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ΑΡΘΡΑ

Τα παρακάτω άρθρα αποτελούν συμμετοχές μελών της ΕΕΕΕΓΜ στο πρόσφατο XIV[th] European Conference on Soil Mechanics and Geotechnical Engineering που διεξήχθη στην Μαδρίτη (24 – 27 Σεπτεμβρίου – βλέπε προηγούμενο τεύχος ΝΕΩΝ).

Design and Construction Aspects of a Large Metro Station Cavern in Urban Environment

Plan et construction d'une caverne souterraine d'une station de métro au sein de l'espace urbain

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ABSTRACT

A large underground cavern, 21 m wide and 110 m long, with 9 meters of cover, has been mined to house an underground metro station for the Athens (Greece) Metro network. The station has been designed and constructed within one of the contracts for the extension of line 3 towards the western suburbs of the city. The large underground opening was mined sequentially following a 'central drift procedure' which proved to be a very efficient procedure in terms of settlement control as well as in terms of construction time and cost. The purpose of the present paper is to describe the design and construction process as well as to present the actual performance of the structure.

RÉSUMÉ

Une large caverne souterraine, large de 21 mètres et longue de 110 mètres, et de profondeur de 9 mètres, a été creusée pour accueillir une station du métro d'Athènes en Grèce. Cette station a été conçue et construite dans le cadre d'un des contrats pour l'extension de la ligne 3 vers la banlieue ouest d'Athènes. La caverne souterraine a été creusée progressivement selon le procédé de la *'galerie centrale'* qui s'est montré être un procédé très efficace pour limiter les tassements` ce procédé a été également efficace sur le plan des délais et du budget prévu. Cette communication présente le plan et la construction ainsi que la performance de cette méthode.

Keywords: Underground Cavern, Metro station, Ground settlements.

1 Introduction

The "Egaleo" station is the last station of a construction contract for the extension of line 3 of Athens (Greece) Metro system towards the western suburbs of the city. For functional reasons (track alignment) this station has been designed with a central platform and an increased track to track distance of 15.0 m. This requirement leads to the construction of a very wide underground cavern with a net excavation span of 21 meters. The large span and the small overburden (9 to 12 meters) made the design and construction of the cavern particularly challenging.

The underground part of the station is located directly underneath a main street of the suburb, which carries heavy traffic required to remain uninterrupted by the construction works. For that reason although a cut-and-cover method of construction was feasible due to the low overburden, an underground mining method was selected for the construction.

Commercial and residential buildings situated in the vicinity of the station are exposed to settlement damage risks induced by the tunneling works. Strict limits to surface ground settlements and angular distortions have been set to control those risks.

A general layout of the station complex is presented in figure 1 and a cross section of the station cavern is presented in figure 2.

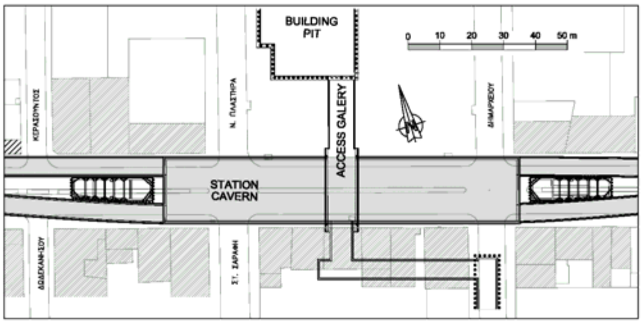

Figure 1. General layout of the station complex. Shaded areas represent buildings.

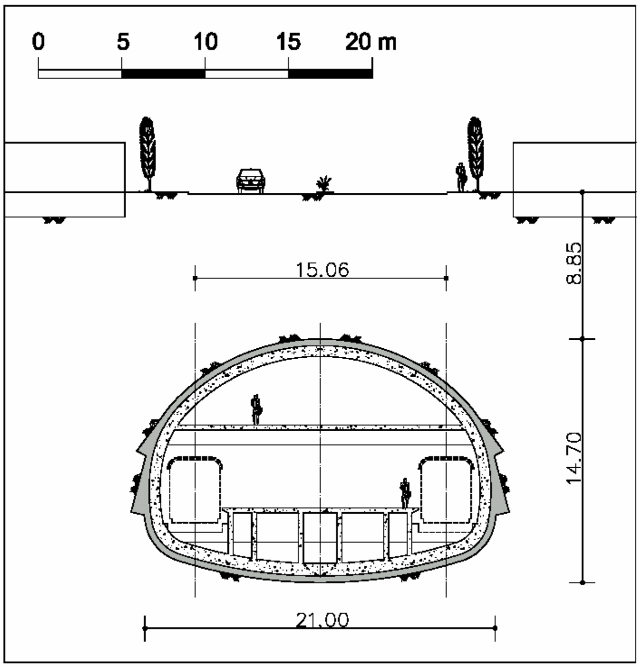

Figure 2. Cross section of the Egaleo station cavern

2 Local geology and geotechnical conditions

2.1 Local geology

The underground part of the station is located within the Athens Schist formation, which covers most of the area of the Athens basin and has been encountered during most of the tunneling works for the previously constructed part of the Athens Metro (Kavvadas et al. 1996, 1999). Athens schist, according to Koukis & Sabatakakis (1999), is a sequence of upper cretaceous flysch-type meta-sediments, which have undergone low grade metamorphism. The basic units of the Athens Schist are meta-sandstones interbeded with meta-siltstones, and in some localities black shales and limestones. In places ophiolitic bodies are also found within this formation. The Athens Schist formation has been subjected to intense folding and thrusting during the Eocene and subsequently subjected to extensive faulting and fracturing. This tectonic procedure is responsible for the very complex geologic structure encountered during tunneling works.

From the engineering point of view, the formation presents frequent changes of lithological facies at sort distances (at the scale of engineering structures) as well as an irregular alteration and weathering pattern. Consequently, the character of the individual facies in terms of strength varies from hard rock to stiff soil.

Figure 3. Polished slickensided surfaces of the meta-siltstone unit of the Athens Schist formation

2.2. Geotechnical assumptions and design parameters

In the vicinity of the station cavern, the Athens schist formation is below a 3-4 meter thick layer of recent fill. The quality of the rock mass was not possible to determine in advance for the entire length of the station cavern, solely on the basis of a small number of exploratory boreholes, given the natural heterogeneity of the geologic formation and its tendency to change abruptly in small distances due to the presence of structural features of tectonic origin (faults e.t.c.). For this reason three design ground profiles were foreseen, namely a good (rock C), medium (rock D) and poor (rock E) quality rock mass, covering the extremes of the expected ground conditions. Since the tunneling procedure could not adjust during construction to the ground conditions encountered at tunnel face, due to the staged excavation of the cavern, the design had to be conservative enough, to cope with the most adverse scenario. However the more optimistic scenarios were also studied in order to bracket the expected performance of the tunneling procedure.

The GSI system (Hoek et al., 1998) was used as a tool to characterize the rock mass during tunneling and the Hoek & Brown failure criterion was used to derive strength and deformability parameters for each rock mass class on the basis of the procedure established by Hoek & Brown (1997). The design values for the three rock mass classes were derived by Kavvadas (2003) and are summarized in table 1.

Table 1. Rock mass classes and respective design parameters

	GSI	E MPa	v	c kPa	φ °
Class C	25-35	750	0.30	110	35
Class D	20-25	500	0.30	90	31
Class E	15-20	300	0.30	70	29

3. Construction method and sequence

3.1. Design and construction issues

In variable and strongly heterogeneous poor quality rock masses, like the Athens Schist, the division of the cavern section to smaller headings is essential for the effective control of face stability. Block falls and wedge sliding along slikesides, joints, or weak zones, is a constant hazard during tunneling in Athens schist. Ground movements are also controlled better reducing the area of the advancing face, since strains occurring ahead of the face contribute in many cases significantly to the final surface settlements.

For very large sections (in our case the total face area of the station cavern is 254 m^2) the tunneling of the cavern section in stages is necessary to reduce the quantities of excavated material, reinforcement and shotcrete placing, per round. Limiting excavation mucking and support time allows earlier support and reduces face instability risks especially when the stand-up time is small.

In the first place it was decided to excavate and support the cavern from top to bottom and its section was subdivided in top heading, bench and invert. Moreover, the sequential excavation of the top heading was also deemed necessary considering its large span and the low overburden of the cavern. The procedure which was employed for the excavation of the top heading, foresees the excavation of a central drift with vertical side walls reinforced with heavy steel beams (HEB sections). Consequently the side drifts are mined, and the partition walls are retained acting as columns and stiffeners of the crown vault. When all the drifts are excavated, the partition walls or pillars are gradually removed. During this critical stage a series of full height "windows" are opened on the partition walls and a final layer of steel mesh reinforced shotcrete is applied along the entire length of the crown area. This continuous layer of shotcrete reduces the risk of defective connections of the shotcrete shell. In the final stage of the top heading construction the remaining pillars are removed and a final layer of mesh reinforced shotcrete is applied at the remaining zones. The procedure (also presented in figure 4) is well suited only for rock like formations where the tunnelling of a central drift with vertical sidewalls is feasible. Large bending moments in the vertical sidewalls might develop in a soft material making this method less attractive.

The excavation of a central drift, along the entire length of the station, provides the opportunity to drain the surrounding rock mass and minimize the problems associated with water inflow, as well as to pre-reinforce the surrounding rock mass with fiberglass nails. It acts also as a pilot drift permitting a reconnaissance of the geologic conditions along the entire length of the underground structure and gives an indication of the settlements that the tunneling works might produce. Of course significant deviations from predicted settlement values at this stage can lead to additional support measures and/or modifications of the tunneling and support method.

4. Numerical Analysis

The analysis of the tunneling procedure has been undertaken by means of two and three dimensional finite difference models, using the code FLAC, and FLAC 3D. The Mohr-Coulomb constitutive model was used for the rock material. The placement of shotcrete in layers as well as its progressive hardening was taken into account in the calculations by changing the respective properties at each construction stage. After a series of preliminary analyses the shotcrete shell was decided to be 40 cm thick and reinforced with lattice girders and steel wire mesh. A pattern of passive rock bolts was also incorporated in the design and considered in the analyses.

4.1. Two dimensional analyses

A basic set of two dimensional analyses for the three rock mass classes (representing three distinct scenarios) was performed. The calculated maximum surface settlements were within acceptable limits reaching 11.0mm, 16.0 mm and 30.0 mm for rock mass classes C, D and E respectively. The angular distortions were also well controlled by the excavation procedure adopted. Interestingly enough, it was observed that an abrupt jump to the evolution of the surface settlements occurs when the partitioning walls (or pillars) are removed. This action reduces strongly the stiffness of the support system and leads directly to settlements. However the fact that the removal of the pillars takes place in a sequential manner and under controlled conditions (full and easy access to construction equipment at this construction stage) it was considered as an advantage of the method. In reality creep and relaxation of the shotcrete shell and the surrounding rock mass allows some redistribution of stresses before the removal of the pillars making the effect less prominent.

Table 2. Mean and standard deviation of rock mass properties considered in the analyses.

	E MPa	v	c kPa	φ °
Mean value	500	0.30	90	31
Standard Deviation	±100	0	±10	±1

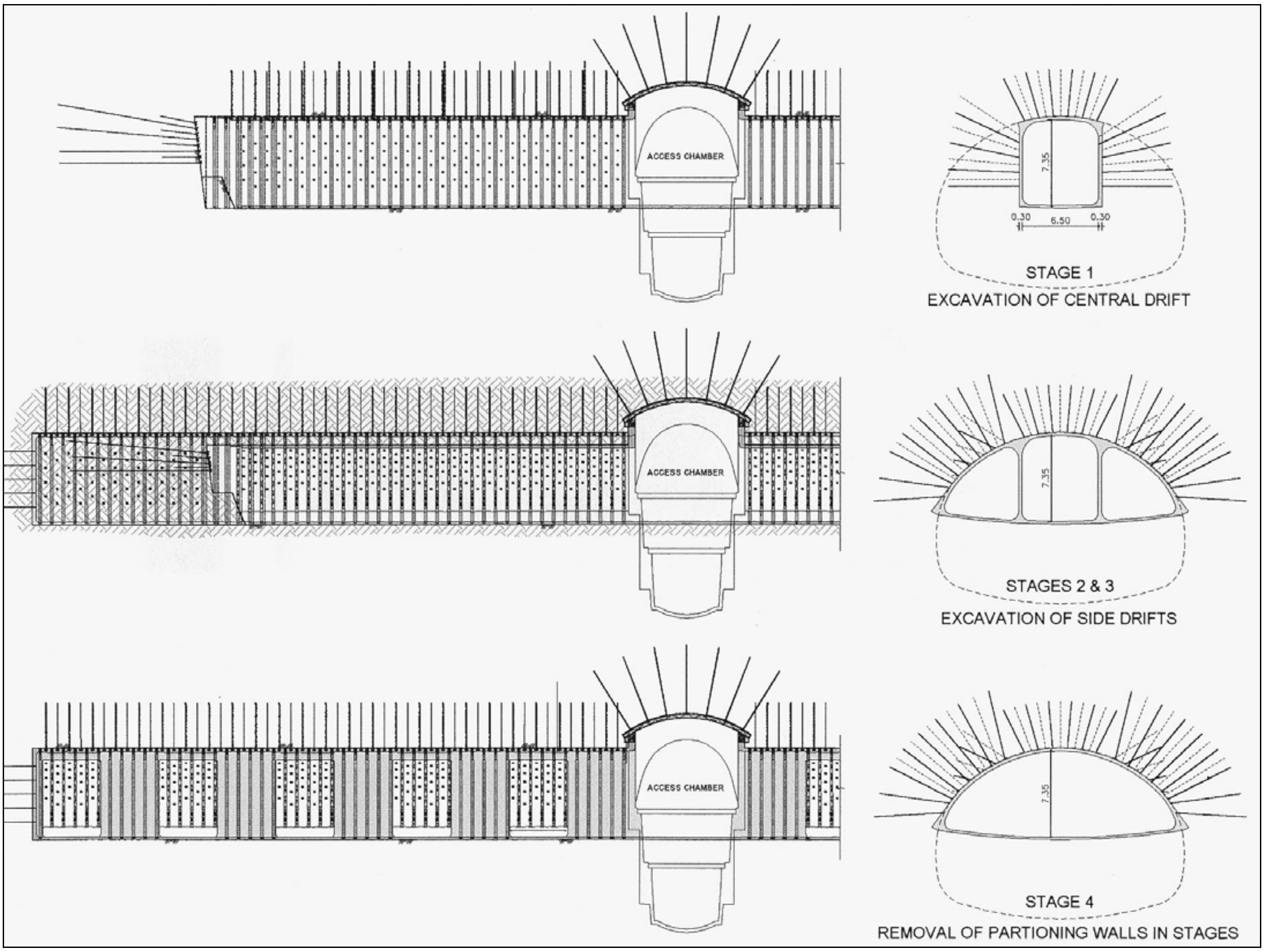

Figure 4. Top heading construction sequence of the station cavern. Notice the opening of 'windows' in the third construction stage and the gradual removal of the pillars.

As mentioned in paragraph 2, one of the dominant characteristics of Athens schist is the abrupt change of rock mass quality due to lithological variations, structural features and weathered zones. Localized weak zones may have a strong effect on shotcrete shell distress and surface settlements. In order to investigate their effect a second set of two dimensional finite difference analyses with randomly varied rock mass properties was performed. In these models strength and deformability properties were varied according to a Gaussian (Normal) distribution. In order to produce grid independent patterns consistent with the geologic structure of the Athens schist, the properties were varied in a part random part periodic fashion. A sub-horizontal pattern an inclined pattern and a sub-vertical pattern, presented in figure 5, were chosen as representative of the possible structural forms of the geologic formation. The strength and deformability properties used in those analyses are summarized in table 2 (mean values are those of class D, GSI=20-25).

The results of the three different random patterns are presented in figures 5. Shear strains and plastic zone follows, as expected, the weaker zones. The more vertical is the pattern, the easiest it is for the plastic zones to reach the ground surface with a direct impact to surface settlements and mainly to angular distortions. Settlement troughs and angular distortions are compared in figure 6 where reference curves for class D and class E are also shown for comparison. The sub-vertical pattern, which is the worst case, lead to settlements not higher than rock class E (worst anticipated scenario), while angular distortions are increased in comparison with those derived by the assumption of homogeneous strata.

4.2. Three dimensional analyses

A two dimensional analysis does not model explicitly the response of the ground ahead of the advancing heading, neither does it give any indication on ground yielding ahead of the tunnel face. Two dimensional analyses neglect the contribution of settlements due to face compliance and account for them indirectly through the first deconfinement stages. A three dimensional model was set up and analyzed with FLAC 3D, for the employed method of staged construction, in order to check the validity of the two dimensional analyses on which the actual design has been based. The finite difference mesh and snapshots of the construction sequence considered are reproduced in figure 7.

The resulting surface settlements showed a remarkable agreement with those derived by the two dimensional analysis for the same ground strength and deformability parameters. The evolution of the vertical displacements calculated by the two and three dimensional analyses for class E (the most adverse scenario) are compared in figure 8. Shotcrete shell axial forces and bending moments were also in general agreement. The pattern of the vertical displacements induced by tunneling and calculated by the three dimensional analysis is presented in fig. 9.

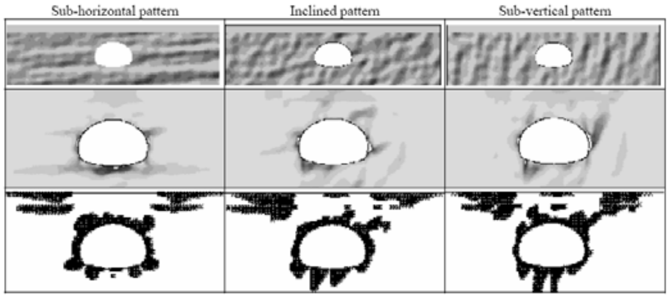

Figure 5. Variation of rock mass properties (upper row) accumulated shear strains (middle row) and plasticity indicators (lower row) for the three patterns examined.

