General Report - Technical Session 3C, Interactive design

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SYNOPSIS Nine papers were submitted to this Session. Notwithstanding the high quality of all of them, one may observe that some show interesting aspects of the problem, but are not actual examples of interactive designs. Besides, in order to retain, as much as possible the originality of the papers, the reporter is going to use languages, words and phrase constructions, very similar to those used by the authors.

1 INTRODUCTION

The theme of this session is Interactive design.

The general reporter presumes that all participants know what an interactive design is. However, it seems appropriate to show some facts that will make clear the importance of this approach.

Poulos (1999) lecturing on foundation settlement analysis said: ..…" *it is vital to recognize that the ultimate success of settlement prediction depends as much (if not more) on appropriate modeling and parameter selection than on the method of analysis used.* In conclusion, it is sobering to recall the following comments of Terzaghi (1951): *"…foundation engineering has definitely passed from the scientific state into that of maturity…….. one gets the impression that research has outdistanced practical application, and that the gap between theory and practice still widens".*

The reporter observes that in many cases, proper modeling of a problem and establishing reasonable parameters for the soil is, by no means, an easy task. In these cases, the observational method, together with reasonable computational tools, may be the only way leading to safe and economical design.

One example of the gap between academics and practicing engineers may be found in the Pile Capacity and Settlement Prediction Event, held during the SEFE V*, in 2004, São Paulo, Brazil.

Four root piles were loaded to failure, two of them under compression and the other two under tension.

The organizers of this prediction event asked the predictors to estimate the ultimate capacity of the piles and the deformation at their top for a load corresponding to half the predicted ultimate value (safety factor of two). Twelve engineers submitted their predictions.

Let's use a simple case as an example: The estimating of the settlement of one of these piles loaded under compression, for a load 50% of the ultimate one.

Predictor Nr. 8 wrote 38 pages, used thirteen methods for computing the ultimate bearing capacity and a program based on finite elements method for predicting the deformation of the pile top.

A radically different approach was used by Predictor Nr. 6 who simply stated "The values of deformations for 50% of the failure loads are going to be rather small and their analytical prediction, very difficult. Empirically, the author estimates that the order of magnitude is going to be 1,0mm for piles loaded under compression and 5,0mm for piles loaded under tension.

Predictor Nr. 8 was a full time Professor (therefore 100% academic) while predictor Nr. 6, although being also an University Professor, was rather a practicing engineer.

The ratio of predicted to measured settlements varied from 1.45 up to 176.67 with an average value of 48.81. More specifically, for predictor Nr. 8 (100% academic) this ratio was 150 and for predictor Nr. 6, (professor + practicing engineer) the ratio was 1.45.

Although being a relatively simple problem, this shows very clearly how difficult it is to make good predictions. The use of sophisticated methods is by no means a guaranty for obtaining reliable results.

As previously stated, the success of any prediction depends basically on the correct modeling of the problem and on the adequate selection of parameters to be used in the computation.

Again, the above mentioned example is of a very simple case. In engineering practice, one has, sometimes, to deal with much more complex problems.

In these cases, the interactive design, based on the observational method, seems to be the only possible alternative for a safe and economical design.

It is quite evident that predictor Nr. 6 relied mostly on his own experience, which probably was based on an intensive use of observational approaches.

Predictor Nr. 8, on the contrary, probably had very little practical experience and therefore, relied almost 100% on theories and on sophisticated computational methods.

Further comments are clearly unnecessary.

*SEFE is a seminar on special foundations, held every four years in São Paulo, Brazil.

2 REVIEW OF THE PAPERS

Remedial works on landslide in complex geological conditions.

Arbanas, Benac and Grošić**,** present a case of a landslide that occurred on a slope alongside a highway cut, during the construction of the Adriatic Highway. Through lateral expansion, the landslide also affected the partly finished highway cut. A high level of underground water, the hydro

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geological properties of the layers and an unfavorable slope of the contact plane caused the sliding. Based on the results of additional investigation works, a landslide remediation project was made. The designed geotechnical structures and potential corrections were made possible through the observational method.

To determine appropriate strength parameters for the breccia deposit, back analyses were carried out. As a representative cross-section the profile P186 (Figure 5) was chosen. Based on back analyses using limit state slope stability analysis (GEO-Slope 1998) the following average Mohr-Coulomb's strength parameters were suggested.

