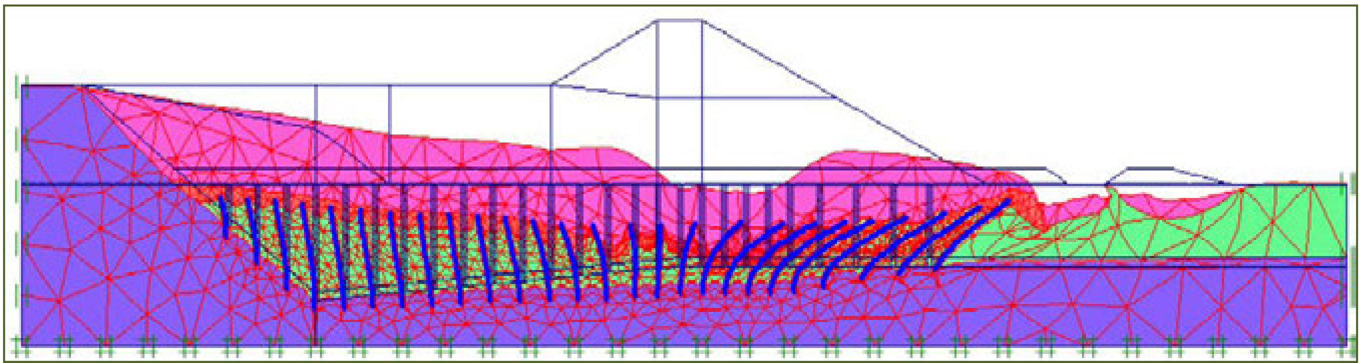
PROCEEDINGS OF THE INTERNATIONAL SYMPOSIUM

TC 211 IS-GI Brussels 2012



Recent Research, Advances & Execution Aspects of
GROUND IMPROVEMENT WORKS
31 May – 1 June 2012, Brussels, BELGIUM

VOLUME V

Organised by:

ISSMGE Technical Committee TC 211 Ground Improvement

Belgische Groepering voor Grondmechanica en Geotechniek
Groupement Belge de Mécanique des Sols et de la Géotechnique

Comité Français de Mécanique des Sols

Edited by:

Nicolas Denies & Noël Huybrechts

SIMSG ISSMGE

GBMS BGGG

CFMS

Recent Research, Advances & Execution Aspects of Ground Improvement Works

Edited by

Dr. Ir. Nicolas Denies
Belgian Building Research Institute (BBRI-CSTC-WTCB), Belgium

Ir. Noël Huybrechts
Belgian Building Research Institute (BBRI-CSTC-WTCB), Belgium

Editorial address

Ground Improvement is a large and important domain in soil mechanics and geotechnical engineering and consists in a wide variety of techniques and methods adapted to a broad range of problems. The amount of contributions to the proceedings of this symposium is certainly a proof of that.

It cannot be denied that during the last decades the importance of the ground improvement market has enormously increased. New methods, tools and procedures have been developed and applied in practice.
In order to support this evolution in a scientific way, research programs have been and are being carried out worldwide, leading to more and better insights and delivering the basis for the establishment of design methods, quality control procedures and standards.

Due to the increasing interest of the construction sector for Ground Improvement techniques, the Belgian Building Research Institute (BBRI) has got more and more involved in projects addressing ground improvement during the last decade, most of them in a fruitful partnership with the Belgian Association of Foundation Contractors (ABEF).
In line with this evolution, the Geotechnical Division of the BBRI supports since 2005 the activities of the ISSMGE TC 211 Ground Improvement, which resulted in June 2012 in the organization of the International Symposium on Ground Improvement Works with more than 1600 pages of publications spread out over 4 Volumes:

- *Volume 1* of the proceedings contains the contributions of the 7 General Reporters, the Louis Ménard lecture held by *Patrick Mengé*, and the specialty lecture of the ISSMGE president *Jean-Louis Briaud*.
- *Volumes 2 to 4* contain more than 140 papers, subdivided in 7 Sessions, each of them dealing with a particular domain of Ground Improvement.

It can be noted that 40% of the papers deal with soil stabilisation and deep mixing, proving the huge interest in these techniques. This is not surprising, as they are outstanding and competitive sustainable construction methods.

At the occasion of the International Symposium, three parallel Short Courses on Ground Improvement were held assessing the following topics:
- Marine Ground Improvement;
- Deep Mixing;
- Rigid Inclusions and Soil Reinforcement.

The *Volume 5* of the present proceedings has been written in order to summarize and harmonize the main contributions of these Short Courses.

We believe that the content of the present proceedings gives a very good overview of recent and on-going research actions and practices with regard to Ground Improvement. Moreover, we are convinced that they will contribute significantly to the further development of quality control procedures and standards.

Finally we would like to thank the Belgian Federal Public Service Economy, the NBN (Belgian standardization organization) and the Flemish Governmental Agency for Innovation by Science and Technology (IWT) for their financial support of the BBRI research programs addressing Ground Improvement techniques.

The Editors,

Nicolas Denies & Noël Huybrechts
Geotechnical Division, Belgian Building Research Institute, Brussels, Belgium

Table of contents of VOLUME V

CONTRIBUTIONS

SUMMARY OF THE SHORT COURSES OF THE IS-GI 2012
LATEST ADVANCES IN MARINE GROUND IMPROVEMENT

SUMMARY OF THE SHORT COURSES OF THE IS-GI 2012 LATEST ADVANCES IN MARINE GROUND IMPROVEMENT

Bert Lietaert, DEME – Dredging International, Belgium, Lietaert.Bert@deme.be
Fanny Maucotel, Menard, France, Fanny.Maucotel@menard-mail.com

ABSTRACT

This paper is the summary of the short course 2 on Marine Ground Improvement which was given during the international symposium on "Recent Research, Advances and Execution Aspects of Ground Improvement Works" in 2012, in Brussels. The need for ground improvement is discussed, with special attention to difficult fill materials and problematic subsoils. Also the importance of a good quality site investigation is stressed out in the light of accurate geotechnical modelling and ground improvement design. The most important ground improvement methods are summarized and illustrated by examples from practice. An important aspect of ground improvement works is the quality control afterwards. The most important techniques are revised and special attention is given to quality control in difficult materials like crushable sands. Throughout the text, the latest advances on marine ground improvement, both regarding the (combination of different) techniques itself as on quality control are incorporated.

1. INTRODUCTION

From May 31 to June 1, 2012, the ISSMGE Technical Committee 211 Ground Improvement hosted in Brussels, Belgium an international symposium on Recent Research, Advances & Execution Aspects of Ground Improvement Works. On the first day of the symposium short courses were organized. Short Course 2 (SC2) focused on Marine Ground Improvement. Specialists with a varying background shared their theoretical and practical knowledge on marine ground improvement. Speakers included: P. Mengé and M. Van den Broeck (DEME, Belgium), S. Bretelle (GHD, Australia), Ph. Liausu (Ménard, France), S. Lambert (Keller, France), J. Dykstra (COFRA, The Netherlands), I. Chu (Iowa State University, USA-Singapore), B. Indraratna (University of Wollongong, Australia) and J. Maertens (Belgium). The content of their presentations is gathered and summarized in the text below. The aim is to summarize the content of the presentations of the SC2, added with the most recent advances on marine ground improvement.

The text starts with reflecting on the need for ground improvement. The chapter focusses on some specific aspects which need to be taken into account when evaluating the need for ground improvement and selecting a suitable technique. First of all, it is explained that for projects requiring the construction of a fill, the need (and amount) for ground improvement is highly affected by the construction method of the fill. Secondly, failure due to liquefaction is briefly explained and how this risk can be minimized by applying ground improvement. Thirdly, problematic fill materials and problematic subsoils, which always require ground improvement if encountered in construction projects, are discussed.

In the next chapter, the focus is put onto the site investigation. In the light of marine ground improvement, the scope of the site investigation and those parts of which are typically related to ground improvement works are discussed. The remainder of the chapter discusses regularly encountered pitfalls related to marine site investigations.

After that, the different ground improvement techniques are discussed. First of all, the most important classification methods are presented. Then, a distinction is made between ground improvement techniques without admixtures and ground improvement techniques with granular admixtures in cohesive soils. Also the combination of different techniques is discussed. The chapter ends with a paragraph on the reuse of dredged materials. Besides the discussion of the different techniques, several examples from practice are given.

The next chapter focusses on the requirements related to ground improvement works and the quality control of the obtained improvements. The most often encountered requirements and their (ir)relevance are discussed. Also for the quality control techniques, an overview is given of the standard applied techniques. Besides that, attention is given to possible pitfalls related to quality control in special materials (e.g. carbonate sands) and alternatives are proposed.

2. THE NEED FOR GROUND IMPROVEMENT

2.1. Introduction

For large infrastructure projects, it is required that on a long term base the subsoil does not affect the integrity of the structures on top. Therefore, designers often pose several requirements towards the subsoil, both towards in-situ soil as well as towards constructed fill. These requirements are related to the physical characteristics of the reclamation materials, the levels of compaction to be achieved, settlements, bearing capacity, resistance to liquefaction,… Ground improvement is required when the fill mass or the underlying soil do not meet these design requirements. The need for ground improvement can be brought back to four principal reasons:

- In order to guarantee sufficient slope stability and bearing capacity; there is often a need to improve the strength of the fill mass and subsoil;
- To prevent excessive settlements or horizontal deformations, the fill mass and subsoil do need to have a certain stiffness;
- Sufficient density of the fill mass and subsoil is required in order to guarantee the resistance against liquefaction;
- The drainage capacity of the fill mass will depend of the permeability of the fill mass

Numerous ground improvement techniques exist to achieve the goals set above. It is not always obvious whether ground improvement will be necessary and if so, determine which technique is the most suitable one for the existing situation. In order to facilitate this decision and related choice for a certain technique, the remainder of this chapter will focus on some specific aspects which need to be taken into account when evaluating the need for ground improvement.

2.2. Hydraulic fill

Large reclamation projects are often built up by hydraulic means. The preferred material for fill construction is obviously a granular material with a limited percentage of fines and stone-sized fragments. The relative density of a hydraulic fill prior to compaction is largely dependent on the placement method.

In dredging industry, common disposal methods of saturated soil under water are spraying, dumping, pipeline discharge (from above the water table) and rainbowing. Common methods for placement of saturated soil above water are free flow through a pipeline and rainbowing. The associated relative densities for each of these disposal methods are indicated in Table 1.

Table 1: Typical relative densities, achieved without any additional compaction or treatment (from van 't Hoff and Nooy van der Kolff., 2012)

Placement method	Relative density
Saturated soil – under water	
- Spraying - Dumping - Pipeline discharge (from above the water table) - Rainbowing	- 20-40 % - 30-50 % - 20-40 % - 40-60 %
Saturated soil – above water	
- Free flow through pipe - Rainbowing	- 60-70 % - 60-80%
Dry soil	
- On a dump truck (filled from funnel) - On the discharge area (compacted by bulldozers) - On the discharge area (compacted by specialized equipment)	- 10-20% - 50-60% - Up to 100%

It is clear that besides the disposal method, also the influence of equipment (bulldozers) operating on the reclamation area cannot be underestimated. If the placement method and site conditions allow, these bulldozers are constantly spreading out the granular material during the reclaiming process. They often cause a significant increase in relative density in the upper part of the fill. This is often observed in practice, however, it remains difficult to estimate this effect in advance.

Besides the choice of an appropriate filling method, grains can also be arranged into a denser state by the process of natural 'auto compaction' (cf. Figure 1). This is also called 'ageing effect'. It is often observed that the measured cone resistance at a particular location increases with time due to ageing. This can be observed both over short time periods (30 days) as over longer time periods (years). This is related to different phenomena such as creep (deformation with time under a constant load), interlocking of grains (related to the angularity), activity of cementing agents, increase in effective vertical stress due to dissipation of excess pore water pressure, etc.

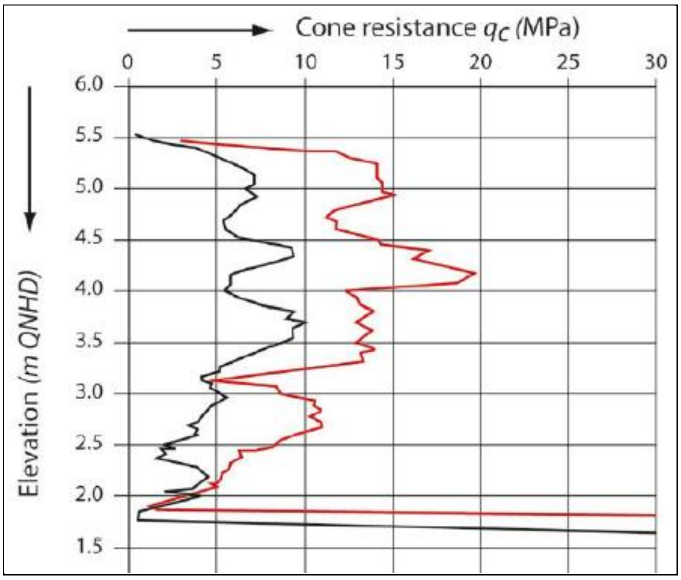


Figure 1: Comparison of cone resistances (above the water table) directly after placement (black line) and after 3 months (red line). Increase in cone resistance due to 'ageing' as a result of cementation of calcareous sand

If the required relative density is higher than the obtained relative density after hydraulic fill, taking into account the positive effects of operating equipment and natural auto compaction, then additional compaction of the fill is required.

2.3. Liquefaction

Failure due to liquefaction in saturated, granular soils is an often encountered risk when dealing with hydraulic fills. For large construction projects in seismic regions, it is important to evaluate the liquefaction resistance of the natural soils or installed fill (cf. Youd et. al., 2001). The liquefaction hazard is mitigated by using ground improvement. Although various methods have been proposed for mitigation of liquefaction (cf. e.g. Port and Harbour Research Institute, 1997), densification is still one of the most widely used methods.

Liquefaction is related to the interaction of the skeleton grains and the pore water. If for some reason pore water pressures are allowed to build up inside a soil volume, the risk exist that these reduce the effective stress to zero and consequently, the soil volume loses its shear strength and failure occurs. This phenomenon is referred to as failure due to liquefaction. The density of the fill plays herein a crucial part.

Granular soils which are loosely packed possibly have a contractive behaviour when subjected to shearing. This implicates that after shearing the particles become more densely packed and consequently, the pore volume decreases. Depending on the permeability of the soil, the pore water will or will not be easily able to be squeezed out of the pores. If pore water cannot easily drain, the pore water pressure will build up, increasing the risk for liquefaction failure.

The contractive behaviour of a granular soil volume is besides the relative density also influenced by other parameters. The most important ones are summarized below.

- The fabric of the sand skeleton should be considered. The fabric is influenced by the age of the deposit; the more recent the sand mass has been deposited, the more sensitive the sand structure or fabric becomes to liquefaction at the same relative density.

- The uniformity coefficient of the granular soil mass is also an important factor: the more poorly graded a sand mass is, the easier the contractive flow behaviour, at the same level of relative density and water content.
- Sands containing an amount of fines of (low plasticity) clays or silts are often referred to as ‘dirty sands’. These sand deposits show a potential for contractive flow failure much higher compared to a clean sand at the same relative density since the fines obstruct the drainage paths for the water.
- The angularity of the particles also has an influence; rounded particles will be more prone to flow phenomena compared to angular particles.
- The initial mean effective stress at the depth of the triggering shear stress increase however remains the most important parameter. At the same relative density, the same granulometric characteristics and the same water content, a contractive flow pattern is more probable the higher the initial effective stress level.

Liquefaction can be triggered by different sources. If the constructed fill is for example situated in a seismic region, the risk exists that external loading, often earthquake primary waves, generate pore water pressure build-up inside the fill which possibly lead to (seismic/cyclic) liquefaction failure. However, also during dredging and filling operations, liquefaction can occur, although no seismicity is involved. This phenomenon is called static or gravitational liquefaction. It can occur in situations with loosely packed sand layers. If the initial density of the sand is less than the critical density at a certain mean effective stress, the sand will show contractive behaviour when sheared, which generates pore water pressure build-up and reduces the effective soil stresses. If filling or dredging slopes are becoming too steep, a small incident can generate shear stress in the soil mass and trigger liquefaction. This phenomenon is also referred to as ‘flow slides’.

2.4. Problematic fill materials

The type of material that is used for filling operations will have a large influence on the need for ground improvement. Although it is generally known that cohesive materials are not the best fill materials, they are sometimes used in the fill. In that case, it can be expected that the engineers involved are aware of the problematic fill material and are well prepared. Granular materials are generally considered as good fill materials. Unfortunately, there are also granular materials which need special attention. For example, some sands are characterised by high (intact) shell content (cf. Figure 2). These sands will have different geotechnical characteristics compared to sands consisting dominantly out of quartz grains. The so called ‘carbonate sands’ need special attention. These are sands of which the grains are easily crushable. If unaware of the particular geomechanical behaviour of these sands, requirements put forward by the designer may not be realistic or achievable.

Figure 2: Shelly sands

Sands with a high content of carbonate minerals, most often CaCO3, are often called **‘crushable sands’**. These carbonate rich sands are often found in (sub)tropical environments, on the shallow waters of the continental shelves. Calcareous sands are usually composed of skeletal or non-skeletal remains of marine organisms and therefore, they have unique characteristics in terms of their mineralogy, surface roughness, particle shape, intraparticle (or secondary) porosity (high void ratio) (cf. Figure 3) and crushability at relatively low stresses. Their unique composition implies that their geotechnical engineering behaviour can be substantially different compared to that of terrigenous sands which consist dominantly of silica (SiO_2).

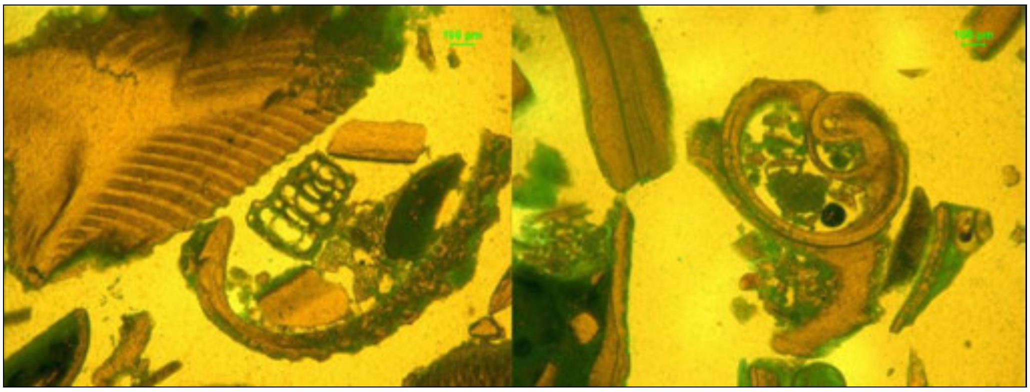


Figure 3: Typical (bioclast) components of crushable sand

These sands are often encountered at large scale land winning projects in the Middle East. They often have to be dredged and have to be used as fill material. Although many different types of ground improvement techniques (dynamic compaction, vibroflotation, roller compaction, HEIC, RIC,…) have been used on this type of material, the effect of the compaction on the crushing is not known widely and also not well documented in literature. However, it is evident that crushing occurs and this crushing might influence the effectiveness and the production rate of the applied technique. It might for example be necessary to densify the compaction grid for dynamic compaction, the vibroflots might need a longer vibration time, more passes might be required with the HEIC,… In a worst case scenario, a particular technique might even become not suitable anymore because of too much increase in fine content.

Since these sands have a different composition and thus geotechnical behaviour, it also implies that the results of standard in-situ and laboratory tests should be interpreted with caution, because correlations between these tests and geotechnical parameters are often derived from extensive testing with standard silica sands. This implies that the use of such tests often requires site specific calibration.

2.5. Problematic subsoils

In reclamation projects, the focus is often put on the characteristics of the fill material. However, the type of subsoil below the fill may also have a large influence on the feasibility of the designer’s requirements. There are some typical subsoils which are known as problematic if encountered in construction projects. If these soils are encountered, chances are high that a particular soil improvement technique will be required. If analysis indicates that soil improvement is required, it is essential to understand the associated geological formation process and the specific characteristics of these subsoils, in order to select the most suitable technique.

2.5.1. Collapsible subsoils

Karst is a phenomenon which is typical for limestone rocks. These limestone rocks, characterized by high carbonate content, are likely to dissolve slowly by infiltration of acid rich rain- or groundwater. The solution process leaves behind a mass of unweathered limestone, containing sinkholes.

Figure 4: Typical landscape affected by karst near Minerve, Hérault, France

These sinkholes are typically formed in two ways. Some develop gradually over many years without any physical disturbance to the rock. In these situations, the limestone immediately below the soil is dissolved by downward-seeping rainwater that is freshly charged with carbon dioxide. With time, the bedrock surface is lowered and the fractures into which the water seeps are enlarged. As the fractures grow in size, soil subsides in to the widening voids, from which it is removed by groundwater. These depressions are usually shallow and have gently slopes (cf. Figure 4). By contrast, sinkholes can also form abruptly and without warning when a roof of a cavern collapses under its own weight. Typically the depressions created in this manner are steep-sided and deep (cf. Figure 5).

Figure 5: 18 m diameter sinkhole in Guatemala City (2010)

It is clear that when working in a limestone rich area, karst may cause problems both on long and short term. Gradual weathering of limestone rock may cause foundation problems on a long term. The rock which was initially excellent as a foundation bearing layer can become unsuitable over time. Short term problems related to karst may be for example the collapse of a cavern under the load of heavy equipment. Also dewatering of a construction site by means of pumping can be problematic if the subsoil is affected by karst.

In a Sabkha environment, one should also be aware for the risk of collapsible soils. **Sabkha's** are often found in hyper-arid to semi-arid areas over the world. They are often defined as gently dipping salt flats along the coast line consisting of alternating layers of algae mats, clastic materials and evaporates (e.g. gypsum) (cf. Figure 6). These evaporates result from evaporation of seawater during periodical inundations. The top of these sabkhas is often characterized by a relatively hard (salt-rich) crust from which the thickness and strength may vary considerably in a lateral direction.

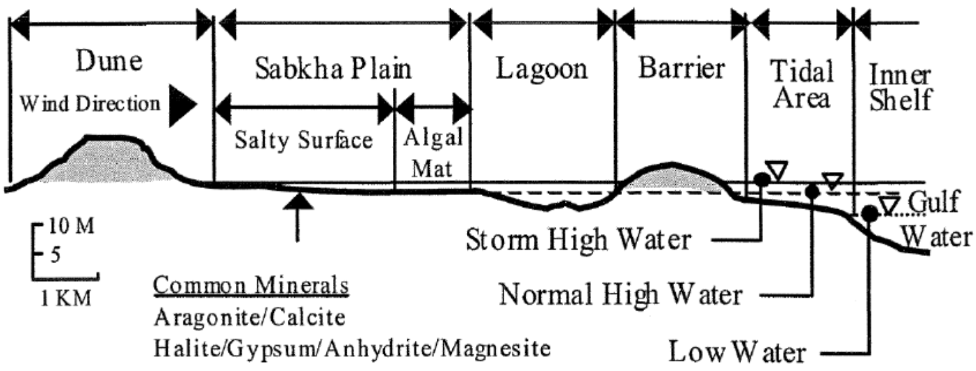


Figure 6: Typical sabkha cross-section

It is clear that such an environment may be the subject of different geotechnical hazards. For example, (local) dissolution of the crust could result in a decrease in strength and may cause problems related to bearing capacity (punch-through), differential settlements, slope stability problems, etc. Also the high salt content may lead to problems; the hydratation and dehydratation of gypsum for example may lead to considerable volume changes. Furthermore, in an attempt to improve the density of this upper part, the use of traditional ground compaction technique may break up this crust even further.

2.5.2. Other problematic subsoils

Sensitive clays may be very problematic when encountered near project sites. The sensitivity of a clay is defined as the ratio between the undisturbed and remoulded shear strength. If this range is > 5-10, the clay is defined as sensitive. Typical for these very soft clays is that they lose their shear strength almost completely if disturbed (remoulded). A possible cause of this disturbance can be related to construction or dredging activities. If the shear strength is lost within a soil volume, the risk exists of a total failure of the soil mass resulting in for example large scale landslides or loss of bearing capacity and foundation failure. In Scandinavian countries, the preferred ground improvement technique when dealing with sensitive clays is soil mixing with cement or lime.

Peat is formed during decomposition of dead organic substances like remnants of plants and animals. The variety of stems, leaves, biological matter and biochemical circumstances are the main causes for the natural heterogeneity of these soils. Peat formed in early post-glacial times may occur at depth, buried beneath more recent sediments. If economically feasible, the preferred option is often to remove the peat layers from below the reclamation area. If not removed, the typically associated geotechnical risks should be considered. Considering the large heterogeneity of peat deposits and the difficulty to take representative samples, it is not evident to define the geotechnical parameters. Peat can contain up to ten times its own weight of water and can shrink by 10 to > 80 % under loading. Under loading, primary consolidation can take place quite fast, creep however can occur for many years. For undrained peat deposits, the undrained shear strength is negligible. Only after consolidation some strength gaining can be expected. Also the decomposition of the organic material has an important influence on the possible volume decrease and deformation when loaded.

