3rd International Conference on
Transportation Geotechnics

4-7 September 2016 | Guimarães | Portugal

Ground Improvement and Soil Stabilisation

Edited by

S. Varaksin, A.A.S. Correia & M. Azenha

Chairman of 3 ICTG2016

Prof. A. Gomes Correia

Proceedings
Workshop 4

University of Minho
School of Engineering

SIMSG ISSMGE

Ground Improvement and Soil Stabilisation

Workshop of

3rd International Conference on Transportation Geotechnics
4-7 September 2016 | Guimarães | Portugal

EXTENDED ABSTRACTS BOOK

Organized by
University of Minho (UM)
Portuguese Geotechnical Society (SPG)
International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE)

Sponsored by

Ground Improvement and Soil Stabilisation

Organizing Committee:

Serge Varaksin
António Alberto S. Correia
Miguel Azenha

Venue:

University of Minho, School of Engineering
Guimarães, Portugal

Date:

4 September 2016

Website:

http://civil.uminho.pt/3rd-ICTG2016/WorkshopsThemes.php

Contact:

Miguel Azenha
miguel.azenha@civil.uminho.pt

ISBN: 978-972-8692-99-5
Universidade do Minho
Departamento de Engenharia Civil, Azurém, P-4800-058 Guimarães
Email: secretaria@civil.uminho.pt
DOI: 10.5281/zenodo.61225

Preface

The Portuguese Geotechnical Society (SPG), the University of Minho and the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE) organized the international Workshop "Ground improvement and soil stabilization", that took place in the School of Engineering of the University of Minho in the 4th September 2016, with support of the technical committee TC211 'Ground improvement'. This workshop is part of the 3rd International Conference on Transportation Geotechnics (3rd ICTG).

Soil improvement and transportation infrastructures are intimately connected, as the necessities of transportation infrastructures have motivated many advances and innovations in the scope of soil improvement, thus bringing economic feasibility to such projects. The main objective of this Workshop was to gather international experts connected to research and teaching or to the industry that are involved in the several types of improvement and soil stabilization. This brought about interesting opportunities for networking and discussion about ongoing works in the domain of transportation geotechnics. The Workshop was also an opportunity for presentation of the most recent research works, new technological developments and new applications in the scope of soil improvement and stabilization. The topics of analysis include dynamic compaction, vertical drains, chemical stabilization, alkaline activation, non-isothermal modelling, vacuum consolidation, reinforced embankments and load transfer platform.

The Editors

Serge Varaksin
António Alberto S. Correia
Miguel Azenha

Ground Improvement and Soil Stabilisation

Table of contents

Ground Improvement and Soil Stabilisation

Controlled Modulus Columns (CMC) Ground Improvement under the Future Embankment of the New Turcot Interchange

Jérôme Racinais[1*], Adrien Viateau[2] and Hubert Guimont[2]
[1]*Ménard, Nozay, France*
[2]*Geopac, Boucherville, Canada*
jerome.racinais@menard-mail.com, *aviateau@geopac.ca*, *hguimont@geopac.ca*

1 Introduction

Turcot interchange is a hub for road traffic in Montréal area interconnecting highways 15, 20 and 720 in addition of facilitating access to the Champlain Bridge. It is also an essential road link between Montreal-Pierre-Elliott Trudeau international airport and downtown Montréal. It consists of inter-related bridges founded on piles (Figure 1).

After 45 years of service, the Turcot interchange, one of the most important interchange in Québec which has a traffic volume of more than 300 000 vehicles per day, must be rebuilt.

Figure 1: Existing Turcot Interchange

The new interchange is expected to be completed in 2018. It will be built lower to the ground and will be made of 8.0 m average height embankments. Construction of Reinforced Earth Walls will reduce the footprint of the embankments.

The soil investigation campaign has highlighted highly heterogeneous soil stratigraphy composed of 5.0 to 11.0 m of compressible soil overlying a dense fluvio-glacial till. It should also be noted that soil is locally contaminated by heavy metals as a result of former activities of the railway industry in the area. Excavation and replacement of contaminated soil is therefore costly and in some areas not feasible due to close vicinity of existing structures.

Expected total settlement at the highest embankment location (h = 10.0 m) was estimated at 1.5 m; it is composed of 1.0 m primary settlement plus 0.5 m secondary settlement over 35 years. These value are not compatible with the pavement structure requirements. Geopac (Canadian Menard Branch) has submitted and designed a Controlled Modulus Columns (CMC) ground improvement solution to have at maximum 25 mm post-construction settlements over 35 years.

2 Geotechnical conditions

From the top to the bottom, soft soil layers are composed of sandy silt, peat, marl, clay and silty sand. Geotechnical parameters of these layers, including volumetric weight, deformation modulus, compression index, swelling index, creep index, void ratio and permeability are detailed in table 1. The number of blows measured by the Standard Penetration test is also provided and gives good indication on the soil compressibility.

Table 1: Geotechnical parameters of the compressible layers

Layer	N_{SPT}	γ (kN/m^3)	E_Y (MPa)	C_c (-)	C_s (-)	$C_{\alpha e}$ (-)	e_0 (-)	k (m/s)
Sandy silt	1-3	12	8.4					$1.0\ 10^{-7}$
Peat	0	10		6.00	1.20	0.35	9.1	$6.3\ 10^{-7}$
Marl	0	12		1.44	0.33	0.10	4.4	$1.5\ 10^{-7}$
Clay	0-1	17		1.09	0.05	0.05	2.2	$1.0\ 10^{-9}$

3 Ground improvement solution : CMC and steel meshes panels

Controlled Modulus Columns (CMC) are mortar or concrete semi-rigid inclusions used to improve on a global scale the geotechnical characteristics of a compressible soil layer. Depending on each specific project, CMC aim to increase the bearing capacity, to reduce the settlement, to ensure the global stability or to combine some or all of these objectives.

The ground improvement solution designed for the new Turcot interchange consists in unreinforced (a 3m long steel bar is however installed on the top part for freezing purpose) CMC ∅420 mm distributed on a square mesh pattern of 1.6 m x 1.6 m to 1.8 m x 1.8 m depending on the embankment height and soil conditions. CMC are performed with soil-displacement augers, resulting in no soil extraction. In some areas due to existing structures (sheet piles, existing piles), CFA concrete piles have been required. The CMC network is installed in the soft soil down to the till made of sandy silt to silty sand (NSPT = 30). CMC are disconnected from the above embankment by the mean of a Load Transfer Platform (LTP) reinforced by layers of steel mesh panels. LTP thickness varies from 1.1 to 1.3 m.

CMC are designed to support the embankment weight whereas the steel meshes panels withstand the major part of the horizontal force coming from the active earth pressure. Combination of CMC and steel meshes panels allow to increase both vertical and horizontal stiffness's of the foundation soil, solving at once settlement, bearing capacity and stability issues.

Advanced finite element models have been developed to determine the loads distribution between structural elements (CMC, steel meshes) and the soil and to predict the evolution of deformations over the next 35 years.

4 Conclusions

The new Turcot interchange is a major roadway project where CMC becomes the most suitable solution for some sections since they both allow to ensure the technical requirements and can be performed in a really challenging context: low headroom below existing bridges, organic and contaminated soils and underground existing structures (piles and sewer).

Ground improvement works are currently (May 2016) on-going. Isolated load-tests on CMC and a field instrumentation are planned and will give the opportunity to compare predictions with measurements.

References

Combarieu O. (1988). *Amélioration des sols par des inclusions rigides verticales, application à l'édification de remblais sur sols médiocres*

Frank, R. & Zhao S.-R. (1982). *Estimation par les paramètres pressiométriques de l'enfoncement sous charge axiale de pieux forés dans des sols fins*

Racinais, J. & al. (2016). *Twenty Years of CMC Successful Application*

Simon, B. (2012). *Rigid inclusions and stone columns*

Varaksin, S. & al (2016). *Ground Improvement vs. Pile Foundations*

Ground Improvement and Soil Stabilisation

Basal Reinforced Piled Embankments: how to design, how to decide?

Arjan A.M. Venmans[1*] and Suzanne J.M. van Eekelen[1]
[1]*Deltares, Delft, Netherlands*
arjan.venmans@deltares.nl, *suzanne.vaneekelen@deltares.nl*

1 How to decide, how to design?

Basal reinforced piled embankments find increasing application in the construction of road embankments on soft soils. Piled embankments perform better than the traditional alternatives with vertical drainage and surcharging. On the other hand, the cost of basal reinforced piled embankments may be higher than those of the traditional methods. This two-part presentation first considers the decision process on making a choice for a ground improvement method, and then presents current developments in the design of basal reinforced piled embankments.

2 How to decide?

Road owners may apply various decision criteria for selecting what they consider the 'best' construction method. These criteria usually combine technical performance, construction costs, whole life costs, life cycle analysis and uncertainty analysis. This presentation outlines some recent developments in the Netherlands.

The decision support tool 'Road Analysis Module for Bridge Approach Constructions' (in Dutch: WAM overgangsconstructies) is an example of the application of whole life costing. The tool considers ground improvement methods for embankments on soft soil in the approach zone between the bridge and the embankment. The purpose of the decision support tool is to quickly check the technical and financial feasibility of several construction methods. The tool accesses the results of 2300 calculations with MRoad, a decision support model for preliminary design (Venmans et al., 2005). Each calculation involved determining (1) the settlements during construction and operation, (2) how often maintenance is necessary and (3) the whole life costs. The calculations have been carried out for the eight typical Dutch ground profiles in Figure 2.

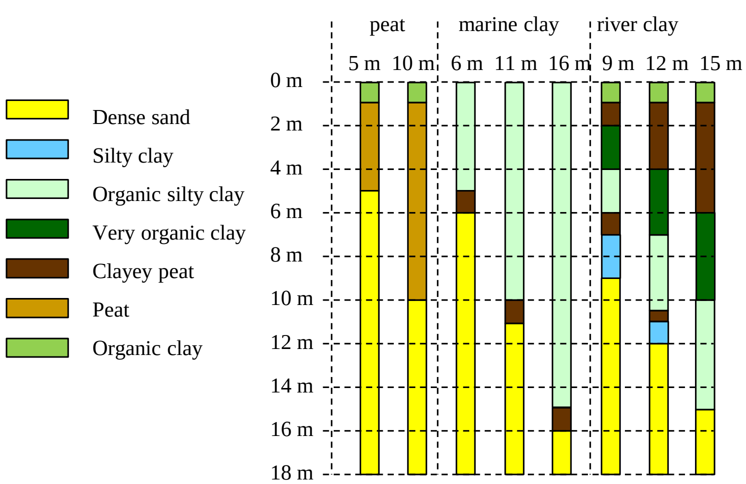


Figure 2: Soil profiles. River clay is more organic and softer than marine clay.

Figure 3 shows the results for the transition zone in a highway with a 7 m high embankment and 6 months construction time. It shows that in this case, a basal reinforced piled embankment is the most economical solution for each soil profile.

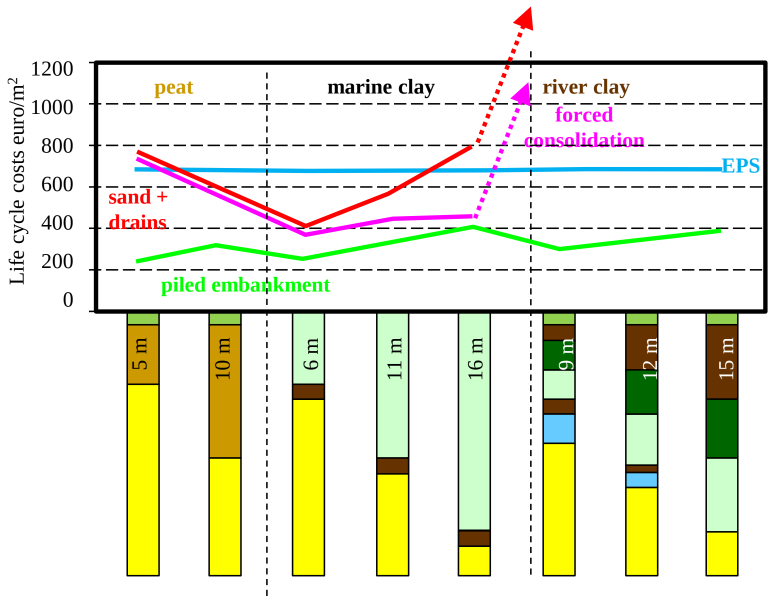


Figure 3: Transition zone, highway, 7 m high embankment, 6 months construction time.