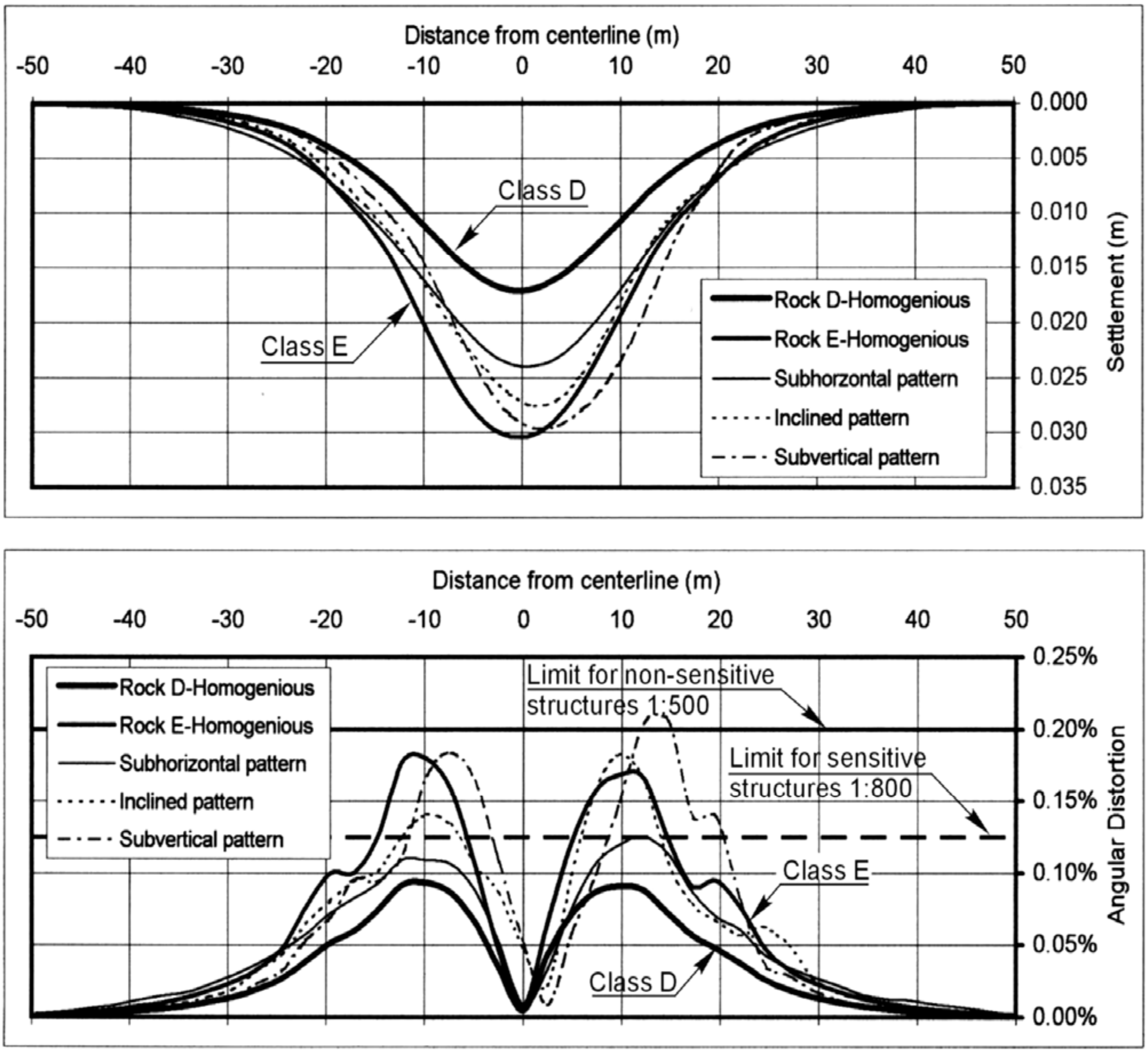

Figure 6. Vertical Displacements and Angular Distortions at the ground surface for the three variability patterns examined. Settlement troughs for classes D and E are also shown for comparison

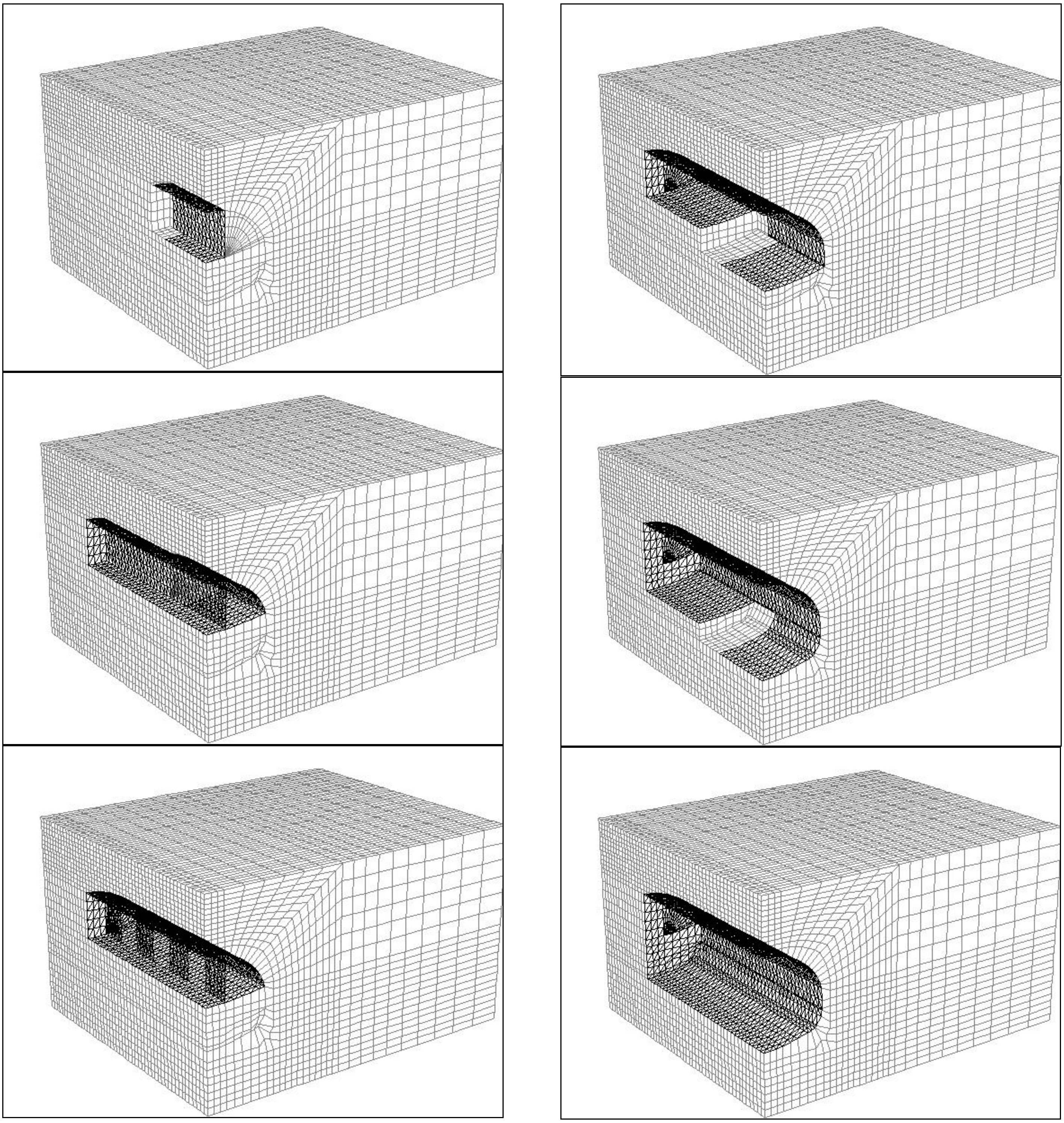

Figure 7. Three dimensional model of the station cavern

The good agreement of the two models (2D and 3D) can be attributed to the fact that for the given set of strength parameters (even in the worst case scenario), ground yielding does not play the dominant role in terms of surface settlements and the induced displacements depend mainly on the ground material and shotcrete shell stiffness. In such cases with a reasonable selection of the deconfinement ratios for each support stage, a two dimensional model is likely to lead to satisfactory results. Of course if softer ground material is present, a three dimensional analysis might be necessary to investigate the effect of soil yielding ahead of the advancing face.

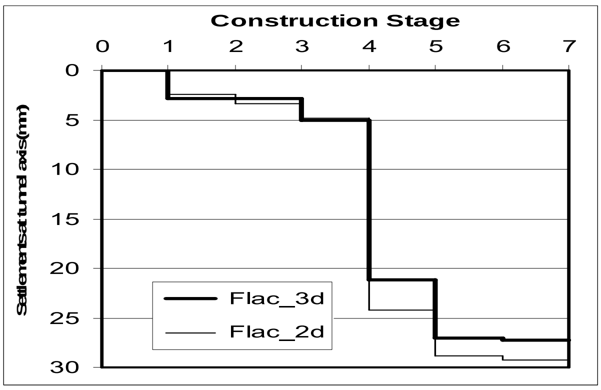

Figure 8. Comparison of the evolution of maximum surface settlement calculated by means of the two and three dimensional model (Rock E - GSI 15-20)

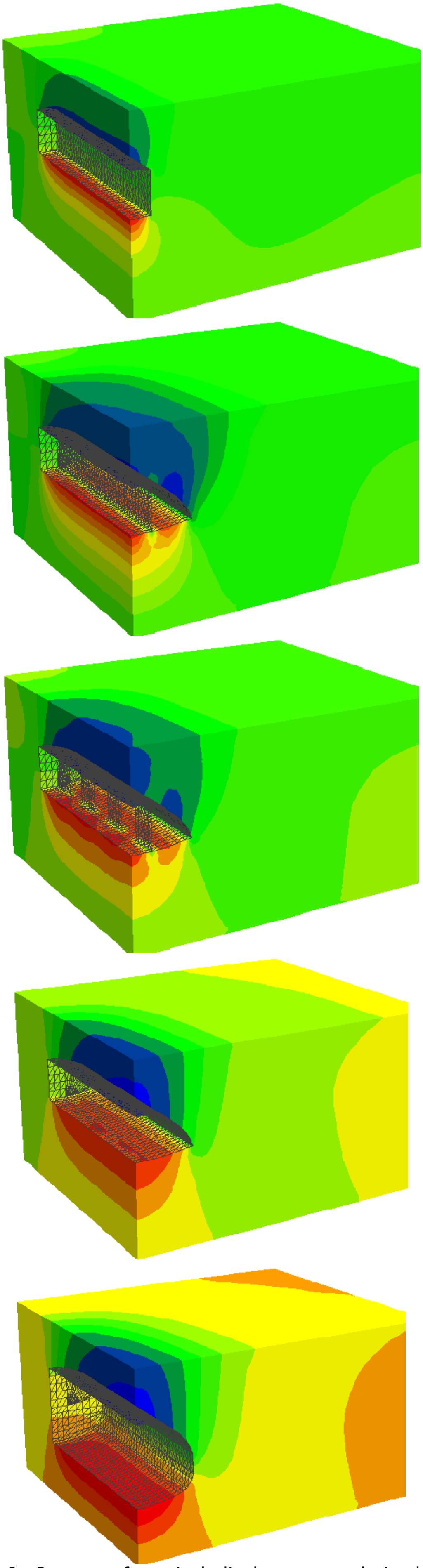

Figure 9. Pattern of vertical displacements derived by a three dimensional analysis. (Rock mass class C - GSI 25-35)

Beginning the excavation of the central drift

Tunneling the central drift

Tunneling the side drifts

Opening of "windows" and application of shotcrete

Complete removal of pillars

Benching

Figure 10. Tunnelling the station cavern in stages.

5. Construction and performance

The construction proceeded without significant problems and surprises and it was found that the design was detailed in such a way to avoid constructability problems. The progressive construction of the station cavern is presented in the photographs of figure 11.

The adopted tunneling procedure, allowed high advance rates and permitted the completion of the tunneling works of the station ahead of schedule. The tunneling started from the access chamber which was located at the middle of the cavern (see also figure 1) and advanced subsequently towards both ends of the cavern. The advance rates achieved at each construction stage are presented in figure 11.

The encountered Athens Schist was composed by meta-sandstone/meta-siltstone alternations, together with lenses or irregular bodies of meta-sandstone (figure 12). Meta-sandstone/meta-siltstone alternations exhibit a well developed, anastomosing foliation which often appears slickensided. Foliation generally dips with low angles towards the east, yet its direction varies strongly in short distances due to extensive folding of the whole formation. Numerous small to large scale, low angle shear zones and high angle fault zones crosscut the whole formation. Fault gouge and cataclastic zones of a few centimeters thick generally mark the shear and fault zones.

The blocks of meta-sandstone exhibit poorly developed foliation and are characterized as massive. Closely to moderately spaced joints characterize the structure and deformation of the meta-sandstone blocks. The GSI values ranges from 25 to 35 for the meta-sandstone/meta-siltstone alternations, whereas for the more competent meta-sandstone blocks, the GSI values ranges from 30 to 45.

In terms of ground water conditions it was found that the broader area of the project is characterized by low capacity perched aquifers, developed almost entirely in the more permeable meta-sandstone bodies, which were drained soon after the tunneling of the central gallery. The water was flowing through fractures (faults, shear zones, joints) within the meta-sandstone bodies and groundwater conditions within the tunnel were characterized as ‘dripping’ or ‘dump’ and rarely ‘wet’.

During excavation at the various stages, the tunnel face was generally stable and no face buttress was needed. A few small-scale over breaks (max $1m^3$ of volume) were recorded and were all structurally controlled.

The surface settlements were very effectively controlled by the support measures foreseen by the design. Surface settlement contours after the excavation of the top heading and after excavation of the complete section are presented in figure 13.

During construction, the continuously monitored surface settlements were compared with the predicted figures of the numerical analyses (2D or 3D), in order to assess the safety of the underground opening and of the buildings located at ground surface.

In figure 14 the results of the three dimensional analysis for the most relevant numerical model (rock class C), and for four characteristic construction stages, are presented (dashed lines) and are compared with the average settlement troughs induced by the actual tunneling works. The comparison shows that the numerical model provided reasonable predictions for the final stages but less satisfactory for the first ones. One possible explanation is that some dewatering settlements that eventually took place during the first excavation stages and which has not been considered explicitly by the numerical analysis, increased the actual settlements observed during the first construction stages. Solid lines in figure 14 represent corrected settlements where a Gaussian curve representing the dewatering settlements has been added to the calculated curves.

6. Conclusions

The central drift method proved to be a very efficient method of a large cavern staged excavation in urban environment, where settlement control and reliability were of outmost importance, while construction time and cost had also to be optimized. It is recognized that the application of the particular method is limited to rock-like formations and it should be avoided if more plastic materials are present. However it suited very well to the actual conditions of the particular project leading to a very efficient construction. Numerical analysis proved to be a valuable tool to verify the procedure and to dimension the support system while during construction provided a basis to assess the safety and performance of the tunneling works.

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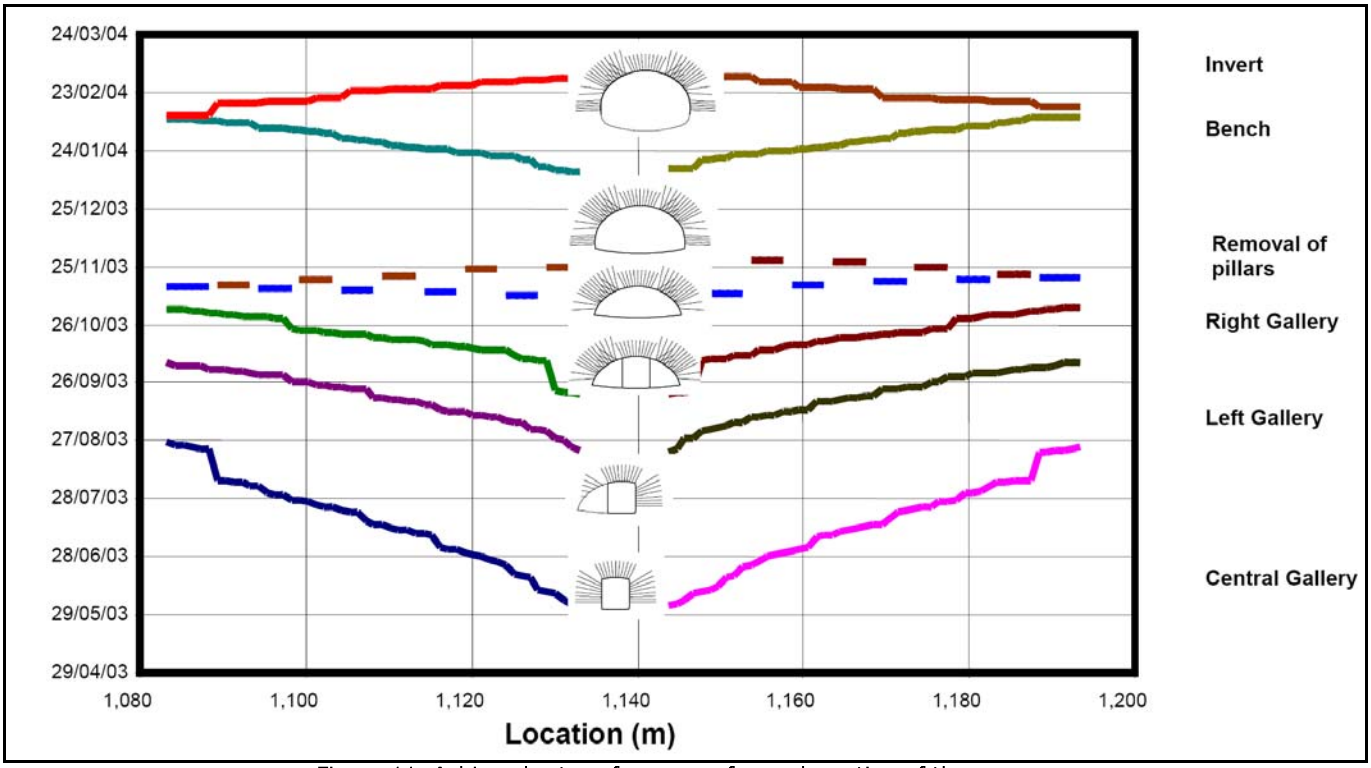


Figure 11. Achieved rates of progress for each section of the cavern.