Also when dealing with **glacial soils** in the reclamation or subsoil, there are some risks associated. There is a great variety in glacial soils, but often two types are distinguished: glacial tills and fluvo-glacial soils. The difference is related to the way of deposition. Glacial tills are deposited directly by the ice and fluvo-glacial soils are deposited by meltwater near the ice front or further away.

Glacial till deposits are unsorted and unstratified units consisting of a mixture of clay, silt, sand, gravel, cobbles and boulders. The composition depends on the rocks which were eroded by the glacier. Locally, the unit can be very variable with zones of soft clay, sand lenses or large boulders. Often the term ‘boulder clay’ is generally applied, although this can be misleading since a till with sandy-gravelly matrix can have possibly almost no clay component. Terminal moraines may be structurally complex where glacial advance has pushed till into ridges with large boulder content (cf. Figure 7). Bearing capacity and compressibility may vary considerable over relatively short distances, depending on the local composition of the till. During site investigations which only consist of testing/sampling at discrete locations, care should be taken that isolated large boulders are not confused with rock layers.

Figure 7: Large boulders found inside the matrix of a glacial till deposit

Fluvo-glacial soils are deposited by streaming water and are therefore more sorted compared to glacial till deposits; the finest (silt-clay) fractions are flushed out and the largest boulders stay in place since they are too heavy to be transported by water. The deposits consist mainly of sand and gravel. The deposition by streaming water implies that these units can have (locally) relatively low relative densities, which may require ground improvement in order to avoid foundation problems.

3. SITE INVESTIGATION FOR MARINE GROUND IMPROVEMENT

3.1. Scope of the site investigation

For the modelling of soils and rocks in geotechnical engineering, a range of constitutive models exist ranging from rather simple to very complex. Discussion can arise whether a rather simple but approximate modelling strategy is better or worse than a very complex but more accurate modelling strategy. In anyway, for both strategies, it is critical to assess the appropriate parameters for the model under consideration. In general, 15 to 35 different physical and mechanical parameters are needed for characterizing the soil in geotechnical modelling. It is clear that the modelling strategy is of no importance as long as it cannot be guaranteed that these parameters can be obtained accurately and are fully representative for the soil behaviour in-situ and under the applied loading conditions. This justifies the need for a proper site investigation. Before the geotechnical engineer starts (preliminary) design calculations, it is essential to consider how representative and undisturbed both in-situ and laboratory testing has been performed.

In a marine environment, projects for which a geotechnical site investigation is required are often related to the developments of ports, breakwater construction, onshore construction on reclaimed land, localized foundation systems for e.g. wind turbines or oil rigs and specific long alignments such as channel dredging or cable laying projects. Except the dredging, soil improvement can be required for any of the above constructions. However, dredging material is very likely to be reused as reclamation fill and its characteristics are then required.

The principal scope of geotechnical site investigation for such projects is to determine the geotechnical parameters which are needed for e.g. pile design, foundation design, settlement and consolidation analysis, liquefaction susceptibility and slope stability calculations.

Site investigations over water are always more difficult and much more expensive than on land. Challenges related to marine site investigation are the specific environmental conditions during the investigation. Compared to investigations on land, there is now the influence of wind, currents, waves, water depth,…This often requires a change or modification of the generally known tools for onshore site investigations. It also requires a specialized team with high quality equipment in order to guarantee a high level of accuracy. Related to that, there is likely to be a major logistical problem involving personnel, materials and supplies. Furthermore, because of these high costs, it is very likely that mobilization will take place only once. Therefore, it is advised to considerate carefully the scope and requirements of the site investigation in advance.

3.2. Characteristics related to ground improvement

3.2.1. Typical near shore stratigraphic layers

In a nearshore environment, there are some typical stratigraphic layers which may be encountered. At the top, there is often the presence of a non-natural fill unit. This a heterogeneous soil volume consisting of sands, clays, silts, gravels and construction debris. This is often not suitable to rely on for foundations support. Consequently, this layer is often removed or requires ground improvement.

In the majority of port sites, there is regularly a typically loose, fine grained, sandy-silty layer at the top. At intermediate levels, this layer is often more dense. Ground improvement is possibly needed if these layers are implemented in the design.

Clay and silt units are generally considered as relatively low strength material. Occasionally, they are mixed with sand lenses and organic material. These units are also considered as relatively compressible. If overlain by new fill, they could settle and cause for example down drag on piles. Ground improvement could be a solution.

Overconsolidated clays and marls consist of hard, stiff clay. They possibly contain also sand and gravel lenses. They are considered to be relatively strong and moderately compressible.

Units consisting of dense and coarse sand are excellent for foundation support. They typically contain 0% - 20% clay or silt, often grouped in lenses. These units are considered to be relatively strong and incompressible.

If bedrock is present, this is also considered to be an excellent foundation support unit. If not extremely weathered, it is assumed to be strong and incompressible. However, a thorough investigation is required in order to determine its hardness, strength, degree of erosion and the presence of possible shear zones due to seismic activity.

3.2.2. What part is specific to soil improvement

Soil improvement is generally presented as an (more economic) alternative solution. As a result, appropriate (re)interpretation of soil factual data is required. The risk herein is that some soils are not properly characterized as they were supposed to be removed. In this case, specific additional site investigation may be required. Specific points of interest related to ground improvement are amongst others:

- Stratigraphy to define the depth of improvement
- Grain size distributions to select the appropriate soil improvement technique
- Density and strength to define the requirement for densification and target improvement
- Consolidation parameters, in horizontal and vertical direction, for the drainage path
- Void ratio to define the quantity of grouting
- Chemical contents and carbonate content for selecting the appropriate grout formula

3.3. Pitfalls

The aim of this paragraph is to indicate the most peculiar issues related to marine site investigation. For any site investigation, it is of extreme importance to be aware of the execution circumstances and the competence of the people on site. This can have a significant influence on the interpretation of the results by the geotechnical engineer and related, to the design of dredging works and nearshore constructions.

3.3.1. Executional aspects

A critical aspect in the execution of a marine site investigation is the availability of a stable working platform. If this cannot be foreseen, it will affect the accuracy of the entire site investigation, even if good quality drilling and testing equipment is foreseen. Related to platforms, the actual problem is that under the influence of waves, tides and possibly strong winds the fixed position of the platform and the verticality of the operations cannot be guaranteed. Therefore, it is recommended to execute nearshore site investigations from a stable jack-up platform (cf. Figure 8). The four spuds of the platform can assure a fixed position and the working platform can be jacked up until the working platform is not influenced anymore by the moving water surface. A jack-up platform is also relatively flexible; if testing at a particular location is finished, the platform is lowered onto the water surface, the spuds are jacked-up and the platform can be towed to the next position. Sometimes, the working platform is fixed on a tripod (cf. Figure 8). This also provides a stable platform, but is less flexible; the vertical position is fixed and moving to another location is less evident. Apart from the accuracy, another important issue is safety. If works are executed from a stable platform, the workmen can operate in a safe working environment. One could also use special drilling vessels and large sea bottom rigs, but these are generally only applied for projects in the oil and gas sector, situated in larger water depths compared to the smaller depths in which dredging and harbour projects are executed.

Figure 8: Stable working platforms for nearshore marine site investigations: jack-up platform Vagant (left) and monopole (right)

In nearshore site investigations it is often required to characterize very soft marine clay units at sea bottom level. If these units are investigated, it is recommended to make use of an anchor bottom plate and an appropriate set of casing and CPT-rod guiding systems (cf. Figure 9). The anchor bottom plate is connected to a 'raiser' casing system. This casing system is installed in order to minimize the influence of currents. The anchor bottom plate is anchored into the soft bottom for extra stability. This will guarantee the verticality during the test and prevents excessive disturbance of the upper meters of the soft subsoil. Inside this first casing system, the actual CPT-casings can be installed. Guiding flanges are installed to guarantee verticality of this second casing system inside the first casing system with anchor bottom plate. If this set-up is applied, it is important to realise that the bottom plate should be installed before mobilisation underneath the platform.

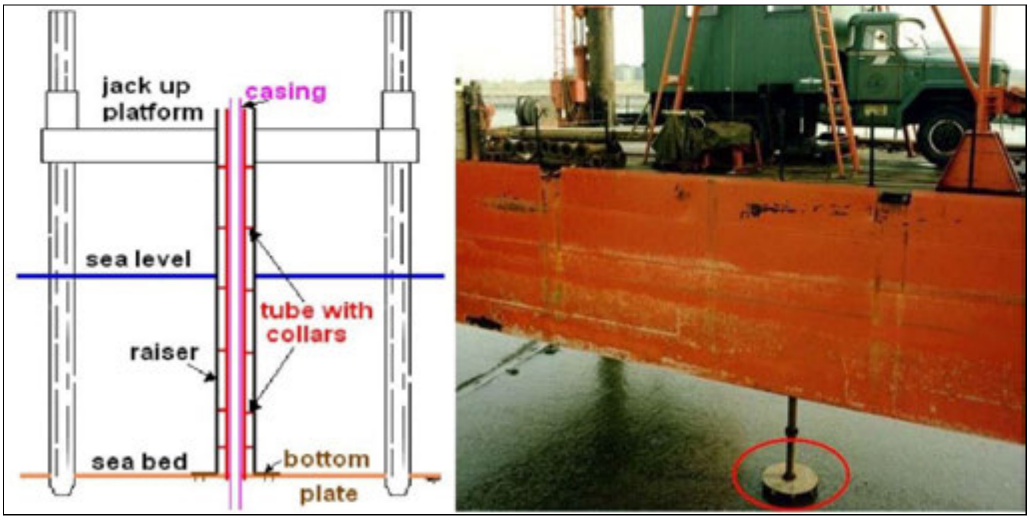


Figure 9: Set-up for reliable cone penetration tests from a jack-up platform in soft clays

If the subsurface contains mixed soils, consisting of gravels, cobbles and/or boulders it is essential to apply the proper drilling technique (cf. Figure 10). For example, if these types of soils are drilled by means of air-water flushing at very high flow rates with limited diameter size casings, the risk for misinterpretation is very large. The technique will give a good indication of the soil fraction in between ca. 1 cm to 10 cm, but other fractions will be missed. Not only will the flushing make the finer grained (matrix) fractions disappear, but due to the limited diameter casing, also the larger fractions will be missed. To get a representative sample, it is advised to drill with a diamond impregnated split double or triple tube core barrel. Although expensive and often resulting in broken core bits, it is a technique which can reveal more about the matrix and grain sizes of mixed soils. Another option to get a representative presentation of the fractions is to take bulk samples with a backhoe. However, even then it is in some particular situations difficult to get a full understanding of the situation. In moraine clay deposits for example, it is not unusual to find boulders up to 1m diameter and larger (cf. Figure 7). For these situations it is still very challenging to quantify the number of boulders and their spatial distribution within a particular soil volume.

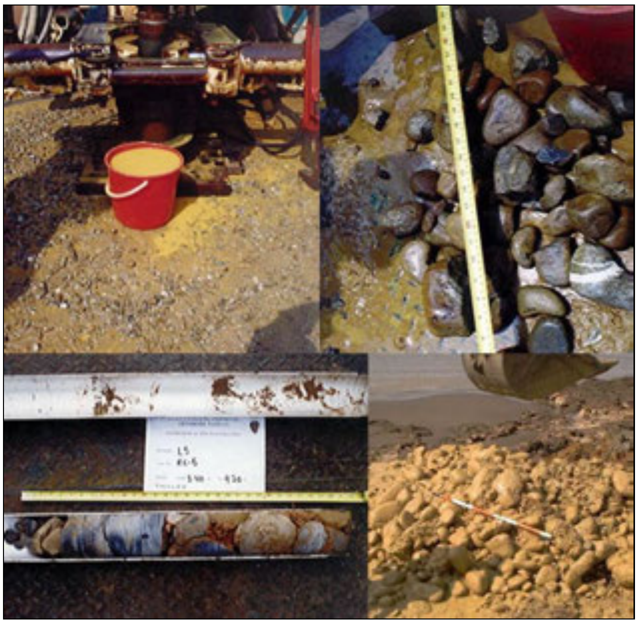


Figure 10: Sampling results in mixed soil unit by applying different techniques; air flush percussion drilling (left, top), air-water flushing at high flow rates in larger casings (right, top), diamond impregnated split double tube core barrel (left, bottom), sample with backhoe (right, bottom)

Unfortunately, in a lot of countries it is not always possible to get experienced people and high quality equipment at site. Often site investigations take place in remote and difficult accessible regions. It is not evident to execute a high quality site investigation in areas such as tropical jungles with dense vegetation, mangroves, deserts, artic regions,… In such situations, the mobilization of the right equipment and people can become very costly and time consuming and it is tempting for the financers of the site investigation to give up on quality (cf. Figure 11).

Figure 11: Examples of low quality equipment and testing/sampling procedures

Another critical step in the process is the storage and transportation of samples to the laboratory for further analysis. Even if the sampling itself is of good quality, this final step can still lead to inaccurate parameter determination and thus low quality design. If expensive (coring and/or piston tube) techniques are applied, it should be evident that the resulting samples are treated with care. First of all, all samples should be labelled correctly in order to assure their traceability. On site, they should be stored away from direct sun and wrapped in plastic foil. This will prevent drying out of the samples. If cored samples are transported in an inappropriate way, the risk of core breakage is high. Adapted core boxes can prevent this. One should realize that only well preserved samples will give representative laboratory testing results for geotechnical modelling, whether the applied models are simple or very complex.

3.3.2. Interpretation of the site investigation results

The quality of a site investigation is unmistakably related to the direct involvement of an engineering geologist/geotechnical engineer, both in the planning phase and during the actual execution of the site investigation. These people (should) have the proper background to situate the site in a broader geological context, define the relevant tests to execute and select high quality testing equipment. On site, they can also rapidly identify possibly misleading results. If not involved during the planning phase, presence at the site during the site investigation allows at least reporting and, if allowed, also reacting when the site investigation is executed with low accuracy. If no one is on site, the geotechnical engineer will eventually end up with a fancy report on his desk, without knowing the details behind it.

During a site investigation, it is important to be capable of situating the site in a broader geological context (cf. Fookes et al., 2001). A classic example is sampling and testing in weathered rock. It is not uncommon that rock formations completely weather into gravel-sand and even clay size fractions. If unqualified people are taking samples or executing tests in such an environment, the soil profile could easily be characterized as being soft and having relatively low strength, while a few (deci)meters deeper, solid rock can be present (cf. Figure 12). For dredging operations for example, this can lead to very low dredging production or even the mobilization of completely unsuitable equipment.

Figure 12: Soil profile with completely weathered rock (laterite, above dotted line) on top of (partly) weathered rock (below dotted line)

As already mentioned earlier, also the other opposite can occur when working in glacial tills. Due to their way of deposition, the occurrence of very large boulders, up to several meter diameter, is not uncommon in these units (cf. Figure 7). These may be confused with rock head during site investigation, with disastrous results in the light of for example piling operations.

And finally, even if high quality data is available, one still has to think how the soil will behave after dredging, transport, placement, treatment or ground improvement!

3.3.3. Contractor's dilemma

During the tender phase of a project, the strategy to follow for the contractor is not always evident. Often it occurs that a site investigation is provided to the contractor without knowing the details of the site investigation ('the good looking report'). Furthermore, in many cases it is impossible for the contractor to undertake further investigations due to time or financial restrictions. In such cases, the contractor must make best use of what is provided.

After interpretation by the contractor, it may turn out that the provided data suggest, but do not demonstrate, adverse conditions at one or more locations in the area of the project site. These adverse conditions possibly imply an alternate geotechnical approach and solution. What should the contractor do in such a situation; loose the work or take the risk? If these adverse conditions are priced, his solution is probably too expensive and he might lose the job. He could also qualify his offer, covering the potential exposure and hope it does not arise or claim it if it does.

In such a case it is advised that the contractor takes into account what is 'reasonably foreseeable' by an experienced contractor acting in good faith and adopting reasonable state of the art means. Therefore, the provided factual data should first of all be interpreted by an experienced geotechnical engineer/engineering geologist. Then, a geotechnical model should be developed which is conform the factual data at the investigated locations and which allows for some reasonable degree of variance between the investigated locations, based on a general appreciation of the geological environment. Unfortunately, what is reasonable remains a question of opinion; to what extent should the interpretation of the factual data allow for variations between investigated locations?

As a conclusion of this chapter, a reference can be made to the following popular catch phrase which summarizes the essence of this chapter: “At a certain point in time, you pay for a site investigation, whether you have one or not”.

4. GROUND IMPROVEMENT METHODS

Many ground improvement techniques exist and there are several ways to classify them:

- Distinction between techniques that have to applied from the ground surface and techniques which need to be executed from a certain depth
- Classification based on the type of material that can be improved by ground improvement techniques
- Distinction based on the behaviour of the ground to be improved and the use of admixtures

As an example three different classification methods are adopted here. First of all, the classification of the Technical Committee 211 of the ISSMGE is adopted in Table 2. This table gives an overview of all the existing techniques and classifies these techniques taking into account the behaviour of the ground to be treated and the use of admixtures.

Table 2: Classification of ground improvement methods as proposed by TC 211 (cf. Chu et al, 2009)

Category	Method	Principle
A. Ground improvement without admixtures in non-cohesive soils or fill materials	A1. Dynamic compaction	Densification of granular soil by dropping a heavy weight from air onto ground.
	A2. Vibrocompaction	Densification of granular soil using a vibratory probe inserted into ground.
	A3. Explosive compaction	Shock waves and vibrations are generated by blasting to cause granular soil ground to settle through liquefaction or compaction.
	A4. Electric pulse compaction	Densification of granular soil using the shock waves and energy generated by electric pulse under ultra-high voltage.
	A5. Surface compaction (including rapid impact compaction).	Compaction of fill or ground at the surface or shallow depth using a variety of compaction machines.
B. Ground improvement without admixtures in cohesive soils	B1. Replacement/displacement (including load reduction using light weight materials)	Remove bad soil by excavation or displacement and replace it by good soil or rocks. Some light weight materials may be used as backfill to reduce the load or earth pressure.
	B2. Preloading using fill (including the use of vertical drains)	Fill is applied and removed to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B3. Preloading using vacuum (including combined fill and vacuum)	Vacuum pressure of up to 90 kPa is used to pre-consolidate compressible soil so that its compressibility will be much reduced when future loads are applied.
	B4. Dynamic consolidation with enhanced drainage (including the use of vacuum)	Similar to dynamic compaction except vertical or horizontal drains (or together with vacuum) are used to dissipate pore pressures generated in soil

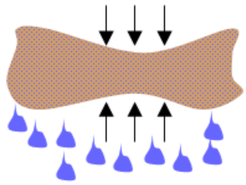
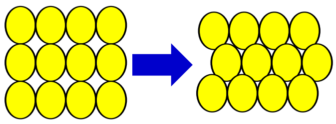
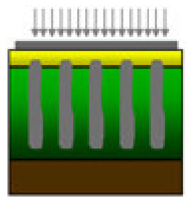
		during compaction.
	B5. Electro-osmosis or electro-kinetic consolidation	DC current causes water in soil or solutions to flow from anodes to cathodes which are installed in soil.
	B6. Thermal stabilisation using heating or freezing	Change the physical or mechanical properties of soil permanently or temporarily by heating or freezing the soil.
	B7. Hydro-blasting compaction	Collapsible soil (loess) is compacted by a combined wetting and deep explosion action along a borehole.
C. Ground improvement with admixtures or inclusions	C1. Vibro replacement or stone columns	Hole jetted into soft, fine-grained soil and back filled with densely compacted gravel or sand to form columns.
	C2. Dynamic replacement	Aggregates are driven into soil by high energy dynamic impact to form columns. The backfill can be either sand, gravel, stones or demolition debris.
	C3. Sand compaction piles	Sand is fed into ground through a casing pipe and compacted by either vibration, dynamic impact, or static excitation to form columns.
	C4. Geotextile confined columns	Sand is fed into a closed bottom geotextile lined cylindrical hole to form a column.
	C5. Rigid inclusions (or composite foundation, also see Table 5)	Use of piles, rigid or semi-rigid bodies or columns which are either premade or formed in-situ to strengthen soft ground.
	C6. Geosynthetic reinforced column or pile supported embankment	Use of piles, rigid or semi-rigid columns/inclusions and geosynthetic girds to enhance the stability and reduce the settlement of embankments.
	C7. Microbial methods	Use of microbial materials to modify soil to increase its strength or reduce its permeability.
	C8 Other methods	Unconventional methods, such as formation of sand piles using blasting and the use of bamboo, timber and other natural products.
D. Ground improvement with grouting type admixtures	D1. Particulate grouting	Grout granular soil or cavities or fissures in soil or rock by injecting cement or other particulate grouts to either increase the strength or reduce the permeability of soil or ground.
	D2. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate to either increase the strength or reduce the permeability of soil or ground.

	D3. Mixing methods (including premixing or deep mixing)	Treat the weak soil by mixing it with cement, lime, or other binders in-situ using a mixing machine or before placement
	D4. Jet grouting	High speed jets at depth erode the soil and inject grout to form columns or panels
	D5. Compaction grouting	Very stiff, mortar-like grout is injected into discrete soil zones and remains in a homogenous mass so as to densify loose soil or lift settled ground.
	D6. Compensation grouting	Medium to high viscosity particulate suspensions is injected into the ground between a subsurface excavation and a structure in order to negate or reduce settlement of the structure due to ongoing excavation.
E. Earth reinforcement	E1. Geosynthetics or mechanically stabilised earth (MSE)	Use of the tensile strength of various steel or geosynthetic materials to enhance the shear strength of soil and stability of roads, foundations, embankments, slopes, or retaining walls.
	E2. Ground anchors or soil nails	Use of the tensile strength of embedded nails or anchors to enhance the stability of slopes or retaining walls.
	E3. Biological methods using vegetation	Use of the roots of vegetation for stability of slopes.

A more recent classification is the one developed by the CUR/CIRIA and which is published in the hydraulic fill manual (cf. van 't Hoff and Nooy van der Kolff., 2012). The classification is indicated in Table 4. The classification focusses on methods which are often used in land reclamation works and takes into account both the field of application, depth of application and depth of influence.

Another simplified classification system has been established by Liausu. The ground improvement methods are classified in two broad categories: the first one gathers the methods which act directly on soil structure and the second one comprises the methods which strengthen the soil by incorporating inclusion (see Table 3).

Table 3 : Classification of ground improvement methods adopted by Liausu

	Direct action on soil structure **No material added**	**Reinforcement by inclusions** **Material added**
Granular Soils **(Sand, gravel, fill, etc…)**	1. Dynamic Compaction 2. Vibrocompaction 3. Rapid Impact Compaction 	***Inclusions of granular material*** 6. Dynamic Replacement 7. Stone Columns ***Rigid Inclusions*** 8. Pile like inclusions such as CMC's (Controlled Modulus Columns), VCC's
Cohesive Soils **(Clay, mud, peat, etc…)**	4. Consolidation by vertical drains + preloading 5. Vacuum Consolidation 	9. Jet Grouting columns 10. Soil Mixing Columns 

The first class consists in densifying the soil by a mechanical compaction, either a static type for cohesive soil (consolidation by Prefabricated Vertical Drains + preloading, vacuum consolidation) or a dynamic type for granular soil (Dynamic Compaction, Vibrocompaction, Rapid Impact Compaction). The second division aims to reinforce the soil by a grid of inclusions which overall gives better mechanical properties to the composite soil (soft soil + inclusion). The inclusion can be rigid, made of grouting material (CMC, Jet grouting Columns, Soil Mixing Columns), or flexible, made of well-compacted natural material (Dynamic Replacement pillars, Stone Columns). In the following sections of this chapter, it will be only reviewed ground improvement methods without admixture (methods 1 to 5 from Table 3) and the ones with granular admixtures (methods 6 to 7). Rigid inclusions and Soil Mixing methods are respectively the subject of symposium short courses 2 (Deep Mixing) and 3 (Rigid Inclusions & Soil Reinforcement) and thus are not dealt with in this text.

