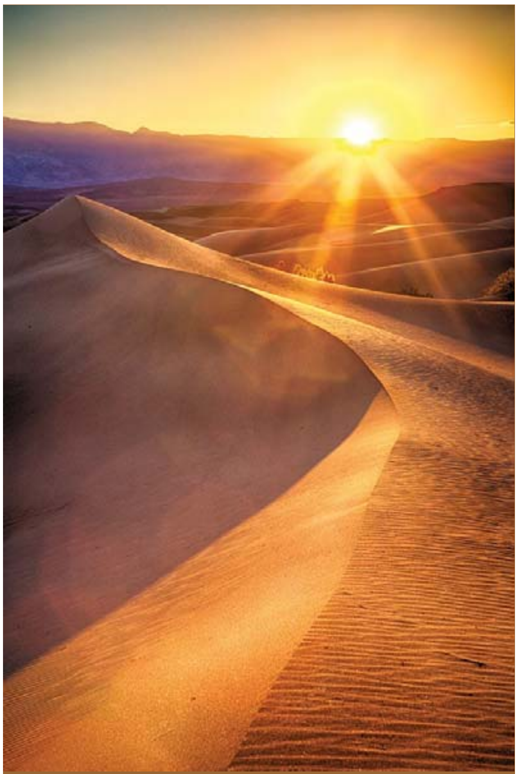

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& ΓΕΩΤΕΧΝΙΚΗΣ
ΜΗΧΑΝΙΚΗΣ

Τα Νέα της Ε Ε Ε Ε Γ Μ

83

Αρ. 83 – ΟΚΤΩΒΡΙΟΣ 2015

Αποτελέσματα Αρχαιρεσιών 21ης Οκτωβρίου 2015 για την Εκλογή νέας Εκτελεστικής και Εξελεγκτικής Επιτροπής

Στην Γενική Συνέλευση της 21ης Οκτωβρίου συμμετείχαν 31 ταμειακώς εν τάξει μέλη της ΕΕΕΕΓΜ, ενώ στις αρχαιρεσίες συμμετείχαν και 20 ακόμα μέλη με επιστολική ψήφο. Τα αποτελέσματα της ψηφοφορίας ήταν τα ακόλουθα:

ΕΚΤΕΛΕΣΤΙΚΗ ΕΠΙΤΡΟΠΗ

1. Γκαζέτας Γεώργιος	33
2. Αναγνωστόπουλος Ανδρέας	30
3. Πανταζίδου Μαρίνα	29
4. Ντούλης Γεώργιος	28
5. Βέττας Παναγιώτης	27
- Μπαρδάνης Μιχαήλ	27
7. Ξενάκη Βάλια	24
8. Παχάκης Μιχαήλ	23
9. Μπελόκας Γεώργιος	18
10. Ιωαννίδης Κωνσταντίνος	12

Οι 9 πρώτοι εκλέγονται ως τακτικά μέλη και ο δέκατος ως αναπληρωματικό μέλος.

ΕΞΕΛΕΓΚΤΙΚΗ ΕΠΙΤΡΟΠΗ

1. Αλεξανδρής Ανάργυρος	30
2. Κοζομπόλης Απόστολος	24
3. Τυρολόγου Παύλος	14

Εκλέγονται όλοι ως τακτικά μέλη.

Π Ε Ρ Ι Ε Χ Ο Μ Ε Ν Α

Αποτελέσματα Αρχαιρεσιών 21ης Οκτωβρίου 2015 για την Εκλογή νέας Εκτελεστικής και Εξελεγκτικής Επιτροπής 1

Σημείωμα του απερχομένου Προέδρου της ΕΕΕΕΓΜ 2

Άρθρα 3

- Permeation potential of colloidal silica for passive stabilization of liquefiable soils 3
- Rheological properties of colloidal silica as a means for designing passive stabilization of liquefiable soils 8
- Numerical analysis of the seismic response of sand passively stabilized against liquefaction 13
- Seismic performance of gravity retaining walls subjected to strong excitation 17
- Challenges of tunnel design and construction in the GCC region 22

Διακρίσεις Ελλήνων Γεωμηχανικών

- Προσκεκλημένη διάλεξη Ηλία Μιχάλη σε συνέδρια σε Gulf Cooperation Council Countries 27

Νέα από τις Ελληνικές και Διεθνείς Γεωτεχνικές Ενώσεις 28

- ISSMGE Webinars 28
- Associazione Geotecnica Italiana: XIV Croce Lecture 28

Προσεχείς Γεωτεχνικές Εκδηλώσεις: 29

- ICONHIC2016 - 1st International Conference on Natural Hazards and Infrastructure: Protection, Design, Rehabilitation 29
- WMESS 2016 - The World Multidisciplinary Earth Sciences Symposium 30
- 5th International Scientific Conference on Industrial and Hazardous Waste Management 31
- Shaping the Future of Geotechnical Education International Conference on Geo-Engineering Education 32
- UNSAT2018 - The 7th International Conference on Unsaturated Soils 33

Ενδιαφέροντα Γεωτεχνικά Νέα 35

- Video: Sinkhole on Queensland Beach! 35
- Huge sinkhole among houses in Hertfordshire 35

Ενδιαφέροντα – Σεισμοί 36

- Αρχαία κατάρρευση ηφαιστείου «σήκωσε τσουνάμι 180 μέτρων» 36

Ενδιαφέροντα – Περιβάλλον 38

- Ανάβρα Μαγνησίας, το Γαλατικό χωριό της Ελλάδας. Ένα χωριό παράδειγμα προς μίμηση! 38

Ενδιαφέροντα – Λοιπά 40

- Η μεγαλύτερη προβλήτα στον κόσμο! 40
- «Αραχνοΰφαντο» μέταλλο είναι κατά 99,9% αέρας 40
- Γέφυρες που προκαλούν ίλιγγο 41
- Κρεμαστή γέφυρα στη Γερμανία προκαλεί ίλιγγο! 42

Νέες Εκδόσεις στις Γεωτεχνικές Επιστήμες 44

Ηλεκτρονικά Περιοδικά 45

Σημείωμα του απερχομένου Προέδρου της ΕΕΕΕΓΜ

Αγαπητοί συνάδελφοι,

Η θητεία μου, ως Προέδρου της ΕΕΕΕΓΜ, λήγει την Τετάρτη, 11 Νοεμβρίου 2015, ημέρα που θα εκλεγή ο νέος Πρόεδρος

Τον Μάιο 2008 και τον Μάιο 2012 τα τότε μέλη της Εκτελεστικής Επιτροπής με ετίμησαν εκλέγοντας και επανεκλέγοντάς με ως Πρόεδρο της ΕΕΕΕΓΜ για δύο διαδοχικές θητείες. Όλα αυτά τα χρόνια προσπάθησα να προβάλω την Γεωτεχνική Μηχανική και το επάγγελμα του Γεωτεχνικού Μηχανικού (Γεωμηχανικού) στην ελληνική τεχνική κοινότητα, καθώς και να προβάλω την ΕΕΕΕΓΜ σε διεθνές επίπεδο. Νομίζω ότι κάτι καταφέραμε

Σε εθνικό επίπεδο η παρουσία μας ήταν αισθητή, τόσο με τον μεγάλο αριθμό, όσο και με την ποιότητα των εκδηλώσεων (στις οποίες συμμετείχαν κάποια από τα μεγαλύτερα ονόματα της Γεωτεχνικής Μηχανικής παγκοσμίως), τον μεγάλο αριθμό των συμμετεχόντων σε αυτές, ιδίως νέων συναδέλφων, αλλά και με την κυκλοφορία του ηλεκτρονικού περιοδικού μας ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ, το οποίο έχει γίνει υπόδειγμα και για τις άλλες επιστημονικές ενώσεις

Σε διεθνές επίπεδο η παρουσία μας αναβαθμίστηκε σημαντικά: Διεκδικήσαμε, κερδίσαμε και διοργανώσαμε το XV European Conference on Soil Mechanics and Geotechnical Engineering που, παρά την κρίση στην οποία είχε ήδη εισέλθει η χώρα μας, είχε μεγάλη επιτυχία. Επίσης, μεγάλος αριθμός μελών της ΕΕΕΕΓΜ συμμετείχε και συμμετέχει ως key persons σε Τεχνικές Επιτροπές της ISSMGE, καθώς και σε panels διεθνών συνεδρίων.

Όλα τα παραπάνω δεν μπορούσαν να γίνουν παρά μόνο με την αμέριστη συμπαράσταση και συνεργασία των μελών των Εκτελεστικών Επιτροπών της ΕΕΕΕΓΜ, τα οποία ολόθερμα ευχαριστώ. Ευχαριστώ επίσης τις κυρίες Μαυροκεφάλου, Αθανασίου και Παντελιά της Γραμματείας του Τομέα Γεωτεχνικής της Σχολής Πολιτικών Μηχανικών ΕΜΠ για την συνεισφορά τους. Θερμά ευχαριστώ, πρέπει επίσης να πω, και στα στελέχη του γραφείου μου ΠΑΝΓΑΙΑ ΣΥΜΒΟΥΛΟΙ ΜΗΧΑΝΙΚΟΙ, και ιδιαίτερα στην γραμματέα μου κυρία Κική Κωστοπούλου, που υπήρξαν βασικό στήριγμα στην όλη προσπάθειά μου. Τέλος, ευχαριστώ την σύζυγό μου, συνεταίρο και συνάδελφο Φάνη Τσατσανίφου για την υπομονή και την ανοχή της όλα αυτά τα χρόνια, όταν πολλές φορές οι υποχρεώσεις μου στην ΕΕΕΕΓΜ προηγούντο των επαγγελματικών και οικογενειακών!

Μετά από 25 χρόνια συνεχούς παρουσίας μου στην Εκτελεστική Επιτροπή, δεν συμμετείχα στις πρόσφατες αρχαιρεσίες. Οι επαγγελματικές υποχρεώσεις μου με υποχρεώνουν σήμερα να εργάζομαι τον περισσότερο χρόνο μου στο εξωτερικό, οπότε η ενδεχόμενη επανεκλογή μου στην Εκτελεστική Επιτροπή θα αποτελούσε εμπόδιο για την εκλογή νέων συναδέλφων, που μπορούν προσφέρουν περισσότερα με την καθημερινή παρουσία τους στην Ελλάδα. Όμως δεν θα χαθώ τελείως! Η νέα Εκτελεστική Επιτροπή μπορεί να υπολογίζη σε εμένα, όπου μπορώ να βοηθήσω. Και βέβαια, εφ' όσον η Εκτελεστική Επιτροπή και ο νέος Πρόεδρος το επιθυμούν, θα συνεχίσω να εκδίδω το περιοδικό μας. Προσωπικός μου στόχος είναι να εκδώσω και το 100ο τεύχος!

Εύχομαι στον νέο Πρόεδρο και στην νέα Εκτελεστική Επιτροπή κάθε επιτυχία.

Με συναδελφικούς χαιρετισμούς

Χρήστος Τσατσανίφος

ΑΡΘΡΑ

Παρουσίαση άρθρων, στην συγγραφή των οποίων μετείχαν Έλληνες, στο XVI European Conference on Soil Mechanics and Geotechnical Engineering, Edinburgh, 13-17 September 2015 (κατ' αλφαβητική σειρά του ονόματος του πρώτου συγγραφέα).

Permeation potential of colloidal silica for passive stabilization of liquefiable soils

Potentiel d'infiltration de la silice colloïdale pour la stabilisation passive des sols liquéfiables

G.I. Agapoulaki , A.G. Papadimitriou , K. Kandris and M. Pantazidou

ABSTRACT Passive stabilization is a new ground improvement technique against liquefaction. It consists of the low pressure injection of colloidal silica in the pores of a liquefiable soil. After well-controlled time, colloidal silica becomes a firm gel and changes the mechanical properties of the soil, making it less vulnerable to strain accumulation and strength degradation related to liquefaction. The effectiveness of the technique relies greatly on the ability to inject (or permeate) the colloidal silica into the soil, well before its gelation. This paper first investigates experimentally the factors affecting the permeation of colloidal silica, via 1D permeation tests in granular soil columns. Then, a simple analytical tool is proposed for estimating the (time-variable) flow rate of colloidal silica through the soil, which is based on Darcy's law after adjustments for the differences in viscosity and density of colloidal silica as compared to that of water. The predictions with this tool agree well with the measurements of the 1D permeation tests.

RÉSUMÉ La stabilisation passive est une nouvelle technique d'amélioration du sol contre la liquéfaction. Elle consiste en l'injection à basse pression de silice colloïdale dans les pores d'un sol liquéfiable. Après un temps bien contrôlée, la silice colloïdale se transforme en gel ferme et modifie les propriétés mécaniques du sol, le rendant moins vulnérable à l'accumulation de déformation et à la dégradation de la résistance liée à la liquéfaction. L'efficacité de la technique dépend fortement de la capacité d'injecter (ou de faire infiltrer) la silice colloïdale dans le sol, bien avant sa gélification. Cet article examine d'abord sur le plan expérimental les facteurs qui influent sur l'infiltration de la silice colloïdale, par des essais d'infiltration 1D dans des colonnes de sol granulaire. Ensuite, un outil d'analyse simple est proposé pour estimer la vitesse d'écoulement (variable dans le temps) de silice colloïdale dans le sol, qui est basé sur la loi de Darcy, après ajustements pour les différences de viscosité et de la densité de la silice colloïdale par rapport à celle de l'eau. Les prédictions avec cet outil s'accordent bien avec les mesures des essais d'infiltration 1D.

1. ON PASSIVE STABILIZATION

Passive stabilization is a new concept of ground improvement based on the low pressure injection (or permeation) of a stabilizer, named colloidal silica (CS), in the pores of a granular soil (Gallagher 2000). Colloidal silica is an aqueous dispersion of silica nanoparticles. Such dispersions have time-increasing viscosity, starting from values similar to that of water (slightly higher than 1cP) thus allowing permeation. However, the viscosity of the material increases rapidly after it reaches a value of 3 to 6cP, transforming itself into a firm gel (Gallagher & Lin 2005, Lin & Gallagher 2006). Once the CS has gelled in the pores, the soil becomes much more resistant to strain accumulation and strength degradation related to liquefaction. The time needed for the CS dispersion to firmly gel is called gel time (t_g) and depends on pH, ion normality, temperature and the percentage per weight of colloidal silica, CS(%), in the dispersion (Agapoulaki and Papadimitriou 2015). However, the time before gelation can be used to inject the material, either by natural groundwater flow or by using boreholes as injection and extraction wells on either side of the foundation area under stabilization. Based on the above, the concept here is to use low pressures for injecting the material, i.e. not requiring special equipment or risking structural damage in the foundation.

By choosing a low pressure injection scheme, the ability of the stabilizer to permeate the granular soil becomes primarily a function of soil characteristics (e.g. permeability, porosity), as well as the rheological properties of colloidal silica (e.g. viscosity, density). Hence, the effectiveness of passive stabilization as a ground improvement method relies on the permeation potential of colloidal silica. This is investigated herein experimentally, by means of 1D permeation tests in columns of granular soil where the flow rate of the material is measured, along with its time-increasing viscosity. These tests are performed in columns with heights ranging from 20 to 100cm using different CS solutions, soil types and hydraulic gradients, and are considered complementary to similar tests conducted by: Gallagher & Lin (2005), Lin & Gallagher (2006).

2. EQUIPMENT AND MATERIALS

The 20cm column used in this research is illustrated in Fig. 1. It consists of a main section of a diameter of 10cm and a height of 20cm, a bottom cap with two valves for the separate injection of CS and water, a top cap for the extraction of CS with one valve and two sampling ports for stabilizer fluid extracted during a test. The bottom and upper valves are connected to three chambers: an inlet (water injection), an outlet (water and/or CS extraction) and an (inlet) CS chamber (CS injection).

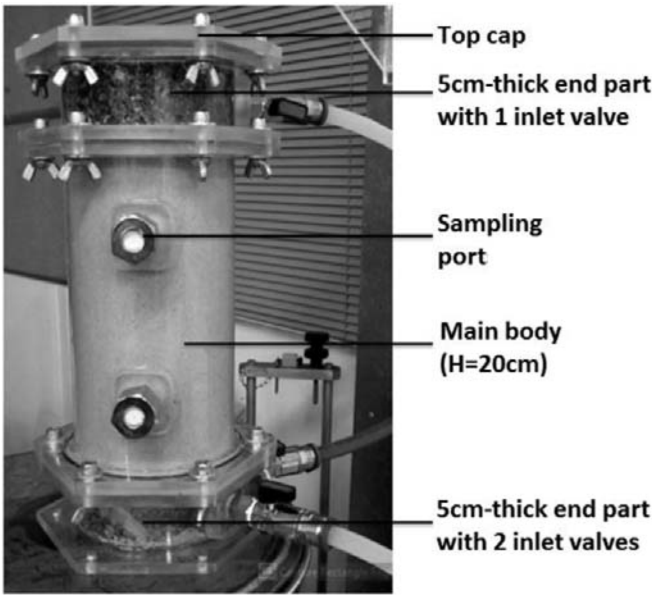

Figure 1. Equipment for vertical colloidal silica permeation testing and pore fluid sampling (example for H=20cm sample)

For the preparation of the CS solution, deionized water was enriched by Ludox®-SM colloidal silica and pure NaCl, thus forming solutions whose pH was controlled by adding (small quantities) of HCl. The pH value of the solution was measured by a Metrohm, type 826 pH mobile pH meter. During the tests, dynamic viscosity (μ_g) measurements of the CS solution were performed by employing a Brookfield LVDV-II+PX rotating viscometer.

The soil used in these studies is M31 sand with maximum and minimum densities ρ_{dmax}=1.717 g/cm^3 and ρ_{dmin}=1.455 g/cm^3, maximum and minimum void ratios e_{max}=0.826 and e_{min}=0.548 and D_{50}= 0.30 mm. For the silty sand mixtures, D6 silt has been used with D_{50}=0.05 mm.

3. COLUMN PREPARATION AND TESTING

For the preparation of sand and silty sand columns, the equipment was assembled as it is shown in Fig. 1. For the uniform permeation of pore fluid, a 5cm-thick layer of coarser soil (gravel) was placed at the bottom part of the sand column within the (5cm-thick) end part with 2 inlet valves (see Fig. 1). Beyond this layer, the sand was placed with the undercompaction method (Ladd 1978). In order to form higher columns, 20cm-tall main bodies are assembled one on top of the other, while at the very top another 5cm-thick coarser soil layer is placed within the (5cm-thick) end part with 1 inlet valve (Fig. 1).

After the preparation of the columns, saturation is achieved by running deionized water through the system at a very low gradient for two days. After saturation, and just before the CS injection, permeability of the sample under constant head was measured. The CS solution was prepared independently for the desired gel time t_g, on the basis of the guidelines of Agapoulaki and Papadimitriou (2015). Before injection it was colored with blue food dye in order to be visible inside the sample.

The goal of the experiments was to perform 1D vertical upward injection of CS using low hydraulic gradients. It has to be underlined here that due to the slightly higher density of the injected CS in comparison to that of water (pre-existing in the pores), if the CS-inlet and water-outlet levels are set at exactly the same height, then density-driven flow of CS through the sample occurs (see Post et al. 2007). If these levels are kept constant, then this density-driven flow will continue at a constantly reducing rate. Hence, in order to establish equilibrium (or no flow conditions) as the initial condition of the experiment, the CS-inlet level was initially set appropriately slightly lower than that of the water-outlet level (see illustration in Fig.6). Then, the CS-inlet level was increased slightly in order to apply a low hydraulic gradient i and thereafter the selected levels of the CS-inlet and the water-outlet chambers were retained constant throughout the test. The volume of CS required to be added in order to retain the selected height of the CS inlet level was used in order to estimate the measured flow rate or discharge (Q_{meas}) of colloidal silica within the soil sample. Concurrently, the time evolution of the (dynamic) viscosity of the CS solution was measured independently, and so was the location of the CS front within the sample. This was performed by optically observing where the dyed CS material lies within the sample, as shown in Fig. 2. Obviously, this type of optical measurement is less accurate than the measurements of flow rate; however sampling of the pore fluid through the ports (see Fig. 1) ascertains that the dyed pore fluid is the intended CS solution.

This paper presents the characteristics and the results of four (4) tests outlined in Table 1. More specifically, Table 1 contains the number of the tests, the CS properties (pH value and ion normality, since CS = 10% in all tests), the height of the column (H in cm), the hydraulic gradient (i), the dynamic viscosity, μ_g, change (increasing or constant), the gel time (t_g in hours) as the time required for the viscosity to reach a value of 4-6cP which essentially disallows flow), the permeability (k) of the sand (for clean water) and its void ratio (e).

Table 1. Characteristics of the 1D permeation tests performed

Test No.	CS[1]	H (cm)	i	μ_g[2] (cP)	t_g (h)	k *10^{-4} (m/s)	e
4	pH=5.53 0.1N	20	0.05	I	3.4	2.84	0.603
6	pH=7.00 0.1N	40	0.05	I	7.4	3.40	0.622
8	pH=5.50 0.1N	100	0.25	I	4.5	3.10	0.645
9	pH=10.00 0N	20	0.05	C	-	2.84	0.603

[1]CS(=10%) properties, [2] type: Increasing or Constant

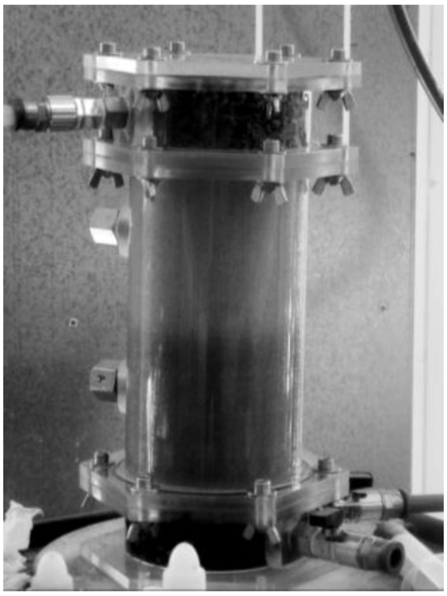

Figure 2. View of dyed colloidal silica at (approximately) midheight of sand sample during test No.4.

The time-evolution of measured flow rate Q_{meas} and dynamic viscosity μg of colloidal silica in Tests 4, 6, 8 and 9 are presented in Figs 3 through 5. The details of these figures will be discussed later. Of interest here is to underline that in Tests No 4, 6 and 8 the CS permeation stopped (the front was stable for a long time) far before anticipated and despite that the levels of water-outlet and CS-inlet were kept constant at their initial heights. This can be explained by the fact that the independent dynamic viscosity measurements showed that the μ_g value at that time was somewhere between 4 and 7cP. In other words, these tests show that the CS solution had become practically "too viscous" to continue permeation through the soil at such relatively low hydraulic gradients (i = 0.05 and 0.25). This result is in line with the literature (Gallagher & Lin 2005, Lin & Gallagher 2006), which considers permeation to be completed when viscosity of CS reaches a value of 3.6 – 4.0cP but for even lower hydraulic gradients.

On the other hand, test No. 9 shows a qualitatively similarly decreasing flow rate with time, but this is attributed to the density-driven upward flow whose discharge (or flow rate) is expected to decrease as the CS front increases through the soil (Post et al. 2007). This assertion is justified given the fact that the CS solution was tailored to retain its initial viscosity constant throughout this test.

In more detail, Fig. 3 studies the effect of different hydraulic gradients i on the measured flow rate of CS through sand. Observe that the effect of i is especially important in the initial stages of the permeation (much higher flow rate for a much larger hydraulic gradient, as expected qualitatively by Darcy's law). Nevertheless, in both tests the flow rate decreases asymptotically with time, as a combined effect of the increase of viscosity and density-driven flow. Note also that the flow rate remains much higher in Test No 8 throughout the test (due to the higher i value), but permeation essentially stops when μg reaches 5-7cP.

