Ο Εκδότης των Νέων σε αναζήτηση δροσιάς ψηλά στο βουνό…

ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ

Τα Νέα της Ε Ε Ε Ε Γ Μ

100

Τα εκατοστίσαμε!!!

Αύγουστος 2005 ÷ Μάρτιος 2017

Αρ. 100 – ΜΑΡΤΙΟΣ 2017

SIMSG ISSMGE

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Πριν από 12 χρόνια, τον Αύγουστο 2015 κυκλοφόρησε το πρώτο τεύχος του ηλεκτρονικού περιοδικού της ΕΕΕΕΓΜ με τίτλο "ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ". Γράφαμε τότε:

Το δελτίο αυτό αποτελεί μια πρώτη προσπάθεια για την τακτική ενημέρωση των μελών της ΕΕΕΕΘ. Σκοπός του δελτίου είναι να ενημερώνει τους συναδέλφους σε τρέχοντα θέματα ενδιαφέροντος, αλλά και να αποτελέσει, το συντομότερο δυνατόν, βήμα διαλόγου αλλά σύντομης παρουσίασης γεωτεχνικών επιστημονικών θεμάτων ευρύτερου ενδιαφέροντος. Εκ των προτέρων ζητάμε από όλους τους συναδέλφους την επιείκειά τους για την μορφή του πρώτου αυτού δελτίου, αλλά επίσης και τις προτάσεις τους για την βελτίωση της μορφής και της ύλης του.

ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΘ

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Περιεχόμενα

Προοίμιο

Το δελτίο αυτό αποτελεί μια πρώτη προσπάθεια για την τακτική ενημέρωση των μελών της ΕΕΕΕΘ. Σκοπός του δελτίου είναι να ενημερώνει τους συναδέλφους σε τρέχοντα θέματα ενδιαφέροντος, αλλά και να αποτελέσει, το συντομότερο δυνατόν, βήμα διαλόγου αλλά σύντομης παρουσίασης γεωτεχνικών επιστημονικών θεμάτων ευρύτερου ενδιαφέροντος. Εκ των προτέρων ζητάμε από όλους τους συναδέλφους την επιείκειά τους για την μορφή του πρώτου αυτού δελτίου, αλλά επίσης και τις προτάσεις τους για την βελτίωση της μορφής και της ύλης του.

Γενική Συνέλευση – Αρχαιρεσίες

Την Τρίτη 24 Μαΐου 2005 πραγματοποιήθηκε η ετήσια Γενική Συνέλευση της εταιρείας, η οποία ήταν και καταστατική και εκλογική.

Από τις προταθείσες αλλαγές στο καταστατικό έγιναν δεκτές οι αναφερόμενες στην αλλαγή του ονόματος του σωματείου, στους σκοπούς του, στις προϋποθέσεις που πρέπει να πληρούν τα υποψήφια μέλη, στην δυνατότητα παράτασης της θητείας της Εκτελεστικής Επιτροπής, στην διάρκεια της θητείας της Εξελεγκτικής Επιτροπής και στο ποσό των χρημάτων σε ρευστό που μπορεί να παραμένει στο ταμείο. Η αλλαγή στο άρθρο περί Γενικής Συνέλευσης και διαδικασίας εκλογής της Εκτελεστικής και της Εξελεγκτικής Επιτροπής (ψήφος δι΄ αλληλογραφίας) παραπέμφθηκε για συζήτηση και ψήφιση στην επόμενη Γενική Συνέλευση, η οποία θα πρέπει να είναι και αυτή καταστατική.

Κατά τις αρχαιρεσίες, που ακολούθησαν, για την εκλογή των μελών της Εκτελεστικής και της Εξελεγκτικής Επιτροπής εξελέγησαν, για την Εκτελεστική Επιτροπή, οι:

ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΘ – Αρ. 1 - ΑΥΓΟΥΣΤΟΣ 2005 Σελίδα 1

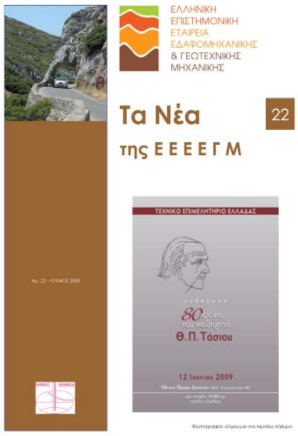
Το πρώτο τεύχος είχε μόλις 6 σελίδες και περιείχε λίγες ειδήσεις από την ελληνική και διεθνή γεωτεχνική κοινότητα. Το επόμενο τεύχος εκδόθηκε 6 μήνες αργότερα, τον Μάρτιο 2006, και από τότε ανά δίμηνο ή τρίμηνο, στην αρχή, και ανά μήνα τα τελευταία 3 χρόνια, αυξάνοντας, σταδιακά τον αριθμό των σελίδων του και την ποικιλίας της ύλης του.

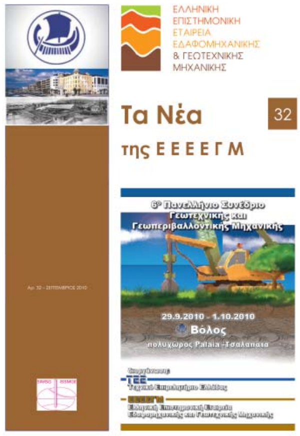
Από τον Απρίλιο 2007 άλλαξε η μορφή του εξωφύλλου του, η οποία διατηρείται ίδια έως σήμερα.

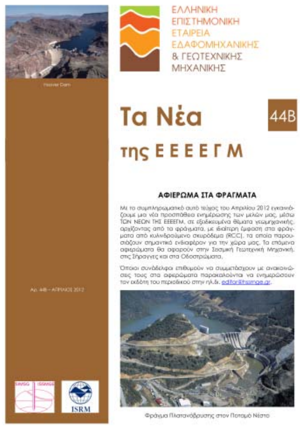
Σε όλα αυτά τα 100+ τεύχη (το + αναφέρεται σε κάποια τεύχη που ήταν διπλά λόγω πληθώρας ενδιαφέρουσας ύλης την συγκεκριμένη χρονική περίοδο) παρουσιάστηκαν άρθρα συμμετοχής συναδέλφων σε διεθνή συνέδρια, άρθρα από διεθνή περιοδικά με ιδιαίτερο ενδιαφέρον εφαρμογών της γεωτεχνικής μηχανικής, αφιερώματα σε φράγματα, σήραγγες και σε συγκεκριμένους μεγάλους σεισμούς, αφιερώματα σε δύο εκ των ιδρυτικών μελών της τότε Ελληνικής Επιστημονικής Εταιρείας Εδαφομηχανικής και Θεμελιώσεων Νίκο Αμβράζη και Θεοδόση Τάσιο, βραβεύσεις ελλήνων γεωτεχνικών μηχανικών, ενημερώσεις για προσεχείς ελληνικές και διεθνείς γεωτεχνικές εκδηλώσεις, ενημερώσεις για τις δραστηριότητες ελληνικών και διεθνών γεωτεχνικών ενώσεων, παρουσιάσεις εκδόσεων γεωτεχνικού ενδιαφέροντος κ.ά.

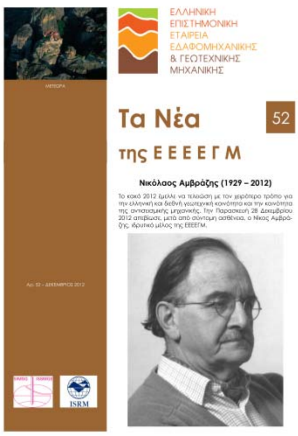
Έχουμε την εντύπωση ότι η ελληνική, αλλά και η διεθνής γεωτεχνική κοινότητα, έχει εκφρασθή πολύ θετικά για το περιοδικό και αυτό μας δίνει την δύναμη για να συνεχίσουμε την προσπάθειά μας για την καλύτερη δυνατή ενημέρωση των μελών και των φίλων της ΕΕΕΕΓΜ.

Ελπίζουμε και στην συμπαράσταση των συναδέλφων για τον εμπλουτισμό της ύλης του περιοδικού. Να τα κάνουμε 200!









Π Ε Ρ Ι Ε Χ Ο Μ Ε Ν Α

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, στα ελληνικά, του ονόματος του πρώτου συγγραφέα).

Comparison of settlement calculation methods for the design of a gravity base foundation in deep water

Comparaison des méthodes de calcul de règlement pour la conception d'une fondation gravitaire en eau profonde

P.S. Smith, N. Hytiris and S. Mickovski

ABSTRACT The vast majority of offshore wind farms today are supported by mono-pile foundations. Until fairly recently mono-piles have been un-challenged as the primary foundation type, but this is now changing. With offshore wind farms moving into deeper water, monopiles are becoming a less suitable option, not only economically but also in terms of function. This paper focuses on concrete gravity base foundations as an alternative to mono-piles for future offshore wind developments. More specifically the focus is on the geotechnical aspects of concrete gravity base foundation design, particularly the calculation of foundation settlement using both traditional analytical and numerical modelling methods. A design case study is performed using site-specific data for a future Scottish Territorial Waters offshore wind farm site. This consisted primarily of geotechnical stability and settlement calculations. The findings indicated that there was a great compliance between traditional analytical methods and numerical modelling approaches.

1 INTRODUCTION

Offshore wind farm developments have now reached the point where reliance on previous knowledge from the oil and gas sector for support structure design cannot continue. Piles utilized for offshore wind purposes have a far greater diameter than the average oil and gas driven pile, meaning that there is little design guidance available and casting doubt over the usual design procedures (Byrne 2011). With sites moving into deeper waters there has been a push to produce a more effective and financially viable foundation solution to the mono-pile. This is due to the fact that monopile foundations are not only increasingly expensive to fabricate and install in deeper waters, but are also less effective in terms of structural stiffness and pose problems when it comes to the decommission stage (Westgate and DeJong, 2005). Concrete Gravity Base Foundations (GBF) have emerged as a potential candidate to replace mono-piles in future developments due to greater stiffness at depth and the possibility of savings in production/installation. Unlike mono-piles, GBF sit directly on the seabed and rely on self-weight/ballast material and footing width to provide stability. The fact that this foundation type performs differently under lateral loading demonstrates the importance of understanding its effectiveness under realistic site conditions.

The settlement of an offshore wind turbine GBF is an important issue; the turbine must remain in a serviceable position during its lifetime. Any differential settlement or tilt that may occur can have an adverse effect on the serviceability of the turbine and the ability of maintenance crews to gain access. If settlement is predicted accurately then it is possible to adjust the foundation transition piece, thus ensuring the turbine tower remains within the serviceability limits (DNV, 2014). Offshore structures are subject to immediate settlements from cyclic loading and changes in loading patterns over time, and long-term settlements relating to primary and secondary consolidation (Dean, 2010).

This study presents work investigating various methods of settlement calculation, from traditional hand calculation methods to the use of finite element software packages such as PLAXIS 2D. These calculations were carried out utilizing site-specific data and soil properties derived from in situ marine ground investigation.

2 DESIGN METHODOLOGY

The design calculations were carried out in accordance with relevant standards (DNV 2014) for a foundation bearing on the seabed surface. The depth below the seabead-foundation interface analysed in this study was taken as the bed rock level at 80m, in accordance with the requirement of a depth equal to two times the foundation width B to be analysed. The calculations were split into three key areas; (i) immediate settlements calculated via traditional analytical methods; (ii) consolidation settlement and rate of consolidation; and (iii) settlement analysis by means of numerical modelling techniques (computer software). The design input parameters such as the wave loading conditions, soil stratigraphy/design parameters at the site and foundation structural properties were provided by the client and are detailed in the forthcoming sections. The results of the calculations allowed the various foundation settlement scenarios to be analyzed comparatively.

2.1 Site description

The site is a Round 3 deep-water development, in the Scottish Territorial Waters with water depths ranging between 40 m and 60 m at the lowest astronomical tide (LAT). The development envisaged building of offshore wind farm for a number of 6 MW turbines with a hub height of ca. 86 m above LAT. The client required investigation into potential designs for a gravity base foundation that would support the turbine under 50-year return wave loading conditions.

At the request of the client, their identity and the location for the works will remain confidential.

2.2 Soil stratigraphy and key design parameters

The details of the soil strata and associated design parameters were derived from the results of offshore Cone Penetration Testing carried out at the site and are shown in Table 1. The sandy clay had a moisture content of 22%, with PL=16 and LL=25.

It should be noted that the strength of all sand layers was conservatively assumed as = 35° due to lack of available test data as it is on the lower end of values for drained dense sand.

2.3 Design loading applied by 1-in-50-year wave

The 50-year return period wave and current design loads (DNV, 2014) were provided as:

- Maximum horizontal load) H_{max} = 38 MN (inclusive of a 1.5 MN wind load)
- Overturning moment M = 450 MNm
- Maximum vertical load V = -10 MN

Table 1. Soil stratigraphy and adopted design parameters. Cone end resistance (q_c), effective unit weight (γ'), relative density (D_r), water content (ω), liquid limit (LL) and plastic limit (PL)

Soil Description	Depth, m	γ' kN/m^3	q_c MPa	D_r %
Loose to medium dense sand	0.0-1.0	9.5	10.6	40
Medium dense to very dense silty sand (1)	1.0-7.0	9.5	10.6	61
Soft to hard sandy clay	7.0-12.2	9.5	5.7	
Medium dense to dense silty sand (2)	12.2-16.3		16.5	52
(2)	16.3-30.5	9.5	19.6	52
(1)	30.5-34.9		33.4	63
Very dense silty sand	34.9-80		40	
Bed rock	80			

2.4 Foundation structural details

The foundation geometry parameters considered in this case study were based on a GBF design concept from the literature (GRAVITAS, 2012; Fig.1), with the following parameters based on 45 m water depth:

- Foundation air gap when submerged: 16 m
- Turbine hub height above LAT: 86 m
- Base diameter (foundation width) D=40 m
- Outer diameter, top of shaft: 6 m
- Net concrete volume: circa 3000 m3
- Solid ballast total: < 30,000 tons
- Submerged weight: V_s = 209 MN

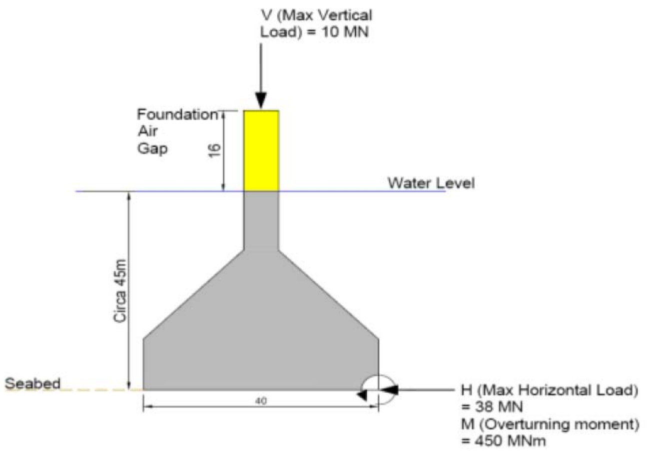


Figure 1. Gravity base foundation section with design wave loads

3 RESULTS

3.1 Effective foundation bearing area and pressure calculations

In order for the foundation settlement calculations to be carried out, the effective foundation bearing area and the pressure exerted to the seabed, as a result of the foundation self-weight and wave loading, were calculated in accordance with relevant standards (DNV, 2014). This effective foundation area is expressed as a rectangular area that is derived from an elliptical shape with a center at the foundation eccentricity (Fig.2). The total vertical load applied to the foundation is assumed to be applied eccentrically at a specific distance (e) from the center of the footing.

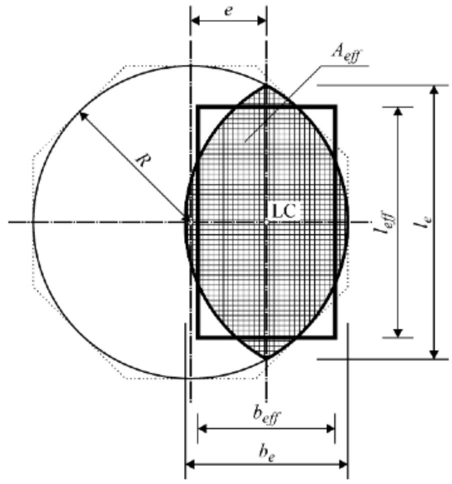


Figure 2. Circular footing with effective elliptical bearing area (DNV, 2014)

3.1.1 Effective foundation area

In order to calculate the effective foundation dimensions the eccentricity (e) of the foundation vertical load (V) was calculated as:

$$e = M_d/V_d = 2.05 \text{ m} \tag{1}$$

where M_d =M from Section 2.3 and V_d = V+V_s (Section 2.3 and 2.4).

The effective elliptical area was then calculated as:

$$A_{eff} = 2\ [R^2 \arccos(e/R) - e\sqrt{(R^2 - e^2)}] = 1092.8\text{m}^2 \tag{2}$$

The length and width of the equivalent effective rectangular foundation area were calculated as:

$$l_{eff} = \sqrt{A_{eff}\frac{l_e}{b_e}} = 34.8 \text{ m} \tag{3}$$

$$b_{eff} = \frac{l_{eff}}{l_e} b_e = 31.4 \text{ m} \tag{4}$$

where le and be are the major elliptical axes associated with the effective foundation area.

3.1.2 Foundation bearing pressures

The details from the previous section were then used to calculate the average, max. and min. bearing pressures. The average pressure across the foundation effective area was calculated as:

$$q_{average} = V_{max}/(l_{eff}\ .\ b_{eff}) = 200.4 \text{ kN/m2}$$

For eccentrically loaded foundations the bearing pressure is not uniformly distributed, resulting in trapezoidal stress distribution. In this case where e < B/6, the maximum and minimum bearing pressures were defined as:

$$q_{max/min} = \frac{P}{A} \pm \frac{M(\frac{B}{2})}{I} = 246\ /\ 102.7 \text{ kN/m}^2 \tag{5}$$

where P is the vertical loading, A is the foundation area, M is the overturning moment, B is the foundation width and I is the foundation moment of inertia.

3.2 Immediate settlement calculated employing traditional analytical methods

3.2.1 Schmertmann's method (S)

The immediate foundation settlement was calculated using a CPT-based method (Schmertmann *et al.*, 1978) due to available data as:

$$S_i = C_1\, C_2\, \Delta_p\, \Sigma_0^{2B}\, (I_z / E_d)\, \Delta_z = 101 \text{ mm} \qquad (6)$$

where C_1 is a depth correction factor taking into account overburden pressure and C_2 is a long term creep factor (both taken as equal to 1 for this study), Δ_p is the net increase of load at the foundation level, B is the width of the loaded foundation area, I_z is the strain influence factor for the center of each layer (Fig. 2) and varies with depth, E_d is the deformation modulus (taken as $2.5q_c$ for axisymmetric footings) varying with depth, and Δ_z is the thickness of each soil layer.

3.2.2 De Beer and Marten's method (DM)

DM method (1975) was also utilized to calculate the total immediate settlement using the available CPT results. The settlement was calculated as:

$$S_i = (H / C_s)\, \ln\, [(p_{o2} + \Delta\sigma_z)/p_{o2}] = 133 \text{ mm} \qquad (7)$$

where H is the stratum thickness, C_s is the is the constant of compressibility, taken as (Meyerhof, 1956):

$$C_s = 1.9\, (C_r / p_{o1}) \qquad (8)$$

where C_r is the static cone resistance and p_{o1} is the effective overburden pressure at depth tested. Additionally, p_{o2} is the effective overburden pressure at the center of the soil stratum prior to application of the foundation loading and $\Delta\sigma_z$ is the vertical stress increase at the center of the soil stratum. It should be noted that the Steinbrenner method was used to assess the vertical stress increase at the center of each of the soil strata. The equation was broken down and calculated for each stratum to take into account variation in stress and constant of compressibility with depth, respectively.

3.2.3 Meyerhof average immediate settlement (M)

Meyerhof (1974) proposed a quick estimate method for the immediate settlement of a footing on sand. As defined by the following equation:

$$S_i = \frac{\Delta p B}{2\overline{C_r}} = 205 \text{ mm} \qquad (9)$$

where B is the least dimension of the footing and $\overline{C_r}$ is the average value of static cone resistance over a depth of B below the footing.

3.3 Consolidation settlement

The consolidation settlement of the sandy clay (Table 1) has been estimated using Schmertmann's (1978) method as:

$$S_c = H \frac{C_c}{1+e_o} \log\left(\frac{P_2}{P_1}\right) = 131 \text{ mm} \qquad (10)$$

where H is the clay stratum thickness (5.2m), C_c is the soil compression index. e_o is the final void ratio, P_1 is the effective overburden pressure at the center of the clay stratum, P_2 is the increase in pressure at the center of the clay stratum due to foundation loading. Due to a lack of oedometer test results, the compression index was taken as:

$$C_c \cong 0.009\, (LL - 10\%) \qquad (11)$$

and the final void ratio was taken as:

$$e_o = \omega\, G_s \qquad (12)$$

where the particle specific gravity of clay has been taken as G_s = 2.68.

3.4 Numerical modeling for settlement prediction

Two commercial software analysis packages were then used to perform the same calculations, these were PLAXIS 2D (PLAXIS, 2012) and OASYS pdisp (OASYS, n.d.). Both of these packages required the input of Young's modulus E, amongst other parameters, which in absence of measured values, was assumed as $2.5q_c$ (Schmertmann *et al.*,1978).

3.4.1 Overall settlement (PLAXIS 2D) (P)

PLAXIS input programme was used to set up the model geometry and assignment of model parameters. The uniformly distributed load of 200.4 kN/m^2 (Section 3.1.2) was set at 40 m width to represent the foundation geometry. The sand layers where modeled using the Mohr-Coulomb failure criterion assuming drained condition upon loading; the clay layer on the other hand was input using the 'Soft Soil' method and assumed to be in an undrained condition to take into account the effect of pore water pressure and consolidation settlement. Fig.4 shows the deformed finite element mesh representing the soil strata with the max. settlement value calculated S = 173mm**.**

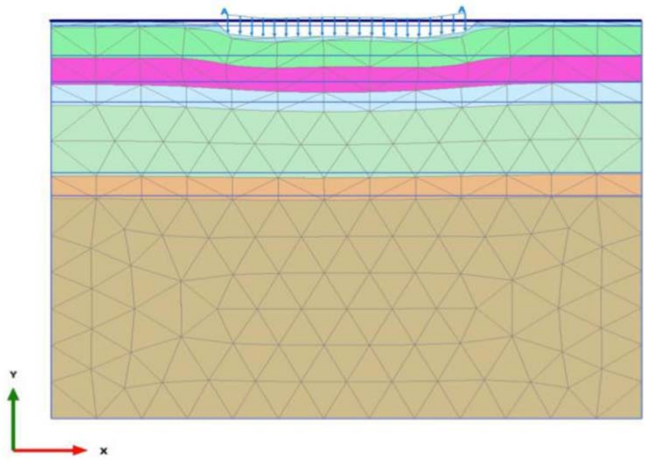


Figure 4. Deformed finite element mesh (uniform loading). Deformation width = foundation width (40m)

The uniform loading calculation was deemed to be inaccurate in its portrayal of the loading conditions as the eccentricity of the foundation loading would result in trapezoidal pressure distribution and thus, differential settlement. Using the maximum and minimum values of foundation pressure (Section 3.1.2) produced a maximum settlement under one edge of the foundation of S_{max}=197 mm and a minimum value at the opposite edge of S_{min}= 62 mm (differential settlement ΔS = 135 mm; Fig.5). This produces an off-vertical tilt of the foundation of 0.2**°**, thus satisfying the criteria adopted by Piere *et al* (2009) of 0.25° at design and 0.75° installation tolerance, giving a total design tolerance of 1°.

3.4.2 OASYS pdisp calculation (O)

The OASYS software did not require as extensive a user input as PLAXIS and was limited in that it only considers immediate elastic settlement and does not take into account the effect of the phreatic level and pore water pressures, particularly in the clay layer. To define the soil strata the software required the values of E and Poisson's ratio v with each entry. The rigid boundary level was taken as 80 m depth in agreement with the requirement for a depth equal to 2B below the foundation to be analyzed. The immediate settlement value calculated was S = 135 mm (Fig. 6) where the vertical displacement with depth is shown.