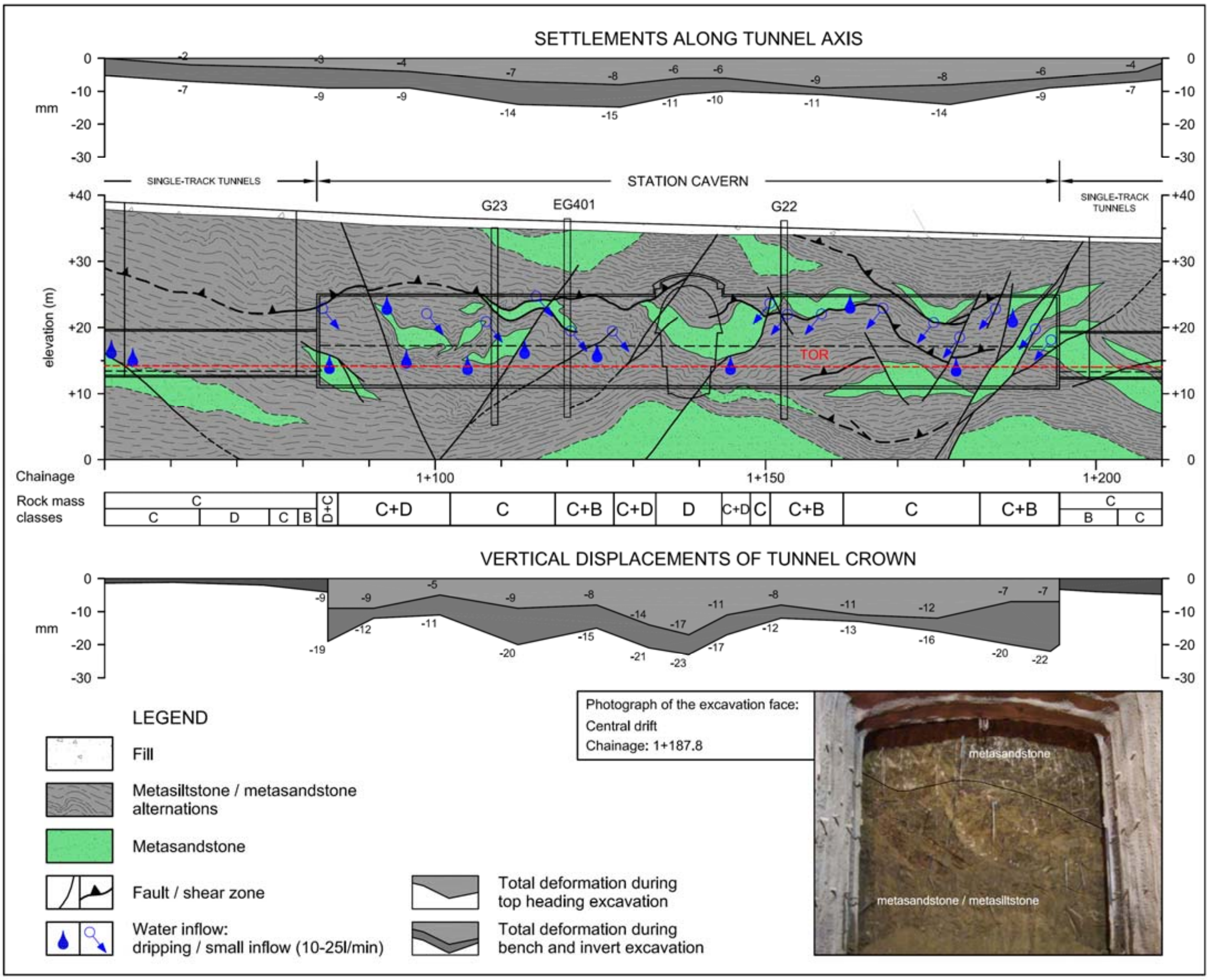
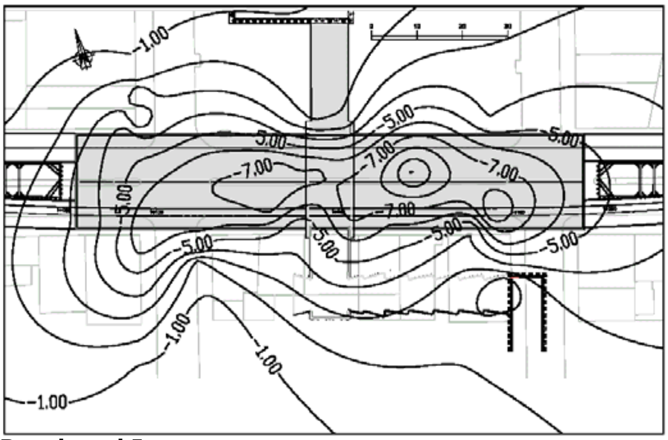


Figure 12. Encountered geology along the station cavern. A summary of convergence measurements (tunnel crown vertical displacement) and maximum values of surface settlements are also shown.

Top Heading



Bench and Invert

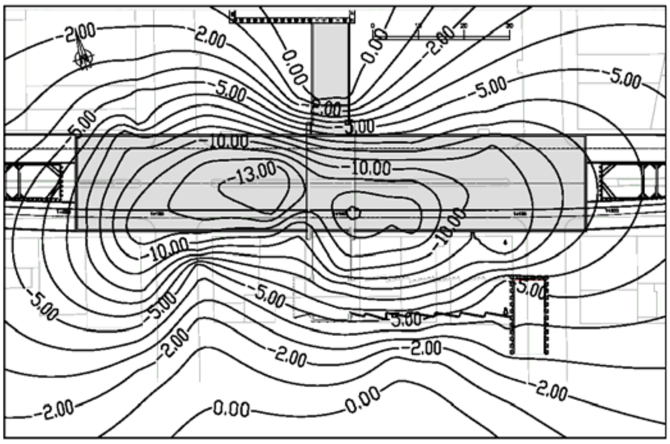


Figure 13 Settlement contours after excavation of the top heading (upper figure) and after the excavation of the complete section (lower figure).

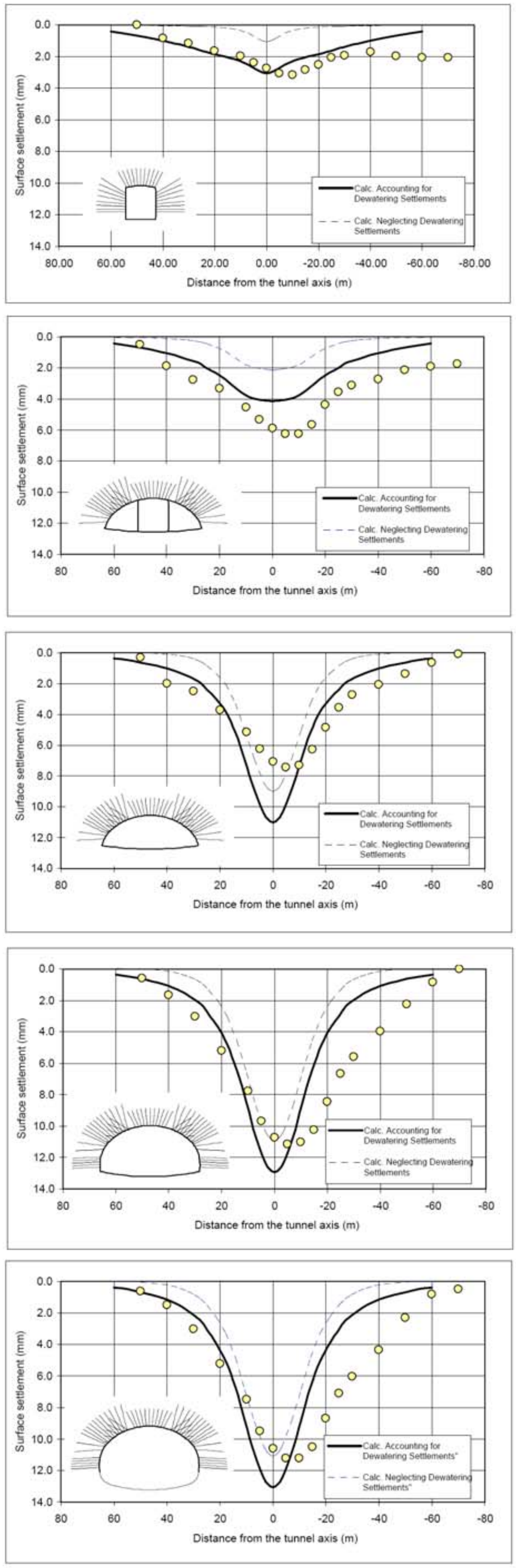


Figure 14 A comparison of the calculated settlement troughs (FLAC 3D-Rock class C) with the observed settlements. Dashed lines are numerical predictions which neglect the effect of dewatering. Solid lines account for some dewatering settlements calculated from the excess settlements of the first construction stage.

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Compatibility of EC-8 ground types and site effects with 1D seismic wave propagation theory

Compatibilité entre les types de sol et les effets de site selon le EC-8 et la théorie de propagation 1D des ondes sismiques

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ABSTRACT

This paper comments on the EC-8 provisions for site effects on seismic actions, from the point of view of non-linear, 1D seismic wave propagation theory. For this purpose, a large number of parametric analyses are performed using a set of recently proposed multi-variable relations, which approximately reproduce the results of 1D wave propagation analysis via the equivalent linear method. By comparing the code provisions to the theoretical estimates, a number of easy to implement and mostly quantitative potential modifications of EC-8 are identified that refer to the ground type categorization and the horizontal elastic response spectra specified by the code. These modifications do not alter the basic line of thought of the code, but enhance the compatibility between its provisions and commonly performed numerical analyses of seismic soil response.

RÉSUMÉ

Cet article présente des remarques sur les provisions du EC-8 pour les effets de site sur les actions sismiques, du point de vue de la théorie de propagation non linéaire 1D des ondes sismiques. Pour cette raison, on a exécuté un grand nombre d'analyses paramétriques en utilisant un groupe de relations multi-variables qui ont été récemment proposées et qui reproduisent approximativement les résultats d'analyses de propagation 1D des ondes sismiques performées en utilisant la méthode linéaire équivalente. Après la comparaison des provisions du code avec les estimations théorétiques, on propose un nombre des modifications potentielles du code EC-8, faciles à établir et quantitatives en majorité, qui concernent la catégorisation des types de sol et les spectres horizontales de réponse élastique spécifiés par le code. Ces modifications ne changent pas la philosophie du code, mais au contraire elles augmentent la compatibilité entre ses provisions et les analyses numériques de la réponse sismique du sol exécutées habituellement.

Keywords: earthquake, site effects, elastic response spectra, EC-8

1. Introduction

The EC-8 divides soil sites into seven (7) "*Ground Types*", five (5) of which (i.e. A, B, C, D and E) have prescribed seismic actions and the remaining two (S1 and S2) require the execution of a special detailed study. The former five (5) ground types are identified in terms of their stratigraphic description, the approximate thickness H of recent soil deposits and the average values of the SPT blow count N_{SPT}, the undrained shear strength c_u and the shear wave velocity $V_{S,30}$ in the top 30m of the ground. In practice, the identification of the ground type is mainly based on H and $V_{S,30}$, the latter being computed irrespective of whether the top 30m include the bedrock or not. Furthermore, each ground type is assigned a different "*Soil Factor*" (S), that modifies the "*Design Ground Acceleration*" (a_g) relative to that of the basic (bedrock) ground type (A), and a different (normalized) horizontal elastic response spectrum $S_e(T)$, where T denotes the structural period. Soil factors S and (normalized) elastic response spectra (hereby denoted as "*NERS*") are defined separately for areas of high and low seismicity, i.e. areas where the design earthquake has a (surface-wave) magnitude $M > 5.5$ and $M < 5.5$ respectively. The former set of S and *NERS* are denoted as Type 1 spectra in EC-8, while the latter as Type 2.

The above description of local soil effects is broadly approximate, but quite rational, as it accounts directly or indirectly for two basic factors which are known, from theory as well as from field evidence, to control the seismic soil response: the dynamic characteristics of the site (through H and $V_{S,30}$) and the seismic excitation characteristics (through the earthquake magnitude M). There is no doubt that, using the non-linear site period T_s, the predominant excitation period T_e and the peak seismic acceleration a_{max} and/or velocity v_{max} to define seismic actions would be a much more rigorous approach. Nevertheless, at the present state of geotechnical and seismological engineering practice, these parameters are not readily available to non-expert users of the code and consequently their use would have an overall negative effect.

For the foregoing reasons, it appears reasonable at the moment to preserve the main line of thought of EC-8 and to focus upon potential modifications which aim to improve its quantitative agreement with well accepted theoretical and field evidence. Hence, in the following, the EC-8 provisions for site effects are examined from a theoretical point of view. Namely, a large number of parametric analyses are performed to answer the following questions:

(a) Do the EC-8 ground types cover adequately and uniquely all soil conditions which are often encountered in practice?

(b) Are the EC-8 soil factors S and *NERS* consistent with the corresponding ground types and seismicity levels?

2. Outline of methodology

The foregoing parametric analyses are performed with a set of theory-based multi-variable relations proposed by Bouckovalas & Papadimitriou (2003). These relations have been shown to approximately reproduce the results of the equivalent linear method (Shake 91, Idriss and Sun 1992), i.e. provide a standard deviation of error up to ±26%. The use of these relations was preferred over actual numerical analyses for purely practical purposes, since their use facilitated greatly the study of all basic problem parameters in over 1000 cases of soil-bedrock-excitation combinations. Furthermore note that these multi-variable relations have already been used successfully in a GIS-aided seismic microzonation study, as a user-friendly accurate alternative of the equivalent linear method (Papadimitriou et al 2004).

To answer questions (a) and (b) of the introduction, the EC-8 code provisions had to be recast in terms of the parameters entering the foregoing multi-variable relations. In particular, these multi-variable relations are based on the definition of the elastic soil period $T_{s,o}$ which in turn requires the definition of the (elastic) shear wave velocity V_S profile of the soil column down to a depth H where lies a uniform bedrock with shear wave velocity V_b.

Hence, there was a need for interrelating this V_S profile to the $V_{S,30}$ value of EC-8 and this was performed via $V_{S,el}$, i.e. the average shear wave velocity of the soil column, which is related to $V_{S,30}$ as:

$$V_{S,el} = \begin{cases} \dfrac{H}{\dfrac{30}{V_{S,30}} - \dfrac{30-H}{V_b}} & ; \ H < 30m \\ V_{S,30}\left(\dfrac{H}{30}\right)^a & ; \ H \geq 30m \end{cases} \quad (1)$$

where a is the power of depth z (in m) in the assumed increasing $V_S = V_{S,30}(z/30)^a$ relation (e.g. a=0 leads to a uniform profile). The value of V_b was varied from 800m/s up to 1200m/s in the analyses, keeping in mind that the underlying bedrock is rarely an extremely stiff rock with $V_b >>$ 1200m/s.

Furthermore, the EC-8 provides different soil factors *S* and normalized elastic response spectra *NERS* on the basis of the (surface-wave) magnitude *M* of the design earthquake (Type 1: $M > 5.5$ and Type 2: $M < 5.5$). Hence, a distinction between strong and weak seismic motion is also made in the parametric analyses. In the multi-variable relations, this distinction is made in terms of the parameters quantifying the design acceleration time history at the outcropping bedrock, namely: a) its peak value a^b_{max}, b) its predominant period T_e and c) the number of equivalent uniform cycles *n* quantifying its duration. Thus, based on experience and being conservative in our estimates, the following ranges of values of Table 1 were adopted in the analyses.

Table 1: Quantification of weak ($M < 5.5$) and strong ($M > 5.5$) seismic motion

Parameter	$M < 5.5$	$M > 5.5$
a^b_{max} (g)	0.1 – 0.2	0.2 – 0.5
T_e (s)	0.1 – 0.25	0.2 – 0.4
n	3 – 6	4 – 8

In the sequel, parametric analyses were performed in order to establish the variation of the soil factor *S* as a function of *H* and $V_{S,30}$ or equivalently of *H* and $V_{S,el}$. The variation is studied separately for strong and weak seismic motion and in terms of its average value over the whole range of a^b_{max}, T_e and *n* outlined in Table 1. This procedure enables commentary on the rationality of the ground type definition of the EC-8 and furthermore it allows for a separate estimation of design values for the soil factor *S* for all ground types and both earthquake magnitude ranges (Types 1 and 2).

Similarly, parametric analyses were performed in order to establish the range of variation of the horizontal elastic response spectrum $S_e(T)$ for the various ground types and earthquake magnitudes ranges (Types 1 and 2). Having studied separately the variation of the soil factor *S*, the emphasis was put on the amplification ratios $A_{Sa}*$ of the normalized (horizontal) elastic response spectra (*NERS*), which are defined as

$$A_{Sa}* = \frac{NERS \text{ for Ground Types A, B, C, D, E}}{NERS \text{ for Ground Type A}} \quad (2)$$

The code provisioned values of $A_{Sa}*$ are estimated on the basis of their definition in the EC-8 and are different for Types 1 and 2. These values are then compared to the values of $A_{Sa}*$ resulting as average (± standard deviation) over the whole range of a^b_{max}, T_e and *n* (outlined in Table 1) for Types 1 and 2 respectively.

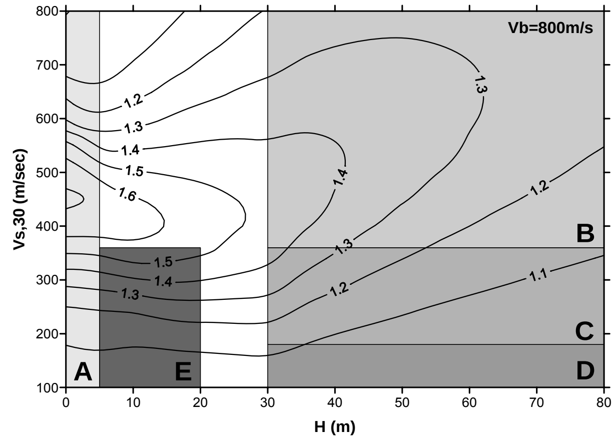
3. Site effects on design ground acceleration

Figure 1 shows contours of computed soil factors, in terms of soil thickness (*H*) and average shear wave velocity ($V_{S,30}$), indicatively for soft bedrock (V_b = *800m/s*). Similar analyses were performed for hard bedrock (V_b = *1200m/s*) conditions, but are not shown here for brevity. In addition, Figure 2 shows the range (average ± std. deviation) of computed *S* values for each ground type, and compares it to the EC-8 soil factors. In Figure 1, we have included the *H*-$V_{S,30}$ range of EC-8 for the proposed ground types A to E. In doing so the expression "*several tens*" used in the definition of ground types was interpreted as "*depth larger than 30m*". Moreover, note that parametric analyses were not performed for cases with $H > 80$m, since, in our opinion, such deep profiles should require a special study.

In Figures 1a and 2a in particular, that pertain to areas with $M > 5.5$ (Type 1), there are a number of noteworthy observations to be made, such as:

- The definition of ground types is not complete, as sites with $V_{S,30} > 360$ m/s and H = 5 – 30m , as well as sites with $V_{S,30} < 360$ m/s and H = 20 – 30m do not seem to belong to any ground type.
- Computed soil factors for ground type A are much higher than the proposed reference value of *S* = 1.00.
- Except for ground type C, the EC-8 soil factors are not in agreement with theoretical predictions. The most remarkable difference is observed for ground type D (deep and soft soil sites) where EC-8 proposes *S* = 1.35, as compared to computed values which range systematically between 0.95 and 1.10.

Similar observations are made in Figures 1b and 2b, which evaluate the EC-8 proposed soil factors for low seismicity areas ($M < 5.5$, Type 2), only that now differences are much larger and concern almost all ground types.