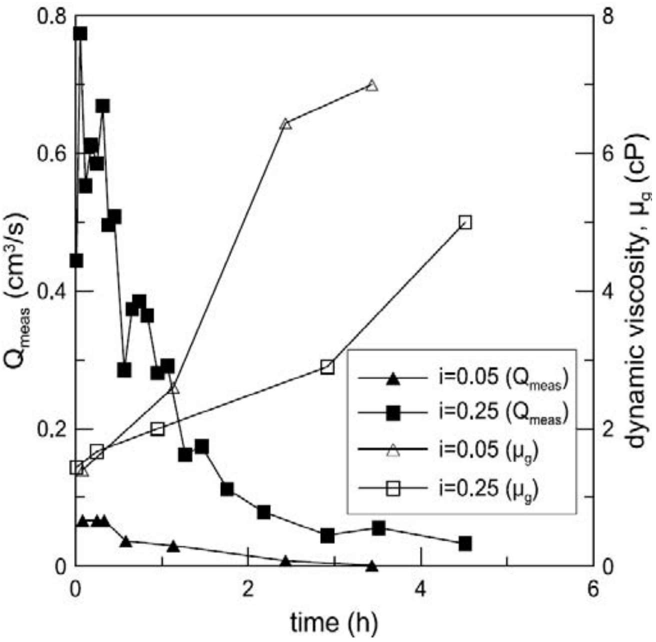

Figure 3. Effect of hydraulic gradient i on measured flow rate (with time) in sand samples permeated with colloidal silica having similar gel times (tests 4 and 8).

In an attempt to disassociate the two flow rate reducing effects, Fig. 4 compares data from Tests No. 4 (also in Fig. 3) and 9. As discussed above, the latter test had a CS with viscosity tailored to remain constantly equal to 1.47cP (its initial value). Observe here that the initial flow rates are practically identical (due to identical values of i and μg), but the flow rate of Test No. 4 with the increasing viscosity decreases faster and essentially stops after 3.4hrs when μg = 7cP. On the other hand, Test No 9 shows that permeation continues further (well after 5hrs), but at a much reduced flow rate. Finally, Fig. 5 studies the effect of different (but finite) gel times on the measured flow rate, under otherwise similar hydraulic conditions. Observe that while the initial flow rates are practically identical (due to identical values of i and μ_g), the flow rate reduces much faster with time when the increase of viscosity is faster. Also note that the permeation essentially reduces to zero when μg reaches 4-7cP, although these instances appear at distinctly different times (3.4 and 7.4 hrs) in the two tests.

4. ANALYTICAL SIMULATION OF THE 1-D COLLOIDAL SILICA PERMEATION TESTS

4.1 Conceptual and mathematical formulation

As the CS solution is delivered from the inlet chamber, it moves upwards within the soil column displacing pore water. As shown schematically in Fig. 6, over time the CS occupies a region of the soil column with height equal to L_g; this region is defined by the bottom layer of the test column and the front of the CS solution. In this region, pore space is filled with the stabilizer solution, which is heavier (stabilizer density, ρ_g > 1000 kg/m^3) and more viscous than water (stabilizer viscosity, μ_g > 1.0cP at the beginning of the injection) for usual CS percentages (e.g. $\rho_g \cong 1050$kg/m^3, $\mu_g \geq 1.25$cP for CS = 10%). In other words, the injection of a CS solution into a saturated porous medium constitutes a flow problem under viscosity and density varying conditions.

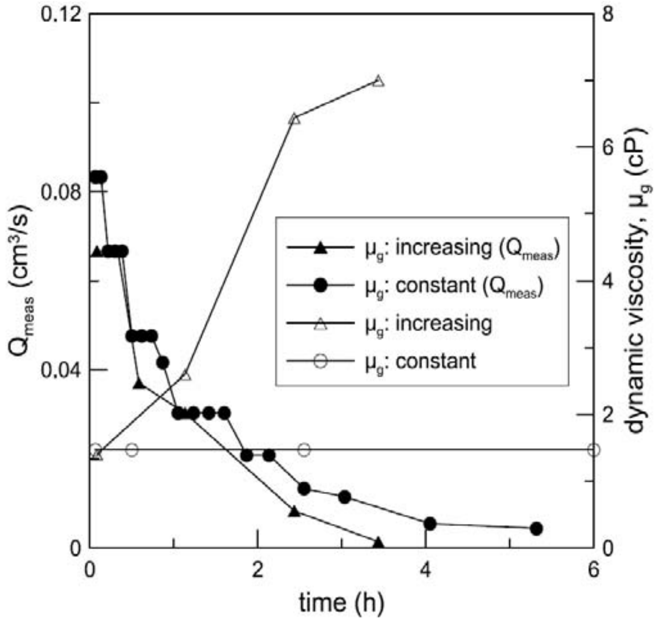

Figure 4. Effect of time-increasing dynamic viscosity of colloidal silica on its measured flow rate (with time) in sand samples (tests 4 and 9)

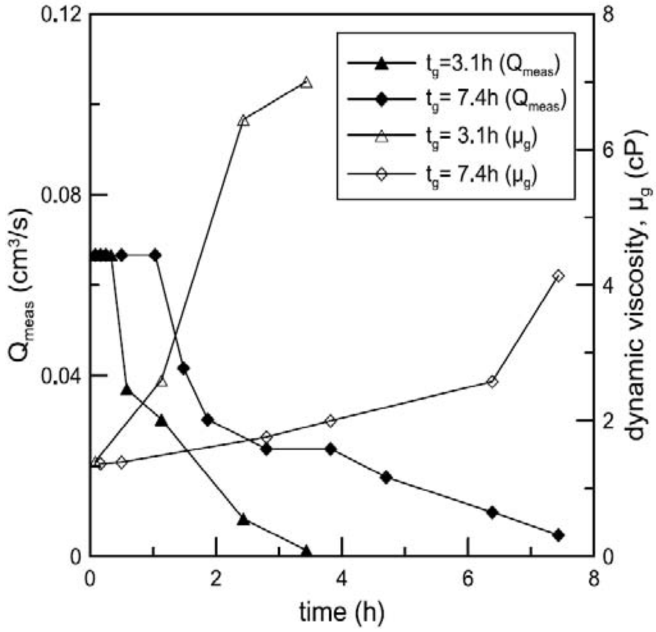

Figure 5. Effect of different gel times of colloidal silica on its measured flow rate (with time) in sand samples (tests 4 and 6)

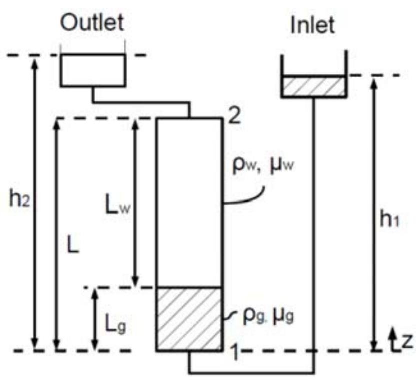

Figure 6. Schematic representation of the permeation tests, as the colloidal silica front advances within the soil column

In many mainstream numerical codes (e.g. MOCDENSE, SEAWAT) variable-density flow is assessed with the concept of equivalent fresh water heads. According to this concept, every measured hydraulic head is expressed in terms of an equivalent fresh water hydraulic head ($h_{f,i}$) according to:

$$h_{f,i} = z_i + \frac{P_i}{\rho_w g} \tag{1}$$

where z_i is the elevation head (m), P_i is fluid pressure (Pa), ρ_w is fresh water density (kg/m^3) and g is the gravitational acceleration (m/s^2). However, the concept of equivalent fresh water heads may produce erroneous flow rate estimates, especially in cases of vertical flow (Simmons 2005). Fresh water head analyses can misinterpret water flow calculations, if density variations are not properly taken into account (Post et al. 2007). To avoid such misinterpretations, Post et al. (2007) suggest the application of the more general form of Darcy's law for fluid flow in a porous medium:

$$Q_{calc} = -\frac{k}{\mu}\left(\frac{\partial P}{\partial z} + \rho g\right)A \tag{2}$$

where k is intrinsic permeability (m^2), μ is fluid dynamic viscosity (kg/m/s), ρ is fluid density (kg/m^3) and A is the cross-sectional area to flow direction (m^2). This form of Darcy's law explicitly demonstrates the two driving forces of flow, i.e. the pressure gradient (∇P) and the gravity force per unit volume (ρ_g). By differentiating and rearranging Eq. (1), Eq. (2) is equivalent to the expression:

$$Q_{calc} = -\frac{k\rho_w g}{\mu}\left[\frac{\partial h_f}{\partial z} + \left(\frac{\rho - \rho_w}{\rho_w}\right)\right]A \tag{3}$$

For the assessment of variable-density flow, Post et al. (2007) suggest the application of Equation (3) with an average fluid density that is given by:

$$\rho_\alpha = \frac{1}{z_2 - z_1}\int_{z_1}^{z_2} \rho dz \tag{4}$$

In the case of colloidal silica flow, the spatial variation of viscosity cannot be neglected. Hence, the calculation of an average viscosity term is required:

$$\mu_\alpha = \frac{1}{z_2 - z_1}\int_{z_1}^{z_2} \mu dz \tag{5}$$

According to the spatial distributions of density and viscosity depicted in Fig. 6, the average rheological properties described in Eqs (4) and (5) are given by:

$$\rho_\alpha = \frac{\left(\rho_g L_g + \rho_w L_w\right)}{L} \tag{6}$$

$$\mu_\alpha = \frac{\left(\mu_g L_g + \mu_w L_w\right)}{L} \tag{7}$$

Note that these average quantities are a function of time, since L_g increases over time. Furthermore, μα is dependent on μ_g, which also increases over time due to the gelling process that the stabilizer undergoes. Finally, if one substitutes Eqs (6) and (7) into Eq. (3), the flow rate can be estimated by the following analytical equation:

$$Q_{calc} = \frac{k\rho_w g}{\left(\mu_g L_g + \mu_w L_w\right)} \times \left\{\left[\left(\frac{\rho_g}{\rho_w}\right)h_1 - h_2\right] - \left(\frac{\rho_g}{\rho_w} - 1\right)L_g\right\}A \tag{8}$$

where h_1 and h_2 are the measured hydraulic heads corresponding to the inlet and the outlet chambers of the test, respectively (see Fig. 6). In order to test the predictive abilities of Eq. (8), in the following section its predictions are compared with the corresponding test data.

4.2 Analytical predictions versus test results

Due to length limitations, the analytical predictions of only two of the tests are compared herein with the data (in Fig. 7). Apart from the stabilizer density, the viscosity of colloidal silica and the position of its front (i.e. the height of the treated region, L_g) are experimentally measured over time.

As shown in Fig. 7, the analytical solution developed by Post et al. (2007) as modified herein reproduces satisfactorily the pattern of flow rates decreasing with time. This reduction results from (a) the increase of the height of the colloidal silica front, and (b) the increase of the stabilizer viscosity. The discrepancy between calculated and measured flow rates may be attributed to the inherently noisy measurements of the stabilizer viscosity, but mostly to the difficulty in accurately determining the height of the treated region (see Fig. 2).

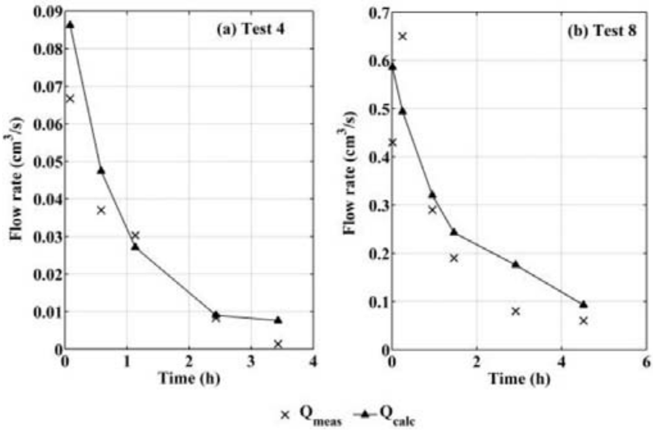

Figure 7. Measured and calculated flow rates versus time for (a) Test 4 and (b) Test 8.

5. CONCLUSIONS

From both the experimental and analytical study of colloidal silica (CS) permeation potential, the following conclusions can be drawn:

1. The CS solution needs to be tailored to the required gel time t_g in order to permeate the required distances, since viscosity values of 4–7cP essentially stop permeation (for i = 0.05 to 0.25).

2. The CS flow rate is affected by both the time-increasing viscosity and the higher density of this material in comparison to that of water.

3. The CS flow rate may be satisfactorily simulated via a simple analytical tool, which is based on Darcy's law after appropriate adjustments for the differences in viscosity and density of colloidal silica as compared to that of water.

ACKNOWLEDGEMENT

This research has been co-financed by the European Union (European Social Fund ESF) and Greek national funds through the Operational Program "Education and Lifelong Learning" of the National Strategic Reference Framework (NSRF) - Research Funding Program: Thales. Investing in knowledge society through the European Social Fund.

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Rheological properties of colloidal silica as a means for designing passive stabilization of liquefiable soils

Propriétés rhéologiques de silice colloïdale comme un moyen pour le calcul de stabilisation passive des sols liquéfiables

G.I. Agapoulaki, A.G. Papadimitriou

ABSTRACT Passive stabilization is a novel ground improvement technique for mitigating liquefaction at developed sites. It is based on the low-gradient (low pressure) injection in the soil pores of colloidal silica, an aqueous dispersion of fine silica particles. This material retains low viscosity values (slightly higher than that of water) up until a well-controlled time, when its viscosity increases rapidly and turns into a gel. When this gel forms in the soil pores, the soil is no longer susceptible to large shear strains and strength degradation related to liquefaction. The (gel) time required for the gelation of colloidal silica is a very important factor for its effective injection at the desired location. It is found to depend on different yet controllable factors such as the pH, ion concentration, ionic strength, percentage per weight of colloidal silica and temperature of the solution. This paper investigates the rheological properties of colloidal silica and quantifies these effects on the gel time as a means for designing this novel ground improvement technique for practical applications.

RÉSUMÉ Stabilisation passive est une nouvelle technique d'amélioration de sol contre la liquéfaction des sites développés. Elle se compose de l'injection (à basse pression) dans les pores du sol de la silice colloïdale, une dispersion aqueuse de particules fines de silice. Ce matériel conserve des valeurs de viscosité faible (légèrement supérieures à celle de l'eau) jusqu' au temps bien contrôlé, lorsque sa viscosité augmente rapidement et se transforme en gel. Lorsque ce gel se forme dans les pores, le sol n'est plus sensible aux grandes contraintes et à la dégradation de la résistance liée à la liquéfaction. Le temps (de gel) nécessaire à la gélification de la silice colloïdale est un facteur très important pour l'injection effective à l'endroit désiré. On trouve que ça dépend à différents facteurs qui peuvent être contrôlés, comme le pH, la concentration ionique, la force ionique, le pourcentage par poids de silice colloïdale et la température de la dispersion aqueuse. Cet article étudie les propriétés rhéologiques de la silice colloïdale, et quantifie ces effets sur le temps de gel comme un moyen pour le calcul de cette nouvelle technique d'amélioration des sols pour des applications pratiques.

1. PASSIVE STABILIZATION

Conventional liquefaction mitigation techniques are either difficult or impossible (e.g. vibro-replacement) to implement at developed sites. In such cases, underpinning, high viscosity/pressure grouting or perforated drains are often prescribed. However, these techniques cannot mitigate liquefaction under the whole structure and may create structural problems (e.g. footing heave in high viscosity/pressure grouting). In addition, the improvement of large areas is costly and may affect the normal operation of the structure. An alternative technique without these drawbacks is passive stabilization (Gallagher 2000), i.e. the low pressure injection into the soil pores of colloidal silica, i.e. an aqueous dispersion of fine silica particles. This stabilizing material retains low viscosity values (slightly higher than that of water), thus allowing its low pressure injection (or permeation) which cannot induce structural problems. In situ, this can be performed either by natural ground flow or by using injection and extraction wells (filled boreholes with different hydraulic heads) on either side of the developed site (see concept in Figure 1). After well-controlled time, the colloidal silica gels in the pores of the soil and alters the mechanical response of the soil skeleton–pore fluid system, making it less vulnerable to reduction of shear strength and plastic strain accumulation related to liquefaction. Significant data quantifying such effects were first presented by Gallagher and Mitchell (2002), whereas an overview of pertinent literature data may be found in Papadimitriou and Agapoulaki (2013). Possibly the most crucial factor for the successful application of passive stabilization is controlling the gel time (t_g) of the pore fluid. Given the low injection pressures used, if colloidal silica gels prematurely, then the stabilizer will not be able to reach the desired location, whereas if it does not gel in time the stabilizer will overtake the desired location. Hence, quality control of t_g is fundamental for making passive stabilization usable in civil engineering practice. This paper complements existing knowledge for the rheological properties of colloidal silica, and proposes design charts for its use in liquefaction mitigation.

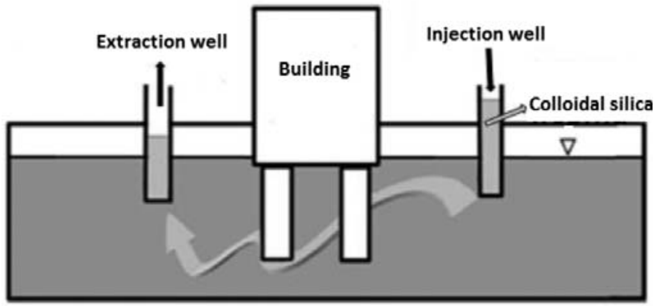

Figure 1. Concept of passive stabilization

2. COLLOIDAL SILICA

Colloidal silica (CS) is an aqueous dispersion of silica nanoparticles. It is biologically and chemically inert, non-toxic and is considered durable (stability of at least 25 years; Whang 1995). Its cost is comparable to conventional chemical grouts. Due to the small size of the silica particles and their negatively-charged surface, electro-static forces control the response of the dispersion. Stabilization is due to gelation, which occurs by the formation of siloxane (Si-O-Si) bonds between silica particles, releasing H2O into the solution. The procedure of gelation is perpetual, and the time required for its formation (gel time; t_g) depends on the electro-static interaction between the particles. Hence, it may be controlled by the factors affecting the electro-static forces between silica particles, similarly to how such factors affect the interaction between negatively-charged clay particles. For example, the introduction of NaCl in the silica dispersion reduces the double layer of water attached to the silica particles that is created by the electrostatic attraction of bipolar H_2O molecules. As a result, the repulsive forces between silica particles are reduced, thus accelerating the formation of siloxane bonds, which is macroscopically measured by an increase in the viscosity (μ) of the solution. Hence, the viscosity (μ) versus time (t) relation and the time (t_g) required for formation of a firm resonating gel are governed by the factors affecting the electro-static forces. Thus, for a given type of colloidal silica, they may be controlled by various factors, the most important of which are the percent per weight of silica CS (%), the pH, the normality of ions in the solution and its temperature (Gallagher 2000).

3. RHEOLOGICAL TESTING RESULTS

Related literature includes measurements of viscosity and gel time of various types of colloidal silica. In particular, Gallagher (2000) showed that the finer the silica particles, the lower the t_g value for given conditions and the smaller the required quantity of silica for achieving gelation. Given the breadth of the literature measurements, it is required that testing focuses on quantifying the various effects on a specific type of silica. The measurements presented here were performed at the Laboratory of Geotechnical Engineering at the University of Thessaly (UTh) in Greece, and focused on Ludox®-SM, one of the finest types of colloidal silica (mean diameter of 7nm).

3.1 Equipment and methodology

In order to investigate all the basic parameters that affect gel time as well as the curves of (μ vs. t) of colloidal silica, a large set of viscosity measurements were performed with a Brookfield LVDV-II+PX rotational viscometer (equipped with a UL adapter for measuring very low viscosities) and a Brookfield TC- 150 water bath for retaining constant temperature. Preparation of colloidal silica solutions was performed by enriching deionized water with Ludox®- SM colloidal silica and pure NaCl, thus forming solutions whose pH was controlled by adding (small quantities) of highly concentrated HCl. Table 1 presents an overview of the variation of these parameters in the performed tests at UTh.

Table 1. CS(%), NaCl concentration and pH of the tests performed

CS(%)	NaCl concentration (normality, N)	pH range
5	0	3.00 - 5.55
5	0.03N	3.00 – 6.00
5	0.1N	4.00 – 7.50
7.5	0	3.00 – 6.55
7.5	0.03N	5.00 – 6.50
7.5	0.1N	5.00 – 7.30
10	0	2.80 – 6.88
10	0.03N	3.00 – 7.55
10	0.1N	3.80 – 7.50

3.2 Typical viscosity vs time response

Figure 2 presents an overview of the viscosity versus time curves for a number of colloidal silica solutions having CS = 10%, no added salt (NS), but quite different pH values (ranging from 3.00 to 6.88). Observe that for all tested pH values the viscosity values remain initially very low (somewhat higher than that of water) and abruptly viscosity increases rapidly to values reaching a few thousands cP (not shown here for clarity). This rapid increase of viscosity turns the solution quickly into a firm resonating gel, offering stabilization to a liquefiable soil if this solution has been permeated into its pores.

While all these colloidal silica solutions have the foregoing similarities in their rheological response, they differ significantly in the time required to reach this firm gel state. There is no strict definition of gel time (t_g) in the literature. Here, the (t_g) is defined when the last measured viscosity value is greater or equal to 100cP, since the time after t_g required to reach a state of firm gel is comparatively negligible.

Given this practical definition of t_g (which is in tune with the literature, e.g. Gallagher 2000), observe in Fig. 2 that the initial stage of very low viscosity values usually runs up until $0.8t_g$ - $0.9t_g$, whereas in the remaining time of $0.1t_g$ - $0.2t_g$ the viscosity increases very rapidly. This type of response was found to apply in all tested (CS, N, pH) combinations of Table 1. Furthermore, observe in Fig. 2 that although all solutions have CS = 10% and no added salt (NS), the values of t_g range from less than 1 day up to more than 12 days, as a result of the different pH, the effect of which is studied in the next section.

3.3 Effect of pH

Based on Fig.2, the effect of pH on t_g is very important. This becomes clearer in Fig. 3, which presents the values of t_g as a function of pH for the tests in Fig. 2. Observe in Fig. 3

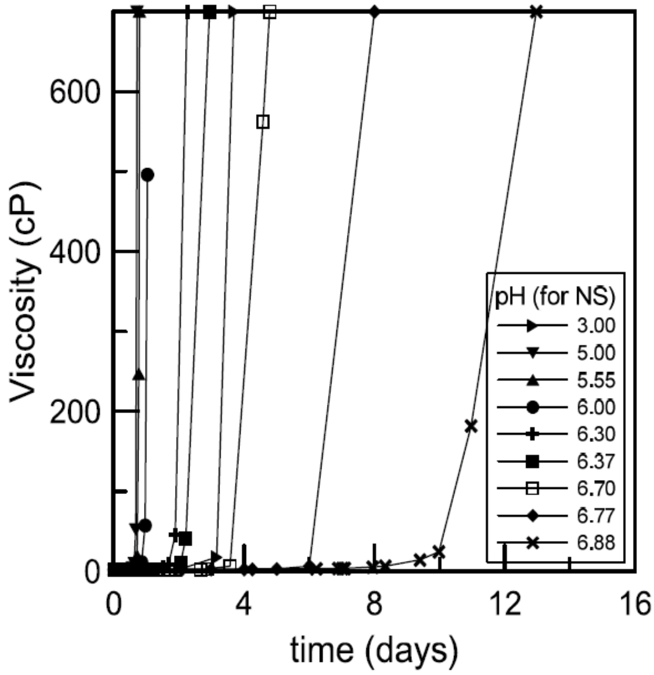

Figure 2. Typical viscosity versus time curves of colloidal silica solutions (CS=10%, no added salt, pH = 3.00 – 6.88).

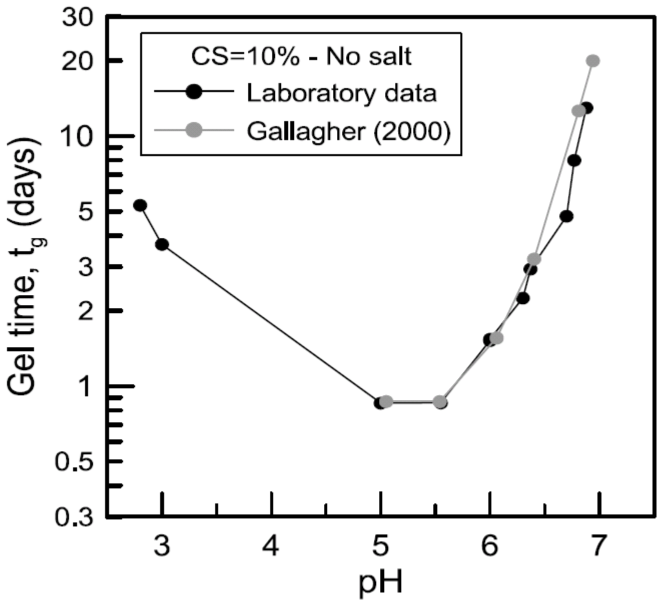

Figure 3. Typical effect of pH on gel time ("gel time curve") for colloidal silica solutions (CS=10%, no added salt, pH = 2.80 – 6.88) and comparison to literature data.

the good agreement with t_g values from the literature and that our tests are complementary to those already published. Furthermore, our tests depict that the "gel time curve" (the curve of pH vs. t_g) has the shape of a (somewhat symmetric) inverted bell, and this was found to hold true for all (CS, N) combinations outlined in Table 1.

