptive styles: small, effusive eruptions occur relatively frequently and build shields and domes of lava, whereas large explosive eruptions occur rarely, at intervals of 10,000–30,000 years. Both types of eruption were thought to incubate in a shallow magma chamber that is continually charged by small batches of melt injected into the chamber from below. However, petrological work suggests that at least 15% of the material ejected during the Minoan explosive eruption arrived in the magma chamber less than 100 years before the eruption. Here we use Satellite Radar Interferometry (InSAR) and Global Positioning System (GPS) measurements of surface deformation at Santorini to show that 10–20 million m^3 of magma have been intruded beneath the volcano since January 2011. This volume is equivalent to 10–50% of the volumes of recorded dome-forming eruptions. GPS and triangulation data show that this is the only volumetrically significant intrusion to have occurred since 1955, shortly after the last eruption. Our observations imply that whether Santorini is in an explosive or dome-forming phase, its shallow magma chamber is charged episodically by high-flux batches of magma. The durations of these events are short in comparison with the intervening periods of repose and their timing is controlled by the dynamics of deeper magma reservoirs.

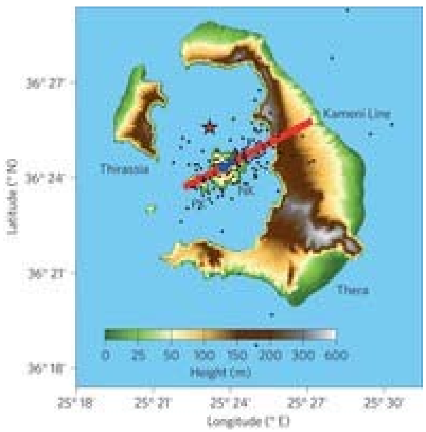


Figure 1: The islands of the Santorini volcanic centre, and the location of the centre of recent volcanic and seismic activity.

PK and NK denote the islands of Palaea and Nea Kameni, respectively; the Kameni Line[4], shown in red, passes through the locations of the vents of all the post-1570 eruptions (blue dots), and is probably an active normal fault that dips north-northwes…

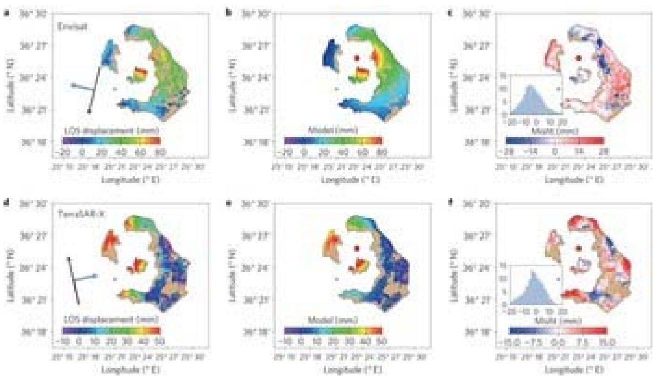


Figure 2: InSAR measurements, and fits to them of the best-fitting spherical inflationary source, whose parameters are given in Table 2.
a, Chain-stacked Envisat interferogram from 3 March to 28 December 2011; brown shading shows areas of land that were de-correlated. Thin arrow indicates orientation of satellite orbit, and thick arrow indicates the look direction of the satellite. Co…

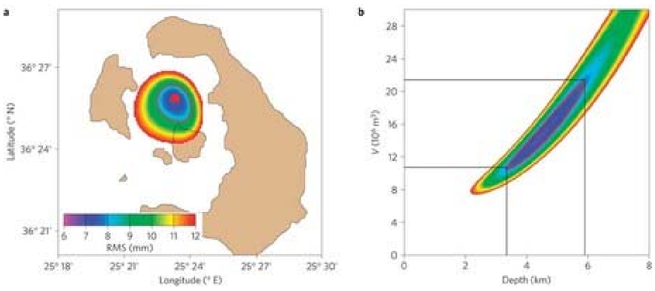


Figure 3: Bounds on the location and volume of the magma body intruded beneath Santorini since January 2011.
a, The colours show the minimum r.m.s. misfit between modelled and observed interferograms for spherical sources[15] beneath each location; where no colour is shown the r.m.s. misfit exceeds 12 mm. **b**, Trade-off between the depth of the spherical source…

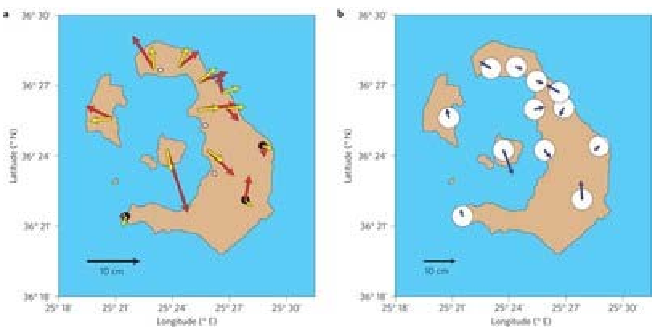


Figure 4: Horizontal surface displacements of Santorini between 1955 and 2011 from triangulation and GPS data.
a, The yellow vectors show the expected displacements if the only deformation in that time interval is that due to the inflation detected since the beginning of 2011 (Table 2).
The red vectors show displacements of monuments between 1955 and 2011, ca…

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Evolution of Santorini Volcano dominated by episodic and rapid fluxes of magma from depth: Supplementary Material

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Uncertainties in Model Parameters

Due to the limited spatial extent of the interferograms it is impossible to assess the formal significance of a given level of misfit between model and inerferogram. Here, we provide figures to support the bounds we place on the model parameters.

