Technical Committee 206
Interactive Geotechnical Design

Comité technique 206
Dimensionnement géotechnique interactif
General Report for TC206
Interactive Design

Rapport général du TC206
Le dimensionnement géotechnique interactif

Ho A.
Ove Arup & Partners Hong Kong Ltd

ABSTRACT: This General Report is to summarise the all papers submitted for TC206 – Interactive Design. A total of 15 papers were received and 6 papers were recommended for oral presentation and the rest for recommended for panel presentation at the 18th ICSMGE Paris, 2-6 September 2013. The submitted papers gave a general picture of the interactive design works around the Globe ranging from experimental and theoretical works from SWOT analysis to geoenvironmental application and potential risk detection for Slope failure to case report on some successful practical works from excavation of diaphragm wall to fibre optic instrumentation in reusing deep foundations. The papers are of good quality and will generate opportunities for the academia and practitioners to discuss and question on various different techniques and approaches to implement interactive design to their works.

RÉSUMÉ : Ce rapport général présente une synthèse des communications correspondant au TC206 – Dimensionnement géotechnique interactif. Un total de 15 articles ont été reçus, 6 ont été recommandés pour une présentation orale et les autres seront présentés lors de la conférence correspondant à ce rapport général au 18ème CIMSG Paris, 2-6 septembre 2013. Les communications donnent une description générale des travaux en dimensionnement interactif de par le monde, depuis les travaux expérimentaux et théoriques jusqu’aux analyses SWOT, aux applications géoenvironnementales et à la détection des risques potentiels dans la réutilisation des fondations profondes. La bonne qualité des communications créera des opportunités pour les universitaires et les praticiens de discuter et échanger sur les différentes techniques et approches permettant de mettre en œuvre le dimensionnement interactif dans leurs réalisations.

KEYWORDS: Observational method, SWOT analysis, monitoring, risk of slope failure, small-strain, fibre-optic sensing technology.

1 INTRODUCTION

A total of 15 papers were received by the TC 206 – Interactive design and 6 papers were selected for oral presentation and the other for panel presentation at the 18th ICSMGE, Paris, 2-6 September 2013. The submitted papers gave a general picture of the interactive design works around the Globe ranging from experimental and theoretical works from SWOT analysis to geoenvironmental application and potential risk detection for Slope failure to case report on some successful practical works from excavation of diaphragm wall to fibre optic instrumentation in reusing deep foundations.

The following section will highlight some keys in the various papers submitted.

1.1 Papers recommended for oral presentation

Paper 1907 “Comparison of monitoring techniques for measuring deformations in an excavation” by DeVos, Van Alboon, Haelterman from Belgium. An online monitoring test set-up was realized in a railway-infrastructure project site in Anderlecht (Belgium). Both advanced and traditional monitoring equipment were installed to measure the deformation of a soil nailed jet grout wall, deformations behind the jet grout wall (on the railway tracks) and forces in the soil nails. The paper presents the results of the measurements in and behind the jet grout wall and on the comparison between the different techniques: FBGS; SAAF (in place inclinometer); Optical strands OSMOS; Traditional inclinometer; Draw Tower Grating; and BOTDR. It concluded that both new and traditional techniques can lead to the same result, when sufficient care is taken in the installation and interpretation. A significant advantage can be seen when continuous monitoring is applied, as the link with execution phases can be made.

The paper is clearly set out and well written and presents a real site monitoring case and compares predicted with actual deformation and bending moment results.

Paper 2397 “SWOT analysis Observational Method applications” by Korf, de Jong and Bles from Holland. A well set out account of Strengths, Weaknesses, Opportunities and Strengths of Observational Method and draws on a wide range of published work. This research is performed as part of “Geoimpuls” in the Netherlands; a joint industry programme, with the ambitious goal to half the occurrence of geotechnical failure in Dutch civil engineering projects by 2015. The Conclusion are given in the form of “Go”, “No Go” listed in terms of importance and “To be overcome” items.

Paper 2029 “Experimental analyses on detection of potential risk of slope failure by monitoring of shear strain in the shallow section” by Tamate & Hori from Japan. The paper consider monitoring locations at shallow depth of slope and introduces a mean to monitor the failure of shallow portion of slope which is particularly important during temporary cut. It may attract discussions in how to bring this into practice so as to enhance safety control measures to safeguard the workers during actual construction stage. It would be good to have a paragraph summarizing their findings from the experiment and / or any further study that may be worthwhile, e.g. any suggested alert, alarm and action shear strain to quantify the potential risk level.

Paper 3059 “The role of fibre optic instrumentation in the re-use of deep foundations” by Bell, Soga, Ouyang, Yan and Wang from UK & China. This paper provided details of a recent project in London to further develop the understanding of foundation reuse by installing fibre optic sensors in both existing piles and a borehole to observe the impact of the demolition process on the changes in piles behavior and ground response. It explained how optical fibre instrumentation was used to monitor pile and ground response under demolition and presented the data captured by the fibre optic instrumentation during the demolition process. It also showed how the use of such instrumentation was fundamental to the successful reuse of the existing piles on this project. It would be good to include the limitations of fibre optic instrumentation; such as can it quantify the vertical extent of section changes or the lateral position of defects; the presence of vertical cracks. And percentage of
existing pile to be instrumented to assess the integrity of existing piles for reuse can also be discussed.

Paper 2971 “New Sensing Technology and New Applications in Geotechnical Engineering” by Wang, Ooi & Gao from Hong Kong. The paper described that soils are inherently a particulate medium, and relevant physical principles behind the macro-scale engineering properties originate from particle interactions. However, it is difficult in general to conduct measurements which can monitor soil particle movement and even characterize micromechanics behind different soil behaviour. The paper presented two examples of advancement of sensing technologies. The first is on using the tactile pressure sensor (film-like sensor) to monitor the evolution of contact normal forces among particles in aged sand. The measurement reveals that the contact forces are continuously redistributed during aging. This ultimately strengthens the soil structure and therefore increases the associated small-strain shear modulus. The second is on using the miniature 3D Micro-Electro-Mechanical-Systems (MEMS) accelerometer to characterize the soil movement in a laboratory flow landslide. The MEMS sensors demonstrate promising results in describing the rich features of local responses of soil movement in the shear zone, e.g. liquefaction, deceleration, contraction and dilation. Some comments on the paper: Are the sensors insensitive to any other property likely to be encountered in its application and to what extent the sensors influence the measured property? Are the sensors designed to be linear or linear to some simple mathematical function of the measurement, typically logarithmic? What is the dynamic error of MEMS accelerometer? A very relevant paper on application of the small-strain theory on flow slides.

Paper 1965 “Monitoring earthwork foundations by fibre optic sensors” by Artières from France. The paper presented that fibre optic sensors are now easily available as part of the large number of fibre optic sensing technologies combined with geotextile properties, such as very good soil friction interface enhancing the transfer of soil displacement to the sensors. More accurate measurements can be obtained due to their smaller and less intrusive size than those of the usual electro-mechanical strain gages. They have high sensing sensitiveness below 0.01% on strain measurement, but also temperature measurement with 0.1°C accuracy, that is combined with a high spatial resolution in the range of 1 m or less and a good durability of the sensors into soil. And they can be used to monitor either local earthworks such as walls and slopes or long infrastructures of several tenths of kilometers such as roads, railways and dikes, all with the same accuracy. Several tenths of earthworks are now monitored globally with this solution for more than 8 years demonstrating its durability. The detection of cavities in the foundation of a large polluted water storage basin was also described. Some comments on the paper: Are the sensors insensitive to any other property likely to be encountered in its application and to what extent the sensors influence the measured property? And are the sensors designed to be linear or linear to some simple mathematical function of the measurement, typically logarithmic? The paper was selected for oral presentation on its relevancy in the application of sensor measurement, typically logarithmic? What is the dynamic error of MEMS accelerometer? A very relevant paper on application of the small-strain theory on flow slides.

1.2 Papers recommended for panel presentation

Paper 2253 “Development of Method for Evaluating and Visualizing 3-dimensional Deformation of Earth Retaining Wall for Excavation” by Matsumaru and Kojima from Japan. The paper described a System to evaluate and visualize retaining wall as three-dimensional curved surface. The validity was confirmed by the simulation of the loading test on the model wall and actual monitoring from the on-site measurement. It proposed to conduct monitoring of retaining walls using this analytical method and simple inclinometers. The paper describes original research/application and would be interested to most researchers.

Paper 2669 “Geotechnical protection of engineering infrastructure objects in large cities under intense anthropogenic impact and long term operation” by Perminov, Zentsov, Perminov, Russia. This article describes more than 30-year experience of scientific and technical support, design, construction and reconstruction of water supply and sewage facilities in St. Petersburg, Sochi, etc. It includes experience of sinking of large diameter shaft/caisson and long term loadings on tunnels in urban areas. Could do with a thorough proof reading and some of the terms used are unfamiliar. It is recommended for Panel presentation as it will be of interest to those who would like to practice in Russia.

Paper 2535 “A geoenvironmental application of an optimisation model” by Azimi, Merrifield, Gallagher & Smith from UK. The paper summaries a network of monitoring wells installed in and around a refinery in mid 1990s as part of a research project aiming to investigate the impact of local groundwater on corrosion of buried foundations and underground storage facilities. A second research project was started in 2000 to delineate the extent of the oil contamination mound(s) beneath the refinery and devise appropriate remedial measures. This paper presented an optimisation technique which assisted with augmentation of the monitoring network, thereby the cost-effective delineation of the oil mounds beneath the refinery. An optimisation model, The Maximal Covering Location Problem (MCLP), was modified and applied to find the optimum number and locations of additional monitoring wells to assist with the cost-effective delineation of the oil contamination mound beneath the refinery.

It is recommended for panel presentation as it may provide further discussions on the degree of confidence in selection of the value of maximal service distance (S) in the order of 100m without consideration of field, laboratory and theoretical investigations.

Paper 2460 “Evaluation of diaphragm wall as-built data to determine the risk of leakage for the Kruisplein car park excavation in Rotterdam, The Netherlands” by Hannink & Thumann from Netherlands. This paper should give the geological profile of the site to let readers have a better understanding of the ground conditions. Representative pumping test results may be included to demonstrate the critical or potential locations of leakage and the contingency measures which shall be required. It is recommended for Panel presentation.

Papers 3083 “Preventive maintenance of water retaining structures based on fiber optic systems” by Fry, Courvaud, Beck & Pinettes from France. The Pare described that EDF develops the concept of preventive maintenance. It means design, building and operation of an early warning monitoring system (leakage and strains), plus model of interpretation and portfolio of technical or legal alarm and interventions. In that framework, EDF has been working since 1994 on the development of the use of the distributed measurements with fiber optic, to improve the monitoring of dykes and flood embankments. The fiber optic technology provides a remote control measurement of the distributed temperature and strain every meter along the embankment. This new technology strategically placed in the fill, allows to reinforce the hydraulic and mechanical behavior monitoring, which is provided to date by conventional instrumentation (leveling, piezometer, discharge rate), with simultaneously a global and detailed surveillance and an early warning system for extreme loadings and crisis (floods, earthquakes, vandalism). It introduces the principle of dikes monitoring using fiber optic and the validation results of this technology from both trial test sites and on EDF’s real sites.
Auscultation des fondations d’un ouvrage en terre par des capteurs à fibre optique

Monitoring earthwork foundations by fibre optic sensors

Artières O.,
TENCATE GEOSYNTHETICS, Bezons, France

RÉSUMÉ : La mesure des déformations dans un ouvrage en terre est maintenant plus facile à réaliser en utilisant la technologie des capteurs à fibres optiques, qui associée aux propriétés des géotextiles, comme par exemple leur excellent frottement d’interface avec les sols, permet un bon transfert des mouvements du sol vers le capteur. Leur faible taille moins intrusive que celle des capteurs électromécaniques classiques donne l’accès à des mesures plus fines. Ils ont une sensibilité élevée, inférieure à 0,01% en déformation, mais aussi de 0,1°C en température, combinée avec une haute résolution spatiale de l’ordre du mètre et une bonne durabilité des capteurs dans le sol. Ils sont polyvalents tant pour ausculter des ouvrages locaux, comme des murs ou des talus, que des infrastructures linéaires de plusieurs dizaines de kilomètres, comme les routes, les voies ferrées ou les digues avec la même résolution. Plusieurs dizaines d’ouvrages sont maintenant auscultés depuis plus de 8 ans, preuve de la durabilité de ces capteurs, comme des remblais sur cavité ou sur inclusions. La détection de cavités en fond d’un grand bassin de stockage d’eau industrielle est décrite.

ABSTRACT: Strain in earthworks is now easier to measure by using fibre optic sensing technologies combined with geotextile properties, such as very good soil friction interface enhancing the transfer of soil displacement to the sensors. More accurate measurements can be obtained due to their smaller and less intrusive size than those of the usual electro-mechanical strain gages. They have high sensing sensitivity below 0.01% on strain measurement, but also temperature measurement with 0.1°C accuracy, that is combined with a high spatial resolution of the range of 1 m or less and a good durability of the sensors into soil. They can be used to monitor either local earthworks such as walls and slopes or long infrastructures of several tens of kilometres such as roads, railways and dikes, all with the same accuracy. Several tenths of earthworks are now monitored globally with this solution for more than 8 years demonstrating its durability, for example on foundations of embankments on piles, cavities or soft soils. The detection of cavities in the foundation of a large polluted water storage basin is described.

MOTS-CLES: Fontis, Cavité, Inclusion rigide, Fondation, Auscultation, Détection, Alerte, Géotextile, Capteur à fibre optique

KEYWORDS: Sinkholes, Piles, Foundation, Embankment, Monitoring, Detection, Warning, Geotextile, Fiber optic sensor.

1 INTRODUCTION

Les capteurs ponctuels de mesure des déformations des ouvrages en terre sont difficiles à installer et occasionnent parfois des problèmes de fiabilité à long terme. Les capteurs à fibre optique solutionnent ces inconvénients. Ils associent les propriétés des géotextiles, comme par exemple un frottement d’interface avec le sol élevé, avec plusieurs technologies de capteurs utilisant les fibres optiques pour mesurer des paramètres importants comme la déformation et la température. Les avantages principaux de cette solution sont sa sensibilité élevée, sa résolution spatiale de l’ordre du mètre voire moins, la durabilité des capteurs dans le sol, leur capacité à mesurer soit des ouvrages ponctuels comme des murs ou des talus, ou de longues infrastructures de plusieurs dizaines de kilomètres comme les routes, les voies ferrées ou les digues, toutes avec la même précision.

Plusieurs dizaines d’ouvrages sont auscultés dans le monde avec cette solution depuis plus de 8 ans, démontrant ainsi sa durabilité. Par exemple, une pile de pont renforcée est auscultée avec succès depuis juillet 2004. A côté des ouvrages en terre, plusieurs ouvrages en terre, tels que bassins, barrages, digues et levées ont été auscultés pour détecter des problèmes de stabilité et de fuite.

Après avoir introduit les principes de la solution d’auscultation et les principaux résultats obtenus sur plusieurs projets anciens, cet article se concentrera sur son emploi pour ausculter les fondations de remblais sur cavités : seront décrits en premier lieu, une section de voie ferrée auscultée en continu depuis octobre 2004, puis la fondation d’un grand bassin de stockage d’eau ausculté depuis 2011.

2 LA SOLUTION D’AUSCULTATION

Les capteurs à fibre optique ont été largement utilisés depuis plusieurs années dans les applications de génie civil, en particulier pour la surveillance des conduites d’hydrocarbure, dans des systèmes de surveillance de l’état de structure, ou des applications hydrauliques comme des barrages ou des digues en béton ou en terre. En associant des fibres optiques sur un géotextile (Figure 1), TenCate GeoDetect® est un capteur géotextile innovant qui améliore les performances des fibres optiques lorsqu’elles sont utilisées en contact avec le sol, du béton ou du bitume : le géotextile crée une excellente interface d’ancrage avec le milieu environnant. Grâce au très bon ancrage du géotextile dans le sol et à la bonne liaison des câbles optiques sur le géotextile, de très faibles déformations du sol peuvent être détectées. Cette interface de frottement améliore également le transfert des mouvements de géotextile vers la ligne optique. De plus, et quand cela est nécessaire, des propriétés de renforcement et raideurs en traction élevées peuvent être associées au capteur comme cela est indiqué au §3.

Différentes technologies de capteurs par fibre optique peuvent être utilisées : les réseaux de Bragg (Fiber Bragg Gratings ou FBG) employés dans le premier exemple d’application au §3, ou la mesure répartie décrite dans le deuxième exemple d’application au §4.
Les réseaux de Bragg sont des modifications locales de l’indice optique à l’intérieur de la fibre optique sous forme d’une série de petits miroirs inscrits sur une distance de quelques millimètres de long et réfléchissant une longueur d’onde donnée, proportionnelle à la température et à la déformation. A l’inverse des réseaux de Bragg qui sont des mesures ponctuelles, les technologies réparties Brillouin ou Raman mesurent tout point le long de la fibre sur des distances de plusieurs dizaines de kilomètres. Ces deux technologies permettent une mesure très précise de paramètres comme la température et la déformation, avec des fréquences de mesure soit statiques (< 1 Hz) soit dynamiques (de 1 Hz à 2 kHz).

La solution d’auscultation comprend le capteur géotextile à fibres optiques, l’instrumentation et le logiciel d’acquisition de données (Figure 1). Différentes stratégies d’auscultation peuvent être prévues lors du dimensionnement, comme une auscultation périodique ou continue à des fins d’alerte précoce.

En comparaison avec des techniques d’auscultation existantes constituées de capteurs ponctuels câblés individuellement, cette solution mesure en continu jusqu’à plusieurs dizaines de millier de points avec une seule instrumentation sur toute la longueur de la structure. La résolution spatiale peut être dans certains cas de 0,5 m. Dès son installation, le capteur géotextile à fibre optiques peut acquérir des valeurs de déformation et de température : la résolution en déformation est inférieure à 0,01% et des variations de température inférieures à 0,1°C peuvent être mesurées avec les logiciels correspondants. La technologie de mesure par fibres optiques ne nécessite aucune calibration avant mesure, seulement une compensation de la température peut être nécessaire pour des amplitudes supérieures à 10°C.

Figure 2 : Le capteur géotextile à fibres optiques renforcé

Figure 3 : Evolution des déformations sous la voie entre 2004 et 2010

4 L’AUSCULTATION DES FONDATIONS D’UN BASSIN

4.1 Description

L’objectif de ce bassin est le stockage d’eau salée industrielle pendant environ 6 mois sur 12 pour pouvoir la relarguer dans la rivière voisine lorsque celle-ci est en crue (en hiver) afin d’obtenir un taux de dilution suffisant. La surface totale du bassin est de 30 ha et a une capacité de stockage de 300 000 m³ pour une hauteur d’eau maximum de 10,7 m. Il est construit à l’endroit d’une ancienne gravière à proximité d’une rivière et est délimité par une digue périphérique de 2 km fondé sur le sol naturel en dehors de la gravière. Le fond du bassin est à 4 m en-dessous du sol naturel. Il est sous le niveau de la nappe et pour cette raison étanché en fond et sur les talus par une structure d’étanchéité incluant une géomembrane PEHD de 2 mm protégée par des géotextiles et un tapis drainant granulaire en fond. Les conditions géologiques et hydrologiques, ainsi que la conception de ce bassin sont décrites en détail dans une autre publication (Artières et al. 2013).

4.2 Observations

Une augmentation des concentrations de produits salés correspondant au même que ceux stockés a été détectée en 2008 dans l’eau de pompage de rabattement de nappe sous le bassin. Après vidange du bassin, des tassements ont été...
observés dans les casiers E1 et D1 le long de la ligne de collecte des sources (Figure 4). Le contexte hydrogéologique de cette zone est très complexe. On a supposé une dissolution du chapeau sulfaté de la dolomie profonde par des eaux salées. Ces tassements étaient visibles sur la géomembrane sous la forme d’une déchirure de 15 cm par 3 cm et étaient dus à un fontis de 6 m² et de 3 m de profondeur. Ceci a expliqué les raisons des fuites. Sur trois autres endroits, le substratum argileux était décomprimé. A partir d’une analyse topographique, la surface totale d’effondrement a été estimée à environ 1500 m² autour des sources.

4.3 Travaux de réparation

Il a été décidé une réparation du fond de la zone du bassin présentant un risque élevé de tassement en installant une structure de renforcement de la fondation par géotextile, associée à un système de détection et de localisation des fontis par capteur géotextile à fibre optique. Dans ce projet, la fonction renforcement a été dissociée de la fonction auscultation. Le géotextile de renforcement évite à la fois que le fontis atteigne de façon soudaine la structure d’étanchéité et que la géomembrane s’allonge au-dessus d’une valeur seuil permettant de maintenir la fonction étanchéité jusqu’à ce que la cavité soit traitée. Il est dimensionné selon l’Eurocode 7 (2005). Aux états limites ultimes (ELU), les justifications sont menées en utilisant l’approche de calcul 2, avec les coefficients partiels définis dans l’Annexe Nationale [NF EN 1997-1 /NA] de l’Eurocode 7.

Figure 4. Numérotation des casiers et sortie des sources

La méthode de dimensionnement est une méthode simplifiée (Blivet et al. 2001) développée après le programme de recherche RAFAEL. Le dimensionnement du géotextile a été conduit en appliquant les trois états limites : un de service et recherche RAFAEL. Le dimensionnement du géotextile a été développée après le programme de

étant une ligne redondante de sécurité. La distance entre les deux câbles de déformation est de 0,6 m (Figure 4). Ces capteurs sont reliés par un câble de liaison de 200 m de long à un interrogateur optoélectronique Brillouin installé dans une cabine en tête de digue. Il n’y a donc aucun appareil électromécanique ni de source électrique dans le bassin. Cette solution est décrète plus en détail dans d’autres publications (Artières et al. 2011, Artières et Dortland 2011).

L’interrogateur connecté à une extrémité de la fibre optique envoie un pulse laser de quelques nanosecondes qui parcourt toute la longueur de la fibre optique. En chaque point de la fibre optique, le pulse laser interfère avec la structure moléculaire de la matière en rétrodiffusant en sens inverse un spectre de lumière schématisé en figure 5. La longueur d’onde des pics secondaires Brillouin dépend notamment de l’état de déformation de la fibre optique. La résolution spatiale de la fibre est d’environ 1 m et la résolution sur la mesure de la déformation est inférieure 0,01%.

L’interrogateur optique connecté à une extrémité de la fibre optique envoie un pulse laser que quelques nanosecondes qui est guidé tout au long de la fibre optique. A tout point de la fibre, le pulse laser interfère avec la structure moléculaire du matériau en rétrodiffusant un spectre de lumière (Figure 5). Les pics de longueur d’onde secondaires Brillouin dépendent de la déformation de la fibre et de sa température. La résolution spatiale le long de la fibre est d’environ 1, ce qui signifie que le système génère un point de mesure tous les mètres. La résolution de la mesure de la déformation est de moins de 0,01%.

4.5 Conception et mise en œuvre de l’auscultation

Une surface d’environ 10.000 m² présente un risque de tassements liés à la remontée aléatoire de cavités souterraines. La solution d’auscultation est implantée sur une zone particulière d’environ 165 m x 20 m incurvée couvrant les casiers E1, D1 et C1 (Figure 6).

Figure 5. Spectre de lumière rétrodiffusé lors de la mesure répartie sur fibre optique (λo est la longueur d’onde de la lumière incidente)

Sur cette surface, 18 bandes du capteur géotextile à fibres optiques ont été installées en 5 boucles optiques distinctes permettant de lire l’une indépendamment des autres. Les bandes sont espacées l’une de l’autre de 1,2 m correspondant à la taille minimale de cavité à détecter. Les deux câbles optiques de déformation portés par chaque bande sont raccordés séparément, ce qui constitue un total de 5 boucles principales et de 5 boucles redondantes de sécurité. Ces dix boucles sont raccordées à un câble optique de liaison de 200 m de long vers l’instrumentation. Ce sont environ 6,000 points de mesures qui couvrent la surface totale avec un pas de 0,6 m x 1 m.

Les bandes de capteur composite géotextile à fibres optiques sont déroulées sur le site selon un plan de calepinage approuvé par le Maître d’Ouvrage. Les bandes sont immédiatement lestées temporairement jusqu’à la fin de l’installation du système d’auscultation (Figure 7). La continuité des lignes optiques est contrôlée avant la pose du gravier de couverture 0/16 et du géotextile de renforcement (Figure 8).
L’association d’une structure de renforcement par géotextile avec un système d’auscultation pour détecter les mouvements du sous-sol est une solution utilisée avec succès depuis plus de huit ans sous un remblai ferré sujet à des risques de remontée de fontis.

Cette solution a également été utilisée pour stabiliser la fondation d’un bassin industriel étanché par une géomembrane après que des tassements aient été constatés. Une structure de renforcement par géotextile a été conçue sous la structure d’étanchéité pour réduire les déformations de la géomembrane pour des tailles de cavité de 3 m. En complément, un système d’auscultation basé sur des capteurs composites géotextile à fibres optiques a été positionné en-dessous pour détecter de façon précoce et localiser les effondrements, suivre l’évolution de la taille des cavités et éventuellement planifier une opération de maintenance si leur taille dépasse les hypothèses de dimensionnement du géotextile de renforcement. La solution d’auscultation a indiqué le développement de déformations lors des réparations à cause de la rugosité élevée du fond de forme, mais aucune déformation liée à un fontis n’est apparue depuis un an et demi après son installation.

6 REMERCIEMENTS
L’auteur remercie Valérie Lefèbvre-Mignon, Arcadis, assistance à la Maitrise d’ouvrage, pour son aide.

7 REFERENCES

4.6 Mesures

Le bassin a été ausculté dès la fin de l’installation du système géotextile à fibre optique en mai 2011 mettant en évidence l’effet de la mise en place de la couche de remblai de 80 cm puis du remplissage en eau du bassin.