Table 4: Classification of ground improvement methods as proposed by CUR/CIRIA (cf. van 't Hoff and Nooy van der Kolff, 2012)

Method	Techniques	Soil Types	Application Depth	Treatment Depth	Suitable for		Improvement			
					Subsoil	Fill	Settlemen t behavior	Strength/ Stability	Liquefacti on	Drainage capacity
Consolidation	pre-loading with or without vertical drains	clay, peat, silt, but also compressible materials such as carbonate sands	drains at depth, surcharge at the surface (sand) or at depth (atmospheric pressure)	up to 30-60 m	X	X	X	X		X (PVD)
Compaction	Vibratory compaction techniques:									
	vibratory roller	granular material	at the surface	up to 0,5-1m		X	X	X	X	
	polygonal drum compactor	granular and cohesive materials	at the surface	up to 1,5-3m		X	X	X	X	
	vibroflotation	granular material (< 15% fines)	at depth	> 30 m	X	X	X	X	X	X
	vibratory probes		at depth	10 – 15 m	X	X	X	X	X	
	Dynamic compaction techniques									
	Dynamic Compaction	granular material	from the surface	up to 8 – 12 m	X	X	X	X	X	
	Rapid Impact Compaction	granular material	from the surface	up to 6 – 7 m	X	X	X	X	X	
	High Energy Impact Compaction	granular material	from the surface	up to 2 – 4 m		X	X	X	X	
Soil replacement	Soil removal and replacement	(Very) soft cohesive soil	From seabottom/surface	0 – 30 m	X		X	X		X
	stone columns	gravel, sand, silt and clay	at depth	20 – 30 m	X	X	X	X	X	X
	sand compaction piles	gravel, sand, silt and clay	at depth	20 – 30 m		X	X	X	X	X
	geotextile encased sand columns	clay, peat	at depth	typically 10 – 15 m	X	X	X	X		X
	dynamic replacement	gravel, sand, silt and clay	at depth	up to 6 - 7 m	X	X	X	X	X	X
	soil removal and replacement	all, mainly very soft soils	at the surface	n/a	X	X	X	X	X	X
Soil mixing	Admixtures (e.g. chalk and lime stabilization), in-situ soil mixing									
	Shallow Soil Mixing	sand, soft clay, silt and organic	both at depth and at the surface	≤ 12 m (SSM)	X	X	X	X	X	
	Deep Soil Mixing	sand, soft clay, silt and organic	both at depth and at the surface	3–50 m (DSM)	X	X	X	X	X	

4.1. Ground improvement without admixtures

4.1.1. In granular soils

4.1.1.1. Dynamic compaction

i. Principle and operation

Invented and developed by Louis MENARD, the Dynamic Compaction (DC) technique consists in pounding the ground by means of a heavy metallic weight (usually between 10 and 40 tons) in order to compact it at depth. The free or quasi-free drop of the pounder from a height of 10 to 30 meters produces a high energy impact which generates vibration waves through the ground (as shown in Figure 13). The compression P-waves induce a pore pressure increase and thus dislocate soil matrix. Then, the shear S-waves and surface Rayleigh-waves shear soil particles and rearrange the soil grains in a denser state by decreasing the voids into the soil. The reiteration of successive impacts at a same point entails a driving of the pounder through the ground and a soil compression. This results onto the surface in a crater named "print". The hoisting equipment is a modified crawler crane weighting 80 to 120 tons.

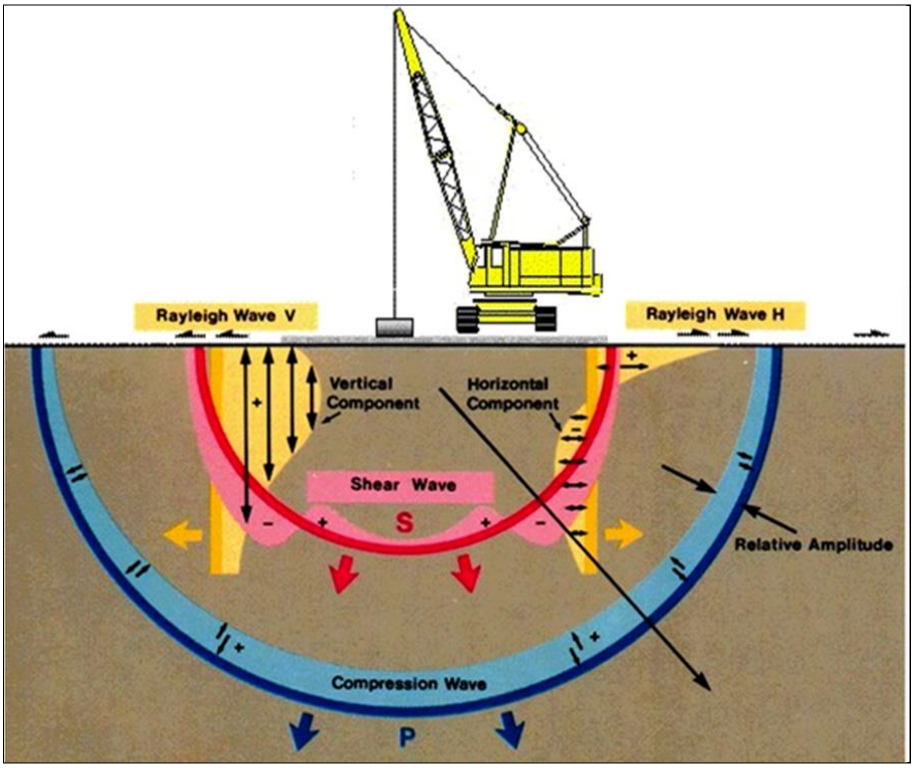


Figure 13 : Dynamic Compaction method

The execution of DC generally needs several phases in order to reach the required design criteria. The typical phasing is the following: the first phase is a phase of high energy and coarse drop grid which aims to compact deep layers (as illustrated in Figure 14), the second one is a phase of intermediate energy and intermediate density falls seeking to compact middle layers and the last one is a ironing phase of low energy and fine drop grid. Energy is thus transferred to the soil in phases and grids.

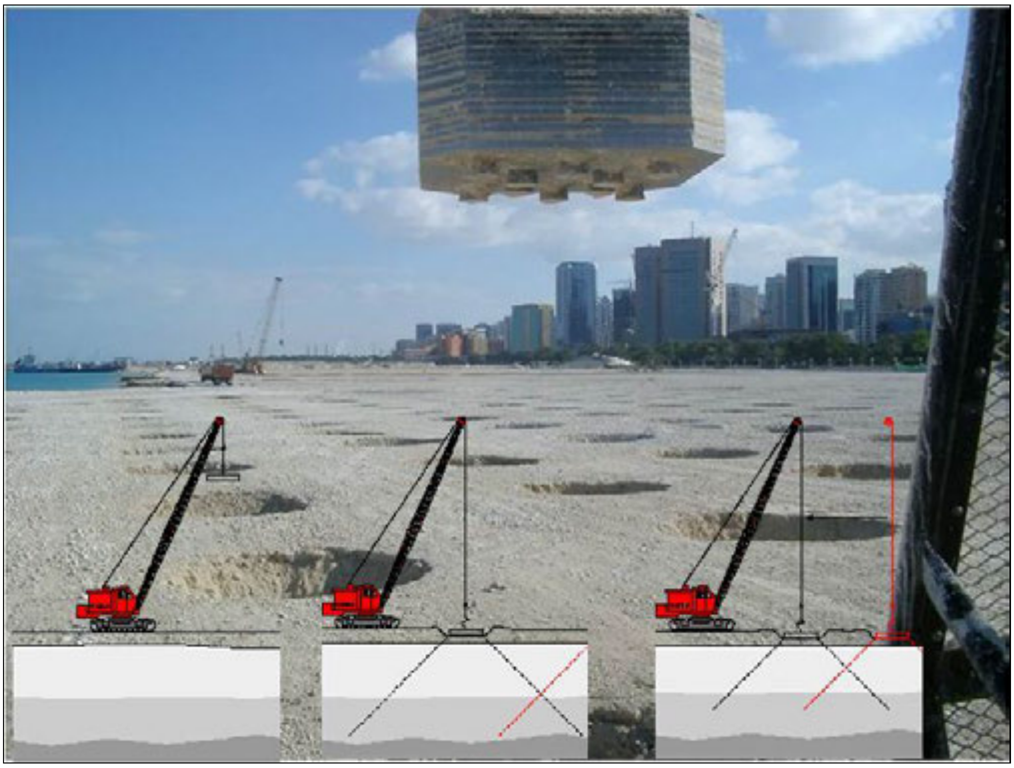


Figure 14: Carrying out of Dynamic Compaction (Menard Group).

The unit energy E_u is the product obtained by multiplying the pounder weight M and the drop height H and is usually given in tons meter.

$$E_u = M \times H \text{ (t.m)} \tag{1}$$

Many studies converge to show that the depth of influence D is related to the square root of the unit energy as per the empirical formula:

$$D = c \times \alpha \times \sqrt{E_u} \text{ (m)} \tag{2}$$

Where: c depends on the type of drop. c = 0.9 for cable fall and c = 1 for free fall. α is a correction factor. α = 0.5 for heterogeneous fills and α = 0.7 for granular materials
The Table 5 summarizes for several unit energy given the practical depth limits of DC. The higher the energy is, the deeper the influence of DC becomes.

Table 5: Practical depth limits of DC

Unit energy	Depth of influence
200 t x m	5/6 m
300 t x m	7/8 m
400 t x m	9/10 m
600 t x m	11/12 m

The intensity or total energy E_t is the product obtained by multiplying the unit energy and the number of blows N, overall reduced to the treated surface S. This is usually expressed in tons meters per squared meters.

$$E_t = \frac{E_u \times N}{S} \text{ (t.m/m²)} \tag{3}$$

If the unit energy enables to determine the depth of influence of DC, the total energy is a indicative of the improvement factor of the treated soil.
For very High Energy Dynamic Compaction (HEDC), specific equipment (pounder weight > 30 tons, crane weight > 120 tons, drop height > 30 m) is involved to compact very deep layers (> 10 to 12 m below natural ground level). Some systems which allow reaching an unit energy more than 600 t.m have been developed, in particular the complete free fall of the weight thanks to the use of a weight release system. After a phase of equal acceleration, the clamping device releases the weight without any line attached. Hence, there are no friction and damping from cables and winches. Following the impact, the device grabs the pounder and lifts it up to the top of the crane. High Energy Dynamic Compaction generates wide deep prints as illustrated in Figure 15 from A72 project in Germany. Print is 6 m deep and 3 m diameter. This represents a significant volume of more than 40 m^3.

Figure 15: Very High Energy Dynamic Compaction for collapse of voids in the ground from A72 project (Germany, Menard Group)

ii. Application field

DC is very effective in granular soils and anthropogenic fill, comprised of inert heterogeneous material. DC can't densify the cohesive soil (clay, peat) because no compression occurs under the impact but rather swelling. It follows that vibration waves have no effect on soil properties. DC can be applied in both saturated and unsaturated soils. Above the limits presented in Table 6, Dynamic Compaction can't be feasible and a solution of Dynamic Replacement is more appropriate.

Table 6 : Limits of soil suitable for DC

Tests	***Criteria***	
	Saturated soil	***Unsaturated soil***
Particle size distribution	Passing through 80 µm < 30 % Passing through 2 µm < 3 %	
Atterberg limits	$I_P < 12$	
CPT	FR < 1.5% $q_c > 3$ MPa	
Proctor	-	$w < w_{OPN}$

iii. Case study

Two case studies combined with other methods are described in sections 4.3.1 and 4.3.2.

4.1.1.2. Vibrocompaction

i. Principle and operation

The VibroCompaction (VC), or vibroflotation, technique consists in sinking a big cylindrical vibrating probe (weighting 15 to 40 kN, with a diameter of 250 to 500 mm and a length of 2 to 5 m) in the ground as per a well-defined grid of compaction points mostly triangular for the treatment to be the most uniform as possible. The implementation of the VC procedure is shown in Figure 16. Jetted water under pressure is injected to the tip of the vibrating probe in order to make easier the penetration of the tool at the depth expected. The vibrations combined with a provision of water lead to a local liquefaction, a rearrangement of soil particles in a denser state and a settlement. The vibrating probe is lifted up gradually in successive passes meanwhile creating around the tool a cylinder/cone of compacted soil. The tightening of the deeply grains (related to void ratio decrease) is reflected at the surface by a cone of subsidence (breaking shear) around the treated point. The cone is eventually filled of material in order to compensate the settlement.

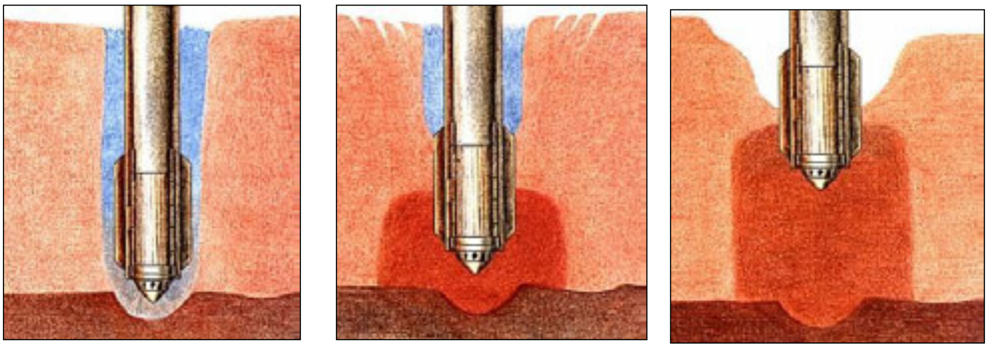


Figure 16 : Vibrocompaction procedure (Vibroflotation group)

The VC method enables the soil to decrease the void ratio and the permeability as well as to increase the friction angle, the stiffness modulus and the relative density. The equipment is a modified crawler crane which suspends the vibrator. Dual or triple vibrators can be used for compaction (see Figure 17). Zhou & al. (2008) have mentioned in these systems benefits of interaction or possible resonance effect.

Figure 17 : Vibrocompaction with vibrators in tandem configuration (Keller group).

With these techniques of soil improvement, the depth is perfectly mastered. The VC can be applied between depths of 3 to 65 m. The method is more compatible for high depths and generates less impact on environment than dynamic compaction. The grid of VC usually varies from 2 m x 2 m to 5.5 m x 5.5 m. The main expected results of this soil improvement in clean sand are:

- A minimal relative density of 60%(usually in a range of 60% to 80% reached);
- A minimal cone resistance of between 10 and 15 MPa;
- A void closure reaching 5% to 15%;
- The elimination of the risk of soil liquefaction during an earthquake causing a ground acceleration in the soil;
- An allowable bearing capacity
- A reduction of residual settlement.

As reminder, the liquefaction potential is defined by the factor of safety FS given by the following formulas:

$$FS = \frac{CRR_{Mw}}{CSR} = \frac{CRR_{7.5} \times MSF}{CSR} \tag{4}$$

Where CRR_{Mw} is the Cyclic Resistance Ratio generated by earthquake shaking with a magnitude of Mw, CSR is the Cyclic Stress Ratio, $CRR_{7.5}$ is the Cyclic Resistance Ratio generated by earthquake shaking with a magnitude of 7.5 and MSF is the Magnitude Scaling factor. Liquefaction occurs when $FS \leq 1$. The targeted FS is usually 1.25 (Eurocode 8).
The CSR corresponds to the seismic demand on a soil layer and can be calculated by the following equation formulated by Seed & Idriss (1971):

$$CSR = \frac{\tau_h}{\sigma'_{v0}} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_{v0}}{\sigma'_{vo}} \cdot r_d \tag{5}$$

Where τ_h is the seismic shear stress, σ_{v0} is the total vertical stress, σ'_{v0} is the effective vertical stress, g is the acceleration of the earth, 0.65 is the average acceleration with reference to a_{max}, a_{max} is the maximal earthquake acceleration, r_d is the reduction factor depending on depth and z is the depth.

ii. Application field

The vibrocompaction method enables to compact loose soil or backfill material by vibrations only. This technique is adapted for non-cohesive granular soils and some slightly cohesive soils. The VC procedure is very effective in relatively clean sand.

To maximize the compaction, the soils subjected to the vibrocompaction have to be a specific grading:
- the proportion of fine particles (≤ 80 µm) must be less than 10-12% and clay (≤ 2 µm) less than 2%;
- the bigger elements must not exceed centimetric grains in order to avoid the blocking and the refusal of the vibrating probe in blocks.
- carbonates content or shells content should be as low as possible because of their crushability under the penetration of the vibrator into the soil.

Massarch and Heppel (1991) proposed to assess the suitability of soils for vibratory compaction based on CPT results (cone resistance and friction sleeve measurements), as illustrated in Figure 18

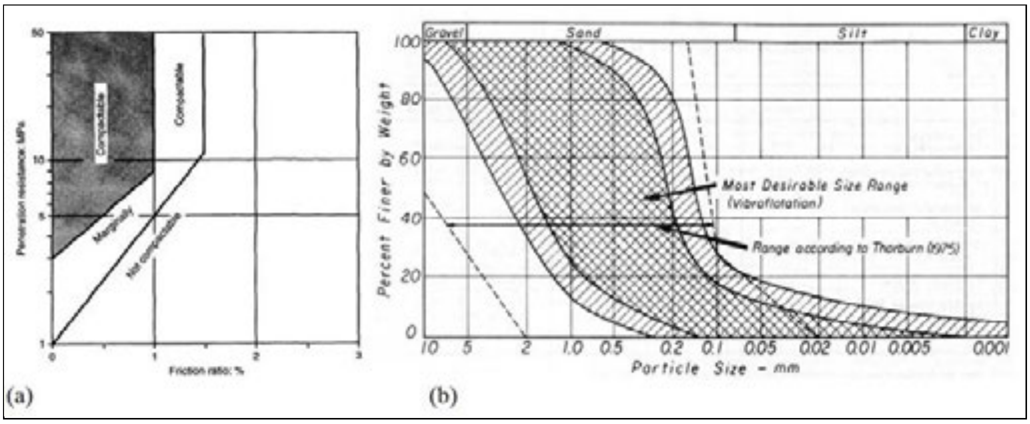


Figure 18: Classification of soil adapted to vibrocompaction based on CPT (after Massarch and Heppel 1991) (a) and grain size distribution (b)

The zone of compactable soil can be superimposed with the CPT soil classification diagram defined by Robertson and al. (1986). The suitability for vibroflotation can be also evaluated by a grain size distribution analysis. The presence of silt/clay pockets can lead to a significant reduction of the vibrocompaction efficiency in this layer as well as in layers above and below (up to 1 m).

iii. Case study

A case studied combined with DC method is described in section 4.3.1 and 4.3.2.

4.1.1.3. Rapid Impact Compaction

i. Principle and operation

The Rapid Impact Compaction was developed in the decade 1990's by BSP (UK) for the repair of bombed runways. This company commercialized the system and developed compactors with increasing weight from 7 to 9 and 12 tons drop weight in order to increase depth of influence. The compaction experiences were very good; however the used machines required major improvements and had a lot of breakdowns. In 2006, Cofra/Boskalis started to develop its own equipment (see Figure 19). Others followed later. Many types of equipment are available presently. The majority of compactors is provided with 7-9 tons weight and the largest ones with 16 tons weight.

Figure 19: Rapid Impact Compaction equipment (Cofra Group)

The principle of Rapid Impact Compaction (RIC) technique is similar as for Dynamic Compaction technique. The RIC technique consists in dropping freely a weight (between 7 and 16 tons) from a height of maximum 1.2 meters onto a circular foot assembly (diameter of about 1.5 m) which remains in contact with the ground. The equipment is a hydraulic hammer mounted on a crawler excavator which is easy and quick to set up compared with the DC crane. The soil is compacted by impact at a fast blow rate, 40 to 60 blows per minute, depending on the drop height and an unit energy of 6 to 18 t.m. The reiteration of successive impacts entails the penetration of the foot into the soil and then a compaction hole remaining after compaction (as shown in Figure 20). Four foot diameters are available: 1.0 m, 1.5 m, 2.0 m and 2.6 m for different soil conditions.

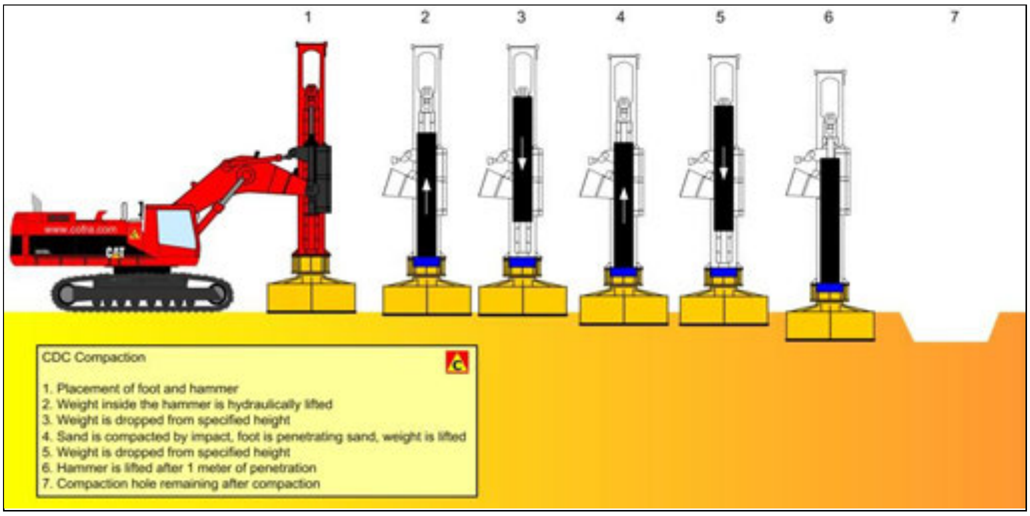


Figure 20 : Rapid Impact Compaction procedure (Cofra Group)

The RIC enables to densify the shallow soil layers up to 8 meters (typically at least 4 to 5 m) below the surface by high vibrations causing the temporary cancellation or exceeding of the friction between particles, resulting in a rearrangement. The depth of influence depends on: energy levels in relation with the shear strength between the particle, intersecting layers, compaction requirements and weight of the hammer. Depending on hydraulic conditions, the compaction is generated by different type of vibration waves (as illustrated in Figure 21):

- by shear waves and Rayleigh waves above water level;
- by compression and shear waves below water table.

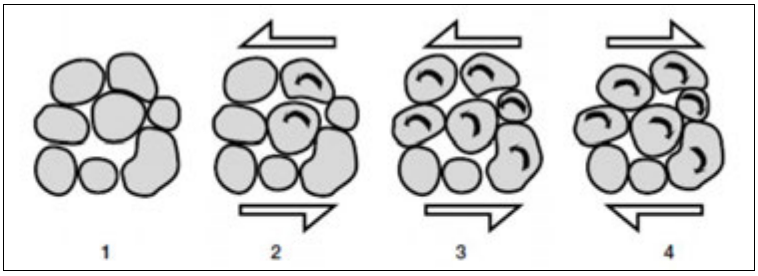


Figure 21 : Soil grain rearrangement during compaction

The grid pattern is determined on the basis of the contract requirements and soil conditions. Several compaction phases can be applied to meet the criteria. The more overlapping the grid is, the denser the compaction becomes.

For Rapid Impact Compaction technique, the theory is more complex. Thus, unlike Dynamic Compaction technique, there is no proven theory which shows a relationship between the unit energy of compaction and the depth of influence. For this reason, a method was proposed by Berry and Narendranathan (2010) based on Momentum and research by Oshima and Takada (1997). Similar work was undertaken by Vink (2012) which predicts compaction levels and depth of influence.

ii. Application field

The Rapid Impact Compaction is used to consolidate granular soils (gravel, sands), some silts, miscellaneous soil material and anthropogenic fills. The depth of influence can be limited to intersecting silt layers.

4.1.2. In cohesive soils

4.1.2.1 Consolidation by prefabricated vertical drains and preloading

i. Principle and operation

The consolidation by Prefabricated Vertical Drains (PVD) and preloading is a mechanical compaction static type. This consists in applying onto the soil a temporary surcharge embankment which must be combined with the installation of vertical drains into the soils, as shown in Figure 22. A hollow steel mandrel containing the PVDs material is driven down to the compressible layer base (up to depths of more than 50 m). The mandrel is attached to a mast mounted on a crawler excavator, as illustrated in Figure 23, a.

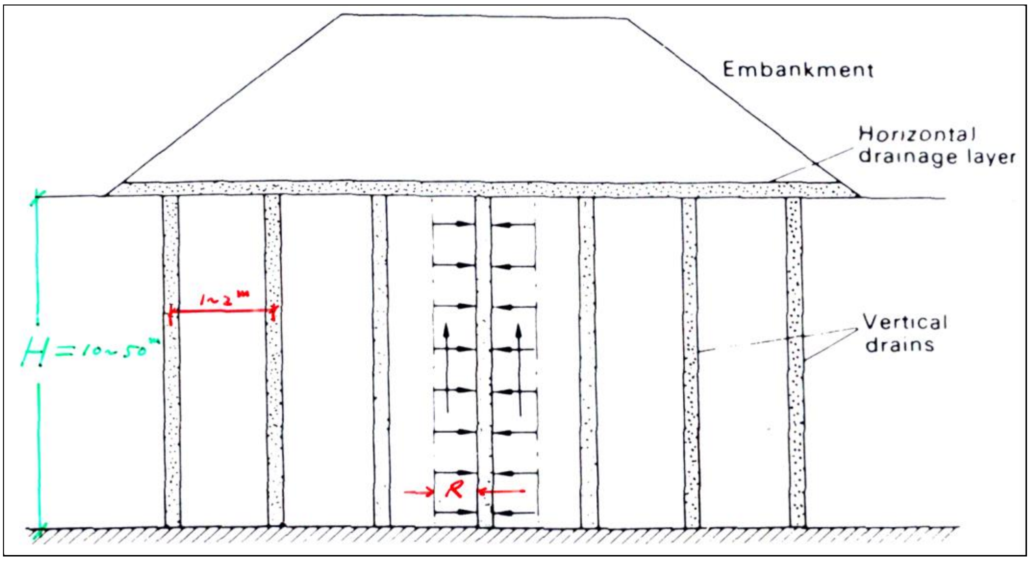


Figure 22 : Preloading using vertical drains method

Once the prescribed depth or refusal is reached, the drain is anchored in the soil by a steel plate while the mandrel is pulled out. Then, a draining platform is installed on the ground and the preloading embankment is raised (see Figure 23, b) in one or several phases.