Figure S.1 shows the influence on misfits of holding the horizontal location of the source fixed, but varying its depth. Figures S.1 c & d correspond to the overall best-fitting solution (Figure 2, main text), which has a source depth of 4.4km and an RMS misfit of 6.8mm. Forcing the source depth as shallow as 3.35km, or as deep as 5.9km, increases the RMS misfit by 1mm, and at either of those two depths, the residuals show a systematic pattern. When the source is forced to be shallower than the best-fit depth (Figure S.1 a & b) residuals far from the source are predominantly positive, and when it is forced to the greater depth (Figure S.1 e & f) residuals far from the source are predominantly negative. These patterns reflect the fact that the characteristic width of the zone of uplift caused by the inflationary source is proportional to the depth of the source [1]. A model source that is placed too shallow will generate too little uplift in the far field and, because the InSAR observable (displacement in the line of sight to the satellite) is sensitive principally to vertical displacements, the residuals (observation minus model) will be positive in the far field. The reciprocal argument applies to a model source that is placed too deep.

Figure S.1 shows that if the source depth is taken outside the range of 4.4+1.5/−1.0km, the pattern of residuals clearly demonstrates that the depth is erroneous. This range should be taken as representing the bounds on source depth, not as formal uncertainties. By similar arguments, Figure S.2 shows that the horizontal coordinates of the source are constrained to within about 1000 metres. Outside these bounds, the misfits show systematic spatial variation that is related to the displacement of the model source from the best-fitting position.

This discussion, in conjunction with Figures S.1 and S.2, provides the basis upon which, in Figures 2 & 3, and Table 2 of the main text, we assign bounds to the parameters of the inflation source.

Wrapped Interferograms

We show here (Figure S.3) wrapped interferograms corresponding to Fig 2 a-d of the paper. Such images highlight details of the deformation pattern, particularly the deformation gradient, but are more sensitive to atmospheric noise. They are included here for the convenience of the community that is accustomed to reading wrapped interferograms.

References

[1] Mogi, K. Relations between the eruptions of various volcanoes and the deformations of the ground sources around them. *Bull. Earthquake Res. Inst. Japan* 36, 99–134 (1958).

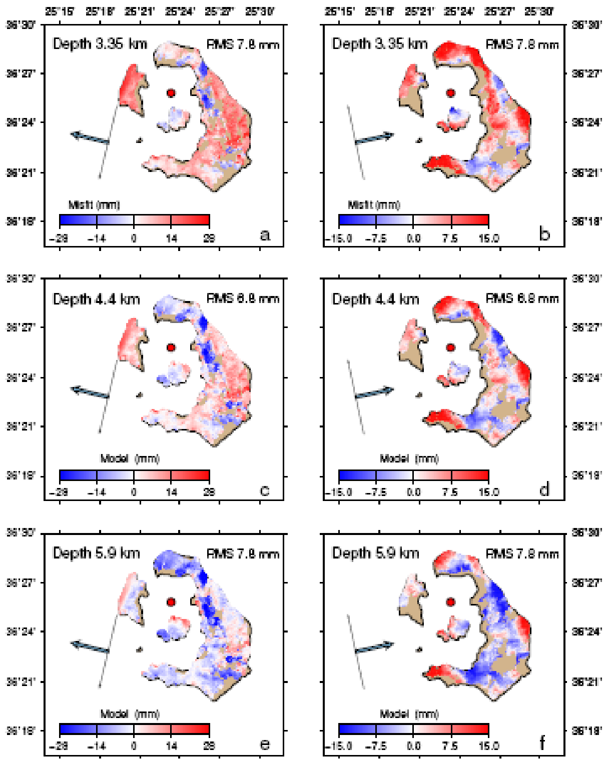

Figure S.1: Misfits (observation minus model) between stacked interferograms from Envisat (panels a,c,d) and TerraSAR-X (panels b,d,f) and model spherical (Mogi) inflationary sources held at different depths. Thin arrow indicates orientation of satellite orbit, and thick arrow indicates the look direction of the satellite. The source is held at its best-fitting horizontal location (red dots) with its depth fixed to 3.35 km (panels a,b), 4.4 km (panels c,d), and 5.9 km (panels e,f). Panels c and d correspond to the best-fitting model (see Figure 2, main text).

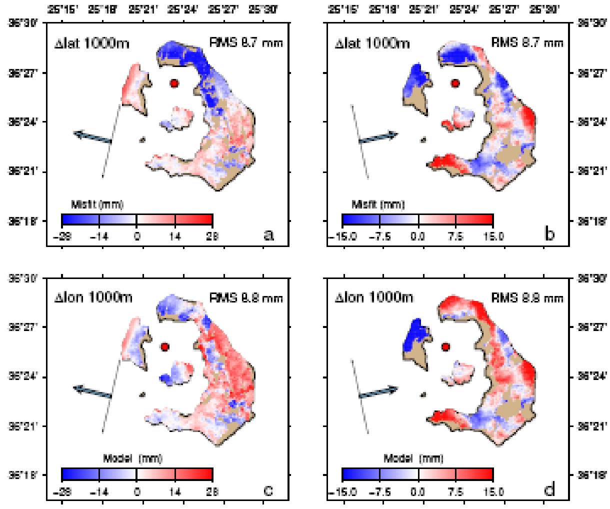

Figure S.2: Misfits (observation minus model) between stacked interferograms from Envisat (panels a,c,d) and TerraSAR-X (panels b,d,f) and model spherical (Mogi) inflationary sources held at different locations (red dots). The source is held at its best-fitting depth (4.4km) with its horizontal location fixed 1000 m north of the best-fitting position (panels a,b) and 1000 m west of the best-fitting position (panels c,d)