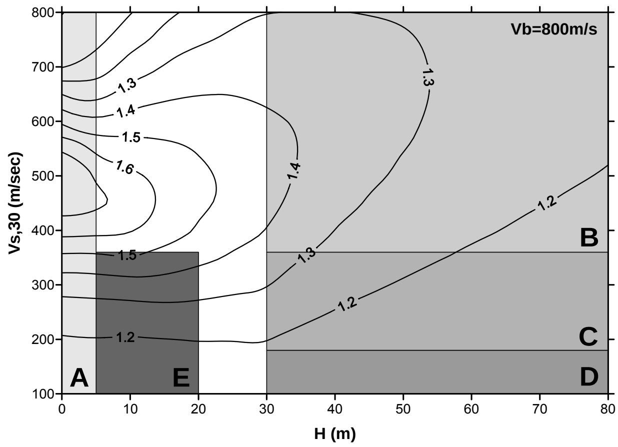


Figure 1. Ground type categorization on the basis of EC-8, as a function of *H* and $V_{S,30}$, and contours of average soil factor *S* variation from 1-D analyses with V_b = *800m/s* (indicatively): a) Type 1 ($M > 5.5$), b) Type 2 ($M < 5.5$)

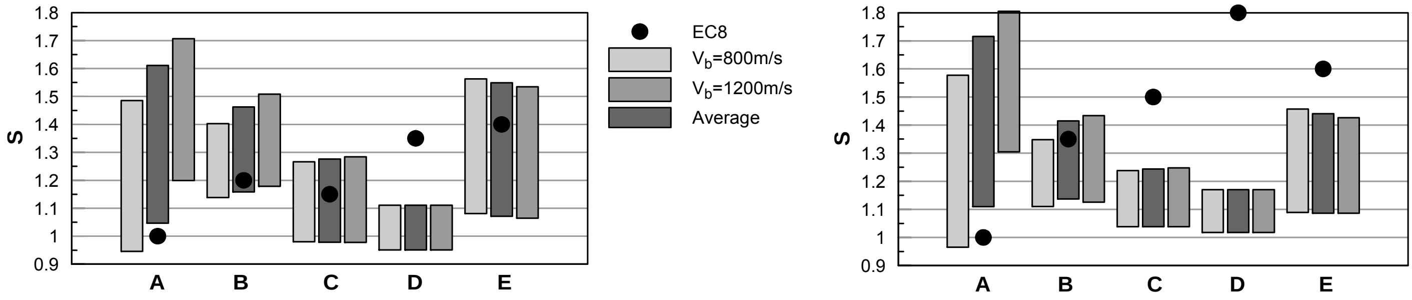


Figure 2. Soil factors S per ground type on the basis of EC-8, as a function of H and $V_{s,30}$, versus respective range of variation from all 1-D analyses: a) Type 1 ($M > 5.5$), b) Type 2 ($M < 5.5$)

As a *first step* to improve the foregoing comparisons, the $V_{S,30}$ (= average shear wave velocity for the top 30m of soil and/or bedrock) was replaced with $V_{S,el}$ (= average shear wave velocity over the thickness H) that is given on the basis of Eq.(1). The new comparisons are shown in Figures 3a & 4a for $M > 5.5$ and Figures 3b & 4b for $M < 5.5$. Observe that soil factors S for ground type A range between 0.85 and 1.30, i.e. they have come closer to the reference value of S = 1.00.

As a *second step*, the ground types are re-defined as follows:

- Ground type E was extended to sites with soil thickness up to 30m.
- Ground type A was broken into two sub-groups (A1 and A2) with the following characteristics:
- Ground Type A1: $V_{S,el} > 360$ m/s & $H < 5$m
- Ground Type A2: $V_{S,el}$ = 100 – 360m/s & H <5m,
  or $V_{S,el} > 360$ m/s & H = 5 – 30m

The new comparisons between theoretically predicted and EC-8 proposed soil factors are summarized in Figures 5a & 6a for $M > 5.5$ and Figures 5b & 6b for $M < 5.5$. Observe that ground type A1 has now become a truly reference soil condition with $S = 1.00 \pm 0.05$. Furthermore, the range of S factors for the new ground type A2 is comparable to that of all pre-existing ground types, implying that it has not only filled an existing gap in the ground type definition process, but it also corresponds to a more or less uniform seismic ground acceleration.

However, there is still significant difference between the EC-8 proposed and the theoretical S factors for other than A1 sites. Thus, as a *third step*, the S factors may be re-defined as shown in the following Table 2.

Table 2: Theory-based soil factors S

Ground Type	Soil Factor S		
	$M > 5.5$	$M < 5.5$	average
A1	1.00	1.00	1.00
A2	1.20	1.30	1.25
B	1.30	1.30	1.30
C	1.15	1.15	1.15
D	1.05	1.10	1.10
E	1.35	1.35	1.35

As expected, S factors for Type 2 spectra ($M < 5.5$) are somewhat larger than those for Type 1. Yet, the difference between the two sets is small and thus, for simplicity, an average set of seismicity independent S factors could be alternatively adopted.

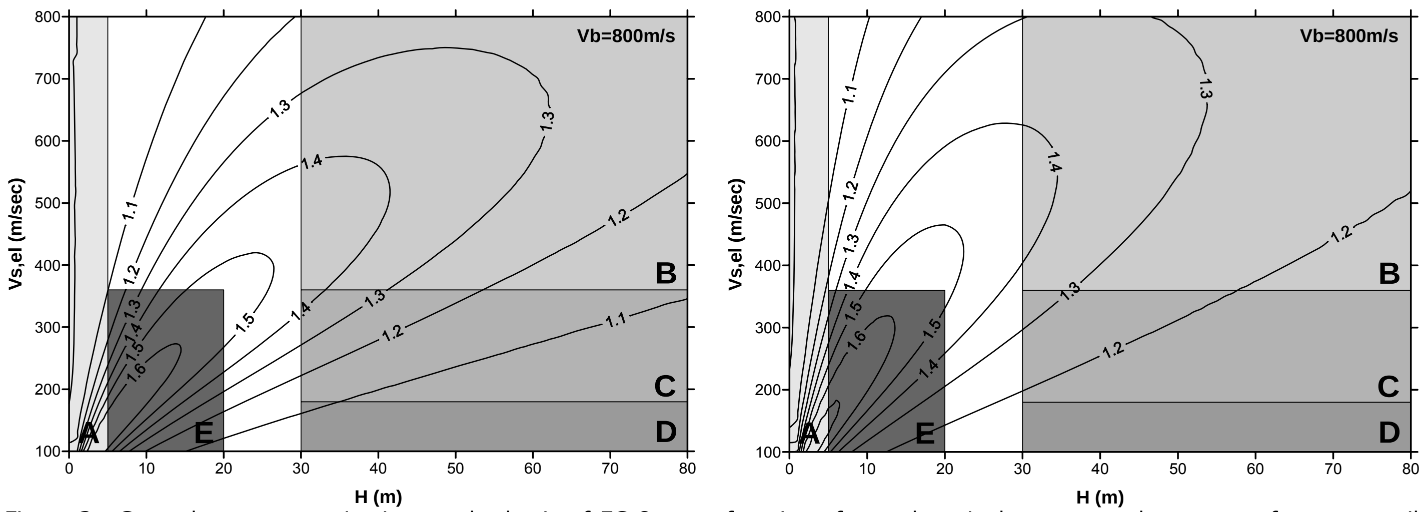


Figure 3 Ground type categorization on the basis of EC-8, as a function of H and equivalent $V_{s,el}$, and contours of average soil factor S variation from 1-D analyses with V_b = *800m/s* (indicatively): a) Type 1 ($M > 5.5$), b) Type 2 ($M < 5.5$)

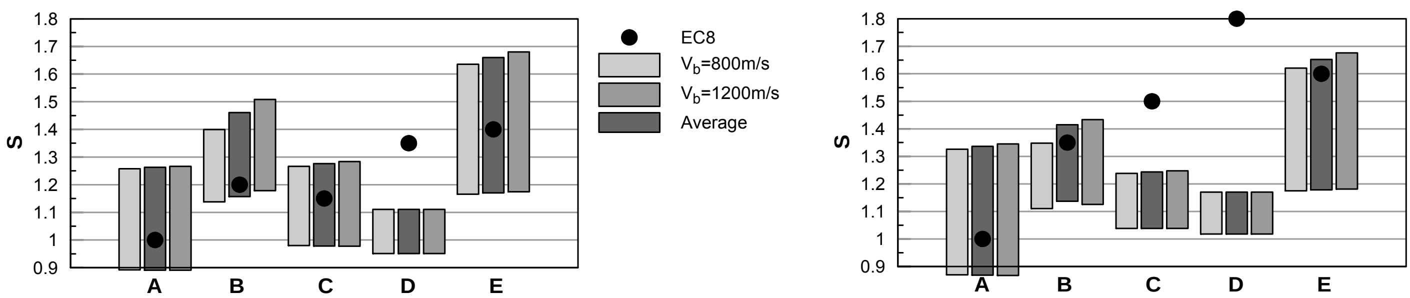


Figure 4 Soil factors S per ground type on the basis of EC-8, as a function of H and equivalent $V_{s,el}$, versus respective range of variation from all 1-D analyses: a) Type 1 ($M > 5.5$), b) Type 2 ($M < 5.5$)

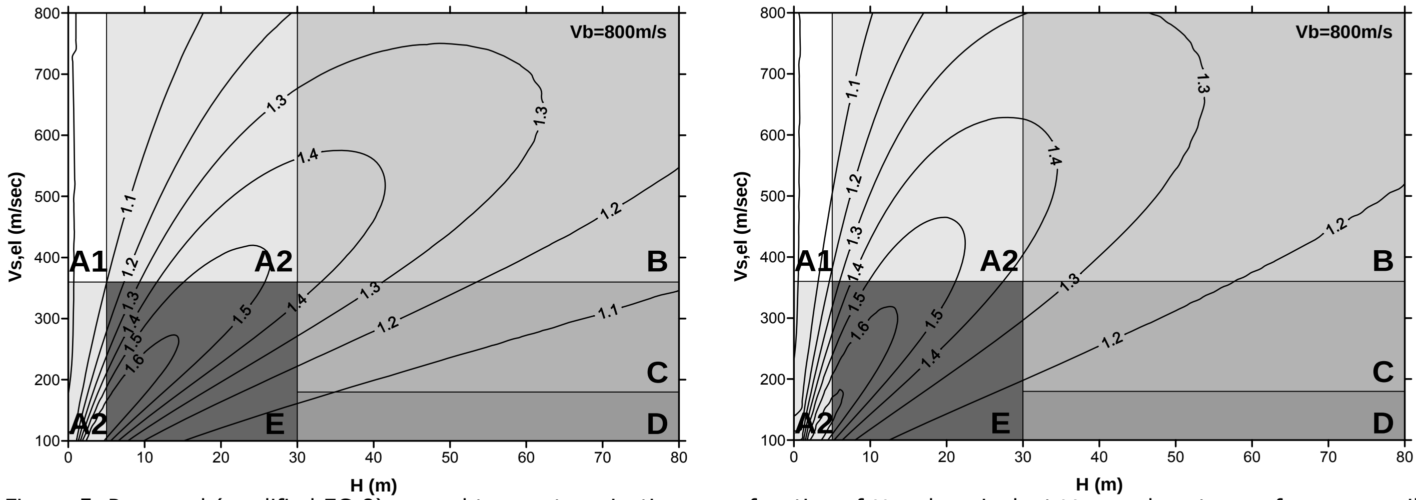


Figure 5. Proposed (modified EC-8) ground type categorization, as a function of H and equivalent $V_{s,el}$, and contours of average soil factor S variation from 1-D analyses with V_b = *800m/s* (indicatively): a) Type 1 (M > 5.5), b) Type 2 (M < 5.5)

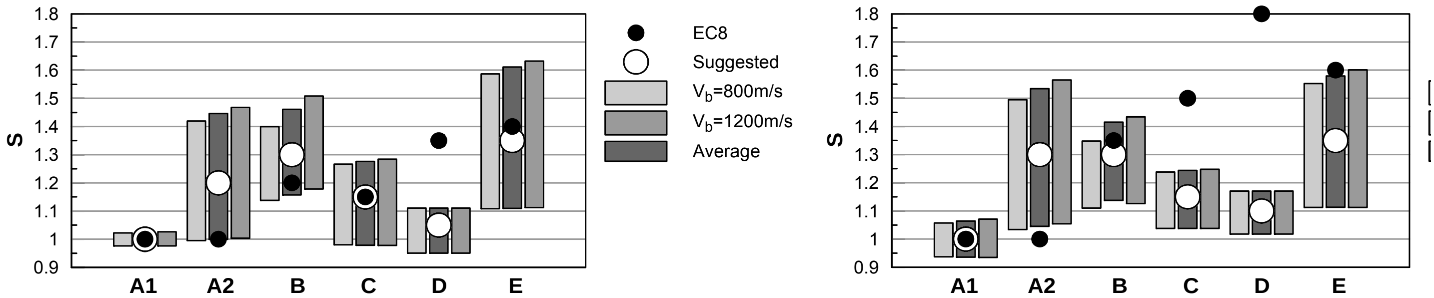


Figure 6. Adjusted soil factors S per ground type of the proposed modified EC-8 as computed from all the 1-D analyses and comparison to the standing code provisions: a) Type 1 (M > 5.5), b) Type 2 (M < 5.5)

4. Site effects on normalized elastic response spectra

Figures 7a & 7b summarize theoretical predictions and EC-8 proposed *NERS* for high seismicity and low seismicity areas, respectively. The comparison is shown in terms of the normalized spectral amplification ratios A_{Sa}* for the different ground types. Theoretical predictions are shown as a gray band, defined by the mean curve and the ± one standard deviation curves.

Observe the difference in the shapes of the theoretically predicted and the EC-8 proposed curves, which is most probably attributed to the smoothing that is commonly applied to code spectra. Eliminating this difference to a satisfactory degree would require drastic modification of the code spectra which, to our opinion, is not presently justified.

Hence, the attention of this paper was focused to improve fitting of the EC-8 *NERS* to the theoretical predictions, while maintaining their current general form. This was accomplished by keeping the reference *NERS* for ground type A unchanged and modifying appropriately the characteristic structural periods T_B and T_C for any of the remaining ground types. The new T_B and T_C values are summarized in Table 3, while the corresponding spectral amplification curves are drawn with bold line in Figures 7a & 7b.

Table 3: Modified structural periods T_B and T_C defining the design horizontal elastic response spectra

Ground Type	$M > 5.5$ (Type 1)		$M < 5.5$ (Type 2)	
	T_B (s)	T_C (s)	T_B (s)	T_C (s)
A1 & A2		0.40		0.25
B		0.50		0.30
C	0.15	0.60	0.05	0.40
D		0.60		0.40
E		0.50		0.30

5. Concluding Remarks

Based on the commentary and comparisons presented above, the following remarks may be made regarding the compatibility of the EC-8 code provisions for ground type categorization and site effects with 1D seismic wave propagation theory:

- The basic line of thought of the studied provisions of the EC-8 (e.g. quantitative ground type categorization, differentiation of both S and NERS according to ground type) is in general agreement with 1D seismic wave propagation theory.

- The definition of ground types is not complete, as sites with $V_{S,30} > 360$ m/s and H = 5 – 30m , as well as sites with $V_{S,30} < 360$ m/s and H = 20 – 30m do not seem to belong to any ground type.

- The use of $V_{S,30}$ for a quantitative index of ground type categorization is practical yet incompatible with 1D seismic wave propagation theory. Alternatively, the use of $V_{S,el}$, i.e. the V_S of the whole soil column irrespective of its thickness H, leads to more accurate ground type categorization.

- The site factors S of the EC-8 are generally conservative, with the possible exception of stiff soil sites (belonging to ground types A or B). Moreover, for some ground types the code-prescribed site factors S are over-conservative (e.g. ground type D, and C or E only for areas with M < 5.5).

- The normalized elastic response spectra (*NERS*) of the EC-8 for areas with high seismicity (M > 5.5) are generally conservative, oppositely to what is observed for areas with low seismicity (M < 5.5). For the latter, the non-conservatism of the *NERS* is counterbalanced for some ground types by the aforementioned over-conservatism of the site factors S in the EC-8.

Based on the above, there is ground for a rationalization of the EC-8 code provisions without altering its basic line of thought. This paper provides ideas for potential modifications to specific elements of the code. A more thorough presentation of potential modifications to the studied EC-8 code provisions may be found in Bouckovalas et al (2006).