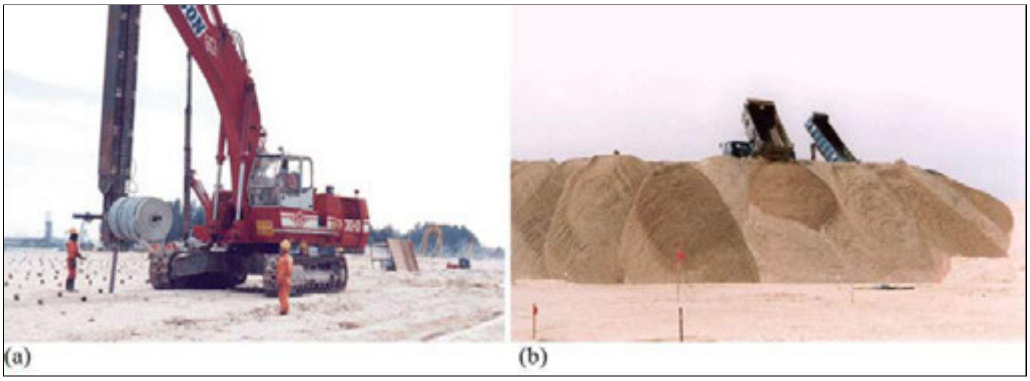


Figure 23: PVD equipment (a) and placement of preloading fill (b)

PVDs are cut about 15 to 20 cm above the working platform and are positioned as per a well-defined grid pattern adjusted to match the consolidation time. The PVDs are made up of a flexible plastic core with nonwoven geoxtextile filter sleeves and their shape can be either wick or cylindrical (see Figure 24).

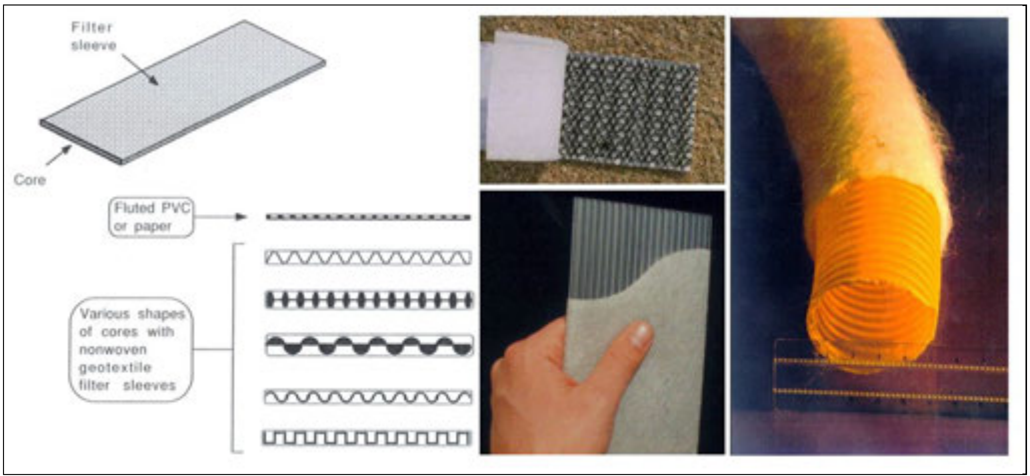


Figure 24: Drain material

Consolidation occurs very slowly (up to many years) in fine-grained soils because of their low permeability and great length of vertical path, slowing the expulsion of water. PVDs bring to the soil lateral drainage paths. The purpose of the PVDs is to shorten the length of the drainage path in the soil which allows highly accelerating the dissipation of excess pore pressures in the ground (evacuation of pore water) and thus the consolidation settlement, both generated by the preloading. Indeed, the increase of total vertical stresses into a cohesive soil is reflected at short term by an equivalent increase of pore pressures. Over time, the evacuation of pore water leads to a reduction of the void ratio (namely consolidation) and an increase of effective vertical stress. At long term, the excess pore pressures are fully dissipated and the increase of effective vertical stresses is equal to the increase of total vertical stresses. Soil consolidation using PVDs increases speed of consolidation and hence lowers consolidation time from years to months. This benefit is shown in Figure 25.

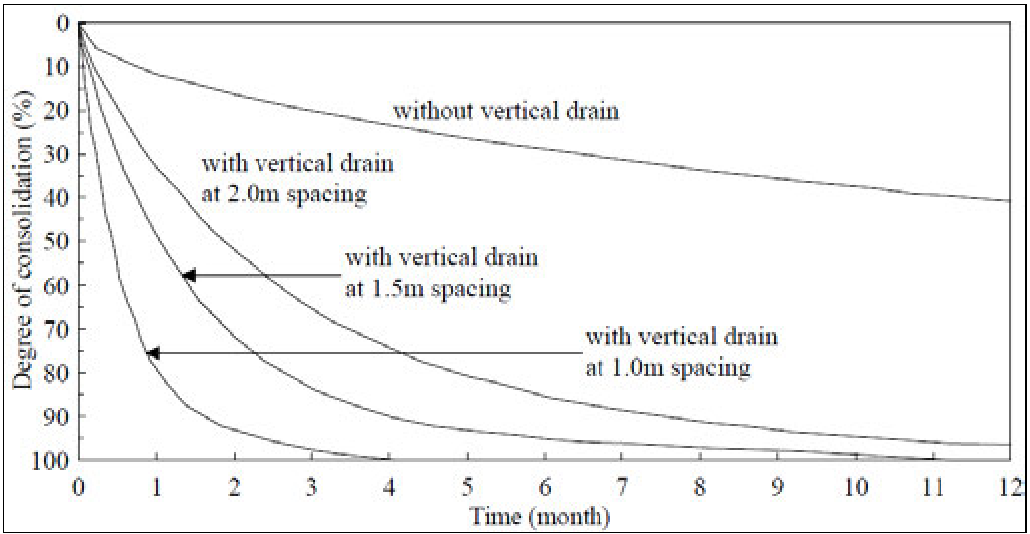


Figure 25 : Potential benefit of vertical drains (after Sathananthan, 2005)

The preloading aims to apply a loading equal to the future structure loading and eventually an additional surcharge (“pre-ageing”). The preloading enables to:
- consume partially or fully primary consolidation settlement, hence reduce residual absolute and differential settlement during the service life;
- anticipate a variable portion of secondary consolidation (creep) settlement;
- increase undrained shear strength in the ground;
- decrease the consolidation period required to reach the settlement.

In some cases, the bearing capacity of the soil does not ensure the stability of the whole preloading fill. The preloading fill should be raised in layers, each layer installation separated by a consolidation period. The number of raising phases and the thickness of the different fill layers are determined and adjusted by a slope stability analysis in which the potential risk of slope failure depends on the undrained shear strength in the soil. A consolidation period leads to a gain of undrained shear strength due to drainage, which can ensure the bearing capacity of the next fill layer. The consolidation rate to reach at the end of a consolidation period exceeds usually 80%.

The one-dimensional consolidation theory has been developed by Terzaghi (1925) and needs basic assumptions including:
- Deformations and flow occur only in vertical direction;
- Compressible layer is homogeneous, isotropic and saturated;
- Model is infinite in horizontal direction;
- Darcy's law is valid;
- Water and soil grains are incompressible;
- Coefficient of compressibility and permeability are constant;
- Deformations follow small strain theory;
- The relationship between effective vertical stress and volume variation (void ratio) is linear.

The one-dimensional consolidation theory is given by the differential equation as follows:

$$\frac{\partial u_e}{\partial t} = c_v \times \frac{\partial^2 u_e}{\partial^2 t} \qquad \text{with } c_v = \frac{k_v}{\gamma_w \times m_v} = \left(\frac{2.3 \times \sigma'_v \times k_v}{\gamma_w}\right)\left(\frac{1+e}{C_c}\right) \tag{6}$$

Where: u_e is the excess pore pressure, t is the elapsed time, z is the vertical distance below the ground surface, c_v is the coefficient of vertical consolidation, k_v is the vertical permeability, γ_w is the unit weight of water, m_v is the coefficient volume compressibility, σ'_v is the vertical effective stress, e is the void ratio and C_c is the compression index. The resolution of this issue leads to the definition of a dimensionless time factor T_v as per the following formula:

$$T_v = \frac{c_v}{{H_{dr}}^2} \times t \tag{7}$$

Where: H_{dr} is the drainage distance. By definition, the consolidation rate U is expressed as the following ratio:

$$U(t) = \frac{\overline{\Delta\sigma'_v(t)}}{\Delta\sigma_v} = 1 - \frac{\overline{u_e(t)}}{\Delta\sigma_v} \tag{8}$$

Where: $\overline{\Delta\sigma'_v(t)}$ is the average increase of effective vertical stress, $\overline{u_e(t)}$ is the average excess pore pressure at a given time t and $\Delta\sigma_v$ is the final increase of effective vertical stress (equal to the initial excess pore pressure, itself equivalent to the surcharge load applied on the soil). As is assumed a linear relationship between variation of effective vertical stress and variation of void ratio, the vertical consolidation rate U_v is also equal to the ratio:

$$U_v(t) = \frac{S(t)}{S_\infty} \tag{9}$$

Where: S(t) is the consolidation settlement at a given time t and S_∞ is the infinite consolidation settlement. The relationship between the average vertical consolidation rate and the time factor can be approximated by the following equation:

$$U_v(t) \approx \frac{\sqrt{4 \times T_v / \pi}}{\left[1 + (4 \times T_v/\pi)^{2.8}\right]^{0.179}} \qquad \text{for } 0 \le U_v \le 100\% \tag{10}$$

The previous equation can be represented by the following empirical equations:

$$T_v = \frac{\pi}{4} \times U_v \quad \text{for } U_v < 0.6 \quad \text{and} \quad T_v = -0.933 \times \log(1 - U_v) - 0.085 \quad \text{for } U_v > 0.6 \tag{11}$$

The pure radial drainage theory has been developed by Barron (1948) and is given by the differential equation as follows:

$$\frac{\partial u_e}{\partial t} = c_r \times \left(\frac{\partial^2 u_e}{\partial^2 r} + \frac{1}{r} \times \frac{\partial u_e}{\partial r} \right) \qquad \text{with } c_r = \frac{k_h}{\gamma_w \times m_v} = c_v \times \frac{k_h}{k_v} \tag{12}$$

Where: r is the radial distance from drains, c_r is the coefficient of radial consolidation and k_h is the horizontal permeability. The resolution of this issue leads to the definition of a dimensionless time factor T_r as per the following formula:

$$T_r = \frac{c_r}{{d_e}^2} \times t \tag{13}$$

Where: d_e is the equivalent diameter of cylinder of soil around drain (see Figure 26).

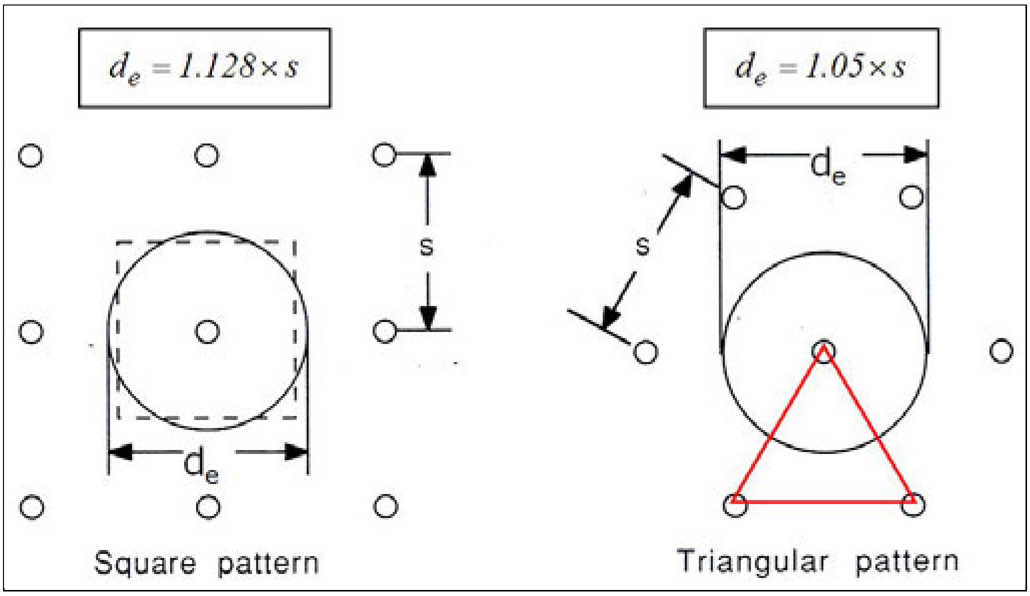


Figure 26 : Diameter of the equivalent soil cylinder tributary to a vertical drain

The relationship between the average radial consolidation rate Ur and the time factor T_r can be given by the following equation (Hansbo's theory, 1981):

$$U_r(t) = 1 - \exp\left[\frac{-8 \times T_r}{F(n)}\right] \qquad \text{with} \qquad F(n) = \frac{n^2}{n^2 - 1} \times \ln(n) - \frac{3n^2 - 1}{4n^2} \approx \ln(n) - 0.75 \tag{14}$$

$$n = \frac{d_e}{d_w}$$

Where: n is the drainage zone ratio and d_w is the equivalent diameter of drain (see Figure 27).

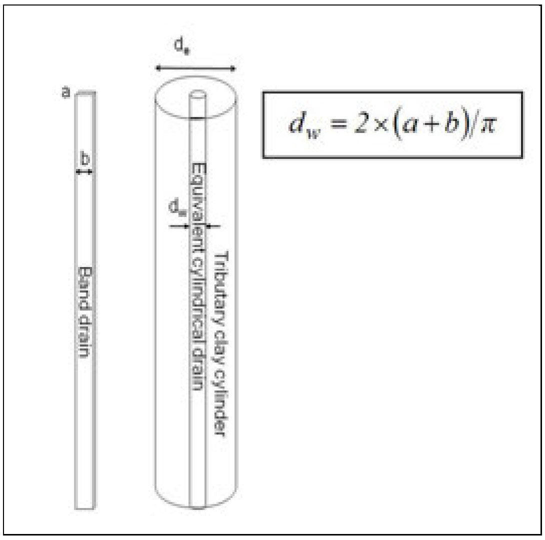


Figure 27 : Diameter of the equivalent cylindrical drain core (after Hansbo, 1979)

In practice, the excess pore pressure ratios are calculated separately based on vertical flow and radial flow alone, and then combined using Carrillo's equation:

$$(1-U_{vr})=(1-U_v)\times(1-U_r) \tag{15}$$

Two events can disturb the performance of drain: the smear effect and the well resistance effect. The smear effect corresponds to a soil remolding which can occur around the vertical drain when the latter is installed by a steel mandrel. Within a annular zone (named smear zone) of diameter d_s (see Figure 28), the remolded soil has a horizontal permeability k_s which is lower than the horizontal permeability k_h of the undisturbed soil. Thus, the smear effect delays the consolidation rate. The well resistance effect corresponds to the apparition of drain resistance to the water flow when the discharge capacity is reached. This well resistance is accompanied by a deterioration of the drain filter (reduction of the cross section) and a clogging of the drain by fine-grained particles. The influence of smear and well resistance effects can be taken into account by additional factors in the calculation of the factor F:

$$F = F(n) + F_s + F_r \qquad \text{with} \qquad F_s = \left(\frac{k_h}{k_s} - 1\right)\times ln(s) \text{ and } s = \frac{d_s}{d_w} \tag{16}$$

$$F_r = \pi \times \left(2L \times z - z^2\right) \times \frac{k_h}{q_w}$$

Where: F(n) is the factor expressing the effect due to the spacing of the drains, F_s is the factor expressing the smear effect, F_r is the factor expressing well-resistance effect, s is the smear zone ratio, L is the length of the drain having one-way drainage and half this value for two way drainage, z is the depth of the drain and q_w is the discharge capacity of the drain.

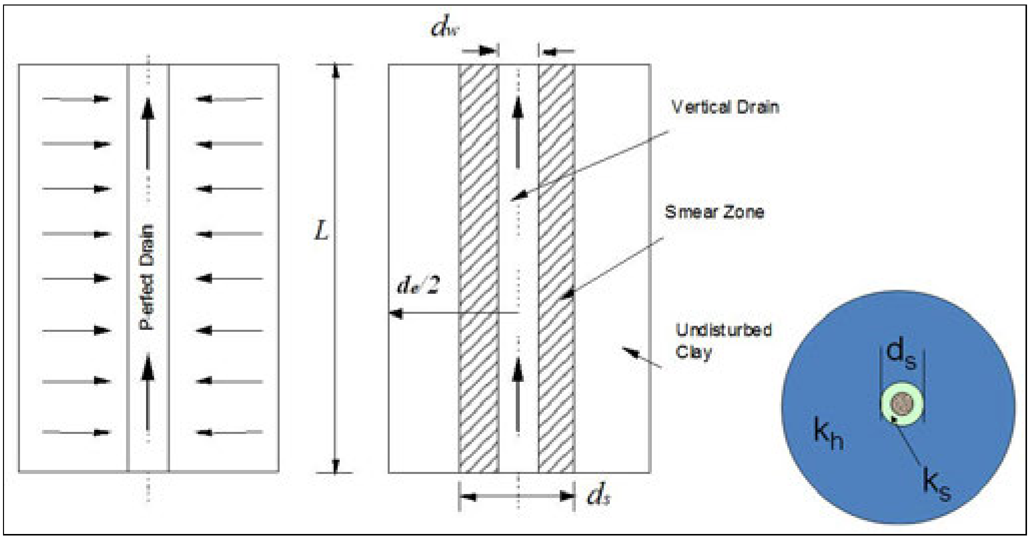


Figure 28 : Smear effect

Hird & al. (1992) and Indraratna & al. (2000 & 2005) converted the vertical drain system from a 2D axisymmetric model (unit cell) into a 2D plane strain model by adjusting the soil permeability, as shown in Figure 29.

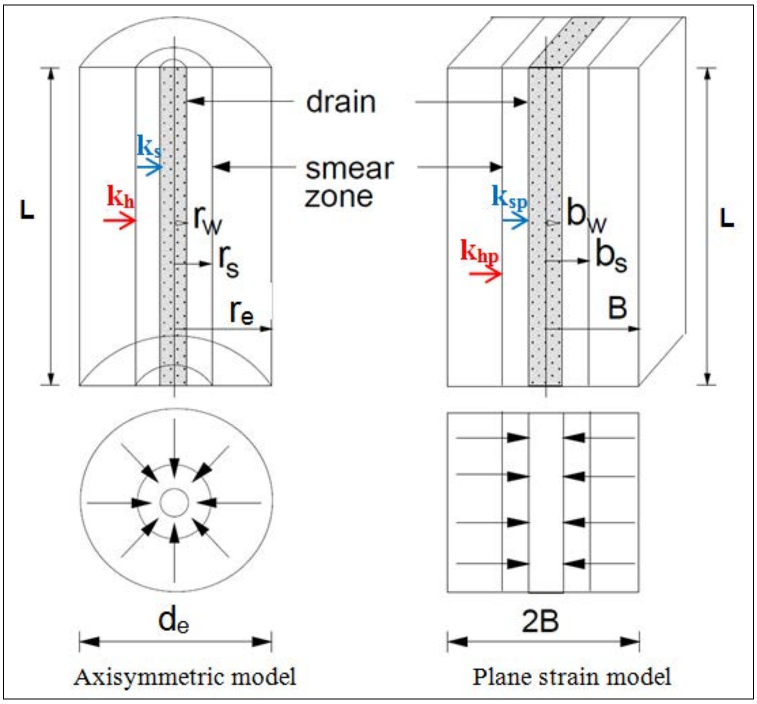


Figure 29 : Conversion of an axisymmetric model into a plane strain model (after Indraratna and Redana, 1997).

The relationship between the average radial consolidation rate U_{rp} and the time factor T_{rp} can be given by the following equation:

$$U_{rp}(t) = 1 - exp\left[\frac{-8 \times T_{rp}}{F_p}\right] \quad \text{with } F_p = \left[\alpha + \beta \times \frac{k_{hp}}{k_{sp}} + \theta \times \left(2L \times z - z^2\right)\right] \qquad (17)$$

Geometric parameters: $$\alpha = \frac{2}{3} - \frac{2b_s}{B} \times \left(1 - \frac{b_s}{B} + \frac{b_s^{\,2}}{3B^2}\right)$$

$$\beta = \frac{1}{B^2} \times (b_s - b_w)^2 + \frac{b_s}{3B^3} \times \left(3b_w{}^2 - b_s{}^2\right)^2$$

Flow parameter: $$\theta = \frac{2 \times k_{hp}{}^2}{k_{sp} \times B \times q_{wp}} \times \left(1 - \frac{b_w}{B}\right)$$

Where: k_{hp} is the equivalent plane strain undisturbed permeability, k_{sp} is the equivalent plane strain smear zone permeability, b_w is the half width of the drain, b_s is the half width of the smear zone, B is the half width of plane strain cell and q_{wp} is the equivalent plane strain discharge capacity of the drain. Indraratna and Redana (1997) showed that if d_e were considered equal to 2B (see Figure 29), then the relationship between k_{hp} and k_{sp} is given by:

$$k_{hp} = \frac{k_h \times \left[\alpha + \beta \times \frac{k_{hp}}{k_{sp}} + \theta \times \left(2L \times z - z^2\right)\right]}{\left[ln\left(\frac{n}{s}\right) + \frac{k_h}{k_s} \times ln(s) - 0.75 + \pi \times \left(2l \times z - z^2\right) \times \frac{k_h}{q_w}\right]} \tag{18}$$

If well resistance effect is ignored, the influence of smear effect can be modelled by the ratio of plain strain smear zone permeability to undisturbed permeability, as follows (after Indraratna & al., 2000 & 2005):

$$\frac{k_{sp}}{k_{hp}} = \frac{\beta}{\frac{k_{hp}}{k_h} \times \left[ln\left(\frac{n}{s}\right) + \frac{k_h}{k_s} \times ln(s) - 0.75\right] - \alpha} \tag{19}$$

If smear and well resistance effects are ignored (after Hird & al., 1992):

$$\frac{k_{hp}}{k_h} = \frac{0.67}{ln(n) - 0.75} \tag{20}$$

ii. Application field

Vertical drains are used to consolidate quasi-saturated (degree of water saturation more than 80%) to saturated cohesive soft soil with low permeability such as silt and clay deposits.

iii. Case study

For dredging and reclamation works in Pulau Bunting, Malaysia PVD's were installed in multiple phases, in combination with surcharge installation (cf. Figure 30). For the construction of a power plant, land needed to be reclaimed. In order to meet the requirements put forward by the client, rapid consolidation both beneath the reclamation area itself as beneath the bordering revetment slopes was necessary. The seabed level, which was originally at -3 m CD, was in a first stage raised to -1 m CD. A first series of vertical drains, with a spacing of 1,20 m, were installed below the slopes at the border of the reclamation area. Consolidation of the soft layers in the subsurface would decrease the risk for slope failures, since the consolidation would increase the shear strength of the soft layers. Then, the reclamation process continued and the level was raised to + 4 m CD. After that, a second series of vertical drains were installed, mainly below the future reclamation platform area itself. In order to meet the requirements towards residual settlements, an extra surcharge was foreseen. The surcharge was build-up with slope 1:6 to level +12 m CD. Eventually, the surcharge was removed to the required design level of +5 m CD and the slope protection was successfully installed.

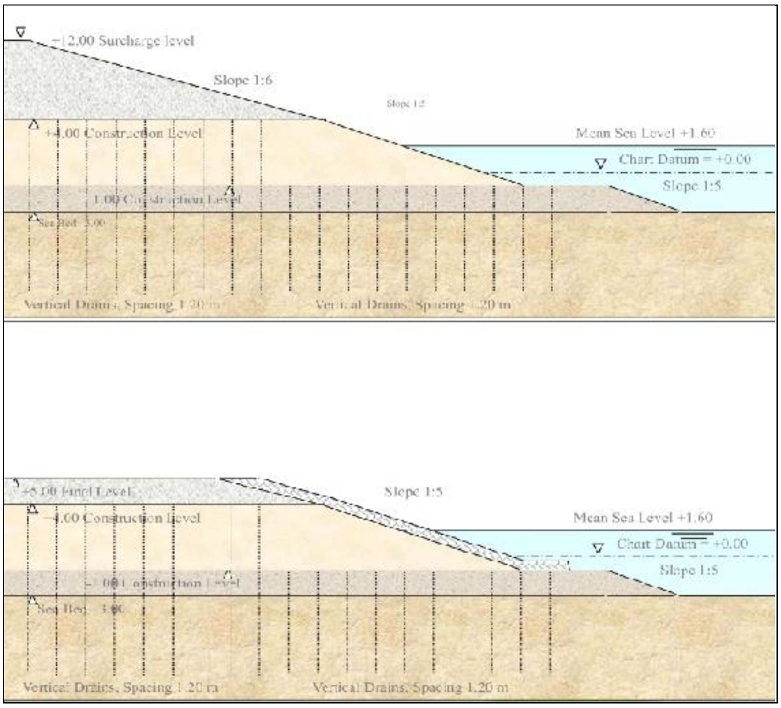


Figure 30: Multiple phase PVD- installation + surcharge in Pulau Bunting, Malaysia

4.1.2.2 Vacuum consolidation

i. Principle and operation

The Vacuum consolidation technique is an atmospheric consolidation procedure. The principle consists in creating a vacuum in fine-grained soils in order to consolidate it. The consolidation is carried out by means of vertical drains (PVD) into the ground, horizontal drains (HD) installed in a draining layer laid on the soil surface and an air and water pumping system, (as illustrated in Figure 31 and Figure 32). A protection fill (about 300 m thick) is installed above the draining layer. The vacuum spreads through the network of drains and enables to apply a depression of between 60 and 85 kPa which must be maintained until end of consolidation. If the soil is totally saturated, the depression generation by pumping is applied immediately and creates an isotropic state of stress around drained soil volume. The vacuum system aims to accelerate the soil consolidation and thus reduce the consolidation time without the need for surcharge fill material. In order to confine the whole system, the area is covered by an impervious membrane anchored in peripheral trenches (see Figure 32). The membrane is protected by a sandy layer. The trenches maintain the soil saturation and hence prevent from groundwater table lowering. The construction of a slurry wall is required if a coarse-grained layer is present above or interbedded in the fine-grained soil. If an additional surcharge fill is necessary, isotropic consolidation allows raising the surcharge fill without waiting a consolidation period during which the undrained shear strength increases. This gain of undrained shear strength is normally needed for ensure the stability of the surcharge fill. The vacuum depression entails no change at short term. Over time, the vacuum depression leads to increase effective vertical stresses by decrease of pore pressures. The total vertical stress remains constant. At long term, the reduction of pore pressures is equal to the vacuum pressure whereas the increase of effective vertical stresses is equal to it. Soil consolidation using vacuum increases speed of consolidation and hence lowers consolidation time from years to months without any surcharge fill.