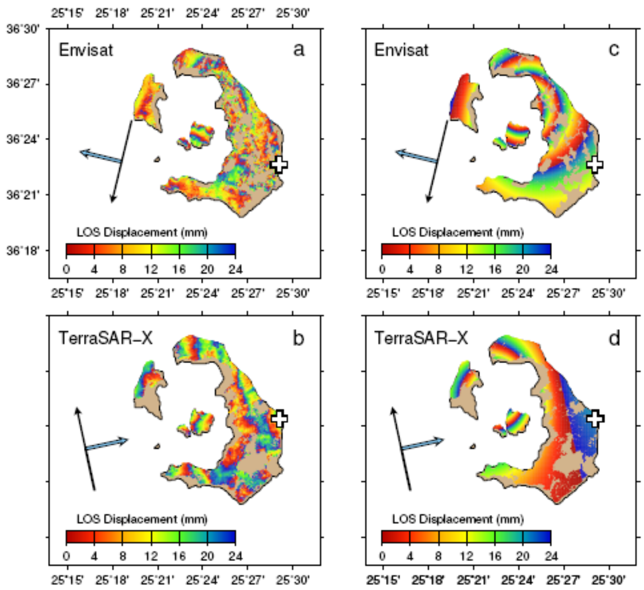

Figure S.3: Wrapped interferograms corresponding to Figures 2a-d. The phase observations are wrapped to a colour cycle of 24mm, which is between the natural half-wavelengths of the Envisat data (28mm) and the Terra SAR-X data (16mm).

Nature Geoscience, 2012, doi:10.1038/ngeo1562, http://www.nature.com/ngeo/journal/vaop/ncurrent/full/ngeo1562.html#/t2

Σε συστηματική υποθαλάσσια σεισμική επιτήρηση Κωνσταντινούπολη και Μαρμαράς

Ο κίνδυνος ενός νέου ισχυρού καταστροφικού σεισμού στα παράλια της γειτονικής Τουρκίας είναι μεγάλος και, για αυτό το λόγο, γερμανοί επιστήμονες, σε συνεργασία με τούρκους συναδέλφους τους, μόλις άρχισαν τις εργασίες πολλαπλών γεωτρήσεων για την εγκατάσταση ειδικού υποθαλάσσιου εξοπλισμού παρακολούθησης.

Οι ερευνητές του Γερμανικού Κέντρου Ερευνών για τις Γεω-επιστήμες Χέλμχολτς (GFZ), μόλις άρχισαν την εγκατάσταση ενός δικτύου σεισμικής επιτήρησης στη Θάλασσα του Μαρμαρά, κοντά στην Κωνσταντινούπολη. Ειδικά σχεδιασμένοι σεισμικοί αισθητήρες θα τοποθετηθούν μόνιμα σε οκτώ πηγάδια γεώτρησης βάθους 300 μέτρων το καθένα, που άρχισαν να ανοίγονται στα ανοιχτά της πόλης και γύρω από τη θαλασσα του ανατολικού Μαρμαρά.

Μ' αυτό τον τρόπο θα υπάρχει ακριβής παρακολούθηση της μελλοντικής σεισμικής δραστηριότητας στην περιοχή, που θεωρείται υψηλού κινδύνου. Οι γερμανικοί σεισμογράφοι θα καταγράφουν και τις πιο ανεπαίσθητες σεισμικές δονήσεις στο βόρειο ρήγμα της Ανατολίας, το οποίο περνάει κάτω από το βυθό της θάλασσας του Μαρμαρά, περίπου 20 χιλιόμετρα απέναντι από την Κωνσταντινούπολη.

Όπως δήλωσε ο καθηγητής Γκέοργκ Ντρέζεν του Κέντρου GFZ, «η Ιστανμπούλ, με τους πάνω από 13 εκατ. κατοίκους της, βρίσκεται σε μια περιοχή που είναι υπερβολικά ευάλωτη από τους σεισμούς. Υπάρχει μεγάλη πιθανότητα για ένα ισχυρό σεισμό μεγέθους έως 7,4 στην περιοχή». Όπως είπε, οι νέοι σεισμογράφοι που εγκαθίστανται, «θα δώσουν σημαντικές πληροφορίες για τις τεκτονικές διεργασίες προτού συμβεί ένας μεγάλος σεισμός».

Η πρώτη πιλοτική γεώτρηση γίνεται στη χερσόνησο Τούζλα και, αφού ολοκληρωθεί και κριθεί επιτυχής μετά από τις σχετικές δοκιμές των πρώτων εγκατεστημένων επιστημονικών οργάνων, θα ακολουθήσει το άνοιγμα και των υπόλοιπων επτά υποθαλάσσιων πηγαδιών γεώτρησης.

Τα σεισμικά δεδομένα θα μεταδίδονται ταυτόχρονα σε πραγματικό χρόνο στο Πότσνταμ της Γερμανίας και στην πρωτεύουσα της Τουρκίας Άγκυρα, όπου θα αξιολογούνται. Πάντως ο επικεφαλής του προγράμματος καθηγητής του Κέντρου GFZ Μάρκο Μπόνχοφ φρόντισε να διευκρινίσει ότι «η πρόβλεψη ενός μελλοντικού σεισμού δεν είναι ο στόχος του έργου, επειδή τέτοια πρόβλεψη δεν είναι δυνατή».

Πηγή: ΑΜΠΕ

(Η ΚΑΘΗΜΕΡΙΝΗ, 10.Σεπτεμβρίου 2012, http://portal.kathimerini.gr/4dcgi/_w_articles_kathciv_1_10/09/2012_460475)

ΕΝΔΙΑΦΕΡΟΝΤΑ – ΠΕΡΙΒΑΛΛΟΝ

CarbonCure makes concrete go green 'without compromise'

Robert Niven can talk at length about the "green" benefits of his concrete-making technology. But if he can't match or beat the price of traditional methods, he knows few producers will listen.

"We have to be green without compromise," said the chief executive and founder of CarbonCure Technologies Inc., a Halifax-based clean-tech company that aims to shake up the global concrete industry.