Next, the presentation will briefly show how the sustainability of construction methods can be compared quantitatively using a life cycle analysis, by calculating the Environmental Cost Indicator (ECI) value using shadow prices (DuboCalc, 2015).

Uncertainty in compression and consolidation parameters and subsoil variability may cause considerable delays and budget overruns. Most design methods do not consider these factors, or their impact on costs. This presentation will summarize a methodology for probabilistic quantitative determination of the whole life costs and their reliability interval (Venmans, 2013).

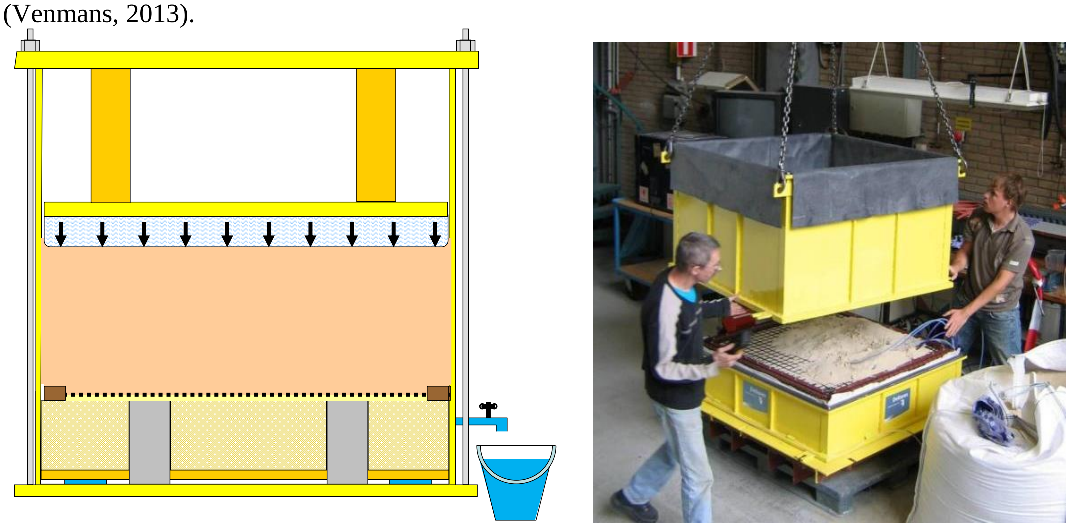


Figure 4: A series of scaled 3D experiments showed the load distribution in a basal reinforced piled embankment.

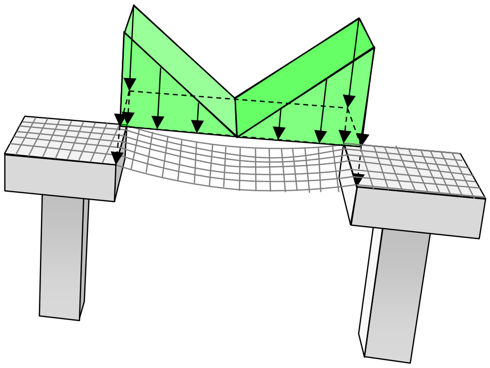


Figure 5: Load is attracted to the GR strips and the net load distribution on GR strips followed from the experiments.

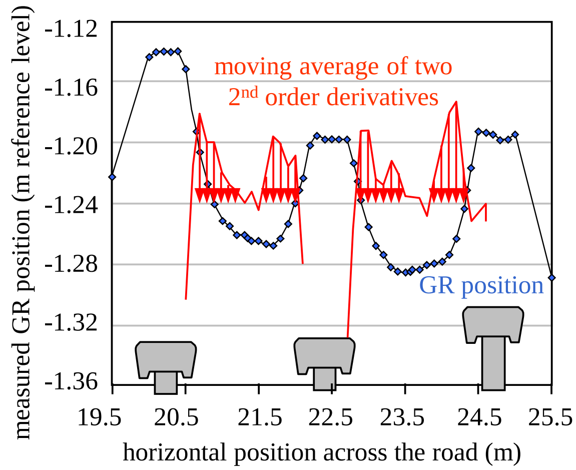


Figure 6: The inverse-triangular net load distribution was confirmed in the field with measurements in settlement tubes attached to the GR. The second derivative of the tube position gave the load distribution.

3 How to design?

Three field studies, a series of nineteen scaled 3D experiments and 3D numerical analysis were conducted on basal reinforced piled embankments. Based on this, the new Concentric Arches was developed, which is a design model for the basal geosynthetic reinforcement (GR) of a piled embankment.

The observed load distribution can be explained by deflection, or vertical deformation. The areas with the least deflection attract most load. Therefore, the piles attract most load. Secondly, the GR close to the pile caps is the GR area that deflects least, because the deflection is limited by the unmoving pile cap. This location therefore attracts more load than the locations further away from the pile cap and so the highest pressures are found alongside the pile cap, with the lowest pressures on the GR strip being found at the central point between the pile caps.

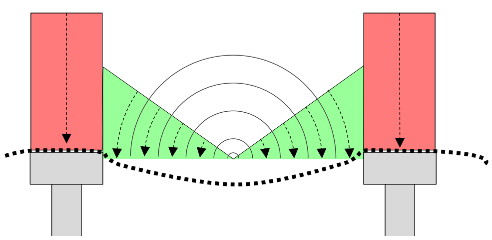


Figure 7: Explanation of the observed load distribution: the more vertical deformation, the less load it attracts. This behaviour can be described with these concentric arches. The larger the arch, the more load it transports.

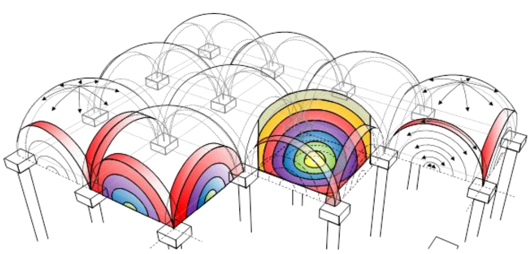


Figure 8: 3D version of the Concentric Arches model of van Eekelen et al, (2013, 2015) and van Eekelen (2015). The load is transported along the 3D hemispheres, outwards, towards the piles and the GR strips. Above the GR strips, the load is forced to follow the 2D strips, towards the piles and subsurface.

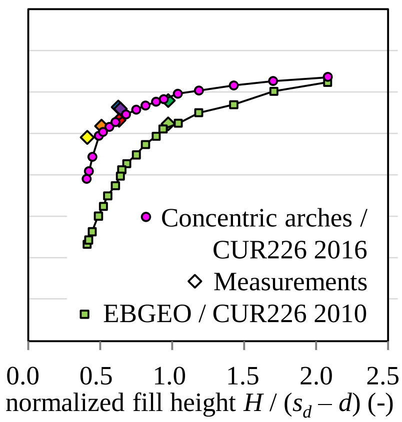


Figure 9: Comparison test results with design guidelines.

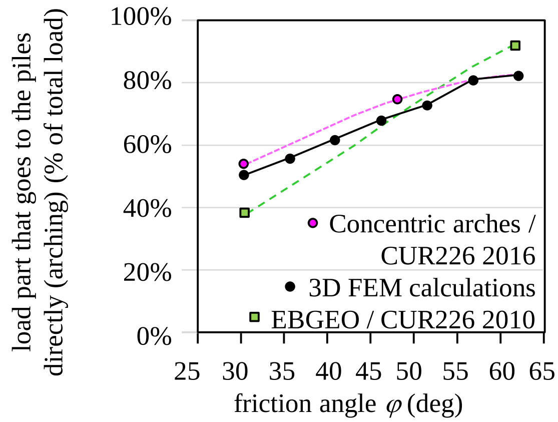


Figure 10: Comparison 3D FEM calculations with design guidelines.

The new Concentric Arches model gives a good match with measurements and numerical calculations and was therefore adopted in the design guideline for basal reinforced piled embankments CUR226 (2016, summarized in Van Eekelen, 2016).

References

CUR 226 (2010). *Design Guideline Piled Embankments*. Stichting CUR, Gouda, ISBN 978-90-376-0518-1 (in Dutch).

CUR 226 (2016). S.J.M. van Eekelen and M.H.A. Brugman, Eds. *Design Guideline Basal Reinforced Piled Embankments.* SBRCURnet & CRC press, Delft, ISBN 9789053676240. https://www.crcpress.com/Design-Guideline-Basal-Reinforced-Piled-Embankments/Eekelen-Brugman/9789053676240.

DuboCalc (2015). https://www.rijkswaterstaat.nl/zakelijk/zakendoen-met-rijkswaterstaat/inkoopbeleid/duurzaam-inkopen/duurzaamheid-bij-contracten-en-aanbestedingen/dubocalc/index.aspx. Last accessed 6 November 2015 (in Dutch).

EBGEO (2010) *Empfehlungen für den Entwurf und die Berechnung von Erdkörpern mit Bewehrungen aus Geokunststoffen.* Deutsche Gesellschaft für Geotechnik e.V. (DGGT).

Van Eekelen, S.J.M. (2015). *Basal Reinforced Piled Embankments*. PhD thesis Technical University of Delft, Netherlands. ISBN 978-94-6203-825-7 (print), ISBN 978-94-6203-826-4 (electronic version). Downloadable at: www.piledembankments.com, including an excel file with the calculation model.

Van Eekelen, S.J.M., (2016). *The 2016-update of the Dutch Design Guideline for Basal Reinforced Piled Embankments*. In: Proceedings 3rd International Conference on Transportation Geotechnics. Procedia Engineering. pp. 582-589 DOI information: 10.1016/j.proeng.2016.06.077.

Van Eekelen, S.J.M., Venmans, A.A.M. (2010). *Piled embankments in the Netherlands; how to decide, how to design?* In: New Techniques on Soft Soils, ed. Márcio Almeida.

Van Eekelen, S.J.M., Bezuijen, A., Van Tol, A.F. (2013). *An analytical model for arching in piled embankments*. Geotextiles and Geomembranes **39**: 78 – 102.

Van Eekelen, S.J.M., Bezuijen, A. van Tol, A.F. (2015). *Validation of analytical models for the design of basal reinforced piled embankments*. Geotextiles and Geomembranes. **43**, No. 1, 56 - 81.

Van Eekelen, S.J.M. and Venmans, A.A.M., (2016). *Piled Embankment or a Traditional Sand Construction: How to Decide? A Case Study*. In: Proceedings 3rd International Conference on Transportation Geotechnics. Procedia Engineering pp. 590-597 DOI information: 10.1016/j.proeng.2016.06.078.

Venmans, A.A.M (2013). *Building with the subsurface for realizing cost-efficient infrastructure*. Proceedings of 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris. Vol. 2, pp. 1781-1784.

Venmans, A.A.M., Förster, U., Hooimeijer, R.H. (2005). *Integral design of motorways on soft soil on the basis of whole life costs*. Proceedings 16th International Conference on Soil Mechanics and Geotechnical Engineering, Osaka. Millpress Science Publishers Rotterdam, vol. 4, pp 2867-2870.

Venmans, A.A.M., Kwast, E., (2011). *Life cycle cost optimization of bridge approach constructions in local and national roads in the Netherlands*. In: Proceedings EPS 2011, Oslo.

WAM-software decision support tool for ground improvement for bridge approach constructions http://dtvirt57.deltares.nl/applications/intraweb/dminiroad/dminiroad.dll.