Les déformations principales ont été enregistrées lors de la mise en place du remblai qui s’explique par la rugosité importante du fond de forme et la mise sous tension des bandes capteur. Un événement pluvieux intervenu lors de la pose des capteurs avait en effet érodé la surface du fond de forme. La moyenne des déformations est de l’ordre de 0,2% avec des pointes autour de 1% et une forte irrégularité. Un remblaiement sur support plan créé généralement des déformations initiales inférieures à 0,1%. Cette mesure a été choisie comme ligne de référence par le Maître de l’Ouvrage pour le suivi du bassin

Le remplissage du bassin à 6 m d’eau en décembre 2011 et en novembre 2012 ne montrent pas d’accroissement sensible des déformations, en moyenne inférieures à 0,1% avec quelques points très localisés entre 0,1 et 0,4 %, ceux-ci étant encore certainement liés à la mise en place des capteurs sur le fond de forme (Figure 9).
A geoenvironmental application of an optimisation model

ABSTRACT: A network of monitoring wells was installed in and around a refinery in mid 1990s as part of a research project aiming to investigate the impact of local groundwater on corrosion of buried foundations and underground storage facilities. Oil contaminated groundwater was evident in some of the monitoring wells. A second research project was started in 2000 to delineate the extent of the oil contamination mound beneath the refinery and devise appropriate remedial measures. Of 30 initial monitoring wells, 15 were found operational inside the refinery in 2000. An optimisation technique is presented herein which assisted with augmentation of the monitoring network, thereby the cost-effective delineation of the oil mounds beneath the refinery. The Maximal Covering Location Problem (MCLP) was adapted and utilised to find the optimum number and locations of additional monitoring wells. The contamination results obtained from the augmented and optimised network of monitoring wells were analysed using a geostatistical tool and the oil contamination hot spots beneath the refinery were delineated cost-effectively.

KEYWORDS: Groundwater, contamination, monitoring, optimisation, MCLP, network augmentation

1 INTRODUCTION

An oil refinery constructed in the early 1970s and operated ever since caused groundwater contamination. A research project was conducted at the refinery in mid 1990s to investigate the impact of local groundwater flow on corrosion of buried foundations and underground storage facilities inside the refinery. As part of that project, a network of 30 monitoring wells was installed in and around the refinery. Oil contamination of groundwater was evident in some of the monitoring wells. A second research project was started in 2000 to delineate the extent of the oil contamination mound beneath the refinery and devise appropriate remedial solution(s). Of 30 initial monitoring wells, 15 were found to be operational inside the refinery at the beginning of the second research project. Monitoring of these wells demonstrated that free phase of oil contamination was present in groundwater at least at two separate locations inside the refinery. However, the contamination data obtained from the existing monitoring network of 15 wells were too sparse for the purpose of oil contamination delineation. Therefore, it was decided to add monitoring wells to the network within the refinery. The two major engineering challenges were identified as:

- How many monitoring wells should be added to the existing network?
- In which locations should these wells be installed?

There is substantial evidence in the literature on the application of Operations Research (OR)-based optimisation methods in different civil and environmental engineering practices (ReVelle et al. 1997). Fields of practice such as transport engineering, urban planning and water resources management are examples where successful applications of OR methods including optimisation techniques have been demonstrated. The Maximal Covering Location Problem (MCLP) is an optimisation model proposed in the literature, primarily devised to find the optimum locations for public facilities, such as ambulance dispatch centres, on a network of demand nodes (Church and ReVelle 1974). The model was modified and applied to the groundwater contamination problem in this study to assist with the cost-effective delineation of the oil contamination mound beneath the refinery.

2 MAXIMAL COVERING LOCATION PROBLEM (MCLP) – CONCEPT, THEORY AND APPLICATION

An example where the MCLP can be used for the optimum usage of the resources is the requirement to add a certain number of a public facility (e.g. ambulance dispatch centres) to an existing network in a city. The city is discretized to a set of demand nodes where additional dispatch centres can be situated. Each demand node is assigned a weight representing its population with the existing dispatch centres being attributed to the nearest nodes. From the network operator’s point of view, coverage of the demand nodes (i.e. population nodes in this example) on the network is the key objective with demand nodes not being located farther than a threshold distance (i.e. maximal service distance, S) from an ambulance dispatch...
centre. In other words, a node is covered if at least one facility is located within the maximal service distance of that node. Otherwise, the node is uncovered. If a network operator instinctively places all the available resources on the nodes with the greatest nodal weight (e.g. population), the outcome will not necessarily be maximum coverage of the population by the public facilities because of the likely overlaps and gaps in the coverage of the demand nodes.

In reality, the 100% coverage target is not always achievable due to limitations in the availability of supply units. If the resources are insufficient to cover all the demand nodes, the objective changes to cover as many nodes as possible within the budget allocated for additional monitoring wells included the addition of a maximum 10 number to the existing network. Therefore, the MCLP model was solved for different values of \( P \) from 20 to 25, noting \( P \) is the total number of existing and additional wells on the network and there were 15 existing wells. There were three key decision parameters; areal plume coverage which corresponds to the vertical axis on Figure 2 and is defined as the percentage of the nodes with weight values above zero covered by one or more wells (i.e. located within distance \( S \) of one or more wells), the total number of existing and added wells \( (P) \), and the maximal service distance \( S \) (horizontal axis on Figure 2). The marked increase in the slope of the curves on Figure 2 in two particular regions demonstrated that with a moderate increase in the value of \( S \), the areal plume coverage would increase considerably compared to the other regions on the curves. If the values of \( S \) and the number of covered nodes (i.e. areal plume coverage) within these regions (intervals) were reasonable for decision making, then it would be possible to focus on these two regions for taking the next steps towards the final decision. The maximal service distance \( (S) \) plays the most important role in dictating the final configuration of the added facilities. This parameter should ideally be calculated through field, laboratory and theoretical investigations. Considering the grid size of the study area, values of \( S \) in the order of 100m were justified in this study.

### 2.3 Results

Figure 3 demonstrates that within the maximal service distance of 76m, it was possible to achieve a maximum areal plume coverage of 60% (i.e. 113 covered nodes out of total 188 nodes could be covered). This amount of coverage was not considered satisfactory. Therefore, the first interval on Figure 2 was not considered further and the second interval was selected. Figure 4 demonstrated that augmenting the network with 10 additional monitoring wells was somewhat insensitive to the magnitude of \( S \), i.e. selection of 10 additional monitoring stations would result in a considerable increase in the areal plume coverage with minimum change in \( S \). Hence, the network was augmented with 10 additional boreholes, corresponding to the areal plume coverage of 85% and \( S \sim 108 \) m. The pattern of additional wells showed no clustering at the areas of high contamination within the refinery, using physical / numerical models to calculate the nodal concentrations (weights) was considered impractical. Kriging as a stochastic interpolator was employed instead to estimate the weights. The groundwater chemical data obtained from all 22 available monitoring wells located within and around the refinery were utilised in this nodal weight estimation.

### Figure 1. The concept of maximal service distance.

The MCLP is expressed as: Maximize coverage (population covered) within a desired service distance by locating a fixed number of facilities (Church and ReVelle, 1974).

The mathematical formulation of the MCLP in the context of augmentation of a groundwater monitoring network is presented in Hudak & Loaiciga (1992). Groundwater monitoring network augmentation incorporates the following stages:

- Discretize the model domain into a network of potential detection monitoring sites (nodes).
- Assign weighting to each node to quantify its relative importance for coverage by a monitoring well.
- Solve MCLP with successive values of \( S \) until target areal plume coverage is achieved.
- Determine the corresponding configuration of the added wells on the network.

#### 2.1 Geometry of the grid (problem domain discretization)

An irregular grid of 188 nodes was defined within the oil refinery area taking into account the local hydrogeology as compared to similar cases where groundwater monitoring networks have been augmented, the limitations against excavation of wells on site, the spacing between the existing wells and the computational limitations for a plausible nodal weight estimation. Each of the 15 existing wells was assigned to the nearest node.

#### 2.2 Nodal weight estimation

Due to the complex hydrogeological setting of the site and presence of a large number of potential sources of oil contamination within the refinery, using physical / numerical models to calculate the nodal concentrations (weights) was considered impractical. Kriging as a stochastic interpolator was employed instead to estimate the weights. The groundwater chemical data obtained from all 22 available monitoring wells located within and around the refinery were utilised in this nodal weight estimation.
estimated chemical concentration (weights). This model located the additional stations (i.e. monitoring wells) at regions with high concentration of contaminant and at the same time prevented clustering of the wells (See the layout in Figures 5 and 6).

Figure 3. Cost-effectiveness curves for two distinct values of $S$ derived from Figure 2.

Figure 4. Variation of maximal service distance ($S$) versus the number of added wells ($P'=P-15$).

Figure 5. Added wells on the discretized network.

Figure 6. Added and existing wells.

Figure 7. Hot spots of oil contamination beneath the refinery (concentrations are in terms of Total Organic Carbon).

3 GEOSTATISTICAL ANALYSES ON THE AUGMENTED DATA SET

A geostatistical analysis, using the same geostatistical tool which was used in nodal weight estimation (i.e. estimation of chemical concentrations at different nodes), was conducted on the extended data set to assist with delineating the locations and the extent of hot spots of oil contamination beneath the refinery. Three different hot spots were identified at three distinct areas. Figure 7 shows the location and the extent of the hot spots.

4 CONCLUSIONS

A geoenvironmental case history of applying an optimisation model in practice is illustrated in this paper. The Maximal Covering Location Problem (MCLP) was employed to enhance the efficiency of an existing network of monitoring wells in an oil refinery in order to assist with delineating the mounds of oil contamination beneath the refinery. After installation of the added monitoring wells at the locations predicted by the model (i.e. monitoring network augmentation), the results obtained from the augmented network demonstrated the robustness of the method. The model helped to prevent clustering of the added monitoring wells in the areas with high estimated values of the attribute (i.e. concentration) and at the same time helped to benefit the monitoring from further sampling at these areas. Using the data from the augmented network of monitoring wells
and a geostatistical tool, the oil contamination hot spots were delineated cost-effectively.

5 REFERENCES


ABSTRACT: The re-use of existing foundations, in particular piled foundations has increased in recent years due to the significant environmental and commercial benefits. However, there has been limited progress in assessing the condition of such piles by considering the effect of initial use and the impact from the subsequent demolition process which often requires a detailed study. This paper will provide details of a recent project in London that successfully reused all exiting piles beneath the site and optical fibre sensors were instrumented to the existing foundations in order to monitor the behavior of piles during the demolition of the existing building. The use of optical fibre instrumentation is believed to be the first of such an approach in observing the behavior of reuse piles during demolition in the UK and as urban environments become more congested particularly below ground, the approach discussed in this paper will become increasingly valuable. The monitoring data is presented and discussed in detail and the role of using these sets of data in assessing the reuse strategy is also highlighted in this paper.

RÉSUMÉ : La réutilisation de fondations existantes, en particulier de fondations sur pieux, a augmenté ces dernières années en raison des avantages environnementaux et commerciaux significatifs. Cependant, l’état des pieux suite à leur première utilisation et le processus de démolition sont souvent négligés. Ce document présente un projet récent à Londres où les pieux ont été instrumentés avec des capteurs en fibre optique, avant la campagne de démolition. Les fibres optiques permettent de mesurer des déplacements le long des pieux lors de la démolition. L’utilisation d’une telle instrumentation est une première au Royaume-Uni. Les données de la campagne de surveillance sont présentées et discutées en détail. Une stratégie de réutilisation des fondations sur pieux est également proposée dans ce document.

KEYWORDS: Re-use foundations, Optical Fibre Sensors (OFS), Brillouin back-scattering

1 INTRODUCTION

Foundation re-use can generate significant environmental and commercial benefits, and is becoming a popular engineering option, particularly in congested urban environments. Due to the many practical constraints, most redevelopments need to be constructed on the existing foundations together with a new pile system; therefore it is crucial to understand the geotechnical behaviour of reused piles and their compatibility with the new structure. This is often difficult without removal of significant parts of the substructure. Previous researches (Leung et al., 2011; Begaj-Qerimi & McNamara, 2010) have shown that pile behavior may change with time, due to consolidation and ageing, residual stress at the pile base and increased soil stiffness; hence reused piles are often stiffer than new piles. On the other hand, the building demolition process could potentially introduce ground heaving and the physical unloading of the reused piles can also generate tension cracks. These differences in pile responses need to be properly assessed in the design of a new pile system.

A recent project in London provided the opportunity to further develop the understanding of foundation reuse by installing fibre optic sensors in both existing piles and a borehole to observe the impact of the demolition process on the changes in piles behavior and ground response. The site is located at 6 Bevis Marks and near to Liverpool Street, London, UK, and it was proposed to reuse all existing foundation piles and the majority of the basement substructure on this project.

This approach produced significant commercial and environmental benefits.

The existing piled foundations are large diameter under-ream piles, and there was a concern that these piles would be damaged during the demolition process. Such damage is usually caused either by the forces generated by the removal of significant load as the building is demolished, the tensile forces within the piles and surrounding soil, or physical damage caused by demolition of the substructure, pile caps and pile breaking down. This can lead to the reuse being questioned and ultimately being discounted.

This paper explains how optical fibre instrumentation was used to monitor pile and ground response under demolition and will present the data captured by the fibre optic instrumentation during the demolition process. It will show how the use of such sophisticated instrumentation was fundamental to the successful reuse of the existing piles on this project.

2 SITE DESCRIPTION

2.1 Existing site and proposed redevelopment

The existing building at 6 Bevis Marks was constructed in the early 1980’s and comprised eight superstructure floors and two basements. The existing foundations system includes (i) piles located inside the basement, which is approximately 7.0m below pavement level, and (ii) piles constructed in the Bevis Marks pavement, which is approximately 3.5m below pavement level.
level as shown in Figure 1. All existing piles were designed and constructed by Cementation Skanska as under-ream piles.

The local geology comprises made ground overlying Terrace Gravels, which in turn rests on London Clay. The top of the London Clay is within 2m of the existing basement level and all piles, both existing and new, are founded within this stratum. The existing piles were designed to carry total building loads of 350,100kN and 33,900kN for those inside and outside the basement, respectively. There are a series of heavily reinforced pile caps and ground beams, and their thickness varies across the site. Typically the ground beams are around 1000mm to 1500mm deep at pile positions, and the pile caps vary from 1500mm to 4150mm deep. This complex pile foundation system practically constrains the available physical space for designing new piles.

The scheme design was developed with the existing building arrangement and existing foundation arrangement in mind. However, with the proposed building being taller (sixteen floors in lieu of eight) and the main stability systems located in slightly different areas, the existing foundations proved to be inadequate as a whole. The result of technical studies demonstrated that the existing piles were overloaded by some 85,000 kN when compared to actual load carried originally.

To enhance the existing piles where they were found to have inadequate capacity, supplementary piles needed to be installed. Figure 2 shows the significant subsurface congestion beneath the site when new piles were installed.

2.2 BOTDR sensing principle

Brillouin Optical Time Domain Reflectometer (BOTDR) was adopted for this project. It provides sensitive strain measurement from the reflected light that travels along the standard single mode fibre optic cable. The entire fibre cable can be considered as the sensor itself. When the light travels in the optical fibre sensor, the majority of it is back scattered as shown in Figure 3. In the back scattered spectrum, where only Brillouin spectra are temperature and strain dependent, the frequency shift of the Brillouin spectrum indicates the local change in the fibre properties induced by the change of strain and temperature. Hence, the change in strain and temperature along the fibre optic sensor is proportional to the frequency shift, which can be detected by the BOTDR analyzer. The analyzer used in this study is capable to sample 1 m averaged strain or temperature at every 5 cm with an accuracy of 50 με and up to a distance of 10km.

![Figure 3. The spectrum of backscattered light.](image)

Due to the principle of Brillouin backscattered sensing, the system registers strain induced by the elongation of the optical fibre itself and the strain resulted from temperature change. Therefore it is necessary to incorporate two different types of optical fibre sensing cables to compensate for temperature effect. Figure 4 shows two types of cables which have been carefully calibrated and widely used for infrastructure sensing (Klar et al., 2006; Mohammed, 2012). Figure 4a shows the strain sensing cable, which consists of four optical fibre members tightly bonded by strong nylon material. This is to ensure the strain can be fully transferred from the nylon coating to the optical fibre itself. Two steel wires at both ends reinforce the cable and make it robust enough to survive in the harsh construction environment. The temperature cable shown in Figure 4b consists of several optical fibres in a gel filled tube, so that it can contract and expand only under temperature effects, independent of mechanical strain.

The use of optical fibre technology has numerous advantages over conventional monitoring systems, it is capable of providing a continuous and full length strain profile and this makes it possible to monitor for cracking of the pile along its full depth. The continuous strain profile also provides a picture of what is happening over the full pile shaft; this would not be possible with traditional single point based system such as vibrating strain gauges or extensometers.

2.3 Instrumentations and field installation

Prior to demolishing the building, an under-ream pile (E47, see Fig. 1) was selected on site to be cored to full depth for...
inspecting and assessing concrete quality. Fibre optic cables were attached to a flexible pipe and installed to the full depth of the cored hole. Table 1 lists the detailed level of the under ream pile. In addition, a new borehole was drilled to 35.5m adjacent to an instrumented pile in the basement and also instrumented to full depth to capture potential ground heave during the demolition process.

This paper will present the data recorded during the demolition process and summarise the results to highlighting the role of the instrumentation in the successful reuse of the deep foundations on this project. It will also include details of challenges faced in using the fibre optic sensors to instrument existing deep foundations, which is believed to be the first of such use in the UK.

Table 1. Summary of instrumented pile

<table>
<thead>
<tr>
<th>Pile</th>
<th>Core length [m]</th>
<th>Coring length [mAOD]</th>
<th>Toe Level [mAOD]</th>
<th>Original Design Toe Level</th>
<th>Shaft Diameter [mm]</th>
<th>Bell Diameter [mm]</th>
</tr>
</thead>
<tbody>
<tr>
<td>E47</td>
<td>20.16</td>
<td>7.46</td>
<td>-13.70</td>
<td>-13.00</td>
<td>1500</td>
<td>3300</td>
</tr>
</tbody>
</table>

3 FIELD DATA ANALYSIS

There are ten sets of strain data that have been collected on five dates from 15/08/2011 to 05/10/2011 up to building level 3 of the demolition programme. All optical fibre strain sensing cables and optical fibre temperature sensing cables, installed in the pile and the borehole close to one of the piles, were monitored as close as possible to the removal of each floor. The data from 15/08/2011 taken when the 6th floor had been demolished and 05/10/2011 when the 3rd floor was demolished is presented in this paper to evaluate the pile and borehole performance. The original geotechnical design load on the pile was around 7,250 kN to 7,500 kN, but the structure takedown load on these piles (i.e. the load they experienced under use) is more likely to be around 65 percent of this at around 4,700 kN to 4,900 kN.

3.1 Pile E47

The first set of data was taken on 15/08/2012 and has been considered as the reference for comparing with the dataset collected on 05/10/2012. The profile of strain change between the two periods is shown in Figure 5(a). In general, the axial strain is reducing from the top to the middle part of the pile (0-9m) in the range between 0 and 100 micro-strain, and the axial strain change becomes reasonably small in the base part of the pile (9-17m). After integration of the strain profile, the calculated overall vertical heave between the two periods is less than 1mm at the pile head as shown in Figure (b).

Pile E47 was initially constructed with a design concrete strength $f_{cu}$ of 30 N/mm², with a corresponding modulus of elasticity varying from 20 to 32 GPa as suggested by BS 8110-2:1985. The concrete strength data taken from the tests on the full length cores in pile indicates that the concrete strength in-situ is between 47 N/mm² to 56 N/mm². The modulus of elasticity of this concrete can be calculated, allowing for creep and degradation, as ranging from 26 GPa to 43GPa. Figure 5(c) shows the load profile by assuming the pile Young’s modulus is 26GPa. Hence, the axial force reduction at the top of pile can be estimated to be around 3,830 kN, which roughly corresponds well to the load removed between the two periods.

It is believed that nominal starter cages were installed into all of the original under-ream piles on this site and are likely to be between 9m and 12m in lengths. Reinforcement details for the existing piles were not available, as is often the case with reuse projects. The cage toes were observed in the strain measurements at 10m to 13m as a change in the strain profile, which indicates the pile is experiencing tension forces around the base of the reinforced section. It is suspected that this is as a result of the different Young’s modulus between the reinforced and unreinforced sections of the pile.

3.2 Borehole

In comparison to the pile reaction to the demolition of the substructure, the magnitude of change in strain along the borehole is less pronounced than the results observed from the pile, which ranges within 50 micro-strain and within the accuracy range of the BOTDR system. The calculated strain profiles are shown in 6 including the strain profile along the borehole and the interpreted vertical heave is about 0.4 mm at the top. Due to the limitation of the BOTDR system, it is difficult to obtain the accurate vertical displacement profile from such small strain measurement, and the data shown in Figure 6 are the best approximation of the ground movement from the borehole measurements.
considering foundation reuse are an essential part of the proofing process and validated the installation records. Although in the future these may be more monitored piles and borehole gave a good indication of the quality of such records. This will in turn constrain the future development sites, the extent of re-use will be limited to the "as built" design and installation records. Although in the future these may be more comprehensive than what are currently available on development sites, the extent of re-use will be limited to the quality of such records. This will in turn constrain the future development options for such sites and is likely to influence the asset values of the site and the existing development, and the viability of redevelopment.

Traditionally, low strain pile integrity testing is carried out to confirm that new piles have been constructed correctly and no discontinuities exist. It is also used to assess the integrity of existing piles for reuse, usually with mixed success. Such testing is often not appropriate for pile reuse as,

- To carry out a low strain integrity test, the top of the pile needs to be exposed and structurally separated from other foundations. This is not possible where pile caps, slabs and basement substructures are to remain in place for reuse.
- Such testing only confirms if there is a crack, not how big it is or what is below it. When demolishing an existing building, the expected ground heave may crack the piles to some degree, such cracks are expected to be small and to close up upon pile reload but these tests cannot confirm this.

An alternative solution is to install fibre optic strain measuring devices into existing piles that have been cored full depth and into future piles on installation, producing a smart foundation system. Making this provision for the future will not only increase the potential for re-use and increase its asset value, but is also likely to make the asset more valuable when compared to other properties where such "future proofing" has not been incorporated. Such an approach will allow monitoring of how the piles actually perform under loading, unloading, reload and during the life cycle of the building. Results could be used to further advance our understanding of actual foundation response during the construction phase and operation of such buildings.

5 ACKNOWLEDGEMENTS

The authors would like to thank Waterman Structures Ltd for their initial foresight in assessing the re-use potential of the foundations on the Bevis Marks project and their considerable involvement in developing and implementing the pile reuse strategy and associated instrumentation and monitoring works.

6 REFERENCES


Comparison of monitoring techniques for measuring deformations in an excavation

Comparaison de techniques d’auscultation pour la mesure de déformations dans une excavation

De Vos L., Van Alboom G., Haelterman K.
Geotechnics Division, Flemish Government, Ghent, Belgium
Maekelberg W.
TUC RAIL, Brussels, Belgium

ABSTRACT: Active monitoring is often suggested as a method to decrease the required safety coefficients in the design stage of a construction. In order to apply active monitoring precise, reliable and interpretable measurements of the actual behaviour of the structure and soil-structure interaction are required. To obtain this data, accurate and robust monitoring tools should be available at an acceptable cost. An online monitoring test set-up was realized in a railway-infrastructure project site in Anderlecht (Belgium). The braced excavation consists of a nailed jet grout wall with HEB profiles, installed immediately next to a railway track. Both advanced and traditional monitoring equipment is installed to measure the deformation of the jet grout wall, deformations behind the jet grout wall (on the railway tracks) and forces in the nails. The present paper focuses on the results of the measurements in and behind the jet grout wall and on the comparison between the different techniques.

RÉSUMÉ : L’auscultation active est souvent suggérée comme une méthode permettant de réduire le coefficient de sécurité du dimensionnement d’un ouvrage. Afin d’appliquer une auscultation active, des mesures précises, fiables et interprétables du comportement réel des ouvrages et de l’interaction sol-structure sont requises. Afin d’obtenir ces données, des outils d’auscultation précis et robustes doivent être disponibles à un coût acceptable. Un essai de surveillance en ligne a été réalisé sur le site d’un projet d’infrastructure ferroviaire à Anderlecht (Belgique). L’excavation consiste en un mur d’étanchéité cloué, avec des profils HEB, installé à proximité immédiate de la voie ferrée. Des technologies aussi bien avancées que traditionnelles ont été utilisées pour mesurer les déformations du mur d’étanchéité, les déformations derrière le mur (sur les voies ferrées) et les forces dans les clous. Le présent article vise à comparer les résultats des mesures dans et derrière le mur et à comparer les différents techniques de mesure.

KEYWORDS: active monitoring, advanced monitoring techniques, monitoring test site.

1 INTRODUCTION.

The Geotechnics Division of the Flemish Government (GEO) realised an online monitoring test set-up to extend the experience with new monitoring techniques and to make a step forward in the application of active monitoring on construction sites. The project was partially funded by the Agency for Innovation by Science and Technology (IWT), allowing 3 firms to develop and perform online monitoring for an excavation. Verification measurements were made by several parties, using both more traditional as well as new monitoring techniques.

1.1 Main objective

The main goal of the project is to evaluate different monitoring results and suitability of proposed monitoring schemes for application in interactive design. This implies that accuracy, installation possibilities, reliability and cost are important aspects to be considered.