Concrete is the most widely used building material in the world. But it is also responsible for roughly 5% of the world's greenhouse-gas emissions, making it the second-largest industrial emitter behind coal. Mr. Niven says his company can help change that.

CarbonCure's technology infuses carbon dioxide (the evil byproduct of industry) into the concrete-making process. The injection of CO_2 during the curing stage produces limestone, which actually makes the concrete stronger. Mr. Niven says the process also results in fewer defects, and producers use less energy and cement during the production phase.

Plus, the resulting concrete blocks serve as storage units for unwanted carbon dioxide, which is drawn from industrial waste sources such as power plants.

"We make concrete that is really indistinguishable from other concrete products," Mr. Niven said in an interview at his Halifax office. "We make a better quality product, at a lower price. And it's green. It's that triple-win that makes it really attractive to producers."

CarbonCure's "bolt on" technology is now being demonstrated at three large concrete plants, in Halifax, San Francisco and Toronto. And CarbonCure blocks are installed in a growing number of buildings, including in Halifax at a new high school and on the Dalhousie University campus.

In June, CarbonCure blocks were installed in a section of the new Hullmark Centre Complex in Toronto, which is being developed by high-rise developer Tridel.

Tower Labs, an organization that uses real world buildings as test labs for new green materials, arranged the Tridel installation. Jamie James, Tower Labs' founder, said he has discussed CarbonCure's technology with a number of key players in the concrete industry.

(Quentin Casey / Financial Post, 17 September 2012, http://business.financialpost.com/2012/09/17/carboncure-makes-concrete-go-green-without-compromise)

«Μικρά μουσεία οι σταθμοί του μετρό Θεσσαλονίκης»
Πλούτο ευρημάτων έφερε στο φως η αρχαιολογική σκαπάνη

«Όλοι οι σταθμοί του μετρό Θεσσαλονίκης θα μπορούσαν να γίνουν μουσεία, μικρά ή μεγαλύτερα. Αν τα αρχαιολογικά ευρήματα στο μετρό της Αθήνας συνέβαλαν στη φήμη του έργου παγκόσμια, αυτά που βρέθηκαν στο μετρό Θεσσαλονίκης θα το κάνουν πρώτο σημείο αναφοράς διεθνώς, αν αναδειχθούν σωστά».

Με τη φράση αυτή, ο πρόεδρος της ΑΤΤΙΚΟ ΜΕΤΡΟ ΑΕ, Χρήστος Τσίτουρας, αναφέρθηκε στον πλούτο των ευρημάτων, που έφερε στο φως η αρχαιολογική σκαπάνη στα εργοτάξια του μετρό Θεσσαλονίκης, τα οποία μπορεί μεν να έχουν προκαλέσει καθυστερήσεις στο έργο, αλλά δεν παύουν να είναι πολύ σημαντικά, μεταδίδει το ΑΜΠΕ.

94.000 κινητά ευρήματα

Πάντως, μέχρι το τέλος του 2012, οι ανειλημμένες ανασκαφικές υποχρεώσεις της 9ης Εφορίας Βυζαντινών Αρχαιοτήτων απέναντι στο έργο πρόκειται να ολοκληρωθούν, όπως ανακοίνωσε η διευθύντριά της, Δέσποινα Μακροπούλου, μιλώντας σε εκδήλωση του ΤΕΕ Κεντρικής Μακεδονίας (ΤΕΕ/ΤΚΜ). Συνολικά, ο αριθμός των κινητών ευρημάτων ξεπέρασε, όπως είπε, τις 94.000. «Δεν είναι όλα τα στραβά ψωμιά της νύφης. Οι αρχαιολογικές υπηρεσίες δεν φταίνε για την καθυστέρηση του μετρό», υποστήριξε η κ. Μακροπούλου.

Στο μεταξύ, συνολική έκταση 30.800 τετραγωνικών μέτρων, έχουν διερευνήσει ανασκαφικά, από το 2006 μέχρι σήμερα, τα συνεργεία της ΙΣΤ' Εφορίας Προϊστορικών και Κλασικών Αρχαιοτήτων, όπως επισήμανε η προϊσταμένη της, Βασιλική Μισαηλίδου-Δεσποτίδου.

Υπενθύμισε ότι οι αρχαιολογικές υπηρεσίες είχαν, κατά τον σχεδιασμό ακόμη του έργου, εκφράσει τις αντιρρήσεις τους για την πορεία του από την σημερινή οδό Εγνατία, χωρίς βέβαια να εισακουσθούν- η ΙΣΤ ΕΠΚΑ από πολύ νωρίς είχε προτείνει την οδό Τσιμισκή ως πιο «ανώδυνη» διέλευση.

Σύμφωνα με την κα Μισαηλίδου-Δεσποτίδου, μέχρι σήμερα έχει ολοκληρωθεί η ανασκαφική έρευνα στους σταθμούς «Νέος Σιδηροδρομικός Σταθμός», «Σιντριβάνι», «Πανεπιστήμιο», «Ευκλείδης» και «Φλέμινγκ», καθώς και στη Διασταύρωση Τροχιογραμμών του σταθμού «Σιντριβάνι», στο χώρο μπροστά από την κεντρική βιβλιοθήκη του ΑΠΘ και στη Διακλάδωση προς Σταυρούπολη του σταθμού «Δημοκρατίας». Οι παραπάνω θέσεις έχουν παραδοθεί στο τεχνικό τμήμα του έργου.

Σε εξέλιξη βρίσκονται οι ανασκαφές στους σταθμούς «Αγίας Σοφίας» και στο αμαξοστάσιο της Πυλαίας, ενώ στο σταθμό «Δημοκρατίας» αναμένεται το τεχνικό έργο για να ολοκληρωθεί η ανασκαφή», ανέφερε.