Ground Improvement and Soil Stabilisation

Behaviour of a compacted subgrade soil and the influence of planar reinforcement in track substructure

Ana Heitor[1*], Buddhima Indraratna[1] and Cholachat Rujikiatkamjorn[1]
[1] *Centre for Geomechanics and Railway Engineering, Faculty of Engineering, Univ. of Wollongong, Wollongong City, NSW 2522, ustralia*
aheitor@uow.edu.au, indra@uow.edu.au, cholacha@uow.edu.au

1 Introduction

Efficient transport systems integrated with the optimal supply chains are vital for the global economy and industrial competitiveness, as well as for our social and environmental well-being. In areas where subgrade conditions can be unfavourable, the resilient performance of the overlying compacted structural fills (i.e. sub-base, base and capping or sub-ballast) under high cyclic stresses and impact loads becomes critical in design (Indraratna et al., 2010). While the cyclic loading performance of the underlying transport formations (subgrade) is traditionally assessed for fully-saturated conditions, most load bearing strata in rail infrastructure remain unsaturated. Past research studies (e.g. Gens, 2010, Alonso et al., 2010, Heitor et al., 2013, Indraratna et al., 2014 and Heitor et al., 2015) showed that the behaviour of such materials is governed by the initial compacted state, i.e. role of matric suction in relation to the degree of saturation. Furthermore, track instability conditions may also result from high cyclic stresses and impact loads (Indraratna et al., 2011) being transmitted to the track substructure layers. One of the most effective methods to reduce the cyclic stresses transmitted to a subgrade layer and minimize particle degradation is to increase the stiffness of the track sub-structure. Among the various techniques available, the use high-strength planar reinforcement such as: geogrids and geocells has been proven to enhance the track substructure performance (Nimbalkar et al., 2012, Ngo et al., 2014, Indraratna et al., 2015).

2 Part A: Small strain behaviour of a compacted subgrade soil

The dynamic properties of the soil such as the small strain shear modulus (G0) are evaluated to characterize the engineering behavior of earth structures subjected to repeated loading (e.g. traffic of heavy, fast moving vehicles). This paper shows that the initial compaction conditions i.e. water content and applied energy, govern compaction effectiveness and, thus, the structure and matric suction of compacted subgrade soil and associated G0. In addition, it also illustrates the dependence of G0 on the suction history that the soil experiences during its service live owing to changes in hydraulic behaviour derived from climatic changes (i.e. rainfall or extended periods of drought) (Fig. 1).

3 Part B: Influence of planar reinforcement in the track substructure

Past research studies have shown that the use of planar inclusions can improve track conditions (e.g. Indraratna et al., 2013), both by promoting smaller particle degradation and the reduction of stresses transmitted to the subgrade. This paper shows a comparison study between different types of geogrids (i.e. different geometry and apertures) tested under relevant track conditions (i.e. mimicking typical train axle loads and speeds). The use of an aperture size of geogrids as a function of the ballast gradation (i.e. 1.15D50) for optimal interlocking of particles is suggested (Fig 2a). The benefits to track stability that arise from proving additional confinement through the use of geocells are also shown (Fig 2b).

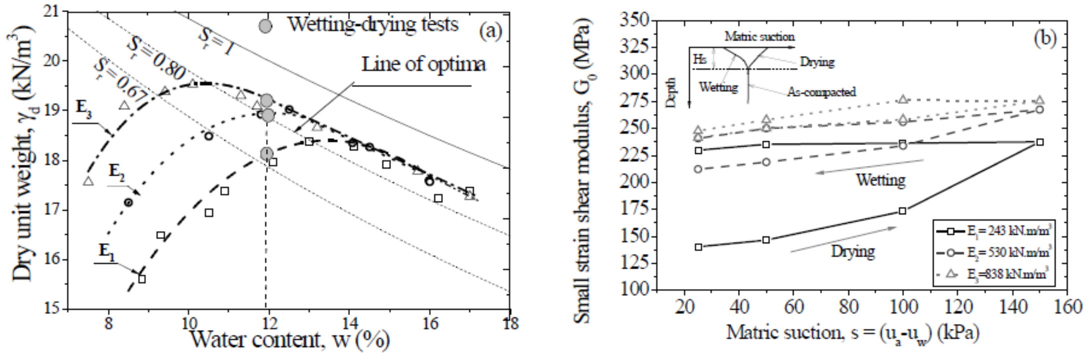


Figure 1: (a) Co mpaction data and (b) variation of G_0 with suction during a drying-wetting cycle of specimens repared at energy levels of 243kN.m/m^3, 530kN.m/m^3 and 838 kN.m/m^3 (data from Heitor et al., 2014)

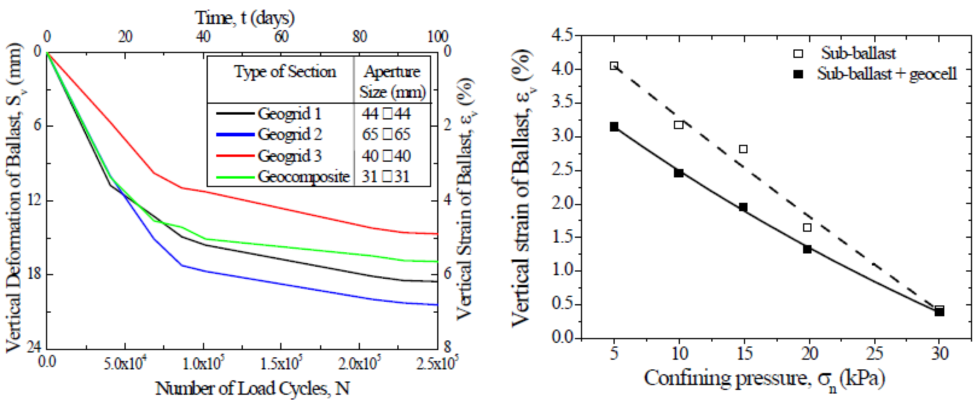


Figure 2: Vertical strains obtained for (a) different geogrids (data from Indraratna et al., 2010) and (b) with geocell for different confining pressures (data from Indraratna et al., 2014)

References

Alonso, E.E., Pereira, J. M.; Vaunat, J., Olivella S. (2010) A microstructurally based effective stress for unsaturated soils. Géotechnique, 60 (12), 913 –925.

Gens, A. (2010) Soil–environment interactions in geotechnical engineering. Géotechnique, 60 (1), 3 –74.

Heitor, A., Indraratna, B. and Rujikiatkamjorn, C. (2013). Laboratory study of small-strain behavior of a compacted silty sand. *Canadian Geotechnical Journal* 50 (2): 179-188.

Heitor, A., Indraratna, B. and Rujikiatkamjorn, C. (2014). Assessment of the post-compaction characteristics of a silty sand. *Australian Geomechanics*, 49(4): 125-131.

Heitor, A., Indraratna, B. and Rujikiatkamjorn, C. (2015). The role of compaction energy on the small strain properties of a compacted silty sand subjected to drying-wetting cycles, *Géotechnique*, 65(9): 717-727.

Heitor, A., Indraratna, B., Rujikiatkamjorn (2015) Effect of suction history on the small strain response of a dynamically compacted soil. *Australian Geomechanics*, Issue 50 (4): 61 – 68.

Indraratna, B., Predamasa, W., Brown, E.T., Gens, A. and Heitor, A. (2014) Shear strength of rock joints influenced by compacted infil. *International Journal of Rock Mechanics and Mining Sciences*, 70: 296–307.

Indraratna B., Thakur P. K., and Vinod J. S. (2010). Experimental and numerical study of railway ballast behaviour under cyclic loading. International Journal of Geomechanics ASCE, 10(4): 136-144.

Indraratna, B. and Nimbalkar, S. (2013). Stress-strain degradation response of railway ballast stabilized with geosynthetics. Journal of Geotechnical & Geoenvironmental Engineering, ASCE 139(5), 684-700.

Indraratna, B., Biabani, M. and Nimbalkar, S. (2015). Behavior of Geocell-Reinforced Subballast Subjected to Cyclic Loading in Plane-Strain Condition. Journal of Geotechnical and Geoenvironmental Engineering, 141(1)

Nimbalkar, S., Indraratna, B., Dash, S. K. and Christie, D. (2012). Improved Performance of Railway Ballast under Impact Loads using Shock Mats, J. of Geotechnical & Geoenvironmental Engineering, ASCE 138(3), pp. 281-294.

Ngo, N., Indraratna, B. & Rujikiatkamjorn, C. (2014). DEM simulation of the behaviour of geogrid stabilised ballast fouled with coal. Computers and Geotechnics, 55, pp. 224-231.

3rd International Conference on Transportation Geotechnics
4 September 2016 | Guimarães | Portugal

Ground Improvement and Soil Stabilisation

Dynamic Rollers in Earthworks: Compaction and Continuous Compaction Control

Johannes Pistrol[1*] and Dietmar Adam[1]
[1] *Vienna University of Technology, Institute of Geotechnics, Vienna, Austria,*
johannes.pistrol@tuwien.ac.at, *dietmar.adam@tuwien.ac.at*

1 Introduction

The quality of earth structures highly depends on the compaction state of fill layers, which can be made up of a wide range of various materials. Compaction is usually accomplished by vibratory rollers; the vibration of the drum is generated by rotating eccentric masses. Moreover, dynamic rollers with different types of excitation have been developed in the last decades, including rollers with directed vibration, feedback controlled rollers and oscillatory rollers.

A high-levelled quality management requires continuous control all over the compacted area, which can be achieved only by work-integrated methods. Roller integrated measurement and continuous compaction control (CCC) result in time and cost savings. CCC provides relative values representing the evolution of the material stiffness all over the compacted area. These values have to be calibrated to relate them to customary values such as deformation modulus of static and dynamic load plate tests defined in contractual provisions and standards.

2 Dynamic Roller Compaction

The concept of vibratory excitation for drums was implemented for the first time in the late 1950s and has become the commonly used type of excitation for dynamic drums. The major benefit of vibratory rollers compared to static rollers is their significantly higher vertical loading due to dynamic excitation, which results in a better compaction at depth.

The eccentric masses of a vibratory drum are shafted concentrically to the drum axis and rotate around this axis with a constant frequency. The rotation of the eccentric masses causes a cyclic translational vibration and a mainly vertical loading of the soil. This implies the main characteristics of vibrating drums, the larger compaction depth and higher ambient vibrations compared to oscillating rollers.

A further development of vibratory rollers was made by company Bomag in 1998 by producing the first roller with directed vibration, which was called Vario® roller. The drum of a roller with directed vibration comprises two counter-rotating eccentric masses of the same mass and eccentricity shafted concentrically to the drum axis. Thus, the eccentric masses generate a directed vibration. The direction can be adjusted manually by rotating the whole excitation unit from horizontal to vertical in defined steps.

The drum of a feedback controlled roller is in accordance with the drum of a roller with directed vibration. However, the inclination of the excitation unit is not adjusted manually but automatically controlled by defined control criteria.

The principle of oscillatory roller compaction was developed in the early 1980s. The dominant direction of compaction of oscillatory rollers results in a lower compaction depth compared to vibratory rollers of the same size and weight. However, an advantage of oscillatory rollers, which makes their application in earthworks a considerable option to vibratory rollers, is given by the significantly reduced ambient vibrations caused by oscillatory rollers. Therefore, oscillatory rollers can also be used in sensitive areas, such as inner city construction sites or on and near bridges.

The drum of an oscillatory roller has two eccentric masses; their shafts are mounted eccentrically but point symmetrically to the drum axis. Two identical eccentric masses with the same eccentricity rotate in the same direction, resulting in a sinusoidal moment around the drum axis that causes a torsional motion in terms of a fast forward-backward-rotation. The described rotation is superposed with the travelling speed of the roller.

3 Continuous Compaction Control (CCC)

In contrast to spot like testing methods continuous compaction control (CCC) is a roller and work integrated method for the identification of soil stiffness. The roller is not only used as compaction equipment but also serves as a measuring device at the same time.

The basic principle of a CCC system is to detect the soil stiffness by evaluating the motion behaviour of the drum. The initial research development of roller integrated measurement dates to 1974 when Heinz Thurner performed field studies for the Swedish Highway Administration with a 5-ton tractor-drawn Dynapac vibratory roller instrumented with an accelerometer. The tests indicated that the ratio between the amplitude of the first harmonic and the amplitude of the excitation frequency could be correlated to the compaction effect and the stiffness of the soil as measured by the static plate load test. In 1975 Thurner founded the company Geodynamik with his partner Åke Sandström to continue the development of the roller-mounted compaction meter. In cooperation with Lars Forssblad (of Dynapac) Geodynamik developed and introduced the compaction meter and the compaction meter value (CMV) in 1978 (Thurner & Sandström).