Given these observations, for each (CS, N) combination there is an "optimum" pH value (pH_{opt}), which corresponds to a minimum gel time (t_{gmin}). This typical response for all (CS, N) combinations has been explained in the literature (e.g. Gallagher 2000). Specifically, when pH < pH_{opt} the hydroxyl ions which create the negative charge of the silica particles become uncharged and therefore gelation is delayed. On the contrary, when pH > pH_{opt} the hydroxyl ions are too many and thus repulsive forces are created between silica particles, again delaying gelation.

3.4 Effects of CS (%) and ion normality

Another important parameter controlling gel time is CS concentration. This value is also related to the cost of passive stabilization. Tests from the literature show that concentrations between 5% and 10% suffice for mitigation liquefaction in practical applications (Gallagher and Mitchell 2002), and this is what dictated the range of CS (%) values here (Table 1).

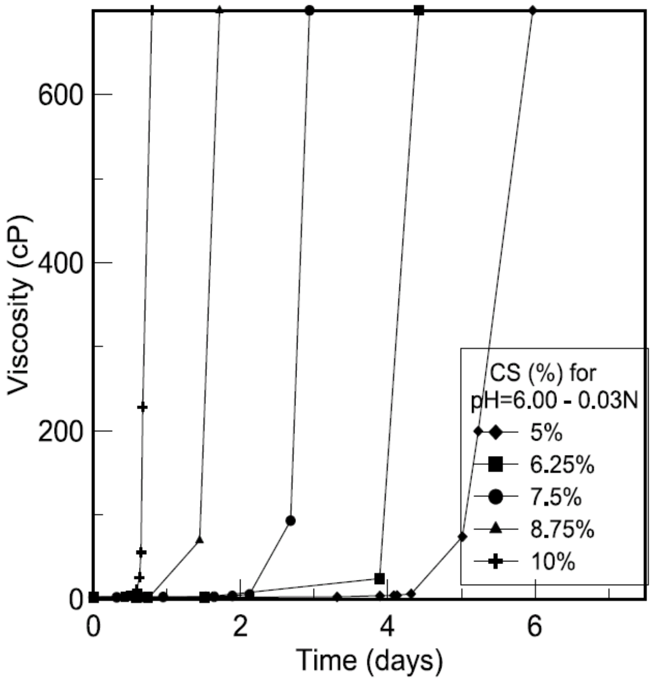

Figure 4. Effect of CS(%) on the viscosity vs time curve of colloidal silica solutions (pH = 6.00, 0.03N)

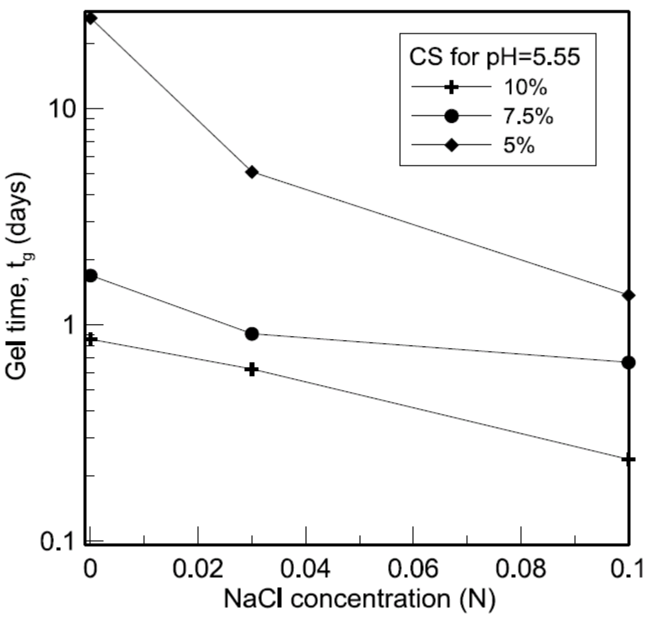

Figure 5. Effect of (NaCl) ion normality on gel time of colloidal silica solutions with different CS(%), but the same pH = 5.55.

Figure 4 shows the viscosity vs time curves in solutions with different CS (%) concentrations and pH = 6.00 & 0.03N. It is clear that the typical response identified in Fig. 2 applies in all these tests, and that there is a considerable monotonically decreasing effect of CS (%) on gel time. Observe that 2-fold increase of CS (%) from 5 to 10% can lead to a 5-fold decrease of gel time. From a physical point of view, this decreasing effect of CS (%) is expectable, since more silica particles mean much more siloxane bonds and thus much quicker gelation.

Ion normality in general (or NaCl concentration in our tests) also affects gel time, as explained in Section 2. To study this effect, Figure 5 shows how NaCl concentration (quantified in terms of normality N) affects the gel time for three (3) different CS concentrations. Observe that for CS = 5% an increase of NaCl concentration from 0.03N to 0.1N causes a decrease of gel time from 5 to 1.4 days, while the corresponding decrease of gel times for CS = 10% is much smaller (at the same pH value for comparison. In other words, NaCl concentration is important, especially for low values of CS (%). Note that higher values of NaCl concentration are physically possible, but would not be used in practice since they would lead to very small gel times (in the order of hours, for the CS values of interest).

3.5 Effect of temperature

The temperature of the colloidal silica solution also plays an important role in its gelation process. For example, there is evidence that a two-fold increase of gel time may occur for a decrease of 10o C (e.g. Otterstedt and Greenwood 2005), but whether this strong effect applies in all (CS, N, pH) combinations and colloidal silica types is yet to be investigated.

For this purpose, a number of viscosity measurements outlined in Table 1 was repeated for different temperatures in order to quantify this effect for the Ludox®-SM colloidal silica. In order to retain the temperature of the tested solutions constant, they were constantly kept within the tub of the Brookfield TC-150 water bath. Figure 6 shows an example of the "gel time curves" for CS = 10%, NaCl ion concentration of 0.1N at three (3) different temperatures T=15, 20, 25o C and three distinct values of pH ranging from 5.0 to 7.0.

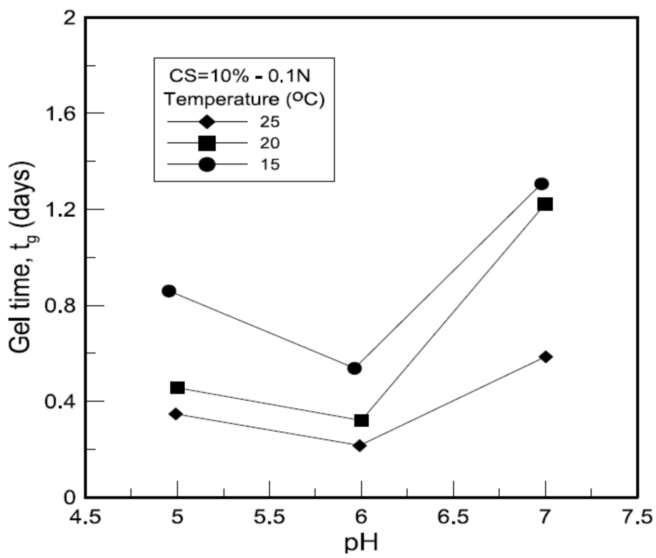

Figure 6. Effect of temperature on gel time for CS=10%, with NaCl concentration equal to 0.1N at different pH values

Our data corroborate the decreasing effect of temperature on t_g. As the temperature increases the gel time becomes significantly smaller. It is remarkable that the tested decrease of the temperature from T = 25° C to T = 15° C leads to an approximate 2.5 times increase of gel time for almost every pH value tested. Similar processing of the data shows that for all tested (CS, N) combinations the "gel time curve" retains the same shape of a (somewhat symmetric) inverted bell, which is shifted upwards by 2.5 times approximately in terms of t_g with a decrease of T by 10° C. Of importance is also the fact that despite the change in the actual values of t_g, the value of pH_{opt} remains essentially unchanged, since it is found a function of CS (%) and ion normality only.

4. DESIGNING FOR GEL TIME

Based on the above, the gel time t_g of colloidal silica is very much influenced by the (CS, N, pH, T) combination. Hence, the quality control of t_g seems crucial for applying passive stabilization in practice. Note that t_g is essentially the required permeation time until the colloidal silica reaches the desired location (under the structure in Fig. 1). Its estimation may be performed on the basis of the geotechnical conditions at a site (as a function of permeability, distance to be travelled via injection, etc.), but this is beyond the scope of this paper. A discussion on these issues may be found in the paper of Agapoulaki et al. (2015) in this conference.

Of interest here is to aid the engineer to select the values of (CS, N, pH, T) that could lead to the desired value of t_g. In order to aid in this effort, this paper presents a set of design charts focusing on patterns of rheological response that seem to be applicable in all cases.

Hence, Figure 7 summarizes the t_{gmin} values of the "gel time curves" for all (CS, N) combinations of Table 1, as well as the pertinent pH_{opt} values for a temperature T = 25° C. Observe the intensely decreasing effect of CS(%) on t_{gmin} (up to 8-fold decrease for a 2- fold increase from CS = 5% to 10%), and the less dramatic decreasing effect of normality N on t_{gmin} (up to 2.5 times decrease for 3.3 times increase from 0.03N to 0.1N). However, note that while the t_{gmin} ranges considerably (from a few hours to a few days), the pH_{opt} does not, i.e. it appears at values of pH between 4.5 and 6.0 in all the data, and based on Fig. 6 these values are not affected by temperature.

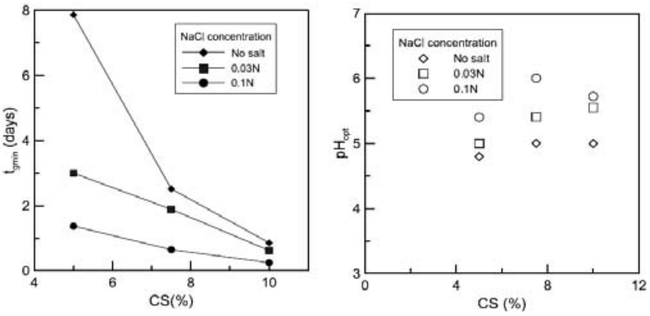

Figure 7. (a) Relation between the minimum gel time, t_{gmin}, of colloidal silica and the CS (%) at three different NaCl normality values. (b) Relation between the optimum pH, pH_{opt}, of colloidal silica and the CS (%) at three different NaCl normality values.

Figure 8 investigates the correlation between the normalized gel time t_g/t_{gmin} and the normalized pH/pH_{opt} ratio at T = 25° C for all cases of Table 1. It is clear that the inverted bell shape is a common trait for all "gel time curves", and that an average design curve (shown dashed) may be defined for all cases.

If one would like to use this design curve to approximately reconstruct his "gel time curve", he could pick values of t_{gmin} and pH_{opt} from Fig. 7, and correct the t_{gmin} for temperature on the basis of Fig. 6. For example, if the chosen CS and NaCl concentrations are 10% and 0N, then t_{gmin} = 0.67 days and pH_{opt} = 5.53 according to Fig. 7. If there is no need for temperature correction and t_g = 1.0 day, then t_g/t_{gmin} = 1.49, and from the average design curve of Fig. 8 the pH/pH_{opt} = 0.81 or 1.1. Hence, the corresponding pH values for the chosen t_g and combination of (CS, N) equal to 4.47 or 6.08. From the experimental data, for CS = 10%, 0.0N, a value of t_g = 1 day corresponds to a pH values of 4.4 or 5.95. Thus, the use of the hereby proposed design tools offers a satisfactory approximation of the laboratory data and a means for designing the recipe of the colloidal silica solution to be injected. Whether one would choose the more acidic or the more alkaline option of pH for attaining the same t_g is an issue related with environmental concerns, as well as quality control issues (e.g. mixing with natural groundwater and its effect on pH and ion normality).

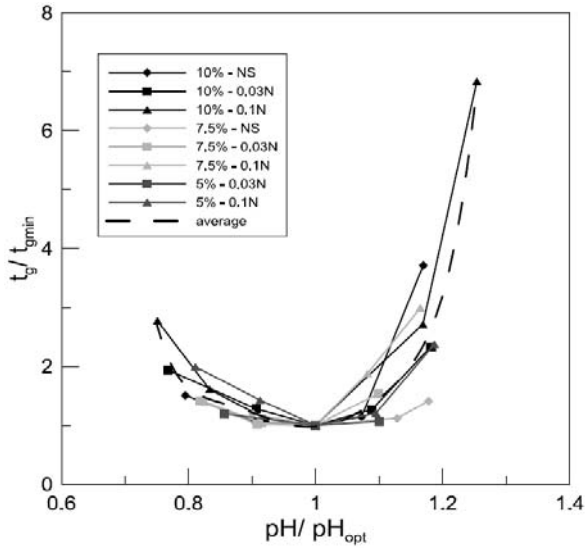

Figure 8. Normalized "gel time curves" at T=25° C for all combinations of (CS, N) of Table 1 and the average design curve.

5. CONCLUSIONS

The rheological testing of a wide range of colloidal silica solutions has provided a satisfying set of data for the application of passive stabilization in practice. This paper focuses on the control of the gel time t_g of colloidal silica, which is paramount for quality control of the method. Based on the presented data:

- The main parameters that control the t_g are silica percentage per weight CS (%), ion concentration and ion strength (in terms of normality N), pH and temperature T. In general, the CS (%), N and T have a monotonically decreasing effect, while the effect of pH is not monotonous.
- For any combination of the controlling parameters colloidal silica has initially (and up until 80-90% of the t_g) very low viscosity (allowing its low pressure injection), which at some point increases rapidly creating a firm gel (at a time of t_g).
- Based on the design tools presented herein, the gel time t_g may be easily controlled to range from a few hours to a few days (and even higher), according to the application at hand.

ACKNOWLEDGEMENT

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Numerical analysis of the seismic response of sand passively stabilized against liquefaction

Analyse numérique de la réponse sismique de sable stabilisé passivement contre la liquéfaction

K.I. Andrianopoulos, G.I. Agapoulaki and A.G. Papadimitriou

ABSTRACT Passive stabilization is a new technique for mitigating the risk of seismic liquefaction in the non-cohesive foundation soil of existing structures. It consists of the low pressure injection (in the soil pores) of colloidal silica, a stabilizer material which transforms into a firm gel after a well-controlled time. This gelation enhances macroscopically the mechanical response of the soil skeleton–pore fluid system. The microscopic mechanism of improvement has not yet been established, thus prohibiting the formulation of constitutive models. This paper explores the possibility of using existing constitutive models for simulating the response of stabilized sands. A well-established plasticity model for sands (NTUA-SAND) is used herein for the simulation of a pertinent dynamic centrifuge test, after re-calibration or in combination with the seemingly relatively compressible colloidal silica as the pore fluid instead of incompressible water.

RÉSUMÉ La stabilisation passive est une nouvelle technique pour atténuer le risque de liquéfaction sismique dans le sol de fondation des structures existantes. Elle consiste en l'injection (dans les pores du sol) à basse pression de la silice colloïdale, un matériel stabilisateur qui se transforme en gel ferme après un temps bien contrôlé. Cette gélification améliore de façon macroscopique la réponse du sol, mais le mécanisme microscopique de l'amélioration n'a pas encore été établi, interdisant ainsi la formulation de modèles de comportement. Cet article explore la possibilité d'utiliser des modèles de comportement existants pour simuler la réponse des sables stabilisés. Un modèle de plasticité pour les sables (NTUA-SAND) bien établie est utilisé ici pour la simulation d'un essai dynamique en centrifugeuse pertinente, après recalibrage, ou en combinaison avec de la silice colloïdale compressible comme le liquide des pores à la place de l'eau incompressible.

1. INTRODUCTION

Passive stabilization (Gallagher 2000) poses as one very promising liquefaction mitigation technique for developed sites. It entails the low-pressure injection of a stabilizer (colloidal silica) in the soil pores, which transforms into a firm gel after well-controlled time. This change alters the mechanical response of the thus stabilized sand, making it less vulnerable to plastic strain accumulation and strength degradation related to liquefaction or cyclic mobility.

This ground improvement technique is currently still at an experimental stage worldwide. One important issue that needs to be addressed before it becomes practically usable is how to control the colloidal silica properties in order to permeate it effectively at the desired distance within a given soil (see also Agapoulaki and Papadimitriou 2015, Agapoulaki et al. 2015, both in this conference).

Another similarly significant issue is to establish the microscopic mechanism of improvement, as the first step for the formulation of constitutive models for this new geomaterial named stabilized sand. Since this is not yet established, this paper explores the potential of using existing constitutive models for sands for the task at hand by appropriately adjusting different aspects of the simulation components.

2. MECHANICAL RESPONSE OF STABILIZED SANDS

The mechanical response of stabilized sands has been studied only macroscopically, via: a) limited insitu testing (Gallagher et al 2007a), b) few physical modeling attempts (centrifuge modeling: e.g. Gallagher et al. 2007b), and c) a number of laboratory efforts outlined below. Yet, each of these laboratory efforts has shed light to partial aspects of the response, since they have been performed on different sands by employing different types of tests.

It has been shown that regardless of the microscopic mechanism, stabilization induces unconfined compression strength in stabilized sands (e.g. Persoff et al. 1999, Papadimitriou & Agapoulaki 2013).

Focusing on the dynamic response at small cyclic strains, published data show that stabilization with colloidal silica leads to a small increase in the elastic shear modulus G_{max}, of 10-25% on average (e.g. Papadimitriou & Agapoulaki 2013, Spencer et al. 2008). On the contrary, at medium cyclic strains, published data (Spencer et al. 2008) show no substantial effect on the normalized shear modulus G/G_{max} degradation (Fig. 1a) and hysteretic damping D increase curves (Fig. 1c) with cyclic shear strain.

However, at large cyclic strains, element tests reveal a much more stable behavior for treated samples as compared to their untreated counterparts. In particular, cyclic (triaxial, simple shear) tests depict liquefaction, denoted by double amplitude cyclic shear strain DA exceeding a preset level (e.g. of 1%, 2% or 5%), after a much increased number of cycles, as compared to the natural sand under the same conditions (e.g. Gallagher and Mitchell 2002, Diaz Rodriguez et al. 2008). For example, Fig. 2 shows the significant increase of liquefaction resistance quantified in terms of the cyclic stress ratio (CSR = τ/σ'_{vo}) for a given number of cycles to liquefaction N_L due to stabilization (treatment) on the basis of cyclic simple shear tests (data from Diaz-Rodriguez et al., 2008). Observe that the increase of liquefaction resistance in terms of CSR for any given number of cycles N_L is typically in the order of 0.08 – 0.13 (as in these tests, processed for N_L = 5, 10 and 15). Note, that similar test data in the literature have reported a minimum effect of 0.01 (on the basis of these tests) and a maximum of 0.2 (Kodaka et al. 2005).

3. NUMERICAL SIMULATION

For the numerical simulation of the seismic response of stabilized sand, two (2) different approaches are examined:

a) recalibrating an existing elasto-plastic formulation for the sand (skeleton), and

b) modifying the properties of the pore fluid

3.1.1 Recalibrating an existing constitutive law

Regarding the first approach, if one disregards the fact that the stabilized sand exhibits non-zero unconfined compression strength, constitutive simulations may be potentially attained by employing existing constitutive models for sands. For this purpose, this paper adopts a bounding surface plasticity critical state model (NTUA-SAND; Andrianopoulos et al. 2010), which has been successfully implemented in FLAC and is readily available for potential users in: http://www.itasca-udm.com/pages/NTUA.html.

The attribute of this model that makes it appealing for the simulation of stabilized sand is the adoption of a Ramberg-Osgood type formulation of the "elastic" moduli, which governs the response under small to medium cyclic shear strains. In this sense, one may re-calibrate the plastic modulus (i.e. the ho model constant that serves as a scalar multiplier of the plastic modulus) to efficiently simulate the significant effect of stabilization at large cyclic shear strain levels (e.g. see liquefaction tests in Fig. 2), without significantly affecting the predicted response at small and medium cyclic shear strains where stabilization plays a less important role.

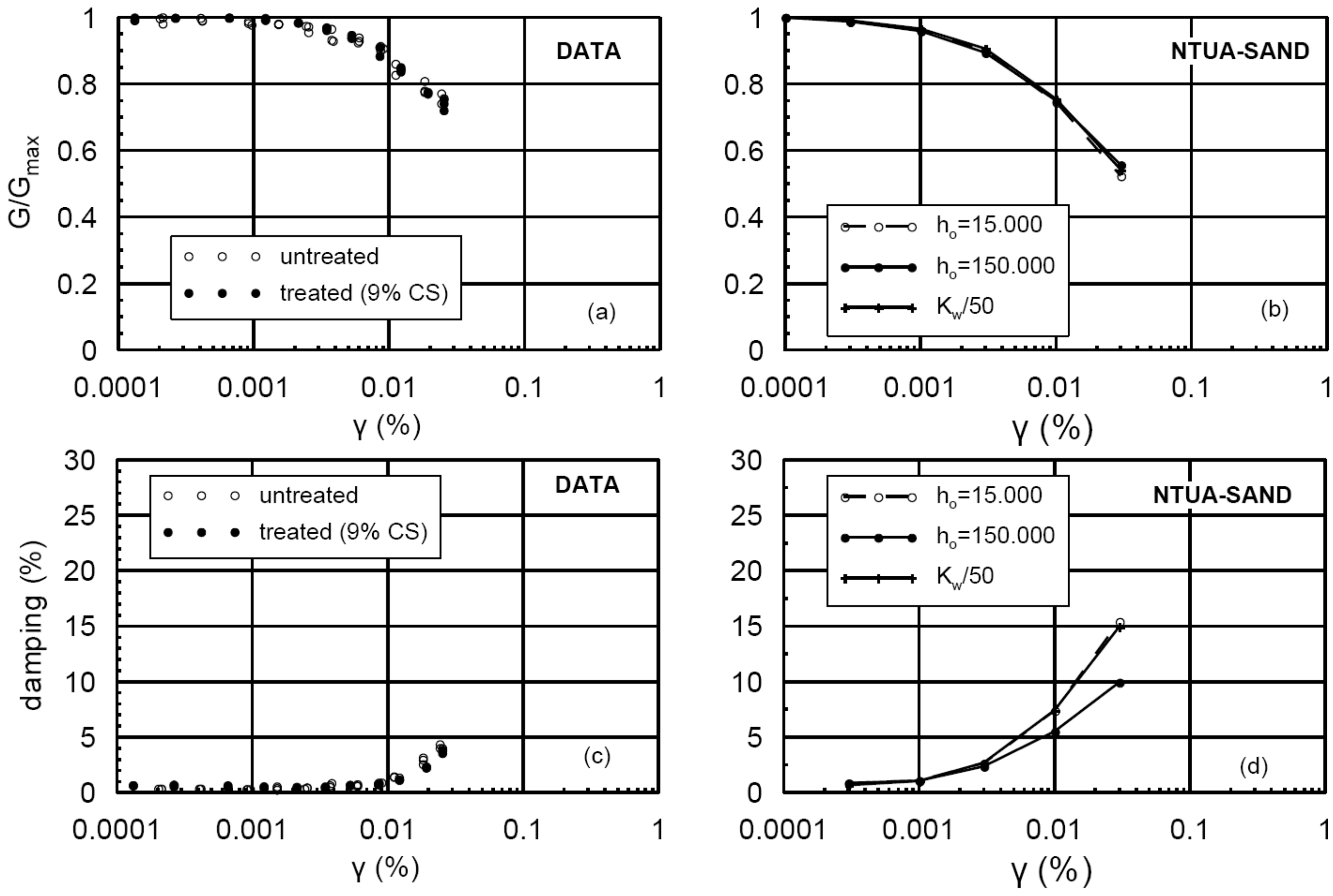

Fig. 1: Effect of colloidal silica on the normalized shear modulus G/G_{max} degradation and hysteretic damping D increase curves with cyclic shear strain level (data from Spencer et al. 2008) and comparison with numerical results using NTUA-SAND

A ten-fold increase of the plastic modulus as an upper-value is adopted here to simulate the effect of passive stabilization with colloidal silica, i.e. by altering the initial value of ho from 15,000 (representing "*untreated*" sand) to 150,000 (representing the "*treated*" sand). All other model constants retain their values (as reported in Andrianopoulos et al. 2010), which correspond to employed Nevada sand.