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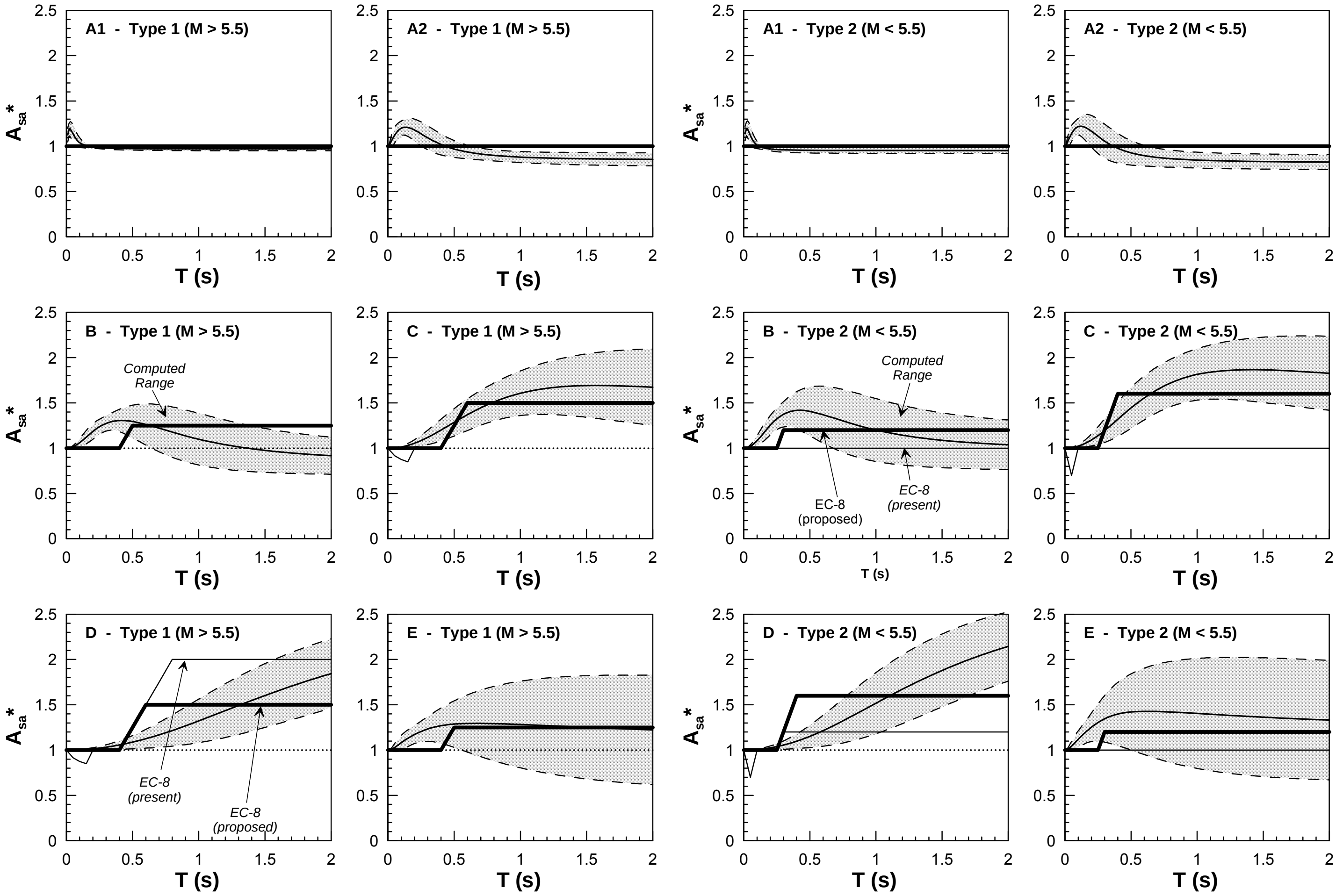


Figure 7.Normalized spectral amplification ratios A_{Sa}* per ground type from 1-D analyses and comparison to the standing and proposed modified EC-8 for: a) Type 1 (*M* > 5.5), b) Type 2 (*M* < 5.5)

(ανατύπωση από τα Πρακτικά του XIVth European Conference on Soil Mechanics and Geotechnical Engineering, 24 – 27 September 2007, Madrid, Spain, pp. 227-232)

Overview of Golden Gate Bridge Seismic Retrofit
Updated January 2007
www.goldengatebridge.org/projects

It was a bone rattling, concrete crushing, nerve-racking 15 seconds. At 5:04 p.m. on Tuesday evening, October 17, 1989, the 7.1 magnitude Loma Prieta earthquake caused 68 deaths, at least 3,700 injuries and an estimated dollar loss of $6 billion to $7 billion. The earthquake reminded the world that the San Francisco Bay region remains vulnerable. Although the Golden Gate Bridge suffered no observed damage from the Loma Prieta quake, since the epicenter was located some 60 miles to the south, the earthquake became a catalyst for the extensive seismic retrofit program that the historic structure is undergoing today.

Perhaps the most impressive statistic resulting from research conducted since the Loma Prieta earthquake is the conclusion by the U.S. Geological Survey (USGS) and other scientific organizations that there is a 62% probability of at least one magnitude 6.7 or greater quake capable of causing widespread damage, impacting the San Francisco Bay region within the next 30 years.

The Golden Gate Bridge represents a vital transportation link to the San Francisco Bay Area, serving more than 40 million vehicles a year. The Bridge is recognized by the American Society of Civil Engineers as one of seven civil engineering wonders of the United States. The Bridge is a national treasure known and admired around the world. Spanning 1.7 miles from abutment to abutment, the Golden Gate Bridge consists of six main structures:

1. San Francisco (south) Approach Viaduct
2. San Francisco (south) Anchorage Housing and Pylons S1 and S2
3. Fort Point Arch
4. Main Suspension Bridge
5. Marin (north) Approach Viaduct
6. Marin (north) Anchorage Housing and Pylons N1 and N2

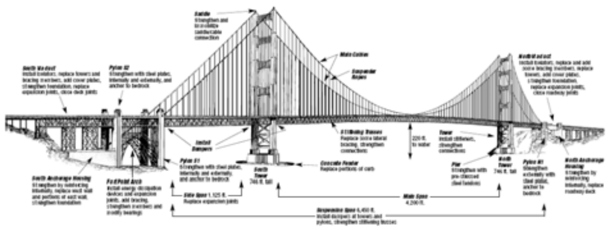
Immediately following the Loma Prieta quake, the Golden Gate Bridge, Highway and Transportation District (District), San Francisco, CA, the operator of the Golden Gate Bridge, engaged a team of consultants to conduct a vulnerability study. The conclusion of the study was that under a Richter magnitude 7.0 or greater earthquake with an epicenter near the Bridge, it would experience severe damage that could close this important transportation link for an extended period of time. If a Richter magnitude 8.0 or greater earthquake centered near the Bridge, there would be a substantial risk of impending collapse of the San Francisco and Marin Approach Viaducts and the Fort Point Arch, and extensive damage to the remaining Bridge structures, including the Main Suspension Bridge. It must be noted here, that as of April 2006, the seismic retrofit of the Golden Gate Bridge is far enough along that the Bridge no longer faces the potential for collapse and until the entire retrofit is completed, the risk of significant damage to the Main Suspension Bridge remains.

After determining that retrofitting the Bridge would be more cost-effective than replacing it, in 1992, the District hired engineering consultants to develop seismic retrofit design criteria. As part of this task, the site-specific design ground motions associated with different magnitudes of earthquakes and expected performance levels were defined as the basis for the Bridge retrofit design. The site-specific, moderate earthquake was defined as one having a 10 percent chance of being exceeded in a 50-year period or having an acceleration of 0.46g. The site-specific, maximum credible earthquake was defined as one having a return period of 1,000 years or having an acceleration of 0.65g, which is equivalent to the 1906 San Francisco earthquake of a magnitude 8.3 on the Richter scale.

Because of financial constraints, the District proceeded with phasing the construction of the seismic retrofit in a manner that reflected the degrees of structural vulnerabilities. In 1996, the three construction phases were established as follows (see figure):

- Phase 1 would retrofit the Marin (north) Approach Viaduct
- Phase 2 would retrofit the San Francisco (south) Approach Viaduct, San Francisco (south) Anchorage Housing, Fort Point Arch, and Pylons S1 and S2
- Phase 3 would Main Suspension Bridge and Marin (north) Anchorage Housing

GOLDEN GATE BRIDGE SEISMIC RETROFIT PROJECT



SEISMIC RETROFIT MEASURES

Phase 1:	North Viaduct Retrofit	Retrofit Completed December 2001
Phase 2:	South Viaduct, Fort Point Arch Retrofit	Started Summer 2001, Scheduled for Completion 2006
Phase 3A:	North Anchorage Housing Retrofit	Construction starts 2006, 3.5 years to Complete
Phase 3B:	Main Span and Tower Retrofit	Construction starts 2008, 3.5 years to Complete

Updated March 2006

Phase 1 - Completed in early 2002

On June 27, 1997, the Board of Directors of the District awarded a contract for the first phase of seismic retrofit construction. It also organized a construction administration team made up of District staff and consultants.

The seismic retrofit measures applied to the Bridge structures consist of various methods of structural upgrades and include both the strengthening of structural components and the modification of structural response of the structures so they can better respond to strong motions without damage. The cost of Phase 1 totaled $71 million, which was funded using Golden Gate Bridge tolls.

The major strengthening measures implemented on the Marin (north) Approach Viaduct included the following:

1. Strengthening the existing foundations
2. Total replacement of the four supporting steel towers and strengthening of Bent N11
3. Replacement and addition of top and bottom lateral bracing and strengthening vertical truss members and truss connections
4. The structural system has also been modified to minimize effects of ground motions on the structure by the following:
   - Connecting five, simply-supported truss spans into a continuous truss;
   - Installing seismic expansion joints at the north and south ends of the viaduct truss; and
   - Installing isolator bearings atop the new steel support towers at the Pylon N2 support and at Bent N11.

The scope of retrofit within the viaduct truss was significantly reduced through the installation of lead-core-rubber type isolator bearings. These bearings enable displacements of the truss relative to its supports, thereby significantly reducing the transfer of seismic forces onto the truss.

The maximum credible earthquake is predicted to create up to 12-inch displacements of the truss. To prevent the truss from crushing against the Marin (north) Abutment and Pylon N2, seismic expansion joints were constructed at these locations by removing a section of the orthotropic steel deck of the viaduct at Pylon N2 and removing and reconstructing the Marin Abutment backwall. These joints enable truss displacements of up to 15 inches, thereby preventing damage that could jeopardize the integrity of the structure.

A primary challenge of Phase 1 was to construct the retrofit measures under continuous traffic. The construction inspection team closely monitored the structure throughout the complex process of installing temporary bracing, constructing and loading temporary supports for replacement of the towers, removing and replacing members, and strengthening members and connections.

The first work undertaken was to connect the viaduct spans to create a continuous superstructure capable of distributing lateral forces to prescribed points while the structure underwent tower replacements. Bent N11 near the Marin (north) Abutment was substantially strengthened to substitute for temporary loss of longitudinal stiffness at the removed supporting towers. Before the individual towers could be replaced, the retrofit sequence required that truss members directly above each of the towers be replaced and truss panel points be strengthened.

The contractor retrofitted the tower foundations in a two-stage operation. The first stage was constructed with the existing towers still in place, which allowed them to schedule this work outside of the project critical path.

During the first stage, cast-in-drilled-hole (CIDH) piling and pile caps were added around the perimeter of the original foundation pedestals. The new concrete to existing concrete interfaces were strengthened with post-tensioning of monostrands, clamping the new footings to the pedestals of the existing foundations. The existing grade beams between the foundation pedestals were also substantially strengthened, and additional grade beams were constructed.

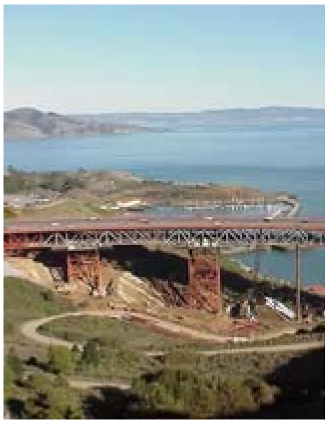
After the existing tower was removed, the second stage of the foundation retrofit proceeded. First, the remaining upper portions of the existing pedestals were demolished. Then, new upper pedestals were constructed and closure pours placed to incorporate these elements into the entire foundation system. The erection of a new tower followed.

The most visually dramatic Phase 1 work was the complete removal and replacement of the four steel support towers with footprints of 50 feet by 75 feet and heights of up to 150 feet. The contractor sequentially replaced the existing towers with new ones that very closely imitate the appearance of the original towers.

Jacking of the superstructure continuously under traffic was an interesting aspect of the tower removal and replacement operation. Once erection of the temporary supports was completed on the sides of the original tower, a series of synchronized jacks lifted the superstructure from the six original tower bearings by loading the six temporary support bearings. The temporary supports and jack were located 25 feet away from the adjoining original tower. At the jacking points, the superstructure had to be lifted by up to 1½ inches to provide for up to ¼-inch lift at the existing bearings. This separation was sufficient for the contractor to proceed with removal of the original bearings, which was to be followed by demolition of the tower below.

The synchronous lift system used by the contractor was controlled at an electronic central control panel that is capable of raising the individual jack rams in precise increments of 0.2 inch and of shutting down the individual jacks once the superstructure was raised the prescribed height.

Viaduct support tower undergoes demolition.



Temporary supports are in place as the viaductsupport tower is removed.

Closer view of temporary supports in place with viaduct support tower removed.

New support tower under construction.

New support tower under construction. (Photos property of GGBHTD)

A total of six jacking points were used per tower; each point consisting of a cluster of four 200-ton jacks. Each jack cluster was tied to a single manifold such that all four jacks received the same hydraulic and electronic signals from the controller. This system included highly accurate (up to 0.04 inch) sensors, which were attached to the superstructure to control its position. Aside from this means of displacement monitoring, a licensed land surveyor was also deployed on a nearby hillside to monitor structure location prior to, during, and after the jacking operations so as to detect any unplanned access movement. Locking collars were placed on the jacks as a means of providing redundancy in the event of a hydraulic failure of the jacking system. Workers monitored the existing tower bearings and reported on their status via radio lift off.

The overall jacking operations typically required approximately a half hour, the majority of which was spent checking and monitoring the status of the lift, with frequent instrument readings and status verifications.

Phase 2 - To Be Completed in 2007

On May 11, 2001, the Board of Directors of the District authorized award of the Phase 2 construction contract. In June 2001, the second construction phase began, and it is the most complex part of the project in terms of design and construction. Federal, state and regional funds totaling $174 million were aggressively sought and authorized to complete this phase. This phase, set to be completed in 2006, encompasses structural retrofit of many different types of structures of the south approach: the south approach viaduct, south anchorage housing, Fort Point arch, and south pylons. Retrofit measures developed for each of these structures reflect their individual behavior under seismic ground motions and their interaction at points of interface while accommodating their already-in-place historic configuration.

Without closing the Golden Gate Bridge to traffic, the steel support towers and bottom lateral bracing of the south approach viaduct will be entirely replaced, and seismic isolation bearings and joints will be installed at the roadway level. The west wall of the south anchorage housing will be replaced and massive internal shear walls constructed. Five million pounds of external and internal steel plating will be added to south pylon walls. The historic architectural appearance of the external surfaces of the pylons will remain unchanged with the addition of a new external concrete cover on top of the new plating.

The Fort Point arch will be retrofitted with new arch bearings and energy dissipation devices, and isolation joints will be installed. Steel members throughout the entire arch will undergo extensive strengthening.

Not only were immense challenges presented in the design and engineering of this phase of retrofit construction, but the construction site itself presents very unique project limitations. The construction site is located in a very compact area bound on the west by the Pacific Ocean and on the east by very steep slopes. Severe weather including strong wind and high waves are nearly constant. Access consists of two narrow roads that must be shared with thousands of tourists visiting the Golden Gate Bridge and the Historic Fort Point Site located directly below the Fort Point arch structure of the Bridge. Construction on the arch is limited to four days per week to allow limited visitation to the Site. The small construction staging areas available near the work site further restrict the logistics of the construction operations.

Phase 3 - To Begin in 2007

The third and final phase of the Golden Gate Bridge Seismic Retrofit Construction Project has been separated into two sub phases as follows:

<u>Phase 3A: Retrofit of the North Anchorage Housing and Pylon N1</u>

Phase 3A will be funded using a combination of federal funds along with regional and state earmarks. It is anticipated that construction bids will go out for Phase 3A in 2007, with construction underway by the end of 2007. This phase project will take approximately 3.5 years to complete.

<u>Phase 3B: Retrofit of the Main Suspension Span, Main Towers, South Tower Pier and Fender</u>

Phase 3B will also be funded using a combination of federal funds along with regional and state earmarks. Phase 3B will begin in 2009 and also take approximately 3.5 years to complete.

The seismic retrofit measures for these phases consist of strengthening foundations, installation of micropiles and rock bolts, construction of reinforced concrete shearwalls, replacement of the housing roof/roadway deck with a precast concrete slab-on-steel stringer deck system involving nighttime lane closures, and other structural modifications.

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ΕΝΔΙΑΦΕΡΟΝΤΑ ΓΕΩΤΕΧΝΙΚΑ ΝΕΑ

Announcing the "Geotechnical Safety Network" (GEOSNet)

Hosted by Geoengineer.org website

Geoengineer.org is pleased to support the Geotechnical Safety Network. We believe that this new activity may contribute to our profession and we are glad to support it with all our resources. Please read the announcement below and consider becoming part of this effort.

This network is an open collaborative platform that will hopefully serve to sustain, energize, and regularize future events on geotechnical safety and risk. We would like to extend a warm welcome to everyone who shares our mission (researchers, practitioners, educators, students) to join us as members or play a more active role as task group coordinators.

You can join GEOSNet by visiting the website. Fill in the application form and e-mail it to Dr. Marco Uzielli if you are interested to join our network.

Message from the GEOSNet

"There is a need for geotechnical design codes and standards to keep pace with the globalization pressure to harmonize across national boundaries, the regulatory pressure to harmonize with structural design, rising public expectations in health & environment, and increasing complexities of big projects with their associated financial/insurance risks.

There are significant practical and research challenges. Examples include the complexities of geotechnical variabilities, the role of numerical methods in design, the roles of full-scale testing, observational approach, etc. versus design calculations in the overall assurance of safety. These challenges are unique to geotechnical engineering, particularly for large complicated projects.

There is a groundswell of related activities taking place in national code committees and international professional societies, within and outside geotechnical engineering.

A series of thematic symposiums/workshops related to limit state design has been organized since the early nineties (the first session on codes and standards was organized in 1989). These events were organized by motivated groups of individuals for different purposes. There is neither regularity nor continuity to these events.

The time is ripe to form a network to promote coordination between related groups, to broaden participation beyond geotechnical engineering, to garner support from stakeholders from the industry and government agencies, and to support a more regular series of activities.

The Geotechnical Safety Network (GEOSNet) was formed during Taipei2006 with Dr CT Chin and Prof KK Phoon as founding chair and co-chair, respectively. The inaugural event for GEOSNet is the First International Symposium on Geotechnical Safety and Risk, which was chaired by Prof HW Huang and was successfully held in Shanghai, China, between 18 and 19 Oct 2007".

ΑΝΑΣΚΟΠΗΣΗ ΓΕΓΟΝΟΤΩΝ ΓΕΩΤΕΧΝΙΚΟΥ ΕΝΔΙΑΦΕΡΟΝΤΟΣ

ΤΑΚΤΙΚΗ ΓΕΝΙΚΗ ΣΥΝΕΛΕΥΣΗ ΕΕΕΕΘ

Η ετήσια τακτική γενική συνέλευση της εταιρείας διεξήχθη την Τρίτη 11 Δεκεμβρίου 2007 στην Αίθουσα Εκδηλώσεων της Σχολής Πολιτικών Μηχανικών του Εθνικού Μετσοβίου Πολυτεχνείου).

Στην Γ.Σ. συμμετέσχον 20 μέλη της εταιρείας, τα οποία ενέκριναν τα πεπραγμένα της Εκτελεστικής Επιτροπής της περιόδου 16.06.2006 – 10.12.2007 και τον οικονομικό απολογισμό της περιόδου 01.01.2006 – 31.12.2006. Στη συνέχεια παρατίθεται η έκθεση των πεπραγμένων και απόσπασμα του οικονομικού απολογισμού.

ΕΚΘΕΣΗ ΠΕΠΡΑΓΜΕΝΩΝ (16.06.2006 ÷ 11.12.2007)

ΕΙΣΑΓΩΓΗ

Όπως είναι γνωστό η τελευταία Γενική Συνέλευση έγινε στις 16.06.2006. Στο χρονικό διάστημα που μεσολάβησε έκτοτε, ένα μεγάλο μέρος του χρόνου και της δραστηριότητας της Εκτελεστικής Επιτροπής αφιερώθηκε στην προετοιμασία του φακέλου διεκδίκησης της διοργάνωσης 15ου Πανευρωπαϊκού Συνεδρίου Εδαφομηχανικής και Γεωτεχνικής Μηχανικής στην Αθήνα το Σεπτέμβριο του έτους 2011. Αυτός ήταν κυρίως και ο λόγος που η παρούσα ετήσια Γενική συνέλευση λαμβάνει χώρα με κάποια καθυστέρηση.