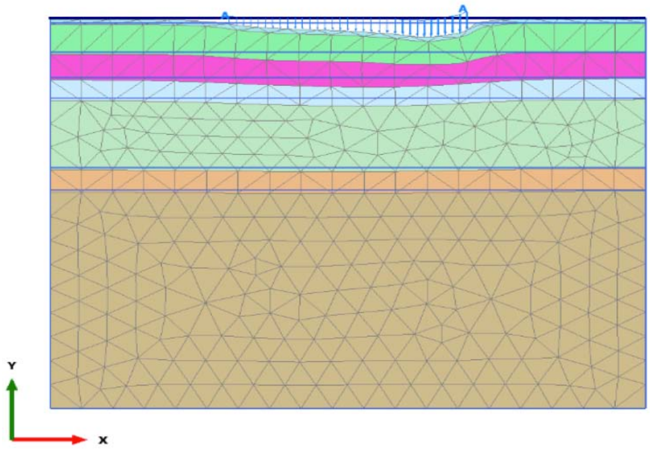


Figure 5. Deformed finite element mesh (trapezoidal loading). Deformation width = foundation width (40m)

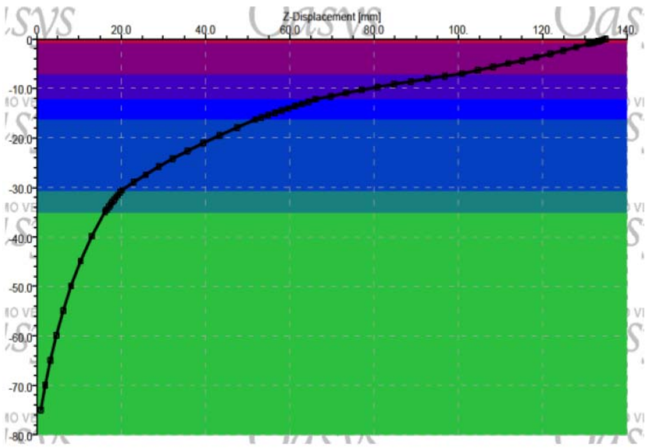


Figure 6. Displacement (mm) with depth below the footing

3.4.3 Overall settlement predictions

The summary of the settlement predictions is shown on Table 2.

Table 2. Summary of the settlement predictions

Settlement Method	S	DM	M	O	P
Immediate [mm]	101	133	205	135	173
Consolidation [mm]	131	131	131	131	131
Differential [mm]	-	-	-	-	135
Total (mm)	232	264	336	266	304

4 DISCUSSION AND CONCLUSIONS

The results showed clear compliance between traditional analytical and numerical modeling settlement calculation methods, with the numerical methods producing slightly higher values than the traditional calculations. This suggests that the settlement problem in traditional soil mechanics can be applied to offshore wind turbines with confidence and the calculations can be refined and improved by the use of numerical modeling software.

Based on the results of this study it is recommended that the S method is favoured prediction method when combined with a numerical modeling tool. This study has shown the benefits of PLAXIS in predicting the settlement of a shallow footing and has the potential to provide highly realistic models of a given site, provided the input data is accurate.

The availability of in-situ testing data (CPT in this case) was of utmost importance for accurate analysis and design avoiding the need for assumptions resulting in over/under estimates. For future designs it is recommended that at least similar in situ testing is carried out prior to the design, perhaps focusing on direct measurement of angle of shear (e.g. shear vane), rather than assuming it, which would optimize the design. S and O / P methods also required the input of the elastic modulus (E_d) of the soil strata which was assumed. Future work should include investigation of the effects of this parameter on settlement for offshore structures, enhancement of methods for its direct measurement, or more realistic approximation (Terzaghi et al., 1996).

The accuracy and reliability of the assumed parameters are the main issues in the design for serviceability (Das and Sivakugan (2007). More detailed and relevant in-situ and laboratory testing as well as improving the quality and accuracy of the existing methods can contribute towards optimization of the design. The reliability (likelihood that measured settlement < predicted), can be improved by accurately measuring actual structure settlements for which new methods and monitoring plans should be developed to cater for the offshore environment (Byrne, 2011).

The calculated values of total settlement in this study would increase due to processes, such as scour and cyclic loading, if appropriate measures are not taken. There is a dearth of design guidance on the effect of cyclic loading on the settlement, and our future research will focus on this aspect.

Additionally, the use of foundation skirts on the foundation base, which penetrate the seabed upon installation and prevent undermining/loss of bearing area whilst also transferring foundation loading to a stronger stratum, has not been investigated in the past and is another area of focus for our future research.

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The design of an anchored pile retaining structure to control landslide regression at Lyme Regis, UK

Conception et calcul d'une structure de retenue ancrée en pieux foré afin de contrôler l'évolution d'un glissement de terrain à Lyme Regis, au Royaume-Uni

C. Candian, A. Goodwin and D. G. Daskalopoulos

ABSTRACT The town of Lyme Regis lies on the coast of southern England within a World Heritage site. Part of the town and the coastal land immediately to its east lie within a major landslide complex. Stabilising works were required to protect land to the east of the town, but stabilisation of the landslide complex as a whole is not practicable due to its scale and environmental constraints. The design solution was a single row of 900mm diameter bored piles at 1.8m spacing with a concrete capping beam supported by pre-stressed ground anchorages embedded into stable strata further uphill. This paper outlines the development of a ground and failure model for the site based on a robust understanding of the geology and failure mechanisms, and the estimation for design purposes of a likely ground model at the end of a 60 year design life. It discusses the analytical approaches used to design the wall and presents selected output to illustrate the effect of global movement patterns on behaviour

1 INTRODUCTION

Lyme Regis and its environs lie on the Jurassic Coast of southern England within a World Heritage site that is also within one of the most unstable and actively eroding stretches of coastline in the UK. The town itself is built on pre-existing and largely active coastal landslip systems that are causing damage to properties and infrastructure through progressive movements with periodic rapid failures.

Previous studies had recognised that stabilisation of the landslide complex as a whole was not practicable, especially given the environmental constraints. It was recognised though that a scheme to protect an area to the east of the town was essential. After extensive consultations High Point Rendel developed a reference scheme with a 60 year design life that included a new seawall, a soil nailed slope, a pile retaining structure, and drainage measures.

A companion paper deals with the design of the soil nailed slope, whilst this paper primarily addresses the appraisal and design of an anchored piled wall. It focuses firstly on the development of the design ground model during which landsliding would continue, and secondly on the optioneering and analysis of the final design solution, 900mm diameter anchored bored piles at 1.8m spacing.

2 GEOLOGICAL SETTING AND CURRENT GROUND CONDITIONS

The site lies to the east of Lyme Regis as shown in Figure 1, with the ground rising gently inland towards Timber Hill. The regional geological succession comprises Cretaceous Upper Greensand deposits unconformably overlying Jurassic Charmouth Mudstone and Blue Lias Formations. Local to the site, the succession comprises Shales with Beef, a basal member of the Charmouth Mudstone overlying Blue Lias, with Upper Greensand deposits inland.

The Shales with Beef comprise thinly interbedded organic-rich mudstones and calcareous mudstones. It contains numerous thin, laterally persistent beds of fibrous calcite ('beef') and beds of tabular and nodular limestone including the Table Ledge and Fish Bed marker beds. Previous studies had revealed that, other than a stable spur to the east, the upper part of the Shales with Beef had been subject to periglacial disturbance to within 1m to 2m of the Table Ledge, at a typical depth of 6m to 8m in the vicinity of the retaining wall. The disturbed deposits were typically firm to stiff clays with localised areas of soft very soft clays at or very near surface.

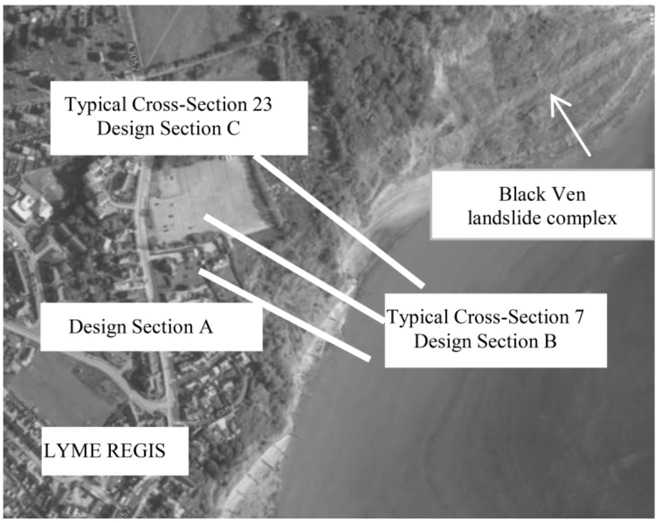

Figure 1. Site Location

The Blue Lias Formation comprises 26m to 38m of thinly interbedded limestones and calcareous, locally laminated, mudstone or siltstone that typically weathers to stiff clays that grade to weak mudstone. The strong fine grained blue-grey limestone beds are typically 0.1m to 0.3m thick and break into hard tabular blocks. The top of the Blue Lias is marked by Grey Ledge, an almost continuous tabular, dense muddy limestone.

Onshore the dip of the Blue Lias strata is about 4º towards the south-east, reducing to around 1.5º beneath the foreshore. The unconformity at the base of the Cretaceous formations dips westwards at around 3°. The strata are cut by minor faults around which local changes in magnitude and direction of strata dip are apparent. There are also more major faults in the area but these do not directly affect the site.

3 LANDSLIDING APPRAISAL

Understanding the instability of the coastal region was an important aspect of the wall design. The complex regional landslip mechanisms have been studied and investigated for many years (e.g. Gallois, 2009) but essentially comprise of the following:

- Local discontinuity controlled block slides and falls of the Blue Lias cliffs
- Translational and compound slides in the Shales with Beef circa 2m above Table Ledge in the periglacially disturbed materials and circa 2m above Fish Bed in undisturbed strata
- Reactivated translational mudslides inland of the retaining wall within the Shales with Beef
- Following movements, the scarp features formed in the Shales with Beef degrade through local mudflows of the strata, especially after heavy rain.
- Studies revealed also that a stable spur exists to the east of the site; this marks a break between the slips close to Lyme Regis and the extensive Black Ven/ Spittles Landslip complex to the east.

Based on previous reports and published work the site was zoned into two areas, namely the East Cliff Main Slip Area and the Allotments Area. The former area is moving significantly despite the toe being protected from sea erosion by the sea wall. The major movements were assessed to comprise shallow rotational and planar slides within the periglacial zone which are heavily influenced by surface and ground water. Deep seated planar slides along the limestone beds were shown by back analysis to not be continuous features, though sufficient ground straining had occurred to reduce the strength to below peak values.

In contrast, the slope and cliff in the Allotments Area appear to be relatively stable at present other than the effects of sea erosion and its inducement of local cliff face instability. The Allotments Area is not protected by a sea wall and historically is known to regress inland at circa 0.5m per year.

4 DESIGN GROUND CONDITIONS

The retaining wall was not designed to prevent instability, rather it is to provide a break line to prevent further inland scarp regression. For design it was critical therefore to estimate the likely profile at the end of the 60 year design life.

In the East Cliff Main Slip Area ongoing movements were expected to continue in front of the wall due to rainfall infiltration and groundwater, even though the rate of movement would likely reduce compared to present due to the introduction of drainage upslope of the wall. The slope formed would be uneven, but an average angle of 11.5º degrees was considered worst case and equivalent to the shallowest parts of the current slope profile.

In the Allotments Area the current stability was considered unlikely to be maintained long term. It was envisaged that new movement trends to the south west were likely in response to the continued instability in the East Cliff Main Slip Area. As the Allotments Area had not been subjected to periglacial slips a worst credible slope angle of 15º in the strata above Table Ledge was envisaged. The steep coastal cliff was expected to regress similar to present.

Based on this understanding of future movement patterns and the ground investigation data, design cross sections were prepared for the end of design life case, typical forms of which are shown in Figures 2a and 2b. Figure 2c shows also the lateral variation in the level of the Blue Lias bedrock along the line of the retaining wall, which for design purposes was reduced by 1m to allow for local variations. In the event, construction records proved the predicted levels to be 500mm or more above this design level.

Groundwater levels were taken to be at or very near surface except for local drawdown close to where drainage measures were to be installed.

5 DESIGN OF ANCHORED PILED RETAINING WALL

Three main cross sections were considered for the anchored piled wall design, namely:

- Western part of the East Cliff Main Slip (Section A)
- Central (and Eastern) part of the East Cliff Main Slip, where previous 'Urgent Advance Works' in 2003/4 had left two rows of piles tied together with steel bars (Section B)
- Eastern, within the Allotments Area (Section C)

5.1 Pile Geometry

The anchored wall design comprises a single row of 900mm diameter anchored piles at 1.8m centres throughout, with a continuous capping beam. The anchors are multi-stage, 4 strand, in Sections A and B, and these were extended 5m on plan into the western end of the Allotments Area as a transition. All piles were 23m long in the East Cliff Main Slip. To the east in Section C the anchors are 3 strand, with the piles incrementally increasing in length from 24m to 27m. All anchors are inclined 30 degrees to the horizontal and installed with a 150kN pre-stress per strand.

Retaining heights considered for design were determined based on the estimated ground profiles at the end of the design life. The values selected were 5.6m (Section A), 5.5m (Section B), and 6.8m (Section C). The wall design had to work around the existing piling in the Central Section. However, no account was taken of the capacity of the existing piles due to the limited embedment of the piles and concerns over their long term performance.

5.2 Geotechnical Design Parameters

The following tables show the characteristic material properties, which were derived based on site specific ground investigation data. These values were further verified early in the design process through limit equilibrium SLOPE/W analyses of the existing slopes and assessment of the factors of safety against slips occurring both along pre-defined potential shear surfaces and through the 'intact' mass. The characteristic values were then factored in accordance with EC7 to determine design values.

It should be noted also that the stiffness values were estimated based on both site specific data and published literature. Account was taken too of the strain dependency of the values, with preliminary PLAXIS analyses showing predicted ground strains of less than 1%.

Table 1. Characteristic Strength Parameters

Formation	Bulk Unit Weight (kN/m^3)	Peak Parameters		Residual Parameters	
		φ' (°)	c' (kPa)	φ' (°)	c' (kPa)
Disturbed Lias /Active Slip Mass	19	20	0	11.5	0
Shales with Beef	19	25	5	11.5	0
Blue Lias (highly weathered)	20	25	12.5	-	-
Blue Lias (weathered)	20	30	25	-	-
Blue Lias (unweathered)	20	35	85	-	-

Table 2. Characteristic Stiffness Parameters

Formation	Undrained Stiffness Values (MPa)		Drained Stiffness Values (MPa)	
	Design	Lower Bound	Design	Lower Bound
Disturbed Lias /Active Slip Mass	20	10	11.5	0
Shales with Beef	40	20	11.5	0
Blue Lias (highly weathered)	40	-	30	-
Blue Lias (weathered)	75	-	50	-
Blue Lias (unweathered)	150	-	112.5	-

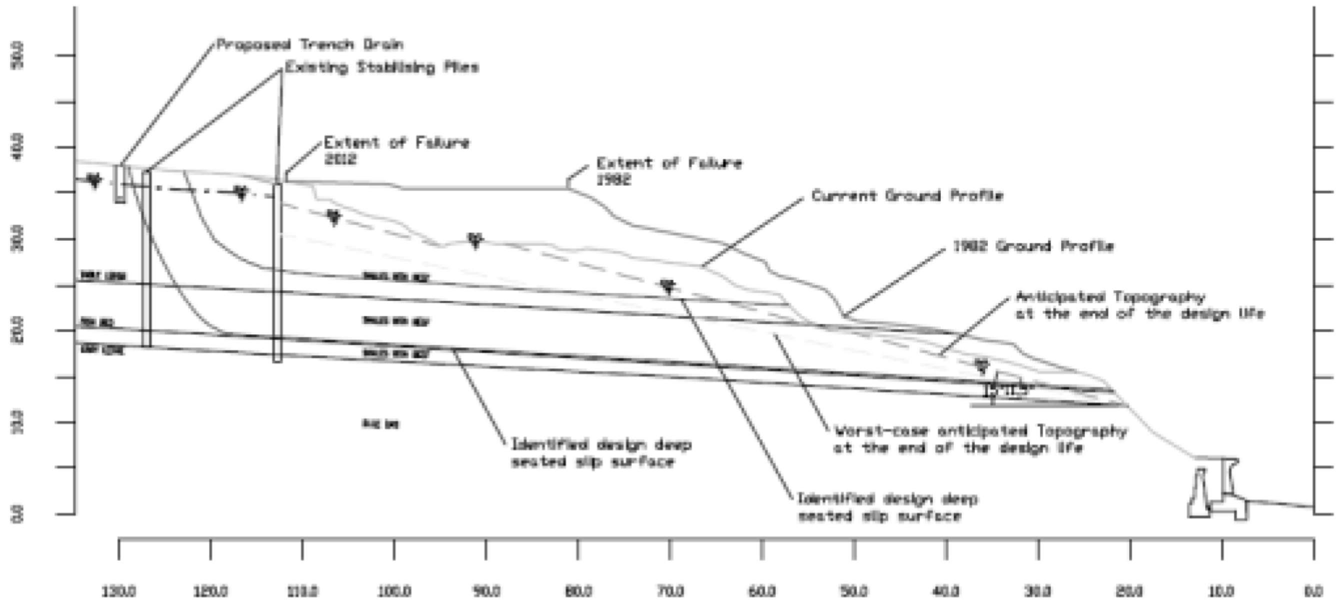

Figure 2a. Typical Cross Section 7 – Central (and Eastern) Part of the East Cliff Main Slip

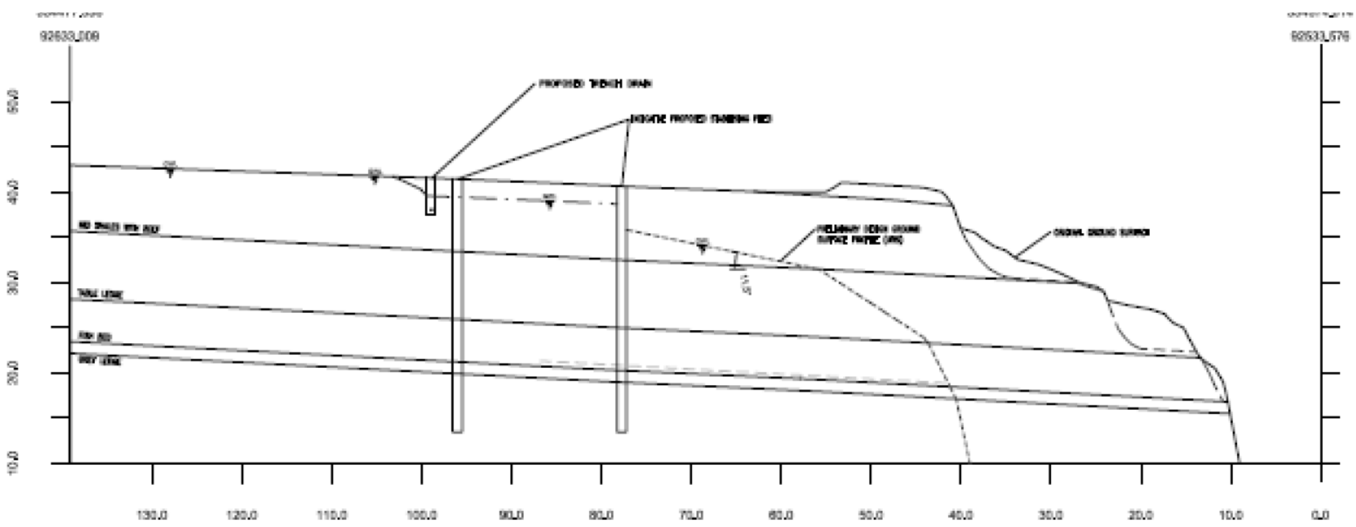

Figure 2b. Typical Cross Section 23 – Allotments Area

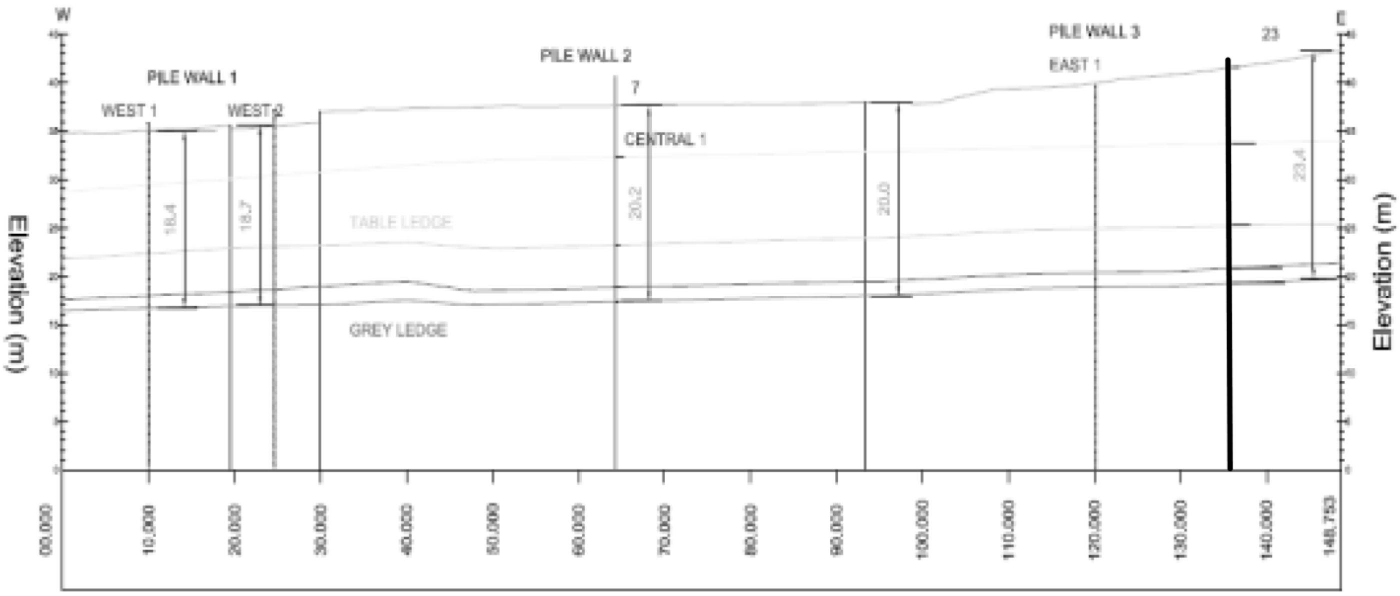

Figure 2c. Lateral Variation in the level of the Blue Lias bedrock along the retaining wall

5.3 Design Approach

The design was undertaken using a combination of approaches:

- PLAXIS analyses (2D) were used to assess the effect of the ongoing instability of the coastal slopes and model soil - structure interactions; in particular they allowed pile wall movements and ground movements close to and remote from the wall to be assessed, and ensure global instabilities could not be induced through local wall movements
- WALLAP analyses were used as a 'rapid modelling' tool to supplement PLAXIS and allow EC7 compliant limit states to be assessed; as the models were unable to model the overall slope movements due to software limitations, 'modelling correction factors' were required to be applied to the WALLAP results to calibrate them to the PLAXIS results

5.3.1 PLAXIS Analyses

The 2D PLAXIS analyses represented the SLS limit state, and as such unfactored parameters were used for the design appraisal. Other key aspects of the modelling approach were:

- Mohr-Coulomb, linear elastic model (i.e. no strain softening) with constant stiffness appropriate to the strain levels
- No tension was allowed in the ground
- The construction stages were modelled sequentially, including degradation of the slope long term
- Excess pore pressures were allowed to dissipate between each construction stage as appropriate

An additional feature of the design was the decision as part of the risk management process to verify that the design solution would also work using lower bound stiffness parameters, as sensitivity analyses had showed these to be influential on the predicted wall performance. Utilisation levels of unity or less were targeted in this very cautious state, implicitly providing an additional safety factor on the design compared with EC7.

For detailed design, analyses were undertaken for the three critical sections (A, B & C) defined above. Major observations were as follows:

- The predicted horizontal displacements in Sections A and C were similar, with downslope movements concentrated above the Blue Lias and occurring in a zone local to the wall without any global effect (see Figure 3)
- Relative pile movements followed the expected pattern for an embedded pile, but the global downslope movement pattern resulted in an absolute toe movement of between about 5mm and 10mm
- In Section C the patterns of movement were significantly smaller than the remainder of the wall, and even after regression of the cliff face was modelled movements were small and again largely confined to the zone above the Blue Lias (see Figure 5).

5.3.2 WALLAP Analyses

WALLAP analyses were undertaken using SLS, DA1-C1 and DA1-C2 limit states, with partial factors applied in accordance with EC7. Key observations regarding the basis of the modelling are:

- the hydrostatic ground water conditions used were an approximation of the variable ground water profile within the overall slope
- the construction sequence was modelled to replicate the expected conditions, similar to the PLAXIS modelling.