Χιλιάδες τάφοι στα «σπλάχνα» της πόλης

Αναλυτικότερα:

- Στο εργοτάξιο του Νέου Σιδηροδρομικού Σταθμού η ανασκαφική έρευνα βεβαίωσε την ανθρώπινη δραστηριότητα σε ένα ευρύ χρονολογικό πλαίσιο και ιδιαίτερα την ταφική χρήση του χώρου από τα τέλη του 4ουαι. π.Χ. έως και τον ύστερο 3οαι.μ.Χ. Αποκαλύφθηκαν 193 τάφοι και ταφικοί περίβολοι, που αποτελούν τμήμα του δυτικού νεκροταφείου της αρχαίας Θεσσαλονίκης.

- Στη διακλάδωση προς Σταυρούπολη του σταθμού Δημοκρατίας, αποκαλύφθηκαν συνολικά 218 τάφοι, τμήματα τοίχων-ταφικών περιβόλων, λιθοσωροί, λάκκοι και πηγάδια. Στον σταθμό Δημοκρατίας καθεαυτόν, ήρθε στο φως τμήμα του δυτικού νεκροταφείου της αρχαίας Θεσσαλονίκης το πλησιέστερο προς το δυτικό τείχος της αρχαίας πόλης, ενώ η έρευνα δεν έχει ολοκληρωθεί.

- Στον σταθμό «Αγία Σοφία» ανασκάφηκαν οικιστικά και οικοδομικά κατάλοιπα, όπως αποχετευτικοί αγωγοί, απορριμματικοί λάκκοι, κατάλοιπα τοίχων και κτιστές κατασκευές, καθώς επίσης ένας χωμάτινος δρόμος, που μαρτυρούν την αδιάλειπτη χρήση του χώρου, ήδη από τους ελληνιστικούς χρόνους.

- Στο Συντριβάνι, αποκαλύφθηκε σημαντικός αριθμός τάφων (1.150) και αρχιτεκτονικά λείψανα όπως κτιριακά κατάλοιπα, κατασκευές, τοίχοι, λάκκοι και αγωγοί, ενώ τεκμηριώθηκε ανασκαφικά η διέλευση χειμάρρου από την περιοχή. Στη διασταύρωση τροχιογραμμών του σταθμού «Σιντριβάνι», ερευνήθηκε ένα ακόμη εκτεταμένο τμήμα του ανατολικού νεκροταφείου της αρχαίας Θεσσαλονίκης. Εντοπίστηκαν 1.100 τάφοι, οικοδομικά λείψανα κτηρίου και σποραδικά αρχιτεκτονικά κατάλοιπα.

- Στον σταθμό «Πανεπιστήμιο» η γη «έκρυβε» υπόγειο κτίσμα ορθογώνιας κάτοψης και περιμετρικά αυτού αρχιτεκτονικά κατάλοιπα και λιθοσωρούς, καθώς και ένα λιθόκτιστο πηγάδι.

- Στον σταθμό «Ευκλείδη», η έρευνα έφερε στο φως αποσπασματικά σωζόμενα οικοδομικά λείψανα των όψιμων οθωμανικών χρόνων, ενώ κατά τη διάρκεια των εργασιών για τη διαμόρφωση του ευρύτερου εργοταξιακού χώρου αποκαλύφθηκε τμήμα κτηρίου (τοίχοι και αγωγοί), που χρονολογείται στους νεότερους χρόνους (τέλος 19ου-μέσα 20ου αι.).

- Στον σταθμό «Φλέμινγκ» η ανασκαφή πραγματοποιήθηκε σταδιακά στον χώρο όπου έγινε η μετατόπιση δικτύων κοινής ωφέλειας, στο χώρο που καταλαμβάνει ο κυρίως σταθμός και στην ανατολική είσοδο του σταθμού αποκαλύπτοντας τμήμα νεκροταφείου της ρωμαϊκής περιόδου (2ος – 4ος αι. μ.Χ.).

- Στον σταθμό «Ανάληψη» παρακολουθούνται οι εκσκαφικές εργασίες στο σταθμό κατά τις οποίες έχουν προκύψει νεώτερα αρχιτεκτονικά κατάλοιπα του 20ου αι.

- Στο Αμαξοστάσιο Πυλαίας, τέλος, η ανασκαφή στον κεντρικό τομέα της περιοχής όπου πρόκειται να κατασκευαστεί αυτό, έφερε στο φως τμήμα οργανωμένου προκασσάνδρειου πολίσματος.

Ο διευθυντής έργων του μετρό Θεσσαλονίκης, Γιώργος Κωνσταντινίδης, ανακοίνωσε ότι, με υπουργική Απόφαση της 30ης Απριλίου 2012, το μετρό θα πρέπει να αναδείξει τμήμα των αρχαιολογικών ευρημάτων.

(newsbeast.gr, Τετάρτη, 26 Σεπτεμβρίου 2012, http://www.newsbeast.gr/greece/arthro/419091/mikra-mouseia-oi-stathmoi-tou-metro-thessalonikis)

Photographer Offers New Perspective on Underground

By studying cities from the point of view of their underground structures, photographer Steve Duncan sheds new light on the often forgotten infrastructure that keeps our cities running.

The Knickerbocker Avenue Extension Sewer, constructed in 1885 beneath the Williamsburg section of Brooklyn, is one of many subterranean marvels Duncan explores.

It was when the water hit chest level that Steve Duncan began to get scared. That was 10 years ago, and the young explorer, then in his early twenties, was somewhere in Queens, New York, underground in a storm drain tunnel. The tide from the Atlantic Ocean was rolling in. Only he didn't know it.