The monitoring scheme consists of measuring and logging:
• deformation of a vertical wall \((x,y,z)\)
• maximum bending moment in a vertical wall
• deformation of the soil \((z)\) behind a vertical wall
• anchorage forces in nails

1.2 Site description and applied equipment

The monitoring site is located in Belgium, Anderlecht (Brussels), where an extra railway track will be constructed alongside the existing tracks. For the foundation of the new bridge, a nailed jet grout wall was installed next to the existing railway. By doing so, the soil could be excavated vertically and the foundation could be realised in an open construction pit. The excavation depth is 12.5m starting from the railways. The jet grout wall starts 4m below the railway level and has a total length of 21m. HEB profiles with a length of 21m are inserted in the jet grout wall. Five rows of nails are installed over the excavated depth. Figure 1 shows pictures before excavation and after excavation. The excavation is executed in different phases. Each time 2m is excavated and consecutively, a row of nails is installed. After installation of the nails, the contractor waits at least 2 weeks before excavating the consecutive part. More information on the site can be found in Van Alboom et al. 2012 and Verstraelen et al. 2013.

Figure 1. Picture of initial situation (left) and picture of the jet grout wall after excavation (right).

1.2.1 Deformation of and bending moments in the jet grout wall

To measure the deformation of the jet grout wall, both advanced and traditional monitoring equipment is installed:
• Fiber Bragg Grating (fiber optics, FBGS)
• SAAF (In-place inclinometer, Inventec)
1.2.2 Settlements behind the jet grout wall
As one of the main concerns during an excavation are the settlements of the soil (and inherently the infrastructure) in the vicinity of the excavation, the vertical deformation of the soil behind the wall is one of the important parameters to be monitored. In this project specifically, deformations of the railroad track are to be avoided. The deformations behind the wall are measured:

- with two horizontal inclinometers (a traditional one and a continuous SAAF inclinometer) which are placed perpendicular to the wall in the ballast underneath the rails. The inclinometers are attached to a Berliner wall, which is located at one side of the rails. It is assumed that this creates a fixed point;
- topographically: the rails are marked with survey nails along a length of 100m and vertical deformations are measured by topographic levelling at different stages of the project;
- with electrical beam sensors placed on the railway sleepers; the beam sensor consists of an electrolytic tilt sensor attached to a rigid metal beam. The beam, one to two meters long, is mounted on anchor bolts that are set onto the sleepers. The sensors are linked end to end, as to allow displacement values to be accumulated from anchor to anchor to provide a profile of differential movements or settlement.
- with two optical strands, placed at a certain angle with the railway on the railroad tracks.

Figure 4 gives a top view of the instrumentation which is placed on or underneath the railroad tracks. It also shows the different HEB profiles which were instrumented.

A more detailed description of all installed equipment is given in Van Alboom et al. 2012.

1.3 Design of the jet grout wall
Calculations of the jet grout wall are implemented in FLAC2D by TUC Rail. They result in a maximum horizontal displacement of 21 mm, a maximum moment of 65KNm and a maximum settlement of 9mm behind the wall in the final excavation phase. A more detailed description of the calculations is given in Verstraelen et al. 2013.

1.4 Sequence of the execution phases
Table 1 gives the sequence of the execution phases, retrieved from photos made on site every hour. As continuous monitoring was performed, the influence of the executed works on the movement of the soil could be assessed.

<table>
<thead>
<tr>
<th>Date</th>
<th>Information</th>
</tr>
</thead>
<tbody>
<tr>
<td>22-23/10/2011</td>
<td>Installation of the instrumented HEB profiles</td>
</tr>
<tr>
<td></td>
<td>in the jet grout wall</td>
</tr>
<tr>
<td>3/11/2011</td>
<td>Excavation up to 1.25m below the top of the concrete</td>
</tr>
<tr>
<td></td>
<td>beam</td>
</tr>
<tr>
<td>06-09/11/2011</td>
<td>Installation of the first row of nails</td>
</tr>
<tr>
<td>06-07/12/2011</td>
<td>Excavation up to 3m below the top of the jet</td>
</tr>
<tr>
<td></td>
<td>grout wall</td>
</tr>
<tr>
<td>08-12/12/2011</td>
<td>Installation of the second row of nails</td>
</tr>
</tbody>
</table>
The measurements made by the 3 tenderers are continuous and data is transferred over the internet. The verification measurements, both with traditional equipment and innovative techniques are performed at discrete moments in time.

2.2 Moments in the jet grout wall

The other optical fiber instrumentation measures the strains in the HEB profiles and derivation of deformations can only be achieved by integration of the measured strains. This implies knowledge of the boundary conditions. Furthermore, the stiffness of the combination jet grout wall and HEB-profile needs to be estimated to obtain the deformation. It was found that there are a lot of assumptions which need to be made to obtain a reliable result. Further analysis will be attempted in the future.

From the inclinometers, the bending moment can be derived as:

\[ M = E I \frac{dE}{dx} \]  
\[ M = \varepsilon E W \]  

with \( E \) Young’s modulus, \( I \) the moment of inertia of the HEB profile and \( \alpha \) the inclination.

For the optical strand, measuring only 1 strain over its full length (top 10m of the HEB profile), the moment can also be derived with Eq.2, only this will result in 1 single bending moment and not in a bending moment as a function of depth.

For the optical strand, as this does not result in a bending moment as a function of depth. Unfortunately, a lot of the FBG sensors placed by FBGS failed during the first weeks after installation. For this reason, they are not included in the graph. For the stiffness of the wall, the stiffness of the HEB profiles was used, as the instrumentation is placed on these profiles.

The discontinuous and continuous measurements were not always measured on the exact same dates. However, all measurements are made after the final excavation phase.

Figure 6 shows that all derived bending moments give comparable values, except for the SAAF. This can be explained by the fact that the deformation measurements of the SAAF are less “smooth”. As the bending moment is a result of the derivative of the measured inclination, this results in unexpected peak values.

A maximum bending moment of about 55 kNm is derived from the measurements. This is again in the same order of magnitude as the calculations (maximum calculated bending moment of 65 kNm). For the optical strand, the derived bending moment is about 35kN.m, which is considerably lower than the other measurements. Due to the smoothing which is used for the BOTDR measurements, the peak bending moment is also reduced and has a value of 39kNm, which is also less than the value obtained by the inclinometer and the FBG technique from the BBRI. The shape of the bending moment curve however, is comparable to the other results.

From the fiber optics, which measure strains at regular intervals, the bending moment can be obtained as:

\[ M = \varepsilon E W \]  

The maximum horizontal displacement, measured after full excavation, amounts to 21 mm (26/04/2012) at a depth of 6.5m below the top of the grout wall for the inclinometer and 21.6mm for the SAAF. This lies very close to the calculated value of 22mm. It is found however that the direction at which the maximum deformation is found for the SAAF deviates slightly from the expected angle (perpendicular to the wall). This is probably due to a very small twist in between the different elements of the SAAF.
The “bump” in the measurements of the SAAF (circled in Figure 8) is caused by the lifting of the tracks on 25/02/2012, as explained above. This “bump” has no influence on the settlement which is measured at the end of the railway tracks.

The measurements made with the electrical beam sensors give comparable results to the measurement of the SAAF. The data measured with the optical strands appears to be uninterpretable. This is probably because it was in no way connected to a fixed point.

3 CONCLUSIONS

An extensive monitoring program was set up to compare different monitoring techniques. It appears that both new and traditional techniques can lead to the same result, when sufficient care is taken to the installation and interpretation. A significant advantage can be seen when continuous monitoring is applied, as the link with execution phases can be made.

4 ACKNOWLEDGEMENTS

The IWT is gratefully acknowledged for their support.

5 REFERENCES


Maintenance préventive des ouvrages hydrauliques par fibre optique

Preventive maintenance of water retaining structures based on fiber optic systems

Fry J.-J., Courivaud J.-R.
EDF-CIH, Le Bourget du Lac, France

Beck Y.-L.
EDF-DTG, Grenoble, France

Pinettes P.
Géophyconsult, Le Bourget du Lac, France

RÉSUMÉ : EDF est maitre d’ouvrage de plus de 500 km de digues, dont certaines qui forment une protection d’aménagements intéressant la sécurité publique. Afin d’améliorer la sûreté de son parc, EDF travaille au concept de maintenance préventive. Il s’agit de concevoir, réaliser et gérer un système de détection précoce d’anomalies (dont les principales sont les fuites et les déformations), un modèle d’interprétation et une chaîne d’interventions précises comportant alarmes techniques, administratives et travaux de réparation adaptés. Dans ce cadre, EDF travaille depuis 1994 à développer l’utilisation de mesures réparties par fibre optique pour améliorer la surveillance des digues. En effet, la technologie fibre optique permet d’avoir une mesure de température et/ou de déformation répartie tous les mètres le long de celles-ci, mesurée en continu. Cette nouvelle technologie, placée stratégiquement dans l’ouvrage, permet de compléter la surveillance habituelle du comportement hydraulique et mécanique, assurée à ce jour avec une instrumentation classique (nivellement, piézomètre, drain), par une surveillance à la fois globale, détaillée et surtout apte à la détection des phénomènes extrêmes et des états de crise (crue, séisme, vandalisme). Cet article présente le principe de la surveillance des digues de canaux par fibre optique ainsi que les résultats de validation de cette technologie tant à l’échelle de sites test que sur sites réels du parc d’EDF.

ABSTRACT: EDF owns more than 500 km of dikes, some of them protecting large critical structures involving public security. In order to improve the safety of its stock of power plants, EDF develops the concept of preventive maintenance. It means design, building and operation of an early warning monitoring system (leakage and strains), plus model of interpretation and portfolio of technical or legal alarm and interventions. In that framework, EDF has been working since 1994 on the development of the use of the distributed measurements with fiber optic, to improve the monitoring of dikes and flood embankments. The fiber optic technology provides a remote control measurement of the distributed temperature and strain every meter along the embankment. This new technology strategically placed in the fill, allows to reinforce the hydraulic and mechanical behavior monitoring, which is provided to date by conventional instrumentation (leveling, piezometer, discharge rate), with simultaneously a global and detailed surveillance and an early warning system for extreme loadings and crisis (floods, earthquakes, vandalism). This paper introduces the principle of dikes monitoring using fiber optic and the validation results of this technology from both trial test sites and on EDF’s real sites.

MOTS-CLÉS : remblai, barrage, digue, crue, séisme, sécurité, surveillance, auscultation, fibre optique, rupture, fuite

KEYWORDS: embankment, dam, levee, flood, earthquake, safety, surveillance, monitoring, fiber optic, failure, leakage

1 INTRODUCTION

En 1995, EDF est le premier gestionnaire d’ouvrages hydrauliques à tester la technologie de mesure de température répartie par fibre optique pour la surveillance des fuites d’une digue. Sur les digues, il est courant d’observer des fuites soudaines, sans que les piézomètres ne détiennent aucune variation de la surface libre. Cette observation, souvent répétée sur des ouvrages variés, montre à quel point le phénomène d’érosion interne, à l’origine de la fuite, est local. EDF, sensibilisé à ces manifestations d’érosion interne, lors de la mise en eau des digues d’Isère Moyenne Aval en 1991, en conclut que la détection précoce de l’érosion interne ne peut pas être envisagée avec l’espacement habituel des piézomètres, mais plutôt avec un espacement de l’ordre du mètre et un système d’acquisition marchant en continu. Dans le cadre d’un groupe de travail constitué avec la CNR en 1993, il apparaît que la méthode thermométrique de GTC, détectant les anomalies thermiques provoquées par un écoulément préférentiel dans le remblai avec des mesures de température dans des forages implantés tous les 20 m, est la méthode la plus efficace pour localiser la zone de fuite. D’autre part, Johansson propose à cette période une interprétation des mesures de thermométries, apte à en extraire la perméabilité et à suivre son évolution dans le temps. EDF en déduit en 1994 que la détection de l’érosion interne par l’augmentation de la perméabilité dans le temps est possible grâce aux mesures à l’interprétation de température, comme l’écrit plus tard Johansson (1997) dans sa thèse.

Cependant, la méthode n’est pas prédictive : l’intervention est curative. Il faut un nouveau saut technologique, pour détecter l’initiation de l’érosion interne, surveiller son évolution au fil du temps et décider d’une intervention ou non. Quel système a l’aptitude de mesurer la température tous les mètres et en permanence ? En discutant avec Jürgen Dornstäeder en 1994, patron de GTC, EDF met en parallèle le contrôle de l’érosion interne et le contrôle des câbles électriques enterrés. Ces derniers sont mis en place avec une fibre optique, en vue de détecter à tout moment, une éventuelle coupure accidentelle du câble. L’outil existe donc à cette époque ! Il reste à l’adapter au contexte de la surveillance des ouvrages hydrauliques. Cela nécessitera plus de 15 années d’étude… Dès le premier test avec une fibre optique d’une centaine de mètres, en juillet 1995, la détection de fuites, dans le canal de drainage en pied de la digue Cusset, montre que la détection des fuites est possible, mais qu’elle est dépendante de la technologie et exige une forte précision de la mesure. D’autre part, le diagnostic est entaché de
dificultés d’interprétation, suite aux nombreux biais physique: changement thermique de l’eau canalisé, rayonnement, pluie, perte de signal dans les soudures ou les rayons de courbure trop courts. Ce premier bilan montre l’importance de la difficulté, du temps et des moyens qu’il faudra développer, pour aboutir à une surveillance fiable. Cependant le fort enjeu de sécurité justifie
les nombreux efforts pour surmonter ces difficultés. EDF veut améliorer la sureté des aménagements les plus sensibles, en démontrant une détection précoce, un diagnostic fiable et une intervention préventive minimisant le coût la réparation. Depuis, EDF, maître d’ouvrage, développe et valide méthodiquement cette méthodologie (Fry 2004).

Cet article introduit le principe de cette maintenance préventive, montre les principaux résultats de qualification et évoque le déploiement opérationnel de cette technologie.

2 LA MAINTENANCE PRÉVENTIVE

2.1 La mesure par fibre optique

Le principe consiste à envoyer un rayon lumineux de laser dans une fibre optique standard, dont les défauts installés à pas régulier, souvent 1 m, vont rétrodiffuser le signal, qui sera analysé par un interrogateur optoélectronique et identifié par son temps d’aller-retour et son spectre en fréquence. Les pics du spectre, dépendant de la température et de la contrainte, mesurent d’une manière indirecte et surtout répartie la température et la déformation le long de celle-ci. Les interrogateurs optoélectroniques disponibles sur le marché actuellement permettent d’obtenir une mesure tous les mètres, avec une portée allant jusqu’à 20 à 30 km. En choisissant bien l’interrogateur et en utilisant des fibres standard télécom multimodes, la précision est de 0,1°C pour une mesure de température seule avec un interrogateur Raman et une distance inférieure à 10 km. À partir d’une fibre optique monomode contrainte dans un câble, la précision des mesures de température et de déformation par un interrogateur Brillouin est de l’ordre de 1°C et de 20µm/m.

2.2 Surveillance active ou passive

La méthode thermométrique initiale de GTC mesure une seule fois la température. Comme ce n’est souvent pas suffisant, GTC développe un test de convection forcé, où la température est imposée par chauffe ou réfrigération (Dornstadter J. 2010). Ce mode opératoire est appelé méthode passive, en opposition à la méthode active, consistant à mesurer la température naturelle. Le débat, autrefois vif entre les partisans de l’une ou de l’autre approche, laisse maintenant une place à chaque méthode.

Johansson (1997) n’utilise que la méthode passive associée à une modélisation numérique des échanges thermiques. EDF montre qu’une interprétation simpliste de la méthode passive dans un milieu non saturé aboutit à un diagnostic faux. Pour surmonter cette incohérence, EDF développe d’autres modèles d’interprétation, valables quelle que soit la position de la fibre optique ou celle de la nappe. Quatre modèles sont complémentaires (Beck & al. 2010), les deux premiers sont physico-statistiques, issus de la pratique de l’auscultation, tandis que les suivants sont basé sur le traitement du signal. La méthode passive a l’avantage de permettre la surveillance permanente et d’identifier les fuites à plus d’un mètre de la fibre. Son inconvénient est de nécessiter la mesure de la température de l’eau stockée ou canalisée et celle de l’air et un modèle d’interprétation complexe à plusieurs niveaux de traitement.

Perzlmaier, Auflerger et Dornstadter (2007) ont été les précurseurs de la méthode active, en entourant la fibre optique d’un câble de cuivre chauffé par effet Joule sur une courte période de temps. La puissance électrique nécessaire est de 3 à 15 W/m. Cette méthode présente l’inconvénient d’être applicable uniquement à des tronçons limités de digues (< 2 km), d’avoir un faible rayon d’action (< 20 cm autour du câble), de ne pas permettre une surveillance en continu et de nécessiter quelques précautions de conception et d’utilisation afin d’assurer la sécurité du personnel.

2.3 Le concept de maintenance préventive

Le choix d’un système de surveillance par fibre optique n’est pas justifié sur tous les ouvrages. Une stratégie basée sur l’évaluation du risque ne montre pas de bénéfice à équiper ni les grands barrages de bonne conception, dont la sécurité est assurée par une surveillance habituelle, ni les petits barrages de risque limité, dont la rupture aurait peu de conséquences. À l’opposé, un système de détection par fibres optiques apporte un gain justifié quand l’ouvrage est de grande longueur, sur une fondation mal connue ou édifié avec une conception non conforme à l’état de l’art actuel, dont le cout de réparation est lourd pour le maître d’ouvrage, alors que des zones de faiblesses sont suspectées sans qu’elles soient localisées, laissant un doute sur la sécurité dans le temps ou en conditions extrêmes.

Le système de surveillance par fibre optique apporte dans tous les cas un complément au réseau habituel d’auscultation. C’est un outil d’aide à la gestion de crise. Que se soit après un séisme ou une crue, il a la capacité de localiser en temps réel les zones à risque et donc d’améliorer la gestion des ressources et d’accroître l’efficacité des interventions. Le choix de la méthode et du modèle d’interprétation est lié à la stratégie du maître d’ouvrage : surveillance long terme ou court terme, pathologies à suivre, gestion du risque sismique. La mise en place de ce système de surveillance est identique à celui de la méthode observationnelle, en réunissant quatre conditions :

1. Les limites admissibles du comportement en température et/ou en déformation sont évaluées par modélisation avant d’être mesurées;
2. Le domaine des variations possibles du comportement thermique ou cinématique est jugé acceptable. Un tel système d’auscultation n’est installé que si la marge de sécurité est jugée acceptable, dans le cas contraire il s’agit de programmer en urgence la réhabilitation;
3. Le programme de suivi est établi pour vérifier si le comportement réel reste dans les limites admissibles. La fréquence d’acquisition est suffisamment élevée pour que le dispositif puisse détecter l’apparition de toute faille, vérifier par la progression du phénomène s’il s’agit d’érosion interne et laisser le temps de réparer. Le choix de la méthode passive ou active doit être justifié par rapport à la cinématique des phénomènes attendus;
4. Un programme d’interventions d’urgence est défini au cas où le suivi révèle un comportement sortant des limites admissibles.

L’avantage de la maintenance prédictive est de limiter les zones endommagées par la détection précoces des pathologies menant à n’import quel mode de rupture. Il existe 3 modes de rupture potentiels : l’érosion externe, l’érosion interne et le glissement. Chacun de ces trois modes est intercepté par le système de surveillance à base de fibres optiques de mesure de température et de déformation. Les mesures de température repèrent le risque de rupture par érosion et les mesures de déformation détectent le risque d’instabilité géométrique. Cela est bien démontré par les résultats du projet Ikdiik en Hollande (Koelewijn A. 2010) et (Beck & al. 2010).
surveillance, ne garantit pas à elle seule la sûreté. Le gouvernement hollandais en prend conscience. Il décide de lancer le projet IJdijk en vue de sélectionner les meilleures technologies de détection précoce de la rupture. EDF, associé à Tencate, Géophyconsult et au projet PAREOT, voit une opportunité hors du commun de tester son approche de la sûreté.

3.1 Rupture par instabilité générale

Le premier test en 2008 concerne la détection du glissement d’une digue, chargée par des containers et dont le pied est excavé. 4 fibres optiques mesurent l’elongation du parement aval (Figures 1 à 3). La surface de rupture est bien détectée.

3.2 Rupture par érosion interne

Le système de détection est de nouveau testé sur les 4 essais de détection de rupture par renard des digues expérimentales du projet IJkDijk en 2009. Les remblais mesurent 3,5 m de hauteur, 15 m de longueur et ont un fruit H/V=2/1 à l’amont et aval. Les fibres optiques passives sont installées avec Tencate dans la fondation au contact du remblai (Figure 4). La rupture est obtenue par érosion régressive en montant par palier le plan d’eau amont. La durée de l’essai varie de 4 à 6 jours. La détection visuelle et la détection par analyse du signal sont représentées respectivement sur les figures 5 et 6.
température par analyse du signal détecte un clair précurseur de la rupture plusieurs jours avant l’analyse brute des données. Sur la Figure 6, l’absence d’anomalie au cours de la première journée d’essai (19-20/10) correspond bien à l’absence d’observation de fuite sur le terrain. Les jours suivants, des fuites sont observées et visualisées par un rectangle bleu, alors que l’entraînement et le dépôt de sable est représenté par un rectangle rouge. Le modèle d’analyse détecte bien une fuite croissante vers l’abscissee 9-10 m, là où la rupture va se produire 5 jours plus tard.

Pour chacun des quatre essais, le modèle d’analyse développé par EDF apparaît plus performant que la visualisation des mesures brutes (tableau 1). Le temps de détection varie d’un test à l’autre, car les fibres n’ont pas pu être posées exactement aux endroits demandés par EDF. Cela montre l’importance de la localisation des fibres dans la conception du système de détection.

<table>
<thead>
<tr>
<th>Test</th>
<th>Durée (jours)</th>
<th>Détect. visuelle</th>
<th>Détect. analy. du signal</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>4</td>
<td>3 heures</td>
<td>3 jours</td>
</tr>
<tr>
<td>2</td>
<td>6</td>
<td>48 heures</td>
<td>5 jours</td>
</tr>
<tr>
<td>3</td>
<td>5</td>
<td>31 heures</td>
<td>3 jours</td>
</tr>
<tr>
<td>4</td>
<td>5</td>
<td>21 heures</td>
<td>1 jour</td>
</tr>
</tbody>
</table>

3.3 Bilan des tests de validation de Ljdkijk

Que ce soit pour la détection du glissement ou de l’érosion interne, le système de fibres optiques accompagné d’un robuste modèle d’interprétation apparaît la méthode la plus performante (robustesse et justesse) parmi les technologies de détection testées par la trentaine de participants du projet international.

Conforté par ces résultats d’inter-comparaison des pratiques internationales, EDF programme et déploie cette maintenance préventive en 2010 sur une série de travaux visant à réhabiliter certains biefs hydro-électriques et à optimiser leur maintenance.

4 APPLICATIONS

EDF dispose en 2012 de quatre ouvrages instrumentés et suivis par fibre optique. Ainsi, le canal de Curban, de 5 km de longueur, est instrumenté par 11 km de fibre optique et est aujourd’hui suivi périodiquement par des mesures de température actives et passives. Des mesures de déformation sont faites à une fréquence plus faible. La surveillance par fibre optique de la remise en eau de l’ouvrage à la fin des travaux de réhabilitation contribue à détecter rapidement les zones de fuites (résultats de surveillance toute les 4 heures) et à diminuer les pertes d’exploitation remboursant son coût d’installation. La surveillance actuelle de l’ouvrage en exploitation montre une bonne corrélation entre les résultats d’analyse des données de température par fibre optique et les débits de drainage sous l’étanchéité rénovée, captés tous les 300 mètres. Ils mettent en évidence une décroissance due au phénomène de colmatage. Les mesures de déformation identifient trois secteurs de déformations différentielles à suivre de près.

5 CONCLUSION

La maintenance préventive par un système de fibres optiques permet une mesure en continue et répartie dans l’espace d’anomalies thermiques ou mécaniques, une détection précoce des risques de rupture, une optimisation des ressources en situation de crise et une économie sur les durées d’indisponibilités ou le coût des travaux de réhabilitation. Elle est adaptée aux ouvrages de grand linéaire ou à ceux qui posent des difficultés de conception ou sont très sensibles au niveau de la sûreté.

6 REMERCIEMENTS

Nous remercions les nombreux partenaires (de Deltares, EDF R&D, GTC, Hydrosreach, Irex, Irstea, Tencate, Université de Grenoble) qui ont accompagné EDF et ont contribué à la qualification de la technologie à base de fibres optiques pour la surveillance des ouvrages hydrauliques.

7 BIBLIOGRAPHIE


Evaluation of diaphragm wall as-built data to determine the risk of leakage for the Kruisplein car park excavation in Rotterdam, The Netherlands

Hannink G.
Engineering Consultancy Division, City of Rotterdam, The Netherlands

Thumann V.M.
Seaway Heavy Lifting Engineering B.V, The Netherlands, formerly Engineering Consultancy Division, City of Rotterdam, The Netherlands

ABSTRACT: In the centre of Rotterdam, the Kruisplein car park is becoming the deepest underground parking facility in the country. It is being constructed since 2009, and will provide around 760 parking places on a total of five floors, of which the deepest is reaching to almost 20 m below ground surface. The retaining wall has been designed as a diaphragm wall reaching from ground surface to approximately 40 m depth. Because of some major leakage incidents in diaphragm wall type excavations in Rotterdam and elsewhere in The Netherlands, additional effort was raised to define and prepare mitigating measures to reduce the risk of leakage for the Kruisplein car park project. An extensive evaluation of as-built data of the diaphragm wall was made, including all available field records of process-parameters. Based on the outcome of the evaluation, it has been concluded that in general the diaphragm walls were of sufficient quality. No major leakage incidents have occurred to date.

RÉSUMÉ : Dans le centre de Rotterdam, le parking de la place Kruisplein est en train de devenir le plus profond garage souterrain du pays. Ce parking est en construction depuis 2009 et pourvra 760 places de stationnement réparties sur cinq niveaux. Le plus profond d'entre eux atteint plus de 20 m de profondeur sous la surface du sol. Le mur de soutènement a été conçu comme une paroi moulée atteignant approximativement 40 m de profondeur. À cause de divers accidents majeurs de fuites d'eau survenus dans des excavations utilisant des parois moulées à Rotterdam et ailleurs dans les Pays-Bas, des efforts supplémentaires ont été fournis pour définir et préparer des mesures d’atténuation pour réduire le risque de fuite lors du projet du parking Kruisplein. Une évaluation étendue des données des parois moulées, incluant toutes les données de chantiers enregistrées lors de leur fabrication, est présentée. Il n'y a eu jusqu'à ce jour aucun incident majeur de fuite sur le chantier de Kruisplein.