- Angle of internal friction $\phi=26^{o}$
- Cohesion c=3 kPa.

In order to confirm the chosen solution, stability analyses have been carried out. The analyses were performed using parameters of rock masses obtained from back analyses.

The measured data enabled the control stress-strain back analysis. Based on the results of these back analyses an active design procedure was established which made possible the required changes in the support construction if observations indicate unacceptable deformations. Measured strains on installed measurement equipment showed a good match with the predicted/calculated strains and the construction was successfully finished in the spring 2007.

COMMENTS AND/OR QUESTIONS

1. It is not clear for the reporter if the project for the landslide remediation was carried out on basis of the results of additional investigational works or on the observational method.

2. What do the authors consider unacceptable deformations?

Design of engineered slopes in flysch rock mass.

Brunčić, Arbanas and Kovačević, present a method for the design and construction of systems for ensuring stability of cuts in flysch formations, using an interactive approach based on stress-strain back analyses. Flysch formations can be regarded as materials in a transitional zone between soft rocks and hard soils. Based on results obtained in laboratory and in-situ investigations in various stages of design and realization of projects, it could be concluded that the strength and deformability parameters for flysch could not accurately be determined by traditional methods, and therefore it would be very difficult to adopt realistic design parameters.

Relevant data about flysch displacement, data about rock mass quality in side cuts, and test data for anchors, were used in stress and strain back analyses. Strain properties of flysch could vary in order to harmonize measured and calculated displacements at side cut locations. The results of these analyses either confirmed the predictions or pointed out to the need to modify the design support systems in order to ensure stability of side cuts in flysch formations.

The measured data enabled the control stress-strain back analysis to confirm parameters for describing the real behaviour of excavated and reinforced rock mass. Based on the results of these back analysis an active design procedure was established which made possible the required changes in the rock mass support systems if the observations indicate unacceptable deformations.

COMMENTS AND/OR QUESTIONS

- How big a deformation should be in order to be considered unacceptable?

Full-scale horizontal pile tests in a residual dolomite profile.

Jacobsz and Vorster**,** present results of horizontal pile load tests carried out on instrumented bored piles in order to back-calculate parameters for foundations analyses.

Near the surface the soil profile consists of gravels and boulders, in a matrix of silty fine sand, dense to very dense. This overlies a zone of Weathered Altered Dolomite (WAD) comprising a silty low-density material highly susceptible to erosion, frequently containing disseminated voids. The WAD is underlined by an extremely irregular dolomite bedrock profile characterized by pinnacles, deep gullies and cavities.

The stiffness of residual dolomitic materials has traditionally been assumed to be low. The available database of stiffness was generally derived from plate load tests and laboratory tests.

- The observed pile deflections showed the stiffness of the residual dolomite profile to be considerably higher than traditionally assumed. The reason is that, in the case of the pile tests, the small-strain stiffness of the ground plays a significant role, whereas, in the case of plate load tests, much higher strains and hence lower soil stiffness control the behaviour. Designs based on plate load results in which small strain stiffness is not recognised, are therefore likely to be very conservative and uneconomical.
- The modulus of sub-grade reaction calculated for cracked and non cracked piles were of similar magnitude.
- The calculated modulus of sub-grade reaction against the pile shaft was found to remain approximately constant with increasing load.
- For the pile geometry analysed, the values of the modulus of sub-grade reaction and the Young modulus can be taken as numerically equal, despite the difference in units.

COMMENTS AND/OR QUESTIONS

1. Observe that in figure 3 the tittles of the axes have been exchanged.

2. Linear elastic theory is not adequate to predict or adjust the behavior of foundation under lateral loading. A non linear approach is much better.

3. The conclusion that the modulus of sub-grade reaction against pile shaft remains approximately constant with increasing loads, seems to be unreasonable.

Monitoring of shear strain in shallow sections of slopes to detect increased risk of slope failure.

Tamate and Itoh, emphasize that slope failures frequently cause labour accidents in construction sites and therefore must be avoided. It is also known that even a small collapse can cause serious injury to workers. In particular, many failures occur in the process of vertical cuts at the toe of slopes. Slope failure must be avoided. Consequently, temporary retaining walls are needed to support slopes.