In the late 1980s Bomag developed the OMEGA value and the corresponding Terrameter® system. The OMEGA value provided a continuous measure of compaction energy and at that time it served as the only CCC alternative to CMV. In the late 1990s Bomag then developed the measurement value Evib, which provided a measure of dynamic soil modulus (Kröber, 2001). Ammann followed suit with the development of a soil stiffness parameter kB (Anderegg & Kaufmann, 2004). These latter Evib and kB parameters signalled an important evolution towards the measurement of more mechanistic soil properties, e.g. soil stiffness and deformation modulus.

Rollers using directed vibration and feedback controlled rollers can be used with CCC systems. When the roller is used as a measurement device the inclination and settings of the excitation unit have to remain constant since the vertical amplitude of the excitation has a large influence on the level of CCC values.

While various CCC systems for vibratory rollers are available no working CCC system existed for oscillating rollers until a short time ago. Therefore, a comprehensive research project on the compaction with oscillating rollers was launched by the German roller manufacturer HAMM AG in 2011 in cooperation with the Institute of Geotechnics at Vienna University of Technology. The aim of the project, which will be continued until September 2016, is a better understanding of the motion behaviour of an oscillating drum and its impact on the compacted soil as well as the development of a CCC system for oscillating rollers and, moreover, the indication of wear of the drum during operation. Within this project large-scale in situ tests were performed with a tandem roller possessing an oscillating drum and a vibrating drum in a gravel pit near Vienna Airport.

The experimental field tests showed a significant influence of the soil stiffness on the motion behaviour of the oscillatory drum and a formation of a secondary vibration with a double frequency compared to the excitation was observed in the vertical soil accelerations throughout all of the performed experimental tests. Because of the upward movement of the drum onto the bow wave during the forward motion and the upward movement onto the rear wave during the backward motion, two periods in vertical direction occur during the same time of one period in horizontal direction. Hence, the vertical acceleration shows a double frequency compared to the frequency of the horizontal acceleration. A novel CCC value for oscillating rollers has been defined based on the characteristic accelerations of the oscillating drum (Pistrol, 2016).

The CCC value for oscillating rollers was evaluated in the scope of the experimental field tests at which the presented CCC value for oscillating rollers properly reflected the increase in soil stiffness with increasing number of roller passes.

References

Anderegg, R., & Kaufmann, K. (2004). Intelligent compaction with vibratory rollers. *Transportation Research Record 1868*, pp. 124-134.

Kröber, W. (2001). Dynamic soil stiffness as quality criterion for soil compaction. *Geotechnics for Roads, Rail Tracks and Earth Structures*.

Pistrol, J. (2016). *Compaction with Oscillating Rollers (Doctoral Thesis).* Vienna: Vienna University of Technology.

Thurner, H., & Sandström, Å. (n.d.). Continuous Compaction Control, CCC. *Proceedings of the International Conference on Compaction*, (pp. 237-245). Paris.

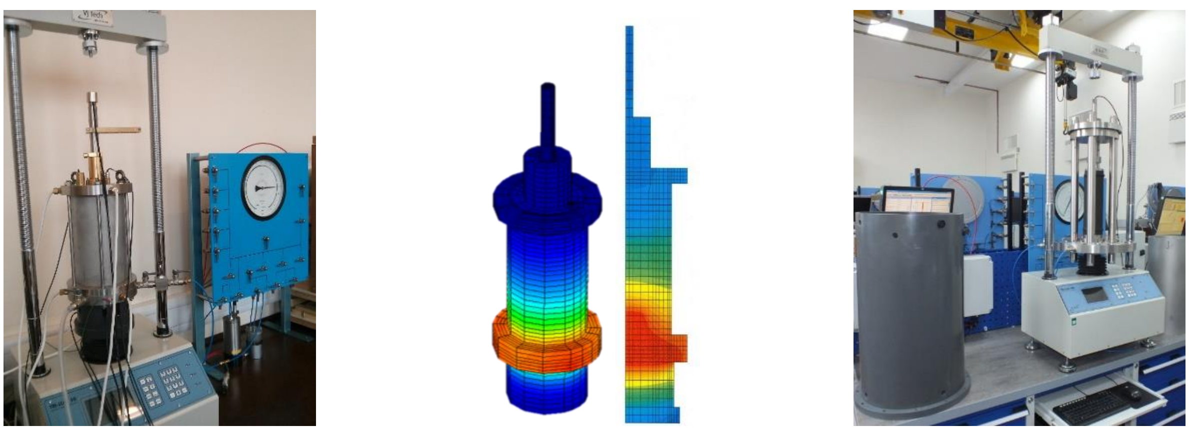
Ground Improvement and Soil Stabilisation

Characterisation and Modelling of Non-Isothermal Soil Behaviour and Implications to Ground Improvement

David M. G. Taborda[1*], Klementyna A. Gawecka[1], Wenjie Cui[1], David M. Potts[1], Lidija Zdravkovic[1] and Aikaterini Tsiampousi[1]
[1]*Imperial College London, London, United Kingdom*
d.taborda@imperial.ac.uk, *klementyna.gawecka09@imperial.ac.uk*, *w.cui11@imperial.ac.uk*, *d.potts@imperial.ac.uk*, *l.zdravkovic@imperial.ac.uk*, *aikaterini.tsiampousi05@imperial.ac.uk*

1 Introduction

The early work on Pasquasia clay and Pontida clay carried out by Baldi et al. (1988) in the context of deep geological repositories for nuclear waste highlighted some of the characteristics of non-isothermal soil behaviour. In particular, this study established that, unlike most engineering materials, normally consolidated clays contract when subjected to drained heating. Subsequent research (e.g. Sultan et al., 2002; Abuel-Naga et al., 2007) confirmed this trend for other soils and demonstrated the need to characterise and simulate accurately the non-isothermal response of geomaterials as it is central to the development of solutions for high level nuclear waste disposal and ground source energy systems, especially when heat is exchanged with the ground through geotechnical structures required for stability, such as piles, retaining walls and tunnel linings. This paper provides an overview of recent research carried out at Imperial College on this topic and explores its applicability to reduction in consolidation time of foundation soils for embankments.



(a) (b) (c)

Figure 11: temperature-controlled triaxial apparatus MkI – (a) general view and (b) finite element modelling of its thermal performance – and (c) temperature-controlled triaxial apparatus Mk II.

2 Temperature-Controlled Laboratory Equipment

Soil-structure interaction phenomena are fundamentally controlled by the behaviour of the ground when subjected to solicitations. Thermo-active structures, in addition to transferring load to the soil, generate significant changes in temperature which need to be accounted for in design procedures. Therefore, at the Imperial College Geotechnics laboratories, a temperature-controlled triaxial apparatus (Figure 11a) was developed, capable of testing samples with diameters of 38 mm or 50 mm, under pressures up to 800 kPa and temperatures up to 85°C. During its design, which is described in Martinez Calonge et al. (2015), extensive studies were carried out using the Imperial College Finite Element Program (ICFEP, Potts and Zdravkovic, 1999), in order to evaluate its thermal performance (Figure 11b). Based on the results obtained using this apparatus, an improved version (Mk II, Figure 11c) was commissioned, which allows samples with diameters up to 100 mm to be tested under higher stress levels (up to 5 MPa) and temperatures ranging from 5 °C to 85 °C. This ability to apply temperatures below ambient level is important as thermo-active structures in the UK tend to be used primarily for heating purposes, thus reducing ground temperatures. Moreover, a special set of temperature-controlled pedestals and top caps allows a thermal gradient to be established in order to quantify the sample's thermal conductivity. Lastly, a temperature-controlled oedometer cell was designed for testing samples of 70 mm in diameter under one-dimensional conditions and temperatures from 5 °C to 85 °C.

3 Numerical Modelling of Non-Isothermal Soil Behaviour

The bespoke finite element code, ICFEP (Potts & Zdravkovic, 1999), has been upgraded to enable the simulation of fully-coupled THM soil behaviour. This procedure, detailed in Cui (2015), involved the development of numerical algorithms for simulating heat transfer in porous media through conduction (i.e. Fourier's law) and convection, as well as the implementation of suitable boundary conditions. Important aspects of the adopted formulation include: (a) the ability to predict the generation of excess pore water pressures due to the greater coefficient of thermal expansion of the liquid phase; (b) the increase of permeability with temperature due to changes in viscosity of pore fluid; and (c) the extension to the THM capabilities to one-dimensional elements.

4 Conclusions

Research on non-isothermal soil behaviour at Imperial College resulted in the development of both experimental techniques and numerical methods. While initial studies involved the evaluation of the thermal performance of the new laboratory equipment, current research focuses on the simulation of the behaviour of geotechnical structures when subjected to temperature fields. The specific case of an embankment built on soft Bangkok clay using prefabricated vertical thermal drains (Pothiraksanon et al., 2010) has been analysed to illustrate how non-isothermal soil behaviour can be explored to reduce consolidation times in normally consolidated clays. Moreover, detailed studies are carried out to establish the influence of (a) the assumptions regarding the modelling of the thermo-mechanical response of Bangkok clay, (b) the use of one-dimensional elements to simulate the prefabricated vertical thermal drains and (c) the variation of permeability with temperature due to changes in viscosity.

References

Abuel-Naga, H.M., Bergado, D.T. and Bouazza, A. (2007). *Thermally induced volume change and excess pore water pressure of soft Bangkok clay*. Engineering Geology, 89: 144 -154.

Baldi, G., Hueckel, T. and Pellegrini, R. (1988). *Thermal volume changes of the mineral-water system in low-porosity clay soils*. Candian Geotechnical Journal, 25(4): 807-825.

Cui, W. (2015). *Development, implementation and application of thermo-hydro-mechanical couplings in finite element analysis.* PhD thesis. Imperial College London, London.

Martinez Calonge, D., Gawecka, K.A., Zdravkovic, L., Sim, W.W. and Taborda, D.M.G. (2015). *Development of a new temperature-controlled triaxial apparatus for saturated soils*. Proceedings of the 16th European Conference on Soil Mechanics and Geotechnical Engineering, Edinburgh.

Pothiraksanon, D., Bergado, D.T., Abuel-Naga, H.M. (2010). *Full-scale embankment consolidation test using prefabricated vertical thermal drains*. Soils and Foundations, 50(5): 599-608.

Potts, D.M. and Zdravkovic, L. (1999). *Finite Element Analysis in Geotechnical Engineering: Theory.* Thomas Telford, London.

Sultan, N., Delage, P. and Cui, Y.J. (2002). *Temperature effects on the volume change behaviour of Boom clay*. Engineering Geology, 64: 135-145.

3rd International Conference on Transportation Geotechnics
4 September 2016 | Guimarães | Portugal

Ground Improvement and Soil Stabilisation

Performance of test embankment under vacuum consolidation

Marcio Almeida[1*] and Esther Marques[2]
[1]*COPPE/UFRJ, Rio de Janeiro, Brazil*
[2]*IME, Rio de Janeiro, Brazil*,
almeida@coc.ufrj.br, esther@ime.eb.br

1 Introduction

Due to stability considerations, embankments over clayey deposits are sometimes built in several stages or with lighter materials, associated or not with vertical drains, depending on the construction schedule. With vacuum consolidation, the equivalent stress of a 4.5m conventional pre-loading fill can be applied to the clayey deposit in four to six months, as a function of the horizontal coefficient of consolidation of the soil and the vertical drains spacing. The increase in vertical effective stress is due to the decrease in pore pressure, so the vacuum consolidation can be applied in one single step, even on very soft soils.

Although the vacuum consolidation is not yet a widespread technique, it has been successfully used in different parts of the world (Choa 1989, Cognon et al. 1994, Qian et al. 1992, Shang et al. 1998, Van Impe et al. 2001, Dong et al. 2001). In order to study the vacuum consolidation technique and heating, applied over a typical eastern Canadian clay deposit, two trial embankments 13m x 13m were executed approximately 30km North of Montreal, at Saint-Roch-de-l'Achigan. The deposit was subjected to vacuum consolidation (trial embankment A) and vacuum consolidation and heating (trial embankment B), down to approximately 7.5m depth. The heating was applied for five months and the vacuum consolidation was carried out over five months. The idea of in situ heating comes from the fact that permeability increases with temperature. It was also observed in consolidation tests, that the heating of a sample in the normally consolidated domain increases its deformation and strain-rate (Leroueil and Marques, 1996). There are a few studies on in situ heating (Miliziano, 1992; Bergenståhl et al., 1994; Edil & Fox, 1994; Moritz, 1995), and it was the first time that a vacuum consolidation pre-loading was installed in Canada and the first time that vacuum consolidation was associated with heating.