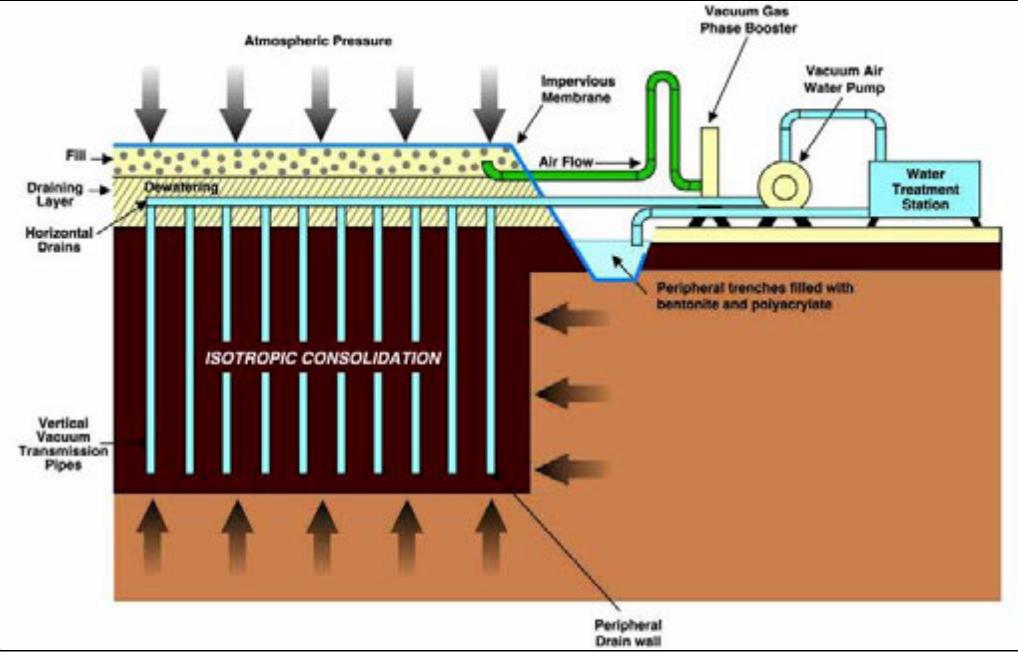


Figure 31 : Vacuum Consolidation principle (Menard system, after Masse & al., 2001)

Figure 32: Site preparation for Vacuum consolidation (Courtesy from Austress-Menard)

The unit cell consolidation theory of radial drainage subjected to vacuum preloading has been developed by Indraratna & al. (2005) and needs basic assumptions including:
- Darcy's law is valid;
- Soil is fully saturated;
- Water and soil grains are incompressible for all practical purposes;
- Deformations follow small strain theory;
- Vertical loads are initially effected (carried) by the pore water pressure u_0,
- Compressive strains within the soil mass occur isotropically;
- Coefficients of compressibility and permeability are assumed to be constant.

The average excess pore pressure ratio in both vertical and horizontal directions can be expressed by (Rujikiatkamjorn and Indraratna, 2007):
(a) Surcharge loading combined with vacuum pressure application:

$$\frac{\overline{u_e(t)}}{u_0} = -\frac{p_0}{u_0} + \left(1 + \frac{p_0}{u_0}\right)\sum_{m=1}^{\infty}\frac{8}{(2m+1)^2\pi^2}exp\left(-\left[\left(\frac{2m+1}{2}\right)^2\pi^2\frac{1}{c_{vr}L_n^2} + \frac{8}{F}\right]T_r\right) \tag{21}$$

With: $c_{vr} = c_r/c_v = k_h/k_v$ (Compressibility or permeability ratio)

$L_n = L/d_e$ (Normalized drain length)

$VPR = p_0/u_0$ (Vacuum pressure ratio)

Where: u_0 is the initial excess pore pressure (equal to the surcharge load $\Delta\sigma$) and p_0 is the vacuum pressure.

(b) Vacuum application only (no effect of u_0):

$$\overline{u_e(t)} = -p_0 + p_0\sum_{m=1}^{\infty}\frac{8}{(2m+1)^2\pi^2}exp\left(-\left[\left(\frac{2m+1}{2}\right)^2\pi^2\frac{1}{c_{vr}L_n^2} + \frac{8}{F}\right]T_r\right) \tag{22}$$

The advantage of the proposed method is that the excess pore pressure, both positive (due to surcharge load) and negative (due to vacuum pressure) can be obtained simultaneously. The overall average degree of consolidation with time U(t) can now be evaluated conveniently by:

$$U(t) = -\frac{\overline{u_e(t)}}{p_0} \tag{23}$$

Substituting equation (23) into equation (22) gives:

$$U(t) = 1 - \sum_{m=1}^{\infty}\frac{8}{(2m+1)^2\pi^2}exp\left(-\left[\left(\frac{2m+1}{2}\right)^2\pi^2\frac{1}{c_{vr}L_n^2} + \frac{8}{F}\right]T_r\right) \tag{24}$$

The previous equation shows that the total consolidation rate at any vacuum condition (p_0) is uniquely related to the time factor, vertical drain configuration and anisotropic soil permeability. Once is known, as suggested by Chai et al. (2005), the associated settlement at a given time t is then evaluated by the following conventional equation:

$$S(t) = \delta \times U(t) \times S_\infty \tag{25}$$

For isotropic consolidation, δ can be calculated by:

$$\delta = \frac{1-\nu}{1+\nu} \tag{26}$$

Where: ν is the Poisson's ratio of the soil skeleton.

In the case of no lateral strain, δ = 1 (e.g. centreline of embankment). For a soil thickness (equal to drain length if PVDs penetrate the entire clay thickness), the total primary consolidation settlement is given by:

$$S_\infty = m_v \times (u_0 + p_0) \times L \tag{27}$$

The benefits of vacuum preloading in comparison with conventional PVD preloading are as follows (Qian et al., 1992):

- The effective stress increases due to vacuum pressure, and the corresponding lateral movement is compressive towards the drain, unlike standard PVD preloading (as illustrated in Figure 33). Consequently, the risk of shear failure can be minimized even at a higher rate of embankment construction.
- The vacuum head can be distributed to a greater depth of the subsoil via the PVD system – size of apertures of PVD core and filter properties influence the effectiveness of the depth of vacuum propagation.
- The extent of surcharge fill can be decreased to achieve the same degree of consolidation, depending on the efficiency of the vacuum system in the field (see Figure 33). Effectiveness decreases dramatically if air leaks occur either through the membrane imperfections or drain-soil interface conditions (research at University of Wollongong).
- Since the surcharge fill height can be reduced considering an equivalent vacuum head, the maximum excess pore pressure generated by vacuum preloading will be less than a greater fill height without vacuum application.
- Vacuum pressure compensates for the inevitable unsaturated condition at the soil-drain interface (air gap due to mandrel withdrawal), resulting in an increased rate of consolidation.

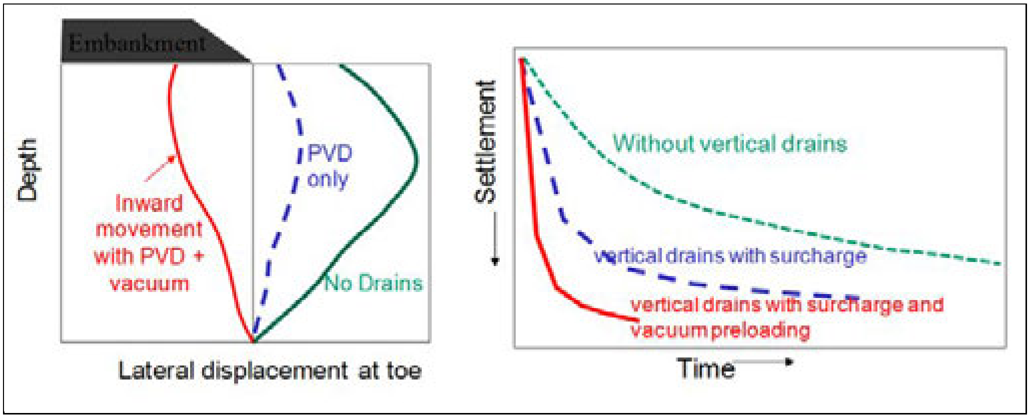


Figure 33 : Some potential benefits of vacuum consolidation

Rujikiatkamjorn (2005) have compared the embankment performance for different consolidation methods, as shown in Table 7. It is noted that 1) a reduction of drain spacing, 2) an increase of surcharge load or 3) an application of vacuum pressure leads to a decrease of consolidation time. However, these last methods must cope respectively with the possible following problem: 1) excessive smear effect due to an over disturbance on soil (diameter of influence zone decreases while diameter of smear zone remains the same), 2) higher risk of slope stability, 3) air leaks which make difficult the maintaining of vacuum pressure.

Table 7: Comparison of consolidation time based on various approach

Parameters	Surcharge preloading	Drain spacing reduction	Higher surcharge load application	Vacuum combined with surcharge preloading
Diameter of influence zone, d_e (m)	1.05	0.7	1.05	1.05
Equivalent diameter of drain, d_w (m)	0.07	0.07	0.07	0.07
Diameter of smear zone, d_s (m)	0.35	0.35	0.35	0.35
Length of PVD, L (m)	20	20	20	20
$n = d_e/d_w$	15	10	15	15
$s = d_s/d_w$	5	5	5	5
Coeffcient of radial consolidation, c_r (m²/yr)	3	3	3	3
Coeffcient of vertical consolidation, c_v (m²/yr)	1.5	1.5	1.5	1.5
k_h/k_s	3	3	3	3
Compression index, C_c	0.29	0.29	0.29	0.29
Preconsolidation pressure, p'_c (kPa)	20	20	20	20
Initial void ratio, e_0	2	2	2	2
Surcharge load, Δp (kPa)	50	50	100	50
Vacuum pressure, p_0 (kPa)	-	-	-	50
Maximum excess pore pressure (kPa)	50	50	100	50
Consolidation time t_{req} to reach U=90% (days)	200	113	86	86

ii. Application field

This procedure is intended for saturated cohesive soft soils with low permeability. Under the effects of Vacuum consolidation technique, the soil behaves differently than under the influence of a preloading fill.

iii. Case study

The vacuum consolidation technique has been successfully applied in many projects. One of these projects is the Ruisbroek project, near Antwerp, where an old harbor dock was filled with dredged sludge. In a first phase, the existing dock was isolated from the nearby canal by constructing bunds and a sheet pile wall. After that, the basin was filled with dredged mud. In order to successfully place the sludge inside the basin, a specially designed barge elevator was used. This barge elevator could successfully transfer the dredged sludge from the supplying barge, across the bund/sheet pile wall, into the storage basin. After the basin was filled, a 60 cm thick sand capping layer was placed on top of the dredged sludge. This layer would serve as a drainage layer for the vacuum application. Prefabricated vertical drains with a length of 15 m were installed. Eventually, after the membrane was installed, all the drains were connected to the pump and vacuum consolidation could start. Eventually, the project was considered as a success; there were no significant delays and the required degree of consolidation was achieved within the prescribed time period.

Figure 34: Project at Ruisbroek; successful application of vacuum consolidation technique

4.2 Ground improvement with granular admixtures in cohesive soils

4.2.1 Stone columns

i. Principle and operation

Two techniques exist to perform Stone Columns (SC): either by wet top feed method (as shown in Figure 35) or dry bottom feed method (see Figure 37).
The wet top feed method consists in sinking a cylindrical vibrating probe (similar as the one used for vibrocompaction) in the ground under vibrations and jetting of water down to the desired depth. The ballast material is fed from the surface through annular space alongside the probe and creates the stone columns. The drilling is "sweeped" several times by pulling up the vibrating probe to the surface and bringing down it quickly. This process enables to clean the annular space in order to help the fall of the ballast by gravity. The vibrator is pulled up in steps (around 1 m) and repenetrates the column in order to compact it. The soil compression leads to a conical crater which is backfilled. The columns executed with this method are usually called "vibro stone columns".

Figure 35 : Stone Columns procedure by wet top feed (Vibroflotation Group)

This method is also applied to offshore project: foundation of offshore retaining walls in order to protect the reclaimed land offshore, seawall foundations... The stone columns are installed from a barge. The ballast is either stored on the sea bottom and inserted by gravity through annular space alongside the probe in the soil (blanket method, see Figure 36a), or stored on a barge and include through an annular metallic tube laid on the soil (see Figure 36b).

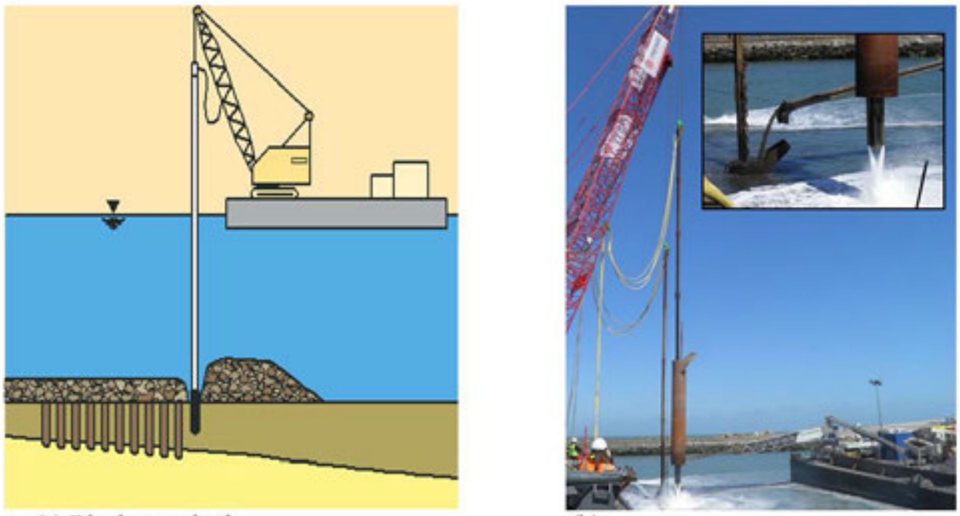


Figure 36 : Offshore stone columns ((a) Keller Group, (b) Menard group)

The dry bottom feed consists in sinking the vibratory probe assisted by vibrations and compressed air jetting down to the prescribed depth. The soil is pushed laterally during penetration of the probe and thus produces any spoil at the surface. Then, the vibrating probe is lifted up gradually in successive passes (about 0.3 m to 0.5 m). The ballast material is added through a tremie pipe alongside the probe. Compressed air (130 kW) is injected in order to facilitate the incorporation of aggregates continually to the bottom of the pipe and thus ensure the continuity of the columns. The vibrator is reintroduced into the ballast in order to compact it. This method is also used in offshore project

Figure 37 : Stone Columns procedure by dry bottom feed (Vibroflotation Group)

The stone columns have a diameter varying according to soil compressibility. Their shape is inflated in soft soil and strangled in stiff soil. The diameter ranges from 0.6 m to 1 m. With this technique, the magnitude of the replacement ratio achieves about 4% to 20%. Stone columns are not necessarily anchored in a stiffer layer. The quarry material which forms the columns is a cohesionless granular material with fraction 0/40 mm perfectly graded. This gives to columns a high flexibility and significant draining characteristics which allow accelerating consolidation in surrounding soft soil. The stone columns enable also increase the bearing capacity, to reduce the magnitude of residual settlement, to eliminate the risk of slope failure and liquefaction during an earthquake or to. Their flexible feature allows tolerating seismic strain while keeping providing a bearing capacity.

The failure modes of an isolated stone column which is loaded vertically at the top have been described in a homogenous soft soil by Datye (1982) for: columns whose base relies on a stiff layer (bulging failure, shear failure) and “floating” columns (punching failure).

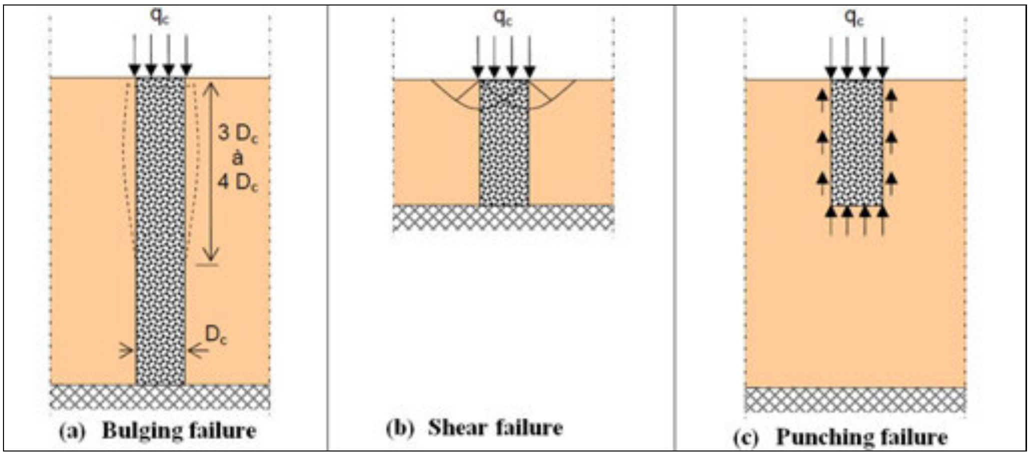


Figure 38: failure mechanism of a Stone Column (after Datye, 1982)

The two significant parameters which enable to describe the efficiency of the treatment by a network of stone columns are: the settlement reduction ratio β and the ratio stress (column/soil) n. There are several different analytical methods defining the column behavior to determine these two parameters. The most commonly used methods are homogenization method and Priebe method (1976, 1995) which consider elastic model for stone columns. Priebe introduced a relation between the ratio stress p_s/p (where: p_s is the external load on the soil and p is the foundation load), the reciprocal area ratio A/Ac (where: A is the grid area and Ac is the section of the column) and the friction angle of the ballast, as illustrated in Figure 39. The Figure 39 shows that the residual stress on the surrounding soil is lower when the area ratio decreases. For a fixed area ratio, the improvement will be more efficient if the friction angle of the ballast is higher.

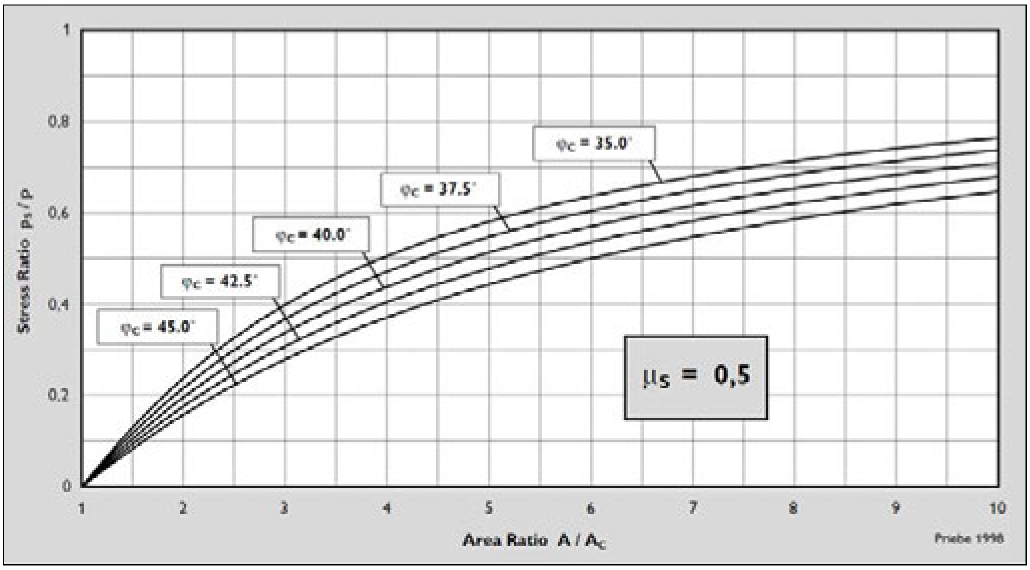


Figure 39: Residual stress on the soil surrounding stone columns (after Priebe, 1998)

In past decades, other analytical methods were undertaken taking into account plastic model (Ghionna & Jamiolkowski, 1981) or elastoplastic model (Goughnour & Bayuk, 1979). Although these methods are more complex, their benefit is to provide results similar to the ones obtained by FEM analysis.

ii. Application field

Stone Columns technique is used for reinforcement of soft soils (clayey as well as gravelly), except organic soils (mud, peat ...) and or containing anthropogenic waste whose mechanical characteristics degrade over time (creep). Thus, these last soils are not able to ensure a lateral confinement sufficient to remain stable columns at long term. Stone columns can be carried out above and below ground table water.

iii. Case study

An example from Stone Columns works is Anpara Thermal Power Station project (Raju, 2011) which is located at Anpara, state of Uttar Pradesh, India. The project was an expansion of the existing power plant by setting up Unit-D of 2 x 500 MW capacity. The project was developed on an abandoned ash pond. The fly ash deposit (widely present in India) was loose to medium dense and was encountered on site over a depth varying from 3 m to 13 m. This layer was underlain by hard clayey silt/medium silty silt down to a depth of 23 m and weathered rock (granitic gneiss) beyond this depth. In general, the geotechnical characteristics of fly ash deposits were not consistent with depth. As a solution of traditional foundations was not viable, a solution of soil improvement has been foreseen in order to achieve the following objectives:

- improve bearing capacity of open foundations of different structures of coal handling plant;
- enhance the lateral capacity of bored cast-in-situ pile foundations of structures like stacker-reclaimer of coal handling plant;
- mitigate the liquefaction potential in an event of earthquake.

The contract requirements were a bearing capacity of 10 t/m² for open foundations, a lateral load capacity of 7 t with ultimate load of 21t for bored cast-in-situ piles, and a allowable settlement limits of 5 mm. The soil improvement works was awarded to Keller which adopted Vibro stone columns using bottom feed method. Before SC works, test trials was carried out to determine the optimal grid and assess the increase in bearing capacity (plate load tests) and in lateral capacity of piles (lateral load tests, see Figure 40) as well as the reduction of settlement. From test trials, it was retained to meet the criteria the following stone columns characteristics:

- For open foundations : a network of stone columns having a diameter of 0.9 m and anchored of 0.5 m into the underlying stiff layer as per a triangular grid of 2 m;
- For bored cast-in-situ piles: stone columns of 0.5 m diameter surrounding the piles.

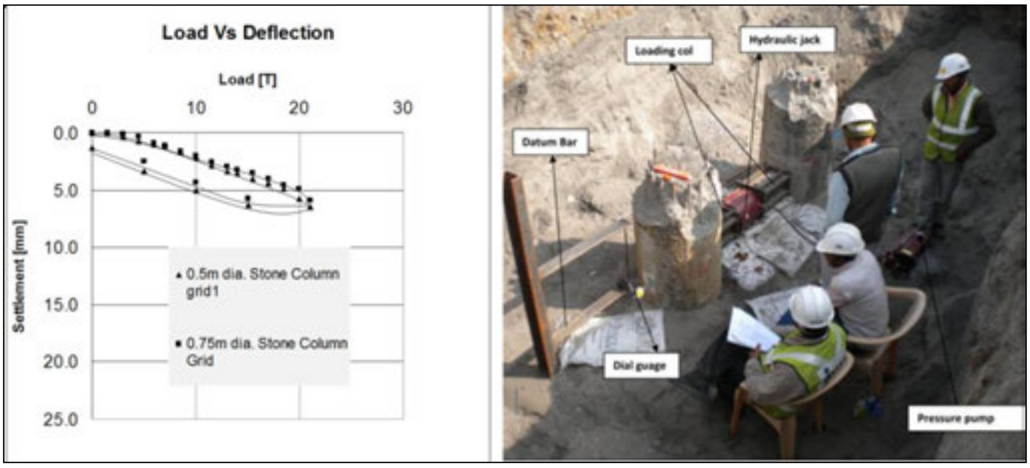


Figure 40: Load-displacement curves and pile testing (Keller group)

4.2.2 Dynamic replacement

i. Principle and operation

The dynamic Replacement (DR) technique is an extension of Dynamic Compaction technique in compressible soils with high fines content where DC is not effective. Similar equipment is used. The method consists in forming a pillar by alternating tamping and print backfiring phases (as shown in Figure 41). The pounder weights of usually between 10 and 20 tons drop quasi-freely from a height ranging from 10 to 30 m. The fall produces a crater which is backfilled with granular material (fraction 0/400 mm with fines content less than 30%). Then, the process is repeated until the prescribed depth or tool refusal is reached. The compaction energy is also transferred to the soil whose mechanical characteristics might be improved.