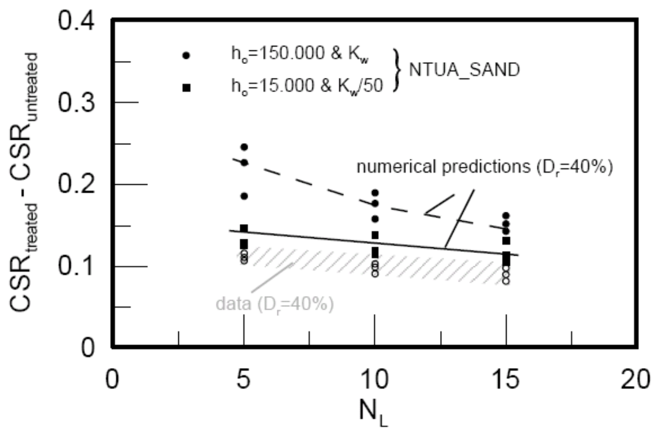

Fig. 2: Effect of colloidal silica on the cyclic stress ratio CSR needed for liquefaction for a given number of equivalent cycles N_L and comparison with numerical results for treated sand using NTUA-SAND with 2 modeling approaches (data from Diaz-Rodriguez et al. 2008)

3.1.2 Modifying pore fluid properties

Regarding the second approach, it is acknowledged that any constitutive model for soils (not only NTUA-SAND) is constructed with the aim to simulate the soil skeleton response, on the basis of tests performed with water as the pore fluid. However, the pore fluid in stabilized sand is not water, but gelled colloidal silica. Hence, the properties of colloidal silica itself could affect the overall response, especially in boundary value problems where the conditions are not fully controlled as in element tests.

The same concerns were raised by Towhata (2008) who, based on results from unconfined compression tests of pure gelified colloidal silica samples, concluded that this material has significant volume compressibility in comparison to that of water (which is practically incompressible). In other words, while any tendency for volume reduction (e.g. due to seismic shaking) translates to excess pore pressure development in untreated sand, in treated (stabilized) sand any similar loading is not expected to generate (significant) excess pore pressures.

Having in mind this observation, numerical analyses were also conducted by reducing the bulk modulus of the pore fluid at various values, with K_w denoting the bulk modulus of water. The model constants of NTUA-SAND in these analyses retain the values corresponding to untreated Nevada sand.

3.1.3 Comparison with laboratory results

Numerical simulations were performed to compare the effect of the two approaches on the simulation of laboratory tests. Regarding the small strain modulus G_{max}, the results (not shown here for reasons of brevity) depict that by introducing a ten-fold increase of the plastic modulus *ho*, an increase of G_{max} by 15% to 22% is predicted, in good agreement with experimental data (e.g. Spencer et al. 2008). However, even if one adopts as pore fluid modulus a value 50 times smaller than that of water (K_w/50) no similar effect on G_{max} is found.

Figure 1 emphasizes on the normalized shear modulus G/G_{max} degradation and hysteretic damping D increase curves with cyclic shear strain level. It compares simulations for "untreated" sand (*ho* = 15,000) and "treated" sand (*ho* = 150,000) in subplots b and d, and compares them to the resonant column data of Spencer et al. (2008) in subplots (a) and (c), respectively. Observe that similarly to the test data, no effect of stabilization is observed in the predicted response for small and medium cyclic strains. Similar behavior is observed when modifying the pore fluid modulus to a value of K_w/50.

Figure 2 compares the relative increase in the cyclic stress ratios CSR required for liquefaction attained at given numbers of load cycles (N_L = 5, 10 and 15) from the tests of Diaz-Rodriguez et al. (2008), to the predicted pertinent values for "*untreated*" (*ho* = 15,000) and according to the first approach "*treated*" sand (*ho* = 150,000). Similarly to the relevant data, significant increase of the cyclic resistance is observed resulting from the ten-fold increase of the plastic modulus. Note that significant increase of the cyclic resistance is also predicted when modifying the pore fluid modulus to K_w/50, although its effect is less pronounced than that of the first approach.

3.1.4 Comparison with centrifuge tests

This section explores the potential of both approaches to simulate a boundary value problem involving stabilized ground. For this purpose, a dynamic centrifuge test is selected (Gallagher et al. 2007b) that refers to the 1D dynamic response of a uniform sand layer under sinusoidal motion. The test replicates the 1D test performed during the VELACS project using loose Nevada sand (Dr = 40%), with the difference being that the Nevada sand in this test is treated with colloidal silica grout with CS = 6% by weight. In prototype scale, the experiment refers to 10m deep layer and is excited by 20 cycles of a 2Hz sinusoidal horizontal input, with uniform peak base acceleration of 0.2g. The simulation is performed with NTUASAND model (Andrianopoulos et al. 2010), since:

a) it has been thoroughly calibrated on Nevada sand data, the sand used herein as well, and

b) 1D shaking of an untreated horizontal sand layer of Nevada sand has been successfully simulated with this model.

The details of the simulation are not presented here for reasons of brevity, since all requirements for accuracy (mesh density, boundary conditions, etc) of the simulation for the untreated sand layer (see Andrianopoulos et al. 2010, for details) are retained.

Before proceeding to comparing data to simulations, note that in comparison to the original experiment for the untreated sand, the treated sand did not liquefy during shaking (Gallagher et al. 2007b). Moreover, the treated sand did not depict deamplification of the ground motion; rather it even showed significant amplification after the 3-4 cycles, with peak accelerations reaching 0.55g (in comparison to 0.2g at the base). The measured response also included significant spikes, which are essentially the cause for the foregoing amplification.

These traits of the measured response are presented in Figure 3, which compares the acceleration time history, as recorded at 2m depth for the treated sand (gray lines), with its counterpart from various numerical simulations (black lines). Focusing on Figs 3a and 3b, note that the ten-fold increase of the plastic modulus, produces minimal benefit in predictive accuracy (e.g. the numerical analysis still depicts deamplification of the seismic motion, following liquefaction of the horizontal layer). Thus, despite that the first approach seems promising at the element level (see Figs 1 and 2) it fails to predict the system response of the stabilized sand.

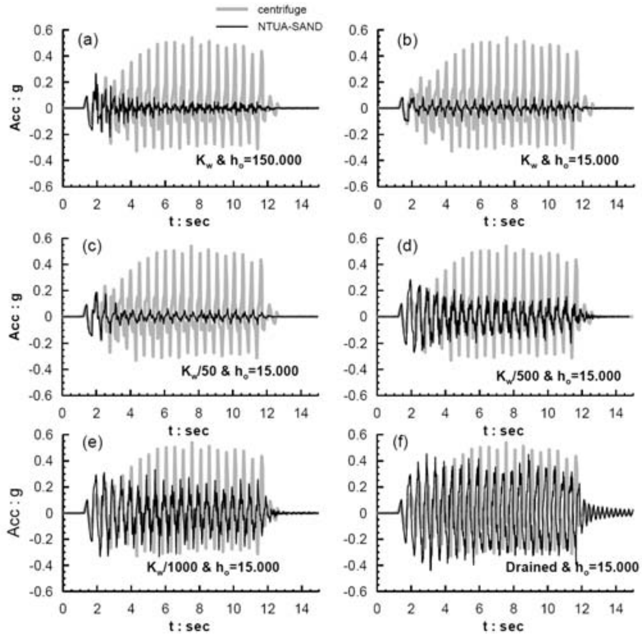

Fig. 3: Comparison of acceleration time-history at a depth of 2m as measured in centrifuge test with the numerical predictions using NTUA-SAND with different values for pore fluid compressibility and under drained conditions (data from Gallagher et al. 2007b)

Regarding the second approach, the numerical simulations are compared to the measured time history of acceleration at 2m depth in Figs 3b to 3e. As expected, by decreasing the pore fluid bulk modulus, the excess pore pressures are reduced and even some amplification of the seismic motion is predicted (0.3g as compared to 0.2g at the base). However, the predicted peak acceleration remains below the measured 0.55g, while dilation spikes are not predicted even for a decrease of the pore fluid modulus by 1000 times. Additionally, the 50-fold reduction of the pore fluid modulus seems inadequate to predict the system response of the stabilized sand, despite its good performance at the element level (see Figs 1 and 2). Finally, Fig. 3f presents the numerical prediction by assuming fully drained conditions (i.e. no excess pore pressures, or zero pore fluid bulk modulus), which plots much closer to the measured data, and this approach poses as a good candidate for a phenomenological simulation.

Similar conclusions are drawn from the comparison in terms of the surface settlement of the stabilized sand layer. As show in Figure 4, a ten-fold increase of the plastic modulus *ho* or a 50-fold decrease of the pore fluid modulus do not essentially alter the predicted value of surface settlement, which remains significantly lower than the measured one (in gray line). Hence, once more, a good performance at the element level seems inadequate for predicting the system response. Predicted settlements tend to increase by further reducing the pore fluid modulus, with the case of K_w/500 representing the best fit to the measured values of surface settlement. It should be underlined here, that the drained analysis seriously over-predicts the measured settlements, and is no longer considered a good candidate for a phenomenological simulation of stabilization. For this task, possibly the case of K_w/1000 could be considered an optimal trade-off in predictive accuracy in terms of both accelerations and settlements.

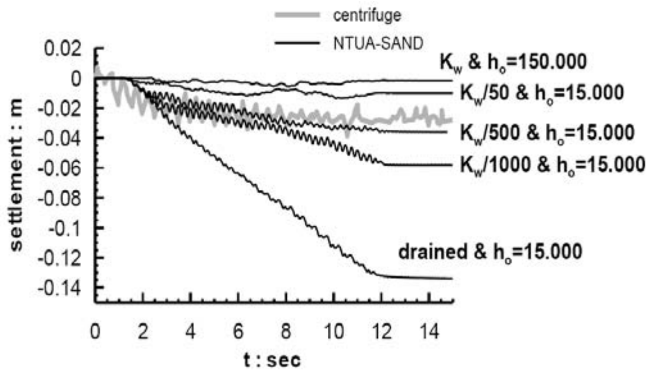

Fig. 4: Comparison of settlement time-history as measured in centrifuge test with the numerical predictions using NTUA-SAND with different values for pore fluid compressibility and under drained conditions (data from Gallagher et al. 2007a)

4. CONCLUSIONS

This paper explores the possibility of using existing constitutive models for simulating the response of sands stabilized with colloidal silica. Since the microscopic mechanism of improvement has not been established yet, this paper attempts a phenomenological approach to the task at hand. Based on this numerical study, the following may be concluded:

- Colloidal silica seems to affect mainly the cyclic response of sands at large cyclic strains, while its effect on the stiffness and damping values at small and medium strains is relatively small,
- Existing state-of-the art constitutive models can simulate at least macroscopically the response of stabilized sands at the element level, by mere recalibration (e.g. increase of plastic modulus) or by introducing a small decrease to the pore fluid modulus.
- Successful simulations of element tests on stabilized sands do not necessarily guarantee successful simulations of the system response in boundary value problems involving stabilized sand.
- The compressibility of colloidal silica after gelation needs to be explored experimentally. Existing evidence shows that it is more compressible as compared to water. Hence, if it is used as a pore fluid (as in passive stabilization) it may lead to significant reductions in pore pressures, which could qualitatively explain the system response.

ACKNOWLEDGEMENT

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Seismic performance of gravity retaining walls subjected to strong excitation

Performance séismique de murs de rétention gravitationnelle soumis à une forte excitation

E. Garini, R. Zakkak, I. Anastasopoulos and G. Gazetas1

ABSTRACT Seismic earth pressures on retaining structures are evaluated in practice by the pseudo-static extension of the Coulomb method known as the Mononobe–Okabe approach, in which the inertia force on the soil wedge is included. In reality, earthquake loading is not permanent but varies with time. We examine the dynamic response of a gravity wall in a two-layered soil stratum subjected to horizontal strong seismic acceleration. The cohesionless soil layers (retained soil and underlying soil) are modelled inelastically with finite-element discretisation. Their properties are studied parametrically. As excitations we employ a number of idealised wavelets and near-fault ground motions recorded in recent earthquakes. We present results for dynamic earth pressures behind the wall, soil displacements, wall settlement and rotation, and accelerations at several points. The relative importance of sliding versus rocking is explored. Comparisons are made with the Mononobe-Okabe method.

RÉSUMÉ Les pressions séismiques de la terre qui agissent sur les murs de soutènement sont évaluées dans la pratique en utilisant l'extension pseudo statique de la méthode connue comme l'approche Mononobe-Okabe, dans le cadre de laquelle les forces d'inertie du sol sont incluses. Pourtant, en réalité, la charge séismique n'est pas permanente, comme c'est supposé dans l'approche pseudo statique, mais elle diffère dans le temps. Cet ouvrage examine la réponse séismique d'un mur de rétention gravitationnelle dans un sol à deux strates, et soumis à de fortes accélérations horizontales de la terre. Nous employons comme excitations quelques ondulations idéalisés et plusieurs accélérogrammes enregistrés pendant de récents tremblements de terre. Le sol est composé de deux couches (la couche retenue et la couche inférieure). En plusieurs points, nous présentons des résultats quant de la pression dynamique du sol sur le mur, le déplacement du sol, la rotation du mur, et les accélérations. Une comparaison avec les résultat de la méthode Mononobe-Okabe est également effectuée.

1. INTRODUCTION

Gravity retaining walls are the type of retaining walls that rely on their weight to retain the material behind it. The weight of the wall has two roles: resisting the overturning of the wall and causing frictional sliding resistance at the base of the wall on one hand, and experiencing inertia loading on the other.

Seismic response of even the simplest retaining wall is a complicated soil-structure interaction problem. The displacements of the wall and the dynamic earth pressures depend on the response of the retained soil, the inertia of the wall itself, the foundation soil and the nature of the input motions. The Mononobe-Okabe method (1926), an extension of Coulomb's method, is the earliest and most widely used analytical method. It gives the total active thrust acting on the wall by applying a pseudostatic inertial force on the soil wedge. The point of application of the thrust is presumed at 1/3 the height of the wall above its base. This method had been modified and simplified by Seed & Whitman (1970). Richards & Elms (1979) determined permanent (inelastic) outward displacements, and Nadim & Whitman (1983) permanent sliding and rotation using the Newmark sliding block concept. Veletsos and Younan (1994) modelled the soil as an elastic medium and obtained elastodynamic solutions. Several other studies have also appeared, among which: AI-Homoud & Whitman (1994), Wu & Prakash (1999), Gazetas et al. (2005), Huang (2005), Dakoulas & Gazetas (2008). In parallel, a significant effort was made in numerical study of seismic earth pressures in centrifuge experiments by Ortiz et al. (1983), Cai & Bathurst (1995), Zeng (1998), Madabhushi & Zeng (2007), Al Atik & Sitar (2010), and most recently by Mikola & Sitar (2013).

The scope of this paper is to shed some light into fundamental aspects of seismic response of gravity retaining walls subjected to near-fault ground shaking, with numerical analyses of a typical gravity wall.

2. MODEL GEOMETRY AND INPUT MOTIONS

A 2-D plane-strain finite element model was constructed using the ABAQUS commercial code. The discretization consists of four-noded quadrilateral, plane-strain elements. As shown in Figure 1, the model includes two identical walls, one opposite to the other, to render the model symmetrical, and hence to minimize boundary effects. In addition, for each excitation the model provides results for two directions of loading, capturing the polarity effect which might be significant for this type of asymmetric systems (Gazetas et al. 2009, Garini et al. 2011).

The wall height and width is 12 m and 8 m respectively. The geometrical limits of the model are 50 m behind each wall. In order to avoid any interaction between the two walls, they were placed at a distance of 100 m. The soil properties are: (i) for the retained soil: ρ = 1.9 Mgr/m^3, E = 100 MPa, φ = 37.5^o, ψ = 7.5^o and (ii) for the foundation soil: ρ = 1.7 Mgr/m^3, E = 50 MPa, φ = 35^o, ψ = 5^o. The wall is made from concrete and its behavior presumed to be elastic. The soil-wall interfaces are all tensionless with potential for sliding. The coefficient of friction is: $\mu = \tan\varphi$ for the retained soil and $\mu = (2/3)\tan\varphi$ for the foundation soil. Soil behavior is described by Mohr-Coulomb constitutive law.

Horizontal and vertical viscous dashpots with free field boundaries were added to absorb the radiated energy from the P and S waves, respectively. Accelerations, applied at the base of the model (at -32 m depth), are either idealized pulses or near-fault seismic records. The latter have been influenced by forward-directivity or fling-step effects from the Kobe, Kocaeli, Lefkada and Aegion earthquakes.

In particular, two idealized wavelets are used as simple representation of near-fault pulses: a threepulse Ricker wavelet and a multi-pulse Tsang wavelet. The near-fault accelerograms are: the Takatori (Kobe 1995), Sakarya (Kocaeli 1999), Lefkada (Lefkada 2003) and Aegion (Aegion 1995). The peak acceleration of each record was normalized from 0.1g to 1g.

3. RESULTS OF DYNAMIC ANALYSES

For the sake of brevity, only a minimum of all the parametric results are presented below. In Figure 2 the contours of plastic strain magnitudes in the soil behind the right wall are illustrated for the case of a Tsang excitation with frequency 2 Hz and PGA = 0.4 g. It is evident that a failure wedge forms in the soil behind the wall, while small plastification is noticed beneath the outer corner of the wall base. Acceleration-time histories were investigated at three points (K, L, M) behind the wall at soil surface (Figure 3). Also shown are the accelerations at (N, O, P) in front of the wall at its base. The following conclusions are of interest:

- Next to the wall (top and base) high spiked components are included in accelerograms (see points K, N), apparently the result of slippage and wave reflections on the wall.

  The surface acceleration amplitudes in the backfill free field are only 60% of the exciting peak acceleration

(PGA)—a deamplification. By contrast in the free field in front of the wall there is an increase of PGA to 0.53 g.

- Next to the wall at the top (points K and L) an asymmetry is noted in the accelerograms: the outward acceleration, developing inward inertia, reaches 0.51 g and 0.45 g at the two points. In the other direction, the wall outward compliance, limit's the developing peak acceleration to 0.33 g and 0.28 g, at points K and L respectively. Recall the potentially-sliding interface effect (named PEPSI) shown by Gazetas & Uddin (1994).
- Figure 4 portrays the rotation time history of the wall, as well as the sliding displacement at the base and at the vertical interfaces between the wall with the retained soil. As a result at the end of the motion, the wall slided outwards 10 cm at its base and rotated just 0.36°.

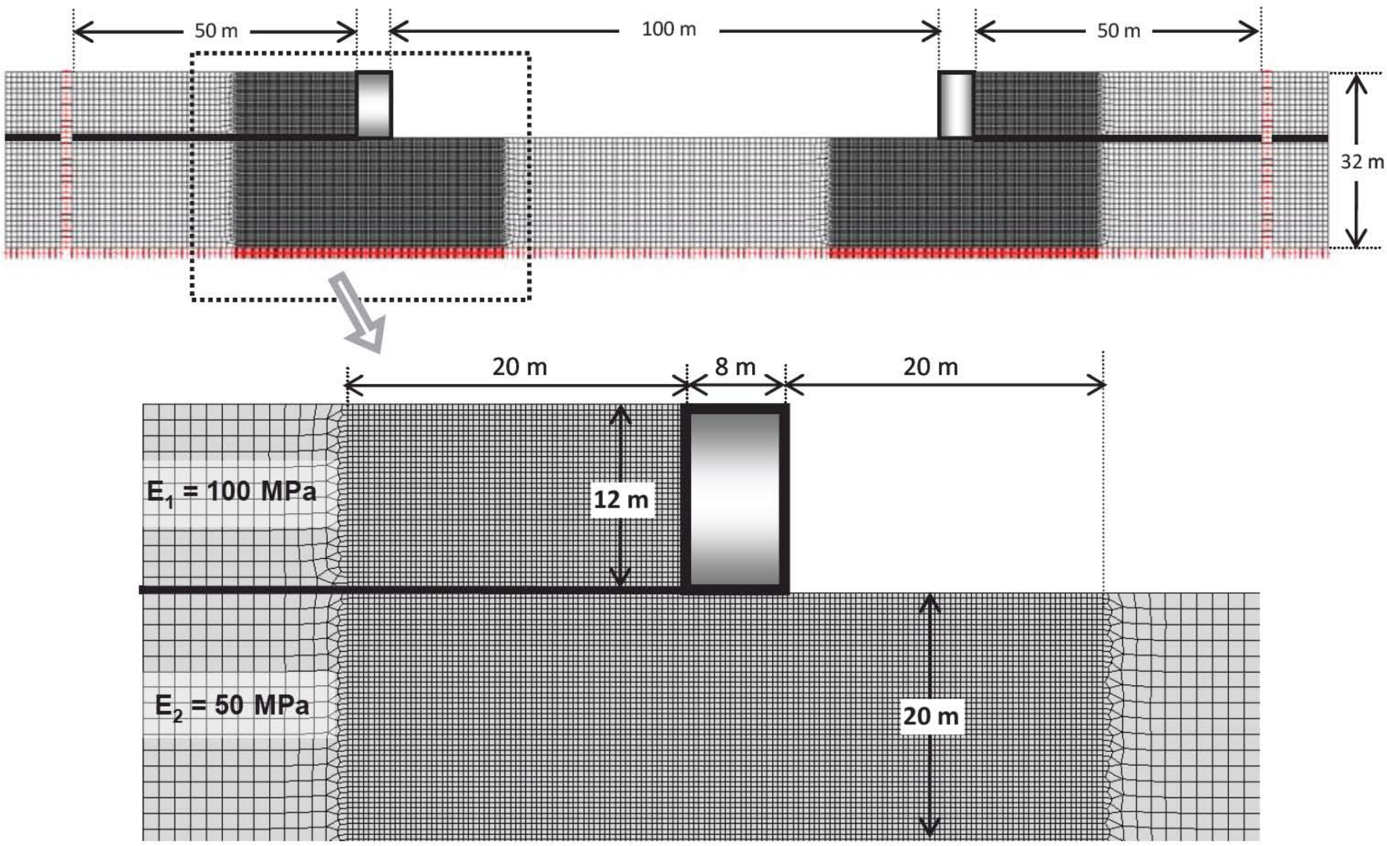

Figure 1. Two-dimensional finite element mesh (in ABAQUS) of the soil-retaining wall system: geometry, dimensions and elastic modulus of soil are presented.

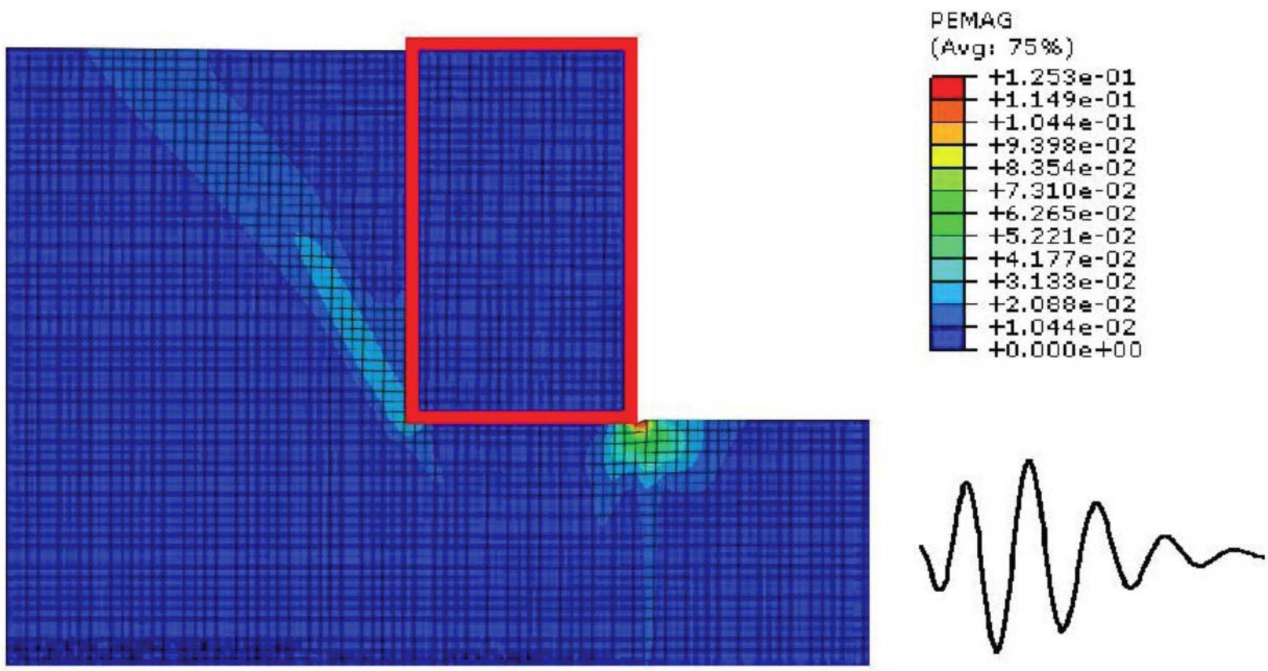

Figure 2. Plastic strain contours due to a Tsang wavelet of 2 Hz frequency and peak acceleration 0.4 g.