KEYWORDS: diaphragm wall, deep excavation, risk of leakage, field records, as-built data, mitigating measures.

1 INTRODUCTION

In Rotterdam diaphragm walls are regularly applied since the construction of the Willem Railway tunnel in the nineties of the past century. For underground infrastructural projects, it is an appropriate building method, because it facilitates deep excavations under dry conditions. By connecting the diaphragm walls to the rather impermeable soil layers of the Formation of Waalre a dry building pit can easily be created. The geological conditions are favorable for this building method: in the centre of Rotterdam the rather impermeable layers of the Formation of Waalre are everywhere present between 35 and 40 m below sea level (this corresponds to the Dutch reference level NAP).

Diaphragm walls have been applied at several locations of the metropolitan light rail project RandstadRail in the beginning of this century. During the extension of metro station CS a major leak through the diaphragm walls arose when the excavation inside the building pit was at its maximum depth (14 m below sea level). As a result a huge amount of water and sand entered the building pit. With a lot of trouble, the leak was fortunately stopped within two days, and the construction of the metro station could be continued (Thumann et al. 2009).

In the same period the preparation of the underground car park Kruisplein was in full swing. The design consisted of 40 m deep diaphragm walls to make a 20 m deep excavation possible. A logical question at that moment was, how to minimize the possibility of a similar incident during the construction of the car park. This paper describes the mitigating measures taken in advance, the supervision during the construction, and the effectiveness of the precautionary measures in practice.

The construction of the Kruisplein car park is part of the overall project Rotterdam Centraal (Hannink & Thumann 2007). This major project comprises of the building of a large Public Transport Terminal in the vicinity of the Rotterdam Central Railway Station. It is designed to facilitate passenger transfer between (inter)national trains including the high-speed train, and local public transport like trams, buses and underground trains. The excavation as required for the construction works of the Kruisplein car park covers about 5.000 m².

Engineering of the car park Kruisplein, and supervision of the execution of the project is performed by the Engineering Consultancy Division of the City of Rotterdam.
2 SOIL CONDITIONS

The ground level in the area is situated at about sea level. The geotechnical profile of the Rotterdam city area consists of anthropogenic layers (from ground level to about 5 m below sea level), and soft Holocene peat and organic clay layers (from about 5 to 17 m below sea level). Below this level Pleistocene coarse sand layers are encountered up to 35 to 40 m below sea level. These sand layers are underlain by the Formation of Waalre, consisting of over consolidated clay and sand layers. Figure 2 shows the result of a CPT. The phreatic groundwater level is about 2 m below sea level.

![Figure 2: Results of a CPT at car park Kruisplein.](image)

3 RISK ANALYSIS

In this project the principal is responsible for the design, and the contractor is responsible for the construction. The contractor is obliged to present plans about the way the risks connected to the building method are controlled, including relevant solutions and mitigating measures.

The possibility of leakage into a building pit with diaphragm walls is small, but the consequences may be very serious in case groundwater flows into the building pit. Especially in an urban environment a sand carrying leak is seen as a huge risk. Piles are founded in the sand layer with the top at 17 m below sea level. This is the same sand layer that may flow into the building pit. There are several possible causes for leakage out of this sand layer:

- the panels of the diaphragm wall are insufficiently connected;
- the base of the panel is not or insufficiently connected to the impermeable layer, for example as a result of the presence of an obstacle;
- the concrete of the panel contains intrusions of sand, peat or clay, that form weak spots in the diaphragm wall;
- the concrete of the panel contains bentonite, that forms weak spots in the diaphragm wall.

The starting point in the design stage of the car park was that the diaphragm walls would be placed at least 1.5 m into the impermeable layers of the Formation of Waalre. At the deepest point of the excavation the diaphragm wall must be able to retain a groundwater pressure difference of about 20 m.

Measures to minimize the possibility of a leaking diaphragm wall were prescribed in the contract. However, to minimize the possibility of leakage, the most was expected of measures that could facilitate the building process.

The outcome of the risk analysis indicated that to minimize the possibility of a leaking diaphragm wall:
- additional requirements should be prescribed in the contract;
- early observations of imperfections during the building process are of utmost importance;
- the execution of the project has to be monitored adequately;
- the analysis of as-built records is essential as to identify hazardous locations.

4 MEASURES IN THE CONTRACT

The building contract included Dutch standard RAW specifications regarding quality control of the diaphragm wall building process. The following additional contract requirements have been defined:

- the verticality of the panels shall be within 0.5% of the depth in both transverse and longitudinal directions;
- the horizontal deviations of the exposed face of a panel shall be less than 100 mm;
- 150 mm wide water-stops (rubber profiles) shall be put into every steel stop end of the diaphragm wall;
- the concrete surface of adjacent panels shall be cleaned from bentonite cake before the commencement of concreting;
- the maximum rate of concrete rising in the trench shall be 6 m/h in the Holocene clay and peat layers;
- a good connection between the floors and the diaphragm walls shall be secured. It is important not to drill unnecessary additional holes for reinforcement bars into the diaphragm walls. Zones of overlap of the reinforcement of the diaphragm wall were not allowed at the locations of the reinforcing bars;
- the maximum aggregate particle size of the concrete shall not exceed 16 mm.

Concreting records of the diaphragm wall panels had to be made to register the following possible execution imperfections:

- the time during which the trench is left open before concreting;
- the deviations of the steel stop ends;
- the deviations of the reinforcement;
- steel stop ends which are left behind;
- discontinuities in concreting.

After completing the diaphragm wall, and before starting the excavation, a check had to be made on the permeability of the building pit by means of generating a 20 m pressure difference between inside and outside of the diaphragm wall. This pumping test was meant to deliver information about the water tightness of the rather impermeable layers at 40 m below sea level. A successful test however, does not exclude leakage in the execution phase, because the diaphragm wall will be excavated at one side, and will deflect. This may result into open joints between the panels.

According to the contract four boreholes for so-called 'sleeping' wells had to be drilled around the building pit, to be able to act quickly in case of a leak. The purpose of these 'sleeping' wells was, in case of a calamity, to decrease the difference in water pressure between the inside and the outside of the building pit as soon as possible. This will make it easier to control the amount of groundwater that penetrates the building pit. The installation of pumps and mains was not required in the contract. The idea was that in case of a calamity the mobilization period would be limited.
5 MEASURES DURING THE EXECUTION

It was considered to be important to specify the most vulnerable processes into a number of documents that made it possible to control the processes:
- the excavation plan describes the sequence of the execution of the panels of the diaphragm wall as to minimize the possibility that a leak will occur during the night;
- the supervision plan for the construction of the diaphragm wall and for the excavation of the building pit is meant to detect imperfections in an early stage during the excavation;
- the calamity plan describes the risks connected to the construction of the diaphragm wall and the available mitigating measures at the moment of signaling a (potential) leak or a threatening calamity;
- results of monitoring activities and records give detailed information on the execution of each panel, and of possible imperfections. It is of utmost importance that the content of the documents and the point in time of handing them in are mutually agreed;

The results of the supervision by both the principal and the contractor are discussed at the building meetings, and a separate regular monitoring meeting with all persons concerned was convened.

6 ASSESSMENT OF DIAPHRAGM WALL QUALITY

All relevant data have been evaluated to determine potential weak-spots in the diaphragm wall along the excavation circumference.

6.1 Pumping test

The pumping test to check the water tightness of the diaphragm walls was executed in June 2010. The measuring results showed that the water tightness of the building pit as a whole met the requirements as formulated in the permit for water extraction. This implicated that the average quality of the diaphragm walls came up to expectations about water tightness in not excavated circumstances.

6.2 Field observations

Field observations by both the contractor and the supervisors of the City of Rotterdam were meant to record regular and any extraordinary circumstances during the building process of each individual panel. In practice, general data on the duration of the consecutive building stages (a.o. excavating, refreshing of bentonite suspension, lowering of steel reinforcement, concreting) and identification of excavated soil type (sand or clay) have been recorded. For some panels, underground obstacles were encountered.

6.3 As-built documents

Information from as-built documents has been thoroughly examined to detect any hazardous sections of the diaphragm panel. These data have been visualized as for example shown in Figures 3 to 5. Figure 3 shows amongst other things the elapsed time between:
- the start of the excavation of the trench and the start of the installation of the steel stop ends;
- the start of the installation of the steel stop ends and the end of the excavation of the trench;
- the end of the excavation of the trench and the start of the cleaning of the concrete surface;
- the start of the cleaning of the concrete surface and the start of the installation of the reinforcement cage;
- the end of the installation of the reinforcement cage and the commencement of concreting.

Long periods of elapsed time for a particular activity may indicate an increased risk of imperfections.

Figure 4 shows the calculated deviation of wall thickness as can be calculated from the concreting progress reports. From Figure 4 it can be identified where the panels are suspected to have a reduced thickness of more than 0.2 m. However, it is recommended to have more detailed information on the progress of the concreting process for future projects, as to increase the resolution (reliability) of this graph.

Figure 5 shows the position of all diaphragm wall panels at 40 m depth, as derived from crane operating monitoring equipment. Most of the panels have been excavated in two or three parts, thus giving at least two monitoring records (inclination and deviation vs. excavation depth) per panel. From Figure 5 it can be identified where the panels are suspected to have insufficient overlap. The diaphragm wall thickness as designed was 1.20 m; the allowable position with respect to overlap (zone width of 1.60 m at 40 m depth) follows from the 2% deviation of the verticality.
Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris 2013

Figure 5. Relative position of the panels at 40 m depth. Dark represents the outside of the diaphragm wall; light the inside. The dotted red lines represent the 2% deviation of the verticality.

6.4 Non-standard measurement techniques
Reference is made to Doornenbal et al. (2011) and Spruit et al. (2011) for more information on the experiments using non-standard measurement techniques to detect imperfections in diaphragm walls, which appeared to be quite successful.

6.5 Increased risk of leakage
Combined interpretation of Figures 3 to 5 reveals areas where an increased risk of leakage has to be anticipated for, as compared to normal conditions. Following aspects have been evaluated using the outcome of the interpretation of the as-built records:
- a missing/damaged water-stop;
- no cleaning of the concrete surface;
- the elapsed time between refreshing of the bentonite and the commencement of concreting greater than 24 hours;
- encountered problems during the stop end removal works;
- concrete characteristics;
- wall thickness reduction > 0.20 m (Figure 4),
- reduced overlap < 0.80 m (Figure 5).

An overview of locations with an increased risk of leakage along the excavation circumference has been generated as shown in Figure 6.

Subsequently, a number of mitigating measures has been defined for each individual panel as to compensate the increased risk. It appeared from the overall risk analysis that careful positioning of four ‘sleeping’ wells around the excavation would provide sufficient means for acting in case of leakage at any of the identified weak spots as shown in Figure 7. Additionally, for a number of panels it has been recommended to call for an intensified inspection and repair program (when necessary) during the excavation works.

7 CONCLUSIONS
The realized diaphragm wall panels were in general of sufficient quality. A number of hot spots along the circumference of the building pit with an increased risk of leakage were identified. The risk profile related to leakage was considered to be at an acceptable level if a number of mitigating measures were executed. These measures were supplementary to the required measures in the contract.

A pumping test to check the water tightness of the diaphragm walls is very valuable in case the subsoil conditions are similar to those in Rotterdam.

Supervision of the construction process appeared to be an important mitigating measure, in combination with the registration of the execution data of the diaphragm wall, and the subsequent analysis of these data.

The extensive acquisition of data as such is not new for the construction of diaphragm walls, but the systematic analysis of the data, as performed for this project, has not been noticed so far. It is recommended to do so for all future projects.

The positioning of ‘sleeping’ wells can be considered as a major mitigating measure, and is a lesson learned from the construction of the (leaking) diaphragm wall at metro station CS.

ACKNOWLEDGEMENTS
The work on this subject of former colleagues Edwin Dekker and Rens Servais is gratefully acknowledged. Records of the execution of the diaphragm walls have been provided by the contractor Besix BV. Colleague Arie van de Heerik collected most of the records, and Ton de Keiser prepared the illustrations in this paper.

REFERENCES
EN 1538 2010. Execution of special geotechnical work – Diaphragm walls.
Optimisation of bridge approach treatment via staged construction

Optimisation du traitement de remblais d’accès à des ponts par phasage des travaux

Hsi J.P., Carson D.J., Lee C.H.
SMEC Australia Pty Ltd, Australia

ABSTRACT: This paper describes an improved approach to bridge embankment transition design and construction staging that was utilised to overcome financial and programme challenges associated with the proposed initial design solution for bridge approach embankments. An alternative staged approach was developed for construction, comprising improvement of the strength and compressibility characteristics of the soft soil foundation by surcharging techniques in combination with use of prefabricated vertical drains (PVD) and high strength geotextile. Unreinforced continuous flight auger (CFA) columns were installed after surcharging to achieve smooth transition at bridge approach embankments. During construction, the behaviour of the foundation under load was closely monitored and back analysis of the performance of the improved foundation was undertaken. Construction stage design optimisations were then made to satisfy the design criteria using actual monitoring data. This approach to bridge embankment transition design provided ability for the entire subsurface profile to accommodate the applied embankment loading. As a result, major cost, programme and environmental benefits were realised during construction by avoiding the installation of approximately 88,900 lineal metres of concrete foundation piles that were specified in the initial design.

RÉSUMÉ : Cet article décrit une approche améliorée pour la conception et le phasage des travaux de remblais d’accès aux ponts. Cette approche a été utilisée pour répondre aux contraintes financières et de planning associées à la solution initiale proposée. Une approche alternative en termes de phasage des travaux a été développée et comprenait l’amélioration de la résistance et de la compressibilité du sol de fondation (argile molle) par l’installation de remblais de chargement, de drains verticaux préfabriqués et de géotextiles haute performance. Des colonnes en béton ont été installées après la période de chargement pour assurer une transition en douceur au niveau des remblais d’accès au pont. En phase construction, le comportement du sol de fondation sous la charge était étroitement contrôlé et une évaluation de la performance en ce qui concerne l’amélioration actuelle du sol de fondation a été réalisée. Sur la base des mesures effectuées sur chantier, certains paramètres de conception ont été optimisés en phase travaux pour satisfaire aux exigences du projet. Cette méthode de conception des remblais d’accès aux ponts a fourni à l’ensemble du sous-sol la capacité de supporter le chargement qui s’applique sur le remblai. Ainsi, de conséquents gains financiers, de temps et environnementaux ont été réalisés en phase travaux puisque cette solution a évité l’installation d’environ 88,900 mètres de pieux en béton, spécifiés dans les études initiales.

KEYWORDS: Ground improvement, bridge approach transition treatment, prefabricated vertical drain, CFA column, preloading.

1 INTRODUCTION

As a state government initiative, the AUD $1.88B Gateway Upgrade Project in Brisbane Australia involves the design, construction, operation and 10 year maintenance (DCOM) of a new Gateway Bridge, existing Gateway Bridge refurbishment, 12km of motorway upgrade and 7km of new motorway.

Located along Brisbane’s north south arterial transportation corridor, the project provides improved connectivity to infrastructure such as Brisbane’s Trade Coast region, Airport and the Port of Brisbane. Construction completion for the entire project occurred during November 2010.

Delivered by Queensland Motorways Limited (QML) in partnership with Leighton Abigroup Joint Venture (LAJV) and principal designers Maunsell SMEC Joint Venture (MSJV), the project involved construction and refurbishment of 30 bridge structures. Fourteen (14) of these bridges are located within the Brisbane Airport Interchange precinct, which is characterised by soft, compressible foundation soils up to 20 m in thickness, with road embankment heights up to 13m.

Initial design for the bridge approach treatment in this area comprised use of various forms of piled embankment supported by a mixture of approximately 4,900 continuous flight auger (CFA) piles, displacement auger piles, pre-stressed concrete piles and dynamic replacement columns.

Following cost and program analysis, an alternative staged ground treatment approach was proposed and adopted for the construction of 14 of the 28 bridge approaches within the Airport precinct. This paper focuses on one such abutment (denoted as BR25A) within this area. Site based geotechnical characteristics are identified together with key aspects of the initial and alternative design approach, summary of the alternative design methodology, comparison between predicted and actual ground settlement and outcomes successfully delivered through utilisation of a staged approach to ground treatment.

2 GEOTECHNICAL CHARACTERISTICS

2.1 Subsurface conditions

Geotechnical investigations indicated that the Airport Interchange is underlain by up to 20m Holocene (upper and lower) and Pleistocene alluvial deposits.

Upper Holocene alluvium within the Airport Interchange area was characterised by variable deposits of clay and silt (UH-C) and sands (UH-S). Lower Holocene alluvium (LH-C) was found to be of more uniform composition, comprising compressible silty clay to up to 20m depth.
Deposited during previous lower sea level, Pleistocene clay (P–C) was found to be characterised by less compressible stratum including stiff to hard clays and medium dense (or denser) sand layer. The ground water level was observed to be within 1–2m of the natural ground surface. The design ground water table was assumed at the ground surface level prior to construction (approximately RL 1.3m).

2.2 Geotechnical design parameters

The geotechnical parameters adopted for design and back-analysis of BR25A bridge approach are summarised in Table 1. Design parameters were derived taking into consideration the potential variability in the ground conditions and were calibrated against monitoring results during construction stage. The coefficient of consolidation in the horizontal direction (c_h) was assumed to be \(2c_v\) and this ratio was found to be appropriate based on the back-analysis of field measurements.

Site investigation data indicated variation in strength, compressibility and hydraulic conductivity with depth and location within the Airport Interchange area. Field results from this vicinity indicate that the undrained shear strength (\(C_u\)) of the compressible clay increases with depth from approximately 10kPa to 60kPa. \(C_u\) values derived from piezocone were calibrated against the shear strength determined from the field shear vane. For geotechnical design, a characteristic \(C_u\) value of \(20 + 0.6z_1\) (kPa) for UH-C and \(23.6 + 2.7z_2\) (kPa) for LH-C was selected, where \(z_1 = 0\) at RL 0 and \(z_2 = 0\) at RL -6. Over-consolidation ratios (OCR) were derived from Oedometer and piezocone data. Figure 1 shows field and laboratory test results.

3 ALTERNATIVE DESIGN DETAILS

The alternative design philosophy involved initially improving the shear strength and compressibility characteristics of the soft soil by 6 months preloading in combination with placement of 4.3m surcharge. High strength geotextile (2 layers of WX600/50) and prefabricated wick drains (1.0m triangular pattern) were utilised for stability control. Refer to Figure 2 for schematic design arrangement nominated during design stage 1.

To facilitate construction haulage, a 2m high temporary berm in the longitudinal direction was proposed and this stabilising effect was incorporated in the design. The use of temporary berm achieved a reduction in the high strength geotextile requirement for stability control.

Following conclusion of preload, installation of final settlement transition treatment was anticipated, following review of actual performance of the embankment during preloading. The ground transition treatment for the alternative approach comprised 3 transverse rows of unreinforced concrete CFA columns (0.6m diameter on a 2.5m square grid with a UCS of 40MPa) overlain by a 20m long geotextile reinforced mattress to provide adequate pavement transition (see Figure 3). Two layers of WX1100/100 were specified in the longitudinal direction and one layer of WX200/50 in the lateral direction for the geotextile mattress. As a Stage 3 optimisation, 1m of embankment fill was excavated and replaced with lightweight fill (flyash) to increase the final over-consolidation ratio of the foundation soils and decrease preload period from 6 months to 2.4 months.

4 ALTERNATIVE DESIGN METHODOLOGY

The alternative design comprised a 3 staged approach to design, which occurred across the design and construction stages for the BR25A bridge approach.

4.1 Stage 1 methodology

Stage 1 involved undertaking design calculations to predict the required ground treatment to meet the settlement and stability criteria for the bridge approach transition. To meet the prescribed settlement criteria of 50mm (max) at the abutment
Table 1: Geotechnical Design Parameters

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (top of layer) (m)</th>
<th>( \gamma_t ) (kN/m³)</th>
<th>( c' ) (kPa)</th>
<th>( \phi' ) (°)</th>
<th>( E' ) (MPa)</th>
<th>( v )</th>
<th>OCR</th>
<th>( C_{\infty} ) (m²/yr)</th>
<th>( C_{ui} )</th>
<th>( C_{uw} )</th>
<th>( C_{um} )</th>
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<tr>
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<td>30</td>
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<td>0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>UH-C</td>
<td>0.5</td>
<td>17.0</td>
<td>2</td>
<td>27</td>
<td>-</td>
<td>-</td>
<td>2.5</td>
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<td>0.03</td>
<td>0.01</td>
<td>6.0</td>
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<tr>
<td>UH-S</td>
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<td>0</td>
<td>30</td>
<td>10</td>
<td>0.3</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>LH-C</td>
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<td>17.0</td>
<td>2</td>
<td>27</td>
<td>-</td>
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<td>1.5</td>
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<td>P-C</td>
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<td>-</td>
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<td>-</td>
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<td>-</td>
<td>-</td>
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<td>-</td>
</tr>
</tbody>
</table>

Note: Figures shown in brackets are values used in back analysis.

Figure 3: Typical operational stage bridge approach transition design arrangement proposed during Stage 1

Soil structure interaction of bridge approach transition treatment was analysed in PLAXIS. The prescribed settlement method was used to analyse the differential settlement within the transition zone due to creep effect. Post construction creep settlement was firstly estimated. Subsequently, the ground behind the CFA columns was then prescribed to settle by an amount equal to the estimated post construction settlement. The embankment change in grade over any 4m length of pavement due to differential settlement was then calculated.

To meet the stability criteria (minimum safety factor of 1.2 in short term and 1.5 in long term), the embankment construction was constrained at a rate of 1m per week. Accordingly, strength gains due to consolidation of the UH-C and LH-C layers were able to be considered in the design. Time rate of consolidation of the UH-C and LH-C layers was further accelerated by use of PVD’s. High strength geotextile in combination with lateral stability berms was utilised to provide additional stability control during construction. Stability analysis and design of soil reinforcement were carried out in accordance with the method outlined in Hsi and Martin (2005).
embankment construction in PLAXIS and then comparing the primary settlement obtained from the modelling to the settlement actually observed in the field (see Figure 4). Calibrations were then made to the soil model to achieve an acceptable match between observed and predicted behaviour. The magnitude of primary settlement inferred by the Asaoka (1978) method using a constant time step of 7 days was compared to the actual field data and numerical predictions as an additional validation check on the degree of consolidation achieved. A further validation was undertaken by comparing the actual degree of excess pore water pressure dissipation recorded by the piezometer against the degree of excess pore water pressure dissipation calculated by FEM during stage 3 back analysis (see Figure 4). The magnitude of creep settlement was estimated based on the methods described in Mesri and Feng (1991), Mesri et al. (1997) and Stewart et al. (1994) and compared with the design criteria. The recommended preload duration was then refined to ensure that the predicted post construction settlement met the design criteria.

5 RESULTS
As shown in Figure 4, the magnitude of primary settlement predicted in Stage 1 was significantly greater than the actual primary settlement recorded during Stage 2 field monitoring. Compressibility and consolidation parameters were calibrated (calibrated parameters shown bracketed in Table 1) to achieve a good agreement between Stage 2 actual settlement results and settlement back calculated at Stage 3. From iterations during the Stage 3 back calculation, the source of the difference between Stage 1 and Stage 3 settlement predictions was partly attributed to the higher modified compression index $C_{c1}$ and modified recompression index $C_{r1}$ adopted during Stage 1 design. As a result, the modified secondary compression index $C_{c2}$ was also amended. As a further validation check, the primary settlement was also calculated using the Asaoka (1978) method. Using this method, primary settlement of approximately 1.79m was estimated, which compared reasonably well to the Stage 3 back calculated primary settlement estimate (1.80m).

The degree of excess pore water dissipated as measured by the piezometer during Stage 2 was compared against the degree of excess pore water dissipation from the Stage 3 back calculation. This comparison provided an additional validation check in relation to the estimated degree of consolidation of the compressible soils. A reasonable agreement between the measured (Stage 2) and back calculated degree of excess pore water dissipation (Stage 3) of the compressible soil was observed (see Figure 4).

6 CONCLUSIONS
An alternative staged approach to design and construction successfully achieved reductions of over 88,900 lineal metres of ground improvement piling that was specified in the initial design. BR25A approach has been presented as a ground treatment design case study; providing key geotechnical considerations, design methodology and a comparison of actual embankment performance with design predictions.

7 REFERENCES
SWOT analysis Observational Method applications

Analyse FFOM à l’implémentation de la méthode observationnelle

Korff M.
Deltaires and Cambridge University

Jong de E.
Geobest

Bles T.J.
Deltaires

ABSTRACT: The paper analyses the strengths, weaknesses, opportunities and threats (SWOT) for the application of the Observational Method in civil engineering practice. International cases, many of which are well known in literature have been analysed along the lines of the SWOT methodology. A specific number of cases has been analysed, having typical Dutch conditions, to determine country specific aspects as well. This paper describes the evaluation of the cases. This results in conditions under which the application of the Observational Method is best suitable and conditions in which it is best to avoid the observational method.

RÉSUMÉ : Cet article présente les résultats d’analyses Forces, Faiblesses, Opportunités, Menaces (FFOM) effectuées pour appliquer la méthode observationnelle au domaine du génie civil. L’analyse FFOM est appliquée à des réalisations internationales, bien connues dans la littérature et pour lesquelles la méthode observationnelle a été mise en œuvre. Un certain nombre de cas est analysé sous les conditions néerlandaises, afin de déterminer les éléments spécifiques pour ce pays. Cet article décrit l’évaluation des cas. Les résultats de cette évaluation sont des situations dans laquelle l’application de la méthode observationnelle est la plus appropriée et des situations dans laquelle il est préférable d’éviter la méthode observationnelle.