Practices for immediate escape are also important to save workers' lives, and warnings must be given prior to failure. Therefore, monitoring the slopes is needed in order to detect increment of the potential risk of failure. A simplified monitoring device, the transducer (*SPS*), was developed by the authors to measure the increment of shear deformation in shallow section of slopes, through prototype model tests and centrifuge model tests.

Prototype model tests were carried out in several sets of model slopes, which were composed of loose sand and soft loam. An inclination of 45˚ and a height of 5m were given to the model slopes. *SPS* was set into the shallow section of slopes, near the shoulder. Vertical cuttings were carried out from the toe of slopes to simulate failure. Centrifuge model tests were also conducted by giving equivalent conditions to prototype model tests to provide additional test data of *SPS*.

Responses of strains (r_s) were measured with the progress of slope cuttings in both the prototype model tests and the centrifuge model tests. In the slope of loam, similar reactions to the creep strain curve were observed. A couple of minutes' time could be provided for escape by identifying either a 2nd creep or a 3rd creep. In the slope of sand, a linear relationship was confirmed between r_s and the settlement at the top of slopes. *SPS* shows similar reactions to conventional transducers for monitoring.

Accordingly, a relationship between the increment of shear deformation in the shallow section of slopes and the increase of the potential risk of slope failure was confirmed. It became clear that monitoring of shear deformations at the works provided an aid the usual safety inspections.

Figure 7 shows another relationship between r_s and t_e prior to failure at Cs2.

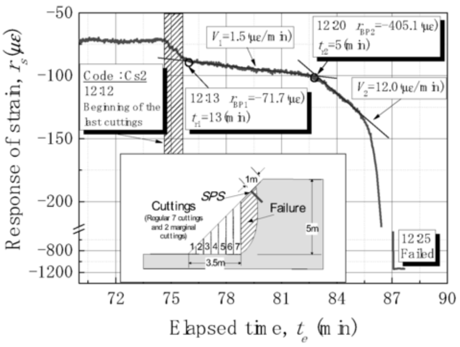

Figure 7. Relationship between r_s and t_e prior to failure at Cs2

The experiments showed that shear strain in the shallow section of slopes increased in association with an increase in the risk of slope failure. They also confirmed that an increase in shear strain corresponds to settlement on the crown of slopes.

The *SPS* would be effective in monitoring increased risk of slope failure in slopes that show a creep strain curve prior to failure.

COMMENTS AND/OR QUESTIONS

It would be very interesting to have data regarding to the use of SPS in other types of soil.

Comparison of soil settlement estimates with finite element method and manual computations.

Li and Wang, compare SPT settlements predictions in soft clays on basis of the conventional one-dimensional primary consolidation analysis and a three-dimensional consolidation-analysis, the latter supposed to be more adequate.

The case analyzed is of a runway close to an airport, with 4.000m in length and 60m in width.

The main conclusion of the authors is that the finite element analyses do not have significant advantage over hand calculations regarding the accuracy of consolidation settlements.

As far as other aspects of the paper are concerned, the report found some apparent inconsistencies in some statements of the authors, as follows;

The airport authority required that the settlement of the runway be less than 5 cm occurred in 30 years after construction.

But in table I, the manually computed settlements are compared with measured ones.

Table I. Manually Computed Settlement and Measured Data (Wang 1995)

Monitoring plate no.	Manually computed settlement (cm)	Measured settlement (cm) in 5.5 months after preloading sands stockpiled (cm)
2-1	55.6	52.4
1-2	33.5	31.1
2-3	55.4	50.6
2-4	55.2	51.2
2-12	55.2	51.5
2-14	55.7	51.5
2-15	53.8	50.5
1-13	44.7	40.2
1-16	40.6	37.8

As one can see, the order of magnitude of the settlements is 50,0cm and not 5,0cm.

Data comparisons indicated that hand calculations based on one-dimensional consolidation theory seems to have a trend to slightly underestimate the consolidation settlement than that based on the finite element analysis utilizing elastic porous model and modified Drucker-Prager model, though all differences are within 10%.

One may then conclude that the finite element analysis yield to computed settlements higher than those obtained through the one dimensional analysis.