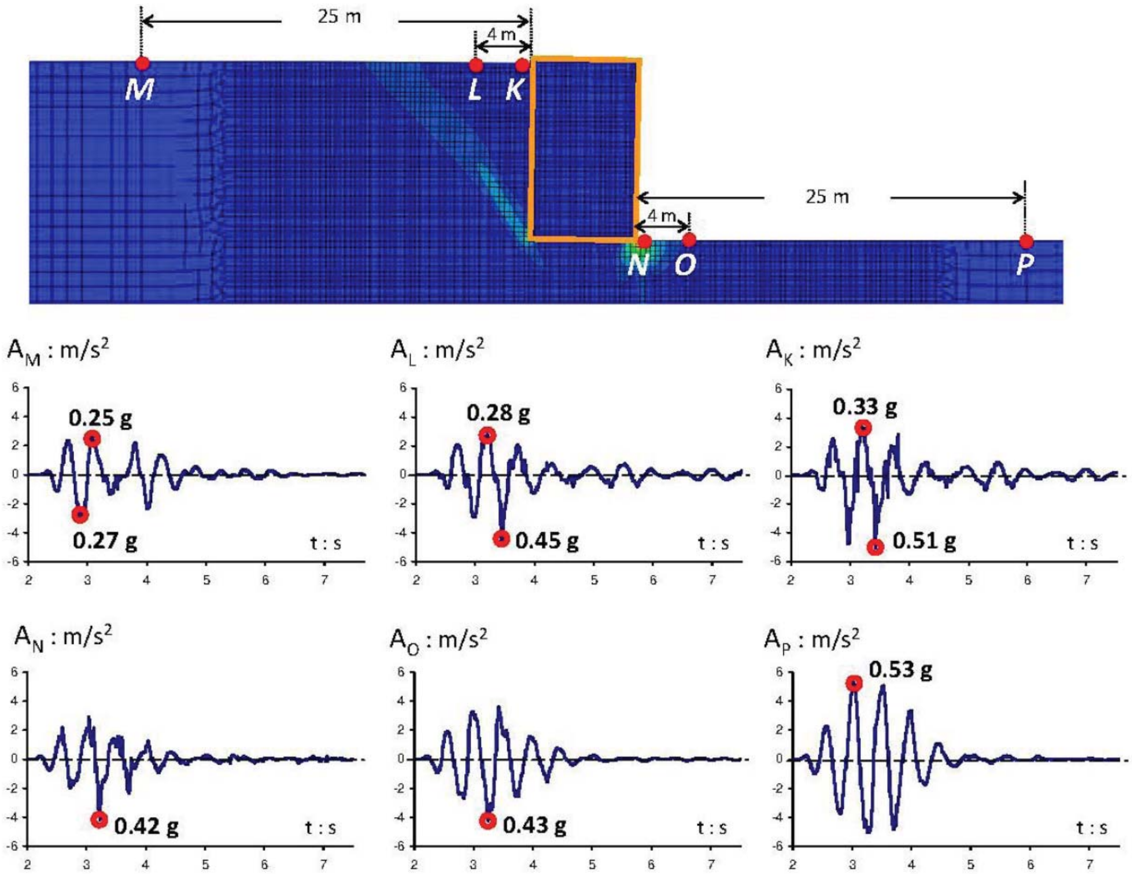

Figure 3. Acceleration-time histories for: (a) three different points (K, L, M) behind the wall, and (b) three different points (N, O, P) at the base of the wall. The excitation of the wall-soil system is a Tsang wavelet of 2 Hz frequency and peak acceleration of 0.4 g.

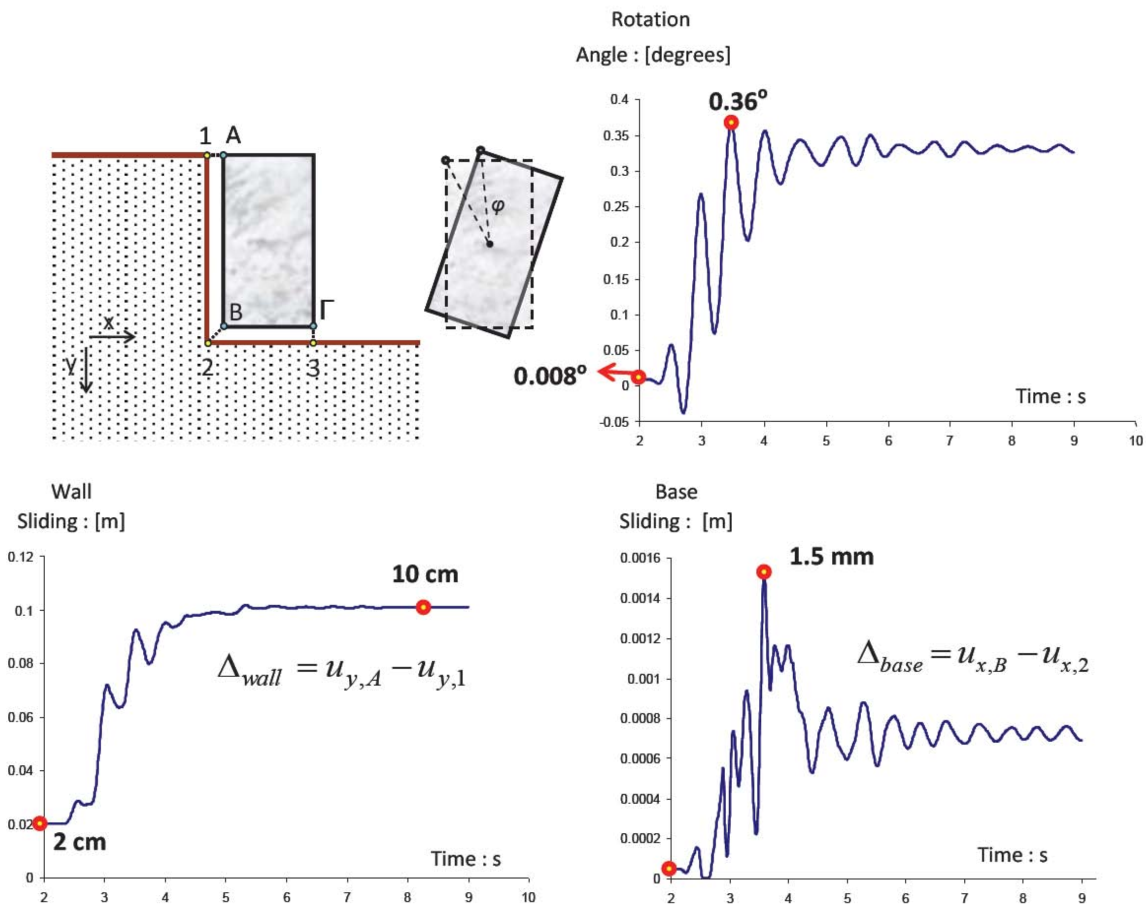

Figure 4. Rotation and sliding of the wall at its base and at the vertical interface behind the wall.

Figures 5 and 6 refer to wall response to Takatori ground motion, the most harmful of those studied herein. It leads to a horizontal displacement of 2.9 m (for PGA = 1g) and differential settlement of 2.1 m. Figure 5 shows the distribution with depth of the peak earth pressures for different peak accelerations, for both right and left wall. For the left wall, earth pressures increase almost linearly with depth for all acceleration levels, but for the right wall the pressures attain an almost uniform distribution with depth — a clear evidence of the *polarity* effect, i.e. from the asymmetry of the record and the response as has been shown by Gazetas at al. (2009) and Garini et al. (2011).

The role of the coefficient of friction, μ, is illustrated in Figure 6. Three values are given to μ: 0.4, 0.6, and 1. As can be seen from Figure 6, earth pressures are smaller for the smallest friction coefficient, μ=0.4. For μ=0.6 and μ=1 they are higher and almost the same. The reduction of earth pressures stems from the larger horizontal displacements due to sliding for smaller μ. Moreover, a comparison with Mononobe-Okabe stresses acting on the wall is performed (dash line in Figure 6). Soil pressure distribution is overestimated by the Mononobe-Okabe mainly when the coefficient of friction is small.

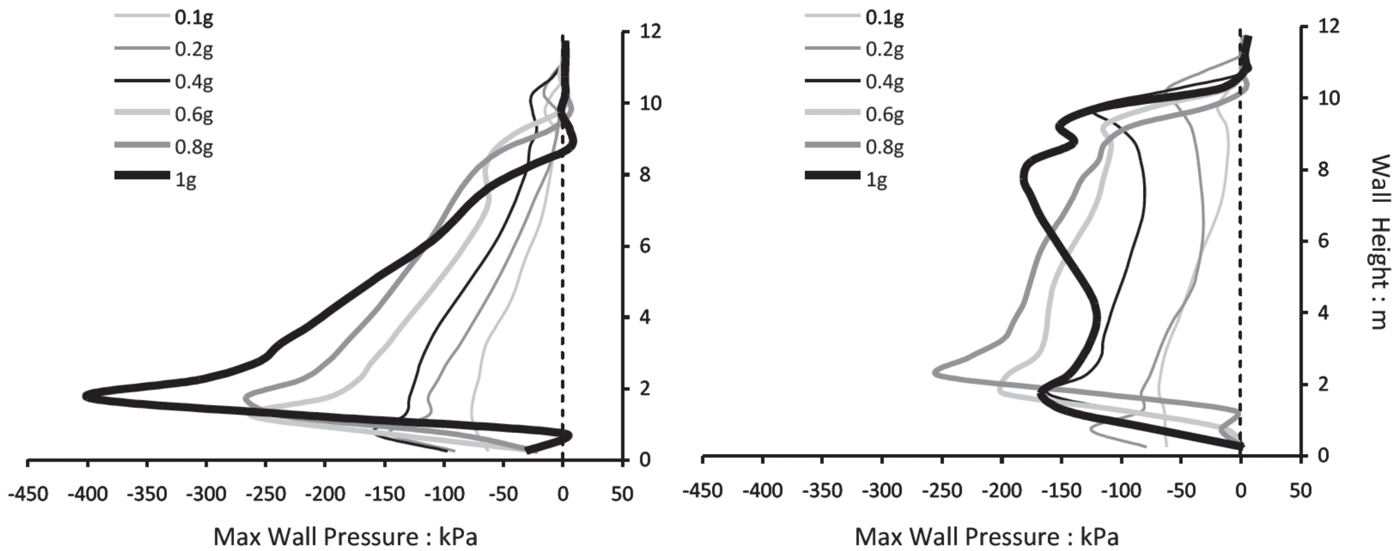

Figure 5. Maximum earth pressure for different peak accelerations on the left and right wall when subjected to Takatori record.

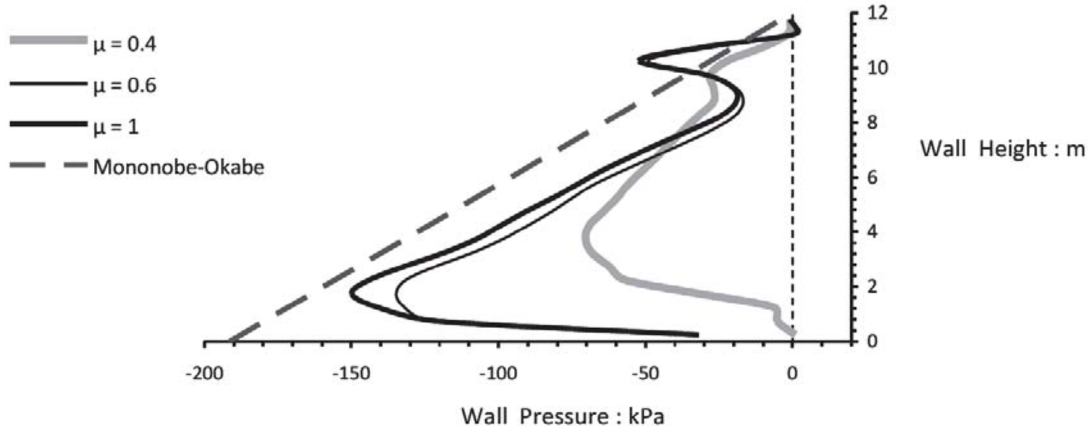

Figure 6. Comparison of earth pressure on the retaining wall when subjected to Takatori seismic excitation with peak acceleration 0.6 g.

4. SUMMARY AND CONCLUSION

The paper studied numerically the seismic response of a typical gravity retaining wall. 2-D analyses were conducted and the response of the retaining wall was investigated for different excitations and three coefficients of friction. Asymmetry of the acceleration experienced by the wall and the soil stems from the outward displacement of the wall. Decreasing the friction coefficient, μ, leads to increased sliding of the retaining wall, increased horizontal displacements and less rotation of the wall. The seismic response of a retaining wall is sensitive to the polarity of the seismic excitation.

ACKNOWLEDGEMENT

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Challenges of tunnel design and construction in the GCC region

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Senior Tunnel and Geotechnical Expert
Deutsche Bahn Engineering & Consulting

*GCC: Gulf Cooperation Council

Invited lecture to the BRIDGES & HIGHWAYS OMAN Conference / Tunnelling Focus Day, 13 September 2015, Muscat, OMAN, RET - Rail Expansion & Technology Congress MENA 2015, 15 September 2015, Dubai, UAE and Qatar Transport Infrastructure 2015 Conference, 13 October 2015, Doha, Qatar.

Challenges of tunnel design and construction in the GCC region

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Outline of the presentation

- On going main tunnelling projects in the GCC countries
- Future main tunnelling projects in the GCC countries
- Tunnelling challenges and their main causes in the GCC countries
- Solutions based on design innovations and well targeted risk analyses
- Solutions fitted in the framework: “LARGE SCALE PROJECTS to be completed in a tight time framework” – Estimations of TBM advance rates
- Conclusions

On going main tunnelling Projects in the GCC countries

- Doha Metro Network - Phase 1

1. Total length: 128 km
2. Underground: 74 km
3. Elevated: 25 km
4. At Grade: 29 km
5. No of Stations: 49

✓ Estimated Completion Year: 2019
✓ Estimated Cost: ~ 20 billion Euros

On going main tunnelling Projects in the GCC countries

- IDRIS (Inner Doha Re-sewage Implementation Strategy)

1. 40 km of deep main truck sewer
2. > 70km of lateral interceptor sewers

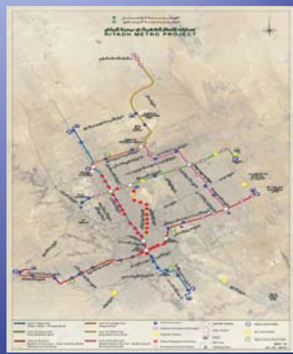
✓ Estimated Completion Year: 2019
✓ Estimated Cost ~ 2.5 billion Euros

On going main tunnelling Projects in the GCC countries

- Riyadh Metro Project

1. Total length = 176km
2. No of Lines = 6
3. No of Stations = 78

✓ Estimated Completion Year 2019
✓ Estimated Cost ~ 20 billion Euros



Future main tunnelling Projects in the GCC countries

- Doha Metro Network – Phase 2

1. Total length: 103km
2. Underground: 40km
3. Elevated: 51km
4. At Grade: 12km
5. No of Stations: 45

✓ Estimated Completion Year 2026

Future main tunnelling Projects in the GCC countries

Tunnelling challenges and their causes

- Geotechnical & Hydrogeological conditions
- Topographical conditions & Adverse geotechnical conditions – Significant overburden heights: Possible squeezing
- Environmental conditions – Groundwater aggressiveness
- Scale of the Projects – Very demanding logistics requirements
- Time framework of the Projects

The “Mega” scale of Metro Projects in the GCC countries

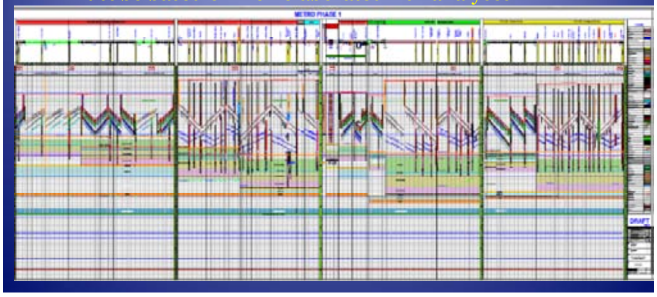
- ***Construction Logistics***:
1. 20 -30 Mio m³ of excavation materials
2. 3 – 5 Mio m³ of Backfill
3. 4 – 6 Mio m³ of concrete
4. 480,000 – 650,000 tunnel segments
- ***Supply and disposal logistics:***
1. Congested cities
2. Limited space
3. Well coordinated materials supply

Geotechnical & Hydrogeological conditions in the GCC region

- Weak rocks (e.g. Limestone formations)
- Karstic features (Voids, Karstic depressions, sinkholes) – Weathered zones
- Swelling potential of soil & rock formations
- Significant groundwater inflows & High groundwater pressures

The Tight Time framework of the Metro Projects in the GCC countries

- Completion time: 5 to 6 years
- Decisions for the chosen construction methodologies must be based on well elaborated risk analyses



Groundwater aggressiveness in the GCC region

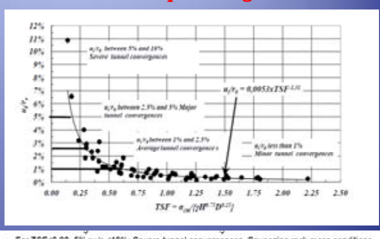
- **Cloride content**:
  between 50 – 55,000 mg/l
- **Sulphate content**:
  up to 5,500 mg/l
- **Magnesium content**:
  up to 1,000 mg/l
- **Groundwater temperature**:
  around 32°C – 35°C
- **pH values**:
  between 7 - 11

Tunnelling Challenges in Weak Rocks (e.g Simsima Limestone)

- Difficulty in measuring rock mass properties & assessing the design parameters
- The design values of shear strength and deformability parameters can be estimated: (i) empirically by using rock mass classification systems and well justified mathematical formulae and / or (ii) by evaluating properly good quality in-situ test results (e.g. dilatometer tests, pressuremeter tests) and laboratory test results

Average to Significant overburden heights & Adverse geotechnical conditions– Squeezing

- Altered Conglomerates and Ophiolites in North Oman



- For TSF≤0.20, 5%<u_t/r_t≤10%: Severe tunnel convergences. Squeezing rock mass conditions.
- For 0.20<TSF≤0.30, 2.5%<u_t/r_t≤5%: Major tunnel convergences. Very weak rock mass conditions.
- For 0.30<TSF≤0.60, 1%<u_t/r_t≤2.5%: Average tunnel convergences. Weak rock mass conditions
- For TSF>0.60, u_t/r_t ≤1%: Minor tunnel convergences. Fair rock mass conditions

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Tunnelling Challenges in Weak Rocks (e.g. Simsima limestone) Design geotechnical parameters

Inspection / Testing of Rockmass → Classification System → Rockmass Index (RMR, GSI) → Empirical formulae → Design values of rockmass properties

- Rock classification systems are not directly applicable in weak rocks (e.g. Simsima limestone)
- Methods commonly adopted are:
1. The use of modified “Classification systems” & modified empirical formulae and /or
2. Monitoring + Back Analysis ⇨ Design values of rock mass properties
3. Elaboration of tests results versus shear strain

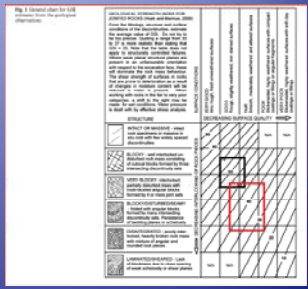
Tunnelling Challenges in Weak Rocks (e.g. Simsima limestone) Classification Systems

- Classification System for Simsima Limestone (Weathered and Fresh)

A.Structure:
1. Compact character and any potential fracturation
2. Presence of inherent primary (sedimentary) discontinuities
3. Presence and amount of vugs

B. Quality:
1. Weathered fraction in the rockmass
2. The quality of the weathered material
3. The participation of any other type of rock, as sedimentary intercalation (e.g. veins of marls, shales, etc. in limestone)

C. Groundwater conditions

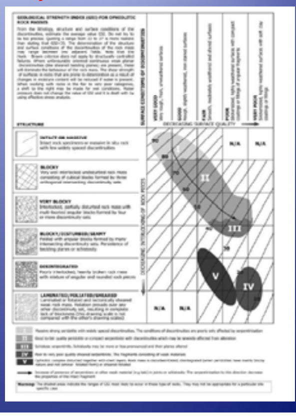


Tunnelling in Weak Rocks & in Rocks with Karstic features -Design & Construction solutions

- Control of face instability:
1. Closed face TBM: Ideal for face instability / deformation control
2. NATM (conventional)
- Reduction of face area by multi – drifting, side drifting, pilot tunnel
- Use of forepoling / spiling / pipe arch
- Installation of fiberglass nails at the face

Tunnelling Challenges in Weak Rocks (e.g. Altered Ophiolites) Classification Systems

- Classification System for Ophiolites in Oman
- ✓ Consideration of various alteration degrees that downgrade significantly the geomechanical properties

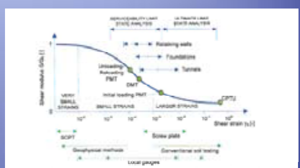


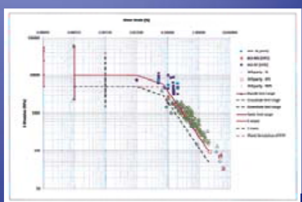
Tunnelling in Rocks with Karstic features Design & Construction solutions

- Cavity mitigation plan in place, prior to the tunnelling operations
- Well described grouting methodologies in place, prior to the tunnelling operations

Tunnelling Challenges in Weak Rocks

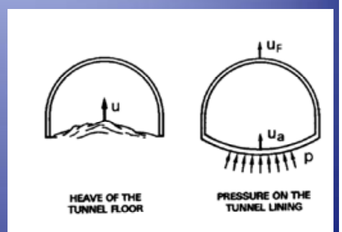
- Representation of the rockmass stiffness as a function of the shear strain amplitudes
- Proposed Rock mass stiffness value at the range of shear strain: $0.5x10^{-3}$ to 10^{-3} (*dilatometer tests and in the initial loading cycles of pressuremeter tests*)





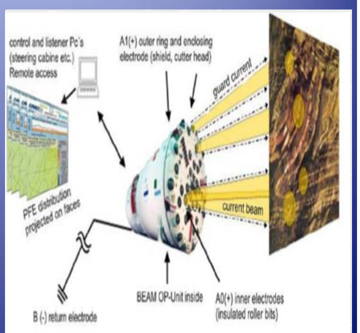
Tunnelling challenges in Rocks with high swelling potential (e.g Rus formation, Mudstones, Marls, altered Conglomerates & Ophiolites,)

- Swelling = Increase in volume when the Rocks come into contact with water
- Swelling occurs in Rocks containing clay minerals and / or anhydrite. **Linked with the chemical composition of the Rocks**
- In tunnelling : Swelling of rocks manifests itself as a heave of the tunnel floor, or as pressure on the invert arch
- Swelling pressures can take place rapidly
- In some cases , swelling processes continue several decades after tunnel completion



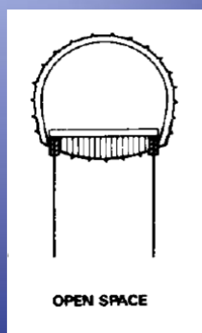
Tunnelling Challenges in Weak Rocks & in Rocks with Karstic features