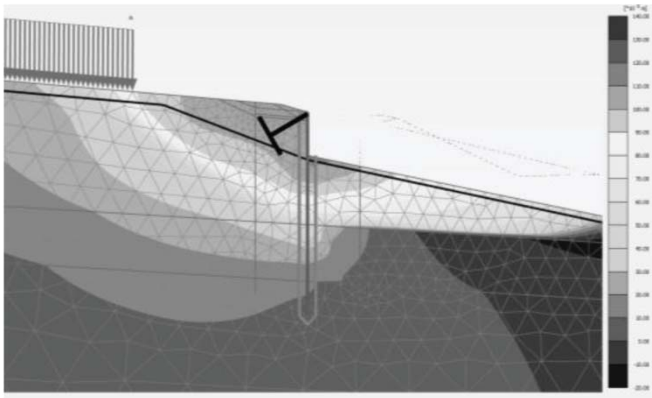

Figure 3. Horizontal movements in Section B

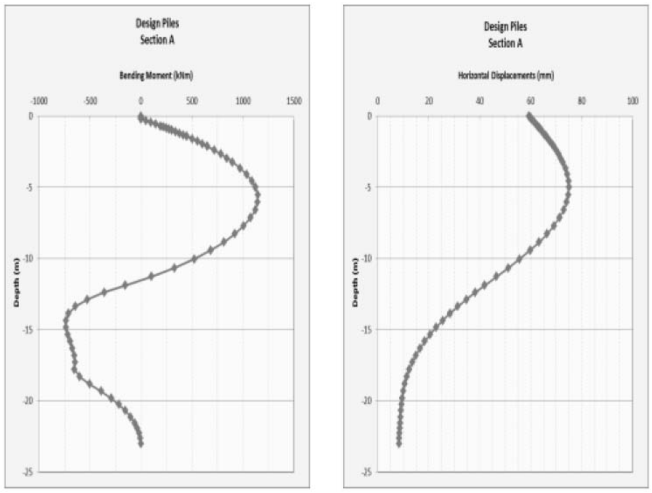

Figure 4. Pile bending moments and deflections in Section A

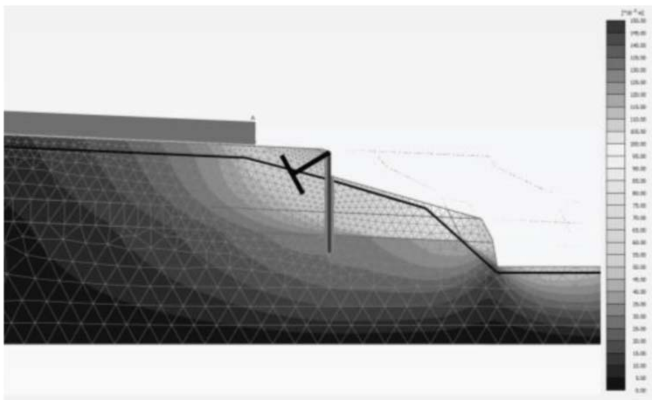

Figure 5. Total movements in Section C

The results of the analyses indicated that Sections A and C were the worst case, and these were taken to be representative of the East Cliff Main Slip area (Western and Central sections) and Allotments area (Eastern section) respectively.

The form of movement and bending moments predicted by the analyses were similar to those derived from PLAXIS, but smaller in magnitude. As PLAXIS modelling was substantially slower than WALLAP, the approach adopted was to define the design envelopes using the WALLAP output and to factor these up using the modelling correction factors discussed in Section 5.3. Final PLAXIS analyses were then undertaken to verify the factored envelopes were reasonable and cautious.

Table 3 summarises the correction factors derived using this method. The factors of up to 1.75 indicate clearly the effect of the global slope movement patterns on the piles and the significant under-estimation of the required structural capacities of the piles if a conventional WALLAP based approach had been used alone.

Table 3. Summary of Modelling Correction Factors

Attribute	Correction Factor
Bending Moment	1.75
Shear Force	1.75
Anchor Force	1.25
Horizontal Movement	1.50

Table 4 summarises the design values used to determine the structural design requirements for the anchors and piles.

Table 4. Summary of pile design values

Section	Design Bending Moment (kNm/pile)	Design Shear Force (kN/pile)	Design Anchor Force (kNm/pile)	Maximum Horizontal Movement (mm)
A	2284	811	545	143
C	1602	904	391	114

6 CONCLUSION

The design of the piled retaining structure within an area of ongoing instability presented significant and unusual challenges:

- The establishment of a robust ground model for design is usual, but in this case it was dependent first on the development of an understanding of the existing ground and ongoing failure modes, and secondly the estimation of the likely ground conditions at the end of the 60 year design life based on a holistic understanding of the site.
- Having established the design ground model, it was recognised that consideration of the wall alone would underestimate its required capacity as global movements were likely to significantly complicate the soil-structure interactions; as a result both PLAXIS and WALLAP analyses were undertaken and confirmed this effect.
- PLAXIS analyses of complex conditions are relatively much more time-consuming and a design protocol was implemented that allowed the high productivity associated with WALLAP modelling to be utilised even in these complex ground conditions through the use of ‘modelling correction factors’ in combination with PLAXIS verification.

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Feasibility study of random fibre reinforced railway ballast using image-based deformation measurements

Étude de faisabilité du ballast ferroviaire aux fibres aléatoires renforcées utilisant des mesures de déformation basées sur images

O. Ajayi, L. Le Pen, A. Zervos and W. Powrie

ABSTRACT The need to develop new means of optimizing the performance and minimizing maintenance requirements of railway ballast has become more pressing. This is due to the growing need for the development of resilient track materials (including ballast) capable of withstanding increasing train speed, load and frequency. In recent times, railway ballast has been reinforced mostly by the use of geogrids placed at strategic depths within the ballast layer. However, experience has shown that the presence of geogrids restricts the type of maintenance activity (tamping) that can be carried out on the reinforced track section. The addition of random fibre reinforcements in ballast has been proposed as an alternative reinforcement technique that can meet the reinforcement requirements of ballast without limiting future maintenance activities. Using novel digital-image based deformation measurement technique for triaxial tests, the deformational evolution of random fibre reinforced scaled ballast was investigated. The mechanism of reinforcement in the reinforced granular material is highlighted and the potential applications in railway ballast discussed.

1 INTRODUCTION

The ballast supporting a traditional railway track has attracted significant research interest in recent times, but there is considerable potential for optimizing its performance and minimizing maintenance requirements. The use of randomly orientated fibre reinforcements in sand has been shown by previous research to improve its mechanical properties. Similar benefits can be expected to be observed in ballast (which is also a granular material) if an in-depth scientific understanding of the mechanics involved is achieved.

The use of randomly orientated fibres to reinforce soil has been investigated by a number of researchers (e.g. Michalowski and Cermak, 2002; Lirer et al., 2011; Diambra et al., 2013). The mechanical behaviour of the mixture may be influenced by fibre properties, soil characteristics and reinforced soil characteristics. There is also a general consensus among researchers that the micromechanics of the fibre/particle interaction significantly influence its mechanical properties (e.g. Michalowski and Zhao, 1996; Diambra et al., 2013). These fibre/particle interaction mechanisms include the relative fibre/particle dimensions, fibre content etc. However, no attempt has been made to study the deformation of fibre reinforced granular materials at both the full field and local micro-scale level.

This paper reports the use of an image-based deformation measurement method developed by Bhandari et al. (2012) to study the local deformation of fibre reinforced triaxial specimens.

2 EXPERIMENTAL WORK

2.1 Materials

The granular material used in the experiments reported in this paper follows a grading parallel to that of standard Network Rail (NR) ballast at 1/5 scale (Figure 1). This offers an attractive and economical means of developing an understanding of the mechanics of full size ballast (D_{100} = 62 mm). Le Pen et al. (2013) demonstrated that although there is a measurable variation of form and roundness with particle size over a range of sieve intervals, these differences are slight and do not militate against the use of scaled material in investigating the factors influencing macro-mechanical behaviour.

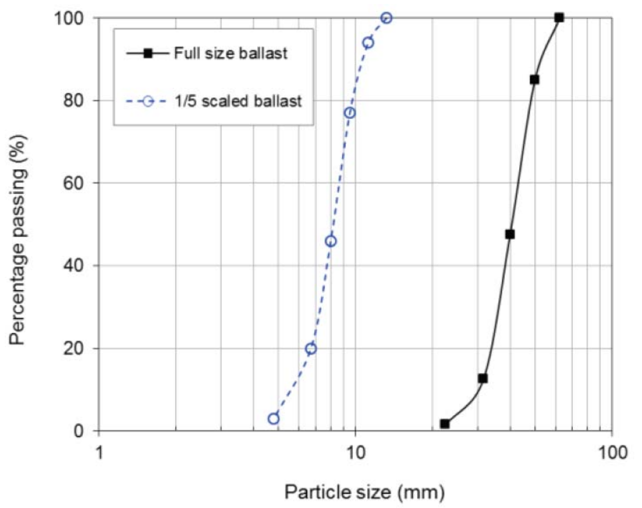

Figure 1. Particle size distribution of 1/5 scaled, and Network Rail standard ballast

The dimensions and the typical mechanical properties of the fibres used are presented in Table 1.

Table 1. Typical values of the basic properties of polyethylene fibres

	Polyethylene
Fibre length	58 mm
Fibre width	20 mm
Fibre thickness	0.5 mm
Specific gravity	0.92
Tensile strength	20.3 MPa[1], 11.2 MPa[2]
Softening temperature	85°C

[1] Longitudinal; [2] Transverse

2.2 Laboratory Tests

The scaled ballast-fibre mixture used in the triaxial tests was prepared by mixing known masses of fibres and scaled ballast in a plastic container. The resulting mixture was random and homogeneous. The procedure for determining the maximum and minimum density of fibre reinforced granular are described in Ajayi et al. (2014).

2.2.1 Triaxial tests

Conventional monotonic triaxial tests on specimens 150 mm in diameter and 300 mm in height were carried out on fibre reinforced and unreinforced scaled ballast specimens. The fibre-reinforced scaled ballast mixture was prepared to achieve the densest state corresponding to the minimum void ratio at a given fibre content.

The triaxial tests were carried out on dry specimens at a confining stress of 30 kPa to replicate the typically low confining stresses within ballast as reported in the literature (e.g. Indraratna et al., 2010; Sevi and Ge, 2012). The triaxial tests and the initial conditions of the specimen are summarised in Table 2.

The *void ratio*, e is defined as the ratio of the volume of voids (V_v) to the volume of grains (V_s). The *Volumetric fibre ratio*, V_{fr} is defined as the ratio of the volume of fibres (V_f) to the volume of solids.

Table 2. Triaxial tests carried out

Specimen	e_o	Fibre content, V_{fr} (%)
1/5 SB	0.76	-
V_{fr} = 6.5%	0.98	6.5

2.2.2 Image-based deformation measurement technique

A digital image-based deformation measurement system for triaxial tests as described in Bhandari et al. (2012) was utilized to study the local deformation of the specimens. The triaxial setup consisted of two digital cameras placed along the radii of the transparent triaxial cell at an interval of about 120° when viewed in plan.

In the image analysis, a resolution of 0.01 pixel was chosen for the sub-pixel interpolation, with a precision better than 0.0033 pixel for a 181 × 181 pixel subset based on the empirical equation given by White et al. (2003) is. The accuracy was found to be better than 0.10 pixels or 0.004 mm in the objectspace at a nominal image scale of 0.04 mm/pixel at the centre of image (Bhandari et al., 2012).

3 DEFORMATION CHARACTERISTICS OF FIBRE REINFORCED TRIAXIAL SPECIMEN

The addition of fibres produces an increase in the peak stress ratio, *q/p′*, of the mixture while suppressing its dilation (Figure 2). The effect of fibre reinforcements on the dilatancy (as defined in Eq. 1) of the specimen is further highlighted in Figure 3. Figure 3 indicates that the reinforced specimen is able to mobilise a higher mobilised strength at a given rate of dilation than the unreinforced specimen.

$$\delta = \delta\varepsilon_{vol} / \delta\varepsilon_q \qquad (1)$$

The results of the image analysis of the triaxial specimens during the pre-peak and post-peak deformation stages of the shearing process (as seen in Figure 2) are shown in Figures 4 and 5 respectively. The horizontal axis of each plot represents the circumferential distance of the specimen and the vertical axis represents the specimen height, both in millimetres. A reference vector is shown below the displacement vector field (Figure 4(a) and 5(a)) and the colour bar displayed to the right of the strain fields (i.e. Figure 4(b) and 5(b)) represents the corresponding strain in percentage. For comparison, the limits of the colour bar for each plot were kept constant.

During pre-peak deformation, a more nearly vertical deformation of the reinforced specimen was observed compared with the unreinforced specimen (Figure 4(a)). The unreinforced specimen exhibited higher shear strain over the prepeak deformation stage than the reinforced specimen (Figure 4(b)).

During post-peak deformation, the reinforced specimen exhibited more nearly vertical deformation than the unreinforced specimen (Figure 5(a)). A more homogeneous distribution of shear strains is also observed in the reinforced specimen.

4 DISCUSSION

At large strains, fibres in granular materials are believed to be undergoing significant stretching thereby mobilising their tensile forces. This in turn creates an apparent confinement of the granular matrix (Diambra et al., 2013) and thus a reduced shear strain distribution.

This suggests that fibre reinforcements in railway ballast can potentially reduce shear deformation (evident through reduced lateral spread) while increasing the mobilised strength of the composite. In addition, the more uniform distribution of strains as observed in the reinforced scaled ballast specimen can potentially lead to a re-distribution of stresses (vertical and horizontal) within the reinforced ballast layer.

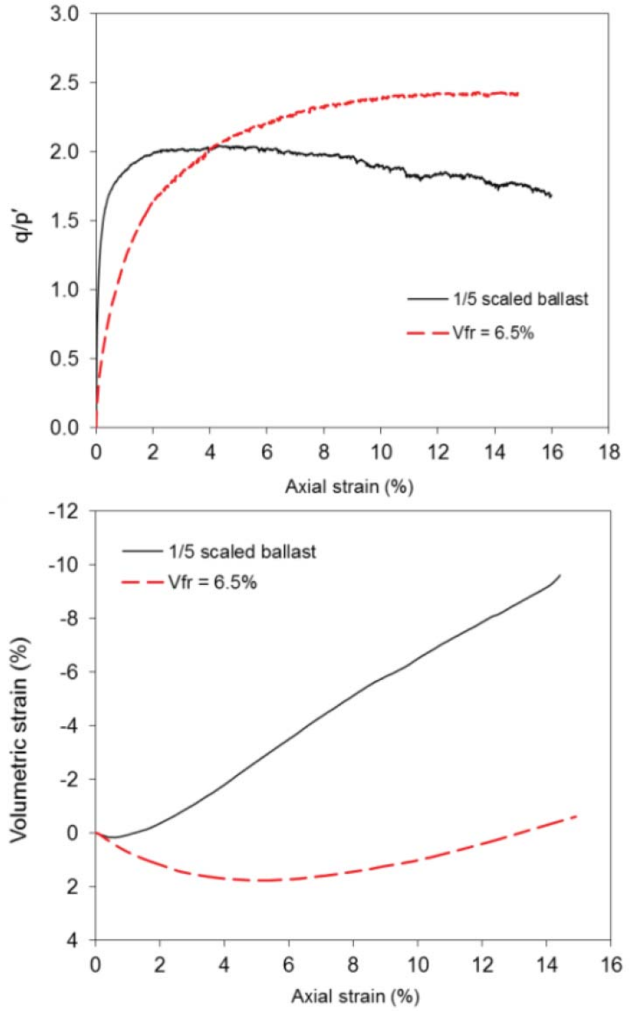

Figure 2. Effects of fibre reinforcements on the development of stress ratio, *q/p* and volumetric strain with axial strain for scaled ballast

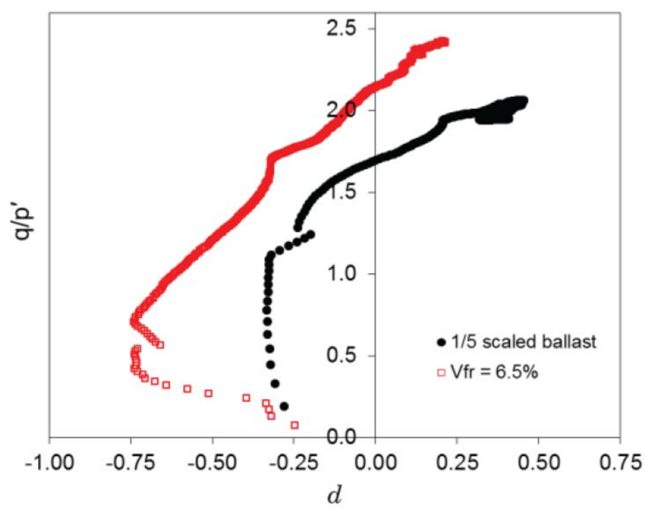

Figure 3. Effects of fibre reinforcements on the rate of dilation, *d*

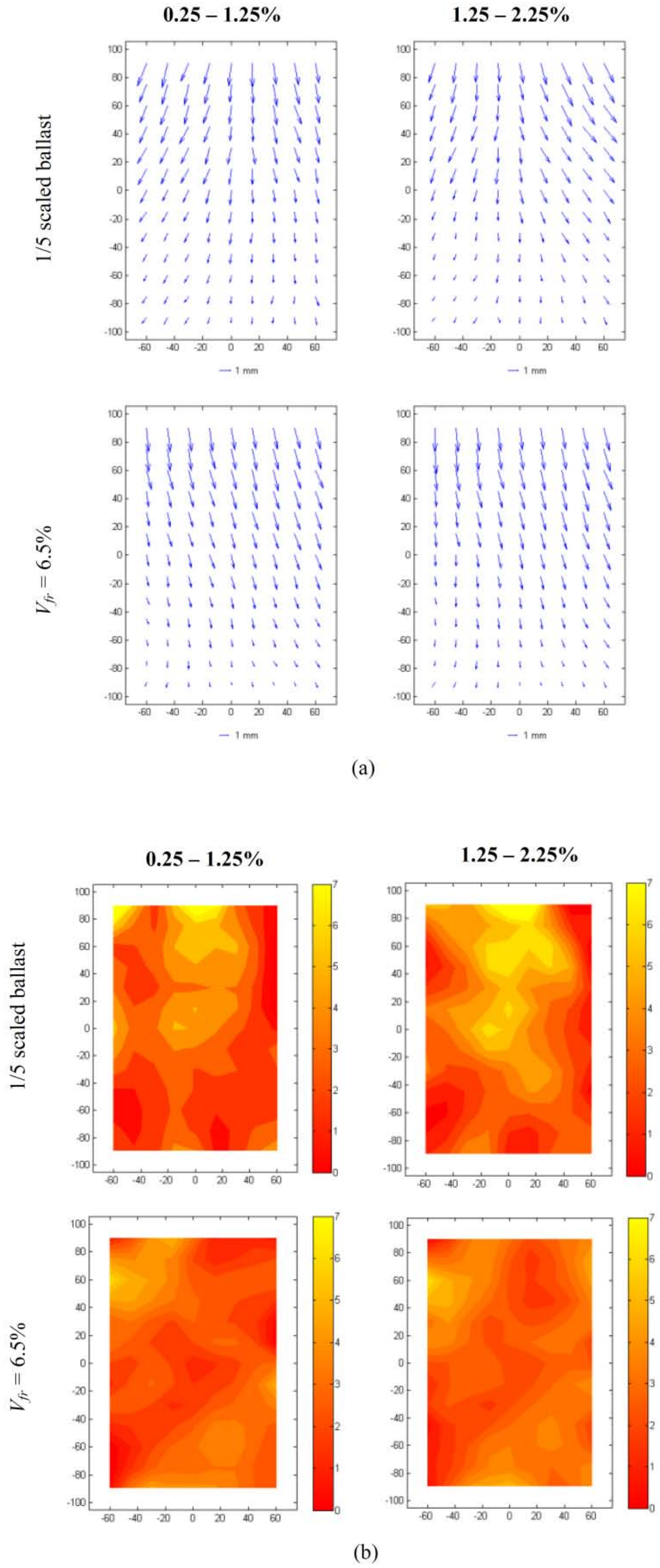

Figure 4. Pre-peak deformation characteristics of unreinforced and reinforced specimens (a) displacement vector (b) maximum shear strain fields

5 CONCLUSIONS

An image-based deformation measurement technique was used to investigate the deformation characteristics of unreinforced and reinforced 1/5 scaled ballast specimens. Fibre reinforcements in scaled ballast produced an increase in the mobilised strength of the mixture while suppressing dilation. At large strains, the addition of fibres to scaled ballast produced a more uniform distribution of shear strains owing to the apparent confinement provided by the tensile resistance of the fibres.

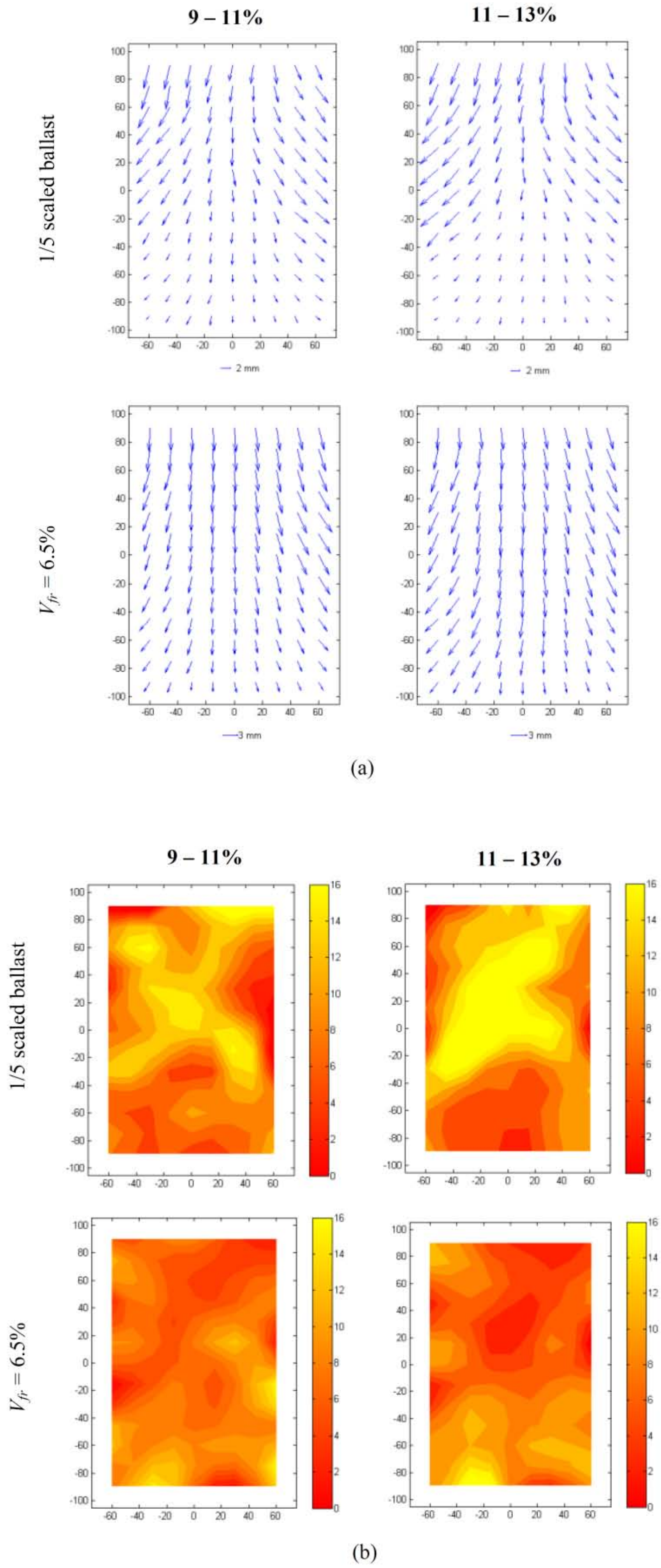

Figure 5. Post-peak deformation characteristics of unreinforced and reinforced specimens (a) displacement vector (b) maximum shear strain fields

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Applied rock engineering for the investigation and restoration of mine workings, case example – Dudley Limestone mines

Ingéniere appliquée rock pour les enquêtes et la restuaration des chantiers miniers, affaire exemple – mines de calcaire caire de Dudley

R. A. MacKean, J. C Sharp, S. C. Bandis, E. A Bacasis, R Morgan

ABSTRACT Wren's Nest Mine was excavated during the 18th, 19th and 20th Centuries, extracting a dipping sequence of Silurian Limestone, primarily for the making of lime. The room and pillar mines were finally abandoned in a precarious state following systematic 'robbing' of the pillars. Significant collapse occurred in 2011 and with further falls and ongoing concern for public safety the final famous upper (daylight) gallery was completely backfilled with granular material in 2004. The site forms an important part of the local heritage and was declared a National Nature Reserve in 1957. In 2006 Dudley Metropolitan Borough Council and other stakeholders wished to remove the granular backfill from the upper gallery and re-establish public access, as a key part of an ambitious, large scale underground facility for the area. The paper summarises the geological and rock engineering studies undertaken to establish design concepts for long-term stabilisation of the underground complex.

1 INTRODUCTION

The Wren's Nest Mine is a part of an extensive area of mining that extends through several levels and depths below Dudley. The limestone on the west side of Wren's Nest dips at 30° to 45°, on the limb of an anticline, and the upper gallery is unusual in that it is open to the surface with daylight entering between the remaining pillars. A plan of the mine is shown on Figure 1 and a view of the uppermost 'daylight' gallery is provided as Figure 2. The pillars shown on the right-hand side of Figure 2 are named the Seven Sisters, of which just five remain, as shown on Figure 1. Details of the internal geometry of the mine workings are shown along a section, Figure 3, the location of the section is marked on Figure 1.