Duncan had gotten interested in exploring the underbellies of cities, and he and a friend had found an inconspicuous manhole entrance in a park in Queens—more convenient than the typical middle-of-the-street variety, which are far more visible to passersby and far more difficult to access. The double channel they entered was 8 ft high and 12 ft wide. Its scale blew him away. "If it was on the surface it would be a landmark as a giant engineering project," he says.

Duncan found and photographed an abandoned Amtrak spur tunnel on the west side of Manhattan, near 40th Street, that once conveyed freight to barges that crossed the Hudson River before any bridges or tunnels had been built across the river.

The park they had entered was close to the ocean side of the borough; Queens is unusual in that its sewer tunnels and storm drain tunnels are separate. The pair had entered the tunnels to explore them at what turned out to be low tide and had been underground for several hours. As they slowly began making their way back to the manhole, Duncan noticed water was flowing in the wrong direction—back down the tunnel rather than out to sea.

As the water begin filling the tunnel, they tried to make their way back. They didn't have an alternate exit plan. They tried pushing up against another manhole they found but it was stuck; cars continually pound down the manholes in the roads.

New York's first subway station was one of its most ornate; situated on a curving platform beneath City Hall, the station opened in 1904 and closed in 1945.

At half a mile from the exit, the water was rising, coming in stronger, and had reached chest level. "We were terrified," Duncan says. "Being in a tunnel makes everything seem a lot scarier." The tunnel was dark, except for their headlamps. As water rushed in, eels came too, slithering past Duncan's ankles. A "vicious current" of water was battering them, he says. "We realized [that] pretty soon we wouldn't be able to fight it."

Duncan wanted the two to go farther inland, tie themselves to something, and wait the water out. His friend pushed them to try one more manhole cover they had stumbled past. This one gave, and the two emerged, half-soaked, on a quiet street in Queens. A mom in a minivan drove past with a look of disapproval. But Duncan and his friend rejoiced that they had escaped.

The 1837 Essex County Jail in Newark, New Jersey—designed by architect John Haviland and added to the National Register of Historic Places in 1991—features both aboveground and belowground cells. It hasn't been used since 1989, and was damaged by fire in 2001.

Looking back, Duncan notes it as a "real lesson on the extent of my ignorance"—where else, after all, will tides come in but through a giant ocean drainage channel? And there were no high-water marks in the tunnels, so it's likely the water would have filled the tunnel entirely; they would have drowned.

But there was another lesson from his misadventure—a realization of just how invisible the world he'd begun to explore really was. "If we did die down there it might be a long time before anybody found out what had happened."

Still, Duncan's brush with death did not deter him from what's become both a professional and a personal vocation -the exploration of hidden, buried, and sometimes lost and forgotten infrastructure systems in cities, systems that lie right beneath our feet. In fact, there are a growing number of such urban explorers, adventurers who explore our cities' hidden and marginalized spaces. Some of the photographs circulating online get grouped under such vivid tags as "decay porn" or "ruin porn," reflecting a growing fascination with documenting or romanticizing ruins, and the adventure of finding them.

But Duncan, who is now 33, has a different objective. As an urban historian, he has the training to put such places within historical context, and to reveal the ways in which these largely invisible tunnels and corridors still shape us. "Systems that come to us from 50 years or 100 years or 150 years [ago]," he explains, "also shape how cities function and how we live in the present day."

The New Milford water filtration plant, in Oradell, New Jersey, was built by the Hackensack Water Company in 1882, and in the 1920s pioneered the activated carbon filtration treatment process. Closed in 1990, its buried infrastructure remains as a testament to 20th-century engineering.

Urban infrastructure, he observes, remains largely the domain of engineers, and as systems become more and more complicated, there's a sense that the average citizen lacks the ability to understand it. Civic infrastructure, he says, used to be a more open topic of public interest. When major Brooklyn sewers were constructed in the 1890s, for instance, more than 10,000 people got tours of construction sites.

"Today you don't get that interest or accessibility," he says. To Duncan, that comes at a price: most people, he says, "don't realize how deteriorated a lot of our underground infrastructure already is.

"Nobody in cities will pay for big infrastructure projects until they see it as a necessary response to catastrophe," he adds. "I'd love it if we all got more educated about it to the point where we sometimes wanted to pay for big projects before it was almost too late."

(T.R. Witcher / ASCE Civil Engineering Website, http://www.asce.org/CEMagazine/Article.aspx?id=25769810840&utm_campaign=Mkt-20120907-SeptDigital-CEASCENews&utm_medium=email&utm_source=Eloqua)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΛΟΙΠΑ

UTA researchers target road expense deterioration

Stefan Romanoschi, an associate professor in the University of Texas at Arlington's department of civil engineering, designed and built this accelerated pavement testing machine. It will test recycled products in pavement for the Texas Department of Transportation.

A University of Texas at Arlington civil engineering professor is addressing Texas' $6.3 billion annual tab for roads and its interest in being more environmentally friendly.

Stefan Romanoschi and his research team have devised a pavement testing machine that accelerates the deterioration of road materials, and with a $1.12 million grant from the Texas Department of Transportation, they will test the durability of pavement mixes that include recycled materials.

Ash from coal power plants, slag from blast furnaces, asphalt shingles from old roofs or production plant scraps, and chunks of road surfaces will be ground and incorporated into parallel strips of pavement at a new testing center in Fort Worth.

By increasing pressure and frequency of tire rotations, the machine Romanoschi helped build can document a decade's worth of pavement deterioration in three or four weeks.

The 30-ton electrically powered testing machine is the first of its kind in Texas. With its large steel frame, it looks like the body of a semitruck.

A motor pulls the central carriage back and forth every six seconds at a speed of around 8 mph. The frame exerts 5,000 or more pounds of pressure on the two-wheel axle to simulate pavement stress and measure durability. There are also controls for temperature.

Two graduate students assist with the research. A full-time technician will oversee the machine's maintenance.