- Face instability and deformations
- Explorations of karstic features:
1. Geological mapping of surface depressions
2. Geophysical investigations
3. Detailed investigations of the pin-pointed geophysical anomalies
4. Exploration ahead of the tunnel face, during TBM operations



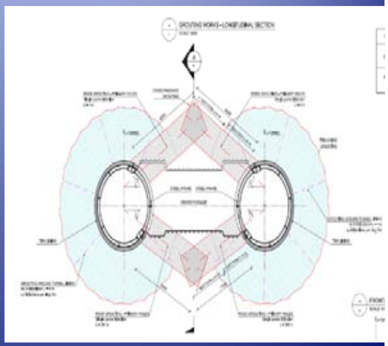
Tunnelling in Rocks with high swelling potential Design & Construction solutions

- Resistant invert arch
- Anchoring system along the tunnel invert
- Open space under the road/rail way plate

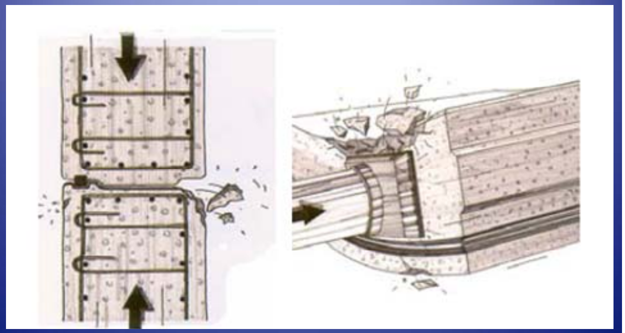


Tunnelling in adverse groundwater conditions
Design & Construction solutions

- **Adverse groundwater conditions:**
1. Significant groundwater inflows (e.g. 3,000m³ /hr – 5,000m³/hr)
2. High groundwater pressures (e.g. > 4 bars)
- **Solution:**
1. Combination of dewatering & grouting
2. Closed face TBM



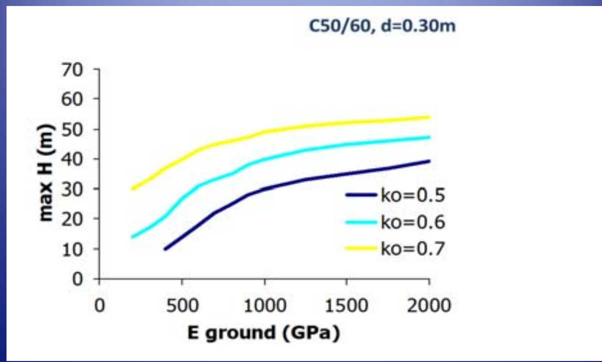
Tunnelling in aggressive groundwater conditions
Design & Construction solutions



Tunnelling challenges in "Squeezing" Rocks
(e.g. altered Conglomerates & Ophiolites,)

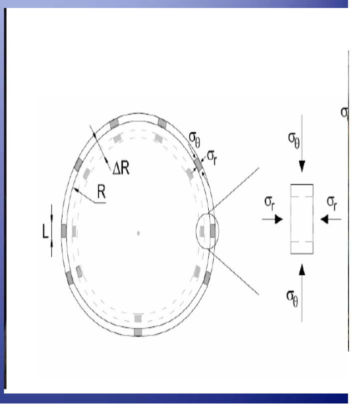
- Squeezing phenomena are related to the development of excessive "time dependent rock loads" & movements
- Squeezing conditions depend on ratio: (σ_{cm} ; rockmass strength / σ_1 ; in in situ vertical stress). It is a yielding mechanism of hard rocks in very deep tunnelling
- Failures of primary lining
- Tunnel collapses

Tunnelling in aggressive groundwater conditions
Design & Construction solutions



Tunnelling in "Squeezing" Rocks
Design & Construction solutions

- Very stiff support
- Ring Closure close to face
- Installation of yielding support :
1. Yielding (Sliding) steel sets with
2. Limiting Stress Controllers (LSC) in the shotcrete



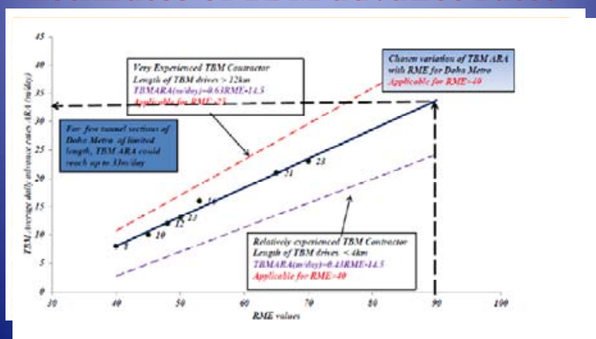
Tunnelling Challenges
"LARGE SCALE PROJECTS to be completed in a tight time framework

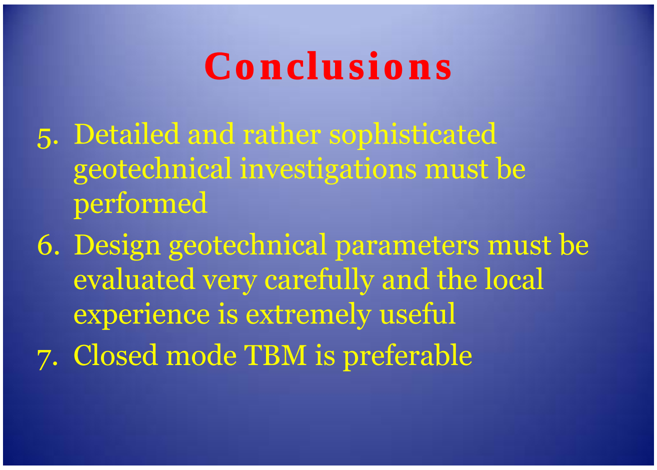
- Mechanized tunnelling – TBM
- Choice of the right type and number of TBMs
- The choice must be based on: (i) the good knowledge of ground and groundwater conditions and (ii) the results of well elaborated risk analyses
- "Safe" estimations of TBM advance rates

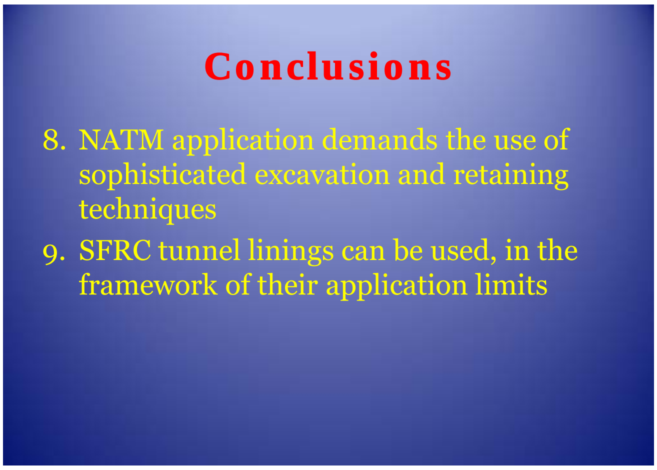
Tunnelling in aggressive groundwater conditions
Design & Construction solutions

- Tunnel final linings with Steel (S) Fiber (F) Reinforced (R) Concrete (C)
- SFR improves the concrete behavior with respect to corrosion because:
1. Fibers reduce the space or porous area for the propagation of corrosive agents
2. Fibers exhibit smaller cathodic surface and consequently limits corrosion currents
3. Fibers reduce the cracks openings due to shrinkage
4. Even total corrosion of the fibers does not cause splintering of the concrete

Tunnelling Challenges – Estimates of TBM advance rates







Ilias K. Michalis for the last 3 years is continuously working with Deutsche Bahn Engineering & Consulting offering his consulting services to the Technical Department of Qatar Rail both for Doha Metro and Long Distance projects. The focus of his services is on to providing technical solutions to the significant tunneling and geotechnical design issues of the aforesaid projects. He has 20 years of experience working as tunnel and geotechnical expert in major infrastructure projects, including among others: (a) Athens and Thessaloniki Metros in Greece, (b) the 700km Egnatia Motorway in North Greece and (c) three major concession motorways in Greece of total length of 700km approximately. He is an author of more than 25 scientific publications and has delivered until now 8 invited lectures to regional (GCC) and international tunnelling and geotechnical conferences.

ΔΙΑΚΡΙΣΕΙΣ ΕΛΛΗΝΩΝ ΓΕΩΜΗΧΑΝΙΚΩΝ

Προσκεκλημένη διάλεξη Ηλία Μιχάλη σε συνέδρια σε Gulf Cooperation Council Countries

Το μέλος της ΕΕΕΕΓΜ Ηλίας Μιχάλης προσεκλήθη και έδωσε διάλεξη σε μια σειρά συνεδρίων στην περιοχή των χωρών του Συμβουλίου Συνεργασίας Περσικού Κόλπου (Gulf Cooperation Council Countries). Προσεκλήθη στο BRIDGES & HIGHWAYS OMAN Conference / Tunnelling Focus Day, 13 September 2015, Muscat, OMAN, στο RET - Rail Expansion & Technology Congress MENA 2015, 15 September 2015, Dubai, UAE, καθώς και στο Qatar Transport Infrastructure 2015 Conference, 13 October 2015, Doha, Qatar.

Η παρουσίαση της διάλεξής του στα συνέδρια δίνεται στις προηγούμενες σελίδες του περιοδικού.

ΝΕΑ ΑΠΟ ΤΙΣ ΕΛΛΗΝΙΚΕΣ ΚΑΙ ΔΙΕΘΝΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΝΩΣΕΙΣ

ISSMGE Webinars

Samuel I.K Ampadu (Ghana)
December 2015
Characterizing lateritic soils

Serge Leroueil (Canada)
January 2016
Consolidation and creep

XIV Croce Lecture

Since 2000 the Italian Geotechnical Society (AGI) organises a yearly Conference dedicated to the memory of the late Professor Arrigo Croce, who was the first professor of Soil Mechanics in Italy, President of the AGI, vice-President for Europe of the ISSMFE and co-founder of the Technical Committee devoted to the Geotechnical Aspects of Preservation of Historic Sites.

Past Lectures delivered by: Prof. Carlo Viggiani,Prof. Michele Jamiolkowski, Prof. Ruggiero Jappelli, Prof. Giovanni Calabresi, Prof. Giovanni Barla, Prof. Beniamino D'Elia, Prof. Giuseppe Ricceri, Prof. Luciano Picarelli, Prof. Alberto Burghignoli, Prof. Eduardo Alonso, Prof. Renato Lancellotta, Prof. Kenichi Soga, Prof. Leonardo Cascini were printed on RIG: www.associazionegeotecnica.it/rig/croce_lecture.

We are now very happy to announce that the 2015 Lecturer will be: **Prof. Sarah M. Springman**, ETH Zürich, who decided to give a lecture with the very stimulating title: ***Lessons learnt for geotechnical engineering practice from field case histories and centrifuge modelling***

The Conference will be held on **December 11th, 2015 in Rome,** in the Conference Room of ANCE - Italian National Association of Building Contractors

All members of ISSMGE are welcome. We attach the registration form (PDF editable) that we would like you to distribute among your members.

We hope to see many of you at the Conference, to attend the interesting Lecture of Prof. Springman and have the chance of visiting Rome, a very pleasant town also in the period before Christmas.

With my best regards,

Nicola Moraci
AGI President

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ΠΡΟΣΕΧΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΚΔΗΛΩΣΕΙΣ

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

ICSGE 2015 - The International Conference on Soft Ground Engineering, 3-4 December 2015, Singapore, www.geoss.sg/icsge2015

The 1st International Conference on Geo-Energy and Geo-Environment (GeGe2015) 4th and 5th December 2015, Hong Kong, http://gege2015.ust.hk

2015 6th International Conference Recent Advances in Geotechnical Engineering and Soil Dynamics, December 7-11, 2015, New Delhi (NCR), India, wason2009@gmail.com; wasonfeq@iitr.ernet.in, sharmamukat@gmail.com; mukutfeq@iitr.ernet.in, gvramanaiitdelhi@gmail.com, ajaycbri@gmail.com

Southern African Rock Engineering Symposium an ISRM Regional Symposium, 5 January 2016, Cape Town, South Africa, http://10times.com/southern-african-rock

Environmental Connection Conference, February 16–19, 2016, San Antonio, Texas, USA, www.ieca.org/conference/annual/ec.asp

ASIA 2016 - Sixth International Conference on Water Resources and Hydropower Development in Asia, 1-3 March 2016, Vientiane, Lao PDR, www.hydropower-dams.com/pdfs/asia20161.pdf

GeoAmericas 2016 3rd Panamerican Conference on Geosynthetics, 11 – 14 April 2016, Miami Beach, USA, www.geoamericas2016.org

International Symposium on Submerged Floating Tunnels and Underwater Structures (SUFTUS-2016), 20-22 April 2016, Chongqing, China, www.cmct.cn/suftus

World Tunnel Congress 2016 "Uniting the Industry", April 22-28, 2016, San Francisco, USA, http://www.wtc2016.us

International Symposium "Design of piles in Europe - How did EC7 change daily practice?", 28-29 April 2016, Leuven, Belgium, www.etc3.be/symposium2016

7th In-Situ Rock Stress Symposium 2016 - An ISRM Specialised Conference, 10-12 May 2016, Tampere, Finland, www.rs2016.org

84th ICOLD Annual Meeting, 16-20 May 2016, Johannesburg, South Africa, www.icold2016.org

2nd International Conference on Rock Dynamics and Applications (RocDyn-2), 18 – 20 May 2016, Suzhou, China http://rocdyn.org

13th International Conference Underground Construction Prague 2016 and 3rd Eastern European Tunnelling Conference (EETC 2016), 23 to 25 May 2016, Prague, Czech Republic, www.ucprague.com

GEOSAFE: 1st International Symposium on Reducing Risks in Site Investigation, Modelling and Construction for Rock Engineering - an ISRM Specialized Conference, 25 – 27 May 2016, Xi'an, China, www.geosafe2016.org/dct/page/1

14th International Conference of the Geological Society of Greece, 25-27 May, Thessaloniki, Greece, www.ege2016.gr

NGM 2016 - The Nordic Geotechnical Meeting, 25 - 28 May 2016, Reykjavik, Iceland, www.ngm2016.com

International Mini Symposium Chubu (IMS-Chubu) New concepts and new developments in soil mechanics and geotechnical engineering, 26 - 28 May 2016, Nagoya, Aichi, Japan, www.jiban.or.jp/index.php?option=com_content&view=article&id=1737:2016052628&catid=16:2008-09-10-05-02-09&Itemid

19SEAGC – 2AGSSEAC Young Geotechnical Engineers Conference, 30th May 2016, Petaling Jaya, Selangor, Malaysia, seagc2016@gmail.com

19th Southeast Asian Geotechnical Conference & 2nd AGSSEA Conference Deep Excavation and Ground Improvement, 31 May – 3 June 2016, Subang Jaya, Malaysia, seagc2016@gmail.com

ISSMGE TC211 Conference Session within the framework of the 19th Southeast Asian Geotechnical Conference "GROUND IMPROVEMENT works: Recent advances in R&D, design and QC/QA"

ISL 2016 12th International Symposium on Landslides Experience, Theory, Practice, Napoli, June 12th-19th, 2016, www.isl2016.it

ℭ ℜ

ICONHIC 2016

1st International Conference on
Natural Hazards and Infrastructure: Protection, Design, Rehabilitation
28-30 June 2016, Chania, Greece
http://iconhic2016.com

We are delighted to invite you to Chania, Greece for the "1st International Conference on Natural Hazards and Infrastructure: Protection, Design, Rehabilitation".

With this new Conference we aim to bring under one roof specialists from the academia and industry on earthquake engineering, landslides, floods, tsunamis and hurricanes As we have witnessed several times in recent years, **our civil infrastructure is exposed to one or several of these threats and it is us, engineers, who mainly undertake the task of designing against them to minimize the risk and reduce fatalities.**

This conference gives us all the opportunity to not only state our case in our own specialty but also to watch and listen how our "scientific neighbors" are coping with in their fields of expertise; **this is the essence of cross-fertilization of knowledge.** To achieve this goal, our conference will include excellent keynote lectures, special and theme sessions as well as 5-in-5 presentations in what we hope to be a most lively event encouraging discussion on challenging topics.

Looking forward to meeting you in Greece on 28-30 June 2016 !

Professor George Gazetas, Conference Chairman

CONFERENCE THEMES

CROSS-CUTTING TOPICS

- Physical Modeling in Lab and Field: from miniature to large scale Testing
- Resilient Infrastructure Design: from structural Robustness to adaptive Systems
- Community Preparedness and Resilience to Natural Disasters
- Cascading Hazards and Multiple Risk Assessment Sensing and Monitoring: from local Instrumentation to Satellite technologies
- Case studies and lessons learned
- Eco-friendly mitigation techniques
- Engineering in harsh Environments (Offshore Challenges, Arctic Areas, Deep-Water Systems

EARTHQUAKES

- Engineering Seismology & Ground Motion Simulation
- Assessment, Analysis and Retrofitting of Structures
- Aging Infrastructure and future Earthquakes: concrete, steel and masonry Structures
- Seismic Design of Foundations and Underground Structures
  Ground Failure & Liquefaction: Analysis and effects on Structures and Lifelines
- Isolation and Energy Dissipation Devices

LANDSLIDES

- Landslides Prevention and Mitigation: Design Practice and New Concepts
- Hazard Assessment and Sensing
- Offshore Landslides and effects on submarine Structures and Pipelines
- Earthquake/Flood induced Landslides : Analysis and Modeling

FLOODS & TSUNAMIS

- Flood-resistant Design: Applications on Dams and Levees
- Tsunami-resistant Design
- Tsunami Generation – Propagation
- Flood & Tsunami Disasters: Prevention – Mitigation

TECHNICAL SESSIONS

Keynote Lectures (KS)

They will be delivered to all participants together by 2 internationally renowned experts moderated by a panel consisting of Steering Committee members. In order to cross-fertilize experience and methodologies, these sessions will have a slightly longer duration allowing for some discussion- which of course could then be continued at the end of the session.

General Sessions (GS)

These sessions will focus on specific research fields. They will be organized and moderated by 2 chairpersons and will be running in parallel sessions. Each GS will include oral presentations of papers submitted to the conference.

Special Sessions (SS)

Special Sessions will include a panel of invited speakers on cross-cutting topics and will be even organized by companies. SS could include presentation of suitable papers submitted to the conference.

5-in-5 oral presentations

During this lively session, authors will be allocated a five minutes slot to underline their research results through a five slides presentation. Presentations will run in parallel in thematic kiosks within the same hall, allowing delegates to seamlessly move from one kiosk to another during the 1-minute break between consecutive presentations. The unique format of this session will encourage interaction between delegates and authors, and give the participants the opportunity to get a flavor of subjects other than their own specialty.

Posters

Physical Poster Boards will bear poster presentations during the coffee and lunch breaks

Please send your email at secreteriat@iconhic2016.com.

4th GeoChina International Conference Sustainable Civil Infrastructures: Innovative Technologies for Severe Weathers and Climate Changes, July 25-27, 2016, Shandong, China, http://geochina2016.geoconf.org

6th International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics August 1-6, 2016, Greater Noida (NCR), India, www.6icragee.com

EUROC 2016 - ISRM European Regional Symposium Rock Mechanics & Rock Engineering: From Past to the Future, 29-31 August 2016, Ürgüp-Nevşehir, Cappadocia, Turkey http://eurock2016.org

3rd ICTG – 3rd International Conference on Transportation Geotechnics 4 - 7 September 2016, Guimaraes, Portugal, www.civil.uminho.pt/3rd-ICTG2016

IAS'5 5th International Conference on Geotechnical and Geophysical Site Characterisation, 5-9 September 2016, Gold Coast, Queensland, Australia http://www.isc5.com.au

The World Multidisciplinary Earth Sciences Symposium– WMESS 2016

5-9 September 2016, Prague, Czech Republic
www.mess-earth.org

WMESS aims to provide a forum for discussion of the latest findings and technologies in different fields of Earth Sciences, to give opportunities for future collaborations, to be a platform for sharing knowledge and experiences in the fields of Earth Sciences and to lead for providing a forum for early career researchers for presentation of their work and discussion of their ideas with experts in different fields of Earth Sciences.

The main mission of the "World Multidisciplinary Earth Sciences Symposium - WMESS" is to lead to contribute in multidisciplinary studies related with atmosphere, biosphere, hydrosphere, lithosphere and pedosphere of the Earth and interaction of the human with them. As another mission it will provide a forum for this diverse range of studies which report very latest results and document emerging understanding of the Earth's system and our place in it.

Topics of the Conference

- Tectonics & Structural Geology
- Engineering Geology
- Geotechnics
- Hydro-Hydrogeological Sciences
- Natural Hazards
- Oil, Gas & Coal
- Geothermal Energy
- Geomorphology
- Geochemistry, Mineralogy, Petrology & Volcanology
- Stratigraphy, Sedimentology & Palaeontology
- Geophysics & Seismology
- Geodesy, Photogrammetry & Cartography
- Informatics, Geoinformatics & Remote Sensing
- Mining Engineering
- Mineral Processing
- Blasting & New Technologies
- Natural Resources
- Environmental Sciences
- Energy, Resources & Pollution & the Environment
- Environmental Legislation
- Biogeosciences
- Geological Heritage & Geoparks
- Urban Planning
- Atmospheric Sciences
- Oceanography
- Climatology
- Glaciology
- Modelling and Soft Computing in Earth Sciences
- Medical Geology
- Occupational Health and Safety

Contact: mess@mess-earth.org

SAHC 2016 - 10th international Conference on Structural Analysis of Historical Constructions 13-15 September 2016, Leuven, Belgium, www.sahc2016.be

13 Baltic States Geotechnical Conference Historical Experiences and Challenges of Geotechnical Problems in Baltic Sea Region, 15 - 17 September 2016, Vilnius, Lithuania, http://www.13bsgc.lt

EuroGeo 6 – European Regional Conference on Geosynthetics, 25 – 29 Sep 2016, Istanbul, Turkey, www.eurogeo6.org

5th International Scientific Conference on Industrial and Hazardous Waste Management
27 - 30 September 2016, Chania, Crete, Greece
http://hwm-conferences.tuc.gr

The conference topics will respond to significant issues that are interested the scientific community worldwide. For this reason the scheduled sessions and special workshops will mainly focus on:

- Industrial and Hazardous Waste Regulation / Legislation
- Industrial and Hazardous Waste Characterization
- Industrial and Hazardous Waste Management Practices Treatment and Disposal
- Industrial and Hazardous Waste Production, Minimization and Recycling
- Hazardous Waste Toxicology - Risk Assessment
- Treatment of Hazardous Waste Landfill and Mine Leachates
- Contaminant Release and Transport
- Management of Contaminated Sites
- Special Waste (Medical, Radioactive, WEEE, Agro-Industrial, Asbestos, Plastics etc.)
- War and Environmental Impact
- Waste - to - Energy Concepts
- Case Studies

Organizing Secreteriat: hwm.conferences@enveng.tuc.gr.

2nd International Specialized Conference on Soft Rocks
29-30 September 2016, Cartagena, Colombia

Contact Person: Mario Camilo Torres Suarez
Address: Calle 12C, No. 8-79 Of. 512
Telephone: +57 1 3340270
Fax: +57 1 3340270
E-mail: sociedadcolombianadegeotecnia@scg.org.co

ARMS 9, 9th Asian Rock Mechanics Symposium, ISRM Regional Symposium, 18-20 October 2016, Bali, Indonesia, http://arms9.com

Shaping the Future of Geotechnical Education
International Conference on Geo-Engineering Education
20 - 22 October 2016, Minascentro, Belo Horizonte, MG, Brazil
http://cobramseg2016.com.br/index.php/sfge-sobre/?lang=en

SFGE 2016 is the fourth in the series of international conferences about geo-engineering education organised by the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) through its Technical Committees and Member Societies. The previous conferences were:

- 1st International Conference on Geotechnical Engineering Education and Training, Sinaia, Romania, 12-14 June, 2000
- 1st International Conference on Education and Training in Geo-Engineering Sciences: Soil Mechanics and Geotechnical Engineering, Engineering Geology, Rock Mechanics (ICETGES), Constantza, Romania, 2-4 June, 2008
- Shaking the Foundations of Geo-Engineering Education (SFGE 2012), Galway, Ireland, 4-6 July, 2012

The latest conference was the first to be organised by TC306, the Technical Committee on Geo-Engineering Education of ISSMGE.