2 Site characterization and vacuum site characteristics

The clay deposit is quite homogeneous, composed of a 2.5m thick clay crust followed by a 10m slightly overconsolidated silty clay deposit very well studied, particularly laboratory tests as shown in Fig. 1. Preconsolidation by vacuum consists in applying vacuum on a deposit by pumping water from a grid of vertical and horizontal drains, decreasing pore pressures inside the deposit.

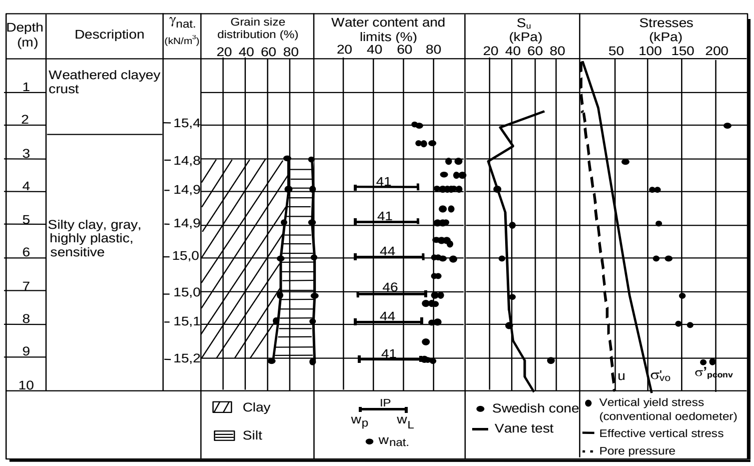


Figure 1: Geotechnical characteristics of Saint-Roch-de-l'Achigan clay deposit (Marques et

At the Saint-Roch-de-l'Achigan site, the 2” circular prefabricated vertical drains were installed with a spacing of 1.15m up to a depth of about 7.5m in the silty clay deposit, under the two trial embankments. Horizontal drains were placed in the 60cm thick sand layer, and they carried out the water pumped from inside the deposit towards the trenches. At embankment B, thin copper tubes (diameter = 3/8") were installed inside the vertical drains and hot water was pumped inside these tubes down to 7.3m depth, in a closed hydraulic system, independent of the vacuum pumping system.

First, all the instrumentation was installed in the trial embankments and the heating was turned on at trial B. At embankments A and B, the vacuum pumping was turned on 30 and 58 days after beginning of heating, respectively. The fills were raised 60 days after the beginning of vacuum application, so the deposit could achieve the normally consolidated domain.

3 Pore pressures

For study purposes the deposit was divided in three sub-layers and the piezometers and the settlement gauges were placed at the middle and bottom of each sub-layer, respectively, as shown in Fig 2, thus allowing calculation of vertical deformations and average pore-pressure of the layers.

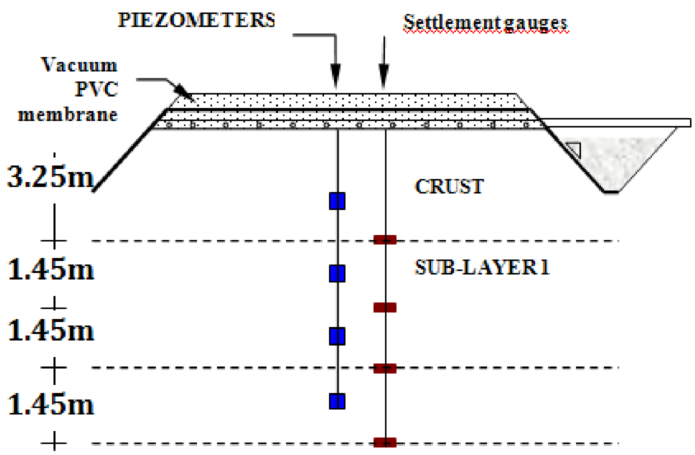


Figure 2: Instrumentation profile.

Figure 3 shows the pore pressure variation and deformation with time of sub-layer 2. It shows also the suction and embankment heights with time and the initial piezometric profile. The Casagrande piezometers and vibrating wire piezometers installed outside the preloading area indicated that the initial pore pressure profile was not hydrostatic. However due to the small spacing of drains, the eight vibrating wire piezometers installed under the trial embankments after the installation of the vertical drains presented an initial hydrostatic profile. Two vacuum meters were installed at the sand fill of each embankment in order to measure the suction inside the sand fill.

The pore pressure decreased with time with vacuum application as shown in Fig. 3. When the fill was heightened, it increased accordingly. After that it continually decreased with consolidation and vacuum application. Sixteen settlement plates were also installed over the trial embankments, and the total vertical displacements of the eight vibrating wire settlement gauges were also measured. Figure 3 shows the vertical deformation of sub-layer 2.

The rate of decrease of pore pressures at embankment B was slightly higher, but not proportionally to the increase of permeability with temperature observed in laboratory tests, where an increase of 2.2 times in permeability was observed, when temperature was increased from 10 to 50ºC (Marques, 2001).

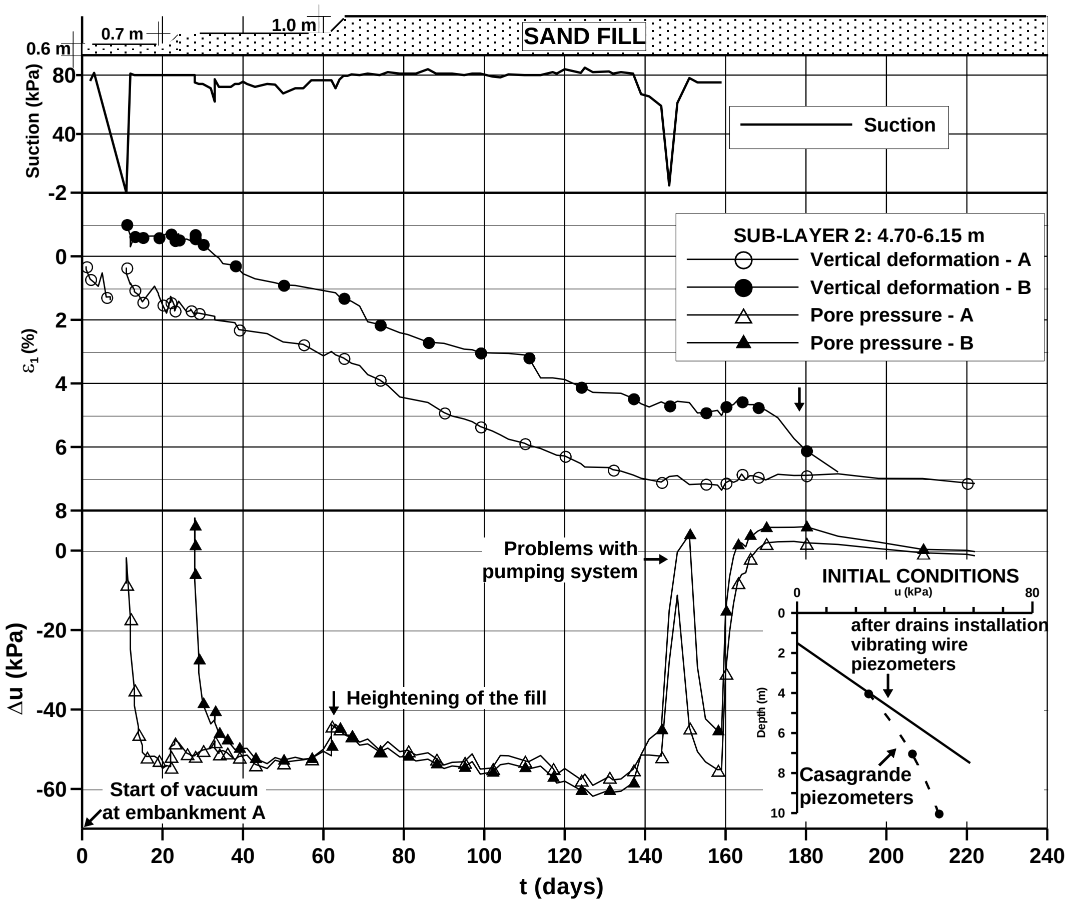


Figure 3: Suction, vertical deformation and pore pressure variation with time at sub-layer

4 Vertical displacements

Sixteen settlement plates were also installed over the trial embankments, and the total vertical displacements of the eight vibrating wire settlement gauges were also measured. Figure 3 shows the vertical deformation of sub-layer 2.

The heating of embankment B was turned on before the vacuum, when the deposit was overconsolidated and there was expansion, a behavior that was also observed in laboratory. However, after the vacuum consolidation was applied, the settlements of the fills were very similar.

An increase in strain-rate was observed on both embankments when the fill was heightened. The strain rates were about 10-9s-1, on both embankments when pumping was turned off, which is a very high in situ strain rate, when compared to conventional fills.

5 Conclusions

The main objectives of the experimental site were to observe the performance of the vacuum system applied on a sensitive Canadian clay deposit, and to study the in situ viscous behavior. Better results under vacuum consolidation at this site could not be obtained due to the high depth of the water table and the initial pore-pressure conditions, which decreased the efficiency of the vacuum system. However, the instrumentation used proved to be adequate to the very special conditions on this site.

The heating proved to be a very expensive technique and the in situ behavior contradicted the laboratory observations (Marques, 2001; Marques et al. 2003) with respect to temperature variation of embankment B. The stress variation due to vacuum application plus fill was only high enough to surpass the preconsolidation pressure, wich could be the main reason for such behavior. However, it is possible that if the deposit had been well inside the normally consolidated domain, the heating effect on the behavior would be more important, mainly with respect to vertical deformations.

6 Acknowledgements

The authors are grateful to the Ministère des Transports du Québec that has made the present study possible.

References

Bergenståhl, L., Gabrielsson, A. & Mulabdic, M., (1994). *"Changes in soft clay caused by increases in temperature"*. In: Proceedings of the 13th International Conference on Soil Mechanics and Foundation Engineering, New Dehli, pp. 1637 - 1641.

Choa, V., (1989). *"Drains and vacuum preloading pilot test"*. Proc. 12th Int. Conf. on Soil Mech. and Found. Eng, Rio de Janeiro, pp. 1347 - 1350.

Cognon, J. M., Juran, I. & Thevanayagam, S., (1994). *"Vacuum consolidation technology - principles and field experience".* Vertical and Horizontal Deformations of Foundations and Embankments, Settlement'94, College Station. Geot. Spec. Pub. no 40, ASCE, pp. 1237 - 1248.

Dong, Z. L., Zhen, X. L. & Kan, M. M. K., (2001). *"Effect of vacuum preloading and road base surcharge on the lateral displacement of surrounding soil in highway ground improvement"*. Soft Soil Engineering, Lee et al. (eds), Swets & Zeitlinger, pp. 459-464.

Edil, T. B & Fox, P. J., (1994). *"Field Test of Thermal Precompression"*. In: Vertical and Horizontal Deformations of Foundations and Embankments, Settlement'94, College Station, Geotechnical Special Publication no 40, ASCE, New York, New York, pp. 1274 - 1286.

Jacob, A., Thevanayagam, S. & Kavazajian, E., (1994). *"Vacuum-assisted consolidation of a hydraulic landfill".* Vertical and Horizontal Deformations of Foundations and Embankments, Settlement'94, College Station. Geot. Spec. Pub. no 40, ASCE, pp. 1249 - 1261.

Leroueil, S. & Marques, M. E. S., (1996). *"Importance of strain rate and temperature effects in geotechnical engineering. Measuring and modeling time-dependent Soil Behavior".* Geotechnical Special Publication no 61, ASCE, pp. 1 - 60.

Magnan, J. P., (1994). *"Maîtrise des amplitudes et vitesses de tassement des remblais sur argiles molles"*. Évolutions récentes. Bulletin de Liaison des Laboratoires des ponts et chaussées, vol. 194, pp. 27 - 40.

Marques, M. E. S., Leroueil, S., Varaksin, S., Demers, D. & D'Astous, J., (2000). *"Vacuum preloading of a sensitive Champlain Sea Clay deposit"*. 53th Conférence Canadienne de Géotechnie, Montréal, vol. (2), pp. 1261 - 1268.