The data from table I indicate that actual settlements are always smaller than those computed on basis of hand calculations.

In this case, the obvious conclusion is that the finite element calculations gave worse results than the hand calculations.

Last, but not least, according to the authors the parameters provided by geotechnical exploration reports were reasonably adjusted in practice in accordance with experiences of local similar projects, and in conjunction with computation efforts until the estimate results utilizing the two formulas were very close.

So, the reasonable predictions have been made possible because the parameter have been adjusted.

In engineering practice, as stated in the beginning of this report, one of the major problem is the parameter selection.

The errors usually made in such selections are probably, much more important than the differences in the results given by the two methods.

Monitoring of deep excavations in Istanbul.

Saglamer and Aslay, present two cases of deep excavations in downtown Istanbul.

Case 1 is a hotel building and two towers for residences. They are surrounded by metro station and high rise buildings.

The soil consists of two layers. An upper heterogeneous fill and a basic layer of the thrace formation which may be classified as a soft rock.

No water level was found. The excavations were 310,0m x 65,0m in plan with depths of 20,0m at the north and 40,0m at the south.

The building has 10 basements.

The retained structure consists of bored piles tied back with anchors.

While the excavation was 25,0m on the west side, inclinometer measurements indicated a sudden increase in lateral deformations. This anomaly was credited to a layer of mudstone with a high swelling potential not detected during soil investigations.

Case 2 is the Congress Valley Project. The soil profile is similar to case 1, and the problem was also associated with expansive mudstone not detected in the borings.

They present the remedial measures taken to reduce the deformations.

Monitoring helped the control of movements and allowed corrections to be made before unacceptable deformations might take place.

COMMENTS AND/OR QUESTIONS

Case 1

- The excavations were 40,0m deep, but in figure 7, the analysis were limited to a depth of about 20,0m. As it is well known deformations tend to increase with the depth of the excavation. So, it would be interesting to know what happened when the maximum depth of the excavation was reached.

- It is unacceptable that in both cases, the very important mudstone layer has not been detected in the borings.

- Do the authors have any explanation for this?

Trial of geotechnical asset management for highway embankments constructed on soft clay foundations.

Takeyama, Okubo, Yokota and Omoto, describe a trial geotechnical asset management for highway placed on very soft clayey grounds at Ebetsu, Hokkaido in Japan. The highway was constructed 30 years ago and is still settling year by year requiring a considerable maintenance.

They carried out class B predictions of the mechanical behaviour of embankments during construction works employing a soil/water coupled finite element code, predictions of long-term settlement of the embankments based on the information obtained at the stage, estimates of maintenance cost of the embankments based on the computed long-term settlement and verification of the proposed method of geotechnical asset management by comparing the maintenance cost estimated based on the above stated method and the maintenance cost actually needed in the past 30 years.

Class B predictions of the possible performances of highway embankments placed on very soft subsoil evidenced in this paper that the current technique of soil/water coupled finite element analyses are reliable enough to be used in long-term prediction of settlement of highway embankments on soft clay foundation.

Confortement d'un silo de ciment par des pieux vibro-battus dans le port de Douala.

Depardon et Charle, présente le cas d'un nouveau silo de stockage de ciment dans la zone portuaire de DOUALA, Cameroun. Lors du premier remplissage tassements importants absolus et différentiels ont été observés au niveau de la longrine support du voile métallique périphérique, du côté le plus proche des berges du fleuve Le Wouri. La valeur maximum de ces tassements a atteint lors de ce chargement 140 mm sur la longrine et environ 320 mm dans le fond du silo. Cette longrine en béton armée est fondée sur pieux forés en béton de 30 mètres de profondeur dans un site portuaire constitué par des sols sableux compressibles et liquéfiables.

Des estimations de tassement sous la charge du stockage (85 kPa sur les bords à 170 kPa au centre) et de la capacité portante des pieux ont été recalculées à partir des méthodes basées sur les essais in situ et par les méthodes traditionnelles basées sur les essais de laboratoire.