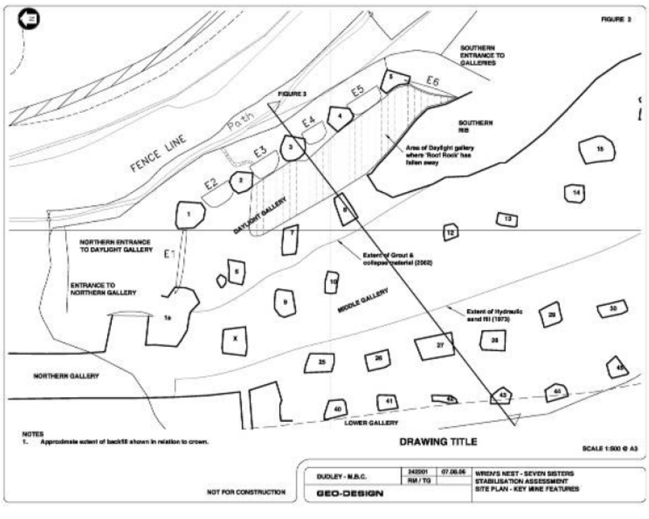

Figure 1. Plan layout of mine showing pillar geometry.

Due to concerns over public safety, the lower levels of the mine have been progressively backfilled. Public access to the viewing platform and the area above the daylight gallery was stopped in 1977.

Significant collapses occurred in 2001 and as a result the third and second galleries were backfilled with grout. Due to further rock falls and ongoing concern over public safety, the upper (daylight) gallery was completely backfilled with granular material in 2004. Granular material was used in the hope that this section of the mine might be reopened following assessment and installation of stabilisation measures that might be required to secure the long term stability of the openings.

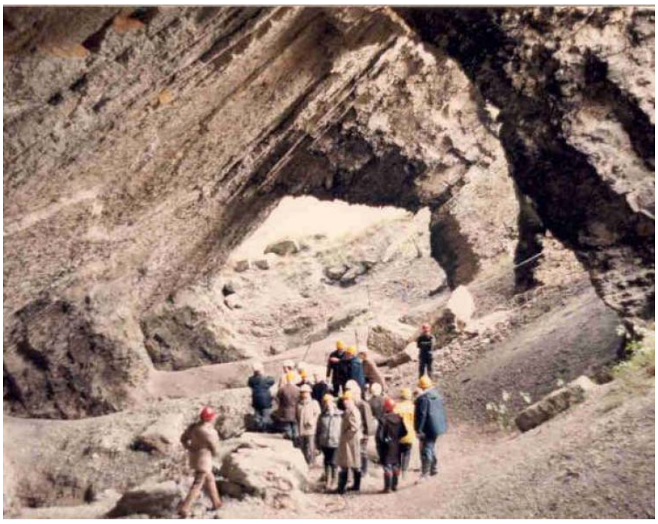

Figure 2. View of Daylight Gallery prior to backfilling

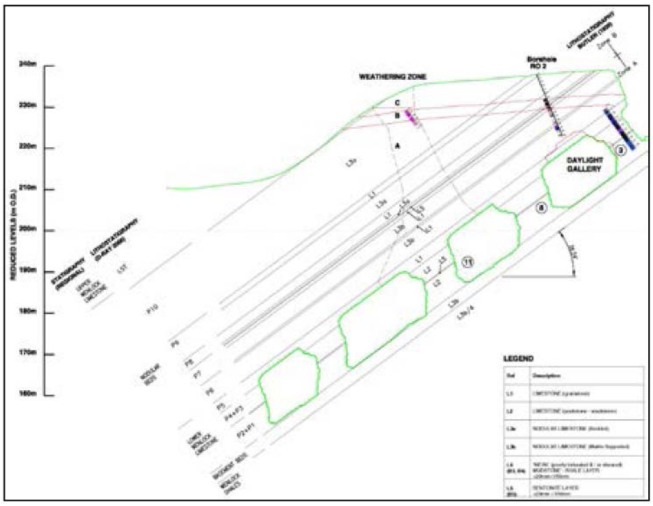

Figure 3. Cross section through mine workings based on 3D laser scan survey undertaken prior to the backfilling in 2004.

This paper outlines the engineering geological basis of rock engineering studies for stability evaluation and review of remediation concepts.

2 INVESTIGATION AND GEOLOGICAL MODEL DEVELOPMENT

2.1 Boreholes and Geophysical Logging

Three rotary cored holes were undertaken during July 2006, as follows :

Table 1. Project Boreholes

Hole Number	Depth to Upper Gallery crown (m)	Inclination degrees from vertical
R01	17.3	20
R02	16.3	23
R03	17.8	14

The boreholes were positioned above the upper gallery, inclined towards the east so as to be approaching perpendicular to bedding.

All holes intersected the granular backfill where a 0.2 to 2.3 m thick void was encountered between the crown and backfill. An additional hole was bored using the 'sonic' drilling method. However, very poor quality core recovery was achieved and the method was abandoned.

All holes were geophysically logged by Natural Gamma, Caliper and Optical Televiewer.

Scaffolding was constructed at key locations to provide direct access to the exposed geological sequence that was logged in detail following the procedures outlined below.

2.2 Geological Setting and Lithostratigraphy

The broad stratigraphy was originally defined by Murchison in 1839 and based on that formative work, the Silurian System was established. The following principal units are present at the site:

Upper Wenlock Limestone/Nodular Beds/

Lower Wenlock Limestone/Basement Beds/

Wenlock Shales

The mines in this study were formed within the lower Wenlock Limestone. Exceptionally detailed unpublished work (Dr Dave Ray) provided further subdivision of the sequence that formed a framework within which to build a lithological and bedding discontinuity model (Figure 3).

The following classification of lithologies was adopted (Table 2):

Table 2. Project Lithological Classification

Ref.	Lithology
L1	LIMESTONE (packstone - grainstone)
L2	LIMESTONE (wackestone)
L3a	NODULAR LIMESTONE (Bedded)
L3b	NODULAR LIMESTONE (Matrix Supported)
L4	'WEAK' (poorly indurated and / or sheared) MUDSTONE - SHALE LAYER >20mm<150mm
L5	BENTONITE LAYER >20mm <150mm

2.3 Rock Mass Structure

The principal discontinuity set which controls rock mass behaviour, for the typical case, is the dipping bedding related structure.

Bedding discontinuities are prominent within the rock mass due to the variety of interbedded and contrasting lithologies, and also the weak, sometimes thick clay layers. The nature of the bedding discontinuities varies according to the associated lithology and a classification was developed to reflect this, shown in Table 3 (examples are shown in Figures 4 and 5).

Each bedding discontinuity 'Type' was comprehensively characterised in terms of their physical properties. Thus, discontinuities in core and outcrop needed only to be classified according to the above systems in order to produce 'Strip Logs' of Discontinuity Type, rather than requiring repetitive description of each individual discontinuity.

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Table 3. Project Bedding Discontinuity Classification

Ref.	Bedding Discontinuity Type	Associated Lithology
B1	BEDDING DISCONTINUITY LST	L1
B2	BEDDING DISCONTINUITY LST	L2
B3	'WEAK' (poorly indurated and / or sheared) MDST - SHALE LAYER >2mm<20mm	L1, L2 & L3
B4	'WEAK' (poorly indurated and / or sheared) MST - SHALE LAYER >20mm <150mm	L4
B5	BENTONITE LAYER >20mm <150mm	L5

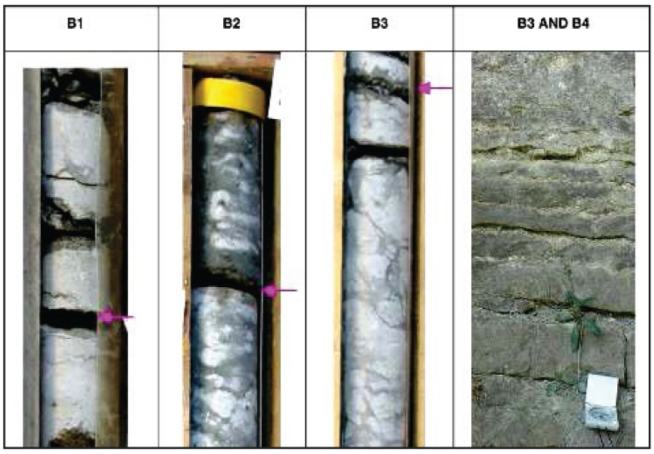

Figure 4. Bedding Types B1, B2, B3, B4 - Examples

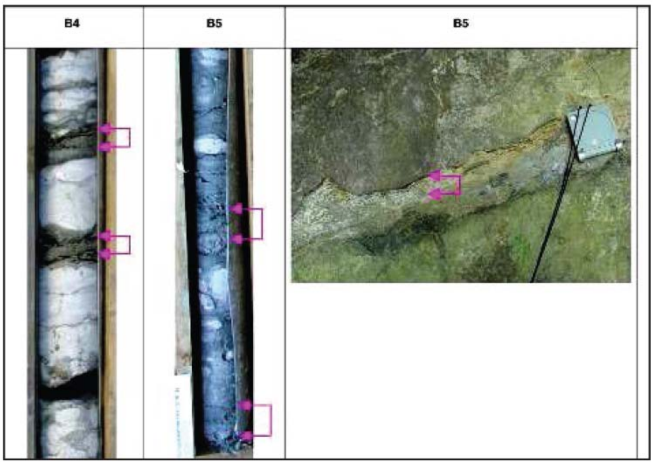

Figure 5. Bedding Types B4 and B5 - Examples

Each bedding discontinuity 'Type' was comprehensively characterised in terms of their physical properties. Thus, discontinuities in core and outcrop needed only to be classified according to the above systems in order to produce 'Strip Logs' of Discontinuity Type, rather than requiring repetitive description of each individual discontinuity.

Correlation between boreholes and outcrop strip logs, within the context of the lithostratigraphical model resulted in the semi-deterministic bedding model illustrated on Figure 6.

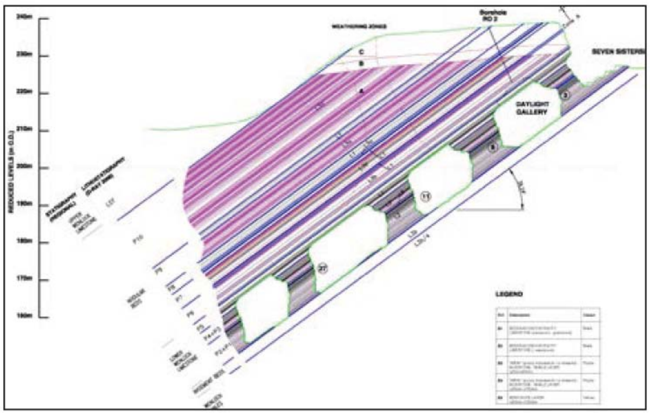

Figure 6 Bedding Discontinuity Model. Colours represent different discontinuity types.

The type and condition of jointing varies within the limestone sequence. Two subvertical orientation sets are present and trend north-south and east-west, consistent with the regional trends. The following three principal types are evident :

- Intra-bed jointing
- Throughgoing joints
- Throughgoing major joints / minor faults

The intrabed jointing is relatively subdued and not always discernable in outcrop.

Throughgoing joints are evident in outcrop and also from prior mapping. More major joints are also present, these features are stained to gallery level and have a brown clay infill. Such features have had a significant control over large scale past instability mechanisms and required careful consideration.

2.4 Weathering

The rock mass is weathered and exhibits a clearly defined weathering profile with depth. It can be summarised as follows :

- Zone C 'Highly weathered'
- Zone B 'Moderately Weathered'
- Zone A 'Slightly weathered'

In Zone A the extent of weathering is pervasive with a significant reduction in material strength. Zone B is characterised by preferential weathering associated with only mudstone / shale interbeds which are significantly weakened relative to Zone A.

A key feature of the weathering profile is that the interface between the zones is not at a consistent level in relation to the crown of the openings, with a more enhanced weathering profile apparent at the southern end compared to the northern end.

3 ROCK MASS CHARACTERISATION

Each Discontinuity Type was defined and characterised based on their physical characteristics such as persistence, roughness, aperture, degree of associated alteration, infill and wall strength.

These characteristics provided the basis for derivation of the joint roughness (JRC), wall strength (JCS) and residual friction indices of the Barton-Bandis (BB) model for shear strength and stiffness modelling.

Important behavioural features of discontinuities include non-linearity, stress, displacement and scale dependency. These aspects are allowed for in the BB model for predicting shear strength and stiffness parameters. The model is implemented as the constitutive model of UDEC-BB.

Figure 7 illustrates the BB model predicted contrasting shear behaviour of the different types of bedding discontinuities and joints as were simulated in the numerical modelling study.

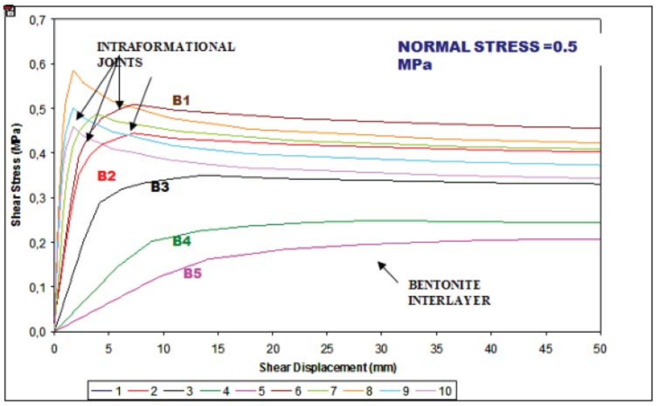

Figure 7. BB model predicted shear behaviour of the different types of discontinuities (bedding and joints) present in the limestone rock mass at the Seven Sisters Mine.

Inspection of Figure 7 shows the contrasting stress strain behaviour of the weak and compliant B5 layer as compared with the stiff but brittle behaviour of the limestone-limestone B1 discontinuities.

4 INTRINSIC STABILITY ASSESSMENT

The mines were 'designed' (worked) as conventional room and pillar openings, albeit strongly influenced by the dipping sequence. In general, the mine was intended to exploit the limestone located beneath a generally more competent limestone unit (the Cap Rock') whilst using the inherent stability provided by that unit to maximise the as-constructed spans. The openings were thus excavated as unsupported openings, relying solely on the intrinsic stability of the rock mass in the crown and pillars.

As noted in the preceding section, the majority of the openings now illustrate varying degrees of instability and the mine as a whole has deteriorated significantly with time and would have continued to do so had it not been backfilled.

In order to assess the degree of stabilisation that may be required for a given excavation configuration (size, shape), it is necessary to first understand the intrinsic (inherent) stability provided by the rock mass. As a first stage assessment, this can be best evaluated in terms of a two dimensional tunnel cross section.

Whilst some factors have a clear influence on stability of room and pillar openings, such as extraction ratio (i.e. the ratio of pillar area to area of the openings), several other factors are equally significant. Of the many factors considered three were identified as having an overriding influence on stability, namely:

- extent of cap rock that remains in place;
- location of weak discontinuities in relation to the openings;
- depth of weathering in relation to the openings.

5 NUMERICAL SIMULATION AND ANALYSIS

The "Distinct Element Method" (DEM) provides the most appropriate approach and representative simulation for analysis of discontinuous rock mass behaviour. The DEM in rock engineering is implemented by the state-of-art software UDEC and 3DEC, principally developed by Dr Peter Cundall [1980].

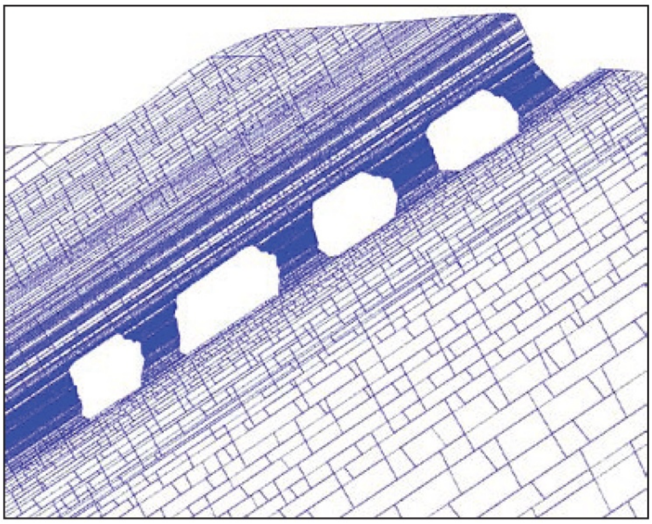

Figure 8. UDEC-BB Model (showing discontinuities only)

Each discontinuity depicted in the semi deterministic geological model (Figure 6) is explicitly represented in the numerical model in terms of location (relative to the openings and geological sequence as per Figure 8) and physical properties via the BB failure criterion.

This approach of combining project specific classifications with DEM allows the explicit representation of discontinuities (and lithology) that control and dictate behaviour of the rock mass. Input parameters were allocated for the full range of materials and discontinuity types including as follows:

Range of Limestone types: UCS=15-45 MPa, Tensile strength = 1.5-5 MPa, c=1.5-5.0 MPa, φ=32°-42°, E=8000-22500MPa, v=0.1-0.2

Bedding discontinuities: the τ-u relations were predicted from the BB model for JRC=5-12, JCS=5-45 MPa, φ_r=18°-35°. Typical shear strength and stiffness parameters of bedding discontinuities were:

Peak friction angle (φ_{peak}) = 22° – 50°

Peak shear stiffness (K_{ss}) = 5 – 125 MPa/m.

6 ENGINEERING ASSESSMENT AND STABILISATION CONCEPTS

The use of UDEC provided detailed simulation of the rock mass as well as representation of a multistaged mining sequence.

Engineering evaluation of the results assisted with identification of the key mechanisms of instability. Shearing along bedding resulted in delamination of the bedded rock mass (Figure 9) and bending under gravity of the separated limestone beds (Figure 10). This resulted in development of particular loading conditions within the crown beam and the supporting pillars. The analysis was used to assess the likely height of delamination and overstressed rockmass above the crown in order to derive a basis for selecting the length of rock reinforcement required. Local joint controlled instability in the pillars (Figure 9) was also highlighted.

Crown deformations resulting from these mechanisms varied depending on depth and extent of weathering. Deformations reached equilibrium, albeit with the crown beam in an overstressed condition, in all but the uppermost gallery. In the more weathered southern openings greater deformation was noted and the model did not reach equilibrium, inferring collapse was likely in the unsupported state.

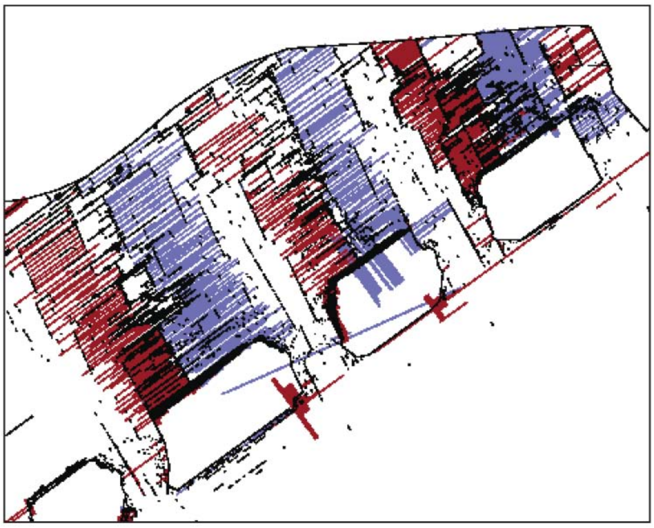

Figure 9. UDEC model output showing Shear displacements (right lateral in blue, left lateral in red and joint opening in black). At 5m above crown of Daylight Gallery total displacement approximately 100mm and shear displacements 10mm.

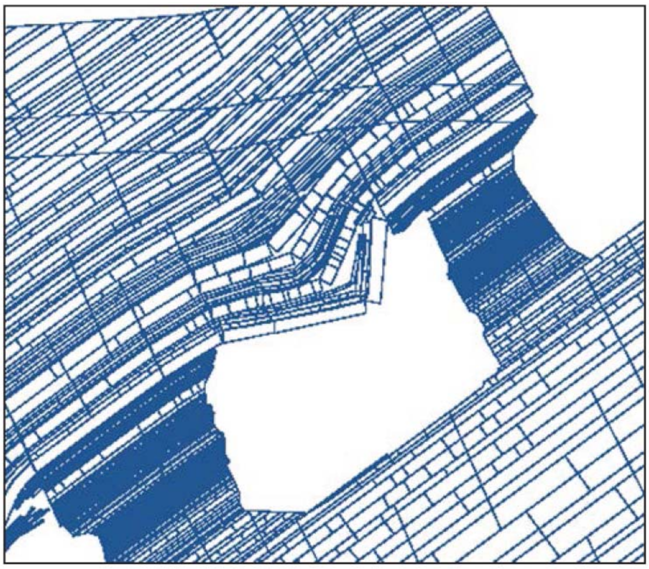

Figure 10. Intrinsic Stability Assessment – Plot showing deformation (x10) in the unstabilised case (equilibrium not reached).

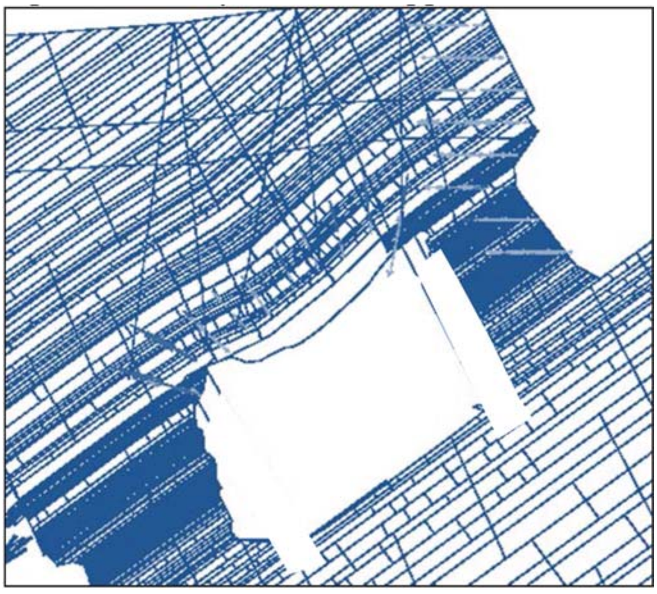

Figure 11. Plot showing the stabilised case (with bolts from surface and from within gallery) deformation (x10)

A series of engineering measures were designed and implemented in the UDEC models to enhance the existing unsupported stability state. The measures comprised pillar strengthening from surface, localised use of bolts and shotcrete and a cast in situ concrete lining for the southernmost gallery. Figure 11 shows the stabilising effect of rock reinforcement installed from surface and from within applied to the crown and haunch area.

An architect's impression of the completed works that would have allowed public access is shown in Figure 12. Having narrowly missed winning the People Millions Lottery in 2008 the scheme awaits an alternative source of funding.

Figure 12. Architects impression of the completed works

ACKNOWLEDGEMENT

The support and vision provided by the staff of Dudley Metropolitan Borough Council are acknowledged together with their huge efforts in attempting to secure lottery funding for implementation of these works. Graham Warton of The Black Country Museum is also gratefully acknowledged.

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Convergence in the Geosciences

Marcia K. McNutt

Abstract

The 21st century presents unprecedented challenges if society is to continue to provide abundant energy, water, and food, and high quality housing and medical care to a growing global population. Deforestation and aquifer depletion are at unsustainable rates, and use of fossil fuels is leading to unprecedented climate change. Geoscientists can confront these challenges by expanding partnerships with other disciplines. "Convergence," the integration of engineering, physical sciences, computation, and life sciences to benefit health, energy, and the environment, has been successfully developed for biomedical research. It is time for the geosciences to embrace convergence, as our future depends upon it.

Suppose on your annual visit to your doctor's office, your doctor diagnosed you with some dreadful, life-threatening disease, and then sent you home to die. You would probably be very disappointed with that doctor and perhaps with the entire medical establishment. In fact, the reason why the National Institutes of Health has enjoyed such broad, bipartisan support for its budget is not just because the biomedical research community has advanced the diagnosis of ailments substantially (which it has), but also because it has delivered cures for those ailments, or at least the hope of a longer, more productive life.

By analogy, our planet is the patient of the geoscientists, and for the most part the diagnoses have only become increasingly dire as to the many afflictions that jeopardize the quality of life of humans on Earth. No wonder many have grown weary of bad news. Consequently, the geosciences have been under siege, singled out for special congressional oversight and targeted for budget cuts. Consider instead the very different situation if in addition to providing the diagnoses, the geoscientists also put forth practical, affordable solutions that society readily embraces. For this to happen, geoscientists should take a page out the biomedical playbook and embrace convergence.

Convergence has its roots in the early part of the 21st century in a proposal that collaboration between nanotechnology, biotechnology, information technology, and cognitive sciences would help improve human performance [*Roco and Bainbridge*, 2002]. This model was embraced and implemented in the biomedical sciences with the goal of making health care more affordable. Biomedical researchers had their basic paradigms in place to guide them for how the major systems in the body work and the tools to explore those systems. But they needed to provide more affordable solutions to a host of afflictions in the face of rising health care costs. By working with engineers, physical scientists, and computational experts biomedical researchers brought multidisciplinary systems thinking and analysis to solve complex problems in creative new ways [*Lane*, 2011]. In a similar vein, geoscientists have good models for Earth's primary systems and excellent tools for probing them. What we need are the partnerships with the life science community to find holistic solutions for a living planet that benefit from advances in biotechnology, with the engineering community to apply innovative solutions to major challenges, with economists to ensure that those solutions are affordable, and with social scientists to understand the barriers to human acceptance of new directions.