KEYWORDS: Observational Method, SWOT, cases.

1 INTRODUCTION

The Observational Method (OM) can produce savings in cost and programme on engineering projects, without compromising safety, and can also benefit the geotechnical community by increasing scientific knowledge. In some countries the use of OM is common practice, see for example Britain with famous papers by Powderham (1994) and Patel et al. (2007) and the CIRIA report 185 (Nicholson et al., 1999) and France with the Irex-RGCU guideline by Allagnat (2005). In many other countries, such as The Netherlands, the method is used in specific cases only and/or more reluctantly. Many papers in literature have described procedures on implementing the OM such as Powderham and Nicholson (1996) and the guidelines mentioned above, but very little attention is usually paid to the conditions in which the OM is most adequate. With use of a SWOT analysis this papers aims to provide such an overview of hurdles and conditions.

This research is performed as part of “Geoinpuls” in the Netherlands; a joint industry programme, with the ambitious goal to half the occurrence of geotechnical failure in Dutch civil engineering projects by 2015. The measures proposed were clustered into five themes by Cools (2011): geo-engineering in contracts, implementing and sharing of existing knowledge and experience, quality of design and construction processes, new knowledge for Geo-Engineering in 2015 and managing expectations. The observational method is seen as a means to obtain robust en cost-effective projects based on measurements in combination with risk-based scenarios. The method provides projects with the possibility to benefit from uncertainties in soil conditions, which results in opportunities.

2 ANALYSIS OF CASE HISTORIES

The paper illustrates the results of a SWOT analysis based on various projects reported in case histories. The focus of this analysis is on the conditions in the projects that make them suitable for the application of the OM. By collecting these aspects, one can check whether for a new project the application of the OM may bring benefits. If this is the case, the authors of this paper wishes to refer to the use of Eurocode 7 and specific guidelines for the correct and optimal procedures. Those procedures are not part of this paper.

Geotechnical monitoring is an essential part of the Observational Method, and if used separately mostly aims to control the construction processes and design assumptions. As part of the OM monitoring is used for design purposes as well. If the monitoring shows that a design can/must be changed with less/more conservative assumptions this is foreseen in the OM. In the SWOT analysis monitoring is also considered, as it is part of the OM. Parts of the SWOT analysis can therefor be used for geotechnical monitoring.

It must be mentioned that for a true SWOT analysis the internal (Strength, Weaknesses) and the external (Opportunities and Threats) must be clearly distinguished. In the case of the application of the OM, this may not be so evident, especially if we consider the soil conditions. In this paper, the soil is considered an internal part of the project. Furthermore, the SWOT analysis focusses on the application of the OM from the start of the project (‘ab initio’) and not as the ‘best way out’, when unwanted events already have appeared.

Strengths (S)

Some project characteristics can be seen as strengths for the application of the OM. If the following characteristics exist, OM could be considered as a serious option.

1. Multiple stages or parts in a project Patel et al. (2007) suggest that for a good application of the OM it is necessary
to have some sort of a variation between parts or stages of the construction. This is essential to make it possible to learn from previous behaviour, which is the essence of the OM. Both projects that are multistage or that are executed according to an incremental construction process are suitable for application of the OM.

Multistage projects for example include a staged excavation or staged application of loads. These provide good possibilities for the OM. Subsequent stages of loading can be based on results measured in previous stages. Examples include the excavation after collapse of the Heathrow terminal described by Hitchcock (2003) and the raising of the embankment of the Betuweroute Cargo Rail on very soft soils as described in the Geotechnet report by Huybrechts (2000). Using the multi-stage construction process, reliability can be controlled by interpreting monitoring of the previous stages and by taking subsequent actions if necessary. Also excavations that progress in depth, for which the struts can be pre-stressed according to the rate of deformation, or when additional struts or soil nails can be installed depending on the deformations, may possess good characteristics for the use of the OM.

Another strength characteristic is present in projects with an incremental construction process. These projects are flexible in the speed with which they progress or consist of several steps. An example may be in NATM tunnelling work, or vibratory installation of (sheet) piles, where the rate of advancement can be controlled based on the monitoring results. Also projects with a long length (in similar soil conditions) for example line infrastructure projects such as roads and rail can provide a good basis for the OM, such as for example described for the Limehouse Link by Glass and Powderham (1994).

2. Short project duration in relation with beneficial short term behaviour of soil. In some cases short term soil behaviour may be a strength, such as when the undrained strength of soils is larger than the drained strength and only short term loading conditions are applicable which have diminished before drainage takes place. Here also the NATM method could be mentioned.

3. Displacements as leading design characteristic. Projects where displacements govern the design are by nature often suitable for the use of the OM. Deformations can usually be monitored accurately and extensively and provide good indication of the mechanisms that have to be controlled. When deformations of adjacent buildings are important, projects can be suitable for the OM, but it must be mentioned that the possible measures and variations might be limited to a specific and tight range of acceptable expected impact, thus giving less space for its application. It can however be considered a strength in the sense that this is the OM if a project relates to existing structures or conditions that are difficult to assess, such as the stability of an existing embankment (Lee, 2012) or old existing structures with unknown response (Chapman and Green, 2004). The application of the OM in those cases might solve otherwise unknown response of the structure. In general projects where epistemic uncertainties, which originate from insufficient knowledge of a property, can be decreased by the use of monitoring might be suitable (Nossan, 2006).

4. Integrated responsibility for both design and construction. Cases where a strong connection exists between design and construction teams and in which good communication between parties is assured, have a strong case for the use of the OM. The OM works well with an alliance contract in which risks (and opportunities) are shared between client and contractor, see section 3 of this paper.

5. Flexible and risk based culture. It can also be considered a strength if the culture of each organization involved is open to some flexibility but also very strict with regard to risk management and monitoring. If staff members are sufficiently experienced and had proper training, preferably related to the use of the OM, this is a main benefit. A management commitment to implementing the OM approach at all levels is also an organizational strength.

6. High ground heterogeneity and uncertainty in failure mechanism. In cases with high uncertainty a ‘standard’ (non OM) design approach forces the designer to make conservative design assumptions, leading to costs that possibly are not necessary and can be avoided. This leads to a potentially high cost differences between a ‘standard’ design and an OM design. It is the advantage of using the OM to justify a set of more favourable assumptions leading to a more cost effective design. This for instance can be the case when a decision needs to be made between a shallow foundation and a piled foundation, as has been experienced by Geolimpuls participants for a LNG terminal with high demands for dissimilar settlements, or in geological heterogeneous areas (for instance close to rivers).

Two combinations of variability are especially suitable for the OM. First, the soil strength or stiffness is not well known or has a large spread, but the load that will be presented is relatively well known (for example in NATM tunnels or deep excavations as described by Kamp (2003) or the railway example by Lee (2012). Secondly, if the opposite is the case and the load is relatively unknown but the soil strength is well known, for example in the case of deep foundations and embankments described by Peck (1969) and many others, the method could also work well. If both are known, or both are unknown, the OM is not suitable and this should be considered a threat.

Weaknesses

Opposite to the benefits are of course also weaknesses for the application of the OM. If any of the following characteristics exist, application of the OM may result in additional challenges or may not be suitable.

1. Too little time between measurements and measures. A major weakness exists if mechanisms involved in the project reveal themself quicker than measures can be implemented. In the case of brittle failure monitoring may not provide previous warning. Brittle failure is a no go for the OM, while late appearance may make application of the OM inefficient since savings of necessary reinforcements can not be decided early enough, such as described by (Korevaar, 2012). Examples are non-ductile failures of structural members such as struts/walling connections in multi-propped basements as described by Patel (2007) or the vertical equilibrium of deep excavations.

2. Measurements that cause failure. In some mechanisms, for example related to the pull out capacity of (micro)piles or anchors, monitoring would require failure of the system, which is not acceptable.

3. Failure mechanism/parameter can not be measured. It can also be problematic if the monitoring system is not able to capture the correct mechanism or relevant parameters. This is often the case as stiffness and strength of soils are only weakly correlated, meaning that deformation measurements do not always indicate a possible failure of the strength of a material.

4. Change of failure mechanism during construction. Other weaknesses could be that during the construction process, the failure mechanisms change, for example if shallow failures become deep failures, primary consolidation becomes creep etc.

5. Costs for changes during construction are higher than profits minus costs for monitoring. The use of OM inevitably requires usually costly continuous measurements that have to be taken, interpreted and analysed during construction. During the design phase to be calculated together with analysis of other cases/experiences in order to know what to expect. These costs needs to be balanced with
expected benefits. Also sometimes measures that might be needed as an outcome of OM are inefficient during construction. This for instance is described by (Schmitt and Schlosser, 2007) for the case of an excavation in Monaco where huge stays bearing on the bottom of the excavation would have caused major consequences for the completion time of the project.

Although it might seem that OM through this weakness is more beneficial in larger projects than in smaller, this not necessarily is the case. For example small embankments lend themselves often for the use of OM.

6. Communication between site and design office. Application of OM requires direct communication between site and design office, being responsible for direct analyses of the measurements. If these different cultures do not find each other easily in a project, this may cause delays in go – no go moments or even proceeding of the work on site without commitment of the design office. However, if communication is planned carefully it can even be considered a strength of OM that is brings design and construction close to each other. Projects where the culture is based on individual profit and loss opposed to mutual benefits, with extremely low bid or difficult market conditions are not suitable for the application of the OM.

Figure 1. Example project with application of OM in Amsterdam, Rokin Station

Opportunities

Opportunities for the use of the OM are present at projects with the following characteristics:

1. Presence of risks with low, but unacceptable a priori probability of exceedance and significant consequences. For the use of OM it is necessary that the full range of possible behaviour is assessed and that it is shown that there is an acceptable probability that the actual behaviour will be within the acceptable limits (Eurocode 7). OM is suitable if the probability is higher than acceptable for a standard design, but is small enough to still have a large chance of successfully completing the project without necessary measures. This also requires the consequences to be large enough to justify the additional costs. Examples can be the impact of vibratory installation of sheet pile nearby a pipeline or possible damage by vibrations to old monumental buildings during driving of piles. The vibrations will most likely be present, but the probability of exceedance might be low enough to use the OM, in order to avoid a priori costly measures in design.

2. Stakeholders. OM lends itself perfectly for good communication with stakeholders involved in the project. For instance a critical attitude of a project’s neighbours can be addressed with a proper explanation of the project risks and the way the project is organized to react pro actively if risks seem to occur. It is shown at the North South Line in Amsterdam during the application of OM in the final excavation of Rokin Station that the stakeholders were reassured by the extensive risk based OM approach. Also the application of OM at the A2 Maastricht proved to be a very good way for communication with the stakeholder (Grote and van Dalen, 2012) The uncertainties related to the strength of the limestone and the subsequent response of the excavation wall, see Figure 2, made application of the OM suitable for a good communication strategy. However, it should also be mentioned here that miscommunication of the use of OM is a threat for the project, since it can easily be interpreted wrongly by stakeholders as a way of window-dressing a risky project.

3. Best way out. Although the authors of this paper think OM should be used ‘ab initio’, OM has proven many times to be a very good opportunity in case unwanted events are (nearly) happening, for instance observed from geotechnical monitoring. Because the original design already is ‘in place’ and can not easily be changed, an OM approach can still save the project.

Threats

Threats for the use of the OM are present at projects with the following characteristics:

1. Quickly changing loads. One of the major and most well known threats is the probability of quick changing loads (causing brittle failure) such as deterioration of soils caused by intrusion of groundwater. Also external loads such as rainfall induced ground water surges or burst water mains as well as the risk for liquefaction all are potential threats for the use of the OM.

2. Unwillingness of authorities. Another type of potential threat may be the willingness for authorities to allow the method, even though according to Eurocode 7, the method is now regulated. Use of OM almost inevitable requires efforts on communication with the authorities in order to explain what OM is, why it is used, and how is ensured that a safe and sound construction will take place. This especially is the case in countries with little experience with OM, such as the Netherlands.

3. Time restrictions. Making an OM design requires more effort in the design phase. If the design capacity is not adapted this may lead to a longer design period. Projects with high planning demands can therefor be impractical for the use of OM, especially if it is expected that OM will not lead to time savings during construction.

4. Calculation methods and tools do not always allow for proper use of OM, in this case related to the necessary inverse modelling. A large amount of data becomes available during construction and needs to be processed. For instance for settlement prediction software, modules exist in which fitting between model parameters and measurements can take place in order to make better forecasts for stages to come. However, for other mechanisms such as deformations of retaining walls or designs using finite element models this is not easily done. Many calculations may need to be performed in advance in order to use OM properly during the construction. This might lead to inefficient use of OM, causing high design costs or even (if mechanisms happen outside the design expectations) the fact that OM can not be used quickly enough during construction.

It can be concluded from all of the above SWOT conditions that the observational method is best suited for projects that are governed by the serviceability limit states. It is applicable, but less suited, for designs governed by the ultimate limit states with ductile behaviour, and it is unsuitable for the ultimate limit states if brittle behaviour takes place.
3 CONTRACTUAL ASPECTS

When discussing the possibilities of the Observational Method in geotechnical engineering, it becomes obvious that contract requirements should facilitate or, to say the least, should not obstruct its use. In The Netherlands projects are awarded based on the so-called UAV (Uniform Administrative Conditions) or the UAV-gc (Uniform Administrative Conditions for integrated contracts). In contracts where the UAV-applies, the client is responsible for the design and the contractor is responsible for the execution of the works (Traditional contract). Since it is virtually impossible to make a design with the Observational Method without expert knowledge of construction methods, there are limitations to the use of the OM in this kind of contracts. Once the contract is awarded, for instance when a contractor is selected based on general conditions and unit prices, it is possible to change the design using the expertise of the contractor.

If the UAV-gc applies the contractor is responsible for both the design and the executions of the works (Design and Construct contract). All though the possibilities for the use of the OM as a design method are significantly greater than compared to the UAV-type of contract, there are still a number of challenges to overcome. One of the main challenges is that in order to get the contract awarded, the contractor first has to be selected. Since the only award criterion that is deemed truly objectively is price, a problem arises in selecting the best offer for the works. The cost price resulting from a design based on OM will vary around the cost price of the most probable way of execution of the works. By nature of the method, it is impossible to submit such a price in a bid. In the Netherlands it was concluded that in order to use the OM as a design method from the start, it is strongly advised and beneficial to execute the project in an alliance between client and contractor.

In this kind of contract client and contractor share a common objective, for example the execution of the project in a safe and cost-effective manner with a minimised risk for the surroundings. All the unknowns in a project that is designed using the OM can be a shared responsibility. Both client and contractor will be fully involved in all decision making and will have an equal part in any additional costs or benefits. Part of the Betuwelijn Cargo Rail Line (Huybrechts, 2000) has been successfully constructed in this way. The challenge of selecting one of the most qualified contractor remains. One of the suggestions to overcome this challenge is to have a “beauty contest” and a known budget price for the total works. In this way the client is able to select a contractor based on value (best value procurement). Contractors are asked to present themselves not only with reference to the projects they carried out in the past (track record), but also with respect to the proposed method of cooperation with the client. The staff that the contractor wants to deploy for the project will be judged not only on their technical know-how, but also on their “soft skills”, since cooperation is the key-word in an alliance type of contract.

Over the last years several projects in the Netherlands have been awarded in this manner. For the OM to be used within such contracts, all other requirements for the successful use of the OM should also be fulfilled. However, an alliance type of contract comes close to the ideal contract framework that was described as being “utopia” in CIRIA Report 185 (Nicholson et al., 1999).

4 PROJECT ASPECTS

Some project examples are given in this section with their relative appropriateness to the use of the OM. It must be mentioned that each project should be considered in their specific settings, both physically and organizationally. In the examples, only the most common aspects have been considered. For deep excavations the use of the OM is usually limited to the focus on the settlements in the surrounding structures or soil. In some cases, struts can be optimized but it may not always be possible to decide in time whether a strut layer actually can be omitted. If long cut and cover lengths are present, the subsequent sections may learn from earlier sections. Chapman describes several cases where the use of the OM was successful; whereas Karlsrud and Andresen (2008) state that the OM is not particularly suitable for deep excavations. It can be dangerous if sudden increases in water pressures may happen, accidents such as strut failure or unforeseen loads next to excavation happen. It is rather difficult to apply the OM to assure the vertical equilibrium of deep excavations, although this was actually done in Rokin station for the Amsterdam North South Line, as best way out, see Figure 1. Usually this aspect is considered as a potentially brittle behaviour, but in this specific case the behaviour was expected to be more ductile since the water carrying sand layer causing the possible uplift was very thin.

For deep foundations the method is usually difficult to apply because strength at failure often governs the design. There are however good examples that for the re-use of existing piles (Huybrechts, 2000) the OM shows some good possibilities. For TBM tunnelling the OM is often used to control the settlements, and for example not to design the tunnel lining, where standardization is always more efficient than optimization over shorter lengths. For NATM tunnelling the method is often mentioned and it should be possible if a safe base design is present. (Muir Wood, 1990) and (Kovari and Lunardi 2000) state however that the OM for NATM is actually not working in a correct way.

Embankments are usually well suited for the application of the OM. Examples are mainly related to settlement control and staged construction, but also include the control of stability (Lee, 2012). In a similar way especially suited for the OM seem projects where a surcharge is placed, a tank is filled or similar loading of soil with storage takes place. The flexible use of (pre)loading has proved very efficient in many cases.

Other types of projects suitable for the application of the OM are pipelines when deformation limits are very strict, because the allowable values are difficult to assess in design. Environmental projects (contaminated sites) have been presented by Morgenstern (1994) and drainage works by (Roberts and Preen, 1994).

Very simple structures (‘in the backyard’) are usually not suitable for the OM, because the costs of the additional monitoring are often larger than the benefits for the project.

In all types of projects where buildings are present at short distance, the method may be beneficial because they can be strictly monitored. On the other hand, much more flexibility in the system is present if no such buildings/structures are present. Usually in dealing with stringent deformation limits it is necessary to have a more robust design, which reduces the effectiveness of the use of OM.

5 CONCLUSIONS

Conclusions in this paper are given in the form of Go/No Go items for clients and project initiators, as well as designers in a very early stage of the project, to determine whether or not the OM could be a wise approach in their specific project, given the specific circumstances. These Go/No Go items are listed by importance, based on the opinion of the authors. Some issues form the Go/No Go list may be given facts for a project, some may be project choices that may benefit (or contradict) the use of the OM. Some items should be taken merely as reminders of how to organize the project most efficiently. These items are labelled in the third category ‘To overcome’.

Go:
- Multistage projects and/or projects with an incremental construction process.
- Presence of risks with low, but unacceptable a priori probability of exceedance and significant consequences.
- Integrated responsibility for both design and construction.
- High ground heterogeneity and/or uncertainty in failure mechanism.
- Displacements as leading design characteristic.
- Short project duration in relation with beneficial short term behaviour of soil.
- Flexible and risk based culture.
- Critical attitude of stakeholders related to the project.
- Best way out.

No go:
- Too little time between measurements and measures.
- Quickly changing loads.
- Change of failure mechanism during construction.
- Measurements only useful after failure.
- Costs for changes during construction are higher than benefits minus costs for monitoring.

To be overcome:
- Communication between site and design office.
- Unwillingness of authorities.
- Time restrictions.
- Calculation methods do not always allow easy use of OM.

7 REFERENCES


6 ACKNOWLEDGEMENTS

This research is performed as part of “Geoimpuls” in the Netherlands; a joint industry programme, with the ambitious goal to halve the occurrence of geotechnical failure in Dutch civil engineering projects by 2015. The authors wish to thank the members of the OM working group for sharing their case histories and experiences.

Figure 2. Application of OM in Maastricht for A2Maastricht tunnel (photo Reen van Beek)
Development of Method for Evaluating and Visualizing 3-dimensional Deformation of Earth Retaining Wall for Excavation

Développement des méthodes d’évaluation et de visualisation de la déformation tridimensionnelle des murs de soutènement dans les excavations

Matsumaru T., Kojima K.
Railway Technical Research Institute

ABSTRACT: Monitoring of deformation of earth retaining wall for excavation is important in order to keep surrounding environment and structures safe during construction. However, there are some problems in monitoring of earth retaining walls. For example, it is difficult for the partial measurement by plum bobs to evaluate the overall behavior of the retaining walls, and the multipoint measurement using multi-element inclinometers tends to be expensive. In this paper, we developed a system to evaluate and visualize retaining wall as three-dimensional curved surface. The validity was confirmed by the simulation of the loading test on the model wall. In order to confirm the effectiveness of the proposed system to actual monitoring, we tried to apply the system to the on-site measurement. Furthermore, we proposed a method to conduct monitoring of retaining walls using this system and simple inclinometers.

RÉSUMÉ : Le contrôle de la déformation des murs de soutènement dans les excavations est important pour assurer la sécurité de l’environnement et des structures lors de la construction. Toutefois, le contrôle des murs de soutènement pose un certain nombre de problèmes. Il est par exemple difficile de procéder à des mesures partielles au fil à plomb pour évaluer le comportement général des murs de soutènement et les mesures multipoint à l’aide d’inclinomètres multiéléments sont plutôt onéreuses. Dans cet article, nous présentons un système d’évaluation et de visualisation des murs de soutènement sous forme d’une surface courbe tridimensionnelle. La validité du système a été confirmée par simulation d’un essai de charpente sur la paroi du mur testé. Afin de vérifier l’efficacité du système proposé dans des conditions de contrôle réelles, nous avons tenté de l’appliquer lors de mesures sur le terrain. Nous proposons également une méthode de conduite du contrôle des murs de soutènement à l’aide de ce système et d’inclinomètres simples.

KEYWORDS: earth retaining wall, 3-dimensional deformation, cubic B-spline function, measurement, incline

1 INTRODUCTION

Monitoring of deformation of earth retaining wall for excavation is important in order to keep surrounding environment and structures safe during construction. However, there are some problems in monitoring of earth retaining walls. For example, it is difficult for the partial measurement by plum bobs to evaluate the overall behavior of the retaining walls, and the multipoint measurement using multi-element inclinometers tends to be expensive.

Considering these problems as backgrounds, we developed a system to evaluate and visualize retaining wall as three-dimensional curved surface. In this system, the cubic B-spline function is adopted as analytical technique, which is employed for describing shape of land as three-dimensional curved surface based on sets of data of the elevation altitude (Nonogaki et. al., 2008). We proposed a method to evaluate inclinometer data as surface without transforming incline into displacement. The validity and the adequacy was confirmed by loading test and field measurement. Furthermore, we reached the way to conduct measurement easily by using the proposed method.

2 EVALUATING AND VISUALIZING DEFORMATION OF RETAINING WALL IN 3-DIMENSIONAL SPACE

2.1 Cubic B-spline function

Figure 1 shows the 3-dimensional coordinate space for describing the deformation of the earth retaining wall. In this figure, x, y, and z axis means the direction of the retaining wall, the depth, and the direction toward which the wall deforms. The earth retaining wall is expressed as smooth and continuous surface by the following equation.

\[ f(x, y) = z \] (1)

In the cubic B-spline function (Nonogaki et. al., 2008), the region for drawing the surface is divided in \( M_x \) and \( M_y \) equally-spaced areas in x and y axis. By setting the \( M_x+7 \) and \( M_y+7 \) equally-spaced nodes, the surface is expressed by the following equation:

\[ f(x, y) = \sum_{j=0}^{M_y} \sum_{i=0}^{M_x} c_{ij} N_i(x) N_j(y) \] (2)

where \( N_i(x) \) and \( N_j(y) \) is the cubic B-spline function, and \( c_{ij} \) is unknown coefficient.

In order to determine the surface, objective function \( Q \) was defined as following equation:

\[ Q(f;\alpha) = J(f) + \alpha R(f) \] (3)

where \( J(f) \) is the functional for evaluating the smoothness of the surface, \( R(f) \) is the function which expresses the sufficiency degree of data, and \( \alpha \) is the parameter balancing for these two functions. The surface is determined by substituting \( c_{ij} \) into equation (2) obtained from \( \beta Q(f;\alpha) = 0 \). \( J(f) \) is written by Shiono et al. (2001).

The function which expresses the sufficiency, \( R(f) \), is mentioned as below. The coordinate \( (x_p\, y_p\, z_p) \) where a measurement equipment is placed, and the measured displacement \( u_p \) has following relationship.

\[ f(x_p, y_p) = z_p + u_p \] (4)

Therefore, using the error average \( e_p \) of squares between the curved surface and the obtained displacement data, \( R(f) \) is evaluated as following equation.

\[ R(f) = \sum e_p^2 / n_h \] (5)
\[ R_p = \sum_{j=1}^{N_{xy}} \sum_{i=1}^{M_i^{(x)}} c_i N_i(x_i) N_j(y_j) - (z_j + u_j) \]  \hspace{1cm} (6)

where \( n_j \) is the number which satisfies equation (4).

**Figure 1.** 3-dimensional coordinate space for drawing deformation of retaining wall

### 2.2 Use of measured inclination

In the monitoring of the retaining wall, we often measure not the displacement but the inclination because of its easiness. For this reason, it is important for developing the method to take incline data for evaluating the deformation. As follows, we show the proposed method for using the incline data.

The function \( R(f) \) is divided into two functions, \( R_0(f) \) and \( R_d(f) \). \( R_0(f) \) expresses the sufficiency degree of displacements, and \( R_d(f) \) expresses that of inclines. Using these functions, \( R(f) \) is expressed as follow equation:

\[ R(f) = R_0(f) + \gamma R_d(f) \]  \hspace{1cm} (7)

where \( \gamma \) describes the weight of the sufficiency of inclines. \( R_d(f) \) is expressed by equation (5).