"What seems like an academic enterprise has huge implications for the state," said Romanoschi, who has been studying pavement for over 15 years and has a photo of cracked concrete hanging outside his office.

Because bitumen, a heavy byproduct in the oil refining process, is a major pavement component, the price of asphalt tracks the price of crude oil, said Darren Hazlett, deputy director of TxDOT's construction division.

"So when the price of crude oil goes up a whole lot, like it is currently and has for the past four, five, six years, the price of asphalt goes up," Hazlett said. "In Arlington they're trying to look to see how we can extend and save resources."

Even a 1 percent reduction in costs will make a huge difference.

"The sooner we get better results, the sooner the state can start saving," Romanoschi said.

Accelerated pavement testing gives states much more confidence in road building, said Bouzid Choubane, the pavement engineer for the Florida Department of Transportation.

"You can get meaningful results — on construction practices, on materials, on design — in a much shorter time," said Choubane, former chairman of the Transportation Research Board's Full Scale and Accelerated Pavement Testing Committee.

In October, once the road pad is prepared for the UTA research team's recycled pavement experiment, the machine will be hitched up to a trailer and moved to the testing center. The machine can be moved to test pavement anywhere in the state.

The Advanced Pavement Research Center is owned and operated by the university. It hopes for future clients that might include tire companies and asphalt producers.

(Eden Stiffman / DALLAS NEWS, 02 September 2012, http://www.dallasnews.com/news/community-news/arlington/headlines/20120902-uta-researchers-target-road-expense-deterioration.ece

Επάρκεια για χιλιάδες χρόνια
Τεράστιο κοίτασμα βιομηχανικών διαμαντιών ήρθε στο φως στη Σιβηρία

Ένα τεράστιο κοίτασμα βιομηχανικών διαμαντιών στη Σιβηρία, η ύπαρξη του οποίου είχε κρατηθεί μυστική επί δεκαετίες, διαθέτει τεράστια αποθέματα, ικανά να προκαλέσουν μια επανάσταση στην παγκόσμια βιομηχανία, σύμφωνα με τους ειδικούς. Με αποθέματα που θα μπορούσαν να επαρκέσουν για χιλιάδες χρόνια, τα διαμάντια στον κρατήρα του Ποπιγκάι είναι σε θέση να προκαλέσουν επανάσταση στη βιομηχανία.

Το κοίτασμα του Ποπιγκάι ανακαλύφθηκε στις αρχές της δεκαετίας του 1970 σε μια ερημική και ακατοίκητη ζώνη της ανατολικής Σιβηρίας, σε απόσταση 400 χιλιομέτρων από το κοντινότερο χωριό, το Καντίγκα, και 2.000 χιλιομέτρων βορείως του Κρασνογιάρσκ, της επαρχιακής πρωτεύουσας.

Λόγω του Ψυχρού Πολέμου το κοίτασμα χαρακτηρίστηκε αμέσως στρατηγικό απόθεμα της τότε ΕΣΣΔ και η ανακάλυψη περιβλήθηκε με τη μεγαλύτερη μυστικότητα.

Το Ινστιτούτο Γεωλογίας και Μεταλλευμάτων Σομπόλεφ του Νοβοσιμπίρσκ της Σιβηρίας δημοσίευσε στις αρχές της εβδομάδας νέες πληροφορίες για το ορυχείο το οποίο βρίσκεται σε έναν κρατήρα διαμέτρου περίπου εκατό χιλιομέτρων.

Ορυχείο διαμαντιών στη Ρωσία

Ο κρατήρας δημιουργήθηκε από την πτώση ενός αστεροειδή πριν από 35 εκατομμύρια χρόνια.

Το κρουστικό κύμα από την πρόσκρουση μετασχημάτισε αυτομάτως τον γραφίτη του υπεδάφους σε μικροσκοπικά διαμάντια, σε ακτίνα δέκα χιλιομέτρων γύρω από το σημείο της πτώσης του αστεροειδούς.

Σύμφωνα με τον Νικολάι Ποχιλένκο, τον διευθυντή του Ινστιτούτου Σομπόλεφ, αυτά τα «βιομηχανικά διαμάντια», που έχουν διάμετρο 0,5-2 χιλιοστά, μοιάζουν με κόκκους σκόνης αλλά είναι γκρίζα, γαλάζια και κίτρινα. Τα διαμάντια του τύπου αυτού χρησιμοποιούνται ευρέως στη βιομηχανία, στις γεωτρήσεις και στην κατασκευή αεροσκαφών.

Όμως τα αποθέματα του Ποπιγκάι σε καράτια είναι 110 φορές περισσότερα από τα παγκόσμια αποθέματα διαμαντιών, σύμφωνα με τους ειδικούς του Ινστιτούτου. Επιπλέον, τα διαμάντια του Ποπιγκάι είναι δυο φορές πιο ανθεκτικά από τα συνηθισμένα βιομηχανικά διαμάντια, εξήγησε ο Ποχιλένκο.

Οι Σοβιετικοί ειδικοί γνώριζαν ότι τα διαμάντια του Ποπιγκάι είναι πιο ανθεκτικά από τα τεχνητά αλλά την εποχή εκείνη «προτιμούσαν να κατασκευάζουν εργοστάσια συνθετικών διαμαντιών και το ορυχείο αφέθηκε στην κατάσταση που ήταν», είπε ο Ποχιλένκο.

Το κοίτασμα εγκαταλείφθηκε και ξεχάστηκε επί 30 χρόνια μέχρι που, το 2009, το Ινστιτούτο Σομπόλεφ αποφάσισε να ασχοληθεί και πάλι μαζί του. Μέχρι σήμερα το 0,3% του κρατήρα που έχει ερευνηθεί διαπιστώθηκε ότι μπορεί να δώσει 147 δισεκατομμύρια καράτια, ενώ τα παγκόσμια αποθέματα διαμαντιών εκτιμώνται στα 5 δισεκατομμύρια.