Marques, M. E. S. (2001). *"Laboratory and field viscous behaviour of clayey soils (in portuguese)".* DSc. Thesis, COPPE/UFRJ, Rio de Janeiro, Brazil, in cooperation with Université Laval, Québec.

Marques, M.E.S., Leroueil, S. &. Almeida, M. S. S. (2003) - *Viscous behaviour of a Champlain Sea clay from Saint-Roch-de-l'Achigan, Québec* - Submited to the Canadian Geotechnical Journal.

Miliziano, S., (1992). *"Effetti della temperatura sul comportamento meccanico delle terre coesive".* Tese de D.Sc., Consorzio entre l'Universitá de Rome La Sapienza e de Napoli Federico II, Itália.

Moritz, L., (1995). *"Geotechnical properties of clay at elevated temperatures"*. In: Compression and Consolidation of Clayey Soils, (ed) Yoshikuni & Kusakabe, pp. 267 - 272.

Qian, J. H., Zhao, W. B., Cheung, Y. K. & Lee, P. K. K., (1992). *"The theory and practice of vacuum preloading".* Computers and Geotechnics, vol. 13, pp. 103 - 118.

Shang, J.Q., Tang, M & Miao, Z., (1998). *"Vacuum preloading consolidation of reclaimed land: a case study*". Canadian Geotechnical Journal, vol. 35(5), pp. 740-749.

Van Impe, W. F., Van Mieghem, J., Vandycke, S. & Van Impe, P. O., (2001). *"Underwater vacuum consolidation of dredged silt – a case history".* Soft Soil Engineering, Lee et al. (eds), Swets & Zeitlinger, pp. 499-506.

Ground Improvement and Soil Stabilisation

Alkali Activated Binders in Soil Stabilisation Applications

Tiago Miranda[1], Nuno Cristelo[2] and Sara Rios[3]
[1]University of Minho, Guimarães, Portugal
[2]University of Trás-os-Montes e Alto Douro, Vila Real, Portugal
[3]University of Porto, Porto, Portugal
tmiranda@civil.uminho.pt, ncristel@utad.pt, sara.rios@fe.up.pt

1 Introduction

Chemical stabilisation is a very efficient method to improve the behaviour of low quality soils, which cannot present an adequate performance based only on mechanical stabilisation. The use of a soil-binder combination specifically developed for sub-grade layers can significantly increase its strength and stiffness, avoiding the extraction and transportation of more interesting materials from distant locations (Ingles & Metcalf, 1972).

The most common binders used for surface chemical stabilisation are based on Portland cement (OPC) and/or lime (Petry & Little, 2002; Xing et al, 2009). The high calcium content of lime is very effective in decreasing the plasticity, and thus increasing the workability of natural clays, due to the pozzolanic properties of the minerals present. Additionally, significant strength and stiffness increase are also to be expected, particularly if dry weight percentages above 6% are used (Little et al, 2010; Al-Mukhtar et al, 2014). When dealing with silty to sandy soils, OPC is usually the most interesting option (Consoli et al, 2011; Bahar et al, 2004). Several new binders have been tested, with more or less success, but in almost every case OPC plays a major role.

Although cement and lime are well capable of producing acceptable soil stabilization for sub-bases, with little or no setbacks whatsoever, the decreasing tolerance regarding the overall production of OPC, mostly due to the high amount of CO2-eq released, significantly hinders the perspective use of such binder in the years to come. Therefore, there is an ongoing overall research effort targeting the development of more sustainable binders. In particular, the use of waste materials is highly encouraged, since it allows an increase in resource efficiency, by allowing a decrease in OPC production, while contributing also to reduce landfilled waste. The alkaline activation technique is particularly adequate to create binders based on residues, such as fly ash or ground granulated blast furnace slag, which constitute very effective options due to their amorphous aluminosilicate microstructure. It consists on a reaction between aluminosilicate materials and alkali or alkali-based earth substances, such as sodium (Na) or potassium (K), or an alkaline earth ion, such as calcium (Ca).

2 Soil Stabilisation with Alkaline Activation

Alkaline activation has recently started to be applied in soil stabilisation applications, using fly ash as precursor, allowing significant strength gains (Zhang et al, 2013; Cristelo et al., 2011; Rao and Acharya, 2014; Phummiphan et al., 2016), but also financial and environmental benefits, very competitive with lime or cement (Cristelo et al., 2015).

However, a significant workload is still needed to allow a consistent application of this new family of binders. For instance, the technical accommodation of the geotechnical (mechanical) and chemical demands has proved hard to achieve. The optimum liquid phase content, obtained by a Proctor test, will only match the ideal liquid phase, in terms of chemical reactions, by coincidence. The Proctor test itself raises some issues, since it needs to be executed with activator, instead of plain water. Otherwise, it yields inaccurate data, namely regarding the maximum volumetric weight. On the other hand, the use of activator, instead of water, introduces a significant error source in terms of the water content associated with each point. Several possibilities have been tested in the last few years, in different soils, and still no satisfactory procedure for designing the ideal mixtures was defined. Figure 1a shows the results obtained from several Proctor tests on a clayey soil stabilized with 10% fly ash (FA) and different activators (always with a silicate / hydroxide ratio of 0.5). Included is the Proctor test made with plain water, clearly showing that this is not a valid option to determine the density of the mixtures. This information needs to be determined using the activator as the liquid phase, even if the test does not always produces a clear result, as shown in Figure 1b, when the same concentrations were used with a silicate (SS) / hydroxide (SH) ratio of 1.0. This issue has a strong influence on mechanical behavior (Figure 2), as a decrease in activator concentration produces a higher liquid content and, at the same time, less effective chemical reactions between the activator and the fly ash.

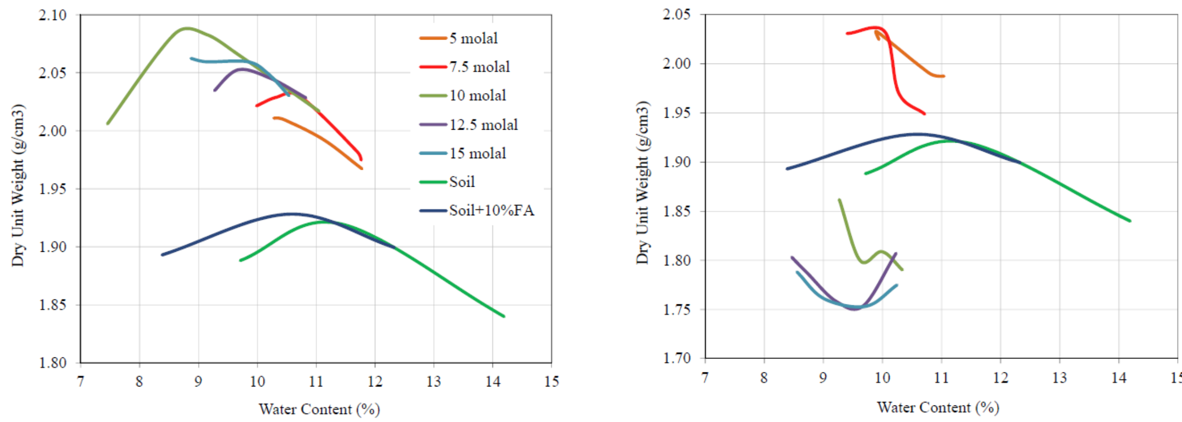


Figure 12: Results from several Proctor tests on a clayey soil stabilized with 10% FA and different activators – SS/SH = 0.5 (left) and SS/SH = 1.0 (right)

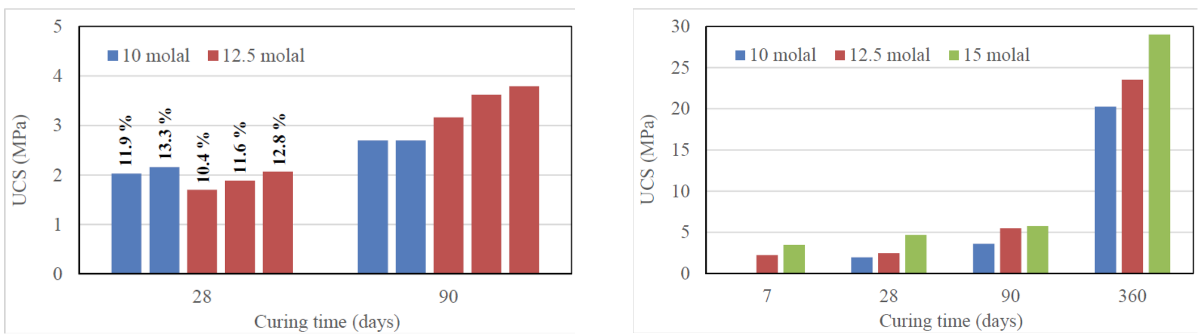


Figure 13: UCS results obtained from two different soils and stabilization concepts

References

Ingles, O.G. and Metcalf, J.B. (1972). *Soil Stabilization: Principles and Practice*.

Petry, T. and Little, D. (2002). *Review of Stabilization of Clays and Expansive Soils in Pavements and Lightly Loaded Structures—History, Practice, and Future.*

Xing, H., Yang, X., Xu, C. and Ye, G. (2009). *Strength characteristics and mechanisms of salt-rich soil-cement*.

Little, D., Nair, S. and Herbert, B. (2010). *Addressing Sulfate-Induced Heave in Lime Treated Soils.*

Al-Mukhtar, M., Lasledj, A. and Alcover, J.F. (2014). *Lime consumption of different clayey soils.*

Consoli, N., Dalla-Rosa, A., Saldanha, R., (2011). *Variables Governing Strength of Compacted Soil-Fly Ash-Lime Mixtures*

Bahar, R., Benazzoug, M. and Kenai, S. (2004). *Performance of compacted cement-stabilised soil.*

Zhang, M., Guo, H., El-Korchi, T., Zhang, G. and Tao, M. (2013). *Experimental feasibility study of geopolymer as the next-generation soil stabilizer.*

Cristelo, N., Glendinning, S. and Teixeira Pinto, A. (2011). *Deep soft soil improvement by alkaline activation.*

Rao, S. and Acharya, P. (2014). *Synthesis and Characterization of Fly Ash Geopolymer Sand*.

Phummiphan, I., Horpibulsuk, S., Sukmak, P., Chinkulkijniwat, A., Arulrajah, A. and Shen, S. (2016). *Stabilisation of marginal lateritic soil using high calcium fly ash-based geopolymer.*

Cristelo, N., Miranda, T., Oliveira, D.V., Rosa, I., Soares, E., Coelho, P. and Fernandes, L. (2015). *Assessing the production of jet mix columns using alkali activated waste based on mechanical and financial performance and CO2 (eq) emissions.*

Ground Improvement Solutions for Harbours

António Cristóvão[1*], Alexandre Pinto[1] and Rui Tomásio[1]
[1]*JETsj Geotecnia, Lda, Lisbon, Portugal*
acristovao@jetsj.com, *apinto@jetsj.com*, *rtomasio@jetsj.com*

1 Abstract

Over recent years the world witnessed the construction of new port infrastructures has they play a crucial role on the development of worldwide trading and regional economic growth. This type of infrastructures commonly requires the occupation of considerable areas of land, allowing the creation of large shipping and storage areas. However, the availability of free land together with deep water coast lines, compatible with the use of large draft vessels, is becoming increasingly hard to find. The solution then involves the construction of deep quay walls and the creation of technically demanding artificial platforms, challenging the engineering capacities. Seeking for inexpensive and appropriate technical solutions, capable to overcome the challenges imposed by the construction of such infrastructures, use has been made of ground improvement solutions. Its wide range of techniques, easily adapted to different technical scenarios and geological conditions, are considered to be an added value in most projects. This paper describes the contribution provided by different ground improvement techniques on the construction of new harbours and maritime facilities.

2 Jet Grouting

Ground improvement using jet grouting is considered to be one of the most versatile technique, has it can easily adapt to a considerable range of situations and different geotechnical conditions.

In most cases, new harbours involve the construction of deep quay walls. In the African shoreline, the new Container Terminal at Lomé – Togo, demanded the construction of a 1000m long quay wall. It comprised a twenty nine meter deep reinforced concrete diaphragm wall, connected to an anchored dead man wall through a forty five meter long steel tie rod system.