Convergent approaches to problem solving go beyond interdisciplinary research. Investigators collaborating on an interdisciplinary project generally reside in disciplinary departments; whereas, convergent research is accelerated because the collaborating researchers are housed in institutes dedicated to convergence. Their students are trained in highly complex, multidisciplinary problem solving, rather than just taking a course or two beyond their major field. Special funding devoted to convergence research avoids the double or triple jeopardy when investigators from different fields must apply to their disciplinary panels for funding in order to collaborate.

Convergent-like approaches have been introduced to the geosciences, especially in confronting natural disasters. For example, the remarkable reduction in earthquake fatalities in nations such as Japan, Chile, and the United States is the result of convergent-like research partnerships between geologists, seismologists, earthquake engineers, architects, social scientists, and public officials. These partnerships have resulted in improved maps of earthquake risk areas, estimates of strong ground motion, engineering designs for earthquake resistant structures, and revised building codes compliant with those designs. The Southern California Earthquake Center has been devoted to this type of research through the National Science Foundation's Science and Technology Centers funding program.

Many other topics in the geosciences would benefit from convergent approaches. For example, consider the issue of environmental health. More than ten million people die annually from environmental factors, and the most common problems are air, water, soil pollution, and chemical exposure. Current approaches to addressing these issues are generally costly and disruptive. Cleaning badly contaminated soil involves excavation and incineration, at the cost of $1500 per ton or $26 million per Superfund site. In too many cases the responsible party is no longer in a position to fund the costs, so they fall to the taxpayer. Usually the best and cheapest approach is to prevent the contamination from happening in the first place, hence involving the input of industrial process engineers working with economists and environmental scientists. When that is not an option, other solutions are to limit exposure to vulnerable populations or improve the public health response for those who have been affected. These solutions require the collaboration of environmental toxicologists, microbiologists, hydrologists, public health professionals, and psychologists.

There is every reason to believe that engineers will be ready and willing to partner in addressing geoscience issues. The National Academy of Engineering has launched the Grand Challenges in Engineering program (http://www.engineeringchallenges.org), a commitment to prepare 20,000 new engineers with societal aspirations. Several of the current themes are relevant to the solutions that geoscientists seek: access to clean water, manage the nitrogen cycle, develop carbon sequestration methods, and make solar energy affordable. The program has expanded internationally through partnerships with the Chinese Academy of Sciences and the British Royal Society.

There have been some notable institutions already established on convergent themes. For example, the Earth Institute at Columbia University blends research in the physical and social sciences with education to find practical solutions for a more sustainable world. The institute includes experts in geoscience, law, public health, public policy, and business. The Broad Institute, founded jointly by MIT and Harvard to advance treatment of human disease, has put a higher premium on technology and collaboration with engineers. Geoscientists need to be open to partnerships with both social scientists and engineers. Convergent science is more likely to thrive if dedicated funding streams prevent collaborators from having to submit separate grant proposals, if university administrators develop policies that encourage convergent research, if promotion and tenure policies recognize these contributions, and if students are educated in complex, system-level problem solving.

If more geoscientists started working in this matter, what outcomes might ensue? At the very least, new partnerships will arise between dissimilar disciplines, and new career paths will emerge for students. We might find new industries clustered around convergence institutes in the geosciences. My hope is that geosciences will be increasingly perceived as contributing to competitiveness, resiliency, health, and quality of life, and thus widely viewed as part of the solution, not part of the problem.

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Monitoring the Victoria Station upgrade

London Underground's Victoria station is at the heart of London's transport network. Currently undergoing over £700m of capacity enhancement improvements and with the North ticket hall now open, discover how specialist contractor ITM Monitoring has closely measured every millimetre of construction movement.

London Victoria upgrade: 3d modelling and close monitoring for movement was paramount

Used by over 80 million passengers each year, Victoria Station is one of London's busiest stations. Over the previous five years main contractor Taylor Woodrow BAM Nuttall JV (TWBN) has begun to provide improvements to the station's capacity, reduce passenger journey times, provide step free access and improve emergency evacuation.

Major works include:

- A new underground north ticket hall at the junction of Bressenden Place and Victoria Street, with an entrance at street level
- Doubling in size of the existing Victoria line ticket hall (south ticket hall)
- 400m of new interchange tunnels, connecting the south and north ticket halls and the District & Circle line platforms
- Full modernisation of the existing station areas including nine new escalators and seven new lifts to provide step free access between street, ticket hall and Victoria line platform levels
- Improved access and new lifts between the National Rail and Victoria line platforms and between the Victoria line and District and Circle lines platforms

In close proximity to existing London Underground (LU) assets, listed buildings, sewers and an interminable flow of passengers, the Victoria Station upgrade posed a significant challenge to the project team and required a comprehensive movement monitoring system, delivered by ITM.

Monitoring that informs the construction process

LU's primary objective in delivering station upgrade schemes is to maximise value across the project from design to delivery. With experience gained on the Channel Tunnel rail link, Green Park step free access and numerous LU projects, ITM was well-placed to ensure LU's ambitious targets were accomplished using a value-engineered design.

The monitoring instrumentation and monitoring contract was procured and is being delivered in stages. With first-hand experience of managing NEC3 contracts, ITM complied rigorously to the construction, design and management (CDM) regulations required by the project and worked closely with both TWBN and LU.

Surrounding buildings were closely monitored using Robotic Total Stations

Stage One saw ITM contribute to the completion of detailed station design. "ITM were employed directly by London Underground to install monitoring equipment and record baseline data to establish how the local infrastructure was reacting to ambient conditions such as seasonal change. This allowed a clear understanding of movement actually induced by the subsequent station upgrade works," comments David Smith, senior construction manager for London Underground.

Substantial traditional surveying and 3D laser scanning works were undertaken of all above and below ground structures providing LU with an accurate 3D representation of the site and adjacent structures to facilitate the creation of a an integrated BIM model. Provision of the model data allowed the project team to design the works from a 3D perspective leading to clash avoidance.

With secant piles forming the walls of the north ticket hall within 3m of a Victorian culvert carrying the River Tyburn; the passenger tunnel linking the north ticket hall to the Victoria line passing under the culvert and skirting over the top of the southbound Victoria line platform tunnel; and an additional escalator squeezing between the twin bores of the Victoria line north and southbound platform tunnels with a mere 300mm clearance, having a spatially accurate and fully co-ordinated 3D model was paramount to construction success.

Passenger tunnel linked the north ticket hall to the Victoria line

Robust monitoring system needed

The early partnership between LU and ITM provided a wealth of background movement monitoring data of the third party structures and LU assets within the zone of influence of the works. As the project progressed through to construction, TWBN were able to use ITM's monitoring data to address key challenges such as tunnelling in shallow water-bearing ground in the paid area links (PALS) tunnels.

TWBN and designer Mott MacDonald utilised precision jet grouting as the ground treatment method for the PALS tunnel works. This process involved installing over 2,000 columns of jet grout into the ground, forming a wall of weak concrete which could be mined through to create the tunnels.

Close proximity to an urban environment meant the interaction between the jet grouting with existing basements, foundations, utilities and tube tunnels needed a robust monitoring system. The surrounding buildings were closely monitored using Robotic Total Stations (RTS), prisms and back-up manual monitoring.

Below ground saw the installation of precise liquid-levelling cells in a number of building basements, accurately monitoring vertical movement in near real-time during this critical stage of the project.

LU commented: "The initial installation was successfully completed including an innovative rope access approach to high level installation. The work involved installing equipment in and on existing buildings which was managed well, interfacing with stakeholders and avoiding conflict."

Instrumentation and data visualisation

Monitoring the vast assets across the project required over 20 different sensor types, installed either above or below ground, to accurately monitor all movement throughout the programme of works.

ITM's approach supplemented an optical monitoring system with the use of Basset convergence strain gauge rings to overcome a problem with optical refraction caused by temperature gradients in the Victoria line running tunnels. In addition to these, the RTS and prisms, as well as continuous electrolevel beams, monitored 3D tunnel settlement, rotation and ovalisation.

Borehole instrumentation such as biaxial horizontal in-place inclinometers (IPIs) and Multi Point Borehole Extensometers (MPBX) were fitted to monitor changes in subsurface ground movements such as settlement and heave.

Above ground, liquid levelling cells, water level loggers, vibration monitors, piezometers and inclinometers were installed, to name a few. An additional 200 prisms were installed and read by seven RTS to accurately monitor the surrounding buildings, a number of which had listed facades.

The 3D coordinate positions of the prisms were read by networked total stations hourly, with each unit linked to a data logger housing a GPRS/3G modem for remote data transmission and control.

ITM use their in-house Star*Adjust™ software to implement a precise least-squared network adjustment to correct for any movement of the RTS themselves due to construction works.

Data visualisation through Argus, now superseded by ITM's Calyx OMS software, houses all manual and automatic monitoring data generated across the site, providing a central repository for the project.

The web-based software is designed for assisting with interpretation of large amounts of instrumentation data and is accessible to the project team 24 hours a day. It provides continual monitoring of site conditions, offering status updates for each sensor and information on whether that sensor is currently in alarm.

Monitoring data better informed construction decisions saving time and labour

Delivering a world class station in the nation's capital

Using monitoring data to better inform construction decisions provides LU with a value-engineered station upgrade, TWBN with a successful project and passengers with a service throughout the construction process.

The monitoring data gave TWBN the ability to refine a number of construction designs and techniques, resulting in significant savings in time and labour.

To undertake extremely complex civil engineering work safely and without incident, despite a highly congested construction zone with live rail traffic and heavy pedestrian footfall, is no small endeavour.

The team at VSU have been commended for their efforts with the project winning the 'Tunnelling Project of the Year' at the 2014 NCE / ITA Tunnelling Awards, 'Complex Infrastructure' category at the 2016 ICE London Civil Engineering awards, as well as 'Specialist Tunnelling Project of the Year' at the 2016 NCE Tunnelling Awards.

ITM are proud to be a significant part of this project, delivering step free access to Victoria underground station for the first time in its 157 year history.

Find out more: Follow the VSU story and download the case study at: http://www.itmmonitoring.com/projects/victoria-station-upgrade/

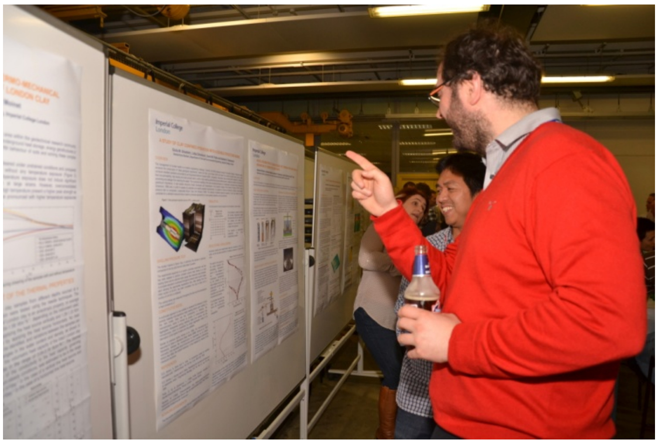
(Rhiannon Walker / Business Development Manager, ITM Monitoring / ICE, 08 March 2017, https://www.ice.org.uk/disciplines-and-resources/case-studies/monitoring-the-victoria-station-upgrade?utm_source=Communicator&utm_medium=email&utm_content=ITM&utm_campaign=ICE+Events+-+Knowledge+Newsletter+MAR+-+15%2f03%2f2017+-+Copy&_ccCt=E7EEgsnYueDz4HJ86Hp81Dx71l8l7fO_EZ72XZRFauD5eWlDrP8br7BLdDPXGoSX)

ΝΕΑ ΑΠΟ ΕΛΛΗΝΙΚΕΣ ΚΑΙ ΔΙΕΘΝΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΝΩΣΕΙΣ

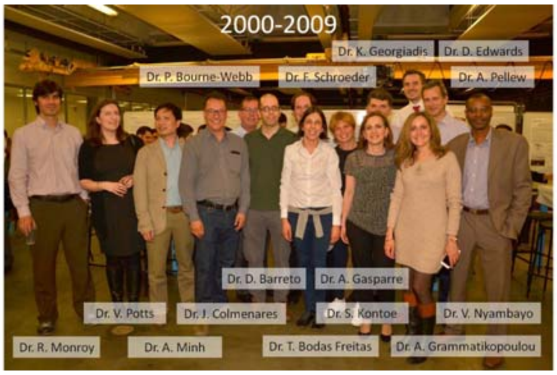
Imperial College Geotechnics PhD Alumni Event

Instigated by the current research group of the Geotechnics Section at Imperial College London, the inaugural Geotechnics PhD Alumni gathering took place on March 14th 2017. Around 50 former PhD researchers were welcomed back to the Department of Civil and Environmental Engineering by the current research group and academic and technical staff from Geotechnics. The earliest PhD graduation at the event was 1973!

Supported by a few drinks and some food, memories flooded back, creating laughter and enjoyment of being together again and catching up with what has happened since. For current students it was an excellent opportunity to show off their research posters and talk to previous generations whose names cover numerous theses and research papers.



Strong encouragements that we received to make this a regular event was a sure sign of a successful and pleasant evening, which people did not hurry to leave even when drinks ran out! We have every intention of repeating this gathering at some frequency, and will inform our alumni well in advance. We also acknowledge the support we received for this event from the Imperial's Graduate School.



ΠΡΟΣΕΧΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΚΔΗΛΩΣΕΙΣ

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

Shallow geothermal energy for buildings and infrastructure, themed issue of Environmental Geotechnics, sam.hall@icepublishing.com.

ICTUS17 The 2017 International Conference on Tunnels and Underground Spaces, 28 August 2017 - 1 September 2017, Seoul, Korea, www.i-asem.org/new_conf/asem17.htm

International Symposium on Coupled Phenomena in Environmental Geotechnics, 6-8 September 2017, Leeds, United Kingdom, http://tinyurl.com/cpeg2017

Brownfield Risk Assessment & Remediation, 13-14 September 2017, London, United Kingdom, https://brownfieldbriefing.com/risk-remediation-2017?ls=lnk

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

AGS GEOTECHNICAL DATA CONFERENCE 2017 Best Practice, Challenges and Future, 20th September 2017, Birmingham, U.K., ags@ags.org.uk

IICTG 2017 Conference "ICT — Past and Future", September 26÷28, 2017, Minneapolis, Minnesota, USA, www.iictg.org/2017-conference

AfriRock 2017, 1st African Regional Rock Mechanics Symposium, 2 – 7 October 2017, Cape Town, South Africa, www.saimm.co.za/saimm-events/upcoming-events/afrirock-2017

Geotechnique Symposium in Print 2017 Tunnelling in the Urban Environment, http://www.icevirtuallibrary.com/pb-assets/Call%20for%20Papers/Geo-Symposium-CFA-AW.pdf

HYDRO 2017 Shaping the Future of Hydropower, 9-11 October 2017, Seville, Spain, hydro2017@hydropower-dams.com

GeoAfrica 2017 3rd African Regional Conference on Geosynthetics, 9 – 13 October 2017, Morocco, http://geoafrica2017.com

3ο Πανελλήνιο Συνέδριο Φραγμάτων και Ταμιευτήρων - Διαχείριση Έργων και Προοπτικές Ανάπτυξης, 12 - 14 Οκτωβρίου 2017, Αθήνα, www.fragmata2017.gr

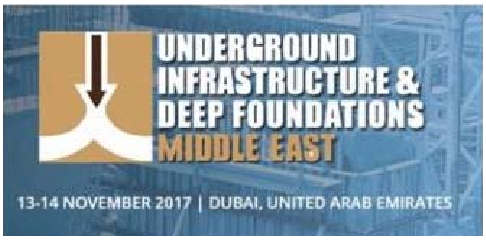
4th International Conference on Long-Term Behaviour and Environmentally Friendly Rehabilitation Technologies of Dams, 17-19 October 2017, Tehran, Iran, www.ltbd2017.ir/en

The 15th International Conference of International Association for Computer Methods and Advances in Geomechanics, 19-23 October 2017, Wuhan, Hubei Province, China, www.15iacmag.org

XIII International Conference "Underground Infrastructure of Urban Areas 2017", 24-26 October 2017, Wroclaw, Poland, http://uiua.pwr.edu.pl/?lang=en

SIFRMEG 2017 Shaoxing International Forum on Rock Mechanics and Engineering Geology, October 28-29, 2017, http://forum.hmkj.com.cn/index.php/Index/show/tid/20

11ο Συνέδριο «Ελληνική Γλώσσα και Ορολογία», 9–11 Νοεμβρίου 2017, Αθήνα, www.eleto.gr/gr/Conference11.html



www.undergrounduae.iqpc.ae

The #1 meeting place for geotechnical experts and underground engineers in the Middle East

Join us this 13 – 14 November in Dubai, UAE, at the 4th Underground Infrastructure & Deep Foundations Middle East Conference and Awards, as we gather the foremost experts, engineers, subject matter experts and solution providers to share best practice, project insight and new technologies. Delivered through a series of case studies, panel discussions and workshops from regional and international professionals, this is the leading event for the underground and deep foundations community in the Middle East.

The UAE remains a prime market opportunity with billion-dollar construction projects on the way. To support new developments like the Dubai metro extension or construction of a sewage system, it is fundamental for the UAE to put in place the best piling and foundation structures and have the most efficient methodologies to create robust underground infrastructure. However, when it comes to constructing alongside existing buildings and infrastructure, project owners, geotechnical contractors and consultants are facing tough challenges, such as waterproofing, insulation, corrosion, and site investigation accuracy issues. The 4th Underground Infrastructure and Deep Foundations Conference will address main geotechnical challenges and illustrate latest solutions to develop the deep foundation and tunnelling innovations.

This year's conference will bring together regional and international professionals from the following departments:

- Ground Engineering
- Design Engineering
- Civil Engineering of Structural Engineering/Infrastructure
- Planning Drainage Sewage
- Infrastructure, Utilities
- Water Networks

For general enquiries:

Tel: 971 4 364 2975
Email:enquiry@iqpc.ae

PARIS 2017 AFTES International Congress "The value is Underground", 13-16 November 2017, Paris, France, www.aftes2017.com

2nd International Conference "Challenges in Geotechnical Engineering" 2017 20-23 November 2017, Kyiv, Ukraine, www.cgeconf.com/en

2nd International Symposium on Asia Urban GeoEngineering, 24-27 November 2017, Changsha, China, hwww.isaug2017.org

ASIA 2018 Seventh International Conference and Exhibition on Water Resources and Renewable Energy Development in Asia, 15 March 2018, Danang, Vietnam, www.hydropower-dams.com/asia-2018-conference.php?c_id=303

World Tunnel Congress 2018 "The Role of Underground Space in Future Sustainable Cities", 20-26 April 2018, Dubai, United Arab Emirates, www.wtc2018.ae

EUROCK 2018 Geomechanics and Geodynamics of Rock Masses, 22-26 May 2018, Saint Petersburg, Russia, www.eurock2018.com/en

16th European Conference on Earthquake Engineering (16thECEE), 18-21 June 2018, Thessaloniki, Greece, www.16ecee.org

CPT'18 4th International Symposium on Cone Penetration Testing, 21-22 June 2018, Delft, Netherlands, www.cpt18.org

NUMGE 2018 9th European Conference on Numerical Methods in Geotechnical Engineering, 25-27 June 2018, Porto, Portugal, www.numge2018.pt

RockDyn-3 - 3rd International Conference on Rock Dynamics and Applications, 25-29 June 2018, Trondheim, Norway, www.rocdyn.org

GeoChine 2018 - 5th GeoChina International Conference Civil Infrastructures Confronting Severe Weathers and Climate Changes: From Failure to Sustainability, July 23-25, , HangZhou, China, http://geochina2018.geoconf.org

UNSAT2018 The 7th International Conference on Unsaturated Soils, 3 - 5 August 2018, Hong Kong, China, www.unsat2018.org

SAHC 2018 11th International Conference on Structural Analysis of Historical Constructions "An interdisciplinary approach", 11-13 September 2018, Cusco, Perú http://sahc2018.com

11th International Conference on Geosynthetics (11ICG), 16 - 20 Sep 2018, Seoul, South Korea, www.11icg-seoul.org

CHALK 2018 Engineering in Chalk 2018, 17-18 September 2018, London, U.K., www.chalk2018.org

ARMS10 - 10th Asian Rock Mechanics Symposium, ISRM Regional Symposium, 29 October - 3 November 2018, Singapore, www.arms10.org

WTC2019 Tunnels and Underground Cities: Engineering and Innovation meet Archaeology, Architecture and Art and ITA - AITES General Assembly and World Tunnel Congress, 3-9 May 2019, Naples, Italy, www.wtc2019.com

7 ICEGE 2019

International Conference on Earthquake Geotechnical Engineering
17 - 20 June 2019, Rome, Italy

Organizer: TC203 and AGI (Italian Geotechnical Society)
Contact person: Susanna Antonielli
Address: AGI - Viale dell' Università 11, 00185, Roma, Italy
Phone: +39 06 4465569
Fax: +39 06 44361035
E-mail: agi@associazionegeotecnica.it

ISDCG 2019

7th International Symposium on Deformation Characteristics of Geomaterials
26 - 28 June 2019, Glasgow, Scotland, UK,

The Technical Committee 101 of the ISSMEG is pleased to announce the organisation of the 7th International Symposium on Deformation Characteristics of Geomaterials (ISDCG) in 2019, in Glasgow, UK. The symposium is co-organised by the University of Strathclyde in Glasgow, the University of Bristol, and the Imperial College in London.

Building on the success of the previous Symposia organised in Sapporo (Japan) Japan in 1994, Torino (Italy) in 1999, Lyon (France) in 2003, Atlanta (US) in 2008, Seoul (Korea) in 2011 and Buenos Aires (Argentina) in 2015, the 7th ISDCG will equally follow both its traditions and active promotion of new technical elements to maintain it as one of the most popular and vibrant events within the geotechnical community. The technical core themes will focus on: (i) advanced laboratory geotechnical testing; (ii) application of advanced laboratory testing in research, site characterisation, and ground modelling; (iii) application of advanced testing to practical geotechnical engineering. In addition to these traditional topics, sub-themes will include cutting-edge techniques and approaches, for example experimental micro-mechanics, non-invasive monitoring systems, nano and micro-sensors, new sensing technologies. A key goal is to engage with the full spectrum of geotechnical specialists, from early career engineers and researchers through to world leading experts.

14th ISRM International Congress
20-27 September 2019, Foz de Iguaçu, Brazil

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

The 17th European Conference on Soil Mechanics and Geotechnical Engineering
1st - 6th September 2019, Reykjavik Iceland
www.ecsmge-2019.com

The theme of the conference embraces all aspects of geotechnical engineering. Geotechnical engineering is the foundation of current as well as future societies, which both rely on complex civil engineering infrastructures, and call for mitigation of potential geodangers posing threat to these. Geotechnical means and solutions are required to ensure infrastructure safety and sustainable development. Those means are rooted in past experiences enhanced by research and technology of today.

At great events such as the European Geotechnical Conference we should: Spread our knowledge and experience to our colleagues; Introduce innovations, research and development of techniques and equipment; Report on successful geotechnical constructions and application of geotechnical design methods, as well as, on mitigation and assessment of geohazards and more.

Such events also provide an opportunity to draw the attention of others outside the field of geotechnical engineering to the importance of what we are doing, particularly to those who, directly or indirectly, rely on our services, knowledge and experience. Investment in quality geotechnical work is required for successful and safe design, construction and operation of any infrastructure. Geotechnical engineering is the key to a safe and sustainable infrastructure and of importance for the society, economy and the environment. This must be emphasized and reported upon.

XVI Asian Regional Conference on Soil Mechanics and Geotechnical Engineering
21 - 25 October 2019, Taipei, China

XVI Panamerican Conference on Soil Mechanics and Geotechnical Engineering
18-22 November 2019, Cancun, Quintana Roo, Mexico
http://panamerican2019mexico.com/panamerican

Technical program

1A – Transportation geotechnics.

1B – In situ testing.

1C – Geo-engineering for energy and sustainability.

1D – Numerical modelling in geotechnics.

1E – Foundations & ground improvement.

1F – Unsaturated soils.

1G – Embankments, dams and tailings.

1H – Excavations and tunnels.

1I – Geo-Risks.

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Nordic Geotechnical Meeting
27-29 May 2020, Helsinki, Finland

Contact person: Prof. Leena Korkiala-Tanttu
Address: SGY-Finnish Geotechnical Society,
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ΕΝΔΙΑΦΕΡΟΝΤΑ ΓΕΩΤΕΧΝΙΚΑ ΝΕΑ

Κατέρρευσε στη θάλασσα το περίφημο «Γαλάζιο Παράθυρο» της Μάλτας

Ένας βραχώδης σχηματισμός στο νησί Gozo, που μοιάζει με αψίδα και εμφανίστηκε σε αμέτρητα τουριστικά φυλλάδια της Μάλτας, κατέρρευσε σήμερα στη θάλασσα.

Οι γεωλόγοι είχαν από καιρό προειδοποιήσει ότι ο σχηματισμός των βράχων, στη βορειοδυτική ακτή του μικρού νησιού Γκόζο, διαβρώνεται με ταχείς ρυθμούς και οι αρχές είχαν απαγορεύσει την πρόσβαση των τουριστών πάνω σε αυτόν.