«Με το σημερινό ρυθμό χρήσης των βιομηχανικών διαμαντιών, τα αποθέματα του Ποπιγκάι καλύπτουν τις ανάγκες για τα επόμενα 3.000 χρόνια» και θα μπορούσαν να υποκινήσουν «μια βιομηχανική επανάσταση στον κόσμο», ιδίως στην κατασκευή αεροσκαφών και αυτοκινήτων, πρόσθεσε ο Ποχιλένκο.

Ωστόσο οι ειδικοί εκτιμούν ότι η εκμετάλλευση του Ποπιγκάι ενδέχεται να αποδειχθεί ασύμφορη επειδή το κοίτασμα βρίσκεται σε μια ζώνη μονίμως παγωμένου εδάφους (permafrost) και δεν υπάρχουν κοντά δρόμοι ή σιδηροδρομικό δίκτυο.

(Newsroom ΔΟΛ, με πληροφορίες από ΑΠΕ / Γαλλικό, 19 Σεπτεμβρίου 2012, http://news.in.gr/science-technology/article/?aid=1231213823)

Curiosity : Αποστολή στον Αρη

Την 6η Αυγούστου 2012 το διαστημόπλοιο Curiosity προσεδαφίστηκε («προσαρειώθηκε») στον κρατήρα Gale του Άρη, αφού διήνυσε 567 εκατομμύρια χιλιόμετρα σε 255 ημέρες (εκτοξεύτηκε από το ακρωτήριο Κανάβεραλ στις 26 Νοεμβρίου 2011). Λίγα λεπτά μετά την προσεδάφιση μεταδόθηκαν και οι πρώτες ασπρόμαυρες εικόνες από τον Άρη.. Για να φτάση ένα ραδιοσήμα από το «Curiosity» στη Γη απαιτούνται 13.8 λεπτά.

Ο κρατήρας Gale ευρίσκεται κοντά στον ισημερινό του Άρη και η διάμετρός του είναι 155 km. Το υψόμετρο του κεντρικού βουνού του είναι 5.000 m. Εικάζεται πως κάποτε φιλοξενούσε μια λίμνη.

Στην παρακάτω ιστοσελίδα παρουσιάζεται μιά πολύ ενδιαφέρουσα σύνθεση της αποστολής και προσεδάφισης του διαστημοπλοίου στον Άρη.

http://www.youtube.com/embed/XRCIzZHpFtY?rel=0

ΗΛΕΚΤΡΟΝΙΚΑ ΠΕΡΙΟΔΙΚΑ

http://www.issmge.org/attachments/article/525/ISSMGE_Bulletin_August_2012%20Vol%206%20Issue%204.pdf

Κυκλοφόρησε το Τεύχος 4 του 6ου Τόμου του ISSMGE Bulletin (Αυγούστου 2012) με τα παρακάτω περιεχόμενα:

- Note from TC 303
- President's Reports
- Technical Article: Preliminary Understanding of the Seti River Debris-Flood in Pokhara, Nepal, on May 5th, 2012 - A Report based on a Quick Field Visit Program
- Obituary: Robert V. Whitman 1928-2012
- News: 2012 SHAMSHER PRAKASH award winners
- News: Candidate for 2012 Shamsher Prakash Annual Prize for Excellence in Teaching of Geotechnical Engineering
- News on Recent Conferences: Second International Conference on Performance-based Design in Earthquake Geotechnical Engineering
- Event Diary
- Corporate Associates
- Foundation Donors
- From the Editor – Call for articles

No. 19 – September 2012
http://www.isrm.net/adm/newsletter/ver_html.php?id_newsletter=76&ver=1

Κυκλοφόρησε το Τεύχος 19 / Σεπτέμβριος 2012 του Newsletter της International Society for Rock Mechanics. Περιεχόμενα:

- A Tribute to John Franklin, 1940-2012
- Update on the 7th Asian Rock Mechanics Symposium - ARMS7, Seoul, Korea
- ISRM 50th anniversary celebrations to end in October in Salzburg
- Call for Abstracts for the ISRM 2013 International Symposium EUROCK 2013
- A successful 2nd South American Symposium on Rock Excavations took place in Costa Rica, in August
- International Conference for Effective and Sustainable Hydraulic Fracturing - HF2013 Invitation to submit an Abstract for the ISRM SINOROCK2013
- 6th Rock Stress Symposium, 20-22 August 2013, Sendai Japan - Call for Abstracts
- ISRM sponsored meetings
- ISRM Rocha Medal 2014 - nominations to be received by 31 December 2012
- New Delhi Asian Rock Mechanics Symposium Keynote lectures now online
- Wudongde hydropower station - a new 10,200MW hydropower project

http://www.icold-cigb.org/userfiles/files/NEWSLETTERS/Newsletter%2012-07-2012.pdf

Κυκλοφόρησε το Τεύχος 12 (Ιούλιος 2011) του **The Dams Newsletter** της **International Commission on Large Dams** με τα παρακάτω περιεχόμενα:

- Sucessful Congress in Kyoto
- 6th WWF stresses Water Food Energy
- The WCD question in Marseille
- WD water storage for sunstaible development
- Central Europe goes for PSP
- ICOLD regional Clubs

www.geoengineer.org

Κυκλοφόρησε το Τεύχος #92 του **Newsletter του Geoengineer.org** (Σεπτέμβριος 2012) με πολλές χρήσιμες πληροφορί-ες για όλα τα θέματα της γεωτεχνικής μηχανικής. Υπενθυμί-ζεται ότι το Newsletter εκδίδεται από τον συνάδελφο και μέλος της ΕΕΕΕΓΜ Δημήτρη Ζέκκο (http://www.geoengineer.org/geonews90.htm).

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