As a consequence of a complementary site investigation undertaken during construction, a weak clayey silty layer was detected in some areas along the quay wall demanding the ground improvement at the wall base. As the diaphragm wall was already constructed, a solution consisting on jet grout columns was then used, allowing preventing punching and contributing to increase the overall wall stability. Jet grouting columns, installed at the passive side of the wall had contributed to increase the overall quay wall stability, whilst, jet grout columns intersecting the wall provided higher skin friction, increasing the bearing capacity.

The versatility of jet grouting columns also enables its use on temporary earth retaining structures, allowing overcoming earth stability problems during construction. A case study of jet grouting walls located at the transition zone between a new quay wall and an existent breakwater are presented. It consists of two different solutions, according to the excavation geometry and the main constraints observed at the site.

The first solution comprised a temporary gravity wall and the second solution refers to the junction of the perpendicular quay wall with the existing breakwater. Both adopted solutions were found to be profitable to overcome demanding local constraints, such as the presence of distributed blocks from the existing rock fill.

To be well succeeded, both jet grouting and vibro techniques require an exhaustive geological-geotechnical campaign, carried out before and during the works, as well as the execution of appropriate field trials.

Preliminary field testing shall constitute the basis of any ground improvement solution, allowing the adaptation of the design assumptions to the real conditions observed at each site. In addition to laboratory field tests on jet grouting samples, suitable to determine the composite soil-cement material stiffness and resistant properties, it is to highlight the advantages of using non-destructive methods to assess the jet columns diameter at the site. Regarding this matter, direct measurements undertaken on an exposed jet grout column were compared with resistivity measurements according to the electric cylinder method. It has been found that the electric cylinder method provides accurate results, making of it a valid method to estimate the jet grout columns diameter. One of the most important advantages of the aforementioned method is that it provides results shortly after the jet grout columns installation, thus, reducing the time ussually consumed on this kind of task.

3 Deep Soil Vibration

Ground-improvement techniques by deep soil vibration have been broadly used in port facilities, aiming to create stable platforms with suitable characteristics for container storage and equipment operation.

At the Lomé Container Terminal, upper frictional soils were found to be inadequate to receive the design imposed loads and therefore vibrocompaction works were carried out to improve soil characteristics.

The proposed port platform required the placement of up to 3.50m thickness of fill material to form a base for the concrete pavement slab.

Being a cost effective and a high production rate technique, vibrocompaction is considered to be a competitive solution on improving the characteristics of platforms formed by granular soils. Once again, field trails were found to be crucial to confirm the adequacy of the solution and determine the adequate grid spacing of the compaction probe. Independent of the treatment grid spacing adopted, vibrocompaction clearly increased the relative density of the granular materials, typically corresponding to behaviour soil type index behaviour (I_c) of between 1.31 and 2.05. Conversely, no evident improvement was found in soils with a behaviour soil type index (I_c) higher than 2.05. Based on the results of soil relative density testing two and ten days after vibrocompaction treatment, it is confirmed that the relative density tends to increase with time after vibrocompaction.

Careful use of vibrocompaction is advisable when at the presence of dense sands, typically with CPT (cone penetration test) cone resistance values varying from 25 to 30MPa, as a decrease of soil initial stiffness may be expected. During vibrocompaction works, any contact with underlying clay layers may lead to soil contamination due to washed in clay material, precluding the achievement of higher relative density results in the upper granular soil layers.

With respect to Quality Control and Quality Assurance, the use of DPSH (deep probe super heavy) testing was found to be a good instrument for vibrocompaction quality control assessment, however, calibration with CPT data performed at the site shall always be undertaken.

A 6.0m thick clayey silty layer, found at a depth of 12.0m on a singular area of the container storage platform, determined the installation of stone columns. As the upper granular soil layers had to be treated by vibrocompaction, the option relied on a combined solution, therefore, stone columns were built from 19.0m to 11.0m depth and vibrocompaction treatment was following executed until ground surface.

The use of stone columns in combination with vibrocompaction has proved to be an optimized and cost effective solution, enabling improving simultaneously clayey silt layers positioned underneath granular and compactable soils.

4 Final Remarks

An overall appreciation of the different ground improvement works presented in this study had clearly demonstrated how jet grouting together with vibro techniques can contribute to solve different engineering problems in new Harbours and maritime facilities construction.

References

EN 12716:2001 Execution of special geotechnical works. Jet grouting
EN 14731:2005 Execution of special geotechnical works. Ground Treatment by deep vibration

Ground Improvement and Soil Stabilisation

The advantages of the use of advanced quality control methods during PVD installation and heavy rapid impact compaction in transportation Geotechnics

Jeroen Dijkstra[1*]
[1]*Cofra B.V. Amsterdam,The Netherlands*
jeroen.dijkstra@cofra.com

1 Introduction

In the recent years large improvements have been made in the registration and visualization of production data during ground improvement. The driving feature behind this improvement is the use of GPS guided logging systems for the registration and the real time positioning of the improvement locations. This extended abstract provides information about the system and the advantages of the use of the GPS data for two ground improvement techniques, prefabricated vertical drains and Cofra dynamic compaction. Both techniques will be elaborated in the next chapters.

2 Prefabricated Vertical Drains

PVD, also known as wick drains, are one of the most commonly used techniques to make soft compressible subsoil with a low bearing capacity constructible. The prefabricated vertical drains are installed using a wide range of stitchers. The heaviest rigs are capable to penetrate qc = 20MPa sand layers. The principal of the installation of the vertical drainage is based on the insertion of a steel mandrel with a drain inside. This mandrel is moved up and down through a system of cylinders and winches, which in turn are propelled by the excavators' hydraulic system. The drain section at the bottom of the mandrel is connected to an anchor plate which closes off the opening to prevent soil from entering the mandrel. The mandrel then takes the drain to the desired depth. When the mandrel is at this depth, it is withdrawn and the resistance created by the anchor plate upon retraction ensures that the drain remains in place at the right depth. After the mandrel is back above the surface, the drain is cut and a new anchor plate is connected to the bottom of the next drain. The principle is shown in Figure 1.

The installation process is monitored by the operator on a screen in the cabin, see Figure 2. The screen is divided into a section showing a GPS referenced AutoCAD top view and a section with the real time data of the installation. The uploaded AutoCAD drawings can provide the operator with all the information required, such as drain depths, underground infrastructure, limitations on height and other restrictions. This system was specially developed for Cofra B.V. and includes the knowledge of several decades of PVD installation. The system has proven itself to be of added value for both the contractor as well as the client and can be used on projects upon the request of the client or engineer. With the use of the data, top views of the installation data as well as cross sections can be made showing amongst others the installation force with depth. This makes it possible to map resistance features, such as intersecting sand layers or extends of soft soils. When unforeseen deviations are encountered, this data can be shared with the client to investigate the effect on the consolidation process, to mark the locations for predrilling or to monitor the area with the use of additional settlement plates. Figure 2 shows an example of the images produced. During the presentation more examples will be shown.

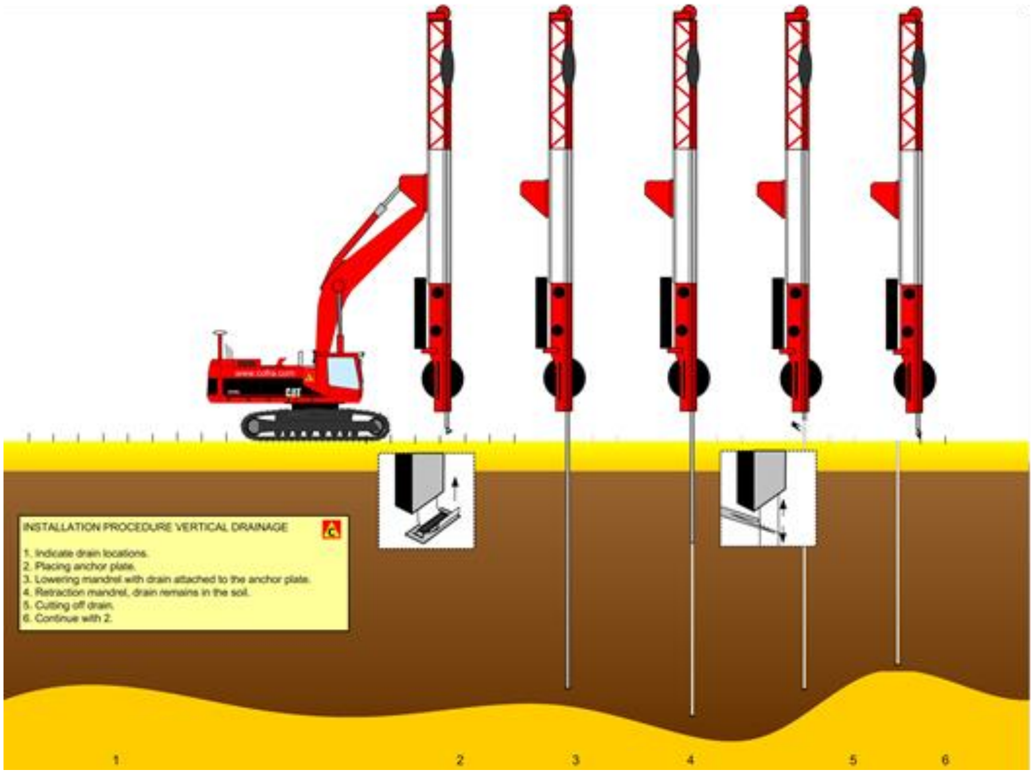


Figure 1: Principle of prefabricated vertical drain installation www.cofra.com

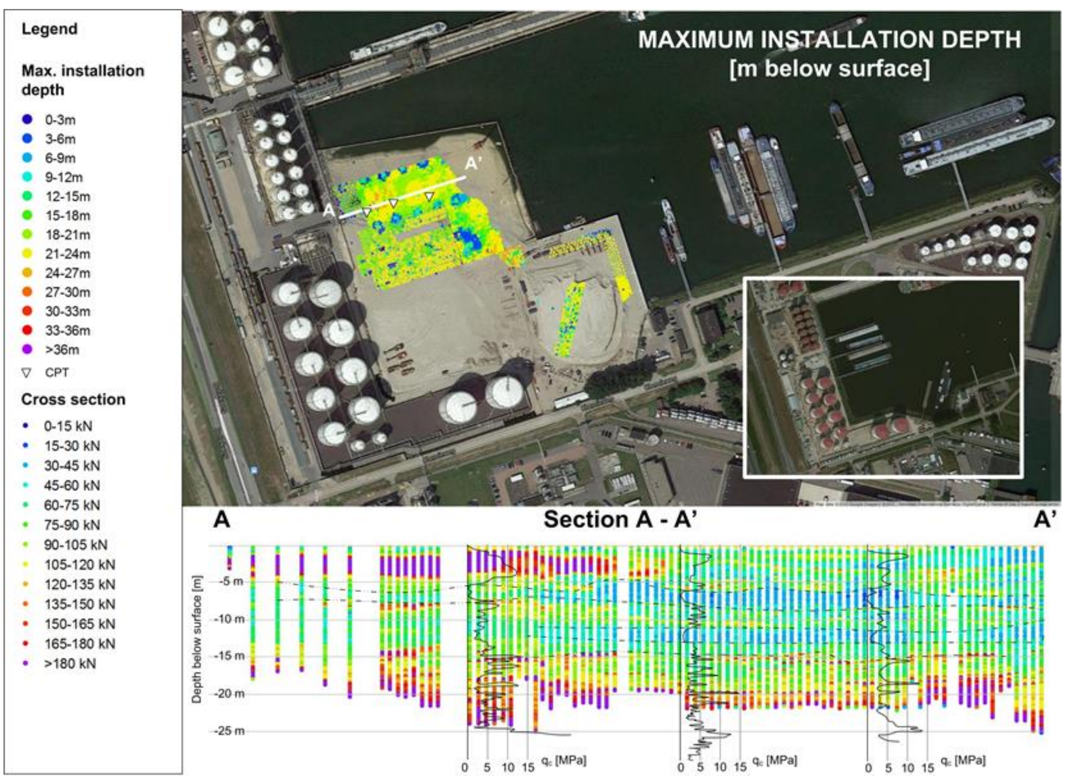


Figure 2: GIS compilation of the logger data showing maximum installation depth and a cross-section through section A-A' (Dijkstra, 2015)

3 Heavy Rapid Impact Compaction (CDC)

The CDC technique or heavy rapid impact compaction is a relatively unknown compaction technique used to compact granular material fast, homogeneously and with a high accuracy. Depending on the substrata and the applied energy, the technique is capable of compacting the soil till a depth of 9 meters below the surface. The equipment consists of a heavy excavator equipped with a specially designed arm onto which the hammer is attached. Within the hammer a 16 ton drop weight is hydraulically lifted till a predetermined height after which the weight is dropped using an hydraulic acceleration. The weight hits the cushion and compaction foot with a speed equal to free fall. The whole process of lifting and dropping takes place with a frequency of 40 times a minute. During the compaction of a location, the foot, with a diameter between 1.5m and 2.6m, remains into contact with the soil. The compaction locations are placed with center to center distances between 2.0m and 3.5m. The compaction of the subsoil is initiated by the vibrations generated during the impact of the hammer onto the foot. Moreover the movement of the foot into the ground, the heavy weight of the equipment and high energy transfer is also causing densification. The dense compaction grid assures that a homogeneous compaction is reached throughout the area because locations are also affected by the compaction of nearby points improving the overall performance. The work sequence as well as an example of the zone of influence are given in the sketches below. The process is illustrated in Figure 3.