On the other hand, \( R_d(f) \) is defined as follows. At the position where an inclinometer located, \((x_q, y_q, \theta_q)\), the derivative of the function \( f \) is described by the following equation.

\[ f(x, y) = -\tan \theta_q \]  \hspace{1cm} (8)

Therefore, the functional \( R_d(f) \) is expressed as following equation:

\[ R_d(f) = \sum_{i=1}^{N_{xy}} \left\{ \sum_{j=1}^{M_i^{(x)}} c_i N_i(x_i) N_j(y_j) + \tan \theta_q \right\}^2 / n_j \]  \hspace{1cm} (9)

where \( n_j \) is the number of the obtained incline data.

### 3 SIMULATION OF LOADING TEST OF MODEL WALL

#### 3.1 Loading test of model wall

Figure 2 shows the photograph of loading test. The wall was 2m in height, 3m in width and 10mm in thickness. The loading was conducted for several cases, changing boundary conditions and displacement. During the loading, the displacement and the incline of the wall were measured using a lot of measurement equipments. In the following simulation, we used only the data obtained from the survey by total station (T.S.) and inclinometers.

**Figure 2.** Loading test of model wall.

#### 3.2 Conditions of simulation

Figure 3 shows the arrangement of measurement equipments used in the simulations. (a) is the arrangement using all 128 points for the survey by T.S., (b) is using only 35 points, and (c) is using 25 inclinometers. Figure 9 shows the pattern of loading.

**Figure 3.** Arrangement of measuring points used in simulations

#### 3.3 Results of simulation

Figure 4 shows the arrangement of measurement equipments used in this simulation. In CASE1, 80 mm displacement was given at the top center of the wall. In CASE2, 30 mm displacement was given at the right middle part.

**Figure 4.** Loading cases used in simulation

Figure 5 shows the simulated and visualized surface using 128 points for T.S. (arrangement (a) as shown in figure 3) in both loading cases. Figure 6 shows the distributions of displacement at the cross section shown in figure 4, in both loading cases. In figure 5, displacement obtained from the contact-type displacement gauges was also plotted. From these figures, it is seemed to be that the simulation could describe the deformed surface in 3-dimension. Furthermore, the simulated displacement for each case almost coincides with measured results using the cross-section displacement gauges, regardless of arrangements or kind of used measurement equipments.

**Figure 5.** Evaluated and visualized deformations of wall

**Figure 6.** Distributions of displacement at cross section.

From these results, it was revealed that the developed method was suitable for the evaluation and visualization of the deformation of earth retaining wall.
4 ADOPTATION OF PROPOSED METHOD FOR FIELD MEASUREMENT

4.1 Field condition of construction and measurement

Figure 7 shows the field conditions of construction and arrangement of measurement equipment. The excavating work was conducted 39 m times 16 m in area, and 9 m in depth. The surface layer of the ground was a very soft alluvial clay layer about 13 m in thickness, with a small N-value of SPT, followed by a gravel layer. The type of the retaining walls was bracing method. The materials of the walls were steel sheet piles. The excavation consisted of three steps as shown in figure 7.

Monitoring the wall was conducted at the south section in order to keep safe the existing tunnel for cars. Monitoring was implemented by multi-element inclinometers. As shown in figure 7, there were four survey lines and six inclinometers were set on each line. For checking monitoring data, the survey of the displacement of the wall using the total station was also conducted at regular intervals around Line No.1 and Line No.2.

Evaluating and visualizing the deformation of the wall in 3-dimensional space was conducted in the region about 33.4 m in width. Two arrangements of the measurement equipments were considered. CASE1 was the arrangement using only 24 multi-element inclinometers. CASE2 was using not only inclinometers but also the displacement obtained from the survey using T.S.

5 ADOPTATION OF PROPOSED METHOD FOR FIELD MEASUREMENT

5.1 Study of using data only upper ground

For the measurement using convenient inclinometers, it seems to be difficult to set inclinometers under the ground. The inclinometers will be set as the progress of excavating work. So, in the case only using the inclinometers located above the
ground, we studied the difference from the deformation evaluated by using all inclinometers, based on the field data mentioned in the previous section.

The number of used inclinometers in this simulation is 4 at 1st step, 12 at 2nd step and 20 at 3rd step. In this simulation, the data obtained from the survey using T.S. was also considered from 1st step.

Figure 10 shows the distributions of the displacement of the wall. The displacement at Line No.1 was quite similar with the one using all inclinometers, CASE1 in previous chapter. On the other hand, the displacement at Line No.4 was quite different because the direct measurement of displacement by T.S. was not conducted around Line No.4. So, we also simulated the monitoring case using multi-element inclinometers were added at Line No.3. The displacement almost coincided with the results using all inclinometers.

In monitoring of field excavating work, direct measurement of displacement or installation of one line for multi-inclinometers would enable the measurement using convenient inclinometers.

Finding optimal arrangement for conventional inclinometers would be executed by evaluating of degree of accuracy and choosing the most suitable surface for all considerable arrangements. Therefore, we conducted the simulation the accuracy of evaluated surface by iterative calculation.

Monte Carlo approach was adopted for iterative calculation and the number of iteration was 1000. The step of excavating work selected for calculation was 3rd step. For the simulation, the surface was evaluated by 8 inclinometers selected from 20 ones above the ground at random.

Figure 11 showed the obtained histogram. The horizontal axis is the evaluated average difference (Matsumaru et. al., 2011) from the surface simulated by using all inclinometers. It was revealed that the accuracy of evaluated deformation of the wall changed largely depending on the arrangement of inclinometers. However, the minimum of the difference was smaller than 1 mm. This mentioned that the monitoring using small number of measurement equipments had the possibility to maintain the accuracy of measurement depending on the arrangement. By conducting iterative calculation about considerable arrangement, the optimal arrangement would be realize.

6 CONCLUSIONS

The purpose of this paper was system to evaluate and visualize deformation of retaining wall as three-dimensional curved surface. As the results, we achieved the following conclusions:

1. We developed a system adopting the cubic B-spline function as analytical technique and also proposed a method to evaluate inclinometer data as surface without transform inclines into displacements. The validity of the method was confirmed by the simulation of the loading tests of the model wall, because correct surfaces were droved using a small amount of data of displacement or using only inclines.

2. The adequacy of the proposed system was examined by applying this method to measurement of the field site of excavating work. From the beginning to the end of the work, the deformation of the wall was represented satisfactorily as three-dimensional surface. Furthermore, it was revealed that the evaluated deformation of the wall coincided with the surveyed displacement by the total station.

3. In order to realize easy monitoring of retaining walls, we checked the arrangement of inclinometers. By using the inclinometers installed above the excavation bottom, the deformation of the wall could be described almost in the same way as by all inclinometers. Furthermore, we checked the validity of the arrangement with smaller number of inclinometers by Monte Carlo approach. Though the evaluated deformation of the wall using smaller number of inclinometers was varied widely, the accuracy of the optimal arrangement was close to the one using all inclinometers.

7 ACKNOWLEDGEMENTS

The field measurement in this study was supported by Mr. Kiyoshi Kuwabara (East Japan Railway Co. Ltd) and Toshiyasu Hisashima (East Japan Railway Co. Ltd).

REFERENCES


Geotechnical protection of engineering infrastructure objects in large cities under intense anthropogenic impact and long term operation

Sécurité géotechnique d’ouvrages du génie civil sous influence anthropogène intense et exploitation à long terme

Perminov N.A.
St. Peterburg University of Means of Communication, Russia

Zentsov V.N.
"Lengiproinzhproekt", Russia

Perminov A.N.
NIPIC Trasspectstroy, Russia

ABSTRACT: This article describes more than 30-year experience of scientific and technical support, design, construction and reconstruction of water supply and sewage facilities in St. Petersburg, Sochi, etc. It describes the specific defects of long-term operation of large-size pumping stations and deep-laid tunnels that cause risks and dangers of their use. It gives the results of geotechnical and design calculations, modeling of underground and tunnel constructions taking into account risk factors determined by defects that occur during construction and operation, and also taking into account external influences, including dynamic ones. The report gives a comparative analysis of calculated and industrial experiments, provides activity and implementation experience of geotechnical support of long-term operation of engineering infrastructure.

RÉSUMÉ : L'article décrit l’expérience de plus 30 ans d’assistance scientifique et technique, en conception, construction et restauration d’infrastructures de distribution d’eau et d’évacuation des eaux usées à Saint-Pétersbourg, Sotchi, etc. L’article détaille les défauts typiques des stations de pompage de grandes dimensions et des tunnels profonds, exploitées sur le long terme et amenés à des niveaux de risque et de danger au cours de leur exploitation. On donne les résultats des calculs géotechniques et de conception, en simulant le fonctionnement des tunnels profonds, compte tenu des facteurs de risque induits par les défauts apparus aux étapes de la construction et de l’exploitation, ainsi que des influences extérieures, y compris les influences dynamiques. Le rapport présente l’analyse comparative des expériences théoriques et pratiques, et fournit les mesures à mettre en œuvre pour la sécurité géotechnique des ouvrages de génie civil exploités à long terme.

KEYWORDS: monitoring, geotechnical analysis, objects of water disposal, deeply lying constructions, tunnels, geoecological safety.

1. GENERAL INFORMATION ABOUT THE OBJECTS OF DEEP ENGINEERING INFRASTRUCTURE IN LARGE CITIES

With long-term operation and intensive development of engineering infrastructure of megalopolises increase the requirements to the ecology and efficient usage of land resources. During engineering development of underground spaces of such a megalopolis, design of integrated measures for protection of town-planning environment against negative anthropogenic impact is of special actuality. Thereupon there must be introduced special safety requirements for the sewage and water treatment facilities.

Sewage (transportation) of waste waters is done through the city sewerage system and tunnel collectors. In the general drainage system these facilities account for up to 60% in large cities and up to 70% in difficult hydrogeological conditions by construction volumes and costs.

Sewage system objects data for the most typical Russian cities with the population over 1 million people is given in table 1.

Table 1. Length of sewerage networks and tunnel collectors in large cities of Russian Federation.

<table>
<thead>
<tr>
<th>City</th>
<th>Sewerage networks length, km</th>
<th>Tunnel collectors length, km</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moscow</td>
<td>8354</td>
<td>550</td>
</tr>
<tr>
<td>St. Petersburg</td>
<td>8245</td>
<td>290</td>
</tr>
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<td>Volgograd</td>
<td>1054</td>
<td>52</td>
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</tbody>
</table>

By now around 88% of all sewage collectors are made of ferroconcrete, around 7% - of metal (steel, cast iron), around 3% - of bricks, plastic, ceramics. Tunnel sewage collectors diameter is from 1.2 to 5.6 m, they are buried from 3 to 60 m underground. For example, in St. Petersburg all sewage network is divided into three basins that serve three main pumping plants up to 70 m deep and up to 66 m in diameter, with productivity of 1.5 mln m³ of sewage per day. For such conditions the main constructive solution for the tunnels are the ferroconcrete tubings with inner ferroconcrete jackets.

Transportation volumes of waste waters in some sections of the tunnels reach 20 m³ per sec, and in case of decrease of their operational reliability or failure will inevitably lead to a technospheric catastrophe.

“Lengiproinzhproekt” institute together with the St. Petersburg State Transportation University has been providing scientific and engineering maintenance, design, construction and rehabilitation of St. Petersburg sewerage system objects for more than 30 years: more than 70 pumping plants, including those with depth of 45 m, 59 m and 71 m, and with diameters of 47 m, 59 m and 66 m; more than 15 km of tunnel sewage collectors with diameters of 1.85, 2.5 and 3.4 m and with depth of 16 m, 24 and 37 m.

Table 2 shows the most typical defects of long-term operated pumping plants and deep tunnels.
Analysis of the materials of the investigations shows that at the moment 60% of gravity sewage tunnels and 80% of pressure sewage tunnels require repairs and sanitation. Instrumental probing (with geological radar) shows that 70-75% of inner surface of pumping plants wells and sewage tunnels have continuity violation and cracks which require strengthening of construct and renewal of waterproof shell.

Table 2. The list of defects typical for the long-term operated (more than 30-45 years) deep pumping plants and tunnel collectors.

<table>
<thead>
<tr>
<th>Location of the defect</th>
<th>Description and photo of the defect</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sunk wells walls</td>
<td>Up to ~25-30m marks. On some sections of sunk well walls there’s leakage through knots. In the knots area there’s leakage of concrete corrosion. Defects are of repetitive nature.</td>
</tr>
<tr>
<td>Sewage tunnels lining</td>
<td>Tubing lining shows leakage. Underground waters go to the collector through cracks and knots in solid ferroconcrete inside lining. There’s leakage of concrete corrosion and salts.</td>
</tr>
<tr>
<td></td>
<td>More than ~45m marks. On the surface of the wall there’re marks of intense leakage through the cracks. Defects are of mass nature. In the knots area there’s leakage of concrete corrosion. Defects are of mass nature.</td>
</tr>
</tbody>
</table>

2. ANALYSIS OF FACTORS INFLUENCING THE SAFETY OF DEEP ENGINEERING STRUCTURES AND MEASURES FOR THEIR ELIMINATION

2.1 Analysis of monitoring data for the construction of sunk large pumping plants and of the inspection results after long term operation

The slotted soil column for construction of sunk wells for main pumping plants in the conditions of St. Petersburg is characterized as follows: top part is presented by quaternary beddings to the depth of 14.0-25.0 meters (middle-density water-saturated dust sand $E=11$ MPa, $C=0$ MPa, $\varphi=30^\circ$; laminar silt sandy loam $E=4$ MPa, $C=0.01$ MPa, $\varphi=15^\circ$; laminar silt loam, very soft $E=9$ MPa, $C=0.025$ MPa, $\varphi=16^\circ$; semisolid silt loam with gravel, pebbles $E=14$ MPa, $C=0.028$ MPa, $\varphi=28^\circ$), lower part is represented by top of positioned Protérozoic bluestone ($E=19$ MPa, $C=0.04-0.06$ MPa, $\varphi=18-21^\circ$).

Figure 1 shows the monitoring results for the construction of a large sunk well using the method of PSTU (Perminov N.A., Lombas S.V., 2004).
diameter of 66m and a height of 71m (with the number of three-dimensional finite elements equal 50828), falling under its own weight at an angle of 15 ° from a height of 140 cm on the compliant soil (average coefficient of elasticity for multilayer soil is taken K = 16500kN/m3). In the model because of the inclination angle the friction forces on the lateral side of the well were applied in the upper part of the shell on one side and in the bottom part on the opposite side.

The results of numerical modeling have shown (see Fig. 2) that in case of a dynamic blow (if the well is dropped from a height of 140 cm) equivalent von Mises stresses in the construct equal \( \text{Sd}_{\text{in}} = 256\text{MPa} \) at the top of the shell and \( \text{Sd}_{\text{in}} = 1538\text{MPa} \) in the area of the bottom rest, which respectively exceeds the limiting strength of concrete class B30 [Spred] to 14 or more times, and the changes in the geometry of the shell are observed.

Thus, already in the process of the well immersion the construction of the well is damaged and the concrete is disintegrated due to breakdowns. Later during operation micro cracks lead to leakage, seepage and corrosion of concrete. To further ensure the safety of operation of facilities of this type it is necessary to strengthen and waterproof the construct by high-pressure injection of polymer resins.

2.2 Geotechnical analysis of technical condition of the sewage tunnels under intensive anthropogenic impact and long term operation

Geotechnical analysis of the sewage tunnel was carried out for the most typical section located in a zone of intense dynamic impact of transport and the impact of new construction.

Figure 3 shows the diagram of the tunnel compressions for more than 35 years of service life.

Figure 2. The results of numerical modeling of a sunk well with diameter of 66 m and a height of 71 m for the conditions of a abrupt landing (breakdown): a) the original position, and b) position after a fall from a height of 1.4 m at an angle of 15 °.

Figure 3. The diagram of comparison of the compressions on the arch axis of the collector: a) engineering and geological section, typical for laying-out sewage tunnels in St. Petersburg, b) the diagram of compressions: 1 - survey results of 2010, 2 - executive survey data of 1975, 3-area of the collector, protected from the influence of the construction be a screen of low modular material.

Uneven tunnel compressions, modified on the arch axis range from 5 to 276 mm. Comparative analysis of engineering and geological section on the tunnel route and its placement on the plan relative to the traffic junction showed that the greatest compressions up to 276 mm are located in the area of the tunnel under intense dynamic effects of the traffic, passing the layer of thixotropic quaternary deposits.

Evaluation of the dynamic impact of the transport was carried out by the study of the oscillatory process with a set of manifold gauges CM TSP installed in the arch and blocks of the recording equipment (Perminov N.A, 2011)

The frequency of the oscillations of the collector during various traffic loads from 15 to 35 Hz, and the vibration amplitude to 35-70 microns was recorded. According to the research (Goldshtein M.N., Lapidus L.S., Reznikov O.M., Storozenko V.I., Sinaevsky N.I, 1973) for this type of ground deposits and the appropriate level of the dynamic effects the decrease of strength characteristics C and \( \varphi \) is up to 35% and 17%, respectively. To ensure the operational
reliability of tunnels vibration protection measures, such as the use of spiral-wound technology for internal lining the tunnel are suggested.

For this section of the collector the numerical modeling was carried out to determine the maximum allowable axis displacement of tunnel lining. The criterion for the safety of the construct is the maximum allowable tensile stress of the concrete in the typical points of lining. Maximum allowable deformation and displacement values are presented in Figure 4.

a)

b)

Data received from long-term field observations for continuous operated embedded constructions being a part of megapolis water discharge system as well as the results of calculation and modeling allowed to conduct geotechnical analysis of the residual bearing capacity and to develop measures to ensure the safe operation of facilities of such type under the conditions of intensive external influences.

As experience shows, the presence of geotechnical tracking of environmentally hazardous facilities of such type for the entire period of their life cycle, including design, construction and long-term operation (even under intensive man-induced impact) provides the safety of the stable functioning of the megapolis utility infrastructure.

4. REFERENCES


Figure 4. Calculated model of the tunnel (a), the diagram of maximum allowable tunnel lining deformation (b): 1 - Safe displacement values after the lining has been strengthened, 2 - reference value of allowed displacement.
Data assimilation strategies for parameter identification of elasto-plastic geomaterials and its application to geotechnical practice

Stratégie d'assimilation de données pour l'identification des paramètres de géomatériaux élastoplastiques et son applications à la pratique géotechnique

Shuku T., Nishimura S.
Graduate School of Environmental and Life Science, Okayama University, Okayama 700-8530, Japan

Murakami A., Fujisawa K.
Graduate School of Agriculture, Kyoto University, Kyoto 606-8502, Japan

ABSTRACT: The objective of this study is to demonstrate the numerical and the practical applicability of the particle filter (PF) to some geotechnical problems, i.e., the parameter identification of elasto-plastic geomaterials and the prediction of the deformation behavior of soil deposits and geotechnical structures, by applying the methodology to hypothetical experiments and an actual construction project. The results of the hypothetical experiments reveal that the parameters identified by the PF, based on the sequential importance sampling (SIS) algorithm, have converged into their true values, and that the approach presented herein can provide a highly accurate parameter identification strategy for elasto-plastic geomaterials. Moreover, the simulation results using the identified parameters are close to the actual observation data, and the ensemble-based approach produces more information about the parameters of interest than simple estimated values obtained from optimization methods. In other words, the identification comes in the form of a probability density function.

RÉSUMÉ : L'objet de cette étude est de démontrer l'applicabilité numérique et pratique du filtrage des particules (FP) pour certains problèmes géotechniques, à savoir, l'identification des paramètres de géomatériaux élastoplastiques et la prédiction du comportement de déformation des dépôts de sol et des structures géotechniques, en appliquant la méthodologie à des expériences hypothétiques et à des projets de construction existants. Les résultats des expériences à partir d'hypothèses montrent que les paramètres identifiés par le FP, basé sur l'algorithme d'échantillonnage d'importance séquentiel (SIS), ont convergé vers leurs valeurs réelles, et que l'approche présentée ici peut fournir une stratégie d'identification paramétrique très précise pour les géomatériaux élastoplastiques. En outre, les résultats de la simulation utilisant les paramètres identifiés sont proches des données d'observation réelles, et l'approche groupée produit plus d'informations sur les paramètres d'intérêt que de simples valeurs estimées obtenues à partir des méthodes d'optimisation. En d'autres termes, l'identification se présente sous la forme d'une fonction de densité de probabilité.

KEYWORDS: data assimilation, particle filter, parameter identification

1 INTRODUCTION

Inverse analyses have been successfully applied to linear elastic problems in which the deformation to be addressed is linear and depends only on the model parameters and the applied load; it does not depend on the loading history. However, the mechanical behavior of geomaterials is commonly described by an elasto-plastic model, and the deformation behavior displays strong nonlinearity and depends not only on the values of the parameters, but also to a great extent on the stress state and the history, whereby the identification of elasto-plastic parameters still remains a major challenge.

Data assimilation (DA) is available as a methodology to tackle the above difficulties (Nakamura et al. 2005). The estimation of the interest dynamic system via DA involves a combination of observation data and the underlying dynamical principles governing the system. The melding of data and dynamics is a powerful methodology, which makes efficient and realistic estimations possible. This approach has recently proven fruitful in earth science, e.g., geophysics, meteorology, and oceanography (e.g., Awaji et al. 2009).

Several kinds of powerful DA methods have been proposed. Among the existing strategies, this study focuses on the filtering techniques referred to as the particle filter (PF, Gordon et al. 1993), because it can be applied to nonlinear and non-Gaussian problems and can provide a simple conceptual formulation and ease of implementation.

Herein numerical and practical effectiveness of the DA strategies using the PF are examined for geotechnical problems through their applications to the numerical experiments and an actual construction project. For this purpose, first, we outline the concepts and methods of DA and refer to the PF. Second, we deal with the parameter identification of elasto-plastic parameters for geomaterials applying the PF to initial and boundary value problems in geomechanics. Finally, we investigate the applicability of the PF to a practical settlement prediction of a well-documented construction project, Kobe Airport Island, comparing the obtained simulation with the observation data, and the practical effectiveness of the DA based on the PF is discussed.

2 DA: CONCEPTS AND METHODS

DA is a versatile methodology for estimating the state of a dynamic system of interest by merging sparse observation data into a numerical model for the system. The state of the system is usually estimated with deterministic simulation models, which are subject to the uncertainty that arises due to a lack of knowledge and a poor understanding of the physical phenomena. Meanwhile, observation data, which represent the true state, but are subject to stochastic uncertainty and randomness, may occasionally be available as a function of a subset of the system variables. Based upon a prognostic model and a limited number of observations, DA attempts to provide a more comprehensive system analysis which may lead to more accurate predictions. This approach has recently proven useful in earth science (Awaji et al. 2009).

Novel sequential data assimilation methods include the Ensemble Kalman Filter (EnKF, Evensen 1994) and the PF which are categorized into nonlinear Kalman filtering. Although the EnKF can be applied to nonlinear systems, it basically assumes a linear relationship between a state and the observation data in calculating a Kalman gain. Therefore, the
EnKF cannot produce satisfactory estimates if its linear approximation is invalid. This means that its application to geomechanics is difficult, because the materials display strong nonlinearity. On the other hand, as the PF does not require assumptions of linearity or Gaussianity, it is applicable to general problems. Therefore, the PF has higher potential for application to geotechnical engineering and can obtain meaningful outcomes. Brief description of the PF is summarized below.

The PF approximates probability density functions (PDFs) via a set of realizations called an ensemble that has weights, and each realization is referred to as a ‘particle’ or a ‘sample’. For example, a filtered distribution at time $t-1$, $p(x_{t-1} | y_{1:t-1})$, where $y_{1:t-1}$ denotes $\{y_1, y_2, ..., y_{t-1}\}$, is approximated with ensemble $\{x_1^{(1)}, x_2^{(1)}, ..., x_N^{(1)}\}$ and weights $\{w_1^{(1)}, w_2^{(1)}, ..., w_N^{(1)}\}$ by the following equation:

$$p(x_{t-1} | y_{1:t-1}) = \frac{1}{N} \sum_{i=1}^{N} w_i^{(t-1)} \delta(x_{t-1} - x_i^{(t-1)})$$  \hspace{1cm} (1)$$

where $N$ is the number of particles and $\delta$ is the Dirac delta function. $w_i^{(t-1)}$ is the weight attached to particles $x_i^{(t-1)}$ and should suffice $w_i^{(t-1)} \geq 1$ and $\sum w_i^{(t-1)} = 1$.

A general approach for filtering is known as sequential importance sampling (SIS) (Doucet et al. 2000). The SIS algorithm is based on using the importance sampling to estimate the expectations of functions of the state variables. The algorithm of SIS is summarized as follows:

1. **Initialization:**
   Generate an ensemble (set of particles) $\{x_0^{(1)}, x_0^{(2)}, ..., x_0^{(N)}\}$ from the initial distribution $p(x_0)$.

2. **Prediction:**
   Each particle $x_{i-1}^{(t)}$ evolves according to the numerical dynamic model given by a numerical simulation method such as FEM.

3. **Filtering:**
   After obtaining measurement data $y$, calculate weight $w_i^{(t)}$, which expresses the “fitness” of the prior particles to the observation data, and assign a weight, $w_i^{(t)}$, to each $x_i^{(t)}$.

4. **Weight update:**
   The set of weighted particles $\{x_i^{(t)}\}$ results in an ensemble approximation of filtered distribution $p(x_{t} | y_{1:t})$.

   Set $t = t + 1$ and go back to Step 2.

Figure 1 shows the algorithm of the PF based on the SIS.

### 3 PARAMETER IDENTIFICATION OF CAM-CLAY MODEL USING THE PF

This chapter focuses on the soil-water coupled behavior of a clay foundation under monotonic loading, where the numerical simulation for hypothetical soil deposit under embankment is implemented to study the efficiency of the PF as a parameter identification method.