SFGE 2016 – Shaping the Future of Geotechnical Education is also organised by TC306 and will take place in Belo Horizonte, Brazil, 19-20 October, 2016, in conjunction with a series of geotechnical events organised by the Brazilian Society for Soil Mechanics and Geotechnical Engineering (ABMS). At least two sessions will be shared by SFGE 2016 and GeoJovem 2016, the Brazilian and South American Young Geotechnical Engineers' conference, thus bringing together educators and students.

The Geotechnical Engineering and Rock Mechanics Brazilian conferences will also create a proper opportunity to enrich SFGE 2016 sessions with contributions from industry regarding geo-engineering education.

The two most recent conferences have created a momentum. Those who have attended can testify that the way they address geo-engineering education has been positively and significantly influenced by those conferences. SFGE 2016 shall be no different, so do plan to attend!

Topics

- Effective education in geotechnical principles
- Innovative geotechnical courses and programs
- Laboratory coursework
- Field coursework
- Project based coursework
- Active learning
- The role of internships
- IT applications
- Opinion articles

Contact: sfge2016@cobramseg2016.com.br

GeoAsia 6 - 6th Asian Regional Conference on Geosynthetics 8-11 November 2016, New Delhi, India, http://seags.ait.asia/news-announcements/11704

RARE 2016 Recent Advances in Rock Engineering 16-18 November 2016, Bangalore, India, www.rare2016.in

AfriRock 2017, 1st African Regional Rock Mechanics Symposium, 12 – 17 February 2017, Cape Town, South Africa, www.saimm.co.za/saimm-events/upcoming-events

World Tunnel Congress 2017
Surface problems – Underground solutions
9 to 16 June 2017, Bergen, Norway
www.wtc2017.no

"Surface problems – Underground solutions" is more than a slogan; for ITA-AITES and its members it is a challenge and commitment to contribute to sustainable development. The challenges are numerous and the availability of space for necessary infrastructure ends up being the key to good solutions. The underground is at present only marginally utilized. The potential for extended and improved utilization is enormous.

EUROCK 2017
13-15 June 2017, Ostrava, Czech Republic

Contact Person: Prof. Petr Konicek

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E-mail: petr.konicek@ugn.cas.cz

19th International Conference on Soil Mechanics and Geotechnical Engineering, 17 - 22 September 2017, Seoul, Korea, www.icsmge2017.org

GeoAfrica 2017
3rd African Regional Conference on Geosynthetics
9 – 13 October 2017, Morocco

11th International Conference on Geosynthetics (11ICG)
16 - 20 Sep 2018, Seoul South Korea
csyoo@skku.edu

10th Asian Rock mechanics Symposium - ARMS10
October 2018, Singapore

Prof. Yingxin Zhou
Address:
1 Liang Seah Street
#02-11 Liang Seah Place
SINGAPORE 189022
Telephone: (+65) 637 65363
Fax: (+65) 627 35754
E-mail: zyingxin@dsta.gov.sg

AFTES International Congress
"The value is Underground"
13-16 November 2017, Paris, France

World Tunnel Congress 2018
20-26 April 2018, Dubai, United Arab Emirates

UNSAT2018

The 7th International Conference on Unsaturated Soils
3 - 5 August 2018, Hong Kong, China
www.unsat2018.org

Following the success of the last conference held in Sydney, Australia in 2014, the HKUST is pleased to invite you to attend the 7th International Conference on Unsaturated Soils (UNSAT2018) in August 2018. This conference will cover a broad range of themes and provide an excellent opportunity for worldwide academics, engineers, scientists, government officials and planners to present and exchange the latest developments in the research and application of unsaturated soil mechanics.

The *2nd Blight Lecture* will be delivered by Professor Delwyn Fredlund at UNSAT2018.

Conference themes

Fundamental soil behaviour

- Micro- and macro-structure
- Water retention
- Stress-strain behaviour
- Multi-physic couplings (e.g., temperature, chemistry, biology etc.)
- Cyclic/dynamic behaviour
- Behaviour of multiphase soils, soft and hard rocks

New equipment and testing methods

- Advanced and new testing equipment
- Sensors for suction/moisture measurements
- In-situ/field testing

Modelling

- Constitutive modelling
- Physical modelling including centrifuge
- Numerical modelling and analysis

Geotechnical engineering problems

- Infrastructural developments such as earth and rockfill dams, roads, embankments and railways
- Natural and man-made hazards such as stability of fill slopes and rainfall-induced landslides
- Foundations in unsaturated soils
- Mining engineering

Energy and environmental issues

- Bio-engineering
- Vegetation effects
- Climate change, soil-atmospheric interaction
- Geo-environment such as capillary barriers, nuclear waste disposal and CO_2 sequestration
- Geo-energy such as energy piles, gas hydrates

Contact Us

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E-mail: unsat2018@ust.hk

☙ ❧

14th ISRM International Congress
2019, Foz de Iguaçu, Brazil

Contact Person: Prof. Sergio A. B. da Fontoura
E-mail: fontoura@puc-rio.b

ΕΝΔΙΑΦΕΡΟΝΤΑ ΓΕΩΤΕΧΝΙΚΑ ΝΕΑ

Video: Sinkhole on Queensland Beach!

The idyllic beach to Inskip Point in Queensland, Australia, had blue water golden sand and attracts campers and nature lovers. However, on Saturday night, an enormous hole opened suddenly and swept away tents and vehicles, causing chaos to campers who evacuated the area.

The idyllic beach to Inskip Point in Queensland, Australia, had blue water golden sand and attracts campers and nature lovers. However, on Saturday night, an enormous hole opened suddenly and swept away tents and vehicles, causing chaos to campers who evacuated the area. Fortunately no injuries were reported, according to Australian media.

It seems that fishermen were the first who realized that something was happening. The campers said they heard a noise like a storm and barely had time to leave before the beach disappears under water -along with tents, caravans, small vehicles and other possessions.

That point of the beach is about 7.5 meters under water. The sea 'swallowed' about 200 meters of beachfront.

The campers evacuated the beach and access is no longer allowed to a large part of it.

A Queensland Government geotechnical engineer thinks that the event may have been a "near-shore landslide" rather than a sinkhole.

Watch the video provided under media below!

https://www.youtube.com/watch?v=zQSFQGoztDI

(Geoengineer.org, Monday, 28 September 2015)

Huge sinkhole among houses in Hertfordshire

Dozens of families were left without water and gas. Miraculously there were no casualties in Hertfordshire in London, UK when a twenty-meter hole in the ground, opened up!

The phenomenon is becoming increasingly alarming, taking a global dimension. The sudden decline of soil in the British area caused no casualties, but left 58 families without water, gas and electricity, as well as the city's infrastructure collapsed.

A large number of inhabitants were forced to evacuate the area, until it is safe to return to their homes and repair the damage to the networks.

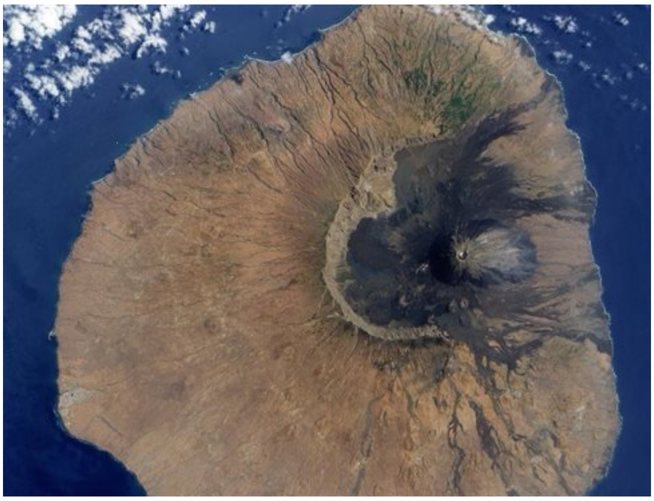
The sinkhole created a diameter of twenty meters and a depth of ten. A few days ago a small hole had appeared in the road, however, authorities felt that it was caused by the intense rains that preceded.

In August a twelve-meter sinkhole had appeared in the main street of Manchester. Such phenomena have been recorded around the globe.

(Geoengineer.org, Wednesday, 07 October 2015)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΣΕΙΣΜΟΙ

Μισό βουνό στη θάλασσα
Αρχαία κατάρρευση ηφαιστείου «σήκωσε τσουνάμι 180 μέτρων»



Σχεδόν το μισό ηφαίστειο στο νησί Φόγκο (δεξιά) έχει πέσει στον ωκεανό

Τα αποτελέσματα της καταστροφής είναι ακόμα ορατά σε αυτό το ηφαιστειογενές νησί του Πράσινου Ακρωτηρίου: η μία πλευρά του ηφαιστείου λείπει, στη θέση της μόνο μια μεγάλη γεωλογική ουλή. Σε ένα άλλο νησί, 55 χιλιόμετρα μακριά, ογκόλιθοι των 700 τόνων δείχνουν να έχουν μετακινηθεί μυστηριωδώς 200 μέτρα πάνω από την επιφάνεια της θάλασσας.

Τι συνδέει τα δύο νησιά; Μια κατολίσθηση βιβλικών διαστάσεων που σήκωσε τσουνάμι 200 μέτρων, ένα από τα μεγαλύτερα στο γεωλογικό αρχείο, προτείνει νέα μελέτη στη διαδικτυακή έκδοση του Science.

Η κατολίσθηση συνέβη πριν από 73.000 χρόνια στο νησί Φόγκο, έξω από τις ακτές της Δυτικής Αφρικής, στην πραγματικότητα ένα ηφαίστειο που ορθώνεται σήμερα σε ύψος 2.829 μέτρων και εκρήγνυται τακτικά, περίπου κάθε 20 χρόνια.

Κανείς δεν αμφισβητεί ότι, κάποια στιγμή στο μακρινό παρελθόν, ο μισός ηφαιστειακός κώνος κατέρρευσε -τα πετρώματα από τα οποία αποτελούνταν βρίσκονται σήμερα πεταμένα στο βυθό. Αυτό για το οποίο δεν ήταν βέβαιοι οι γεωλόγοι είναι το εάν η πλαγιά κατέρρευσε ολόκληρη απότομα, ή αν διαλύθηκε σταδιακά, σε μια σειρά μικρότερων κατολισθήσεων.

Η νέα έρευνα, την οποία υπογράφει διεθνής ομάδα ερευνητών, καταλήγει στην εξήγηση ότι ολόκληρη η πλαγιά, περίπου 160 κυβικά χιλιόμετρα βράχου, έπεσε με τη μία στη θάλασσα.

Οι μάρτυρες αυτού του καταστροφικού συμβάντος βρίσκονται ακόμα και σήμερα στο Σαντιάγο, ένα άλλο νησί στο αρχιπέλαγος του Πράσινου Ακρωτηρίου. Οι ερευνητές παραξενεύτηκαν από ογκόλιθους, ορισμένοι από τους οποίους έχουν μέγεθος φορτηγού και βάρος 770 τόνους, οι οποίοι βρίσκονται διάσπαρτοι μέσα στο νησί, ακόμα και σε υψόμετρο 200 μέτρων και σε απόσταση 600 μέτρων από τις ακτές.

Οι ογκόλιθοι στο νησί Σαντιάγο διαφέρουν από τη γεωλογία του τοπίου (σ.ε. βλέπε επισυναπτόμενη περίληψη μελέτης)

Οι γιγάντιοι βράχοι μοιάζουν αταίριαστοι σε ένα τοπίο από φρέσκα ηφαιστειακά πετρώματα, λένε οι ερευνητές. Μοιάζουν όμως με τους ασβεστόλιθους και τους βασάλτες που βρίσκονται στην άκρη του νησιού.

Η μόνη ρεαλιστική εξήγηση που μπόρεσε να δώσει η ερευνητική ομάδα είναι ότι οι ογκόλιθοι βρίσκονταν κάποτε στις απόκρημνες ακτές του νησιού και μεταφέρθηκαν στο εσωτερικό του από κάποιο γιγάντιο τσουνάμι. Με βάση την ενέργεια που θα απαιτούνταν για να συμβεί κάτι τέτοιο, οι ερευνητές υπολογίζουν ότι το κύμα πρέπει να είχε τουλάχιστον 180 μέτρα ύψος και να έφτασε σε υψόμετρο τουλάχιστον 200 μέτρων καθώς κατέκλυζε τα υψώματα του Σαντιάγο.

Για να χρονολογήσουν το συμβάν, οι ερευνητές εξέτασαν τους ογκόλιθους και μέτρησαν την περιεκτικότητα σε ήλιο-3, ένα ισότοπο του αερίου ήλιου που παράγεται από την προσκρουση κοσμικών ακτίνων όταν ο βράχος είναι εκτεθειμένος. Αυτό έδωσε μια ηλικία 73.000 ετών, εντός του ορίου των ηλικιών που είχε δώσει μια προηγούμενη μελέτη για την κατολίσθηση ή τις κατολισθήσεις, πριν από 65 έως 124 χιλιάδες χρόνια.

Ένα κύμα ύψους 180 μέτρων μπορεί να ακούγεται υπερβολικό, αν σκεφτεί κανείς ότι τα τσουνάμι που σάρωσαν τον Ινδικό το 2004 και την Ιαπωνία το 2011 δεν ξεπερνούσαν τα 40 μέτρα.

Κι όμως, στο πρόσφατο παρελθόν έχουν καταγραφεί πολύ μεγαλύτερα κύματα. Το ρεκόρ ανήκει στο συμβάν της 9ης Ιουλίου 1958, όταν ένας σεισμός έριξε 90 εκατομμύρια τονους βράχου στον απομονωμένο κόλπο Λιτούγια της Αλάσκας: το κύμα που έσκασε στην απέναντι πλαγιά έφτανε σε ύψος τον απίστευτο αριθμό των 550 μέτρων.

Υπάρχει βέβαια μια σημαντική διαφορά: τα τσουνάμι της Αλάσκα και του νησιού Φόγκο, ξεκίνησαν από τοπικές κατολισθήσεις, ουσιαστικά σημειακές πηγές στον χάρτη. Αντίθετα, τα τσουνάμι του Ινδικού και της Ιαπωνίας προκλήθηκαν από τη μετατόπιση του βυθού σε μήκος εκατοντάδων χιλιομέτρων, και η καταστροφική ενέργειά τους έφτασε πολύ μακρύτερα.

Ενδεχόμενη νέα κατολίσθηση στο Φόγκο δεν θα είχε τόσο μεγάλες συνέπειες, θα κατέστρεφε όμως και πάλι το απέναντι νησί του Σαντιάγο, με πληθυσμό 250.000 ανθρώπων.

Το ηφαίστειο του Φόγκο εξερράγη για τελευταία φορά πέρυσι, με αποτέλεσμα να καταστραφούν κτήρια και να εκτοπιστούν 1.200 άνθρωποι.

Όπως σχολιάζει ο Ροκάρντο Ραμάλο, πρώτος συγγραφέας της μελέτης, «οι κάτοικοι του Πράσινου Ακρωτηρίου έχουν σήμερα να ανησυχούν για το πώς θα ξαναφτιάξουν τις ζωές τους μετά την τελευταία έκρηξη.

«Το Φόγκο όμως μπορεί να καταρρεύσει και πάλι, γι΄αυτό και πρέπει να επαγρυπνούμε».

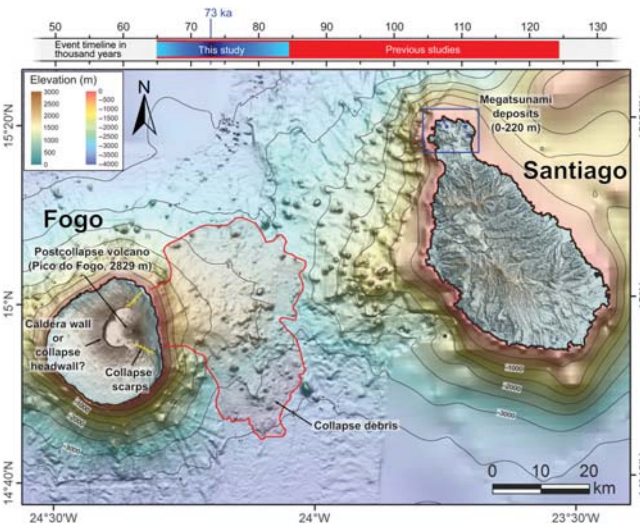
(Βαγγέλης Πρατικάκης / Newsroom ΔΟΛ, 04 Οκτ. 2015, http://news.in.gr/science-technology/article/?aid=1500030297&ref=newsletter)

Hazard potential of volcanic flank collapses raised by new megatsunami evidence

Ricardo S. Ramalho, Gisela Winckler, José Madeira, George R. Helffrich, Ana Hipólito, Rui Quartau, Katherine Adena and Joerg M. Schaefer

Abstract

Large-scale gravitational flank collapses of steep volcanic islands are hypothetically capable of triggering megatsunamis with highly catastrophic effects. Yet, evidence for the generation and impact of collapse-triggered megatsunamis and their high run-ups remains scarce or is highly controversial. Therefore, doubts remain on whether island flank failures truly generate enough volume flux to trigger giant tsunamis, leading to diverging opinions concerning the real hazard potential of such collapses. We show that one of the most prominent oceanic volcanoes on Earth—Fogo, in the Cape Verde Islands—catastrophically collapsed and triggered a megatsunami with devastating effects ~73,000 years ago. Our deductions are based on the recent discovery and cosmogenic. He dating of tsunamigenic deposits found on nearby Santiago Island, which attest to the impact of this giant tsunami and document wave run-up heights exceeding 270 m. The evidence reported here implies that Fogo's flank failure involved at least one fast and voluminous event that led to a giant tsunami, in contrast to what has been suggested before. Our observations therefore further demonstrate that flank collapses may indeed catastrophically happen and are capable of triggering tsunamis of enormous height and energy, adding to their hazard potential.



Onshore and offshore evidence for Fogo's flank collapse.

Fogo's flank collapse is documented by collapse scars onshore and by an avalanche debris field extending offshore, as illustrated in this digital elevation map of Fogo and Santiago and the surrounding seafloor. The study area on northern Santiago is marked with a rectangle, and the blue dots represent the general location of the tsunamigenic deposits reported in this study. The event timeline at the top compares the age interval previously suggested for Fogo flank collapse (in red) with the depositional age interval for Santiago's tsunamigenic deposits obtained in this study (blue). Topography of Fogo and Santiago corresponds to digital elevation model at 1:5000 scale. Seafloor mosaic is composed of low-resolution bathymetry (30 arc-second interval grid) and higher resolution bathymetry (200 m).

http://advances.sciencemag.org/content/1/9/e1500456.full

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΠΕΡΙΒΑΛΛΟΝ

Μερικά θέματα, ακόμα και αν δεν είναι του άμεσου ενδιαφέροντος των γεωτεχνικών μηχανικών, αξίζουν να αναφέρονται και να σχολιάζονται, όπως η παρακάτω αναφορά στο χωριό Ανάβρα.

Ανάβρα Μαγνησίας, το Γαλατικό χωριό της Ελλάδας. Ένα χωριό παράδειγμα προς μίμηση!

Η ανάπτυξη αυτής της απομακρυσμένης κοινότητας, αποτελεί παράδειγμα προς μίμηση για την ελληνική επαρχία, τους πολιτικούς της παράγοντες, αλλά και για κάθε πολιτικό.Ο δήμαρχος του ορεινού χωριού της Ανάβρας Μαγνησίας, αποδεικνύει ότι υπάρχουν περιπτώσεις Ελλήνων που πηγαίνουν κόντρα στην διαφθορά και την μιζέρια, και ενδιαφέρονται πραγματικά για τον τόπο τους.

Λίγα λόγια για την Ανάβρα, ένα «γαλατικό » χωριό, όπως το χουν ονομάσει κάποια δημοσιεύματα, που αντιστέκεται στη μιζέρια και τα ήθη των καιρών

Πρόκειται για ένα απομακρυσμένο ορεινό χωριό που κατάφερε μετά από συστηματική προσπάθεια, όχι μόνο να σταθεί όρθιο, αλλά να γίνει πρότυπο ανάπτυξης. Για τη μεγάλη ανατροπή που έρχεται από την Ανάβρα του νομού Μαγνησίας. Στις δυτικές πλαγιές της Οθρυος, σε υψόμετρο 1.000 μέτρων και σε απόσταση 40 χιλιομέτρων από την κοντινότερη κωμόπολη (τον Αλμυρό), οι 700 κάτοικοι, όλοι τους κτηνοτρόφοι, απολαμβάνουν εισοδήματα από 30 έως 100 χιλιάδες ευρώ και μια ποιότητα ζωής που μπορεί να συγκριθεί μόνο με την πλούσια Ελβετία.

Εδώ ο κόσμος δεν φεύγει προς τα αστικά κέντρα, αλλά επιστρέφει στο χωριό του. Με ποσοστό ανεργίας στο μηδέν και με μέσο όρο ηλικίας τα 40 έτη, ο πληθυσμός διπλασιάστηκε μέσα στα τελευταία 15 χρόνια.

Οι υποδομές του υποδειγματικές: Το αιολικό πάρκο, που δίνει έσοδα 100.000 ευρώ ετησίως στην κοινότητα, τα τρία υπερσύγχρονα κτηνοτροφικά πάρκα που στεγάζουν το χειμώνα (όταν η Ανάβρα αποκλείεται από τα χιόνια) 25.000 ζώα, το πρότυπο σφαγείο, που θυμίζει χειρουργείο, το διώροφο πάρκινγκ των 60 θέσεων, το γυμναστήριο με τα τελευταίας τεχνολογίας μηχανήματα, τα γήπεδα ποδοσφαίρου και μπάσκετ, το λαογραφικό μουσείο και φυσικά το περιβαλλοντικό-πολιτιστικό πάρκο, έκτασης 240 στρεμμάτων.

Η κοινωνική μέριμνα κατέχει πρώτιστη θέση: νηπιαγωγείο και δημοτικό του «κουτιού», αγροτικό ιατρείο (πάντοτε στελεχωμένο), δωρεάν στέγαση για τους δασκάλους και τους γιατρούς, «Βοήθεια στο σπίτι», σχεδιασμός για γηροκομείο, ακόμα και για πισίνα!

Ποιο είναι άραγε το μυστικό της επιτυχίας; Πώς μια μειονεκτική περιοχή κατάφερε το «θαύμα»;

Η μάχη με τη μιζέρια άρχισε τις αρχές της δεκαετίας του ’90, όταν τα ηνία της κοινότητας πήρε ο Δημήτρης Τσουκαλάς, ένας άνθρωπος που άφησε την Αθήνα για να γυρίσει στο χωριό του και να προσφέρει στη γενέτειρά του. Με ένα διάλειμμα 4 χρόνων, είναι από τότε κοινοτάρχης της Ανάβρας. Η κατάσταση που συνάντησε ήταν απελπιστική. Τα γελάδια, οι χοίροι και τα πρόβατα κυκλοφορούσαν ελεύθερα στο χωριό. Στους χωματόδρομους. Δεν υπήρχε πουθενά άσφαλτος. Το χειμώνα περπατούσες μέσα στη λάσπη, το καλοκαίρι η σκόνη σε έπνιγε.

«Προτεραιότητά μας ήταν η κατασκευή κτηνοτροφικών πάρκων, για να μπει τέλος στην αναρχία που επικρατούσε. Με φως, νερό και σωστή δόμηση, σταβλίζουν τα ζώα τους το χειμώνα. Τους υπόλοιπους μήνες βόσκουν ελεύθερα στα βουνά. Αυτός είναι και ο λόγος που το κρέας τους φημίζεται για την ξεχωριστή γεύση του. Η κτηνοτροφία είναι η πηγή των εισοδημάτων στην Ανάβρα», λέει στο ΕΤ.Κ ο πρόεδρος της κοινότητας.

«Ισως εγώ είμαι ο πιο φτωχός του χωριού, αφού η σύνταξη που παίρνω από τη ΔΕΗ, όπου έφυγα με το βαθμό του επιθεωρητή, κυμαίνεται στα 2.500 ευρώ».