Ο σχηματισμός, γνωστός ως Azure Window (Γαλάζιο Παράθυρο) καθώς δημιουργούσε μια αψίδα πάνω από τα γαλάζια νερά που είναι αγαπημένος προορισμός για τους δύτες, κατέρρευσε καθώς η Μάλτα επλήγη από σφοδρή θαλασσοταραχή και θυελλώδεις ανέμους.

(8 Μαρτίου 2017, http://www.lifo.gr/now/world/136140)

The Azure Window developed through sea and rain erosion of a cliff face over a period of about 500 years. The arch was one of Malta's main tourist landmarks, and it was a popular backdrop in photographs.[1] It was included in a Special Area of Conservation,[3] and in 1998, it was included on Malta's tentative list of UNESCO World Heritage Sites, along with the rest of Dwejra Bay.

The Azure Window in 2006

Between the 1980s and the 2000s, parts of the top slab of the arch collapsed, significantly widening the arch. A large slab of rock on the outer edge of the cavity collapsed in April 2012, further increasing the size of the window. Another rock fall occurred in March 2013. A geological and geotechnical report was prepared four months later, and it determined that the arch was "relatively stable and will continue to remain so for a number of years", although it warned that rock falls will continue and it might be hazardous for people to go close to the arch.

Further rock falls and fissures were reported in subsequent years. Fishermen avoided going near the arch with their boats, and warning signs were put up to discourage people from walking on top. However, many people still went on the arch on a regular basis, and videos were uploaded on YouTube of people cliff diving from the window as rocks were falling down.

View of the Azure Window in 2009

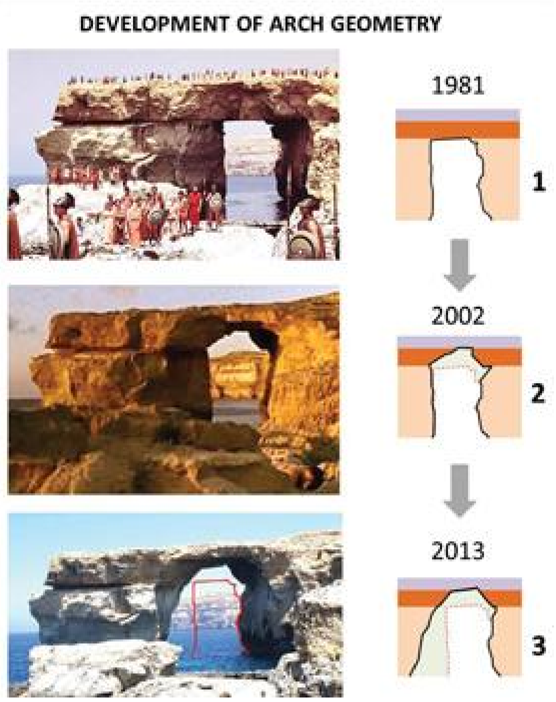
In December 2016, an emergency order was published prohibiting people from going on the arch, with trespassers facing a fine of €1500. However, this law was not enforced, and visitors were still walking on top of the arch days before it collapsed in March 2017.

In 2012 after the partial collapse

Collapse

The arch collapsed at around 08:40 UTC (09:40 a.m. local time) on 8 March 2017, after severe storms with gale-force winds. The pillar supporting the arch collapsed, and the entire structure crashed down into the sea, with nothing remaining visible above sea level.

The collapse was reported in both local and international media. Prime Minister Joseph Muscat and Leader of the Opposition Simon Busuttil both tweeted about the collapse of the Azure Window, and it also became the subject of many Internet memes on Maltese social media. The Environment and Resources Authority called the collapse a major loss to Malta's natural heritage.



Geology

The Azure Window was a natural arch with a height of about 28 m (92 ft) and a span of around 25 m (82 ft). It was located at the tip of a headland known as Dwejra Point. The arch was comprised of two types of Lower Coralline Limestone, known as Member A and Member B. Member A formed the arch's pillar and base, while Member B formed the upper layer including most of the unsupported arch.[3]

The arch was located close to the Inland Sea, a large circular sinkhole reached by a small arch that developed along a joint in the rocks. The Fungus Rock, an islet that was formed when the bridge of a natural arch collapsed leaving a stack, is also found nearby. Another natural arch, the Wied il-Mielaħ Window, is located about 3.7 km (2.3 mi) northeast of Dwejra.

https://en.wikipedia.org/wiki/Azure_Window

Norway may build the world's first tunnel for ships

Stretching over 15 miles, Norway's Lærdal Tunnel has been the longest road tunnel in the world since it was built in the 1990s.

Now, the country is trying to construct the world's first tunnel for ships — which would see between 70 and 120 vessels per day.

As noted by BBC News, the tunnel started gaining public support in the 1980s, and the Norwegian Coastal Administration (NCA) has since embarked on several feasibility studies to see how it could be realized.

Terje Andreassen, the tunnel's project manager, tells Business Insider that the NCA will submit its findings to Norway's Department of Transport in May 2017. The project would need to be fully financed by the Norwegian Parliament, which has already designated approximately $117 million ($1 billion NOK) for the NCA's research.

A rendering of the Stad Ship Tunnel in Norway.

If the plans are approved, construction could start as early as 2019, Andreassen says.

The NCA worked with Snøhetta, an Oslo-based architecture firm, to create renderings of what the tunnel would look like. Check them out below.

If Norway's Stad Ship Tunnel gets approved, it would be located on the Stadhavet Sea, an area known for heavy winds and hurricanes.

Stormy weather makes it almost impossible for ships to travel in the area. Vessels often wait for hours or even days before it becomes safe enough to cross, Andreassen says. His team at the NCA is conducting research on the feasibility of building the tunnel.

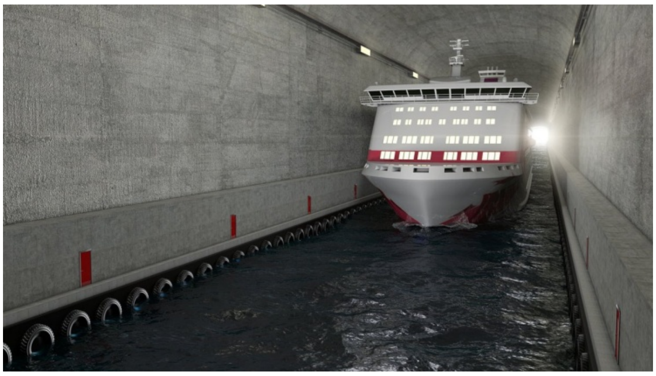
The Stad Tunnel would allow for safer passage along the sea, Andreassen says. Here's a rendering of what it would look like:

From the north, ships would access the tunnel near the town of Selje. The southern entrance would be through the Molde fjord, where there would also be a new pedestrian bridge.

The NCA estimates that the mile-long tunnel would see between 70 and 120 passenger and freight ships per day. It would measure 148 feet high and 118 feet wide.

Various kinds of vessels would pass through it, including small ferries, which Andreassen says currently can't travel along the Stadhavet Sea when the weather's bad. All ships would navigate the tunnel on their own.

If the ambitious project moves forward, the tunnel's construction team would need to drill and blast through rock to create a channel.



The tunnel would be the first of its kind in the world — but it wouldn't be cheap. The NCA estimates that its construction could cost around 2.3 billion NOK, or approximately $267 million.

(Leanna Garfield / BUSINESS INSIDER, Mar. 15, 2017, http://www.businessinsider.com/norway-worlds-first-tunnel-for-ships-2017-3/#if-norways-stad-ship-tunnel-gets-approved-it-would-be-located-on-the-stadhavet-sea-an-area-known-for-heavy-winds-and-hurricanes-1)

How To Fix Oroville Dam
California must design a lasting replacement for the destroyed spillway

On Sunday, Feb. 12, California officials ordered the immediate, mandatory evacuation of 188,000 residents from towns below the Oroville Dam. Two days later, when federal and state officials deemed the dam safe, the evacuation order was rescinded, and people were allowed to return to their homes. It isn't often that hundreds of thousands of people in the U.S. have to leave their homes because of worries about a catastrophic structural failure.

As an experienced civil engineer, I would like to make one point very clear: Oroville Dam was never in danger of failing. Because the Oroville Dam crest is 21 ft higher than the emergency spillway crest, the dam will never be overtopped.

After the flow from the main spillway was shut off, construction workers clear the rock debris strewn across the bottom of the Oroville Dam spillway on February 27, 2017.

But the California Dept. of Water Resources must take this opportunity to design a lasting replacement for the destroyed spillway. I'll explain why.

Located on the Feather River about 65 miles north of Sacramento, Calif., Oroville Dam is 770 ft high—the nation's tallest. Unlike Hoover Dam and Glen Canyon Dam, Oroville Dam is a zoned-earthfill structure.

Water can be released from the reservoir (Lake Oroville) behind the dam in four ways: power plant, river outlets, main spillway and emergency, or auxiliary, spillway. Because rock debris is damming up the powerhouse tailrace, the Oroville power plant is out of service. Due to various operational problems, the river outlets cannot be used. This leaves only the main spillway and the emergency spillway to let water out of the lake.

The main spillway is a 3,050-ft-long, concrete-lined chute, which extends from the flood-control outlet structure down to the Feather River. The gated headworks at the outlet structure has eight radial gates to meter water flow into the spillway chute. The main spillway is rated at 150,000 cubic ft per second (cfs).

The final component for releasing floodwaters from Lake Oroville is the emergency, or auxiliary, spillway. This is a concrete, 30-ft-high ungated overflow weir, with a crest length of 1,730 ft. When floodwater passes over this weir, the water tumbles down an unlined slope to the Feather River.

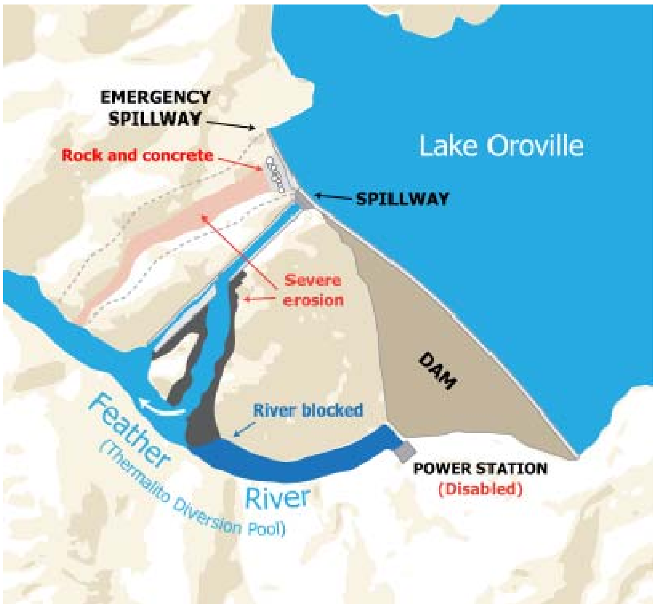
Heavy Rainfall Spikes Lake Levels

After large storms hit the Feather River tributaries in early February, Lake Oroville's water levels began rising dramatically. Normally, this doesn't happen until the snow melts and the spring rains bring additional water into the reservoir.

These early thunderstorms had a tremendous impact on the reservoir behind Oroville Dam. From Feb. 6 to Feb. 11, high inflows entered Lake Oroville, with four days over 100,000 cfs. On Feb. 8, the reservoir took in a whopping 240,000 acre-feet of water in a 24-hour period. Over the course of six days, from Feb. 6-11, Oroville Lake levels rose 52 ft!

On Feb. 11, the lake rose 3 ft, and water started flowing over the emergency spillway shortly after 8:00 a.m. This was the first time that water had moved over the emergency spillway since 1967, when the Oroville Dam was built. Water continued flowing over the emergency spillway until 8:00 p.m. on Feb. 12.

The California Dept. of Water Resources estimates that about 6,000 to 12,600 cfs of water passed over the emergency spillway on Saturday, Feb. 11, and Sunday, Feb. 12. When compared with the main spillway releasing 100,000 cfs, this is a drop in the proverbial bucket.



A relocated main spillway could be inserted between the existing spillway and the dotted lines in this simplified plan view of the dam and related structures.

As the water made its way down the steep slope to the Feather River, significant erosion took place. Huge canyons were carved out of the hillside, and sections of the boat-ramp access road were destroyed.

The Spillway Problems

Of particular concern was the erosion that took place at the toe of the emergency spillway. After the emergency spillway stopped flowing, engineers could assess the damage that took place on Saturday and Sunday. Holes near the 30-ft-high emergency spillway were readily apparent.

Some people feared that this concrete weir would be undermined and fail, thereby releasing a 30-ft-high wall of water onto the towns below the dam. This is why the authorities called for the immediate and mandatory evacuation.

Since Feb. 12, contracting crews have been working around the clock to place rock and concrete over the affected areas. Workers are placing rock with trucks, loaders and excavators. After the rock is deposited, transit-mix concrete trucks deliver concrete for placement by multiple truck-mounted boom concrete pumps.

In a questionable practice, workers loaded bags with small rocks or sand and flew these bags into place with helicopters. Even though concrete was placed around the rock bags, these bags may not have survived the rushing water.

With several helicopters shuttling back and forth from the loading area to the deposit areas, it looked more like a Hollywood movie set than a serious spillway repair effort!

The problems with the main spillway were brought to light on Feb. 7, when a large hole appeared in the spillway's concrete slab, near the location of an earlier patch job. By Feb. 10, the spillway crater had grown to 500 ft long x 300 ft wide x 45 ft deep.

In the process, the concrete side wall collapsed and was washed away. As the crater got larger, more and more water shot out of the spillway and carved a new canyon, carrying the water down to the Feather River. Depending on the flow volume, most of the water leaves the spillway chute at the location of this large crater; very little water stays in the chute until the lower end.

What caused the failure of the main spillway's concrete slab?

Main Spillway Solutions

On Feb. 14, I sent a letter to the Dept. of Water Resources in which I suggested that the concrete in the main spillway's chute had been destroyed by powerful "cavitation" forces. Cavitation can occur in spillways and tunnels when high-velocity water encounters irregularities in the concrete surface. At these locations, cavitation creates high-pressure shock waves that are capable of destroying solid concrete. Proper aeration of the water flow can minimize the harmful effects of cavitation.

The department should immediately design the main spillway's replacement so that construction can take place this summer, during the low-flow season. The department has placed the rebuilding cost of the main spillway at about $100 million to $200 million.

Besides tough site conditions, the biggest obstacle could be the need to handle the main spillway's water flows during construction. How will the contractor divert 60,000 to 100,000 cfs of high-velocity water around the repair site during construction? Because this will be extremely difficult, construction must be pursued on a 24/7 basis to accomplish all the work in the dry season.

A logical alternative is to design a new main-spillway chute to replace the 50-year-old spillway. Of course, anti-cavitation measures would be incorporated into the new design, along with new energy-dissipation features at the Feather River (possibly a flip bucket and stilling basin).

The new spillway chute would have an alignment almost parallel with the existing spillway, a short distance to the northwest. Most of the construction work could be accomplished even if the existing spillway has to carry water. When the new spillway chute is built, it will be tied into the existing spillway, near the gated headworks, with a training wall at a very shallow angle.

On Feb. 27, the Dept. of Water Resources shut off the flow of water in the main spillway to allow workers to clear rock and concrete debris from the Feather River, allowing for the partial operation of the powerhouse turbines. With no water flowing, people could see how rushing water can utterly destroy a weakened concrete spillway. Roughly half the spillway's length has been reduced to short stretches of concrete, rubble and huge canyons.

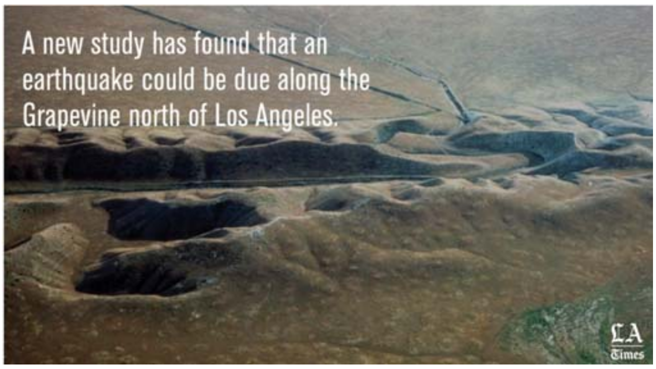
This makes total replacement of the main spillway in a new location even more attractive—and that is the course of action I urge the department to take.

Henry W. Burke, BSCE and MSCE, a former employee of Kiewit Corp., is a civil engineer with extensive knowledge of heavy construction. He is based in Omaha, Nebraska and can be reached at hwburke@cox.net.

(Henry W. Burke / ENGINEERING NEWS-RECORD, March 19, 2017, http://www.enr.com/articles/41664-how-to-fix-oroville-dam)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΣΕΙΣΜΟΙ

A section of the San Andreas fault close to L.A. could be overdue for a major earthquake



Researchers say the land has been pushing at a rate of more than 1 inch a year since 1857. (March 7, 2017)

Southern California could be overdue for a major earthquake along the Grapevine north of Los Angeles, according to a sobering new study by the U.S. Geological Survey.

The research found earthquakes happen there on average every 100 years. The last major temblor occurred 160 years ago, a catastrophic geological event that ruptured an astonishing 185 miles of the San Andreas fault.

The land on either side of the fault has been pushing against the other at a rate of more than 1 inch a year since 1857, the researchers said, accumulating energy that will be suddenly released in a major earthquake, when the land along the fault would move by many feet.

"So you expect that amount of accumulation of energy will be released in the future in a large-magnitude rupture, somewhere along the San Andreas," said the lead author of the study, USGS research geologist Kate Scharer.

A repeat of the 1857 earthquake could damage aqueducts that ferry water into Southern California from the north, disrupt electric transmission lines and tear up Interstate 5, whose Grapevine section runs on top of the San Andreas fault at Tejon Pass.

Scientists observing trenches at this site discovered that earthquakes on this section of the San Andreas fault occurred on average once every 100 years.

Central Los Angeles could experience a couple of minutes of shaking, which could feel like a lifetime compared with the 1994 Northridge earthquake, which shook for roughly 15 seconds.

"This would be more broadly felt across the basin," Scharer said. "It would impact our ability to be a world-class city."

There has been a long drought of major earthquakes on the southern San Andreas fault, which has slowly been accumulating strain as the Pacific plate grinds northward against the North American plate. The seismic stress has been building up without relief since the presidency of Franklin Pierce.

The quake will probably be something few Southern Californians have ever experienced. Of the 10 earthquakes Scharer and her colleagues found in the past 1,200 years, the most common magnitude was about magnitude 7.5.

Such an earthquake would tear up land along the fault's length and displace it by an average of 9 feet.

The site studied by Scharer and her colleagues is next to Frazier Mountain at the top of Tejon Pass, close to the meeting point of Los Angeles, Ventura and Kern counties. Unlike other sections of the southern San Andreas, this region — about a 100-mile section of the fault — had not been studied in detail before, and scientists did not know what earthquakes occurred there before the 19th century.

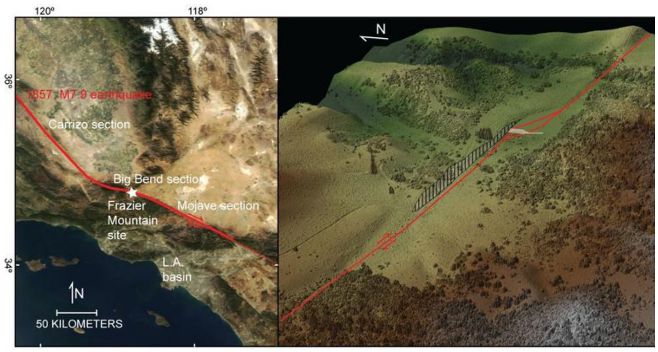


Image showing the location where scientists studied a section of the San Andreas fault, identified by the red line, and found that earthquakes there happened on average once every 100 years. The most common earthquake found was magnitude 7.5. (USGS)

So Scharer and other scientists undertook the most intensive study of its kind in this section of the San Andreas fault, where researchers could hear the roar of traffic from Interstate 5. Trenches were dug deeper than ever before at this location to better understand the history of earthquakes there.

"To get 1,200 years of records, we have to do lots of excavations and go quite deep," Scharer said.

They found 10 major earthquakes over a 1,000-year period. But "it doesn't happen like clockwork," Scharer said. There was once a gap of only 20 years between two major quakes. On the other end, there was a gap of about 200 years between quakes.

So while this part of the San Andreas fault could be overdue for a large earthquake, it's also possible it could be decades longer before the Big One strikes. Of the identified gaps between earthquakes, three took longer than 160 years to strike this part of the San Andreas again.

"Longer gaps have happened in the past, but we know they always do culminate in a large earthquake. There's no getting out of this," Scharer said.

There was one possible silver lining. The most common magnitude they found at this site, 7.5, means that the 7.9 earthquake experienced in 1857 was unusually strong.

In fact, of the newly discovered earthquakes, there was only one other estimated at 7.9 — one that struck around 1550.

The conclusion that 7.9 earthquakes there are relatively rare could be good news, said Caltech seismologist Egill Hauksson, who was not involved in the study.

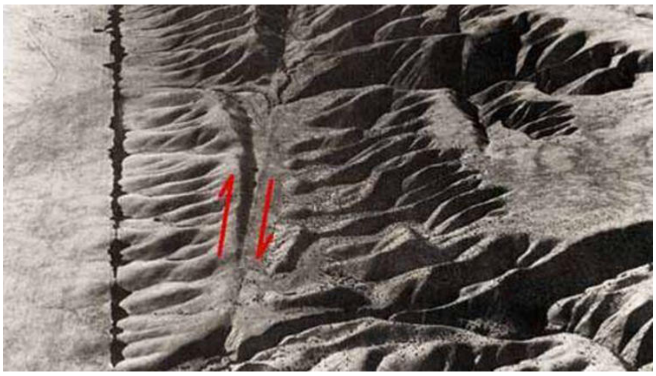


Image showing San Andreas fault (USGS)

"That means it released a lot of energy on the fault," Hauksson said of the 1857 temblor. It also could explain why the southern San Andreas fault has been so quiet since then.

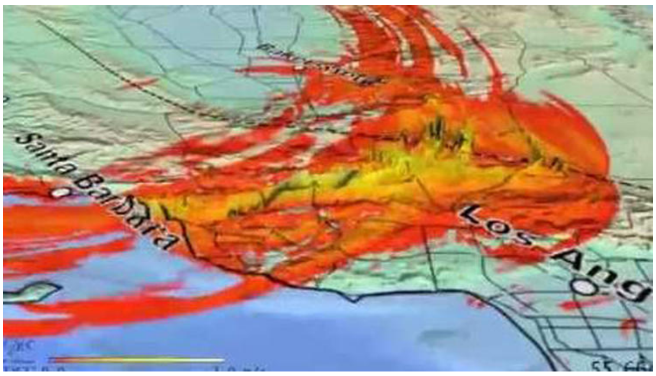
The difference between a 7.5 and a 7.9 earthquake is significant, Scharer said. A 7.5 earthquake produces roughly 15 times less energy than a 7.9. And while a 7.5 earthquake displaces land along the fault by an average of 9 feet, the bigger earthquake moves land by 20 feet.

"A road can handle a couple of feet of displacement," Scharer said, "but when you start to get into a dozen feet, it's a real challenge."

The study was published online Thursday in the Journal of Geophysical Research, a publication of the American Geophysical Union.

The 1857 quake was so powerful that the soil liquefied, causing trees as far away as Stockton to sink. Trees were also uprooted west of Fort Tejon. The shaking lasted one to three minutes.

The San Andreas fault is 30 miles from downtown Los Angeles, but big earthquakes on the San Andreas fault are expected to heavily shake the nation's second largest city.



The simulated shaking from a possible magnitude-8 earthquake on the San Andreas fault. (Southern California Earthquake Center)
https://www.youtube.com/watch?v=V2Ow0Yuv5co

A simulation of a possible magnitude 8 unzipping from Monterey County into L.A. County shows heavy shaking waves reverberating across a wide swath of the Los Angeles Basin and nearby valleys, whose soft soils can trap shaking energy like Jell-O.

Because the 1857 quake hit when the population was so small, only two people were killed.

Other parts of the San Andreas fault could also rupture. Further southeast of the Cajon Pass, such as in San Bernardino County, the fault has not moved substantially since an earthquake in 1812, and further southeast toward the Salton Sea, it has been relatively quiet since about 1680 to 1690.

"The springs on the San Andreas system have been wound very, very tight," Thomas Jordan, director of the Southern California Earthquake Center, said at an earthquake conference last year, adding that the southern San Andreas is "locked, loaded and ready to roll."

In 2008, the U.S. Geological Survey simulated a magnitude 7.8 earthquake on the southern San Andreas that would begin at the Salton Sea and spread west toward the San Gabriel Mountains in Los Angeles County. Such a quake could cause more than 1,800 deaths, 50,000 injuries, $200 billion in damage and severe, long-lasting disruptions.