Παράλληλα όμως το διάστημα αυτό ήταν και αρκετά πλούσιο σε γεγονότα και δραστηριότητες που εμπίπτουν στους σκοπούς της Επιστημονικής μας Εταιρείας.

Μετά την τελευταία Γ. Σ. η Εκτελεστική επιτροπή συνεδρίασε 14 φορές.

ΝΟΜΙΜΟΠΟΙΗΣΗ ΤΗΣ ΤΡΟΠΟΠΟΙΗΣΗΣ ΤΟΥ ΚΑΤΑΣΤΑΤΙΚΟΥ

Στις 16/02/2007 εκδόθηκε η απόφαση του Πρωτοδικείου Αθηνών με την οποία εγκρίθηκε η τροποποίηση του Καταστατικού της Εταιρείας, σύμφωνα με τις αποφάσεις των δύο προηγούμενων Γενικών Συνελεύσεων και στις 30/03/2007 έγινε η καταχώρηση στο Βιβλίο Σωματείων του Πρωτοδικείου. Από την ημερομηνία αυτή ισχύει το τροποποιημένο Καταστατικό Υπενθυμίζεται ότι οι τροποποιήσεις αφορούν κυρίως στην επωνυμία της Εταιρεία που τώρα έγινε ΕΛΛΗΝΙΚΗ ΕΠΙΣΤΗΜΟΝΙΚΗ ΕΤΑΙΡΕΙΑ ΕΔΑΦΟΜΗΧΑΝΙΚΗΣ & ΓΕΩΤΕΧΝΙΚΗΣ ΜΗΧΑΝΙΚΗΣ, ώστε να συμφωνεί με την επωνυμία της αντίστοιχης διεθνούς Εταιρείας, και στον τρόπο διεξαγωγής των αρχαιρεσιών για την ανάδειξη των μελών της Εκτελεστικής και της Εξελεγκτικής Επιτροπής με τον οποίο δίνεται η δυνατότητα ταχυδρομικής αποστολής ψηφοδελτίου. Το τροποποιημένο Καταστατικό και η Διαδικασία Ψηφοφορίας που εγκρίθηκε από την προηγούμενη Γενική Συνέλευση δημοσιεύθηκαν στο πανηγυρικό τεύχος (αρ. 8) των «Νέων της ΕΕΕΕΓΜ» που εκδόθηκε και διανεμήθηκε με την ευκαιρία του εορτασμού των 40 χρόνων της ΕΕΕΕΓΜ.

Αντίγραφα του Καταστατικού και της Διαδικασίας μπορούν να ζητηθούν από τη Γραμματεία (κα Γ. Αθανασίου, τηλ. 210 7723434, e-mail: geotech@central.ntua.gr).

Με την ευγενική προσφορά και επιμέλεια της κας Ειρήνης Τσαμανδουράκη, συζύγου του αείμνηστου καθηγητή Π. Παπακυριακόπουλου, η ΕΕΕΕΓΜ απέκτησε νέο καλλιτεχνικό λογότυπο.

ΝΕΑ ΜΕΛΗ

Από την τελευταία Γενική Συνέλευση μέχρι σήμερα ενεγράφησαν στην ΕΕΕΕΓΜ τα κάτωθι μέλη (κατά σειράν εγγραφής):

Αναστασόπουλος Ιωάννης	Δρ. Πολιτικός Μηχανικός
Δούλαλα – Rigby Χάιδω	Πολιτικός Μηχανικός
Μαρονικολάκη Ειρήνη	Πολιτικός Μηχανικός
Παπαδάκος Γεώργιος	Πολιτικός Μηχανικός
Ξενάκη Βασιλική	Δρ. Πολιτικός Μηχανικός
Ιωαννίδης Κωνσταντίνος	Πολιτικός Μηχανικός
Μαρίνος Βασίλειος	Τεχνικός Γεωλόγος, MSc, DIC

Επίσης ενεκρίθη η εγγραφή των κάτωθι:

Ιωαννίδης Ιωάννης	Πολιτικός Μηχανικός
Γρυπάρης Φαίδων	Πολιτικός Μηχανικός
Ιακωβίδου Μαρία	Πολιτικός Μηχανικός, MSc, DIC
Παπαχαραλάμπους Γεώργιος	Πολιτικός Μηχανικός, MSc, DIC
Σιταρένιος Παναγιώτης	Πολιτικός Μηχανικός
Μουλίνος Γεράσιμος	Δρ. Πολιτικός Μηχανικός
Βρεττός Χρήστος	Δρ. Πολιτικός Μηχανικός
Κεραμίδας Ευτύχιος	Πολιτικός Μηχανικός, MSc, DIC
Κοζομπόλης Απόστολος	Πολιτικός Μηχανικός, MEng.

ΕΚΔΗΛΩΣΗ ΓΙΑ ΤΑ 40 ΧΡΟΝΙΑ ΤΗΣ ΕΕΕΕΓΜ

Στις 15 Μαϊου 2007 έλαβε χώρα στην αίθουσα του ΕΒΕΑ πανηγυρική εκδήλωση για τα 40 χρόνια από την ίδρυση της ΕΕΕΕΘ/ΕΕΕΕΓΜ, κατά την οποία επιδόθηκαν αναμνηστικές πλακέτες στα Ιδρυτικά Μέλη, στους διατελέσαντες Προέδρους της Εκτελεστικής Επιτροπής και στον επί 26 συνεχή έτη Γεν. Γραμματέα της.

Στην εκδήλωση δόθηκε διάλεξη από τον συνάδελφο και πρώην Πρόεδρο Ηλία Σωτηρόπουλο με θέμα: «Εδαφομηχανική – Παρελθόν, Παρόν και Μέλλον» και ακολούθησε δεξίωση.

Επίσης κυκλοφόρησε πανηγυρικό τεύχος των «Νέων της ΕΕΕΕΓΜ» (αρ. 8).

ΔΙΑΛΕΞΕΙΣ

14.09.2006 Διάλεξη του καθηγητή **Robert Mair**, του Πανεπιστημίου του Cambridge (επανάληψη της "Rankine Lecture", 2006) με θέμα: "Tunneling and Geotechnics – New Horizons".

11.12.2006 Διαλέξεις Νέων Διδακτόρων Γεωτεχνικών Μηχανικών με θέματα της διδακτορικής τους διατριβής (σε συνεργασία με την ΕΕΕΕΘ του ΤΕΕ).

- **Πρόδρομου Ψαρρόπουλου**, Δρ. Πολ. Μηχ. ΕΜΠ, με θέμα: «Εδαφοδυναμική Προσομοίωση στη Σεισμική Ανάλυση Βάθρων και ακροβάθρων Γεφυρών.

- **Έλενας Κούμουλου**, Δρ. Πολ. Μηχ. Cambridge Univ. με θέμα: «Προσομοίωση στον Φυγοκεντριστή της Κίνησης Βαρέων μη Υδατοδιαλυτών Υγρών Ρυπαντών».

19.12.2006 Διάλεξη της συναδέλφου **Αντας Αθανασοπούλου**, Πολ. Μηχ. MSc, υποψήφιας διδάκτορος Πανεπ. Berkeley με θέμα: «Διερεύνηση της Συμπεριφοράς των Συστημάτων Αντιπλημμυρικής Προστασίας της Νέας Ορλεάνης κατά τον Τυφώνα "Κατρίνα" της 29ης Αυγούστου 2005» (σε συνεργασία με την ΕΕΕΕΘ του ΤΕΕ).

18.06.2007 Διάλεξη του καθηγητή **Mounir Khalel Berrah** της Ecole National Polytechnique της Αλγερίας με θέμα: «The Complete Stochastic Deamplification Approach: An Efficient Tool to Describe the Spatial Variability of Earthquake Motion" (σε συνεργασία με το ΕΜΠ).

ΗΜΕΡΙΔΕΣ – ΔΙΕΘΝΗ ΣΥΝΕΔΡΙΑ ΣΤΗΝ ΕΛΛΑΔΑ

Κατά την περίοδο που εξετάζεται, έλαβαν χώρα στην Ελλάδα, με ευρεία και ενεργό συμμετοχή των μελών της ΕΕΕΕΓΜ, οι ακόλουθες ημερίδες και διεθνή συνέδρια:

11.01.2007 Ημερίδα «Γεωτεχνικές Εφαρμογές Γεωσυνθετικών Υλικών», στην Αθήνα, (συνδιοργάνωση ΕΕΕΕΘ/ΤΕΕ και ΕΣΓΥ).

01.02.2007 Ημερίδα "Νέες Εξελιγμένες Μέθοδοι Μηχανικής Διάνοιξης Σηράγγων", στην Αθήνα, (συνδιοργάνωση ΕΕΕΕΘ/ΤΕΕ, ΕΕΣΥΕ και Πολυτεχνικού Συλλόγου).

20 ÷ 22.06.2007 Διεθνές Συνέδριο με θέμα "Advanced Characterization of Pavement and Soil Engineering Materials:, στην Αθήνα, (συνδιοργάνωση ΕΜΠ, Delft και Πανεπ. Illinois, υπό την αιγίδα των ISAP, ISCP και HESPER).

25 ÷ 28.06.2007 4^{o} Διεθνές Συνέδριο Γεωτεχνικής Σεισμικής Μηχανικής (4th Int. Conf. on Earthquake Geotechnical Engineering), στη Θεσσαλονίκη, (συνδιοργάνωση από επιτροπή των TC4 της ISSMGE, το Εργαστήριο Γεωτεχνικής του Αριστοτελείου Πανεπιστημίου και την ΕΕΕΕΓΜ).

Λεπτομερείς αναφορές στις εκδηλώσεις αυτές έχουν περιληφθεί στα τεύχη των «Νέων της ΕΕΕΕΓΜ» υπ' αριθμ. 7 και 9.

14^{o} ΠΑΝΕΥΡΩΠΑΪΚΟ ΣΥΝΕΔΡΙΟ ΣΤΗ ΜΑΔΡΙΤΗ

Το Συνέδριο πραγματοποιήθηκε στη Μαδρίτη στο διάστημα 24 ÷ 27 Σεπτεμβρίου, με συμμετοχή 830 συνέδρων, μεταξύ των οποίων 16 μέλη της ΕΕΕΕΓΜ και τρεις Έλληνες συνάδελφοι εγκατεστημένοι στο εξωτερικό. Ο Γενικός Γραμματέας Α. Αναγνωστόπουλος ήταν μέλος της Διεθνούς Επιστημονικής Επιτροπής του Συνεδρίου και ο Αντιπρόεδρος Χ. Τσατσανίφος ήταν Γενικός εισηγητής στην Κύρια Συνεδρία 4.

Τα μέλη μας Γ. Μπουκοβάλας και Κ. Πιτιλάκης συμμετείχαν στο workshop της επιτροπής ERTC12 στα πλαίσια του Συνεδρίου.

Ο συνάδελφος Γ. Αναγνώστου, Καθηγητής στο Πανεπιστήμιο της Ζυρίχης ήταν panelist στη Συνεδρία Συζήτησης 3.1.

Τα μέλη μας Γ. Γκαζέτας και Γ. Αθανασόπουλος ήταν panelist στις Συνεδρίες Συζήτησης 1.1 και 4.3 αντίστοιχα, αλλά δεν μπόρεσαν να παρευρεθούν στο Συνέδριο.

Εκ μέρους της ΕΕΕΕΓΜ υπεβλήθησαν στο Συνέδριο και δημοσιεύτηκαν στα Πρακτικά, έξι (6) άρθρα μελών της.

Λεπτομέρειες για το Συνέδριο υπάρχουν στο τεύχος 10 των «Νέων της ΕΕΕΕΓΜ».

ΔΙΕΚΔΙΚΗΣΗ ΚΑΙ ΑΝΑΛΗΨΗ ΤΗΣ ΔΙΟΡΓΑΝΩΣΗΣ ΤΟΥ 15^{ου} ΠΑΝΕΥΡΩΠΑΪΚΟΥ ΣΥΝΕΔΡΙΟΥ

Η Εκτελεστική Επιτροπή, ακολουθώντας τις ισχύουσες διαδικασίες της ISSMGE, υπέβαλε στις 17.04.2007 έγγραφο αίτημα προς τον Γεν. Γραμματέα και τον Αντιπρόεδρο για την Ευρώπη, για την ανάληψη της διοργάνωσης του 15^{ου} Πανευρωπαϊκού Συνεδρίου Εδαφομηχανικής και Γεωτεχνικής Μηχανικής στην Αθήνα το 2011, με θέμα "Geotechnics of Hard Soils – Weak Rocks".

Ακολούθως προέβη στις απαραίτητες ενέργειες για την προετοιμασία του σχετικού φακέλου διεκδίκησης, προκειμένου να διεκδικήσει την ψήφο των υπολοίπων Ευρωπαϊκών Γεωτεχνικών Ενώσεων μελών της International Society for Soil Mechanics & Geotechnical Engineering (ISSMGE), κατά τη Γενική τους Συνέλευση στα πλαίσια του 14^{ου} ECSMGE στη Μαδρίτη.

Η σύνταξη του φακέλου απαιτεί τη συνεργασία με εξειδικευμένο γραφείο διοργάνωσης διεθνών συνεδρίων. Για το σκοπό αυτό η Ε. Ε. επικοινώνησε με ένα αριθμό μεγάλων τέτοιων γραφείων, από τα οποία έδειξαν ενδιαφέρον και υπέβαλαν προσφορές τα εξής τέσσερα:

- TRIAENA TOURS & CONGRESS S.A.
- AC & C PROFESSIONAL CONGRESS ORGANIZER
- ZITA CONGRESS & TRAVEL
- TRAVEL DIRECTIONS, GR

Με αξιολόγηση της εμπειρίας και των προσφορών τους η επιλογή περιορίστηκε μεταξύ των δύο πρώτων και κατόπιν τελικής αξιολόγησης, στην οποία θα αναφερθεί στη συνέχεια ο συνάδελφος Γιώργος Ντούλης, επελέγη το πρώτο από τα προαναφερθέντα γραφεία.

Με τη βοήθεια του γραφείου αυτού συντάχθηκε ο φάκελος διεκδίκησης και ταχυδρομήθηκε στις Ευρωπαϊκές Γεωτεχνικές Ενώσεις και στους αξιωματούχους της ISSMGE αρκετές μέρες πριν από την Γενική Συνέλευση της 25^{ης} Σεπτεμβρίου 2007 στη Μαδρίτη.

Στη Γενική Συνέλευση παρέστησαν ο Πρόεδρος, ο Α΄ Αντιπρόεδρος και ο Γεν. Γραμματέας της ΕΕΕΕΓΜ και την παρουσίαση της διεκδίκησης έκανε ο δεύτερος εξ αυτών συνάδελφος Χ. Τσατσανίφος.

Αντίπαλος της ΕΕΕΕΓΜ και της Αθήνας, ήταν η British Geotechnical Association (BGA) και το Εδιμβούργο.

Κατά την ψηφοφορία η ΕΕΕΕΓΜ κέρδισε τη διοργάνωση με ψήφους 22 έναντι 9 επί 31 ψηφισάντων.

Ήδη η Ε. Ε. βρίσκεται στη διαδικασία σύστασης Οργανωτικής Επιτροπής και Επιστημονικής Επιτροπής, καθώς και σύναψης Σύμβασης με το γραφείο διοργάνωσης, ώστε να προχωρήσουν οι οργανωτικές διαδικασίες.

ΥΠΟΣΤΗΡΙΞΗ ΣΥΜΜΕΤΟΧΗΣ ΝΕΩΝ ΓΕΩΤΕΧΝΙΚΩΝ ΜΗΧΑΝΙΚΩΝ ΣΕ ΔΙΕΘΝΕΙΣ ΕΚΔΗΛΩΣΕΙΣ

Με δαπάνες της ΕΕΕΕΓΜ έλαβαν μέρος στα Πανευρωπαϊκά Συνέδρια Νέων Γεωτεχνικών Μηχανικών (YGEC) οι κάτωθι συνάδελφοι:

- Π. Ανδρέου : 17th YGEC, Ζάγκρεμπ Κροατίας (20÷22 Ιουλίου, 2006)
- Α. Αραπάκου και Α. Δρουδάκης : 18th YGEC, Ανκόνα, Ιταλίας (17÷20 Ιουνίου, 2007)

ΔΙΕΘΝΕΙΣ ΣΧΕΣΕΙΣ

Γενική Συνέλευση της ISSMGE

Η Συνέλευση έγινε στο Brisbane της Αυστραλίας στα πλαίσια του 10th Australia – New Zealand Conference on Geomechanics, 21 ÷ 24 Οκτωβρίου 2007.

Η ΕΕΕΕΓΜ δεν εκπροσωπήθηκε, αλλά εξουσιοδότησε τον Αντιπρόεδρο για την Ευρώπη (R. Frank) να δώσει εκ μέρους της θετική ψήφο για τη σύσταση της Ομοσπονδίας Διεθνών Γεω-Τεχνικών Ενώσεων (Federation of International Geo-Engineering Societies – FIGS) όπου οι τρεις διεθνείς ενώσεις (ISSMGE, ISRM και IAEG) διατηρούν την αυτονομία τους, αλλά συνεργάζονται σε κοινές δράσεις, συνιστώντας Κοινές Τεχνικές Επιτροπές (Joint Technical Committees) για διάφορα θέματα.

Συμμετοχή σε Τεχνικές Επιτροπές της Διεθνούς Ένωσης

Στις Τεχνικές Επιτροπές της ISSMGE συμμετέχουν μέλη της ΕΕΓΜ ως ακολούθως:

TC 3 Geotechnics of Pavements
Α. Λοΐζος (core member)

TC 4 Earthquake Geotechnical Engineering & Associated Problems
Κ. Πιτιλάκης (core member)
Γ. Γκαζέτας
Γ. Μπουκοβάλας
TC 5 Environmental Geotechnics
Μ. Πανταζίδου
Δ. Κούμουλος
TC 17 Ground Improvement
Α. Πλατής
TC 18 Deep Foundations
Α. Κωμοδρόμος
TC 28 Underground construction in Soft Ground Conditions
Π. Βέττας
Σ. Σχινά
TC 33 Geotechnics of Soil Erosion
Μ. Σακελλαρίου
TC 34 Prediction Methods in Large Strain Geomechanics
Ι. Βαρδουλάκης (core member)
ERTC 12 Implementation of Eurocode 8
Κ. Πιτιλάκης
Γ. Γκαζέτας
Γ. Μπουκοβάλας
Μ. Παχάκης (για τη συμβατότητα με τον EC-7)

Οι Κοινές Τεχνικές Επιτροπές (JTCs) της FIGS, στις οποίες έχουν επίσης δηλώσει συμμετοχή μέλη μας δεν έχουν ενεργοποιηθεί.