Βάσεις ανάπτυξης

Η πρόοδος συνεχίστηκε με την κατασκευή του σφαγείου βιολογικής γραμμής, το μοναδικό δημόσιο στην Ελλάδα, με πιστοποίηση από τη ΔΗΩ. Ετσι μπήκε τάξη και δημιουργήθηκε η βάση για την ανάπτυξη της βιοκτηνοτροφίας, με συνέπεια οι παραγωγοί να κερδίζουν σημαντικά ποσά από τις επιδοτήσεις της Ευρωπαϊκής Ενωσης, με χρήματα της οποίας έγιναν οι παραπάνω υποδομές.

Ξύπνιος, σχολαστικός και με όραμα ο πρόεδρος παρακολουθεί ανελλιπώς κάθε κοινοτικό πρόγραμμα και το εκμεταλλεύεται δεόντως. Το μεγαλύτερο επίτευγμά του, όπως λέει ο ίδιος, είναι ότι κατάφερε να αλλάξει τη νοοτροπία των συγχωριανών του. Στην αρχή έφεραν αντιρρήσεις στα σχέδιά του, αλλά με επιμονή και υπομονή τούς έπεισε.

Τώρα τους καλεί να δημιουργήσουν τυποποιητήρια για τα προϊόντα τους, ώστε να ολοκληρωθεί το φάσμα της βιολογικής παραγωγής. Η Ανάβρα δεν βάσισε την ανάπτυξή της στο κράτος. Ηθελε έσοδα δικά της. Ετσι, πριν από τρία χρόνια ολοκλήρωσε το έργο του αιολικού πάρκου, με τις 20 ανεμογεννήτριες και ανάδοχο την ισπανική εταιρία Gamesa.

Εσοδα

Το ρεύμα το αγοράζει η ΔΕΗ, ενώ η κοινότητα για τη χρήση του χώρου, που της ανήκει, εισπράττει έως και 100.000 ευρώ το χρόνο.

Υπό δημοπράτηση βρίσκεται και η ανάπτυξη υδροηλεκτρικού εργοστασίου, από το νερό των πηγών της Ανάβρας. Από εκεί θα εισπράττονται άλλες 100.000 ευρώ. Η επιστροφή στις ρίζες είναι μια σταθερή πολιτική για τον κ. Τσουκαλά. Για να ενισχυθεί κι άλλο ο πληθυσμός, έκανε επέκταση του οικισμού και δίνει οικόπεδα σε άστεγους δημότες στο ? της αντικειμενικής τους αξίας και με αποπληρωμή σε 5 δόσεις.

Μετανάστες σε... ορεινό παράδεισο

«Δεν μας λείπει τίποτα. Εγώ, παρόλο που γεννήθηκα στην Αθήνα, ήρθα στο χωριό των γονιών μου, μόλις γνώρισα εδώ, κατά τη διάρκεια των διακοπών, το σύζυγό μου τον Παναγιώτη. Κάναμε τρία παιδιά. Δουλεύουμε με τον άνδρα μου φροντίζοντας τα 100 γελάδια που έχουμε. Πηγαίνουμε γυμναστήριο, πίνουμε τα τσιπουράκια μας στις 5 ταβέρνες του χωριού και αν θέλουμε νυχτερινή ζωή πεταγόμαστε μέχρι τον Δομοκό ή τον Αλμυρό», λέει στον ΕΤ.Κ η Νίκη Μηλιώνη.

Οι ηλικιωμένοι αισθάνονται ασφάλεια με τη μόνιμη παρουσία της αγροτικής γιατρού. Η Ελένη Τριανταφύλλου δέχεται 8 με 9 άτομα καθημερινά στο ιατρείο. Πήγε στην Ανάβρα στις 5 Αυγούστου και για τους επόμενους 9 μήνες που θα μείνει εκεί νιώθει τυχερή που βρέθηκε σε ένα τόσο φιλικό περιβάλλον. Μάλιστα, δεν επιβαρύνεται από την τσέπη της, γιατί μενει σε διαμέρισμα που της παραχώρησε η κοινότητα (όπως και οι τρεις δάσκαλοι).

Ο Αποστόλης Καπέλος και ο Πολύζος Κανατούλης είναι δυο νέοι κτηνοτρόφοι. Παίκτες στην ποδοσφαιρική ομάδα του χωριού (Α.Ο. ΟΘΡΥΣ), κάθονται στο καφενείο και συζητούν πώς θα αντιμετωπίσουν τον ΔΟΜΟΚΟ, την ισχυρότερη ομάδα της περιοχής. Περηφανεύονται για την προαγωγή στην Α" Ερασιτεχνική, αλλά και για τα επιτεύγματα του χωριού τους.

«Να πάτε να δείτε το περιβαλλοντικό πάρκο», μας λένε. Συναντάμε έναν παράδεισο. Αιωνόβια πλατάνια, σε μια διαδρομή πέντε χιλιομέτρων, με γεφύρια, παιδικές χαρές, πετρόκτιστη διακόσμηση και με τους λαγούς και τα ελάφια να ξεπετάγονται μέσα από τα ξέφωτα, με φόντο τον υδάτινο κόσμο του Ενιππέα (παραπόταμου του Πηνειού).

Εδώ κρατάει ακόμα το έθιμο της δρυστέλας , όπου οι νοικοκυραίοι πλένουν ρούχα και χαλιά με τα ορμητικά νερά των πηγών. Μια περιφραγμένη έκταση, συνολικού εμβαδού 240 στρεμμάτων, που αποτελεί ταυτόχρονα και μουσείο παράδοσης. Αλλη μια ευκαιρία για να εισπράττει η κοινότητα έσοδα, καθώς πολλά σχολεία εκδηλώνουν ενδιαφέρον για επίσκεψη και αναμένεται να μπει συμβολικό εισιτήριο για τα έξοδα συντήρησής του.

Το πλέον μεγαλόπνοο σχέδιο, αυτή την περίοδο, είναι η επικείμενη εγκατάσταση συστήματος τηλεθέρμανσης. Με προϋπολογισμό 1.700.000 ευρώ (από ευρωπαϊκά κονδύλια) και με μελέτη από το ΤΕΙ Κοζάνης η Περιφέρεια έχει πει ήδη το «ναι». Ενας κεντρικός λέβητας θα τοποθετηθεί στο πάνω μέρος του χωριού και από εκεί θα διοχετεύονται υπόγειοι αγωγοί από τους δρόμους της Ανάβρας με καυτό νερό. Το κάθε σπίτι θα συνδέεται με το σύστημα και θα έχει ολόκληρο το χειμώνα ζεστό νερό και θέρμανση, με μια ελάχιστη επιβάρυνση. Θα λειτουργεί με την καύση βιομάζας (κοπριές των ζώων, ξερά φύλλα, άχυρο κ.ά).

Νηπιαγωγείο

(NEWS MME PRESS ROOM / 9 Μαρτίου 2012, http://newsmme.blogspot.qa/2012/03/blog-post_280.html)

ΕΝΔΙΑΦΕΡΟΝΤΑ – ΛΟΙΠΑ

Η μεγαλύτερη προβλήτα στον κόσμο!

Η πόλη Progreso, στην μεξικανική πολιτεία του Γιουκατάν, μπορεί να υπερηφανεύεται για τη μεγαλύτερη προβλήτα στον κόσμο… Κατασκευασμένη από οπλισμένο...

Η πόλη Progreso, στην μεξικανική πολιτεία του Γιουκατάν, μπορεί να υπερηφανεύεται για τη μεγαλύτερη προβλήτα στον κόσμο… Κατασκευασμένη από οπλισμένο σκυρόδεμα, η προβλήτα απλώνεται έξω από τον Κόλπο του Μεξικού σε μια απόσταση 6,5 χιλιομέτρων, και μοιάζει περισσότερο με μια γέφυρα που φτάνει σε κάποια μακρινή χώρα. Το ασυνήθιστα μεγάλο μήκος είναι αναγκαίο για να μπορούν τα μεγάλα πλοία να ελλιμενίζονται, δεδομένου ότι η ακτή του Γιουκατάν είναι πολύ ρηχή.

Η προβλήτα ήταν αρχικά μήκους 2.100 μέτρων και κατασκευάστηκε μεταξύ 1937 και 1941, αντικαθιστώντας μια ξύλινη που χτίστηκε στις αρχές του περασμένου αιώνα. Το 1988, επιπλέον 4.000 μέτρα προστέθηκαν στο μήκος της για καλύτερη εξυπηρέτηση και διευκόλυνση. Η προβλήτα έχει αντέξει στο σκληρό θαλάσσιο περιβάλλον σε συνεχή λειτουργία για πάνω από 70 χρόνια χωρίς καμία σημαντική επισκευή ή συντήρηση, πέρα από τις επιδιορθώσεις ρουτίνας που γίνονται συχνά-πυκνά.

(Πηγή: perierga.gr, 9 Οκτωβρίου 2015, http://www.pentapostagma.gr/2015/10/h-%ce%bc%ce%b5%ce%b3%ce%b1%ce%bb%cf%8d%cf%84%ce%b5%cf%81%ce%b7-%cf%80%cf%81%ce%bf%ce%b2%ce%bb%ce%ae%cf%84%ce%b1-%cf%83%cf%84%ce%bf%ce%bd-%ce%ba%cf%8c%cf%83%ce%bc%ce%bf.html#ixzz3o9oetxFv)

Ελαφρύ σαν πούπουλο
«Αραχνοϋφαντο» μέταλλο είναι κατά 99,9% αέρας

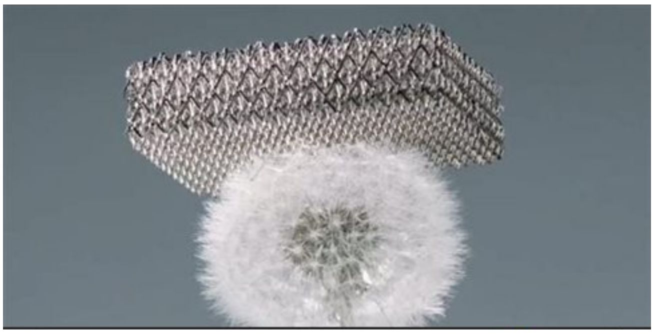
Το πιο ελαφρύ μέταλλο στον κόσμο, το οποίο θα μπορούσε να οδηγήσει στη δημιουργία μιας νέας γενιάς «πουπουλένιων» αεροσκαφών και διαστημικών σκαφών, ανέπτυξε η αμερικανική αεροβιομηχανία Boeing.

«Αεράτο» το νέο υλικό

Το υλικό microlattice, όπως ονομάζεται, αποτελείται σύμφωνα τους ειδικούς κατά 99,9% από αέρα και σχεδιάστηκε με πρότυπο την δομή των ανθρώπινων οστών.

Το μυστικό του νέου υλικού κρύβεται σε έναν σκελετό από κοίλα λεπτεπίλεπτα σωληνάκια, με τοιχώματα περίπου 1.000

φορές λεπτότερα από μια ανθρώπινη τρίχα, τα οποία συνδέονται μεταξύ τους δημιουργώντας ένα «αραχνοΰφαντο» μέταλλο, ικανό να στέκεται ακόμα και επάνω στο ντελικάτο κεφαλάκι από «χνουδωτούς» σπόρους μιας πικραλίδας.



Το νέο «πουπουλένιο» υλικό της Boeing μπορεί να σταθεί επάνω στο ντελικάτο κεφαλάκι από «χνουδωτούς» σπόρους μιας πικραλίδας

Σύμφωνα με τους επιστήμονες της Boeing, το καινοτόμο υλικό από... αέρα κοπανιστό είναι τόσο ανθεκτικό ώστε αν τυλιχθεί γύρω από ένα αβγό να μπορεί να το σώσει σε περίπτωση πτώσης του από ύψος... 25 ορόφων.

Ελαφρύτερα αεροσκάφη, «πουπουλένιες» διαστημικές αποστολές

Οι επιστήμονες υπογραμμίζουν ότι το νέο υλικό συνοδεύεται από πρωτοφανή μηχανική συμπεριφορά δεδομένου ότι πρόκειται για μέταλλο.

Παρά την πολύ χαμηλή του πυκνότητα εμφανίζει εξωπραγματική δυνατότητα απορρόφησης των εντάσεων, αλλά και επαναφοράς του όταν εκτεθεί σε συνθήκες συμπίεσης, η οποία μάλιστα ξεπερνάει το 50% της ισχύος της δύναμης που έχει δεχθεί.

Οι ίδιοι επισημαίνουν ότι το εν λόγω υλικό, πέρα από τη δημιουργία μιας νέας γενιάς αεροσκαφών θα μπορούσε να βοηθήσει την NASA να μειώσει το βάρος αλλά και τον όγκο των διαστημικών σκαφών ακόμα και κατά 40%, γεγονός που θα διευκόλυνε σημαντικά την εξερεύνηση του Διαστήματος.

(Ειρήνη Βενιού / BHMAScience Newsroom ΔΟΛ, 14 Οκτ. 2015, http://news.in.gr/science-technology/article/?aid=1500032743&ref=newsletter)

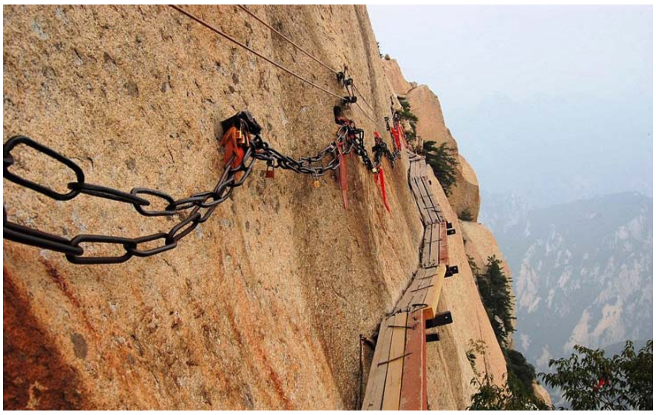
Γέφυρες που προκαλούν ίλιγγο

Απότομες, στενές, ψηλές και κρεμαστές. Απίθανα δημιουργήματα της αρχιτεκτονικής που προκαλούν ίλιγγο.

Από την Κριμαία έως το Περού και από την Κίνα μέχρι την Γαλλία οκτώ μοναδικές γέφυρες αποτελούν πρόκληση για κάθε επισκέπτη μια και η διάσχιση τους παρομοιάζεται ακόμα και με μια βόλτα με το τρενάκι του τρόμου.

Κριμαία

Κίνα

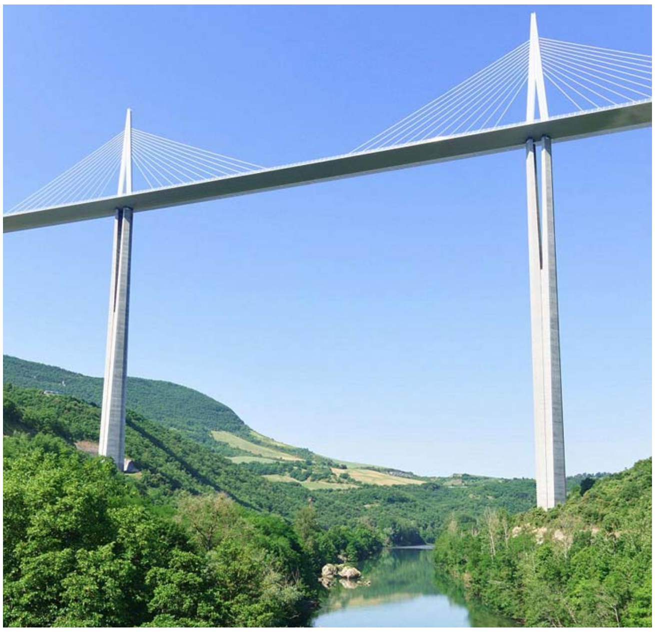


Πακιστάν

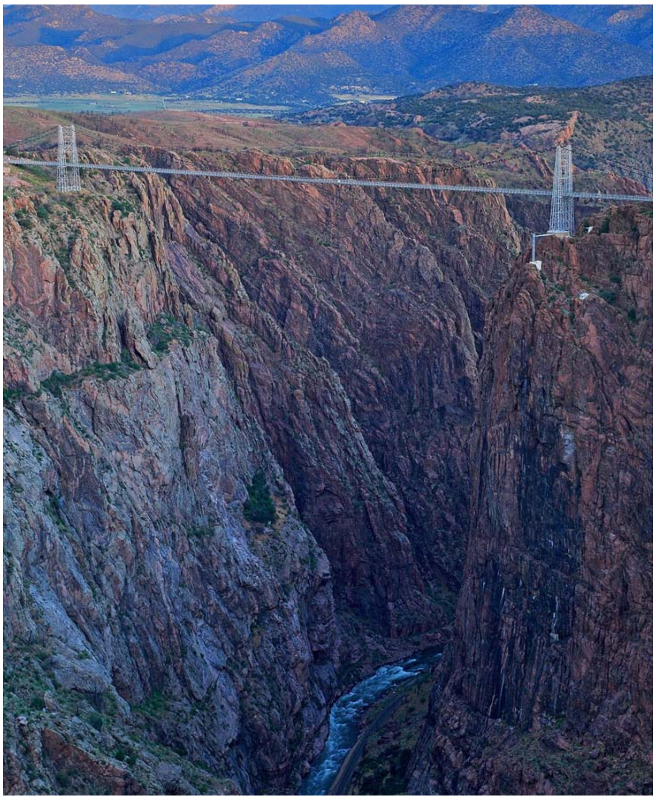
Μαλαισία

Νότια Κορέα

Γαλλία



ΗΠΑ



Περού

(iefimerida, 21.10.2015, http://www.iefimerida.gr/news/232028/arhitektones-me-trela-gefyres-poy-prokaloyn-iliggo-eikones)

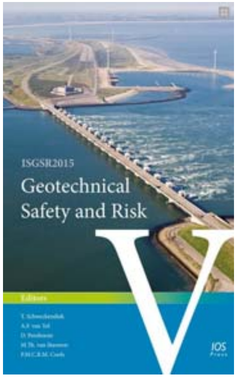
Κρεμαστή γέφυρα στη Γερμανία προκαλεί ίλιγγο!

Η νέα γέφυρα στο φαράγγι Geierley στη δυτική Γερμανία αποτελεί μία πρόκληση για τους επισκέπτες, άλλωστε αυτός ήταν και ο σκοπός της κατασκευής της στην περιοχή, να προσελκύσει περίεργους -και όχι μόνο- τουρίστες που θέλουν να τη διαβούν ατενίζοντας κάτω από τα πόδια τους το απόλυτο κενό. Η γέφυρα χρειάστηκε μόνο 130 μέρες για να φτιαχτεί και δεσπόζει πλέον στο καταπράσινο δάσος ανάμεσα στις πόλεις Morsdorf και Sosberg, κοντά στα σύνορα με το Λουξεμβούργο.

Η οικονομία της περιοχής έπρεπε να ενισχυθεί και για το λόγο αυτό σκέφτηκαν να δημιουργήσουν εδώ ένα αξιοθέατο που θα τραβούσε την προσοχή όλων. Σύμφωνα με την οικονομοτεχνική μελέτη, η γέφυρα θα προσελκύσει 170.000 επισκέπτες το χρόνο, με αρκετούς από αυτούς να διανυκτερεύουν στα καταλύματα των δύο πόλεων. Έτσι η τοπική οικονομία θα ενισχυθεί πέραν του γεγονότος ότι ένας περίπατος στο δάσος με θέα από ψηλά είναι ό,τι καλύτερο θα μπορούσε να ζητήσει κάποιος.

(perierga.gr, 8 Οκτωβρίου 2015, http://www.pentapostagma.gr/2015/10/k%cf%81%ce%b5%ce%bc%ce%b1%cf%83%cf%84%ce%ae-%ce%b3%ce%ad%cf%86%cf%85%cf%81%ce%b1-%cf%83%cf%84%ce%b7-%ce%b3%ce%b5%cf%81%ce%bc%ce%b1%ce%bd%ce%af%ce%b1-%cf%80%cf%81%ce%bf%ce%ba%ce%b1%ce%bb%ce%b5%ce%af.html#ixzz3o9qhw9g9)

ΝΕΕΣ ΕΚΔΟΣΕΙΣ ΣΤΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΠΙΣΤΗΜΕΣ



Geotechnical Safety and Risk V

Schweckendiek, T. , Van Tol, A.F., Pereboom, D., Van Staveren, A., Cools, P.M.C.B.M.

Geotechnical Risk and Safety V contains contributions presented at the 5th International Symposium on Geotechnical Safety and Risk (5th ISGSR, Rotterdam, 13-16 October 2015) which was organized under the auspices of the Geotechnical Safety Network (GEOSNet) and the following technical committees of the of the International Society of Soil Mechanics and Geotechnical Engineering (ISSGME):

- TC304 Engineering Practice of Risk Assessment & Management
- TC205 Safety and Serviceability in Geotechnical Design
- TC212 Deep Foundations
- TC302 Forensic Geotechnical Engineering

Geotechnical Risk and Safety V covers seven themes:

1. Geotechnical Risk Management and Risk Communication
2. Variability in Ground Conditions and Site Investigation
3. Reliability and Risk Analysis of Geotechnical Structures
4. Limit-state design in Geotechnical Engineering
5. Assessment and Management of Natural Hazards
6. Contractual and Legal Issues of Foundation and (Under)Ground Works
7. Case Studies, Monitoring and Observational Method

The 5th ISGSR is the continuation of a series of symposiums and workshops on geotechnical risk and reliability, starting with LSD2000 (Melbourne, Australia), IWS2002 (Tokyo and Kamakura, Japan), LSD2003 (Cambridge, USA), Georisk2004 (Bangalore, India), Taipei2006 (Taipei, Taiwan), the 1st ISGSR (Shanghai, China, 2007), the 2nd ISGSR (Gifu, Japan, 2009), the 3rd ISGSR (Munich, Germany, 2011) and the 4th ISGSR (Hong Kong, 2013)

(IOS Press, October 2015)

ΗΛΕΚΤΡΟΝΙΚΑ ΠΕΡΙΟΔΙΚΑ

http://www.issmge.org/en/resources/issmge-bulletin/784-vol-9-issue-5-october-2015

Κυκλοφόρησε το Τεύχος 5 του 9ου Τόμου του ISSMGE Bulletin (Οκτωβρίου 2015) με τα παρακάτω περιεχόμενα:

Research Highlights
Federal University of Rio Grande do Sul (UFRGS), Brazil

Report from Member Society
Kazakhstan Geotechnical Society (KGS) – First Kazakhstan-USA Geotechnical Engineering Workshop

Young Members' Arena
Experimental mechanics at the grain-scale in sand: Recent work from the geo-mechanics lab of Grenoble

Conference Report

The XVI European Conference of Soil Mechanics and Geotechnical Engineering - Edinburgh, UK on 13th – 17th September 2015

International Workshop on Geotechnology for Natural Hazards

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Hot News
Geomodels in Engineering Geology – an introduction

Event Diary

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Κυκλοφόρησε το Τεύχος #128 του **Newsletter του Geoengineer.org** (Οκτωβρίου 2015) με πολλές χρήσιμες πληροφορίες για όλα τα θέματα της γεωμηχανικής. Υπενθυμίζεται ότι το Newsletter εκδίδεται από τον συνάδελφο και μέλος της ΕΕΕΕΓΜ Δημήτρη Ζέκκο (secretariat@geoengineer.org).

Newsletter #22 - October 2015
www.itacet.org/Newsletter/22_2015/documents/NL_22_Ocotbre_2015.pdf

Κυκλοφόρησε το Τεύχος #22 του ITACET Newsletter (Οκτωβρίου 2015) με τα παρακάτω περιεχόμενα:

- President's address: Eng. Abdullah bin Abdulrahman Al-Muqbel
- Interview: Pauli Arenram, ITATech Committee Chairman
- Coming soon

  Shanghai17-18.10.2015. Challenges in Undersea Deep Long Distance Tunnels(UDLDT): Geotechnical aspect and Structural Design of Shield Tunnels
- Next events

  Tunnelling and Landslides. 10-11.012016. Riyadh, Saudi Arabia

  Rockfall Protection Techniques 12.01.2015 - Riyadh, Saudi Arabia
- Events report

  Deminar On Waterproofing & Drainage. Flums Hochwiese, Switzerland

  Health & Safety & Logistic In Tunnel Construction. Buenos Aires, Argentina

  Training Session Mechanised Tunnelling. Guadalajara, Mexico

  Soft Ground Urban Tunnelling & Rock Tunnelling In Karst WTC2015, Dubrovnik, Croatia.

  Tunnelling and Deep Excavation for Infrastructure in Urban Area Yangoon Myanmar
- ITACET Foundation Scholarship 2015

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