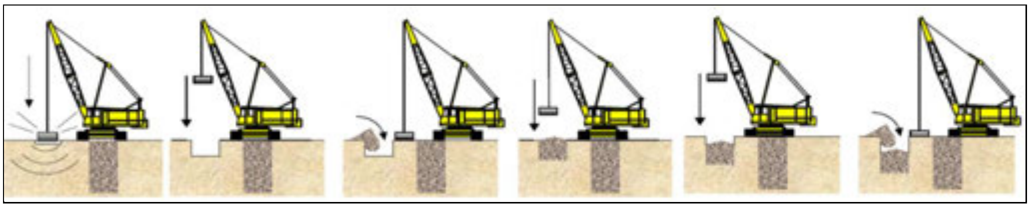


Figure 41 : Dynamic Replacement procedure (Menard Group)

In the event that the bearing capacity of the DR crane is not ensured by the ground, a minimum 0.5 m thick working platform is necessary and must be above ground water level. The DR process can be applied depths from 4 up to 8 m below the working platform. In general, the diameter of DR pillars ranges from 2 m to 3 m and grid varies from 4 m x 4 m to 7 m x 7 m which is equivalent to a replacement ratio of between 10 to 25%. The backfill material gives to pillar draining characteristics which allow accelerating pore pressure dissipation (generated by a loading or an earthquake) in surrounding compressible soil. In some cases, before tamping, a pre-excavation (for example, 2 m deep and 2.5 m side) partially backfilled with a “plug”of granular material is required at the location of the future pillar in order to: increase improving depth, pass through shallow dense and compact layers or limit soil swilling on the surface. The pounders are different from the ones used for DC: they have a smaller section and a higher height which enables to make it easier the punching through the soil (as illustrated in Figure 42). DR pillars are designed just as stone columns.

(a) DR tamper (b) DC dropweight

Figure 42: Pounder weights (Menard Group)

ii. Application field

The soils which can be improved by DR technique are silt and clay deposits, and anthropogenic backfill. Furthermore, this process is used in peat and organic layers due to the relatively low slenderness of the DR pillars.

iii. Case study

A case study combined with DC methods is described in sections 4.3.1 and 4.3.2.

4.2.3 Sand compaction piles

i. Principle and operation

Invented in Japan and widely used in Asian countries, the Sand Compaction Piles (SCP) technique is a sort of flexible inclusion whose construction process is different from the ones for stone columns (different equipment). The technique consists in forming columns of compacted sand where sand is fed through a casing pipe into the soil then compacted by means of either vibration, dynamic impact or static excitation (see Figure 43). The method is applied to both onshore and offshore projects. The state-of-the-art, design and construction issues can be found in a book written by Kitazume (2005).

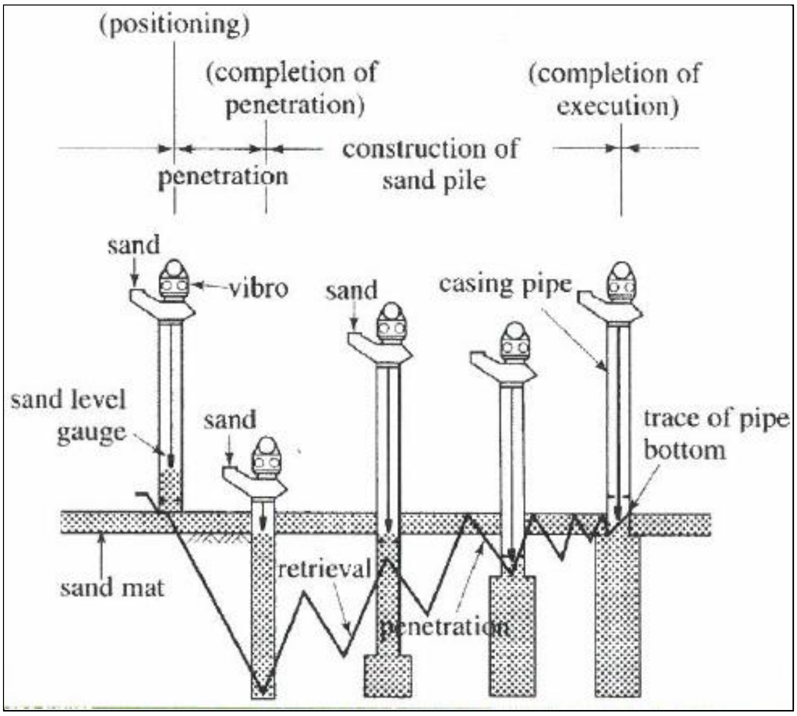


Figure 43: Sand compaction piles procedure

ii. Application field

SCP technique is used for reinforcement of both clayey and sandy soils. In sandy soil, the use of SCP method aims to reduce settlement and prevent liquefaction. In clayey soils, the purposes are the same as for stone columns technique.

4.2.4 Geotextile confined columns

i. Principle and operation

The Geotextile Confined Columns (GCC) technique consists in performing a borehole by driving or vibrating a steel casing (diameter of about 80 cm) into the ground and then installing cylindrical closed bottom Geotextile "sock"(tensile strength of between 200 and 400 kN.m). The latter is wholly filled with sand, as shown in Figure 44. Once completed, the casing is removed. Raithel & Kempfert (2000) and Raithel & al. (2005) proposed refined analytical and numerical procedure to show the benefits of Geotextile confinement. This procedure allows relieving the load on the soil without altering significantly the soil structure.

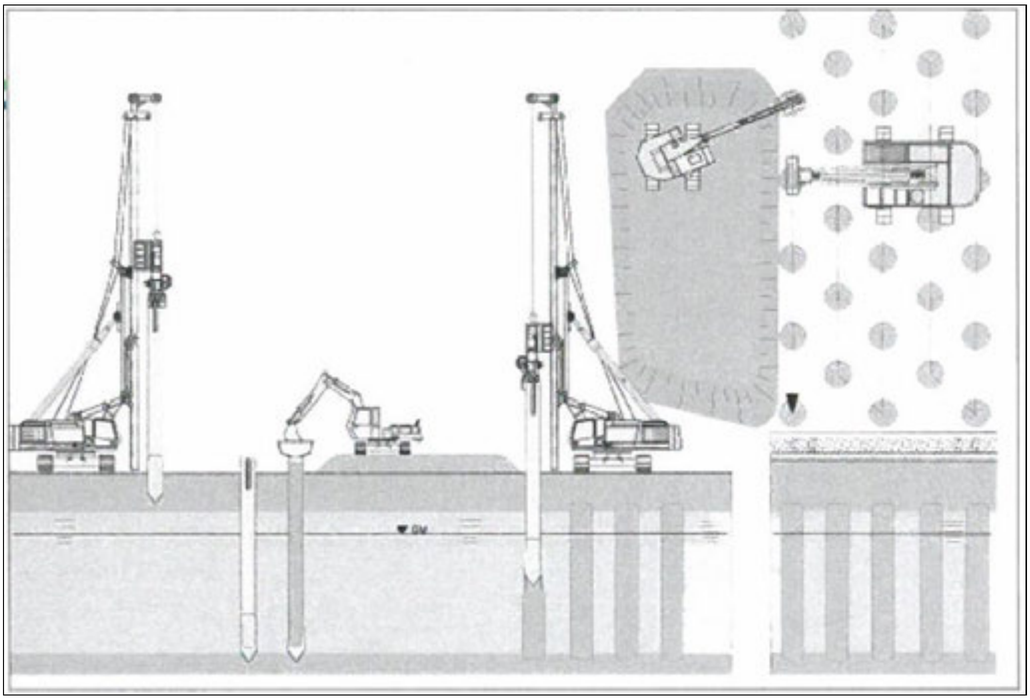


Figure 44: Geotextile confined columns procedure

ii. Application field

SCP technique is efficient in soft soil.

4.3 Combination of ground improvement methods

4.3.1 Dynamic compaction and dynamic replacement

An example from practice is the Independent Water & Power Project (IWPP) which is located in Shuaiba, Saudi Arabia (110 km from Jeddah). The project was a combination of a desalination plant and a power plant and consisted of 12 evaporators, 3 water tanks and several related buildings. The tanks had a great diameter of 106.6 m and a height of 20 m. The project was awarded to a consortium of EPC contractors composed of Doosan and Siemens. The area to be treated was 15 hectares. The contractual specifications to reach were:

- a bearing capacity of 150 kPa and a maximum settlement of 25 mm for evaporators and buildings;
- a bearing capacity of 200 kPa and a maximum settlement of 75 mm for water tanks.

Thus a solution of soil improvement has been foreseen in order to cope with problems of settlements as well as with punching stability problems.

The ground investigation had revealed the presence of two types of soil profiles on site. The first profile (zone A) showed loose to dense silty sand (15% to 35% fines content and low friction ratios) down to a depth between 6 and 10m. This layer overlay coralline limestone. The second profile (zone B) exhibited soft silt or very loose very silty sand (60% fines content at 4.45 m depth and higher friction ratio, as shown in Figure 45) over the upper 4 to 5.5 m of ground and below the bedrock. The groundwater levels observed in the boreholes suggested a range between 2 to 3 m depth below Natural Ground Level.

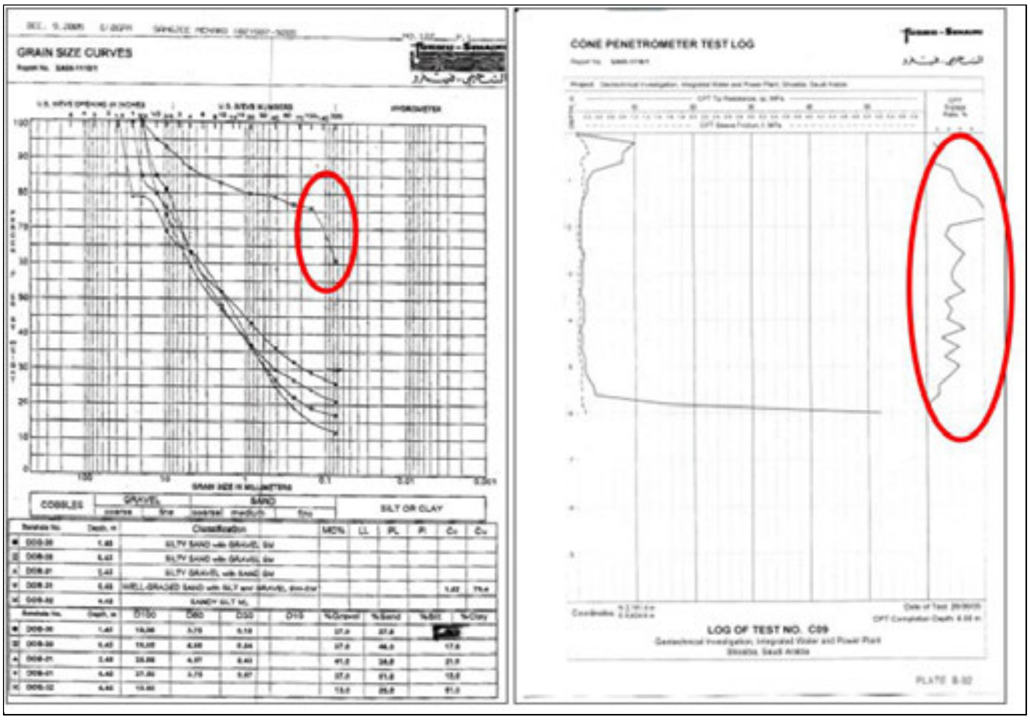


Figure 45: Grain size curves and typical CPT test in zone B from Shuaiba IWPP (Saudi Arabia)

As the proportion of fines content was too high (more than 12%), a vibrocompaction solution was not possible. For all structures, Menard proposed a solution of ground improvement treatment suitable for each zone depending on the soil characteristics:

- zone A: implementation of Dynamic Compaction;
- zone B: execution of Dynamic Replacement (The Dynamic compaction technique was not relevant due to a significant fines content).

A stone columns solution was not retained because of economical reasons.

The acceptance criteria of the project relied on the increase of pressure limit in improved soils. Thus, numerous pressuremeter tests were carried out and proved the achievement of criteria (see Figure 46). The results of hydrotests for water tanks were broadly lower than the maximal required: ring settlement ranging from 28 to 37 mm.

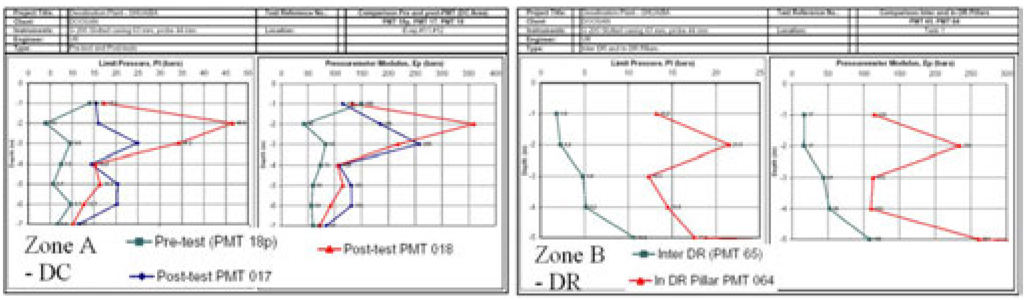


Figure 46 : Compared improvement in DC and DR areas from Shuaiba IWPP (Saudi Arabia)

Another example is the King Abdullah University for Science and Technology (KAUST) project which is located near the Red Sea, 80 km north of Djedda, Saudi Arabia. The project was a vast university campus spreading over a 6 km² area. The geology of the site was comprised of 6-8 m deep heterogeneous soils: loose sands, soft clayey silts (sabkha). Besides, the tender proposition presented very few in-situ tests carried out on site (around 50 boreholes). The major requirement was to ensure the bearing capacity of buildings (with up to 150 tons) at unknown locations. Thus, a pile solution was not feasible. The project was awarded to Saudi Aremco, which decided to perform soil improvement before the issue of design drawings. Menard proposed a solution of ground improvement treatment by Dynamic Compaction and Dynamic Replacement to consolidate 2,600,000 m² in the required period of 8 months. For the project, Menard mobilized 13 rigs to meet the deadline. A criterion was needed to select the improvement methods as both DC and DR technique were used. From many pressuremeter tests carried out on site, site specific relationships between the limit pressure and applied energy were established with different set improvement factor/efficiency/fines content (see Figure 47). It can be identified in Figure 47 the zones suitable for DC and DR, the limit between the 2 zones corresponding well to a fines content of about 30%.

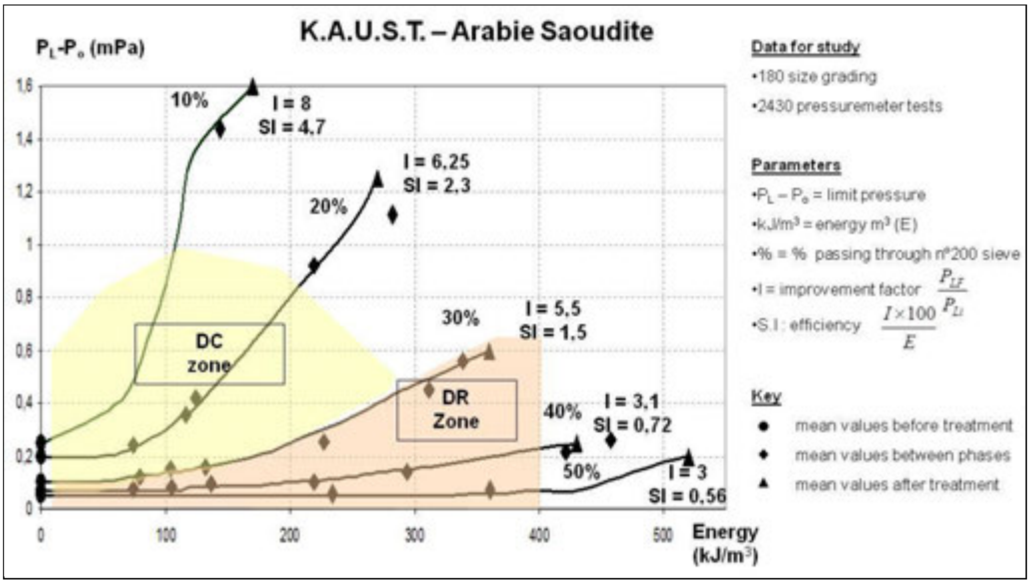


Figure 47: Criterion for the method choice (DC/DR) based on pressuremeter from KAUST project (Saudi Arabia)

4.3.2 Dynamic compaction and vibrocompaction

An example from combination Dynamic Compaction and VibroCompaction is the Pasir Panjang project located in Singapore. The project was a new container terminal constructing by the Port of Singapore Authority (PSA), as illustrated in Figure 48. The soil improvement works was awarded to Menard and aimed to densify loose hydraulic reclaimed sand fill in order to increase the bearing capacity and reduce the risk of soil liquefaction. The requirement was to densify the upper 10 m hydraulic sand fill. The acceptance criterion defined by the client was a value of cone penetration resistance exceeding 15 MPa. This value corresponds to a relative density between 70% and 100%. The relative density D_r can be assessed from correlations with the cone resistance q_c (Jamiolkowski & al. 1985, Baldi & al. 1986 and Schmertmann 1976).

Figure 48: Construction of a new PSA Container Terminal at Pasir Panjang, Singapore

The percentage of fines ranged from 2% to 5% over a depth of 0 to 8 m and 14 to 20% over a depth of 8 to 10 m (shown in Figure 49).

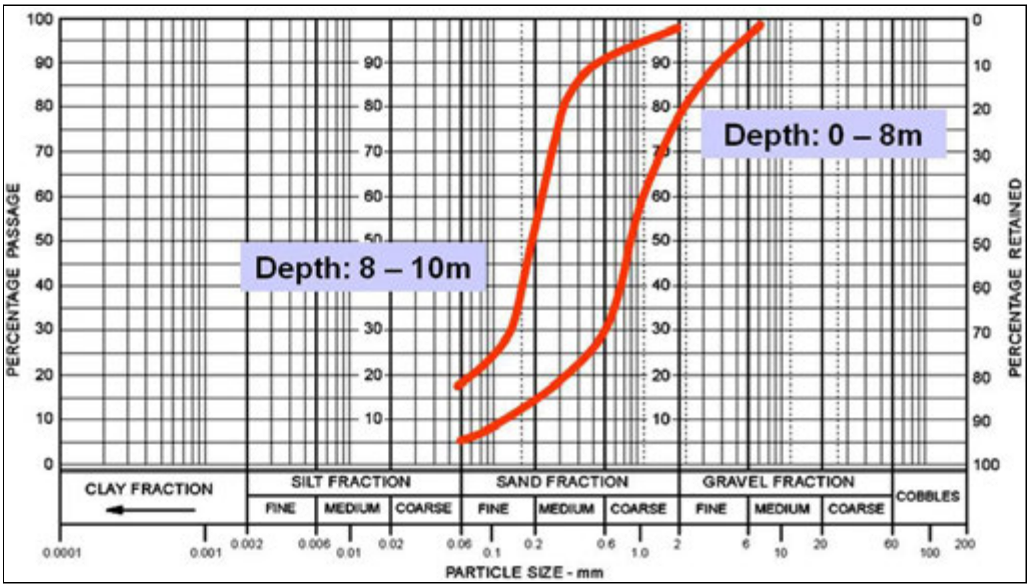


Figure 49 : Grain size curves from PSA Container Terminal project (Singapore)

Based on technical (qc > 15 MPa) and economical considerations (higher production), a combination of vibrocompaction, and dynamic compaction was selected:

- Stage 1: densification from 5 to 10 m by vibrocompaction by taking advantage of overbudden effect to increase the ease of compaction;
- Stage 2: densification of 0 to 5 m by Dynamic Compaction taking advantage of higher compaction effect at shallower depth.

For Vibrocompaction works, a triangular grid of 2.8 m was carried out (instead of a triangular grid of 2.2 m in the event that only a VC method had been chosen). The criteria of compaction were a compaction pressure of 260 bars (hydraulic vibrating probe) or a time intervals of 60 s, whichever is sooner. For Dynamic Compaction works, the energy used was 15 tons x 20 m drop (instead of 18 tons x 22 m drop in the event that only a DC method had been selected). The compaction was performed in 2 phases both

with a squared grid of 6 m x 6 m and a number of 14 blows per print (instead of 16 print in the event that only a DC method had been opted). A last phase of ironing was implemented with a squared grid of 2 m x 2 m.
Based on a geotechnical campaign made of 60 CPT after the works, the combination of VC and DC satisfied the q_c = 15 MPa except for the upper 50 cm, as illustrated in Figure 50. The upper 0.5 m required an additional surface roller compaction. The total enforced settlement after improvement works reached 74 cm (composed of 47 cm after VC works and 27 cm after DC works), which represented about 10% of treatment depth.

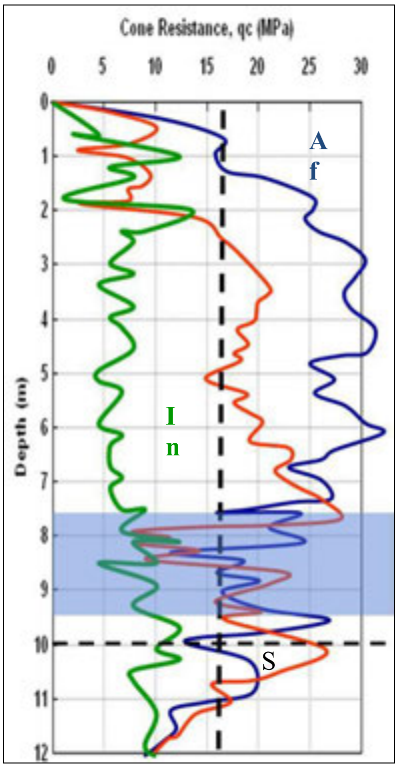


Figure 50 : Typical cone resistance curves from PSA Container Terminal project (Singapore)

As shown in Figure 50, compaction was less effective in layer from 7.5 to 9.5 m deep. The reasons for ineffective compaction could be manifold:

- fines content was higher (between about 14 and 20 %);
- For VC method: cohesion provided by these fine materials prevented momentary breaking of friction between particles through (limited) vibration forces;
- For DC method: lower permeability prevented rapid dissipation of excess pore pressure induced by compaction. Under saturated conditions, repeated impacts only produced displacement ant not densification of the ground.

4.3.3 Dynamic replacement and prefabricated vertical drains

An example from combination Dynamic Replacement and Prefabricated Vertical drains is the Z'Abricots Pond yacht Harbour project which is located in the bay of Fort de France, Martinique. The project consisted of 2 docks with quays and reclaimed areas behind the quays. The geology of the site was comprised of 2 to 8 m deep mud to peat layers above bedrock (weathered tuffite). The ground water table was at the same level as the ground surface. Calculations showed that stability of the structures was not ensured without general substitution of the in-situ soil up to 5 m. As this solution presented economical and environmental problems due to soil substitution, it was proposed an alternative solution by the consortium of companies. The alternative solution consisted in treating the soil by a combination of DR and PVD in order to:

- avoid full excavation and replacement of mangrove;
- reduce the magnitude of residual settlement during service life;
- decrease time consolidation.

The consolidation resulting of soil improvement enabled to ensure the stability of temporary slope during dredging and the stability of final breakwater. The PVD and DR works were performed before dredging from a platform backfilled on soil. The phasing of works is presented in Figure 51. Wicked PVD were installed firstly down to the bottom of mud layer as per a squared grid of 1.4 m x 1.4 m. PVD aimed to dissipate quickly excess pore pressure generated by pounding. DR pillars were executed from a 12 tons weight dropping from a height of 18 m as per a squared grid of 5 m x 5 m (average unit energy of 200 t.m).