Επαφές με Αξιωματούχους της ISSMGE

Υπήρξαν συναντήσεις και φιλοξενία (γεύμα) του Προέδρου της ISSGME (P. Sêco e Pinto) και του Αντιπροέδρου για την Ευρώπη (R. Frank) τόσο στην Αθήνα όσο και στη Μαδρίτη στα πλαίσια της διεκδίκησης της διοργάνωσης του 15^{ου} ESMGE.

ΕΚΔΟΣΕΙΣ

- Συνεχίστηκε, με επιμέλεια του Αντιπροέδρου Χ. Τσατσανίφου, η έκδοση του ενημερωτικού δελτίου «ΤΑ ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ». Μέσα στην εξεταζόμενη περίοδο εκδόθηκαν επτά (7) τεύχη (αρ. 4 έως 10) εκ των οποίων τα δύο (8 και 9) σε πολυτελή έκδοση. Το τεύχος 9 εκδόθηκε στην αγγλική και διανεμήθηκε στις ευρωπαϊκές γεωτεχνικές ενώσεις με το φάκελο διεκδίκησης του 15ου Πανευρωπαϊκού Συνεδρίου.

- Συγκεντρώθηκαν τα γραπτά κείμενα των προσκεκλημένων και ειδικών ομιλιών, καθώς και των χαιρετισμών του 5ου Πανελληνίου Συνεδρίου Γεωτεχνικής (Ξάνθη, 2006) με φροντίδα του Προέδρου και του συναδέλφου Γ. Ντουνιά και έπειτα από κάποια στοιχειώδη εκδοτική επεξεργασία εστάλησαν μέσω του ΤΕΕ στο τυπογραφείο για την έκδοση του 4ου τόμου των Πρακτικών. Όταν ολοκληρωθεί η έκδοση θα κυκλοφορήσει ανακοίνωση.

ΙΣΤΟΣΕΛΙΔΑ

Στα πλαίσια της διοργάνωσης του 15ου Πανευρωπαϊκού συνεδρίου θα δημιουργηθεί μέσω του γραφείου διοργάνωσης ιστοσελίδα του Συνεδρίου η οποία στη συνέχεια θα παραμείνει ως ιστοσελίδα της ΕΕΕΕΓΜ.

ΠΡΟΣΕΧΕΙΣ ΕΚΔΗΛΩΣΕΙΣ

Υπάρχουν σχετικές ανακοινώσεις στα «ΝΕΑ ΤΗΣ ΕΕΕΕΓΜ» τεύχος 10.

- Στα τέλη Ιανουαρίου 2008 προβλέπεται να δοθεί η 5η Αθηναϊκή Γεωτεχνική Διάλεξη. Ομιλητής θα είναι ο Ομότιμος Καθηγητής Ε.Μ.Π. Α.Αναγνωστόπουλος, Γεν. Γραμματέας της ΕΕΕΕΓΜ. Σχετική ανακοίνωση θα κυκλοφορήσει σύντομα.

- Στις 07.04.2008 προγραμματίζεται διάλεξη του καθηγητή A. Gens του Πανεπιστημίου της Βαρκελώνης (επανάληψη της Rankine Lecture 2007).

- Έχουν αρχίσει ήδη συνεννοήσεις με το ΤΕΕ για τη διοργάνωση του 6ου Πανελλήνιου Συνεδρίου Γεωτεχνικής και Γεωπεριβαλλοντικής Μηχανικής το 2010. Υπάρχει πρόταση να διεξαχθεί στην Κύπρο.

Αθήνα, 11.12.2007
Για την Εκτελεστική Επιτροπή,
Ο Πρόεδρος Ο Γεν. Γραμματέας

Μιχ. Παχάκης Α. Αναγνωστόπουλος

ΠΡΟΣΕΧΕΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΚΔΗΛΩΣΕΙΣ

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

"Thinkdeep – Amsterdam" International Symposium on Underground Space Challenges in Urban Developments, 28 -30 January 2008, Amsterdam, Holland - www.thinkdeep.nl

GeoAmericas 2008 – The First Pan American Geosynthetics Conference and Exhibition, 2 – 5 March 2008, Cancun, Mexico - www.geoamericas.info

GeoCongress 08 – The Challenge of Sustainability in the Geoenvironment, 9 – 12 March 2008, New Orleans, USA - www.geocongress.org

International Conference on Geotechnical Engineering ICGE'08, 28 – 30 March 2008, Tunis, Tunisia - www.enit.rnu.tn/fr/manifestations/ICGE08/index.html

3rd International Conference on Site Characterization, 1 - 4 April 2008, Taipei, Taiwan.

6th International Symposium "Geotechnical Aspects of Underground Construction in Soft Ground – IS - Shanghai 2008", 10 – 12 April 2008, Shangai, China - www.tc28-shanghai.org

European Geosciences Union
General Assembly 2008
Vienna, Austria, 13 – 18 April 2008
meetings.copernicus.org/egu2008

The EGU General Assembly will bring together geoscientists from all over Europe and the rest of the world into one meeting covering all disciplines of the Earth, Planetary and Space Sciences. Especially for young scientists the EGU appeals to provide a forum to present their work and discuss their ideas with experts in all fields of geosciences. The EGU is looking forward to cordially welcome you in Vienna!

Οι επί μέρους επιτροπές της συνέλευσης είναι:

ESSI – Earth and Space Science Informatics
ES – Educational Symposia
AS – Atmospheric Sciences
BG – Biogeosciences
CL – Climate: Past, Present & Future
CR – Cryospheric Sciences
ERE – Energy, Resources and the Environment
GMPV – Geochemistry, Mineralogy, Petrology & Volcanology
G – Geodesy
GD – Geodynamics
GM – Geomorphology
GI – Geosciences Instrumentation and Data Systems
HS – Hydrological Sciences
IG – Isotopes in Geosciences: Instrumentation and Applications
MPRG – Magnetism, Palaeomagnetism, Rock Physics & Geomaterials
NH – Natural Hazards
NP – Nonlinear Processes in Geosciences
OS – Ocean Sciences
PS – Planetary and Solar System Sciences
SM – Seismology
SSS – Soil System Sciences
ST – Solar-Terrestrial Sciences
SSP – Stratigraphy, Sedimentology & Palaeontology
TS – Tectonics and Structural Geology

Στα πλαίσια των εργασιών της επιτροπής ERE θα πραγματοποιηθούν οι συνεδρίες ERE10 «Natural stone resources for historical monuments» και ERE 11 «Aggregates – the most widely used geological material» με τα ακόλουθα αντικείμενα:

ERE10 «Natural stone resources for historical monuments»

Monuments built of natural stone make important part of cultural heritage. Unfortunately, our current knowledge of historical stone resources is incomplete. Authentic types of natural stone (or stone showing similar properties) must be often supplied during reconstruction of historical monuments.

This special session will focus on the following topics:

- Summary of recent knowledge on historical stone resources
- Lithotheques and electronic databases of historical stone resources
- Techniques used for the determination of natural stone provenance
- Methods applicable for exploration of historical quarries
- Criteria for re-opening of historical quarries and possible application of authentic stone resources during reconstruction of historical monuments
- Assessment of non-authentic stone types for replacement on historical monuments

Selected papers are planned to be published in a special issue of an international journal.

Convenor: Assoc. Prof. Richard Přikryl
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Phone: +420-221951500
E-mail: prikryl@natur.cuni.cz

Co-convenor: Assoc. Prof. Ákos Török
Budapest University of Technology and Economics, Department of Construction Materials and Engineering Geology, Sztoczek u. 2, H-1521 Budapest, Hungary
Phone: +36-1-4632414
e-mail: torokakos@mail.bme.hu

More information about the conference can be found on meetings.copernicus.org/egu2008/ and about the session on: www.cosis.net/members/meetings/programme/view.php?m_id=49&p_id=300&PHPSESSID=59ac5c47b3ef2c090caac1d183698a66>

ERE 11 «Aggregates – the most widely used geological material»

Subtle balance between sustainable development and needs of the society for the infrastructure means among others a meaningful utilisation of natural resources. Infrastructure is in many aspects roads, railways and houses, i.e. structures requiring huge amount on construction materials. Geomaterials - natural stone (dimension, aggregates) - represent dominant raw material used in current building sector. The annual world production of aggregates (crushed stone, sand and gravels) exceeds 16 billion tons which makes about 50 vol. % of all raw materials extracted from the Earth by humans.

This meeting is focused on any geological aspect of aggregates but namely on the on the following points:

- impact of aggregate quarrying on the environment
- correct use of aggregates (to meet specifications and needs of market)
- non-conventional materials and waste use as the aggregates
- testing of aggregates and quality demands (EU standards and other testing methods)
- behaviour of aggregates in specific environments
- alkali-silica reactivity and its testing
- influence of petrographical parameters and genetic factors on mechanical properties of aggregates

Selected papers are planned to be published in a special issue of an international journal.

Convenor: Assoc. Prof. Richard Přikryl
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More information about the conference can be found on: http://meetings.copernicus.org/egu2008/ and about the session on:
http://www.cosis.net/members/meetings/programme/

Fifth International Symposium on SPRAYED CONCRETE Lillehammer, Norway, 22 – 24 April 2008

www.sprayedconcrete.no/index.html

The Norwegian Concrete Society has been a leading participant regarding the use of wet-mix sprayed concrete for many years. It is therefore with great pleasure that we announce the Fifth International Symposium on the MODERN USE OF WET-MIX SPRAYED CONCRETE FOR UNDERGROUND SUPPORT. After holding the fourth symposium outside Norway, in Davos, Switzerland in 2002, the event is again going to be held in Norway in the city of Lillehammer.

We sincerely hope that many of our international colleagues will again take the opportunity to present papers, exchange experiences and discuss the latest developments in wet-mix sprayed concrete.

The main symposium themes will be design, construction and durability of wet-mix sprayed concrete in underground structures. In view of the latest incidences in Norway regarding failure in the rock support system in a highway tunnel, the support philosophy has been heavily debated in Norway. The results from the investigations, discussing the main reasons of the failure, will be presented in papers at the symposium.

Wet-mix sprayed concrete technology has been further developed to a much higher level since the first symposium in 1993, with the latest developments in chemical additives, application equipment and fibers. It will continue to change, and the fifth International Symposium in Lillehammer will ensure that delegates are kept abreast of the new developments in the use of this material.

The city of Lillehammer is situated 200 km north of Oslo, the capital of Norway. Lillehammer has approximately 25.000 inhabitants and is a modern city at the same time receiving awards for its architecture and protected buildings in the centre of the city. The city is internationally known as the host of Olympic Winter Games in 1994.

We are looking forward to see you at the symposium in Lillehammer in the spring of 2008.

The Symposium will focus on wet mix fibre reinforced sprayed concrete for underground support. There will be four main sessions consisting of invited and submitted papers, followed by discussions.

Support design
Geological conditions
Load capacity
Rock mass classification
Squeezing and swelling rock
Water leakage
Frost action
Fire performance
Rock burst and spalling
Design methods and criteria
Case histories

Durability
Environmental exposure
Early and long term capacity
Sprayed concrete mix design
Saline water
Alkali aggregate reaction
Frost
Deformations, loading
Critical parameters
Service Life prediction
Case histories of performance

Codes and specifications
Health and safety
Design of reinforcement
Concrete materials
Admixtures for sprayed concrete
Use of fibres, ductility
Quality Assurance
Nozzle man certification
Testing and documentation
Fire protection with sprayable system and sprayed concrete

Construction
Spraying equipment
Use of fibres and admixtures
Spraying technique
Water and frost protection
Case histories

Secretary:
Mrs. Siri Engen
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P.O.Box 2312, Solli
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Fax: +47 22 94 75 01
E-mail: info@sprayedconcrete.no

International Young Scholar Symposium on Rock Mechanics 2008, 25 – 28 April 2008, Beijing, China - www.isrm.net/eduacation

Geotechnical Earthquake Engineering and Soil Dynamics IV, 18 – 22 May 2008, Sacramento, Ca., USA – www.geesd.org

International Geotechnical Conference "Development of Urban Areas and Geotechnical Enfgineering", 16 – 19 June 2008, Saint Petersburg, Russia - www.georec.spb.ru/eng/conf/080616

2nd International Conference on Geotechnical Engineering for Disaster Mitigation and Rehabilitation (GEDMAR08), May 30 – June 2 2008, Nanjing, Chine - www.geohohai.com/news/english/2008/1.shtml

First International Conference on Education and Training in Geo-Engineering Sciences: Soil Mechanics and Geotechnical Engineering, Engineering Geology, Rock Mechanics Constantza, Romania, 2 - 4 June 2008 - www.ppm.ro/srgf

Development of Urban Areas and Geotechnical Engineering, 16 - 19 June 2008, Saint Petersburg, Russia - www.georec.spb.ru/eng/conf/080616/

Geosynthetics Asia 2008, 17 – 20 June 2008, Shangai, China - www.4acg-2008sh.com

Journées Nationales de Géotechnique et de Géologie de l'Ingénieur – JNGG'08 : Insertion des Grands Ouvrages dans leur Environnement, 18 – 20 June 2008, Nantes, France, www.ec-nantes.fr/jngggo8

2nd International Conference on Debris Flow
Debris Flow Monitoring, Modelling,
Hazard Assessment, Mitigation Measures,
Case Studies, and Extreme Events, Erosion,
Slope Instability and Sediment Transport
18 - 20 June, 2008 - The New Forest, UK
www.wessex.ac.uk/conferences/2008/debris08/index.html

Debris and hyper-concentrated flows are among the most frequent and destructive of all water relater processes. They mainly affect mountain areas in a wide range of morpho-climatic environments and in recent years have attracted more and more attention from the scientific and professional communities and concern from the public awareness, due to the increasing frequency with which they occur and the death toll they claim.

Higher population pressure on natural resources in hazard-prone areas and development of activities that have the potential to increase the magnitude of hazard call for improvements in the criteria used to identify debris flow risk areas and to design suitable prevention and mitigation measures.

The Conference will provide a forum for engineers, scientists and managers from laboratories, industries, governments and academia to interchange knowledge and expertise in the fields of erosion and slope instability, sediment transport, debris flow and debris flood data acquisition, debris flow phenomenology and laboratory tests, using the most advanced, state-of-the-art methodologies in monitoring, modelling, mechanics, hazard prediction and risk assessment.

Topics of the conference:

- Debris flow modelling
- Debris flow phenomenology
- Debris flow mobilisation
- Debris flow disaster mitigation
- Case studies
- Debris flow rheology and laboratory tests
- Debris flow and landslide phenomena
- Debris and hyper-concentrated flows
- Structures and their effects on debris flow
- Problems and mechanics of solid-liquid flows
- Shape failure and landslides

Conference Secretariat:
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Conference Manager / Debris Flow 2008
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2nd BGA International Conference on Foundations – ICOF 2008 "Founded on Research, Design and Practice, 24 – 27 June 2008, Dundee, Scotland, United Kingdom - www.dundee.ac.uk/civileng/icof2008

San Francisco 2008
42nd U.S. Rock Mechanics Symposium and 2nd U.S. - Canada Rock Mechanics Symposium, 29 June – 2 July 2008, San Francisco, CA, USA, www.armasymposium.org

10th International Symposium on Landslides and Engineered Slopes, June 30 to July 4 2008, Xi'an, China, www.landslide.iwhr.com

www.mercea08.org

The 2008 Seismic Engineering International Conference commemorating the 1908 Messina and Reggio Calabria Earthquake (MERCEA'08) is organized by:

- The University of Reggio Calabria
- The University of Messina
- ENEA (Italian National Agency for New Technologies, Energy and the Environment)
- C.I.Di.S (Centro Interuniversitario di Dinamica Strutturale Teorica e Sperimentale)
- The University of Palermo
- The University of Catania

The Conference will provide a forum to discuss the state-of-the-art, the best practices and the new research results in the field of earthquake engineering and geotechnics.

The Conference addresses the following topics:

1. The 1908 Messina and Reggio Calabria earthquake
2. Site characterisation, microzonation and site effects
3. Soil liquefaction and liquefaction countermeasures
4. Slopes, embankments, dams and waste fills
5. Foundations, and soil-structure interaction
6. Earth retaining structures and geosynthetics
7. Codes and guidelines
8. Structural engineering
9. Emerging technologies
10. Numerical methods
11. Passive protection devices and seismic isolation
12. Advanced technologies in construction and retrofit of structures
13. Seismic risk
14. Stochastic methods
15. Structural safety and reliability
16. Structural dynamics
17. Urban planning and policies for seismic risk reduction

MERCEA'08 Secretariat
Domenico Gioffrè, Maria Clorinda Mandaglio, Francesco Nucera
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6th International Conference on Case Histories in Geotechnical Engineering and Symposium in Honor of Professor James K. Mitchell, 11 – 16 August 2008, University of Missouri – Rolla - www.6icchge2008.org

2nd International Workshop on GEOTECHNICS OF SOFT SOILS, 3 – 5 September 2008, University of Strathclyde, Glasgow, Scotland, www.iwgss.org

EuroGeo4 - 4th European Geosynthetics Conference, 7 – 10 September 2008, Edinburgh, Scotland, United Kingdom - www.eurogeo4.org

milos.conferences.gr/?geoenv2008

The aim of this workshop is to provide a forum for the world's leading scientific and technical communities, to interact and address the main issues and the key challenges of the mining and civil construction industry in the beginning of the 21st century in order to reduce its environmental and health impacts, life cycle assessment, risk analysis, hazard detection and control, environmental and health consequences and liability, waste management, monitoring and projection techniques, geotechnical issues, propabilistic modelling, geoenvironmental engineering, dam and embankment design and case studies.