Tous les éléments concourent au diagnostic suivant:

tassement des sols compressibles atteignant 40cm, principalement dus à la couche B de sables vasards jusqu'à 10m de profondeur,

fluage latéral des sols mous vers le fleuve laissant craindre un risque de poinçonnement ou de glissement circulaire du stockage et de la couche B,

enfoncement d'une partie des pieux lié d'une part à une insuffisance d'encastrement dans la couche D et d'autre part au frottement négatif engendré sur une très grande épaisseur par tassement des couches B et C qui entrainent les pieux vers le bas.

hétérogénéité des sols avec des caractéristiques mécaniques plus faibles dans la zone proche du fleuve, ce qui explique les tassements différentiels.

De plus, les pieux sont soumis à des efforts horizontaux importants. Ils ont peut-être permis d'éviter une rupture par cisaillement circulaire de la couche compressible.

Le silo était en situation critique.

La solution retenue est une reprise des fondations par battage de pieux métalliques en H de 45 mètres de profondeur, à l'extérieur du silo, liaisonnés ensuite à la longrine béton par l'intermédiaire de platines.

La méthode appliquée est celle décrite dans le guide français « La méthode observationnelle pour le dimensionnement interactif des ouvrages » (Allagnat et al 2005), appliquée à la réhabilitation d'ouvrages anciens, conformément aux prescriptions de l'Eurocode 7, Calcul Géotechnique.

Dans le cas présent le dimensionnement des pieux conduit à des profondeurs de base de 45m pouvant être adaptées et approfondies si nécessaires en fonction des courbes de battage. Les valeurs de nombre de coups de battage minimales ont été établies.

Pour le risque de déformations de l'ouvrage dû à la liquéfaction et à l'entrainement des sols sous les effets de vibration et de battage des pieux il a été défini des valeurs de seuils des tassements de la longrine :

Seuil d'alerte : w1 = 2mm au cours du battage d'un pieu
Seuil d'arrêt : w2 = 5mm pour un pieux ou groupe de pieux
w3 = 25 mm en cumulé

COMMENTAIRES ET/OU QUESTIONS

1. Le probléme principal était que dans le projet original, le frottement négative n'avais pas été jugé corretment.

Les ouvrages de soutènement de Troinex sur l'autoroute A41 Nord, un exemple de conception interactive.

Laurent, Vetillard et Quandalle. présente le cas de les ouvrages de soutènement de Troinex, partie de la liaison autoroutière de montagne française A41 Nord. La conception et la construction ont été confiées à un concessionnaire privé. Des délais exceptionnellement courts, ainsi qu'un contexte géotechnique et environnemental complexe, ont été des challenges pour une équipe d'ingénierie intégrée au groupement de constructeurs. La solution retenue a consisté en des soutènements par longrines en béton et par remblai renforcé, ancrés au substratum par des tirants précontraints, avec un réseau de tranchées drainantes et de drains subhorizontaux maîtrisant l'eau souterraine tout en préservant une zone humide.

Un processus de validation, incluant l'acquisition de données géotechniques complémentaires et le suivi en continu de l'évolution du versant et des travaux, a été associé à la solution. Grâce à une interactivité forte entre concepteurs et constructeurs, elle a ainsi pu être efficacement ajustée aux conditions réelles au fil de la construction. Un dispositif de surveillance du site en service, associé à des seuils d'alerte et des axes de sécurisation en cas de problème, a par ailleurs été mis au point

Le cadre organisationnel de l'opération a permis à une équipe d'ingénierie, de constructeurs et de concessionnaires travaillant en collaboration étroite, de définir une solution technique optimisée, facile d'entretien et d'auscultation, et évolutive en cas de nécessité, tout en respectant des délais extraordinairement courts pour la conception et la construction d'ouvrages de soutènement sensibles dans un site complexe.

COMMENTAIRES ET/OU QUESTIONS

- Quelle vitesse de formation est considéré comme critique?

REFERENCES

Carvalho, D; Albuquerque Jr., P. J. R.; Nogueira, R. C. R.; Paschoalin Filho, J. A.; Garcia, J. R.; Fontaine, E. B., 2004. Contest for Predicting the behavior of root piles. *SEFE V, São Paulo, Brazil.*

Poulos, H. G., 1999. Common procedures for foundation settlement analysis – Are they adequate? K*eynote lecture Australian - New Zealand.Conference on geo-mechanics.*

Terzaghi, K., 1951. The influence of modern soil. Studies on the design and construction of foundations.