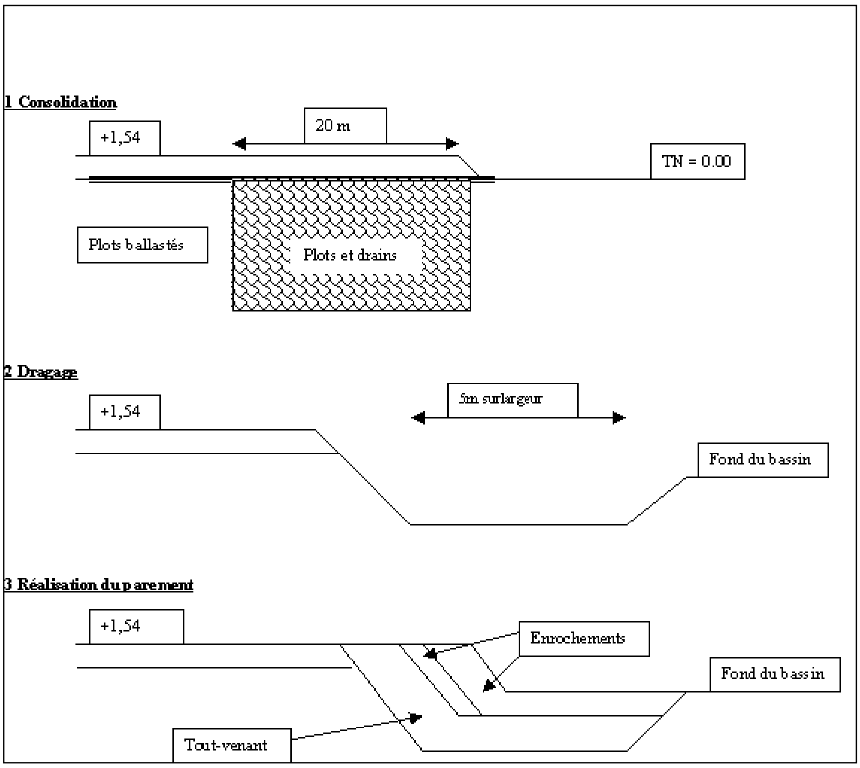


Figure 51 : Phasing of works from Z'Abricots Pond Port project (Martinique)

The acceptance tests carried out were pressuremeter tests within the pillar and had confirmed the achievement of technical specifications (Pressuremeter modulus and limit pressure higher than respectively 8 MPa and 1 MPa).

4.4 Reuse of dredged material

Marine works often have to deal with the reuse or treatment of very soft clays, silts and mud/sludge. Especially harbours need continuous removal of mud in order to guarantee accessibility of the port. Moreover this mud is sometimes (slightly) contaminated which puts limitations to its dumping possibilities. Characteristic for this type of material is its very high water content and low undrained shear strength. Because of this, authorities often have problems how to deal with these kinds of materials. However, several techniques exist to improve the characteristic of these materials, which makes them suitable to reuse after dredging or store them in an effective way.

The first step is to select the most appropriate dredging technique; mechanically or hydraulically. Mechanical dredging techniques are often preferred, since the amount of water added to the material during the dredging process is minimal compared to the hydraulical dredging techniques. After dredging, it is essential to reduce the water content as fast as possible in order to improve the strength of these materials. In reclamation projects, the rehandling process often starts with accelerating the sedimentation of fine grained particles. A water-mud mixture consists of many suspended fines and these fine particles are dominantly negatively charged. Due to their electrostatic charge, they repel each other and remain continuously in motion involving longer suspension times. In order to speed up sedimentation, flocculants are added to these mixtures. Flocculants are often positively charged kation complexes which bind the negatively charged clay particles. The attraction between these two results in the formation of colloids which combine together and precipitate. Once the sedimentation is completed, extra measures can be taken to further increase the consolidation of these sediments.

Installing vertical drains and/or a surcharge is not always practically feasible or economically interesting. In these cases, lagooning could be an option. Then nature takes over; the soft sediments are spread out over a relatively large surface and evaporation of the water takes place under the influence of the wind and the sun (cf. Figure 52). Regularly or even continuously reworking of the sediments will speed up the evaporation process significantly. One could think about several complex rehandling techniques, but one of the simplest and most effective techniques is just driving with an (swamp) excavator through the sediments on a regular base. Once the material has gained sufficient shear strength (ca. ≥ 10 kPa), it is placed in ridges by an excavator. Projects exist in which the water content was reduced to < 55% in only 6 months.

Figure 52: Lagooning at FASIVER site, Zwijnaarde, Belgium

Another possibility is mechanical dewatering of these soft sediments by means of filter press systems (cf. Figure 53). During this process, water is literally pressed out of the sediments. It is evident that for this type of dewatering, the investment is quite large. A nice example from practice is the Amoras project in Antwerp, Belgium, which is further discussed below.

Figure 53: Mechanical dewatering by chamber filter presses at the Amoras project, Antwerp, Belgium

Unfortunately, even if the techniques discussed above are applied, these materials remain relatively soft and special attention is required if these units will serve as a base for further granular fill. If large volumes of granular material are placed on top of these soft materials, the risk of a slope failure or squeezing needs to be studied. In such cases, fill by hydraulic means is always recommended above dry earth movement works. Working by hydraulics means allows to ‘spray’ relatively thin layers of granular material evenly on top of the soft deposit. This minimizes the risk of squeezing or slope failure. Furthermore, if the thickness of the installed sand layer is everywhere more or less the same, similar consolidation results will be achieved over the entire site.

If large volumes of dredged, relatively soft material need to be stored, the fill is often constructed in a so called ‘sandwich structure’ (cf. Figure 54). Here, the fill consist of an alteration between sand layers and the soft material that needs to be stored. The idea is that the sand layers, which are more or less 1m thick and have excellent drainage characteristics, are speeding up considerable the consolidation of the intermediate soft, cohesive materials.

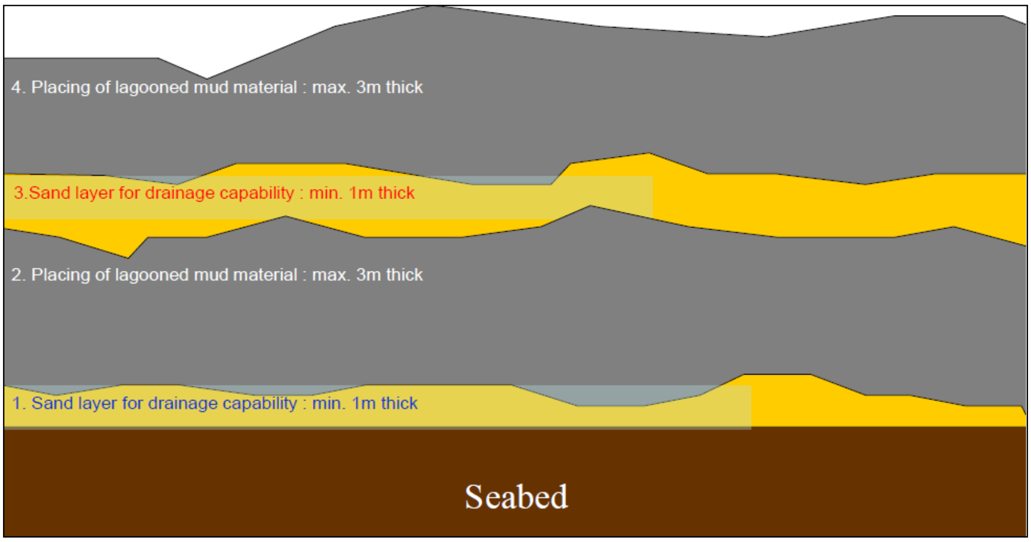


Figure 54: General principle of “sandwich structured construction” for faster dewatering capabilities

4.4.1 Amoras project

The Amoras project is situated in the harbor area of Antwerp, Belgium. Amoras is an acronym that stands for ‘Antwerpse Mechanische Ontwatering, Recyclage en Applicatie van Slib’ (in English: Anwerp Mechanical Dewatering, Recycling and Application of Sludge). The aim of the project is to provide a long-term sustainable treatment and storage of the dredged material from the Port of Antwerp. The entire process consists of several steps, including sand separation, decantation, water separation and storage.

In a first phase, dredged material is received in an underwater cell which is completely isolated from harbor waters and has a storage capacity of 150 000 m³. The deposited dredged material is then pumped by a dredger to the sand separation plant.

In the sand separation plant, two large sieve drums and a number of hydrocyclones separate the sand fraction from the finer silt and clay fractions. The sieve drums separate the material larger than 5 mm, the hydrocyclones separate the remainder of the sand fraction. The silt and clay fractions are then transported first to a buffer tank and from there, through a 4 km long pipeline, to a settling pond at the Bietenveld site.

Figure 55: Aerial view on the Amoras project

The settling pond at the Bietenveld site, with a diameter of 350 m, is further subdivided into 4 quadrants, each having a surface of 250 000 m², in which the sludge is deposited through an automatic valve system. This system decides in which quadrant material is placed. Three of the four quadrants contain the less contaminated sludge and the fourth quadrant is for more contaminated material. A large, rotating dredging gantry with two dredging pumps spans the entire consolidation pond. These pumps can reach the entire pond, are automatically controlled and can operate independently from each other and the water level inside the pond. In the ponds, the material has time to settle and start consolidating.

Then, the material is pumped to the dewatering plant, equipped with 12 membrane filter presses (cf. Figure 53). These presses, with in total 193 filter chambers work under a pressure of 16 bar and can process in one pressing cycle up to 21,5 m³ of material. The filtrate water is transported by gravitation to the water purification plant, which purifies the water by biological means.

The dewatered filter cakes are transported by a conveyor belt system to the ‘zandwinningsput’ deposit site. This site, which is an abandoned sand borrow area and currently used as a sludge pond, is foreseen to process in total 14,5 million ton of dry matter, which implicates a final stacking height over 50 m high and 30 year of exploitation.

The geotechnical issues related to the design of the deposit site lie within the characterization of both the dewatered sludge and the in-situ sludge at the storage site. Numerous tests have been executed and some typical parameters of the dewatered filter cakes are indicated in Table 8. The testing has indicated that the behaviour of the dewatered sludge, drained or undrained, is dependent on the surcharge load. A drained behaviour is observed under small surcharges. Under a small surcharge, the water within the voids is still able to escape and the strength and compressibility characteristics are determined by the strength at the contact points between the lumps. Under larger surcharges, the excessive pore water can no longer escape through the voids and an undrained behaviour is observed.

In the northern part of the future deposit site, the water level is lowered and sand is pumped on top of the in-situ sludge. Vertical drains are installed to accelerate the consolidation of the in-situ sludge. In the southern part of the future deposit site, the in-situ sludge is squeezed out, by controlled filling operations, with the dewatered filter cakes.

Table 8: Characteristics of dewatered sludge at the Amoras project

Characteristics of dewatered sludge	
Granulometry	10 % sand 65 % silt 25 % clay
Volume weight	γ_n = 13,2 kN/m³ γ_d = 5,1 kN/m³
Plasticity Index	PI = 60 %
Water content	110 to 150 %
Undrained cohesion	$c_{u, peak}$ = 5 kPa $c_{u, residual}$ = 2 kPa
Drained strength characteristics	c'=2 kPa and ϕ'=10° (in-situ sludge) c'=5 kPa and ϕ'=25° (dewatered sludge)

The Amoras project is a large scale example of the reuse of dredged material. The (future) geotechnical challenges of this project lie within accurate prediction of the settlements, horizontal deformations and stability issues related to both the in-situ sludge as the dewatered filter cakes.

5 REQUIREMENTS AND QUALITY CONTROL

If for a project ground improvement methods are applied, it is required afterwards to monitor and verify the specified effect thereof. Each of the ground improvement techniques discussed in the previous chapters requires its own particular method in view of monitoring and quality control. Some examples are:

- Vertical drains: control of achieved consolidation, settlements, strength increase,…
- Deep and surface compaction: control of density increase, bearing capacity, liquefaction susceptibility,…
- Stone columns: strength increase, resistance to deformation,…

This chapter will not give an overview of every possible method of quality control for every method of ground improvement. The scope of this chapter is rather to focus on (the relevance of) some regularly encountered requirements and the issues related to quality control (in particular soils).

5.1 Generally applied requirements

5.1.1 Reclamation materials

For most fills there are requirements towards granulometry. A good quality fill has a limited amount of large fragments and also limited amount of fines. It is not uncommon that it is required to limit the fragments > 200 mm and to keep the percentage of fines, i.e. particles < 63 µm, below 10% to 15%. Often there are also requirements regarding plasticity. In some, more exceptional cases, requirements are put forward regarding chemical content, mineralogy, shape and angularity of the material.

5.1.2 Compaction

Compaction requirements define the level of compaction to be achieved in the fill. There is often a distinction between levels of compaction to achieve above and below the water table. Regarding the specification of the requirements, there are different specification possibilities. The most common requirements are related to relative density, degree of compaction, absolute bulk density and minimum cone penetration resistance.

First of all, when analysing the requirements, one should be aware of the fact that relative density can be defined in two ways. As indicated in formula (28) and (29), one definition is based on void ratio and another on porosity. The relative density related to the void ratio, R_e, is generally used in Anglo-Saxion engineering practice and is also known as density index ID. Also the abbreviation Dr is commonly applied.

$$R_e = ID = D_r = \frac{e_{max}-e_{situ}}{e_{max}-e_{min}} = \frac{\rho_{d,situ}-\rho_{d,min}}{\rho_{d,max}-\rho_{d,min}} \times \frac{\rho_{d,max}}{\rho_{d,situ}} \qquad (28)$$

$$R_n = \frac{n_{max} - n_{situ}}{n_{max} - n_{min}} = \frac{\rho_{d,situ} - \rho_{d,min}}{\rho_{d,max} - \rho_{d,min}} \tag{29}$$

Formula (30) is the definition of relative compaction (RC), also known as degree of compaction (D_{comp}) in which the dry density corresponds to the maximum dry density (MDD), achieved according to a standard test. In practice, this often comes down to comparing the situ dry density to the maximum proctor density (mpd). One should also be aware of the fact that the requirement of achieving a particular degree of relative compaction is not the same as achieving the same degree of relative density. From the formulas (28), (29) and (30) it is obvious that achieving 95% relative density is far stricter than achieving 95% of relative compaction.

$$RC = D_{comp} = \frac{\rho_d}{\rho_{d,max}} = \frac{\rho_{d,situ}}{mpd} \tag{30}$$

Although relative compaction and relative density are not the same principle, they do refer to the same geotechnical parameters; the minimum and maximum achievable density of a particular soil. Considering the determination of these parameters, it is important to pay attention to the applied testing method for minimum and especially maximum density in relationship to the type of material (cf. Figure 56).

For the maximum density determination of a material, several techniques are possible:
- The Proctor test (ASTM D698, ASTM D1557, BS 1377: Part 4: 1990: Chapter 3)
- Vibratory table test (ASTM D4253-93)
- Vibrating hammer test (BS 1377: Part 4: 1990: Chapter 4)

In contracts, it is common to specify maximum density in terms of the Proctor test. The principle of this test is to fill a cylinder in layers and compact each layer by dropping a hammer 25 times into the cylinder. The test is repeated at different water contents. This results eventually in a maximum achievable density and corresponding optimal water content. When this test is applied, one should be aware of the difference between the standard proctor test and the modified proctor test. If the modified proctor test is executed, more energy is put into the compaction; the falling weight is heavier, the drop height is higher and the amount of blows of the hammer into the cylinder is also higher. It is evident that the modified proctor test will result in a higher maximum density compared to the normal proctor test.

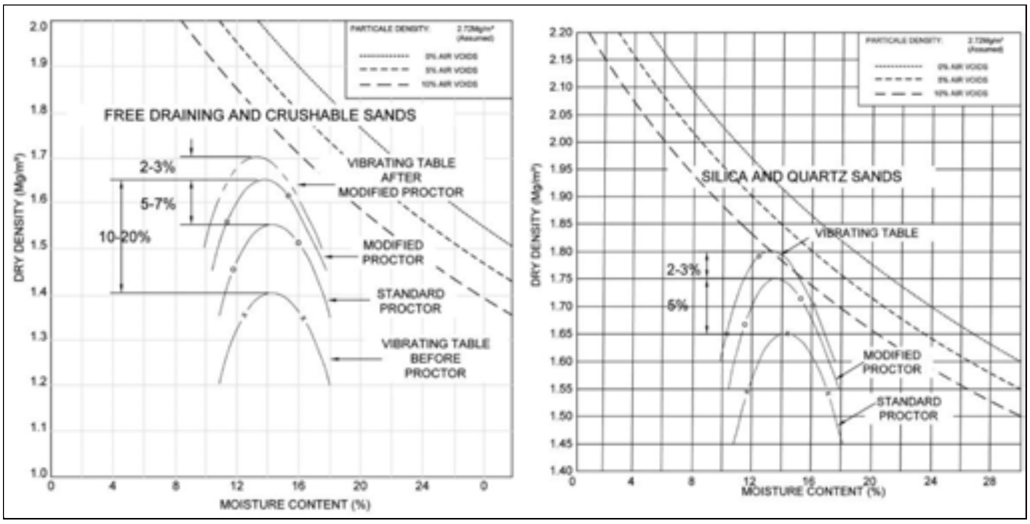


Figure 56: Differences in achievable maximum dry densities, depending on the testing method, for free draining and crushable sands (left) and for silica and quartz sands (right)

For the minimum density determination of a material a reference is made to:
- BS 1377; Part 4: 1990 – Chapter 4
- ASTM D 4254-91

5.1.3 Settlements

Requirements related to (residual) settlements are most often translated into allowable settlements after handover. For these settlement calculations, both the loads imposed by the fill and future service loads should be taken into account. For some particular subsurfaces, continuous deformation under constant load, also called creep, can be quite large and is an important factor in the achievability of the requirements. Where important structures are foreseen, the residual settlements are often more strict. In some particular cases, a certain degree of consolidation under a certain service load is required. Besides residual settlements, there are often also requirements to maximum differential settlements over the reclamation area. Since these are related to the inhomogeneity of the subsurface, these are often more difficult to predict, certainly when insufficient soil data is available.

5.1.4 Bearing capacity

The bearing capacity to achieve is often described as a certain stress applied to the soil (e.g. 50 kPa, 150 kPa). If this stress is related to a service load, it is in fact more related to settlements than to bearing capacity. For a good analysis towards bearing capacity, especially size and depth of the loading surface should be specified.

5.1.5 Liquefaction

For an assessment of dynamic liquefaction in seismic regions, both Peak Ground Acceleration and Magnitude should be given (cf. Youd et al., 2001). The resistance of a granular soil volume to liquefaction is often translated into a requirement of minimum SPT-N blow counts, a particular q_c-value or a certain shear wave velocity (cf. Figure 57). Difference has to be made between the edge areas with slopes and revetments and the large reclaimed land contoured by these structures. At the edges, the risk for generation of significant shear stresses will be larger and therefore, the criteria over there will be more strict. Consequently, different ground improvement techniques could be required at the edges in order to meet the criteria.

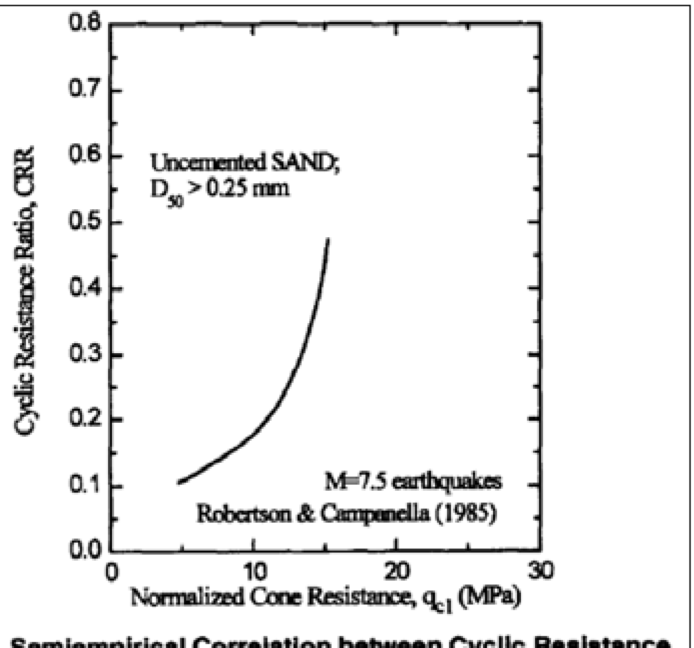


FIG. 1. Semiempirical Correlation between Cyclic Resistance and CPT Penetration Resistance for Clean, Uncemented Sand (Robertson and Campanella 1985)

Figure 57: Classic example of a semi-emperical correlation between cyclic resistance ratio and normalized cone resistance

5.2 Quality control

Quality control methods tend to focus on the fill. In order to avoid client-contractor discussions afterwards, it is of major importance to pay in advance extreme attention to the specified quality control sampling procedures and/or testing procedures. If these procedures are not suited for the applied ground improvement technique or soil material occuring, the risk exists that the requirements will not be achieved.

In fact, a large part of the requirements specified above relate to the same basic soil mechanical principle: the fill should be sufficient ‘dense’ or ‘compacted’. This requirement can be translated into soil mechanical parameter such as high relative density, high friction angle, high cone resistance value in a CPT-test, etc. The remainder of this chapter will therefore focus especially on quality control related to compaction.

5.2.1 Traditional compaction quality control methods

Fills often have a substantial thickness. Furthermore, in hydraulic land reclamation works, fills can be build up rather fast in relatively thick layers, above and below water. Consequently, it is not always evident to control the achieved compaction over the entire fill. Often, several techniques are used or combined. Whether a control method is more appropriate for top layers or for deeper layers, depends on the tested parameter.

5.2.1.1 Control of thin surface layers

Quality control for thin surface layers comes down to the determination of the in-situ density of these layers. Several direct and indirect techniques are available. The basic principle of the direct methods is to dig a hole on the reclamation and measure as accurate as possible the volume of the hole and the mass of the excavated material. Generally accepted, direct techniques are:
- The rubber balloon method (BS 1377: Part 9: 1990: Par 2.3, ASTM D2167)
- The sand replacement method (BS 1377: Part 9: 1990: Par 2.1/2.2, ASTM D4914)
- The core cutter method (BS 1377: Part 9: 1990: Par 2.4)

Figure 58: Technician performing in-situ density determination at 1 m deep excavation by applying the sand replacement method

For indirect methods, a parameter which is related to the density is measured. Site specific calibration is required. The most common techniques are:
- Nuclear density measurements (BS 1377: Part 9: 1990: Par 2.5, ASTM D5195)
- Electrical resistivity measurements

Unfortunately, in-situ density verification by one of these techniques often leads to discussions. First of all, the location of the test is often not clear. Should the test be executed at the final surface of the reclaimed area or also at deeper levels? This could require making excavations on the reclamation site and testing at the bottom of these excavations (cf. Figure 58). And what to do during the build up itself?

Do these techniques imply that the reclamation should be build up in thin surface layers and a test is executed each time before a next layer is put into place? This could lead to large delays and is definitively not applicable for hydraulic projects.

Secondly, the test method itself can lead to discussion. The fill consists mainly of sand with gravelly fragments. Direct methods can lead to discussion whether larger gravel fragments can be included in the test yes or no. It can also be expected that there will be a larger scatter in the results, dependent on the operator. This makes the reproducibility of the test rather low and its applicability rather questionable.

Another issue is that these in-situ tests are executed at several locations over a relatively large area, but each time compared to a single maximum dry density laboratory tests. This implies that possibly two different materials are compared to each other. Therefore, it is advised to take also a sample for laboratory testing at each location where the density is determined in-situ.

5.2.1.2 Control over total thickness

If control over the entire thickness of the fill is required, thus also below the water table, other techniques need to be applied. This comes down to the measurement of a parameter which can be related to the relative density. These techniques have the advantage that they yield in a picture of the homogeneity, strength and achieved compaction over the entire fill thickness rather than control at discrete points.

A widely applied approach is the control by CPT-measurements. At several locations across the reclamation site, the result of the compaction can be assessed over the entire depth of the fill, above and below water. If the requirement is related to a minimum cone resistance, the results are visible directly. If the requirements are related to relative density, the relationship between the cone resistance and the relative density should be determined. Different correlations exist for different types of material. A generally accepted formula (31) for this relationship is:

$$D_r = \frac{1}{C_2}\ln\frac{q_c}{C_0 \times \sigma'^{C_1}} \qquad (31)$$

In which C_0, C_1 and C_2 are soil constants and σ' the effective stress. However, this way of quality control is not applicable for ca. the first meter of the reclamation area. Due to the so-called 'scale effect' of the cone dimensions compared to the limited penetration, reliable results cannot be produced. It is also clear that at shallow depths, both the cone resistance and the effective stress are very low, which would lead to very low relative densities. Also if the requirement is specified in terms of a minimum cone resistance, this cannot be applicable for the first meter of the reclamation.

Furthermore, it is important to realize that soil properties are not based on relative density only; the influence of effective vertical and horizontal stress, the angularity of the grains and the crushability of the grains should be considered when interpreting the results. This sometimes raises questions regarding the (in)validity of relative density for quality control of cohesionless soils (cf. Hamidi, 2011).

Another possible method of quality control is the monitoring of settlements. This results into information relative to the total height of the treated soil mass, but gives no indication of the achieved compaction with depth. Furthermore, to get accurate results, a very dense grid of settlement points is required. Therefore, this quality control method is in practice not often applied.

5.2.2 Quality control for functional requirements

More and more, clients tend to choose for a performance based design. In order to assure the serviceability and durability, the emphasis is placed on the control of structural deformation rather than on a particular safety factor, a particular cone penetration resistance over a certain depth, a certain relative density, etc. The requirement is in this case often defined as a maximum deformation under a certain load. It is common practice to verify these requirements by a large plate loading test or a zone load test (cf. ICE, 1987).

The plate loading test is a test in which the deformation (settlement) of the soil below a circular loading plate is measured. The test is executed by applying increments of load and observing the subsequent plate settlements. It is common practice to wait until settlement from the application of one increment is complete before applying the next increment. Cycles of loading and unloading may be undertaken in the course of the test in order to assess how much settlement is irreversible and how much 'elastic'. The magnitude of the load to be applied in each increment, the rate of application of load, the need to undertake cyclic loading and so forth depend on the nature of the project and the nature of the materials being tested. The volume of soil being affected by the test depends on the diameter of the loading plate and is, in the view of foundation applications, relatively small in comparison with the size of most foundations widths.