The soil-water coupled finite element analysis using the Cam-clay model were used in this example. The finite element mesh and the loading history are shown in Figures 2 and 3, respectively. Table 1 lists the parameters of the clay foundation. The placement of the observation points is also shown in Figure 2; the vertical displacements and the horizontal displacements are located at S1-S3 and at L1-L3, respectively. Some of the parameters are chosen to be identified and their values are called ‘true values’ as listed in Table 2, and we carried out 100 Monte Carlo Simulations using the sets of particles which were generated with uniform random numbers in the range shown in Table 2.

Figure 4 shows the time evolution of the identified parameters ($\lambda$, $\kappa$, and $\mu$). Identified parameters are computed as the weighted mean value of the particles computed by

$$\bar{\phi}_i = \sum_{i=1}^{N} w_i^{(1)} \phi_i^{(1)}$$ \hspace{1cm} (2)$$

<table>
<thead>
<tr>
<th>Parameter</th>
<th>True value</th>
<th>Range</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\lambda$</td>
<td>0.239</td>
<td>0.090 – 0.290</td>
</tr>
<tr>
<td>$\kappa$</td>
<td>0.091</td>
<td>0.015 – 0.115</td>
</tr>
<tr>
<td>$\mu$</td>
<td>1.084</td>
<td>0.854 – 1.454</td>
</tr>
</tbody>
</table>

Table 1. Cam-clay parameters of the model foundation.

Table 2. True values of the parameters to be identified and range of particle generation.
where \( \theta_i \) and \( \theta^{(i)} \) indicate the identified parameter at time step \( t \) and the parameter of particle number \( i \) at time step \( t_i \), respectively.

The parameter identification of unknown parameters approaches the true values, although the identification starts with an incorrect \( \theta_i \) in all cases purposefully. These results verify the effectiveness of the PF for the parameter identification of the elasto-plastic model, which presents strong nonlinear behavior.

4 APPLICATION OF THE PF TO SETTLEMENT BEHAVIOR OF KOBE AIRPORT ISLAND CONSTRUCTED ON RECLAIMED LAND

The objective of this chapter is to investigate the applicability of the PF to an actual settlement prediction of a well-documented geotechnical construction project, Kobe Airport Island. To accomplish this objective, firstly, the settlements of the island are evaluated using a soil-water coupled finite element analysis with the Cam-clay model. Then, the parameters are identified using the PF. Finally, comparing the recomputed settlement using identified parameters with the observation data, the practical effectiveness of the methodology based on the PF is discussed. Some outcomes obtained from this application example were reported in Murakami et al. (2012).

Kobe Airport was constructed on an artificially reclaimed island just off the coast of Kobe. Figure 5 shows the cross section of the construction site. Vertical sand drains were installed in the soft clay layer in order to accelerate the settlement and increase the strength (e.g. Yamamoto et al. 2010).

![Figure 5. Cross section of the construction site (Yamamoto et al. 2010).](image)

The soil-water coupled finite element analysis with the Cam-clay model was adopted for analyzing the deformation behavior of the seawall and the foundation subjected to the construction and reclamation work. Figure 6 shows the finite element mesh. In the model ground, the top surface, bottom surface and the sides of sand/gravel layers were assumed to have permeable boundary conditions, whereas the sides of clay layers were assumed to have impermeable boundary conditions. The sand layers and reclaimed ground were assumed to be linear elastic, and the clay foundations were represented by the Cam-clay model.

The mass permeability concept, which was proposed by Asaoka et al. (1995), was incorporated into this analysis. Mass permeability is the permeability representative of a clay foundation, which includes the effects of inhomogeneity, partial drainage, and load intensity. We also adopted the concept in the same sense. The analysis in this chapter focuses on the settlement behavior of only the improved alluvial clay foundation, because the soil layers which are just below the improved ground, called Ds1-Ds3, are thick, have high rigidity (the N-value obtained from SPT is more than 100), and do not significantly affect the settlement of the island.

Firstly, we considered the improved ground to be homogeneous by incorporating the mass permeability concept. Then, using the PF, some parameters of the treated ground, the so-called mass parameter were identified to simulate settlement of the ground under the airport island. Although the some parameters affect settlement of the ground, the compression index \( \lambda \) and the permeability \( k \) were treated as the only parameters to be identified, because these two parameters directly govern consolidation behavior of clay grounds. Finally, the simulations were implemented using the identified mass parameters and observation data were compared to evaluate the practical usability of the PF.

The representative parameters of the improved grounds, referred to as mass parameter \( P_{coeff} \) in this study, are determined here by using equation (3) for simplicity.

\[
P_{coeff} = \frac{P_i \cdot h_i}{h_i + h_i + \cdots + h_n} \quad (i = 1, 2, \cdots, n)
\]

where \( P_i \), \( h_i \), and \( n \) are the parameters, the thickness of each layer, and the number of soil layers, respectively.

We conducted Monte Carlo simulations with 200 particles over the feasible space listed as follows:

\[
0.30 \leq \lambda \leq 0.60, \quad 1 \times 10^{-6} \leq k \leq 1 \times 10^{-3}.
\]
Each parameter was assumed to follow uniform randomly and was generated independently. All 200 simulations were conducted up to 676 days after the construction was started. Only the settlement values observed on the seabed were used for parameter identification.

Figure 7 shows the time evolution of the identified parameters. In the figure, the estimates for $\lambda$ hardly change through the assimilation. In particular, after the 300th day, the path changes dramatically. On the other hand, in the result of $k$, the identified parameter shows almost constant value through the assimilation.

Figure 8 shows filtered PDFs of a settlement value at the 148 days after construction began. In this figure, the vertical axis represents the weight of the particle, while the horizontal axis represents settlement value. It can be seen from the Figure 8 that the distribution of the weight approximately follows the normal distribution which has sharp peak around -3.5m. From the result, we can see that the use of a large number of particles contributes to the accurate estimation of the arbitrary PDFs for settlements. This is the remarkable advantage of the PF.

The simulation results for the time-settlement relationship at observation points 3BC-2 and 3BC-4, which were placed on seabed, via the identified parameters are shown in Figure 9. The identified parameters mean the values at the end of the identification process, that is, $t = 456$ days. In the figures, dotted line represents the result of direct analysis. Although the results of direct analysis underestimate the observation data, the simulations using the identified parameters yielded predictions with high accuracy.

Figure 7. Time evolution of identified parameters.

Figure 8. Filtered PDF of a settlement value.

Figure 9. Simulation results using the identified parameters.

5 CONCLUSIONS

In this study, we have investigated the numerical and the practical effectiveness of the DA strategies using the PF for geotechnical problems through their applications to the hypothetical experiment and the actual construction project.

The parameters identified by the PF have converged into their true values, and the presented approach has shown effective parameter-identification method for elasto-plastic geomaterials. Moreover, the simulated time-settlement behavior using the identified mass parameters has provided a good agreement with the actual observation.

In conclusion, the DA using the PF has been proven a powerful strategy for identifying elasto-plastic parameters of geomaterials and more accurate predictions of the mechanical behavior of geotechnical structures.

6 REFERENCES


Experimental analyses on detection of potential risk of slope failure by monitoring of shear strain in the shallow section

Analyses expérimentales sur la détection d’un risque potentiel de rupture de pente par la surveillance de la contrainte de cisaillement en pied du talus

Tamate S., Hori T.
National Institute of Occupational Safety and Health, Tokyo, Japan
Mikuni C., Suemasa N.
Tokyo City University, Tokyo, Japan

ABSTRACT: A large scale model test was carried out in this study to investigate the relationship between the potential risk of slope failure and an increase of the shear strain in the shallow section. A model slope made of soft deposit of Kanto-loam with 30 degrees inclination and 3.5m height was prepared. Compact shear strain meters as well as inclinometers and extensometers were installed to measure the movements of slope prior to failure. Seven steps of cuttings were carried out from the toe to make the slope unstable. The model slope did not fail soon after completion of the final cutting, and it lasted 7 minutes before it finally failed. Clear increases in the shear strain $\theta$ had been measured as the cuttings progress. The obtained data of $\theta$ and the displacement showed good agreement in their reactions. Accordingly, it is proven that the potential risk of slope failure was detectable by monitoring of the shear strain in the shallow section for simplicity.

RÉSUMÉ : Un essai sur modèle à grande échelle a été réalisé dans cette étude pour étudier la relation entre le risque potentiel de rupture de pente et une augmentation de la déformation de cisaillement dans la partie peu profonde. Une pente modèle composée de dépôt mousse de Kanto-limoneux a été préparée avec une pente de 30 degrés et une hauteur de 3,5 m. Des capteurs de déformations de cisaillement compacts ont été développés et installés avec des inclinomètres et des extensomètres pour mesurer les mouvements de pente avant la rupture. Sept phases d’excavations ont été réalisées au pied pour rendre la pente instable. Le modèle de la pente ne s’est pas écroulé tout de suite après la coupe finale, et il y a eu un intervalle de 7 minutes avant la rupture. Une nette augmentation de la déformation de cisaillement $\theta$ a été mesurée lors des excavations. Les données de $\theta$ obtenues et le déplacement montrent un bon accord dans leurs comportements. Ainsi, il a été montré que le risque potentiel de rupture d’une pente est détectable par le suivi de la déformation de cisaillement au pied du talus pour la simplicité.

KEYWORDS: slope failure, monitoring, shear strain in shallow section, large scale model test.

1 INTRODUCTION
Slope failures frequently cause occupational accidents at construction sites. It is also known that even a small collapse can cause serious injury to workers. Therefore, slope failures must be avoided for safety reasons, and temporary retaining walls are needed to support slopes at worksite. In addition, the practice of immediate escape is also important to save human lives, and warning must be given prior to failure. Consequently, monitoring of the slopes is needed to detect increase of the potential risk of failure.

This paper will first summarize hazards that exist in the slope works. Next it will explain a large scale model test carried out to simulate the slope failure. It will also introduce a compact shear strain meter developed to measure increase of shear strain in the shallow section of slopes. Finally, its applicability will be discussed in consideration of results from the test.

2 HAZARDS IN WORKS ON SLOPES
Excavations and cuttings are frequently performed in many aspects of slope works. Cuttings at the toe of slopes are common in the building of retaining walls. However, cuttings may cause the slope to be unstable even though its duration is only short term. Itoh et al. (2005) reported that the volume of collapsed soil blocks in almost 60 % of all accidents was less than 50m$^3$. Accordingly, serious damage to the workers are caused by small amount of collapsed soil blocks. In addition, safety must be maintained at the recovering operation after disasters. Collapsed soil deposited is soft and loose after seismic failure, and shear strength of remaining soil slopes was also reduced by seismic acceleration. Therefore, slopes will become unstable after earthquakes. Meanwhile, when quick operations are also required to save refugees, sufficient time for both installation of temporary structures to support the unstable slopes and survey of the ground conditions in detail will not be given.

Hazards exist to people who work on the slopes as well as under the slopes. Moreover, the time for escape may be not given to the adjacent workers. Consequently, warning prior to the failures is important to save workers as well as an installation of temporary supports.

3 A LARGE SCALE MODEL TEST
3.1 Preparation of model ground and method of model test
A model slope of 3.5m height, 4.0m width and 30 degrees inclination was made by filling soil material as shown in Figure 1. Any compaction was not provided to simulate loose deposit of collapsed soil after seismic disasters. Several sheets of tarpaulins are placed to lubricate the friction between the retaining wall and the soil so that the plane strain condition was undertaken.

Soil material used in the test was Kanto loam that has soil properties as shown in Table 1. Figure 2 shows the relationship between the cone penetration resistance $q_c$ and the depth from the top of slope $d$. Values of $q_c$ roughly show a linear increase to $d$ because the self-weight was loaded for consolidation.
back as shown in Figure 1. As for S6 and S7, the cutting angle was performed from S1 to S5 that was 0.5m long from front to extended wires were connected to pegs on the slope.

DTP were set on a beam bridging over the retaining walls while giving 0.9m interval at the center column (CL). Sensor units of the movement of the slopes.

was increased to 70 degrees and 75 degrees, respectively. Thirty minutes interval time between the steps was provided to observe was installed to measure increment of the interpreted shear strain \( \theta \) at the columns of both R05 and R10 in the same manner as the installation in ASG.

### Table 1. Soil properties of Kanto loam.

<table>
<thead>
<tr>
<th>Property</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Density of soil particles, ( \rho ) (g/cm³)</td>
<td>2.799</td>
</tr>
<tr>
<td>Sand (0.075 ~ 2mm) %</td>
<td>62</td>
</tr>
<tr>
<td>Silt (0.005 ~ 0.075mm) %</td>
<td>45.3</td>
</tr>
<tr>
<td>Clay (Diameter&lt;0.005mm) %</td>
<td>48.5</td>
</tr>
<tr>
<td>Liquid limit ( w_L ) (%)</td>
<td>158.3</td>
</tr>
<tr>
<td>Plastic limit ( w_p ) (%)</td>
<td>97.7</td>
</tr>
<tr>
<td>Plasticity index I_p</td>
<td>60.6</td>
</tr>
<tr>
<td>Dry density ( \rho_{dry} ) (g/cm³)</td>
<td>0.665</td>
</tr>
<tr>
<td>Optimum water content ( w_{opt} ) (%)</td>
<td>10.20</td>
</tr>
</tbody>
</table>

Seven steps of cuttings were carried out by construction machinery in the toe of slope. 60 degrees angle in the cuttings was performed from S1 to S5 that was 0.5m long from front to back as shown in Figure 1. As for S6 and S7, the cutting angle was increased to 70 degrees and 75 degrees, respectively. Thirty minutes interval time between the steps was provided to observe the movement of the slopes.

#### 3.2 Installation of sensors

Three kinds of sensors, extensometers (DTP), inclinometers (ASG), and compact shear strain meters (MPS), were installed to measure the movement of the slope during the test. Figure 3 shows the installed positions of the sensors. Two sets of DTP were installed to measure increment of the displacement \( d \) by giving 0.9m interval at the center column (CL). Sensor units of DTP were set on a beam bridging over the retaining walls while extended wires were connected to pegs on the slope.

### 4. Measurement of shear strain in the shallow section

#### 4.1 Development of compact shear strain meter

A compact shear strain meter called MPS was developed to measure the shear strain in the shallow section of slopes (Tamate. 2010). MPS made of compact size rod 0.6m in length, 10mm in diameter, and 3.6N in weight is shown in Figure 4. A screw point of 80mm in length attached in the lower end enables to penetrate the unit into the ground without pre-boring. A taper end of 100mm in length is used to provide a lateral compression to the surrounding soil so that MPS reacts to the slope movement by its bending deformation. A connector terminal for data transmission composed of 7 poles is mounted at the center of a hexagonal plug for rotational installation by a hand drill.

A bending pipe of 420mm in length is positioned between the screw point and the taper end. Four strain gages are pasted on both front and back of the pipe so that the electrical output increases with the bending deformation. A heat shrinkable tube covers the pipe to protect the gages from surrounding soil at installations. Since duration for the installation was less than 10 seconds, easy and quick installation is available in practice as shown in Photo 1.

#### 4.2 Basic idea for the monitoring

Figure 5 shows a schematic image of distribution of displacement and shear strain in the slope. An increment of the displacements occurs near the slip plane whereas the increment converges as the distance increases. Accordingly, major shear strain develops along the slip plane. However, this study aims to monitor the shear strain in the shallow section of slopes from the easily application viewpoint (Tamate et al. 2009). MPS was developed to measure the small shear strain in the shallow sections easily.
A calibration of MPS was carried out to investigate the relationship between the electrical output from MPS $r_s$ ($\mu\varepsilon$) and the interpreted shear strain $\theta$ (%). $\theta$ was defined as the ratio of the settlement $s$ to the effective length $L$ of MPS as shown in Equation 1.

$$\theta(\%) = \frac{s}{L} \times 100$$  \hspace{1cm} (1)

A vertical load $F$ was applied to the end of MPS that was supported by cantilever beam as shown in Figure 6. A clear linear relationship is obtained between $r_s$ and $\theta$ as well as between $F$ and $s$. Since 653 $\mu\varepsilon$ in $r_s$ was output at 1% in $\theta$, a high resolution on the shear strain was confirmed.

5 EXPERIMENTAL ANALYSES ON MOVEMENT IN SLOPE

5.1 Comparison of reactions by sensors

Figure 7 shows the reactions from three kinds of the monitoring sensors. Horizontal axis means the actual time at the test. The first cutting (S1) begun at 13:00 and the final cutting (S7) was completed at 16:20. The slope collapsed twice at 16:27 and 16:36. This meant that 7 minutes remained prior to the first failure and 16 minutes existed up to the second failures after the completion of S7 as shown in Photo 2. An increase of values appeared in each curve as reactions to increase of potential risk of slope failure.

First small increase can be seen at S3 in $\theta$ and its value shows the step increase from S3 to S7. After S7, however, values of $\theta$ kept on increasing from 16:20 to 16:27. Both MPS2 and MPS4 installed at the upper side of the slope show the same increase. Both two curves bent at 16:27 when the first failure occurred. Moreover, values of $\theta$ were built up again at 16:36 of the second failure.
Meanwhile, values of reacted to the second failure so that similar to the 2nd creep that was well known as the plastic deg to the installed in the column of R05 whereas MPS3 and MPS4 were installed in the R10 as shown in Figure 3.

Accelerated these increases from -3 of MPS1 and MPS3 installed inside of the failure block deformation prior to failures. In addition, the value of both soil at the first failure. Upper MPS 2 and MPS4 had recorded a

data until the second failure. Four curves commonly show a linear increase at around -5 of S7 and S6. Same phenomena in the increase can be seen in both sets of DTP and MPS. Meanwhile, curves on angle of inclination did not show clear reactions corresponding to the series of cuttings. A value of ASG1 was kept stable while a value of ASG2 gradual increase from S3. There was no clear reaction on excluding those at the moment of the second failure while the slope was getting unstable.

Figure 8 shows the relationship between and in addition to the relationship between and . shows a clear increase prior to the first failure when hovered between 4 and 7. also reacted to the second failure so that still increased where . Meanwhile, values of increased very little from -0.4 to -0.8 deg to the d though -0.4 deg of the initial drift appeared.

5.2 Increment of the shear strain in the shallow section prior to failure

Figure 9 shows the relationship between and an elapsed time recorded by 4 sets of MPS. Both MPS1 and MPS2 were installed in the column of R05 whereas MPS3 and MPS4 were installed in the R10 as shown in Figure 3. is a modified value that is calculated as zero at the beginning of the first failure. Accordingly, negative values mean the remaining time until the first failure. Recorded data by both lower MPS1 and MPS3 ended at 0 of because these dropped together with collapsed soil at the first failure. Upper MPS 2 and MPS4 had recorded data until the second failure. Four curves commonly show a linear increase at around -5 of , and this phenomena was similar to the 2nd creep that was well known as the plastic deformation prior to failures. In addition, the value of both MPS1 and MPS3 inside of the failure block accelerated these increases from -3 of . Same acceleration on the increase was seen in the values of both MPS2 and MPS4 before the second failure.

Figure 10 shows the entire relationship between the inverse velocity of the shear strain and by a logarithmic scale on a vertical axis. is defined as a value of the increment of per minute. of both MPS1 and MPS3 were distributed at higher values from 1,000 and 30,000, when was between -35 and -10. This means at least that little appeared 10 minutes before the first failure. However, shows the drastic decrease corresponding to the beginning of the final cutting of S7. However, the slope did not fail soon. A couple minutes of time lag existed prior to both failures, and this causes people’s misunderstanding of the stability.

The right side figure shows an expanded view by a linear scale on a vertical axis. indicated the values between 30 and 80 min/% while 4 minutes between -7 and -3 of . Accordingly, the values of were interpreted as between 0.01 and 0.03 %/min. Consequently, the shear strain increased at mostly constant rate in the same manner as the 2nd creep. Moreover, in MPS1 and MPS3 linearly decreased from -2 and -4 of respectively. This proves that values of were accelerating these increases just before failures, the same as the 3rd creep. Accordingly, a clear increase of shear strain in the shallow section of the slope was confirmed in the large scale model test. In addition, it was proven that this phenomenon reflects an increase of potential risk of slope failure. Therefore, a couple of minutes could be provided for escape by identifying either the 2nd creep or the 3rd creep.

6 CONCLUSIONS

A large scale model test was carried out in this study to investigate relationship between the potential risk of slope failure and an increase of the shear strain in the shallow section. Developed compact shear strain meters as well as conventional sensors of inclinometers and extensometers were used in the test to measure the movement of the slope prior to failure. Seven steps of cuttings were performed in the toe to make unstable. The model slope did not fail soon after a completion of the final cutting, and around 7 minutes of the time lag existed until the beginning of failure. Clear increases in the responses of shear strains in the shallow section were measured with the progress of the cuttings. The obtained data of and the displacement showed good agreement in their reactions. Accordingly, it is proven that the potential risk of slope failure was detectable by monitoring of the shear strain in the shallow section for simplicity.

7 ACKNOWLEDGEMENTS

The authors would like to thank Prof. Toshiyuki Kadada, Mr. Nozomu Yamamoto of Tokyo City University, Dr. Kazuya Itoh and Dr. Naotaka Kikawa of the National Institute of Occupational Safety and Health, Japan for their cooperation in the large scale model test and analyses.

8 REFERENCES


Soutènements de grande hauteur soutenus par butons ou multi-ancrages à Monaco : de la modélisation au comportement réel

Retaining wall with struts or multi-anchored for a deep excavation in Monaco: from modeling to real behaviour

Utter N., Dervillé B.  
Soletanche-Bachy, France  
Beth M.  
Soldata, France

RÉSUMÉ : La construction de l’immeuble Teotista comporte la réalisation d’un parking enterré de 6 sous-sols, donc d’une excavation profonde présentant toutes les particularités des projets monégasques, à savoir un environnement exigu, des ouvrages mitoyens sensibles, et la nécessité d’entailler une forte pente d’Éboulis. La hauteur terrassée se trouve ainsi variable de 20 m côté aval à 30 m côté amont, nécessitant la mise en œuvre d’une paroi à contreforts prolongée par des micropieux dans le marno-calcaire rencontré en profondeur, et d’un butonnage fortement dissymétrique sur 5 niveaux, engendrant concentration d’efforts et mise en butée du terrain aval.

La complexité de la structure et du phasage, ainsi que la sensibilité de l’environnement, ont nécessité la mise en œuvre de différents types de modélisations (méthode des coefficients de réaction et méthode des éléments finis), et motivé la mise en œuvre d’une instrumentation qui a permis de confronter à la réalité ces différentes modélisations dans toutes les phases de construction.

Il a ainsi été possible de mettre en évidence la validité des calculs traditionnels dans leur domaine d’application, mais aussi de progresser dans l’appréhension correcte des frontières entre modèles de comportement et méthodes de calcul associées.

MOTS-CLÉS : soutènement, butons, précontrainte, contreforts, méthode observationnelle

KEYWORDS: retaining wall, struts, pre-stress, buttresses, observational method

1 UN PROJET A FORTES CONTRAINTES

1.1 Une emprise limitée

Situé en Principauté de Monaco, le projet de la Tour Teotista comporte l’édification d’un immeuble de 20 étages dans un environnement particulièrement exigu. La surface au sol n’est que de 30 x 30 m et prend la forme d’un talus d’environ 25° dont la cote varie de 132 NGM à l’amont et 118 NGM à l’Aval.

1.2 Un voisinage dense

Le projet est situé dans un contexte urbain dense et bordé : - au Sud par le bâtiment Garden House, immeuble R+7 fondé à 107 NGM.
- au Nord par la Villa Béatrice, habitation R+9 fondée sur semelle à 118 NGM à proximité immédiate du projet.
- à l’Ouest par l’Avenue Hector Otto Supérieure et particulièrement à proximité du Patio Palace, immeuble R+14 de 5 niveaux de sous sols.
- à l’Est par l’Avenue Hector Otto Inférieure donnant sur les Villas du Parc, bâtiment de 10 étages fondé à 108 NGM.

1.3 Un contexte géologique difficile

Les sondages font apparaître une stratigraphie constituée de remblais surmontant une épaisse couche d’éboulis et de colluvions à matrice argileuse. Une frange d’altération d’épaisseur variable surmonte un substratum compact constitué de formations marno-marno-calcaires d’âge crétacé. La résistance élevée des couches profondes (Em = 250 à 500 MPa) et la géométrie étroite du chantier ont nettement orienté la technique de soutènement utilisée. Le projet a consisté en un soutènement mixte composé d’une paroi mouillée prolongée par une fiche en micropieux à partir de 1 m sous le fond de fouille.

L’utilisation d’une hydrofraise était rendue impossible par l’emprise restreinte du chantier, et l’utilisation d’une benne preneuse n’aurait pas permis d’assurer entièrement l’excavation des couches compactes.
1.4 Des tolérances de déplacements strictes associées à une forte contrainte architecturale

Le projet comporte 6 niveaux de sous-sols et la cote du fond de fouille est située à 103 NGM à l’Amont et 100,4 NGM à l’Aval. Dans ce contexte urbanisé très dense et suite aux désordres observés lors de la construction du Patio Palace mitoyen, les critères de déplacements imposés par le contrat sont très sévères : les soutènements ont été dimensionnés pour que le déplacement en tête de paroi amont ne dépasse pas 2 cm, là où pour un soutènement de 30 m, il est généralement admis que les déplacements observés seront de l’ordre du 1/100ème de la hauteur. Le tassement différentiel sous les immeubles avoisinants est limité à 0,8 pour mille.

En outre une contrainte architecturale forte qui impose dans la toute dernière phase des travaux, planchers de la tour construits, de détruire une partie des contreforts du soutènement amont afin d’obtenir une paroi en console sur 8 m de haut. La contrainte principale visait donc à réduire au maximum les déplacements de la paroi pendant les phases provisoires d’excavation pour garantir ce critère lors de la destruction des contreforts.

2 UN PHASAGE DE CONSTRUCTION COMPLEXE

Pour répondre à ces contraintes, il a fallu maîtriser l’espace réduit du chantier, gérer l’ensemble des techniques de travaux mis en jeu et combiner ainsi les multiples plateformes associées à chaque soutènement.