The collapse of five high-rise steel buildings, with 5,000 people inside them, was seen as possible in this USGS scenario, as would be the destruction of 50 concrete buildings; 800 people might be in completely collapsed concrete buildings, and 7,000 more in those that partially collapsed. About 900 brick buildings could be irreparably damaged.

Some cities, like Los Angeles and Santa Monica, have responded by passing laws requiring owners retrofit some of the most vulnerable buildings known to engineers. Santa Monica recently moved forward on a sweeping law that would require evaluations, and if needed, retrofits of wood apartments, brittle concrete buildings and flawed steel towers.

(Rong-Gong Lin, Contact Reporter / Los Angeles Times, March 7, 2017, http://www.latimes.com/local/lanow/la-me-ln-earthquake-san-andreas-grapevine-20170307-story.html)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΓΕΩΛΟΓΙΑ

How One Brilliant Woman Mapped the Secrets of the Ocean Floor

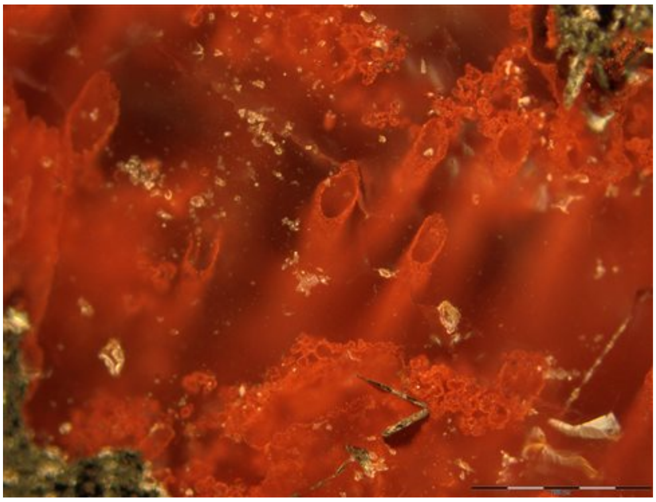
This video tells the story of a female geologist, Marie Tharp, whose groundbreaking work in mapmaking helped bring the theory of continental drift into the mainstream. Her ideas (which a male colleague initially dismissed as "girl talk," 2:34) have helped shape what we know about Earth and its geologic history.

The characters and themes—a persistent woman, scientific truth winning out—make this story feel very contemporary. Plus, it's all accomplished through clever animations of both the characters and the visualizations of the complex ideas they discuss.

http://video.nationalgeographic.com/video/short-film-showcase/how-one-brilliant-woman-mapped-the-secrets-of-the-ocean-floor?utm_source=NatGeocom&utm_medium=Email&utm_content=video_20170306_b&utm_campaign=Content&utm_rd=1084349954

(Lisa Covi, Production Manager / National Geographic)

Απολιθώματα 3,77 δισ. ετών
Εντοπίστηκαν ίχνη μίας από τις αρχαιότερες μορφές ζωής



Η ανακάλυψη ενισχύει την ιδέα ότι η ζωή ξεπήδησε από καυτές υδροθερμικές πηγές στο βυθό των θαλασσών

Οι επιστήμονες πιστεύουν ότι ανακάλυψαν στο Κεμπέκ του Καναδά, τα αρχαιότερα απολιθώματα μικροοργανισμών που έχουν βρεθεί στον πλανήτη μας και τα οποία έχουν ηλικία τουλάχιστον 3,77 δισεκατομμυρίων ετών.

Πρόκειται για τα ίχνη που άφησε μία από τις αρχαιότερες μορφές ζωής πάνω στη Γη.

Μικροσκοπικά ίχνη σαν νημάτια και κύλινδροι, τα οποία είχαν σχηματισθεί από τα βακτήρια που ζούσαν σε ορυκτά με σίδηρο, βρέθηκαν εγκλωβισμένα μέσα σε στρώματα χαλαζία.

Τα συγκεκριμένα ιζηματογενή πετρώματα στην περιοχή Νουβιαγκιτούκ του βορειοανατολικού Καναδά, είναι από τα αρχαιότερα στην Γη και πιθανότατα αποτελούσαν τμήμα ενός υποθαλάσσιου υδροθερμικού συστήματος, πλούσιου σε σίδηρο, το οποίο παρείχε το κατάλληλο περιβάλλον για τις πρώτες μορφές ζωής πριν από 3,77 έως 4,3 δισεκατομμύρια χρόνια.

Οι ερευνητές από πολλές χώρες (Βρετανία, ΗΠΑ, Καναδά, Αυστραλία, Νορβηγία), με επικεφαλής τους Δρ Ντομινίκ Παπινό και Μάθιου Ντοντ της Σχολής Γεωεπιστημών του University College του Λονδίνου (UCL), έκαναν τη σχετική δημοσίευση στο περιοδικό Nature.

«Η ανακάλυψη ενισχύει την ιδέα ότι η ζωή ξεπήδησε από καυτές υδροθερμικές πηγές στο βυθό των θαλασσών, λίγο μετά το σχηματισμό της Γης», δήλωσε ο Μάθιου Ντοντ.

«Η ζωή στη Γη αναπτύχθηκε σε μια εποχή που τόσο η ίδια όσο και ο Άρης είχαν υγρό νερό στις επιφάνειές τους. Συνεπώς, αναμένουμε ότι θα βρούμε ενδείξεις για ζωή στον Άρη πριν από περίπου τέσσερα δισεκατομμύρια χρόνια. Αν όχι, τότε η Γη θα αποτέλεσε μια ειδική περίπτωση».

Μέχρι τώρα, ως αρχαιότερα γνωστά απολιθώματα μικροοργανισμών, ηλικίας 3,46 δισεκατομμυρίων ετών, θεωρούνταν αυτά που είχαν βρεθεί στη Δυτική Αυστραλία.

Τόσο στην Αυστραλία, όσο και στον Καναδά, η πρόκληση που αντιμετωπίζουν οι επιστήμονες, είναι να βεβαιωθούν ότι τα ίχνη πάνω στα πετρώματα έχουν όντως βιολογική προέλευση, άρα αποτελούν απολιθώματα μικροοργανισμών, αφού οι τελευταίοι δεν έχουν σκελετό για να απολιθωθεί.

Στην περίπτωση του Κεμπέκ, τα μικροσκοπικά νημάτια και οι κύλινδροι έχουν γίνει από αιματίτη, μία μορφή οξείδωσης (σκουριάς) του σιδήρου. Οι ερευνητές εξέτασαν και απέρριψαν κάθε εναλλακτική μη βιολογική εξήγηση για την προέλευση αυτών των ιχνών, π.χ. να προήλθαν από μεταβολές στην θερμοκρασία και στην πίεση λόγω των γεωλογικών διαδικασιών.

Τα αρχαία αυτά ίχνη έχουν τα ίδια χαρακτηριστικά με τα βακτήρια, που σήμερα ανακαλύπτονται γύρω από υδροθερμικές «καμινάδες» στον βυθό και τα οποία επίσης προκαλούν οξείδωση του σιδήρου.

(Newsroom ΔΟΛ, 01 Μαρ. 2017, http://news.in.gr/science-technology/article/?aid=1500131751&ref=newsletter)

Evidence for early life in Earth's oldest hydrothermal vent precipitates

Matthew S. Dodd, Dominic Papineau, Tor Grenne, John F. Slack, Martin Rittner, Franco Pirajno, Jonathan O'Neil & Crispin T. S. Little

Abstract

Although it is not known when or where life on Earth began, some of the earliest habitable environments may have been submarine-hydrothermal vents. Here we describe putative fossilized microorganisms that are at least 3,770 million and possibly 4,280 million years old in ferruginous sedimentary rocks, interpreted as seafloor-hydrothermal vent-related precipitates, from the Nuvvuagittuq belt in Quebec, Canada. These structures occur as micrometre-scale haematite tubes and filaments with morphologies and mineral assemblages similar to those of filamentous microorganisms from modern hydrothermal vent precipitates and analogous microfossils in younger rocks. The Nuvvuagittuq rocks contain iso-

topically light carbon in carbonate and carbonaceous material, which occurs as graphitic inclusions in diagenetic carbonate rosettes, apatite blades intergrown among carbonate rosettes and magnetite–haematite granules, and is associated with carbonate in direct contact with the putative microfossils. Collectively, these observations are consistent with an oxidized biomass and provide evidence for biological activity in submarine-hydrothermal environments more than 3,770 million years ago.

Nature **543**, 60–64 (02 March 2017)
doi:10.1038/nature21377

(http://www.nature.com/nature/journal/v543/n7643/full/nature21377.html?foxtrotcallback=true)

Ανθρωπογενείς πέτρες
Πώς τα ορυχεία του Λαυρίου εμπλούτισαν τη Γη με νέα ορυκτά

Από τα συνολικά 5.208 ορυκτά της Γης που έχουν αναγνωρισθεί επίσημα από τη Διεθνή Ορυκτολογική Ένωση, τα 208, δηλαδή το 4% ή το ένα στα 25, έχουν ανθρωπογενή προέλευση, καθώς έχουν σχηματισθεί χάρη στις ανθρώπινες δραστηριότητες. Τα περισσότερα από αυτά γεννήθηκαν μόλις κατά τα τελευταία 200 χρόνια, υπάρχουν όμως και αρχαίες περιπτώσεις όπως στα αρχαία ορυχεία του Λαυρίου.

Από τα 208 αναγνωρισμένα ανθρωπογενή ορυκτά, τα 29 περιέχουν άνθρακα, ενώ ορισμένα από αυτά έχουν βρεθεί στην Ελλάδα, όπως ο νεάλιθος και ο φιντλερίτης στο Λαύριο.

Δείγμα νεάλιθου από το Λαύριο. Το ορυκτό αυτό είναι έμμεσο προϊόν του ανθρώπου (Πηγή: RRUFF)

Αυτό προκύπτει από μια νέα επιστημονική έρευνα, η οποία για πρώτη φορά κατέγραψε σε ένα παγκόσμιο κατάλογο (http://rruff.info/ima) όλα τα ορυκτά που έχουν προέλθει κατά κύριο λόγο ή αποκλειστικά χάρη στους ανθρώπους.

Η προέλευσή τους οφείλεται στο ότι, εξαιτίας των ανθρώπων, δημιουργήθηκαν οι συνθήκες για να έλθουν σε επαφή μεταξύ τους χημικά στοιχεία και να προκληθούν γεωχημικές αντιδράσεις, που αλλιώς δεν θα είχαν συμβεί στη φύση. Αν, για παράδειγμα, οι μεταλλωρύχοι δεν είχαν σκάψει τη στοά κάποιου ορυχείου (π.χ. στο Λαύριο), μπορεί να μην είχαν σχηματισθεί στα τοιχώματά του οι κρύσταλλοι ενός άγνωστου έως τότε ορυκτού.

Οι άνθρωποι έχουν -μετά την αύξηση του οξυγόνου πριν από 2,2 δισεκατομμύρια χρόνια- τη μεγαλύτερη συνεισφορά στην ποικιλία των ορυκτών της Γης. Αυτή η διαπίστωση, σύμφωνα με τους επιστήμονες, αποτελεί άλλο ένα επιχείρημα ότι ο πλανήτης μας έχει πλέον εισέλθει σε μια νέα γεωλογική εποχή, την Ανθρωπόκαινο, που έρχεται να διαδεχθεί την Ολόκαινο (άρχισε πριν 11.700 χρόνια με το λιώσιμο των πάγων) και στην οποία οι άνθρωποι αφήνουν πλέον όλο και πιο έντονα το αποτύπωμά τους.

Οι ερευνητές, με επικεφαλής τον Ρόμπερτ Χέιζεν, επικεφαλής γεωεπιστήμονα του Ινστιτούτου Επιστημών Κάρνεγκι στην Ουάσιγκτον και διευθυντή του διεθνούς Παρατηρητηρίου Βαθέος Άνθρακα, επισημαίνουν στη δημοσίευσή τους στο American Mineralogist (https://drive.google.com/file/d/0B1bF9zth54L8UHVzQWZWQmdOWXc/view) ότι τα περισσότερα ανθρωπογενή ορυκτά έχουν προέλθει από εξορυκτικές δραστηριότητες.

Ορισμένα άλλα έχουν βρεθεί σε μεταλλουργεία, ενώ κάποια έχουν σχηματισθεί στις σωληνώσεις των γεωθερμικών μονάδων, ακόμη και σε ναυάγια πλοίων, στις αρχαιολογικές ανασκαφές ή μέσα στις αποθήκες των μουσείων. Σε όλες αυτές τις περιπτώσεις, ένα φυσικό υλικό ήλθε σε επαφή με ένα άγνωστο για εκείνο περιβάλλον (π.χ. με το θαλασσινό νερό ή με το ξύλο στα ράφια μιας αποθήκης) και, μέσα από μια χημική αντίδραση, σχηματίσθηκε ένα νέο ορυκτό. Αν οι άνθρωποι δεν είχαν κάνει την μεταφορά με το πλοίο ή την αρχαιολογική ανασκαφή, αυτό το ορυκτό μπορεί να μην είχε υπάρξει ποτέ στη φύση.

Αν και μερικά ανθρωπογενή ορυκτά μπορούν να προκύψουν επίσης και μέσω φυσικών διαδικασιών, πολλά άλλα όχι. Στην ιστορία του πλανήτη μας η εξέλιξη των ορυκτών δεν σταμάτησε ποτέ, καθώς με το πέρασμα του χρόνου τα χημικά στοιχεία συναντιούνται σε διάφορους συνδυασμούς μέσα στη Γη, σε συγκεκριμένες τοποθεσίες, βάθη και θερμοκρασίες, για να «γεννήσουν» νέα ορυκτά.

Το λεγόμενο Μεγάλο Οξειδωτικό Συμβάν, δηλαδή η μεγάλη αύξηση του οξυγόνου στην ατμόσφαιρα της Γη πριν από περίπου 2,2 δισ. χρόνια, είχε ως αποτέλεσμα να δημιουργηθούν σχεδόν τα δύο τρίτα των περίπου 5.200 ορυκτών.

Μετά την εμφάνιση των ανθρώπων, οι δραστηριότητές τους συνέβαλαν άμεσα ή έμμεσα, ώστε να σχηματισθούν και άλλα ορυκτά, τα περισσότερα μετά τον 18ο αιώνα, όταν άρχισε σταδιακά η βιομηχανική επανάσταση. Όπως είπε ο Χέιζεν, «πιστεύουμε ότι και άλλα ορυκτά συνεχίζουν να σχηματίζονται σήμερα με τον ίδιο σχετικά γρήγορο ρυθμό».

Οι άνθρωποι επιδρούν στον ανόργανο ορυκτό κόσμο, προκαλώντας τη δημιουργία νέων ορυκτών ως ένα αθέλητο υποπροϊόν των διαφόρων δραστηριοτήτων τους, ενώ συνεχώς μετακινούν τεράστιες ποσότητες πετρωμάτων και ορυκτών από το ένα μέρος της Γης στο άλλο. Οι ερευνητές θεωρούν πιθανό ότι υπάρχουν εκατοντάδες ανθρωπογενή ορυκτά που ακόμη δεν έχουν αναγνωρισθεί, μέσα σε παλιά ορυχεία, χυτήρια, εγκαταλειμμένα κτίρια κ.ά.

Ο κατάλογος των 5.208 ορυκτών δεν περιλαμβάνει όσα οι άνθρωποι παράγουν βιομηχανικά. Πρόκειται για χιλιάδες νέα συνθετικά υλικά που έχουν ιδιότητες των ορυκτών, χωρίς να υπάρχει κάτι αντίστοιχο στο ηλιακό μας σύστημα, ίσως και και σε όλο το σύμπαν (ημιαγωγοί, κρύσταλλοι λέιζερ, μαγνήτες, μπαταρίες, τούβλα, τσιμέντο, χάλυβας, τιτάνιο, συνθετικοί πολύτιμοι λίθοι κ.α.). Τα υλικά αυτά, σύμφωνα με τον Χέιζεν, θα παραμείνουν στο γεωλογικό «αρχείο» για τα επόμενα δισεκατομμύρια χρόνια ως υπόμνηση ότι κάποτε υπήρξαν άνθρωποι στη Γη.

(Newsroom ΔΟΛ, με πληροφορίες από ΑΠΕ-ΜΠΕ, 02 Μαρ. 2017, http://news.in.gr/science-technology/article/?aid=1500131870)

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Hidden crystals offer clue to a geological riddle

A South African scientist believes that zircons found on Mauritius may be the remnants of a buried continental fragment

Surrounded by idyllic turquoise sea, Mauritius may hold a hidden secret.

Oceanic crust lives fast and dies young, usually being dragged down a subduction zone after 250m years or so. By contrast, continental crust lives to a ripe old age, with some of the oldest continental crust on Earth – dating to 4bn years – found in Canada and Greenland.

But geologists have long been puzzled as to why there isn't more continental crust bobbing around. Now new research indicates that some of it might be hidden underneath ocean-island volcanoes.

Mauritius is a young volcanic island which emerged around 9m years ago. But it hides a secret. Last year, geologist Lewis Ashwal from the University of Witwatersrand in South Africa, discovered tiny zircon crystals hiding within Mauritian volcanic rocks, and these crystals turned out to be 3bn years old. Zircons are found mainly in granite on continents, and they are super-hardy, surviving immense pressures and temperatures relatively unscathed.

In this case Ashwal thinks that these zircons are the remnants of a continental fragment which was pulled apart around 85m years ago and subsequently buried under volcanic lavas. When the volcanism that created Mauritius began, around 9m years ago, Ashwal reasons that it swept up some of these hardy zircon crystals as the lava punched through the ancient crust sitting on the sea floor. His findings are published in Nature Communications.

Meanwhile, it isn't just ancient continental crust that we have trouble keeping track of. Last month scientists confirmed that Earth had an extra continent surrounding New Zealand (now known as Zealandia) which had been overlooked until now because 94% of it lies beneath the ocean.

(Kate Ravilious / The Guardian, Sunday 5 March 2017, https://www.theguardian.com/science/2017/mar/05/hidden-crystals-offer-clue-geological-riddle-terrawatch)

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Θησαυρός στην άβυσσο

Το πρώτο ορυχείο στο βυθό ετοιμάζεται για χρυσές μπίζνες

Αυτό το γιγάντιο ρομπότ θα αναλάβει να αντλεί τα θρυμματισμένα πετρώματα σε πλοίο στην επιφάνεια

Αν το εγχείρημα πετύχει, θα μπορούσε να οδηγήσει σε έναν νέο πυρετό του χρυσού: καναδική μεταλλευτική εταιρεία ετοιμάζεται να γίνει το 2019 η πρώτη που εξορύσσει χρυσό και άλλα μέταλλα από τον πλούσιο βυθό του ωκεανού.

Η καναδική Nautilus Minerals έχει εξασφαλίσει συμφωνία με την κυβέρνηση της Παπούα Νέας Γουινέας για την αξιοποίηση αποθέσεων μετάλλων στο βυθό του Αρχιπελάγους του Βίσμαρκ στον Ειρηνικό Ωκεανό, ενάμισι χιλιόμετρο κάτω από την επιφάνεια.

Στην περιοχή αυτή, που βρίσκεται κοντά σε όρια τεκτονικών πλακών, καυτό νερό που περιέχει διαλυμένα μέταλλα αναβλύζει από οπές στο βυθό. Καθώς το νερό ανεβαίνει και ψύχεται, τα μέταλλα απελευθερώνονται και πέφτουν.

Όπως δήλωσαν στο LiveScience.com εκπρόσωποι της Nautilus, οι αποθέσεις στο βυθό του αρχιπελάγους είναι 10 φορές πλουσιότερες σε χαλκό, σε σχέση με συγκρίσιμα χερσαία ορυχεία, με το ποσοστό του μετάλλου να φτάνει το 7%, συγκριτικά με 0,6% κατά μέσο όρο στην ξηρά. Οι αποθέσεις περιέχουν επίσης 20 γραμμάρια χρυσού ανά τόνο, συγκριτικά με 6 γραμμάρια στην ξηρά κατά μέσο όρο.

Η εταιρεία έχει ετοιμάσει ήδη τα τρία γιγάντια ρομπότ που θα αναλάβουν την επιχείρηση, από τα οποία το ελαφρύτερο έχει μέγεθος σπιτιού και ζυγίζει 200 τόνους: Το πρώτο ξύνει το βυθό για να τον ισοπεδώσει και να ανοίξει το δρόμο για το δεύτερο, το οποίο σπάει τα πετρώματα με μεταλλικά δόντια που θυμίζουν δεινόσαυρο. Το τρίτο ρομπότ, μια υπερμεγέθης ηλεκτρική σκούπα, ρουφάει το θρυμματισμένο πέτρωμα μαζί με νερό, σε μια σούπα με τη σύσταση του υγρού τσιμέντου. Στην επιφάνεια, το νερό φιλτράρεται ακόμα και από μικροσκοπικά σωματίδια και επιστρέφεται στον ωκεανό.

Το πρόβλημα είναι ότι η διαδικασία αυτή σηκώνει ιζήματα από το βυθό, τα οποία εξαπλώνονται στο νερό και το θολώνουν, με πιθανές παρενέργειες σε οργανισμούς που τρέφονται με φίλτρα και γενικότερα στο ωκεάνιο οικοσύστημα.

«Υπάρχει η σοβαρή ανησυχία ότι η τοξικότητα από τη διαταραχή του βυθού μπορεί να ανέβει το τροφικό πλέγμα και να φτάσει σε τοπικές κοινότητες» λέει η Νάταλι Λόουρεϊ της αυστραλιανής περιβαλλοντικής οργάνωσης Deep Seat Mining Campaign, η οποία ζητά να απαγορευτούν τα υποβρύχια ορυχεία.

Από την πλευρά της, η Nautilus υποστηρίζει ότι η αναρρόφηση των θρυμματισμένων πετρωμάτων περιορίζει το θόλωμα του νερού σε τοπική κλίμακα.

Το υποβρύχιο ορυχείο της Nautilus θα βρίσκεται μέσα στα χωρικά ύδατα της Παπούας Νέας Γουινέας, και η εταιρεία χρειάστηκε μόνο την άδεια της κυβέρνησης. Για την εξόρυξη σε διεθνή ύδατα απαιτείται η έγκριση της Διεθνούς Αρχής Βυθού του ΟΗΕ, η οποία μέχρι τώρα έχει δώσει πάνω από 25 άδειες εξερεύνησης μεταλλευμάτων.

Το εγχείρημα της Nautilus, πάντως, θεωρείται κομβικής σημασίας. Όπως παραδέχεται η Νάταλι Λόουρεϊ, «αν το σχέδιο προχωρήσει, θα ανοίξει την πόρτα για αυτήν την βιομηχανία».

(Βαγγέλης Πρατικάκης / Newsroom ΔΟΛ, 29 Μαρ. 2017, http://news.in.gr/science-technology/article/?aid=1500136551)

ΕΝΔΙΑΦΕΡΟΝΤΑ - ΛΟΙΠΑ

(αφιερωμένο σ' αυτούς που υπηρέτησαν στο Μηχανικό - ο εκδότης υπηρέτησε στην αεροπορία ...)

From conflict to communities: 75 years of the Bailey Bridge

Former Royal Engineer Alan Pearson, who now works for Mabey, reflects on his time helping communities with the Bailey Bridge's modern-day successors.

A Bailey Bridge spanning a river in Mali

My attachment to the Bailey Bridge started during my career as a Royal Engineer. My belief then and to this day was that there is not a single better piece of engineering.

Invented by Donald Bailey during the Second World War, the bridge could be easily assembled in a wide range of configurations. I was aware of the important role it played in winning World War II, and its simple design meant it was still doing an effective job for the military decades later. But now, as we approach the 75th anniversary of the Bailey, I want to make sure its contribution to civilian life is similarly celebrated.

When I left the Army more than 22 years ago, I started working with Mabey – manufacturer of the lighter, stronger, modern successors to the original Bailey such as the Super Bailey, Mabey Universal, Compact 200 and the Logistic Support Bridge (LSB). The benefits of the Bailey Bridge outside the military setting had always been obvious to me – it can be rapidly built and assembled by hand, keeping costs for governments, local authorities and aid workers down to a minimum. But it wasn't until I started travelling around the world to train diverse communities in how to build these modular steel bridges that I could see the dramatic impact they are having on the lives of real people – from those living in major cities, to indigenous communities in underdeveloped countries.

I've now visited 94 different countries with Mabey, leading teams of local people to build these life-changing bridges. I might be there because natural disasters have brought down existing structures, or perhaps as part of wider country efforts to improve basic infrastructure. Regardless of the circumstances, I've seen communities be completely transformed. Here are the three biggest impacts as I see them:

A Bailey Bridge across the Niari river in Congo

New connections open new revenue streams

As soon as local businessmen realise there's going to be a new bridge, they smell money! When we arrived in Congo in 2015 I heard about a regular ferry taking tourists from one side of the river to another, but in the rainy season this was dangerous, and in high season there was rarely enough water. Our new 130-metre bridge was going to change this and local people couldn't believe it would take us just 35 days to install – they had thought we would be there for months! Businessmen sped up their plans to open new restaurants and petrol stations to serve tourists and passers-by, and a new community was established before our very eyes.