Topics

- Risk assessment in mining, metallurgical and waste disposal sites
- Life cycle assessment
- Hazard detection and control
- Remediation of contaminated soils and mining/metallurgical sites
- Mine quarry reclamation / revegetation
- Mine closure - post mining land use
- Environmental geochemistry of ore deposits, tailings and waste rocks
- Solid waste management
- Geotechnical aspects of mine waste
- Dam and waste dumps design, construction and stability
- Advanced modeling techniques in geotechnical and geoenvironmental engineering
- Advanced monitoring techniques (remote sensing, decision support and alerting techniques)
- Engineering geology applications
- Mine waste management - New EC directives

Conference Secretariat:
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Fax: +30 210 9767208
Workshop Secretariat E-mail: geoenv2008@heliotopos.net

"Stress Wave", 8 – 10 September 2008, Lisbon, Portugal, www.stresswave2008.org

5th International Geotechnical Seminar "Deep Foundations on Bored and Auger Piles", September 8 ÷ 10, 2008, Ghent, Belgium - terzaghi.ugent.be

1st International Conference on Transportation Geotechnics - www.nottingham.ac.uk/ncg

11th Baltic Sea Geotechnical Conference "Geotechnics in Maritime Engineering", 15 – 18 September 2008, Gdansk, Poland - www.11bc.pg.gda.pl

ITA – AITES World Tunnel Congress and 34th General Assembly of ITA – AITES, 19 ÷ 25 September 2008, Agra, India - www.cbip.org

The 12th International Conference of IACMAG - International Association for Computer Methods and Advances in Geomechanics, 1 ÷ 6 October 2008, Goa, India

AFTES – International Congress "Building underground for the future", 6 – 8 October 2008, Monaco - www.aftes.asso.fr

NUCGE 2008 – International Conference on Numerical Computation in Geotechnical Engineering, October, 27-29 2008, Skikda, Algeria - www.univ-skikda.dz/conference/accueil1.html

14th World Conference on Earthquake Engineering (14WCEE), 12-17 October 2008, Beijing, China - www.14wcee.org

ICSE-4 Fourth International Conference on Scour and Erosion, Tokyo, 5 - 7 November, 2008 - icse-4.kz.tsukuba.ac.jp

3° Πανελλήνιο Συνέδριο Αντισεισμικής Μηχανικής και Τεχνικής Σεισμολογίας, 5 – 7 Νοεμβρίου 2008, Αθήνα – www.civil.ntua.gr/3-PCEEES

The First World Landslide Forum - Implementing the 2006 Tokyo Action Plan on the International Programme on Landslides (IPL) - Strengthening Research and Learning on Earth System Risk Analysis and Sustainable Disaster Management within UN-ISDR as Regards "Landslides", 18-21 November 2008, United Nations University, Tokyo, Japan - www.iclhq.org

"Safe Tunnelling for the City and Environment" ITA-AITES World Tunnel Congress 2009 and the 35th ITA-AITES General Assembly, Budapest Congress and Word Trade Center, Budapest, Hungary, 23 - 28 May 2009 - www.wtc2009.org

Géotechnique SYMPOSIUM IN PRINT 2009, May 2009, www. geotechnique-ice.com

IS-Tokyo 2009 "International Conference on Performance-Based Design in Earthquake Geotechnical Engineering - from case history to practice", 15 – 17 June 2009, Tokyo, Japan.

☙ ❧

WCCE - ECCE - TCCE JOINT CONFERENCE

EARTHQUAKE & TSUNAMI

22 - 24 JUNE 2009

İSTANBUL – TURKEY

www.imo.org.tr/eqt2009

The World Council of Civil Engineers (WCCE), the European Council of Civil Engineers (ECCE) and the Turkish Chamber of Civil Engineers (TCCE) are jointly organising a conference on the civil engineering disaster mitigation activities concerning earthquake and tsunami to be held in İstanbul, Turkey on June 22-24, 2009.

All those interested in various aspects of the seismic risk reduction problem are invited to participate in the above mentioned WCCE-ECCE-TCCE Joint Conference. Papers reporting experimental or analytical research and those reflecting interesting practical applications will be most welcome.

OBJECTIVES

The major objective of the Joint Conference is to contribute to the mitigation of life and material losses in earthquake and tsunami through improved civil engineering practice. The emphasis of the conference will be on the Millennium Development Goals, through seismically safe schools, hospitals, dwellings etc., or more generally, seismically safe and sustainable built environment.

As the title implies, the scope of the conference is limited to the civil engineering related disaster mitigation activities concerning the problems of earthquake and tsunami.

CONFERENCE THEMES

- Lessons Learnt from Recent Earthquakes & Tsunamis
- Disaster Mitigation vis-à-vis the Millennium Development Goals
- Seismic Design Considerations and Codes
- Seismic Risk Assessment and Retrofitting of Existing Structures
- Earthquake & Tsunami Modelling

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☙ ❧

17th International Conference on Soil Mechanics and Geotechnical Engineering "Future of Academia & Practice of Geotechnical Engineering", 5 – 9 October 2009, Alexandria, Egypt - www.2009icsmge-egypt.org

IX International Conference on Geosynthetics, Brazil, 2010 - www.igsbrasil.org.br/icg2010

XVth European Conference on Soil Mechanics and Geotechnical Engineering, 13 – 19 September 2011, Athens, Greece.

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ΝΕΑ ΑΠΟ ΤΟΝ ΚΟΣΜΟ

Modern 'Silk Road'

Eight countries in Central Asia have signed-up to a US$ 18.7 billion strategy to develop transport infrastructure in the region. Afghanistan, Azerbaijan, China, Kazakhstan, Kyrgyz Republic, Mongolia, Tajikistan and Uzbekistan, supported by the Asian Development Bank (ADB) and other lending institutions want to make the region a vital transit route for trade between Europe and Asia – a modern-day equivalent of the ancient Silk Road.

According to the ADB, although central Asia lies at the centre of the Eurasian continent, less than 1% of all trade between Europe and Asia currently goes through the region. Inadequate transport infrastructure and cumbersome border processes have resulted in nearly all trade going by sea.

The plan unveiled this month calls for US $18.7 billion to be invested over the next decade in six new transport corridors, mainly roads and rail links. About half of the funds are likely to come from multilateral organisations like ADB, while the rest will come from the countries themselves.

The plan also calls for the improvement of border crossings to speed trade flows. Customs and immigration procedures are currently bottlenecks for trade in the region.

"This is a large and ambitious strategy. It encompasses dozens of projects and will require more than US$ 18 billion in investments over the next decade," said ADB President Haruhiko Kuroda.

Silk Road

Historically, Central Asia was crossed by multiple routes linking east and west, known as the Silk Road and dating back more than 2000 years. It was an important economic artery that stretched more than 10000 km from the Mediterranean to China's Yellow River Valley.

The proposed new transport corridors do not follow the exact routes taken by the Silk Road and will not only be orientated east-west, but also north-south, connecting the Central Asian Republics, Russia and China with South Asia and the Gulf.

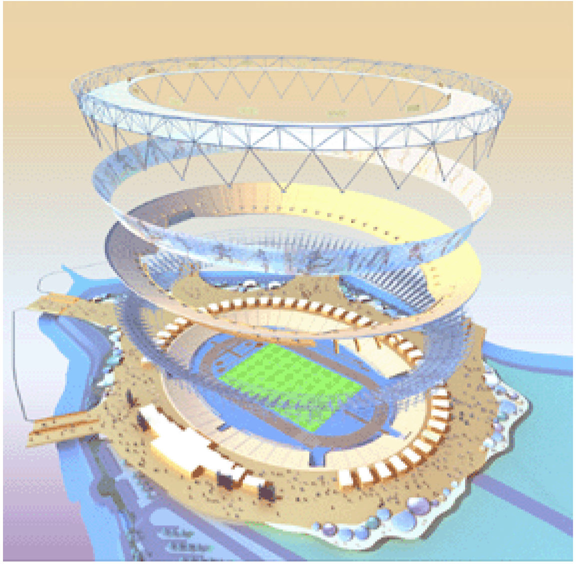
"Creation of safe and reliable transport corridors, together with measures on simplified trade relations, will provide an opportunity to improve the general investment climate and to increase the rate of economic development of the region," said Tajik president Emomali Rakhmon.

(INTERNATIONAL CONSTRUCTION, November 6, 2007 Editor: Chris Sleight)

Olympic Stadium design unveiled

The design for the main stadium for the 2012 London Olympic Games was unveiled today. Following the games, the 80000 capacity venue will have 55000 seats removed, converting it to a smaller permanent sports facility.

The stadium will be built by the Team McAlpine consortium, which is led by contractor Sir Robert McAlpine, and includes HOK Sport, Buro Happold, HED and Savills Hepher Dixon.



The Olympic Delivery Authority is the public body responsible for organising the 2012 games and is client for the project. It is chaired by John Armitt, formerly of Costain, who said, "London's Olympic Stadium is designed to be different. 'Team Stadium' have done a fantastic job against a challenging brief - their innovative, ground-breaking design will ensure that the Olympic Stadium will not only be a fantastic arena for a summer of sport in 2012 but also ensure a sustainable legacy for the community who will live around it.

The central 'bowl' of the stadium will be sunk below ground level to bring spectators close to the action. A 28 m span, cable-supported roof will cover two thirds of the seated area, while a fabric curtain will wrap around the structure providing additional protection and shelter.

Besides the novel use of 55000 demountable seats, facilities such as catering and merchandising will be grouped into self-contained 'pod' structures, adding to the spectator experience around the access level of the Stadium.

(INTERNATIONAL CONSTRUCTION, November 6, 2007 Editor: Chris Sleight)

Burj Dubai reaches 601 m

Concreting elements at the Burj Dubai have this week topped out at 601 m making the structure the tallest tower in the world.

Contractors JV of Samsung-Besix-Arabtech have completed the structure core inside the original timetable despite using an unusually high proportion of Doka wall-forming systems for the construction of a skyscraper.

The honeycombed structural design, with many airframe-like stiffening reinforcements, necessitated the forming of 430000 m² of wall - twice as much as for the floor elements.

Samsung's project director Kyung-Jun Kim, said, "As the in-situ concrete core was being built ahead of the floor-slabs, construction progress on the whole building was entirely dependent on the self-climbing Doka formwork solution."

The final height of the Burj Dubai is expected to reach around 800 m when it is completed at the end of the year.

(INTERNATIONAL CONSTRUCTION, November 13, 2007 Editor: Becca Wilkins)

Catching up with gypsum

One form of common demolition waste is currently generally being under-recycled – plasterboard. In the UK, the Waste & Resources Action programme (WRAP) is spearheading efforts to encourage improved reclamation of this gypsum-based interior wall paneling.

According to Dave Marsh, WRAP's material recycling project manager, there are two obvious main sources for plasterboard waste – installation (off-cuts and damaged boards) and removal during refurbishment or demolition. Precise figures are hard to obtain, but Mr Marsh told D&Ri that he estimates that 300,000 tons is generated during the former operation, with another 600,000 tons coming from the latter.

With additional materials coming from other sources, it is therefore reasonable to estimate that every year close to 1 million tons of gypsum product waste is generated in the UK alone, although he was at pains to point out that there is a rather large possible margin of error to these figures.

Interestingly, it is WRAP's view that most if not all of this material is readily recyclable to the point where the resultant product can be used in a variety of applications, from incorporation into the plasterboard production process, unfired clay block production, cement manufacture and numerous others.

However, despite this, the vast majority of the UK's plasterboard waste ends up dumped in landfills. Mr Marsh estimates that there is currently around 500,000 tons of recycling capacity available on an annual basis in the UK, but that throughput is only around 250,000 tons, i.e. a recycling rate of only 25% and a substantial unused capacity. "There is a long way to go," he said, "lets put it that way."

Given the UK demolition industry's recent (and not so recent) track record of relatively high recycling rates for the more readily recyclable products, this figure has to be a disappointment. So why is the rate so low?

Hurdles to overcome

There are three main barriers to the recycling of plasterboard waste. The first is cultural: "There is still a perception that collecting plasterboard waste for recycling is not feasible or is too costly – this is not true and WRAP is currently conducting a lot of work to overcome this barrier," said Mr Marsh.

The message that the organisation is trying to get over to the construction industry is that while disposal may appear to be the cheap option, this is actually not the case. What is often not taken into account is the costs involved over and above the cost of hiring a waste container – the value of the material being thrown away, the labour required to handle it, the cost of landfill etc, Overall, the true cost of disposal could be up to 10-15 times the hire cost of the waste container and the cost is increasing, with the UK's landfill tax now being ramped up year on year. Currently it stands at £24 (US$50) per ton, rising to £32 (US$66) next year and by 2011 it will be £48 (US$100).

The second barrier is related to on-site working. "Often, those actually working the site are not engaged in high-level policies of reducing waste to landfill, often as a result of poor communication." Frequently a site often will have a waste segregation system in place but unless site workers can see a good reason to follow the scheme there is very little incentive for them to follow it, with waste therefore ending up in the nearest container.

WRAP is therefore working on good practice guidance for on-site works and encouraging companies to communicate this through to its site employees.

The third barrier is one of lack of infrastructure. Plasterboard recycling is still at a relatively immature stage. There are only a few recycling facilities for this material type and as a result, frequently the material has to be carried considerable distances from the site, imposing greater costs and creating transport issues. WRAP has initiated two capital competitions targeted assisting the development of the required infrastructure. "Building up the infrastructure will make the transport of plasterboard waste more easy and more cost-effective," said Mr Marsh.

Currently, there are two main routes for recycled materials – a take-back scheme operated by plasterboard manufacturers with certain contractors (but this only applies to new plasterboard waste from construction projects) and a handful of independent recycling centres around the country. WRAP's capital scheme is aimed at increasing the number of these independents, as well as enhancing the capabilities of existing facilities.

Contamination concerns

Contamination is a big issue when it comes to recycling plasterboard but not perhaps in the way readers might think. In this instance, contamination is taken to mean when the waste reaching the recycling centre contains too much other debris, such as wood, metal, plastics etc, In this event, it is possible the recycling centre might reject the whole load and return it to the site whence it came, or sort it themselves and pass the cost of doing so on to the sender, or even possibly dispose of the waste to landfill with the initial consignee again bearing the cost.

Waste plasterboard from demolition activities has in the past been perceived as being more likely to be contaminated with other materials as above, and this perhaps one reason why a greater percentage of waste material from this source is not being currently recycled by demolition contractors. Plasterboard recyclers have initially been targeting the waste from new construction, since it is generally perceived as being cleaner and freer from contamination by other materials.

However, contamination issues can be relatively easily be addressed by following a good material segregation regime on site, even where waste from demolition operations is concerned. As Mr Marsh says: "If you have a plasterboard waste container that is a third full of metal studding, then the odds are there is also a scrap metal stockpile/container on site, and you are loosing the potential scrap value of that metal studding. Good site practices would see the metal studding being place in the appropriate place, rather than with the plasterboard."

Another obvious point is to keep the waste plasterboard dry – plasterboard is very good at taking up any available moisture, so any container should be well covered to prevent the ingress of moisture. "In one study, we discovered that a plasterboard waste container left uncovered overnight gained 10% in weight and that increase in weight instantly goes on the disposal coast," said Mr Marsh.

The actual process to produce a quality end product has to be carried out in quite a controlled way, using equipment that is similar to standard crushers, tuned and set up to match the requirements of plasterboard. According to Mr Marsh: "It could be feasible to do it on site, but at the moment the route we would encourage people to follow would be to undertake recycling in a purpose designed facility. There are shredders being marketed to process plasterboard on site but the problem I see is that the material they produce is in a form that is not readily reusable on site."

Some of the work that WRAP is currently engaged in is looking at the use of recycled gypsum products in this way, for construction road bases and other similar applications but this is using gypsum produced in a dedicated facility and the reuse of material processed on site has yet to be addressed.

Chipping away

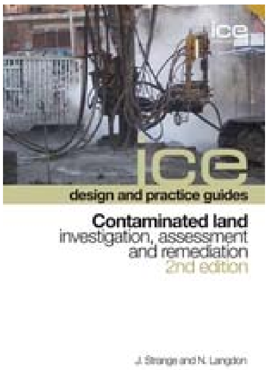
From the early days of WRAP's plasterboard process, it had always been aware of the fact that when plasterboard is removed during demolition or refurbishment, the pieces were invariably quite large and when these are placed in a waste container, a large number of voids result. This is not the most efficient way of handling waste, since the result is a lot of volume but not a lot of weight, which in turn means greater transport being required for a given weight of waste.

"This was an area we felt we ought to address," said Mr Marsh. "Taylor Woodrow undertook a trial on our behalf and identified a plasterboard chipper that was under development that cut waste plasterboard into small pieces, rather than a shredding process." The result is that more material could be placed into waste containers. The trials conducted by Taylor Woodrow demonstrated that using such a chipper could deliver considerable savings on site, in terms of labour costs and also carbon savings.

WRAP has now published a report on the project. For further information on this, and other WRAP initiatives, visit www.wrap.org.uk

(DEMOLITION AND RECYCLING INTERNATIONAL, November 21, 2007, Editor: Lindsay Gale)

ΝΕΕΣ ΕΚΔΟΣΕΙΣ ΣΤΙΣ ΓΕΩΤΕΧΝΙΚΕΣ ΕΠΙΣΤΗΜΕΣ



ICE design and practice guides: Contaminated land - investigation, assessment and remediation

2nd edition

J. Strange and N. Langdon

The first edition of this ICE design and practice guide became one of the most popular ICE publications after it was published in 1994. Since the first edition, there has been a multitude of legislation on the environment and the adoption of many of the principles of the European Landfill Directive. This has meant specialists, regulators and engineers are grappling with interpretation of guidance frameworks, new levels of testing accuracy and increased public perception of environmental damage. This new edition provides an up-to-date overview of the main principles and important aspects of a complex subject.

The first part of the guide sets the use of the investigation methods within a risk management context and highlights those aspects where different techniques or a different emphasis is needed to ensure that contamination is adequately addressed. The guide describes risk assessment as a means of evaluating the significance of any contamination identified, and looks at the development of the Conceptual Site Model as part of the assessment process.

The second part of the guide outlines the methods for setting targets for remediation, and explains the overall selection process needed to determine the most appropriate remediation strategy. Alongside the traditional civil engineering techniques used for remediation in the last twelve years, this guide lays out the new innovative solutions to achieving cost effective and technically sufficient investigation, assessment and remediation today

(Thomas Telford / ICE, November 2007)

BodendynamikGrundlagen, Kennziffern, Probleme und Lösungsansätze

Studer, Jost A., Laue, Jan, Koller, Martin

Die Bedeutung der Bodendynamik hat in den letzten Jahren erheblich zugenommen. Erhöhte Sicherheits- und Komfortansprüche erfordern bei der Bemessung von Bauten und Anlagen die Berücksichtigung dynamischer Lasten aus Erdbeben, Wind oder industriellen Aktivitäten. Dies verlangt moderne und praxisgerechte Berechnungsverfahren zur Ermittlung des Deformations- und Festigkeitsverhaltens von Fundationen und Erdbauwerken.

Die neu bearbeitete dritte Auflage des Buches stellt die wesentlichen physikalischen Grundlagen dar, zeigt, wie die grundlegenden Kennziffern ermittelt werden und gibt deren praktisch wichtige Wertebereiche an. Lösungsansätze für die wichtigsten Problemstellungen in der Praxis werden aufgezeigt. Zum besseren Verständnis sind die mathematischen Berechnungen so einfach wie möglich gehalten.

(Springer, 2008)

ΠΕΡΙΟΔΙΚΑ ΓΕΩΤΕΧΝΙΚΟΥ ΕΝΔΙΑΦΕΡΟΝΤΟΣ

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