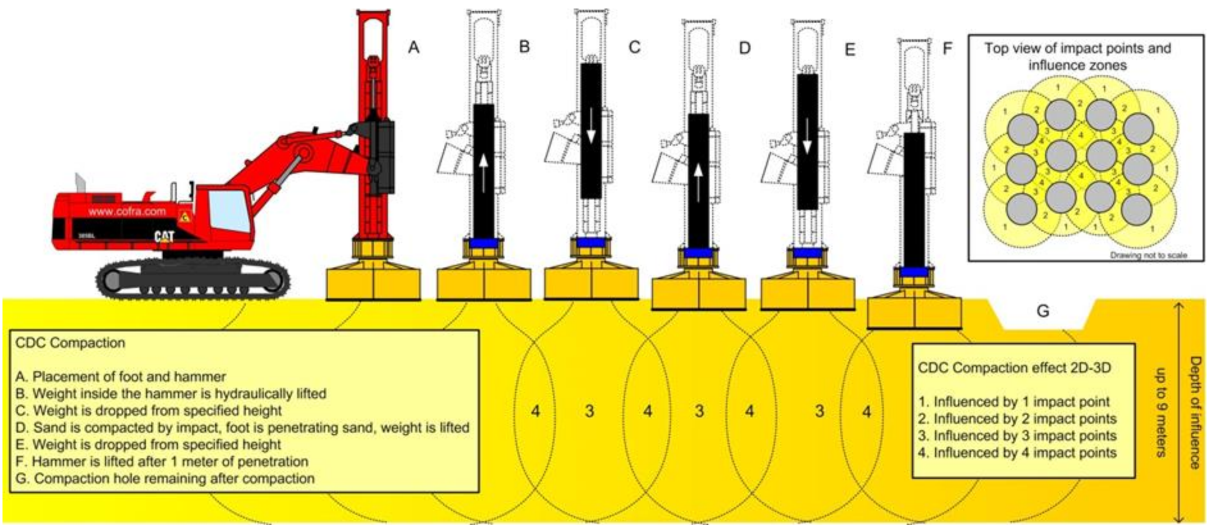


Figure 3: Illustration of RIC work method and overlap of influence zones RIC www.cofra.com

The GPS guided quality control system is also an important part of the compaction method. During the compaction operations several parameters are logged and displayed on the display of the operator. The operator has the possibility to indicate three different stop criteria on the display, being: number of blows, the induced settlement (average settlement per blow of the last blows) and total settlement. Due to the mostly heterogeneous soil conditions (mainly regarding compaction levels) on project sites the more advanced induced settlement (settlement per blow) is often used as a stopping criterion for each compaction point to achieve homogeneous compaction. An example of project data where no induced settlement is used is given in Figure 4. It can be seen that areas are present where a softer profile is to be expected. More information can be found in the paper (Vink et al,2016) written for this conference.

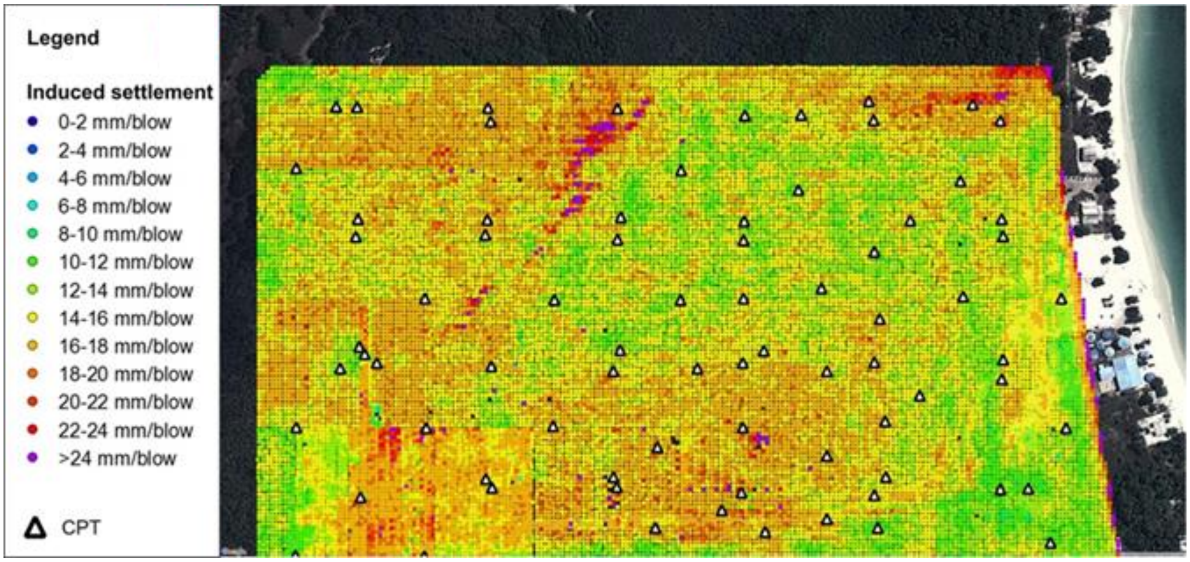


Figure 4: Induced settlement data of a project (Dijkstra, 2016)

References

Dijkstra J.W., Laumen T, Paasen L.A. (2016, may). Optimizing the relationship between CDC compaction induced settlement data and the average CPT cone resistance after compaction, 19th Southeast Asian Geotechnical Conference & 2nd AGSSEA Conference (19SEAGC & 2AGSSEA) Kuala Lumpur.

Dijkstra J.W., Vestering C. (2015, December). The advantages of the use of GPS based logging systems for vertical drain installation projects, International Conference on Soft Ground Engineering (ICSGE2015), Singapore

Cofra, website Cofra B.V (2016, May) . www.cofra.com

Vink J.W., Dijkstra J.W. (2016, September) CDC compaction at Berth 9 Quay Extenxion Felixstowe, UK, 3rd ICTG 2016, The 3rd International Conference on Transportation Geotechnics, Portugal

Ground Improvement and Soil Stabilisation

Rigid Inclusions a Ground Reinforcement Solution rather than a Ground Improvement Solution

Baldomiro Xavier[1*]
[1]*Teixeira Duarte Engenharia e Construções SA., Portugal*
bx@teixeiraduarte.pt

Soil Improvement is an old technic that has been around for decades. It is widely used everywhere but, it is still a matter that calls for a specialist's analysis. It is a subject mainly ruled by empiric knowledge which is a ground where most people don't like to set their foot on. The technic's concepts are basically simple and most engineers can get into it by using simple logic relations in Soil Mechanics instead of sophisticated computer software with several encapsulated black boxes.

When a large area is to be charged with heavy distributed loads and the existing ground does not have the necessary loading capacity, there is a need for a strengthening intervention in order to prevent the soil's collapse or the occurrence of unacceptable settlements. In those situations soil improvement is known to be the most efficient/competitive solution, when compared to other types of soil strengthening interventions. It is fast to implement and besides some environmental restrains, like noise or induced vibration, it seems to be a clear winner.

The main issue is the soil itself. Is it capable of being improved?

Normally a soil is loose because it is poorly graded and it can be improved only to some extent. A soil with SPT of 5 blows can easily become a 10 blows soil, but achieving an improvement grade of even 30 blows is most likely to be a very hard and time consuming job. That is why in manmade compacted embankments it is required the use of specially selected (sandy) soils which are then submitted to densification processes by repetitive action of vibrating rollers over thin layers of soil in order to achieve the desired compaction.

When soil improvement is known to be inadequate, soil reinforcement is the strengthening method that follows. By introducing external elements into the soil, the ground will benefit from the resistance of the inclusions and gain an improvement on its capacity due to soil/inclusion interaction. Existing technology allows the use of several types of inclusions such as, sand columns, stone columns, soil-cement columns, mortar or concrete columns, etc.

Depending on how they are introduced into the ground, besides forming stiff structural underground elements, to some extent they can also promote the soil densification in between those columns.

Like setup in driven piles, skin friction can improve several times, but relaxation over time will do the opposite, so it is not wise to rely on setup in the case of piles, working as single elements. But here, in what soil reinforcement is concerned, there is a mesh of elements working together rather than a pile working on its own, so the gain of resistance can be taken into account. However, a significant portion of the invested energy is lost due to the soils uplifting (heaving) during the installation process, as the soil always chooses the easiest way to rearrange. On the other hand, as soil becomes denser, it will limit the usability of some technologies like, for instance, full displacement inclusions, putting the lower layers of soil to be improved, beyond reach of the treatment.

Sand columns and stone columns are the most common methods in soil improvement design. However these columns cannot exist without the support of the soil they are supposed to reinforce, and they normally share with the improved soil the resistance to the external loads nearly at equal shares.

In the case of very loose soils under heavy loads, it is necessary to rely on a denser mesh of columns. This need is related to the fact that the resistance of the column is governed by de confining soil. On the other hand, the column's capacity is inversely proportional to its length, so in the case of a thick layer to be improved stone columns are no longer an efficient option.

By adding cement to the columns or even by cementing the soil, one can easily get self-sustained columns with a tenfold increase in resistance, for a reasonable increase in cost, by turning them to a much more reliable/rigid type of inclusions. Under these circumstances, the columns with an enhance modulus of deformability will take almost all the loads with much less overall settlements. Good results can be achieved by putting the soil and columns to work together under a simple but comprehensive rule, through the equilibrium of skin friction over the soil-column interface. Whenever the soil is weak, it will transfer the loads to the column through negative skin friction, so that the columns can carry the loads to the stiffer layers of soil, underneath. A well compacted interposing layer between the loads and columns is convenient, but a transfer platform is no longer mandatory.

Historically the first inclusions used as soil improvement elements were timber piles, driven into the top layers of soil in order to promote local densification, especially in wetlands. It can be found in downtown Lisbon under the heavy walls of six stories buildings that stand over a thick layer of alluvial sand with the water table just three meters below the surrounding streets. Thanks to this ground improvement procedure many historical buildings, rebuild after he massive 1755 earthquake, still endure remarkably well.

Nowadays there are many types of rigid inclusions, from jet-grouting, colmix, displacement piles, full displacement piles, auger piles and a few more variants. To distinguish its functionality from the piles they are named columns. As far as the material's resistance is concerned each solution fits between soil-cement and concrete, giving a wide range of options available to each design. The technology for the installation of the inclusions plays an important role in the improvement process; in some cases it is convenient to use different procedures to cover the full scope of the job.

An inclusion of colmix is probably more precise in shape than a column of jet grouting, but the high energy of the jet is more favourable in densifying the soil. If precision and control are required, concrete columns should be the correct option. With the modulus of deformability and dimensions well established they can either be driven, installed as full displacement elements or as auger bored piles. Auger piles can reach the design depth without major concerns, but full displacement piles or even driven piles are frequently limited by the capacity of the installation rigs, leaving part of the design requirements unfulfilled.

Author index