Using local labour boosts communities

The beauty of the Bailey successors is that they are so easy to construct, you really only need one experienced trainer and a willing workforce of local people. Back in 2007, the state of Kashmir was trying to recover from a catastrophic earthquake which killed 50,000 people. As part of aid efforts, the British government supplied around thirty bridges and I headed over to train a team of local engineers and tradesmen to build them. One of the proudest moments of my career was returning years later to hear that this team had gone on to train another 20 people, who had trained another 20 people to build more and more bridges. In the worst circumstances imaginable, they learned new skills and were spreading their knowledge. It's evident that when local people do the work, they take greater ownership of the bridge – and of course they get paid, so more money is ploughed back into the local community.

Bridge building in Namibia

Bridges really do save lives

The biggest impact of a new bridge is, without doubt, ensuring people can get from A to B as safely as possible. It's something we might take for granted in developed countries, but there are still too many people around the world risking their lives to get access to vital amenities. When I arrived in Mali in 1997, local people told me that each year 20-40 people died crossing the Bakoye river. It was shocking to see vehicles which had been washed away, and hear stories about people taking the treacherous journey by canoe or by boat. As soon as local people realised we were building a safe way to cross, they immediately stopped attempting to reach the other side and simply camped out until we were finished. The journey was so dangerous it was worth holding on for.

As the 75th anniversary of the Bailey Bridge draws closer, it gives me great pleasure to highlight how it has enabled faster, safer and more efficient access across a range of challenging locations. It is a true feat of engineering, in both military and civilian settings.

(the engineer, 8th March 2017, https://www.theengineer.co.uk/from-conflict-to-communities-75-years-of-the-bailey-bridge/?cmpid=tenews_3175735)

Italy: Two die as motorway bridge collapses near Ancona

A motorway bridge has collapsed near the north-east Italian city of Ancona, killing two, Italian officials say.

The aftermath of the bridge collapse in Ancona

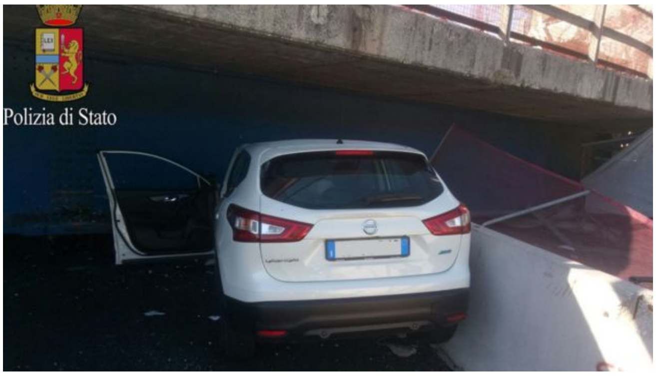
The victims were in a car below the bridge when it fell down onto the A14 road, which runs between Loreto and Ancona South.

The collapse involved a temporary structure which was supporting the bridge, Autostrade, the Italian highways agency said.

Two Romanian workers were also injured, Italian media reported.

The two people who died were a husband and wife, originally from the province of Ascoli Piceno, whose white Nissan Qashqai was crushed as the bridge came down, Ansa news agency reported.

The bridge had been closed to traffic for maintenance work, it said.



The married couple were killed as they drove under the bridge, reports say

Ansa quoted a witness named Francesco as saying the collapse "happened suddenly" and a female driver in front of him braked and stopped just 10m (33ft) from the collapsed bridge.

"We were saved by a miracle," he said.

The affected road is now closed in both directions.

(EIN NEWSDESK, 9 March 2017, http://world.einnews.com/article/370217865/FDxfeirIKonG9ptE?lcf=ZX9dkeSQfK-5FADPuwjBkQ%3D%3D)

Are Circular Runways the Future for Airports?

The idea of a circular runway has recently hit the news but it isn't a new idea. In the January 10, 1960 edition of Arthur Radebaugh's Sunday comic "Closer Than We Think" includes a curious invention that was supposed to literally catapult us into the Jet Age: The circular runway.

From the *Chicago Tribune:*

"The heart of tomorrow's airfield may be a circular catapult-like mechanism for sending planes into the air. It would mean runways much smaller than those now required.

For military purposes, American Engineering Company has already designed a giant wheel that is turned with great force by jet power. Cables from this wheel serve as catapults for fighting aircraft.

The next step would be to use rocket power to catapult planes from a dish-shaped concrete wheel. One spin on such a "circle runway" would produce the same starting speed that now requires a thousand feet or more of conventional runway, and with much less fuel."

The US Navy actually tested a similar circular runway concept in 1965. The big difference between the Navy's tests and the runway envisioned by Radebaugh? The Navy's was much, much larger and didn't have that sci-fi "rocket power unit" to propel the plane.

According to the New York Times, the US Navy pilots who tested this innovative runway found that take-off and landing was "*surprisingly easy*."

Landing is accomplished by approaching the runway in a 15 degree bank — that is, the wing facing the center of the circle is lowered 15 degrees from the horizontal. Once touchdown is accomplished, the runway seems to take care of the rest. The plane finds its natural line on the runway, depending on its speed.

The idea for the circle runway didn't originate in the 1960s. In fact, the circle runway concept dates back to at least the second decade of the 20th century — when air travel itself was still very much in its infancy.

But the idea's infancy can actually be tracked back to late 1910s, an inventor named Mr. H. T. Hanson developed a scheme that was undoubtedly one of the first modern circular runway proposals. He imagined an entire series of circular runways built over the roofs of many buildings, even in cities like New York.

From the June 1919 issue of *Popular Science*: "He would build the platform in the form of a circular, high-banked track — a track that would be constructed of light but strong iron gratings, so that sun and air would still find their way to the streets below."

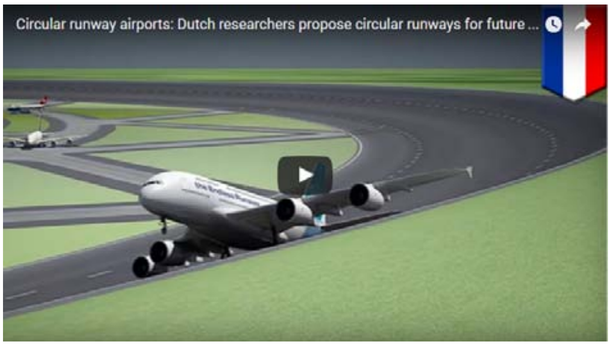
The idea of a circular runway for airports could help revolutionise air travel, shortening flight times, saving fuel costs and promoting airport efficiency.

One man brought this idea into the 21st century a few years ago and has been working with a team at the Netherland Aerospace Centre to make his dream a reality.

Henk Hesselink believes that by creating a 360° runway it will not only conserve space but it will be greener for the environment.

The EU has actually funded this project called, 'The endless runway' (ENDLESS RUNWAY) conceived a revolutionary design for a runway, based on a circular track that runs around the airport. Such a runway would permit planes to take-off in any direction and land from any direction, shortening trajectories, avoiding runway crossings and facilitating landings in any weather.

More specifically, the project team proposed a circular runway that would have a radius of 1.5 to 2.5 km, allowing changes to existing airports more readily. Such a runway would also be 400 m wide, striking a balance between limiting centrifugal forces and safety considerations. Interestingly, several aircraft can operate such a 10 km runway at the same time.



(https://www.youtube.com/watch?v=BaFcDN6Qe1k)

To achieve this, the project evaluated three operational models. The first was designed for low-wind scenarios where any part of the circle can be used in any direction. The second involved a high-wind scenario that was similar to an airport with two parallel runways. While a third model considered changing winds, involving an aircraft sequence that gradually 'moves' with the wind direction.

With these scenarios in mind, the project team found that the concept could shorten take-off and landing tracks overall by 10 % in comparison to straight runways. The model also proves to be better from an operational standpoint as wind direction will not really matter and total land use can be a lot smaller than that of conventional airports. While construction costs will be 10–60 % higher, the concept offers benefits in terms of shorter trajectories, less taxi time and continuous capacity, ideal for increasing air capacity in the world.

ENDLESS RUNWAY has successfully demonstrated the feasibility of the circular runway project, highlighting benefits, future requirements and outlook. With the new Trump administration in the US looking at modernizing their airports will we see circular runways in America before Europe or will maybe the Chinese beat everyone to it…

Fabrizio Poli is Managing Partner of Aircraft Trading Company Tyrus Wings. He is also an accomplished Airline Transport Pilot having flown both private Jets and for the airlines. Fabrizio is also a bestselling author and inspirational speaker & has been featured on Russia Today (RT), TRT World, Social Media Examiner, Bloomberg, Channel 5, Chicago Tribune, Daily Telegraph, City Wealth Magazine, Billionaire.com, Wealth X, Financial Times, El Financiero and many other Media offering insight on the aviation world. Fabrizio is also regularly featured as an Aviation Analyst on Russia Today (RT) and TRT World. Fabrizio is also aviation special correspondent for luxury magazine, Most Fabullous Magazine. Fabrizio is also considered one of the world's top 30 experts in using Linkedin for business. You can tune in weekly to Fabrizio's business Podcast Living Outside the Cube available both in video & audio. You can also follow Fabrizio's aviation videos on Tyrus Wings TV. Fabrizio's latest book "Health4Flyers", the first natural-health book for pilots, flights attendants and frequent flyers is now available worldwide.

(Fabrizio Poli, March 16, 2017, https://www.linkedin.com/pulse/circular-runways-future-airports-fabrizio-poli)

Η αρχιτεκτονική της πίστης

Χτισμένες σε διαφορετικές εποχές, περιοχές και για διαφορετικά θρησκεύματα, έχουν ξεχωρίσει για την πρωτότυπη αισθητική τους και τις κατασκευαστικές τους λεπτομέρειες.

1. HallgrImskirkja, Ισλανδία

Βρίσκεται στο κέντρο του Ρέικιαβικ και αποτελεί ένα από τα διασημότερα τουριστικά αξιοθέατά του. Η λουθηρανική εκκλησία Hallgrímskirkja (που άρχισε να χτίζεται το 1945 και ολοκληρώθηκε έπειτα από δεκαετίες) θεωρείται εθνικό μνημείο, ενώ είναι αφιερωμένη στον κληρικό και ποιητή του 17ου αιώνα Hallgrímur Pétursson, από τον οποίο πήρε και το όνομά της. Πέρα από το μνημειώδες εξπρεσιονιστικό κτίσμα του αρχιτέκτονα Guojon Samuelsson (που είναι εμπνευσμένο από τα ηφαίστεια και τους παγετώνες του ισλανδικού τοπίου), εντυπωσιακό είναι και το τεράστιο εκκλησιαστικό όργανο με τους 5.275 αυλούς, στο εσωτερικό του ναού, όπως και η κολυμπήθρα από κρύσταλλο Βοημίας και βασάλτη (πέτρωμα από λάβα). Στην κορυφή της εκκλησίας, σε ύψος 73 μ., υπάρχει παρατηρητήριο που προσφέρει πανοραμική θέα στην πόλη.

2. Οχυρωμένη εκκλησία Viscri, Ρουμανία

Επτά χωριά της Τρανσυλβανίας που φημίζονται για τις οχυρωμένες εκκλησίες τους έχουν ανακηρυχθεί, στο σύνολό τους, Μνημεία Παγκόσμιας Κληρονομιάς της UNESCO. Ένα από αυτά τα χωριά είναι το Viscri, το οποίο στις μέρες μας κατοικείται κυρίως από Ρομά και έχει ως σήμα κατατεθέν την ομώνυμη οχυρωμένη γοτθική εκκλησία. Το πρώτο κτίσμα της Viscri θεωρείται ότι θεμελιώθηκε γύρω στο 1100 από τον ουγγρικό λαό των Szeklers, προτού εμφανιστούν οι Σάξονες κατακτητές. Η συγκεκριμένη γεωστρατηγική λεπτομέρεια είναι σημαντική και αρχιτεκτονικά, αφού είχε ως αποτέλεσμα η οροφή της εκκλησίας να χτιστεί ίσια και όχι θολωτά.

3. Βασιλική της Γεννήσεως, Ιερουσαλήμ

Είναι χτισμένη πάνω στο Grotto, το σημείο που έχει αναγνωριστεί από την παράδοση ως τόπος γέννησης του Ιησού. Εκτός από ιερός τόπος για χριστιανούς και μουσουλμάνους, η Βασιλική της Γεννήσεως –στη δυτική όχθη της Παλαιστίνης, 10 χλμ. νότια της Ιερουσαλήμ– είναι και ένα από τα Παγκόσμια Μνημεία της UNESCO. Η εκκλησία χρονολογείται τον 6ο αιώνα μ.Χ., θεμελιώθηκε πάνω στην αυθεντική βασιλική του 339 (την ανέγερση της οποίας ξεκίνησε η Αγία Ελένη, μητέρα του Μεγάλου Κωνσταντίνου) και είναι ο αρχαιότερος χριστιανικός ναός που λειτουργεί έως τις μέρες μας. Το σημερινό σύμπλεγμα περιλαμβάνει, επίσης, τον ναό της Αγίας Αικατερίνης, μαζί με μοναστήρια και ξωκλήσια.

4. Εκκλησία Borgund Stave, Νορβηγία

Χτίστηκε μεταξύ 1180 και 1250 και θεωρείται η πιο καλοδιατηρημένη από τις 28 εκκλησίες του είδους που έχουν απομείνει στη Νορβηγία, από τις χιλιάδες που υπήρχαν κάποτε. Πρόκειται για ξύλινες μεσαιωνικές εκκλησίες, γνωστές και ως stave churches, από τη νορβηγική λέξη «stav», που σημαίνει «ξύλινο υποστύλωμα». Η Εκκλησία Borgund Stave, που έχει σχεδιαστεί σε ρυθμό βασιλικής, δημιουργήθηκε από έμπειρους τεχνίτες, οι οποίοι επεξεργάστηκαν με επιδεξιότητα περί τα 2.000 κομμάτια ξύλου για να τη φτιάξουν, ενώ για την κατασκευή της δεν έχει χρησιμοποιηθεί ούτε ένα καρφί. Η εκκλησία έχει άλλη μία αρχιτεκτονική καινοτομία, καθώς μέχρι τότε τα υλικά κατασκευής ακουμπούσαν απευθείας στο έδαφος, ενώ στην Borgund φτιάχτηκε για πρώτη φορά μια πέτρινη βάση. Σήμερα ο χώρος λειτουργεί κατά κύριο λόγο ως μουσείο.

5. Notre Dame du Haut, Γαλλία

Στη διάρκεια της Γαλλικής Επανάστασης, 40 οικογένειες ντόπιων είχαν την ευκαιρία να αγοράσουν το παρεκκλήσι Notre Dame du Haut της κοινότητας Ronchamp, στην ανατολική Γαλλία, διότι δεν ήθελαν το θρησκευτικό σημείο αναφοράς της περιοχής τους να περάσει σε άλλα χέρια. Έπειτα από σοβαρά πλήγματα, που σχεδόν το διέλυσαν (όπως η φωτιά του 1913 και οι βομβαρδισμοί του Β΄ Παγκοσμίου Πολέμου), ξαναχτίστηκε τη δεκαετία του 1950 από τον Le Corbusier. Ο πρωτοπόρος Γαλλοελβετός, με κύριο υλικό του το σκυρόδεμα, δημιούργησε έναν μοναδικό ναό, ο οποίος άλλαξε τα δεδομένα της θρησκευτικής αρχιτεκτονικής όπως την ήξερε ο κόσμος. Το κτίσμα μοιάζει να ρέει συνδυάζοντας καμπύλες και ευθείες επιφάνειες, οι οποίες ορίζουν τον άξονα Ανατολής-Δύσης και εξασφαλίζουν –βασικό για τον Le Corbusier– φυσικό φωτισμό πολλές ώρες την ημέρα.

6. Εκκλησία της Μαρίας Μαγδαληνής, Ιερουσαλήμ

Χτίστηκε το 1888 από τον τσάρο Αλέξανδρο Γ΄ της Ρωσίας, στη μνήμη της μητέρας του, αυτοκράτειρας Μαρίας Αλεξάντροβνα. Βρίσκεται στο Όρος των Ελαιών, πάνω από τον Κήπο της Γεθσημανής, και είναι αφιερωμένη στη Μαρία Μαγδαληνή, μία από τις σημαντικότερες γυναικείες φιγούρες του χριστιανισμού, που λατρεύεται ως αγία από ορθόδοξους, καθολικούς, διαμαρτυρομένους αλλά και λουθηρανούς. Ο ναός ξεχωρίζει για τους απαράμιλλης ομορφιάς τρούλους του, ενώ το στυλ του καταδεικνύει τις τάσεις που επικρατούσαν στη μοσχοβίτικη αρχιτεκτονική κατά τον 16ο και 17ο αιώνα. Όσο για την πρόσοψή του, παρότι μοιάζει μαρμάρινη, στην πραγματικότητα είναι φτιαγμένη από ψαμμίτη.

7. Εκκλησία του Αγίου Γεωργίου, Αιθιοπία

Ανήκει στο θρησκευτικό σύμπλεγμα της Λαλιμπέλα στη βόρεια Αιθιοπία, το οποίο έχει χαρακτηριστεί «όγδοο θαύμα του κόσμου», είναι Μνημείο Παγκόσμιας Κληρονομιάς της UNESCO και περιλαμβάνει έντεκα μονολιθικές μεσαιωνικές εκκλησίες. Η εκκλησία του Αγίου Γεωργίου (Biete Ghiorgis), μία από τις πλέον διάσημες, είναι απομονωμένη και συνδέεται με τους υπόλοιπους ναούς με ένα σύστημα από τάφρους. Το υποβλητικό θρησκευτικό μνημείο χρονολογείται τον 12ο-13ο αιώνα, έχει σχήμα σταυρού και είναι κατασκευασμένο από τόφφο (ηφαιστειακό πέτρωμα).

(Ελευθερία Αλαβάνου / Η ΚΑΘΗΜΕΡΙΝΗ, 03.01.2017, http://www.kathimerini.gr/889599/gallery/ta3idia/me-aformh/h-arxitektonikh-ths-pisths)

8. Παρεκκλήσι Bosjes, κοιλάδα Bosjesman, Νότια Αφρική

Το παρεκκλήσι Bosjes σχεδιάστηκε από τον Νοτιοαφρικανό Coetzee Steyn του λονδρέζικου Steyn Studio και το ολοκλήρωσαν οι TV3 Architects, ενώ πρόκειται να λειτουργήσει τέλος Μαρτίου.

Μία ώρα μακριά από το Κέιπ Τάουν, τοποθετημένο μέσα στην κοιλάδα Bosjesman –γεμάτη με ελαιόδεντρα, ροδακινιές και αμπέλια–, η γλυπτική φόρμα του «ακολουθεί» τη σιλουέτα των βουνών με τους αμπελώνες που βρίσκονται στη δυτική πλευρά της πόλης.

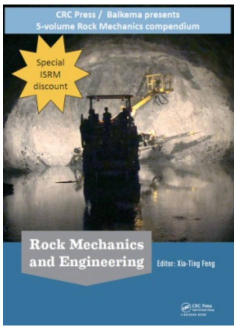
Η κατασκευή του έγινε από σκυρόδεμα, με την οροφή του να μοιάζει με ύφασμα που κυματίζει στον αέρα λίγο πριν ακουμπήσει στη γη.

Παρά την ανάλαφρη εικόνα του, έτσι όπως δείχνει να αψηφά τη βαρύτητα, η δομή του κτίσματος εκπέμπει δυναμισμό.

Ο χώρος εσωτερικά είναι πολύ λιτός, με πολλά δομικά στοιχεία κρυμμένα, για να μην παρεμβαίνουν στη δωρική εικόνα του. Η έμπνευση για το παρεκκλήσι πηγάζει από τους ιεραποστολικούς σταθμούς που ίδρυσε η αδελφότητα της Μοραβίας σε αυτή την περιοχή τον 19ο αιώνα. www.romaconventiongroup.it

(Παγώνα Λαψάτη / Η ΚΑΘΗΜΕΡΙΝΗ, http://www.kathimerini.gr/901972/article/ta3idia/ta3idiwtika-nea/eis-to-onoma-toy-patros)

ΝΕΕΣ ΕΚΔΟΣΕΙΣ ΣΤΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΠΙΣΤΗΜΕΣ



"Rock Mechanics and Engineering"

Prof. Xia-Ting Feng, Editor

The five-volume set "Comprehensive Rock Engineering", which was published in 1993, has had an important influence on the development of rock mechanics and rock engineering. Significant and extensive advances and achievements in these fields over the last 20 years now justify the publishing of a comparable, new compilation.

"Rock Mechanics and Engineering" represents a highly prestigious, multi-volume work edited by Professor Xia-Ting Feng, with the editorial advice of Professor John A. Hudson. This new compilation offers an extremely wide-ranging and comprehensive overview of the state-of-the-art in rock mechanics and rock engineering and is composed of peer-reviewed, dedicated contributions by all the key experts worldwide.

Key features of this set are that it provides a systematic, global summary of new developments in rock mechanics and rock engineering practices as well as looking ahead to future developments in the fields. Contributors are world-renowned experts in the fields of rock mechanics and rock engineering, though younger, talented researchers have also been included.

This multi-volume work sets a new standard for rock mechanics and engineering compendia and will be the go-to resource for all engineering professionals and academics involved in rock mechanics and engineering for years to come.

The "Rock Mechanics and Engineering" set includes the following 5 volumes:

1. Principles
2. Laboratory and Field Testing
3. Analysis, Modeling & Design
4. Excavation, Support and Monitoring
5. Surface and Underground Projects

CRC Press offers ISRM members a 30% discount. Click here to get the discount codes.

(CRC Press, June 12, 2017)

Shallow Geothermal Systems: Recommendations on Design, Construction, Operation and Monitoring

Deutsche Gesellschaft für Geotechnik e.V. / German Geotechnical Society, Deutsche Gesellschaft für Geowissen

The recommendations summarise the state of the art. Their aim is the proper exploitation of the ground for geothermal purposes without adversely affecting the ground or the groundwater on the one hand and the operation of the system and nearby buildings on the other. The recommendations should be used during consulting, design, installation and operation in order to achieve optimum and sustainable use of the ground at a specific location. Authorities responsible for supervising and approving projects can use the recommendations as a guide when taking decisions and making stipulations.

The Geothermal Energy Study Group was set up in Bochum in 2004 and became the joint DGGV/DGGT study group in 2007. Some 20 specialists from universities, authorities and engineering consultants are active in the group and meet two or three times a year.

(WILEY Ernst & Sohn, June 2016)

ΗΛΕΚΤΡΟΝΙΚΑ ΠΕΡΙΟΔΙΚΑ

www.isrm.net/adm/newsletter/ver_html.php?id_newsletter=136&ver=1

Κυκλοφόρησε το Τεύχος No. 37 του ISRM Newsletter (Μαρτίου 2017) με τα ακόλουθα περιεχόμενα:

- 17th ISRM online lecture by Prof. Charles Fairhurst
- 2017 ISRM International Symposium - AfriRock 2017 "Rock Mechanics for Africa"
- EUROCK 2017, Ostrava, Czech Republic, 20-22 June
- “Rock Mechanics and Engineering”, a 5-volume set edited by Prof. Xia-Ting Feng
- Macedonia joined the ISRM
- ISRM Rocha Medal 2019 - nominations to be received by 31 December 2017
- YSRM 2017 & NDRMGE 2017 – Challenges and Innovations in Rock Mechanics and Engineering, Jeju Island, South Korea, 10-13 May 2017, an ISRM Specialised Conference
- Progressive Rock Failure Conference, Monte Verità, Switzerland, 5-7 June 2017, an ISRM Specialised Conference
- GeoProc 2017, Paris, France, 5-7 July, an ISRM Specialised Conference
- Shaoxing International Forum on Rock Mechanics and Engineering Geology (SXFRG), Shaoxing, China, 28-29 October 2017, an ISRM Specialised Conference
- 51st US Rock Mechanics/Geomechanics Symposium - Registration and exhibits now open
- NRMS 2017 - 3rd Nordic Rock Mechanics Symposium, Helsinki, Finland, 11-12 October 2017
- The Spanish Society for Rock Mechanics celebrates the 50th anniversary
- First JTC1 Workshop on Advances in Landslide Understanding, Barcelona, 24-26 May 2017
- Digital Library at OnePetro
- ISRM Suggested Methods page
- ISRM Sponsored meetings

www.geoengineer.org

Κυκλοφόρησε το Τεύχος #143 του **Newsletter του Geoengineer.org** (Μαρτίου 2017) 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).

Ενδεικτικά αναφέρονται:

- Tunnelling beneath Stonehenge: A controversial construction project
- Malta's Azure Window collapses after storm
- Design for Bogotá's first metro line begins
- Rubbish landslide in Ethiopia leaves more than 100 people dead
- John Wolosick named Engineer of the year by the Georgia Society of Professional Engineers
- Impressive submarine landslide outcrop captured in photos
- $3bn Kuwait causeway project on track for launch in 2018
- Cowi employs drones for dam surveys in Zambia
- Stanford scientists develop software tool to reduce risk of manmade earthquakes
- Oklahoma faces seismic risk due to drilling activity

http://campaign.r20.constantcontact.com/render?m=1101304736672&ca=5b2b34e4-c25b-47a2-aa3e-a25e31e6b91f

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