2.1 Soutènement Amont-Aval

La différence de hauteur entre le terrain naturel à l’amont et à l’aval induit une dissymétrie de poussée qu’il est nécessaire de maîtriser pour éviter tout basculement de l’ouvrage. La mise en place de tirants longs ancrés dans le substratum étant interdite à l’amont, l’ensemble des efforts transite de l’amont vers l’aval par un système complexe de dalles, contreforts, butons et planchers.

En phase travaux, la paroi moulée amont de 0,82 m d’épaisseur s’appuie sur 9 niveaux de bandes de planchers réalisées en descendant. D’environ 3 m de large, elles permettent de transférer et concentrer les efforts au niveau des 4 contreforts de 3,75 x 0,50 m espacés tous les 12 m. Cinq lits de butons de gros diamètre, φ1200 mm (see Figure 10) équilibrant jusqu’à 10 000 kN chacun, retransmettent ensuite les efforts des contreforts amont aux contreforts aval. Ces butons sont précontraints afin de garantir le minimum de déplacements. Les contreforts aval de 4,75 x 0,50 m retransmettent ensuite les efforts à la paroi moulée aval de 0,62 m d’épaisseur par l’intermédiaire des bandes de planchers. Elles transfèrent l’intégralité des efforts des contreforts aux parois puis ensuite au terrain en butée.

En phase service, la paroi moulée amont s’appuie sur les planchers et le noyau dur de la tour dont l’inertie est primordiale pour la stabilité globale de l’ouvrage. Les micropieux d’ancrage des parois sont par ailleurs sollicités pour reprendre les éventuels efforts de traction, au séisme notamment.

2.2 Soutènement Villa Béatrice

Le long de la Villa Béatrice, le soutènement exécuté est une paroi moulée de 0,62 m d’épaisseur maintenue en phase provisoire par 6 lits de tirants précontraints et en phase définitive par les planchers de la tour Teotista. La réalisation des tirants précontraints longs engage les tréfonds de la Villa Béatrice sur 15 m de longueur.
2.3 Soutènement Garden House

Les fondations du Garden House se situant à 105 NGM soit environ à 4,60 m seulement du fond de fouille, il a été possible de réaliser une micro berlinoise constituée de tubes 177,8 ép 25 mm disposés tous les 1,30 m.

Figure 5. Elévation le long du Garden House

La partie haute du voile ne reprend aucun effort. Deux butons de principe sont cependant disposés à titre conservatoire.

3 DIMENSIONNEMENT

3.1 Modélisation et méthodes de calcul.

Afin d’estimer au mieux les déplacements du futur ouvrage, les calculs ont été effectués pour chaque soutènement au moyen du programme PARIS basé sur la méthode du coefficient de réaction (Schmitt 2009). Ce logiciel a permis de modéliser et dimensionner l’ensemble des parois, des butons, planchers et moyau central de la Tour. Le programme PLAXIS a permis, quant à lui, d’évaluer les tassements et d’appréhender le comportement global du massif.


4.1 Dispositif de mesures

Le dispositif d’auscultation propre de la paroi comporte 7 inclinomètres et un ensemble de 20 cibles. L’instrumentation des avoisinants comprend 31 prismes, 8 capteurs de vibrations et 16 cibles réparties sur les différentes habitations jouxtant le chantier. Un ensemble de 16 extensomètres sur armatures ont été mis en place dans la paroi moulée amont. Les butons Amont-Aval sont équipés d’un système de précontrainte à perte compensée. La fréquence des mesures est continue pour le suivi des tassements et de transmettre les efforts entre la paroi et le sol.

4.2 Suivi du soutènement Amont-Aval

La courbure des inclinomètres est globalement identique au calcul et les déplacements mis en jeu restent du même ordre de grandeur que ceux calculés. Le comportement réel de l’ouvrage est maîtrisé et aucune action corrective n’a été menée durant l’exécution des travaux.

Le déplacement maximal obtenu au fond de fouille est resté inférieur à 10 mm montrant qu’il est possible par l’application de préconvention de garantir des déplacements exceptionnellement faibles pour une telle hauteur soutenue. Le conducteur de l’ouvrage n’a été mené durant l’exécution des travaux.

La paroi moulée a donc été modélisée en considérant les panneaux comme indépendants et gérant les efforts provenant de l’amont vers l’aval. Tout comme les butons mis en place il a été vérifié que ces soutènements étaient capables de reprendre les efforts mis en jeu. Au droit de la Villa Béatrice, le ferrailage prévu pour les panneaux de paroi moulée a notamment été vérifié en considérant un cas de figure pénalisant où aucun frottement ne serait considéré pour la transmission des efforts entre la paroi et le sol et entre les parois au niveau des joints de panneaux.

La partie haute du voile ne reprend aucun effort. Deux butons de principe sont cependant disposés à titre conservatoire.

4.3 Le rôle des soutènements latéraux

Les soutènements latéraux au droit de la Villa Béatrice et du Garden House ont une double fonction. Ils permettent bien évidemment de soutenir les terres mais aussi de re-transférer les efforts provenant de l’amont vers l’aval. Tout comme les butons mis en place il a été vérifié que ces soutènements étaient capables de reprendre les efforts mis en jeu.

Figure 6. Modélisation PARIS – transmission des efforts Amont-Aval.

La courbure des inclinomètres est globalement identique au calcul et les déplacements mis en jeu restent du même ordre de grandeur que ceux calculés. Le comportement réel de l’ouvrage est maîtrisé et aucune action corrective n’a été menée durant l’exécution des travaux.

Le déplacement maximal obtenu au fond de fouille est resté inférieur à 10 mm montrant qu’il est possible par l’application de préconvention de garantir des déplacements exceptionnellement faibles pour une telle hauteur soutenue. Le conducteur de l’ouvrage n’a été mené durant l’exécution des travaux.

La partie haute du voile ne reprend aucun effort. Deux butons de principe sont cependant disposés à titre conservatoire.
4.3 Soutènement Villa Béatrice

Les mesures inclinométriques montrent que, si les déformées sont du même ordre de grandeur que celles estimées, la courbure est en revanche notablement différente de celle estimée par la méthode au coefficient de réaction, faisant apparaître un « mouvement d’ensemble ». Pourtant, la vérification de la stabilité de chaque massif d’ancrage par la méthode de Kranz fait apparaître des coefficients de sécurité largement satisfaisants (supérieurs à 1,5), voir la Figure 9. L’origine de cette déformée se trouve dans la déformation d’ensemble du massif de sol sollicité par les différents niveaux d’ancrage. La déformation théorique de ce « gabion » d’environ 15 mètres de large par 25 mètres de haut due au cisaillement induit par la poussée des terres, calculée par intégration des formules de Bresse, est en effet tout à fait comparable à celle mesurée (Bustamante et Gouvenot 1978).

4.4 Optimisation du dimensionnement.

Après destruction des contreforts amont, les déplacements en tête de paroi moulée ont été moins prononcés que prévu. Il a donc été possible d’optimiser des places de parking en procédant au découpage des contreforts sur une hauteur supplémentaire de 2 niveaux entre le RDC et le R+2.

5 CONCLUSION

L’auscultation du soutènement et des mitoyens a permis de réaliser un projet complexe en toute sécurité et de livrer au Maître d’Ouvrage un bâtiment conforme à ses spécifications. Le suivi observationnel a montré la remarquable aptitude du programme PARIS multiparois à simuler le comportement fortement interactif des parois amont et aval, et ce sans aucun recalage des paramètres géotechniques utilisés pour le projet: il faut y voir à la fois une validation du modèle de calcul lui-même, et le fruit d’une expérience antérieure du comportement des terrains monégasques et des paramètres de calcul associés. Le suivi géotechnique a par ailleurs permis d’optimiser le soutènement d’une part, et de mettre en évidence d’autre part, la particularité des écarts avec multi-ancrage, dont l’étude ne saurait se limiter à celle de la stabilité des massifs d’ancrage, mais doit intégrer l’effet de la déformation d’ensemble.

6 REFERENCES :

New Sensing Technology and New Applications in Geotechnical Engineering

Nouvelle technologie de détection et nouvelles applications à l’ingénierie géotechnique

Wang Y.H., Ooi G.L., Gao Y.
Department of Civil and Environmental Engineering, The Hong Kong University of Science and Technology, Hong Kong

ABSTRACT: Soils are inherently a particulate medium, and relevant physical principles behind the macro-scale engineering properties originate from particle interactions. However, it is difficult in general to conduct measurements which can monitor soil particle movement and even characterize micromechanics behind different soil behaviour. Advancement of sensing technologies in recent years offers us the opportunity to do so. Two examples are presented in this paper. The first is on using the tactile pressure sensor (film-like sensor) to monitor the evolution of contact normal forces among particles in aged sand. The measurement reveals that the contact forces are continuously redistributed during aging. This ultimately strengthens the soil structure and therefore increases the associated small-strain shear modulus. The second is on using the miniature MEMS accelerometer to characterize the soil movement in a laboratory flow landslide. The MEMS sensors demonstrate promising results in describing the rich features of local responses of soil movement in the shear zone, e.g. liquefaction, deceleration, contraction and dilation.

RÉSUMÉ : Les sols sont intrinsèquement un milieu particulaire et les principes physiques pertinents derrière les propriétés mécaniques à macro-échelle proviennent d’interactions entre particules. Cependant, il est difficile en général d’effectuer des mesures qui peuvent suivre le mouvement des particules du sol et même de caractériser la micromécanique derrière différents comportements du sol. L’avancée des technologies de détection ces dernières années nous offre la possibilité de le faire. Deux exemples sont présentés dans cet article. Le premier utilise le capteur de pression tactile pour suivre l’évolution des forces de contact normales entre des particules dans du sable âgé. La mesure révèle que les forces de contact sont continuellement redistribuées au cours du vieillissement. Cela renforce finalement la structure du sol et augmente donc le module de cisaillement associé à petites déformations. Le second utilise l’accéléromètre miniature MEMS pour caractériser en laboratoire le mouvement du sol dans un glissement de terrain. Les capteurs MEMS démontrent des résultats prometteurs pour la description des caractéristiques abondantes des réponses locales du mouvement du sol dans la zone de cisaillement, par exemple, la liquéfaction, la déceleration, la contraction et la dilatation.

KEYWORDS: tactile pressure sensor; aging mechanism; MEMS • landslide initiation mechanism.

1 INTRODUCTION

Measurements of micromechanical interactions among soil particles are invaluable to constitute insights into the underlying physics of different soil behavior encountered in geotechnical engineering. However, it is difficult in general to conduct measurements which can monitor soil particle movements and even characterize micromechanics involved. In recent years, manufacturing industries witness breakthroughs in both miniaturization of sensors and improvement of sensing technologies. Therefore, we have an opportunity to carry out measurement at the particulate scale. In this paper we would like to introduce two applications in laboratory testing using such new technologies: (1) tactile pressure sensor to characterize the underlying mechanisms of aging in sand, and (2) 3D Micro-Electro-Mechanical-Systems (MEMS) accelerometers to capture soil movement in the initiation process of a laboratory flow landslide.

Aging can occur in all types of soils: for instance, increases in their shear strength and shear modulus are observed as time elapses. Such aging effects in sands have been reported not only from laboratory tests but also via field observations (e.g., see review in Schmertmann 1991, Mitchell and Soga 2005, Wang and Tsui 2009; Gao et al. 2013). Nevertheless, at present, the associated underlying mechanisms remain inconclusive. In the first study, the tactile pressure sensor installed in a tailor-made oedometer was used to characterize the evolution of contact normal force among particles in dry sand during aging. The bender element sets were also utilized in parallel to monitor the associated changes in the small-strain shear modulus, $G_{\text{max}}$. The ultimate goal of this experiment is to provide evidence that could account for the underlying mechanisms of aging effects. Initiation mechanisms of flow slides are likewise still an unsettled open discussion. There had been published studies linking the initiation process of flow slides to pore pressure rise and presence of fines in soil (Iverson et al. 2000, Wang and Sassa 2003); however, exactly how the shear zone develops, liquefies or decelerates due to dilation prior to complete fluidization is not measured. The miniature size of 3D MEMS accelerometers (in the range of mm) makes the measurements of localized soil responses inside slope possible since they can move like a soil particle without any pronouncing inertial effect. Reported MEMS sensor responses shed lights on a variety of soil movement involved in the initiation process of flow landslide, which will be discussed in detail in the second study.

2 USING TACTILE PRESSURE SENSOR FOR THE STUDY OF AGING MECHANISMS

2.1 The I-scan system

Tactile pressure sensors are ultra-thin and flexible and comprise numerous individual tiny sensing elements, called sensels. These features resolve the problems associated with the conventional load cells and enable us to accurately measure stress inside soils. Fig. 1 presents the pressure mapping system used in the first study, i.e., the I-Scan® system (Tekscan Inc., MA., USA). This system consists of software, scanning electronics (called a handle), and a tactile pressure sensor. There are 1936 sensels in the sensor adopted in this study (model 5076). The sensel is a force-sensitive resistor, whose impedance...
changes in response to different loading. When a force is applied to the sensor, the analog-to-digital converter assigns a digital output (DO) value between 0 and 255 (i.e., 8-bit resolution) to each sensel, depending on the corresponding impedance value. This DO can then be correlated to the pressure or other engineering units through calibration.

2.2 Experimental setup and plan
Fig. 2 presents the experimental setup. A tailor-made oedometer with inner dimensions of 100 × 100 × 40 mm was used. Two sets of bender elements were utilized to obtain the small-strain, shear moduli $G_{hv}$ and $G_{hh}$, where the first and second subscripts specify the directions of wave propagation and polarization, respectively; $h$ means the horizontal direction and $v$ stands for the vertical direction. Each set of bender elements consisted of one source and one receiver. The distance $d$ between the source and the receiver is fixed at 80 mm throughout the test. The corresponding shear wave velocity $V_s$ and shear modulus can be derived by $G = \rho V_s^2 = \rho \left(\frac{d}{t}\right)^2$ where $\rho$ is the soil density. The tactile pressure sensor was put between the upper and bottom box to measure the stress distribution inside soils. The calibration of tactile sensors including the creep (or drift) effect followed the procedure suggested by Gao and Wang (2012). The testing material was dry Leighton Buzzard sand (fraction E).

During the aging process, the applied vertical stress onto the sand sample was kept constant at $\sigma_v = 197.21$ kPa for three days. The associated shear modulus changes was continuously monitored using the bender element tests and the evolution of contact normal forces among particles was constantly characterized by the tactile pressure sensor and the I-scan system.

2.3 Experimental results and discussion
Fig. 3 presents the variations in $G_{hv}$ and $G_{hh}$ during the aging process. The variations are presented in terms of the modulus change, i.e., $(G_t - G_{in})/G_{in}$ where $G_t$ and $G_{in}$ are the moduli at any time $t$ and at the initial stage of $t = 10$ min, respectively. As expected, the stiffness continues to increase, suggesting the sample is strengthened during the process of aging. In addition, the increase is greater in $G_{hv}$ than in $G_{hh}$.

Fig. 4 shows the probability distributions of measured, normalized contact forces (in the vertical direction, $F_z$) by the tactile pressure sensor before and after three days of aging under $\sigma_v = 197.21$ kPa. It can be readily seen that the contact forces are redistributed after three days of aging.

In order to further discuss such behavior, the contact forces are categorized into two groups, i.e., strong and weak forces. The categorization of strong and weak forces (i.e., the contact normal forces at strong and weak contacts or at strong and weak
networks) follows the suggestion of Radjai et al. (1996). When the normal contact force \( F_n \) is greater than its mean \( \bar{F}_n \) (i.e., \( F_n > \bar{F}_n \)), it is regarded as a “strong” force; otherwise it is a “weak” force. Also found in Fig. 4 is that the probability distributions of weak forces decrease after aging. That is, the force redistribution leads to increasing contact normal forces in the weak force network such that some of the contact normal forces that originally belong to weak ones can be changed to the group of strong forces. In addition, the force distribution becomes more homogenized after aging because the associated coefficient of variation (CV) in \( F_n \) reduces from 0.726 to 0.705. Since the weak forces become fewer and the contact forces become more homogenized in the sample, the soil structure is strengthened and so is the associated \( G_{hv} \). A comprehensive data set and more detailed explanations with the aid of DEM simulations can be found in Gao (2012) and Gao and Wang (2013).

3 USING THE 3D MEMS ACCELEROMETER FOR THE STUDY OF LANDSLIDE INITIATION PROCESS

Micro-Electro-Mechanical-Systems (MEMS) is a classification of devices, as well as the means of fabrication and manufacturing. In 1959, Richard Feynman took the helm of describing the “problem of manipulating and controlling things on a small scale” and thence pioneers like Analog Devices have since miniaturized conventional sensors from the size of a closed fist to that of a quarter of fingernail (Feynman 1959). The MEMS technology allows for batch-wise etching production, thereby minimizing manufacturing cost and at the same time promising standard accuracy and quality across sensors. Up until the recent 5 years only does stable 3-dimensional MEMS accelerometer become available in the market. The ADXL335 accelerometer model by Analog Devices is selected for the second study to characterize localized soil responses prior to and during landslide initiation.

The Analog Devices’ ADXL335 is a miniature accelerometer which measures \( 4 \times 4 \times 1.45 \, \text{mm} \) (Length \times Width \times Thickness) in size and comes at a low price, about couple US dollars per piece. It utilizes low power, typically functioning at 3.0 \( \mu \text{A} \) and 350 \( \mu \text{A} \). The accelerometer is capable of 3-axis sensing and measures full-scale acceleration within \( \pm 3.6 \, \text{g} \) with a frequency bandwidth ranging from 0.5 to 1600 Hz for the X and Y axes, and a range of 0.5 to 550 Hz for the Z axis. Fig. 5 illustrates the ADXL335 surface mounted on an in-house designed printed circuit board (PCB) since soldering by hand is impossible for the tiny pins; the PCB is \( 11 \times 11 \times 2 \, \text{mm} \) (Length \times Width \times Thickness) in size and the circuitry directs the corresponding pins to larger soldering points. The package was coated with several layers of air-dry polyurethane for waterproof. Also shown in Fig. 5 are the positive directions of X, Y and Z axes. The MEMS accelerometer is attractive not just because of its light-weight, miniature size, low-cost and standardized quality; it also boasts of the unique features of measuring the static acceleration of gravity in tilt-sensing applications, as well as dynamic acceleration resulting from motion, shock or vibration.

These unique features provide us with two kinds of information coming in one package. The DC bias offset in signal conditioned voltage output gives us the tilt angle in reference to the gravity. When the accelerometer is static, we can calculate the current angles of tilt in three dimensions about the accelerometer’s center of mass, known as roll, pitch and yaw; when it is in motion, we can calculate the direction of movement by finding the vector sum of the acceleration. In total 10 MEMS accelerometers were installed in both vertical and horizontal array so that dilative or contractive behavior between layers of soil could also be identified.

![Figure 5. Analog Devices’ ADXL335 surface mounted on PCB.](image5)

3.1 Calibration of 3D MEMS Accelerometer

All the MEMS accelerometers were connected to a logging computer through National Instruments’ NI-USB 6353 analog-to-digital converter with 16-bit resolution. Sampling frequency of every axis was set at 10,000 Hz. Due to batch-wise production, factory performance results of Analog Devices’ ADXL335 are compiled from 1000 pieces to determine the mean bias offset value (Analog Devices 2010); however, to further verify whether the sensors we purchased fall within the range as documented, simple calibration using an earthquake shaking table and a high-frequency vibration exciter was carried out. The mean zero bias offset value for X-axis was found to peak at 1.51 V, and for Y and Z axes the value was 1.49 V; all sensors are functioning as detailed in the datasheet. The sensors were also left operating overnight to check for possible noise drift over time. Nothing anomalous happened and the sensors performed normally as documented in the manual. Subsequent conversions from voltage to acceleration which required parameters from the datasheet were cited directly thereafter.

3.2 Laboratory Water Flume

Fig. 6 presents a side view of the well-instrumented laboratory water flume. The rectangular soil prism made of acrylic is of size \( 100 \times 45.2 \times 20 \, \text{cm} \) (Length \times Width \times Height). A saturation box was affixed at the back to provide standardized antecedent condition before each experiment. In addition to the MEMS accelerometer array (for the positions of accelerometer M1 to M10 see Fig. 6), basal porewater pressure transducers were also installed as indicated by the little squares; a video camera was watching the process from the top. A layer of bottom porous stone was affixed to the flume rack to provide similar friction angle as the soil specimen at the bottom boundary.

![Figure 6. Laboratory water flume setup.](image6)
3.3 Characterization of Shear Zone Behavior with MEMS

In this section, we will demonstrate qualitative results from one of the water flume tests set out to investigate how shear zone liquefaction evolves in loose slope without any fines present. MEMS accelerometer M8 situated right within the shear zone in the rear middle of the soil mass was selected to illustrate the landslide mechanism (see Fig. 6). The loose slope was saturated by slow and little groundwater inflow at start, and a fixed interval later a sudden rise of groundwater inflow twice the original volume was first invoked at the bottom of the slope toe (indicated by T1 in Fig. 6); minutes later the rear bottom (T2 in Fig. 6). Both groundwater supply events are also indicated by vertical black lines in Fig. 7. There are in total 3 sliding events recorded and their occurrence instances are punctuated by the vertical red lines in Fig. 7. The groundwater supply T1 at the slope bottom induced an abrupt liquefaction; inside the shear zone, Y axis of M8 registered a huge contraction and the Z-axis tracked a rotation towards the back then a forward charging jerk. However, the movement of the soil mass ceased as sudden as the initiation; the basal porewater pressure transducer tells us that the porewater pressure built up just then was dissipated. Although by now the groundwater supply T2 at the rear bottom was invoked, the soil mass did not exhibit any noticeable activity, a stark contrast to the loose slope with fines experiment in which complete fluidization occurred by the moment when T2 was invoked. Subsequently what we observed was replenishment of porewater pressure right before sliding event L2 happened and stopped almost instantaneously again; however, this time Y axis of M8 recorded a dilation instead. The porewater pressure abated after L2 then was built up again and finally the whole soil mass fluidized, initiating event L3. The movement was slow; it took 23.9 seconds in total to slide out of the flume. The activity inside shear zone as captured by M8 was a slow dilating rotation, upward and forward as the soil slowly discharged out. In short, the MEMS accelerometers demonstrated promising results in describing the rich features of sliding events, including local responses of soil movement in the shear zone, e.g. contractions, dilations and the rolling components.

4 CONCLUSION

Two new sensing technologies (sensors) were successfully utilized in this paper to reveal aging mechanisms and to monitor local soil movement in a flow landslide event. Using the tactile pressure sensor allows us to measure the contact forces among soil particles and therefore is able to obtain experimental evidence that can explain the underlying mechanisms of aging effects. During aging the contact forces continue to be redistributed. This ultimately leads to increasing contact normal forces in the weak force network such that some of the contact forces in the weak force network such that some of the contact normal forces that originally belong to weak ones can be changed to the group of strong forces. In addition, the force distribution becomes more homogenized. As a result, the soil structure is strengthened and so is the associated small-strain shear modulus. Because of its miniature size and high sensitivity, the MEMS sensor is allowed to be buried in a laboratory slope to characterize the features of soil movement inside the shear zone during a flow landslide event, such as liquefaction, deceleration, contraction and dilation. All of these observations complement theoretical work and provides us insights into the initiation mechanisms of a flow landslide.

5 ACKNOWLEDGEMENTS

This research was supported by the Hong Kong Research Grants Council (GRF 621109 and 620310).

Figure 7. Characterization of sliding features within shear zone in loose slope without fines captured by MEMS accelerometer M8, demonstrating a gradual failure mode.

6 REFERENCES

Feynman, R. 1959. There’s plenty of room at the bottom. URL: http://www.zyvex.com/nanotech/feynman.html
Gao Y. 2012 Experimental characterizations and DEM simulations of aging, creep and structuration in sand. Ph.D. Thesis. The Hong Kong University of Science and Technology.
Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering

CHALLENGES AND INNOVATIONS IN GEOTECHNICS

Actes du 18e Congrès International de Mécanique des sols et de Géotechnique

DÉFIS ET INNOVATIONS EN GÉOTECHNIQUE

The 18th International Conference on Soil Mechanics and Geotechnical Engineering (Paris 2013) was devoted to “Challenges and Innovations in Geotechnics”. The call for abstracts was based on a large series of themes covering most aspects of Geotechnical Engineering and around 800 abstracts were received from the Member Societies. According to the ISSMGE new vision for strengthening the role of the Technical Committees, the papers were distributed to the corresponding Technical Committees, which then selected the General Reporters, the Invited Speakers and the Poster presentations. The involvement of TCs (which could also propose Workshop Sessions on an independent basis) was enthusiastic and successful, which probably explains the success of the Conference with more than 1500 delegates.

The four volumes of the Proceedings contain the Terzaghi Oration, the Honour lectures, the Special lectures followed by the papers presented according to the relevant TC and introduced by the TC General Report. All volumes, together with late papers, will also be made available online free of charge. These volumes will provide a state of the art and serve as an essential reference for practitioners, academics and researchers involved in Soil Mechanics and Geotechnical Engineering.

Le 18e Congrès International de Mécanique des sols et de Géotechnique (Paris 2013) a été dédié aux «Défis et Innovations en Géotechnique». L’appel à résumé était ouvert sur un large éventail de thèmes couvrant la plupart des aspects de la Géotechnique; les Sociétés Membres ont sélectionné environ 800 résumés. En cohérence avec le souhait de la Société Internationale (SIMSG) de renforcer le rôle des Comités Techniques (CTs), les contributions ont été transmises aux CTs correspondants, en charge de choisir les Rapporteurs Généraux, les Orateurs Invités et les Présentations sur Posters. L’implication des CTs, qui pouvaient en outre proposer des sessions d’Atelier à leur convenance, a été enthousiaste et fructueuse ; ceci explique probablement le succès du Congrès avec plus de 1500 délégués.