The principle of the zone load test is similar to that of the plate loading test. The difference lies in the dimensions; the 'plate' applied in the zone load tests is much larger and squared (cf. Figure 59). Therefore, the zone load test verifies the bearing pressure over a much wider and deeper zone compared to the plate load test. It is not uncommon that the applied loads are similar to the foreseen design loads and 25% overload. The zone load test is then a full-scale performance test on site simulating the specified future loading conditions.

Figure 59: Typical zone load test set-up

5.2.3 Quality control in crushable sands

When dealing with crushable sands, it is advised not to implement classic quality control methods. Crushing of these materials can/will occur both during dredging, hydraulic transport, compaction and testing. However, estimating how these materials will behave under loading remains challenging. At what stress level and to which amount crushing will occur is very hard to predict.

Because of this, it is clear that the proctor test is not suitable to use in crushable sands. The heavy local impact of the falling hammer will crush the calcareous grains which implies the generation of more fines compared to the 'original' material at the beginning of the test. The generated amount of fines can be assessed by comparing the particle size distribution curve before and after testing (cf. Figure 60). Because of these extra fines, the maximum density which can be achieved will be higher than without smashing the grains. If later the in-situ densities are compared with the results of the laboratory testing, two different materials are in fact being compared to each other; an original one with a crushed one.

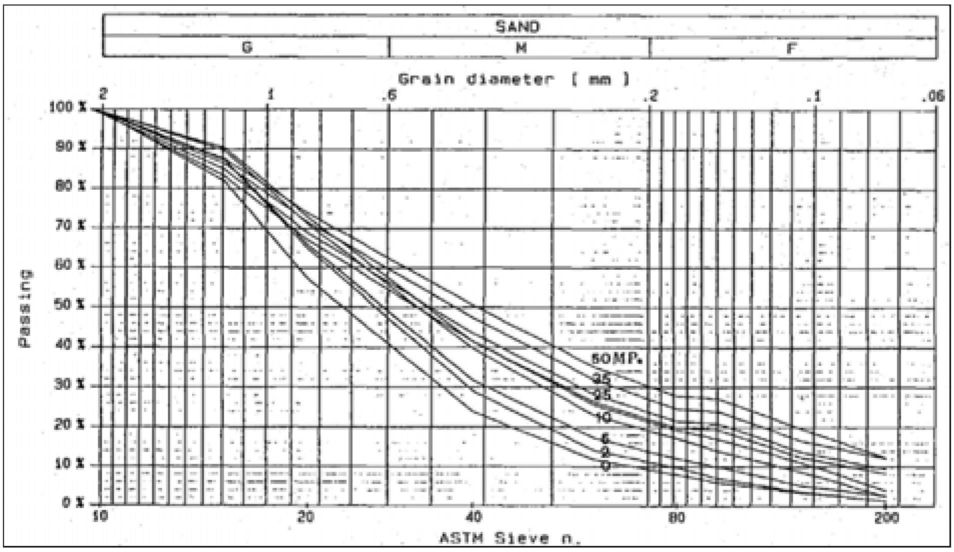


Figure 60: PSD of Quiou Sand in oedometer tests with consolidation pressure 1 MPa up to 50 MPa

For crushable materials, it is advised to determine the maximum density by means of a vibratory table. In the vibratory table test (ASTM D4253), which can be executed both wet and dry, a specific amount of material is put into a cylinder, which is then placed on the vibrating table. The amplitude of the vibration, time of vibrating and surcharge are specified in the standard.

Also quality control by CPT-measurements is not evident in crushable sands. First of all, a high degree of crushing occurs during penetration of the cone. In carbonate sands, the threshold mean stress level to start relevant crushing remains well within CPT cone values. However, it should be noticed that these typical cone resistances, measured at any reclamation area, are commonly 50 to 500 times higher than the future maximum loads on such type of site. Consequently, the crushing can be expected to be much more pronounced than under the common real loading. Also here, two different situations are in fact compared to each other.

Furthermore, commonly used correlations between cone resistance q_c and relative density Dr are only valid for (non-crushable) quartz-sands. It is generally accepted that for the same relative densities, the cone resistance q_c will be lower in crushable sands compared to silica sands. This implies that new calibrations are required in order to define the site specific (or even material specific) relationship between cone resistance and relative density. This can be done by calibration chamber testing (cf. e.g. Wehr, 2005). The calibration chamber testing eventually results in a so-called shell factor or correction factor, which defines the ratio between cone resistances measured in the site specific sand and cone resistances measured in silica sand (cf. formula 32). The set-up, execution and interpretation of these calibration chamber tests often require academic assistance and considerable amount of time. Therefore, on-site calibration chamber testing by the contractor is not always evident (cf. Figure 61). Depending on the amount of calcium carbonate in the sand, the obtained shell factors can vary significantly; typical values from practice range between ca. 1,3 and 2,2.

$$f_{shell} = \frac{q_{c,silica}}{q_{c,carbonate\ sand}} \approx 1{,}3 - 2{,}2 \qquad (32)$$

Figure 61: Typical set-up for onsite calibration chamber testing

For the evaluation of liquefaction potential of carbonate sands, the application of a shell factor is questionable and might lead to even more uncertainty. To the contrary, it is known that carbonate sands are also characterized by a pronounced angularity, which is an important advantage to resist liquefaction. Also the chemical composition of the grains allows for a further chemical binding (cementation) in between the grains, which hinders the processes leading to liquefaction.

The major problem in quality control of ground improvement methods in carbonate sands remains the definition and practical testing of the relative density of the sand. As a solution, one could chose to leave the principle of relative density and use void ratio or state parameter for evaluation of compaction degree and liquefaction potential. The negative side is that in this case a lot of testing is required and therefore might this approach only be applicable when dealing with very large projects. Another solution might be to change the specifications to stiffness testing (e.g. seismic cone penetration test and SASW) or change the specifications to performance testing (e.g. zone load test).

5.2.4 Quality control for other ground improvement techniques

5.2.4.1 Dynamic compaction

The quality controls to carry out on DC works consists in monitor several parameters: energy per print by heave penetration tests, soil behaviour by settlement measurements, soil liquefaction in fine saturated soils by pore water pressure measurements, improvement of soil mechanical characteristics by in situ tests (pressuremeter, penetrometer) an load tests, and damages to environment by vibration measurements. Indeed, the propagation of surface Rayleigh waves generated by DC impact causes vibrations to neighbouring structures. The aim of vibration measurement is to estimate the particle velocity and thus check this does not exceed limiting values. The Particle Velocity (PV) is the measure used to assess the possible damages to existing structures. The Peak Particle Velocity (PPV) is the maximum PV during an event. The French Codes (1987) gives limiting values of PPV for structures as presented in Figure 62. The frequency of vibrations induced by DC impact usually ranges from 8 to 20 Hz. Some empirical correlations have been developed by MENARD from site vibration measurements between distance from impact and particle velocity (see Figure 63).

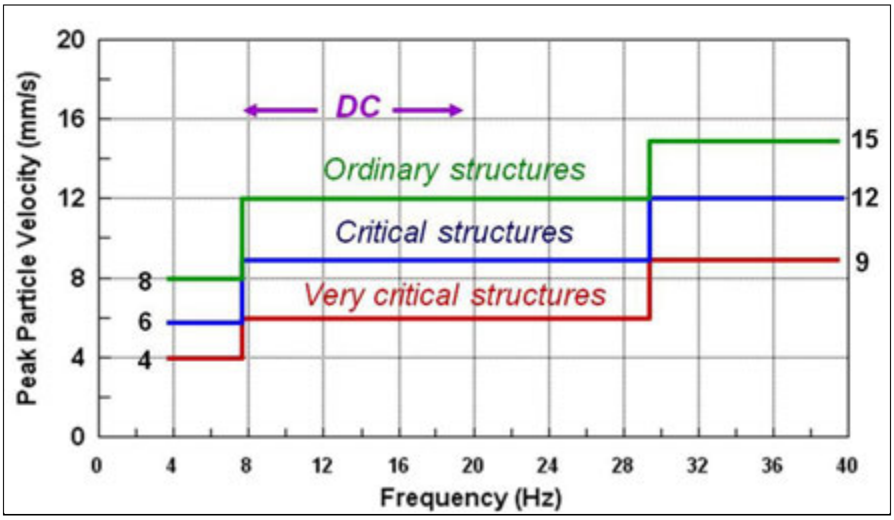


Figure 62: Influence of DC on environnement

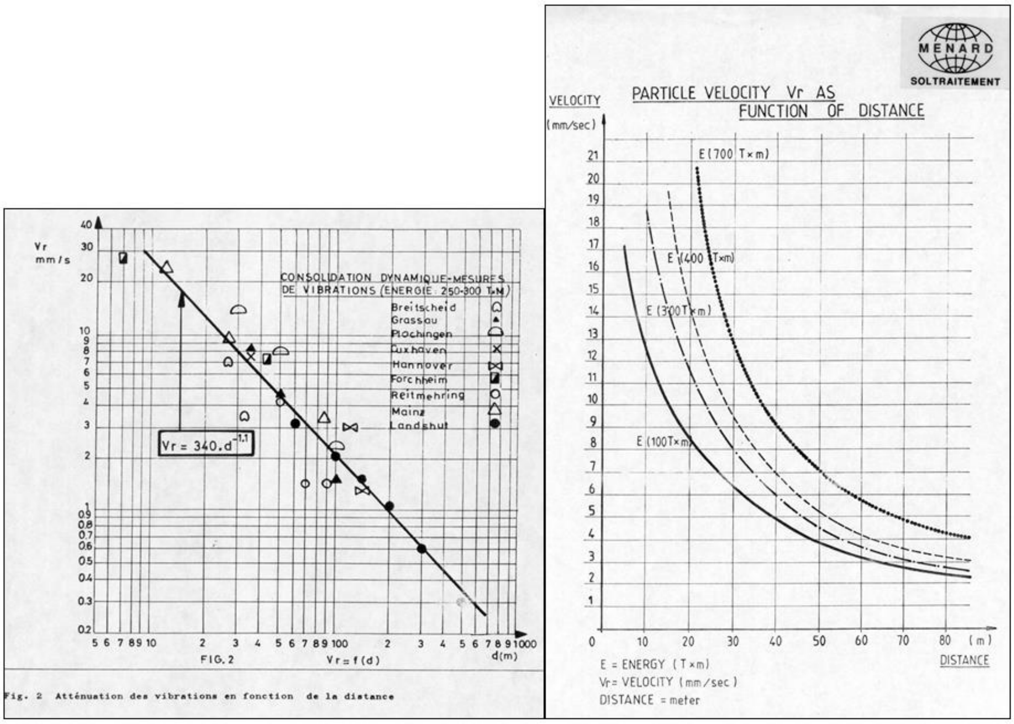


Figure 63 : Empirical correlations from site measurements

A possible solution which enables to reduce the effects related to vibration waves on existing structures is to dig a trench. The main purposes of the trench located ahead of the structures are to accumulate vibrations and thus prevent them from damaging the structures.

5.2.4.2 Vibrocompaction

In general, the acceptance criteria of the project rely on the increase of cone resistance or relative density and if necessary bathymetric surveys to check settlement reached for offshore works.

The quality control are carried out during and after compaction works. During compaction, the following parameters are recorded: depth, amperage or hydraulic pressure, void closure (strain), sand or gravel consumption per compaction point, time of treatment, verticality of the probe (facultative) and vibrations on existing structures in close vicinity. The quality controls after compaction works consists in monitor: soil behaviour by settlement measurements and improvement of soil mechanical characteristics by in situ tests (Cone Penetration Tests, Standard Penetration Tests or rarely Pressuremeter Tests).

Before vibrocompaction works, compaction test trials should be carried out to determine the optimal grid, check the densification, the compactibility and the liquefaction potential of the soils, and define production parameters (jetting method: water or air and pressure, compaction time and steps height) to apply during works.

The improvement of soil properties occurs sometimes after a waiting time (10 to 15 days), which depends on particle grain size and method of treatment (air and/or water). This event is named “ageing”.

5.2.4.3 Rapid Impact Compaction

The quality controls are carried out during and after compaction works. During compaction works, the use of a GPS-Logger allows to know the location, count the number of blows, and measure the settlement and the induced settlement corresponding to the settlement per blow. The quality controls after compaction works consists in monitor several parameters: soil behaviour by settlement measurements (logger data), improvement of soil mechanical characteristics by in situ tests (Cone Penetration Tests, Standard Penetration Tests or Pressuremeter Tests), seismic test (MASW) or plate load tests. After a compaction phase, logger data is assessed for requirement of eventual additional pass.

Before the RIC works, compaction trials should be used to finalize design and to determine stop criterion during compaction works. From quality controls after compaction, a site specific correlation between induced settlement (or settlement rate) and site investigation results (for instance cone resistance value qc) can be established.

The Rapid Impact Compaction produces limited vibrations whose levels are lower than those generated by Dynamic compaction. This allows working closer to existing structures. Indeed, the adjustment of fall height and tamper diameter reduces the vibration levels. Nevertheless, the vibrations tend to increase with the number of blows, the densification of the soil and a heavier weight.

Using a falling dropweight of 16 tons, the peak particle velocities have been measured to less than 20 mm/s at 10 m from centre of foot and less than 10 mm/s at 15-20 m from centre of foot.

A comparison of the compaction techniques in terms of depth of influence, fines content limit and production per rig is shown in Table 9.

Table 9: Comparison compaction techniques

	Depth of influence	***Fines content limit***
Dynamic Compaction	10-15 m	~ 15%
Rapid Impact Compaction	5-7 m	~ 10-15%
Dynamic Replacement	5-7 m	No fines content limit within in-situ soils (new material < 10% fines)
Vibroflotation	30 m	~ 10%
Roller compaction	0.5 m (2 m HEAC)	~ 10-15%

5.2.4.4 Prefabricated vertical drains, surcharge and vacuum consolidation

The selection of PVD is controlled by laboratory test on the drain itself (straight drain discharge capacity tests, Buckled drain test, tensile strength tests, permeability test of filter) as well as on the soil (permeability tests). Therefore, one cannot borrow specifications without considering the site conditions and the nature of the project. The following parameters must be also recorded during installation of vertical drains: depth, location and verticality. The evolution of several geotechnical parameters are monitored throughout the consolidation period. The different instruments used to validate the design and the phasing of the embankment construction are: settlement plates and multi-gauge settlement sensors (for multi-layers) monitoring settlements which enable to assess the achieved consolidation rate (Asaoka or hyperbolic methods); pore pressure sensors monitoring pore pressures which enable to assess the coefficients of vertical and radial consolidation in order to check design basis (failure if u exceeds the load applied); and inclinometers monitoring horizontal movements to check slope stability.

Chu (2004) reported in a book some useful design (approach of using a higher surcharge to shorten consolidation time) and practical considerations (selection, quality control tests, selection of design parameters and smear effect) in the use of PVDs in soil improvement projects experiences.

For the successful implementation of a drain project the design must take into consideration many factors such as the site and soil conditions, the client's requirements, the quality control of the drains, the method of installation, the experience of the contractor and the evaluation and interpretation of the soil instrumentation, laboratory and in-situ test data. A holistic approach to drain design has therefore to be adopted and experience plays an essential role in achieving the desired results.

The quality control used for PVD consolidation is applicable for vacuum consolidation. Additional controls must be checked during installation: horizontal drains (locations, continuity and depth), membrane and pumps. During pumping, the monitoring for vacuum comprises vacuum gauges for each pump and below the membrane which enable to check respectively the vacuum pumps depression and the depression increase in the ground.

5.2.4.5 Stone columns

The following parameters should be recorded during SC works: depth, amperage or hydraulic pressure, time of treatment, stone volume and diameter of columns (at the head). After execution, the typical test is: load test on column head to verify bearing capacity of the column. The successful completion of the column may be verified by in situ tests (Cone Penetration Tests, Standard Penetration Tests or Pressuremeter Tests).

5.2.4.6 Dynamic replacement

The quality controls to carry out on DR works are summarized in Table 10.

Table 10: Quality controls for Dynamic Replacement project

Parameters to monitor	**Measurement techniques**
Energy per print	Heave penetration tests
Replacement ratio	Stone volume measurements
Depth and mechanical characteristics in pillars	In situ tests (CPT, PMT)
Mechanical characteristics in surrounding soil	In situ tests (CPT, PMT)
Bearing capacity of a pillar	Load tests
Damages to environment	Vibration measurements

5.2.5 Quality control based on seismic waves

As an alternative for the classic compaction control techniques, quality control by seismic waves is sometimes applied, especially when dealing with more difficult soil types like crushable sands or soft clays (cf. Dong-Soo et al., 2012).

The principle behind the testing is that velocities of seismic shear waves propagating through the compacted fill are measured. As indicated by formula (33), these shear wave velocities (v_S) are directly related to the small strain shear modulus G_0 and are sensitive to a specific soil type through the density ρ. Furthermore, also good correlations exist with other geotechnical parameters like SPT-N value and CPT cone resistance q_c. Since G_0 and v_S have a quadratic relationship, it is obvious that a precise registration of the velocity is of crucial importance.

$$G_0 = \rho \times v_S^2 \qquad (33)$$

One technique in seismic quality control is the evaluation of the compaction by means of spectral analysis of surface waves (SASW). The essence of this test is measuring the propagation of Rayleigh type surface waves (cf. Figure 64). From this velocity measurement, the stiffness of the subsurface to a certain depth can be calculated. Also correlations between shear wave velocities and in-situ densities exist. Since during the execution of the test, both the source and receivers are located on the ground surface, the method is cost-effective and well-suited for in-situ testing of hard-to-sample soils such as those used in structural fills, pavement bases, and other engineered fills with coarse-grained soils

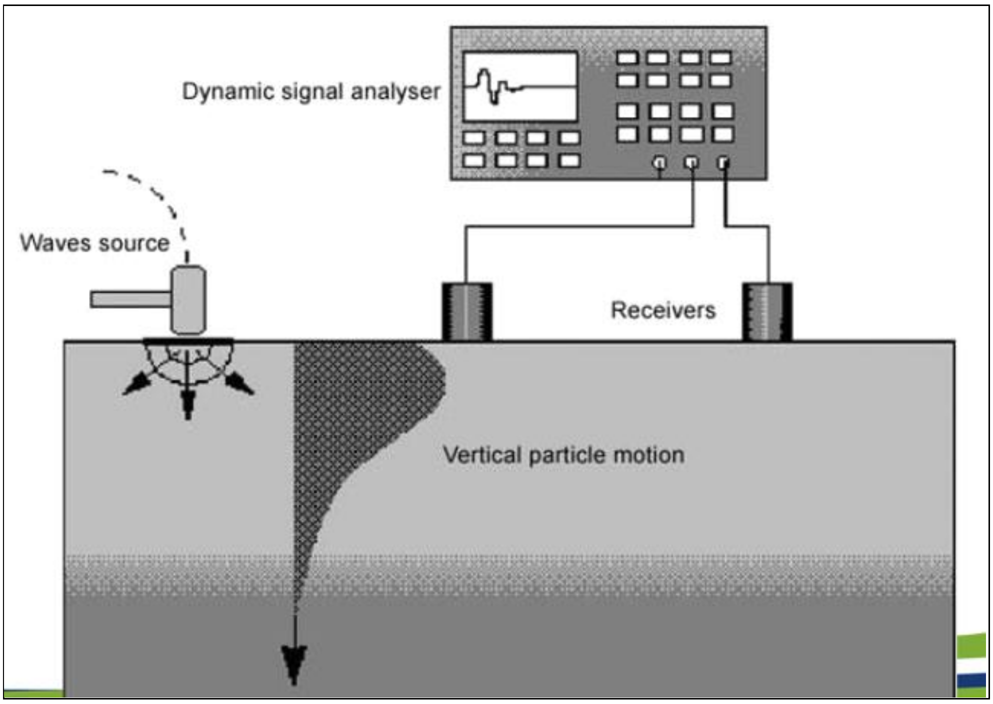


Figure 64: Schematic diagram of SASW testing

In recent years, the application of (Continuous) Seismic Cone Penetration ((C)-SCPT) test for quality control of ground improvement works is gaining popularity. For the execution of an SCPT, a seismic cone is pushed into the ground just like with a normal CPT. At the depth of interest, or at regular intervals, the penetration is paused. At that time, a seismic source is triggered at the surface and sends seismic waves through the soil which are recorded by the sensors installed in the seismic cone. Not only the pauses at regular intervals, but especially the need to unclamp the CPT string for each test and minimize the interference of other possible noise sources (e.g. shutting down the engine of the truck/drilling machine) , makes the SCPT a time consuming test. The unclamping of the string is required in order to prevent that seismic waves do not travel directly through the CPT-rods to the receiver. The development of the C-SCPT makes this test much more time efficient. In the C-SCPT configuration seismic source waves are generated as the penetrometer is pushed into the ground without stopping to unclamp the string, turning the rig/truck engine off and generating seismic waves at the surface. The fact that seismic shear waves can directly derive the small-strain rigidity of the subsoil and the fact that the speed of the seismic waves is sensitive to the encountered soil type and shows good correlations with other geotechnical parameters, makes their application for quality control very promising. Verifying the liquefaction potential of a granular fill is a classic example. Further optimization of the C-SCPT testing method itself and the corresponding data processing makes it very likely that these techniques will be more and more applied in practice for quality control.

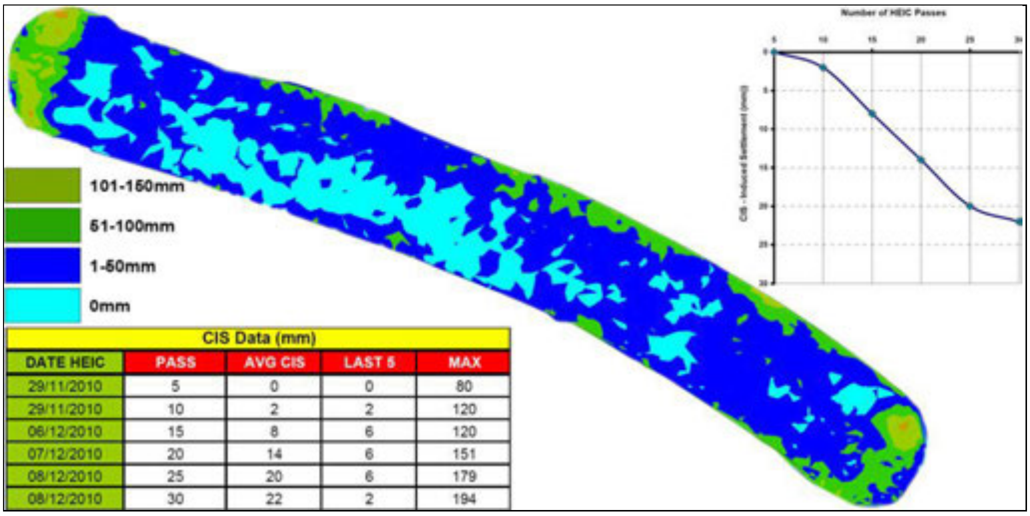
5.2.6 Other recent advances

Traditional approaches to compaction quality control have several limitations: the small ratio between the tested volume of soil and the total treated volume of soil, the lack of correlation between laboratory and field compaction test results, poor reproducibility of the results, long duration of certain testing methods, etc. Therefore, new possibilities and techniques are looked for. The techniques based on velocity measurements of seismic waves are already discussed above. However, there are also other examples.

Another good example is the emergent quality control method for twin drum High Energy Impact Compaction (HEIC) developed by Landpac (cf. Kelly and Gill, 2012). The principle of HEIC is the transfer of falling kinetic energy to the soil upon impact of the non-circular rotating mass. This kinetic energy transfer will generate a compaction of the soil at the point of impact. The Continuous Impact Response (CIR) and Continuous Induced Settlement (CIS) measurement systems are relatively recent measurement systems for quality control of the generated compaction.

The technique behind the CIR measurement system is that at each impact of the compaction masses, the peak deceleration is measured. These measurements are correlated back to particular engineering properties using traditional testing methods like density determination, CPT, DCP, PLT, ZLT, CBR,… Each measurement point is also recorded relative to its position on site by an integrated GPS system. In that way, 'ground improvement maps' can be generated which allow to indicate the relative strength of the various stages of the ground improvement process and monitor the progress between the various stages. Because of the correlation with traditional techniques, it also allows to spread the conventional test results over the entire site. The maps also allow highlighting weak areas which need further treatment.

Simultaneously with the deceleration measurement, the relative settlement which is induced by the compaction progress can be measured. Also here it is possible to generate maps which give an overview of the Continuous Induced Settlements (CIS) on site (cf. Figure 65). These maps allow indicating the continuous settlement throughout the process, indicating relative settlements on site, monitoring areas and volumes and monitoring absolute level of the threatened area.



CIS Data (mm)				
DATE HEIC	PASS	AVG CIS	LAST 5	MAX
29/11/2010	5	0	0	80
29/11/2010	10	2	2	120
06/12/2010	15	8	6	120
07/12/2010	20	14	6	151
08/12/2010	25	20	6	179
08/12/2010	30	22	2	194

Figure 65: Typical CIS data for each batch of 5 HEIC surface coverages (to a max of 30). Data from the London Gateway Container Terminal project in the UK (from Kelly and Gill, 2012)

The combination of CIR and CIS monitoring techniques allows the contractor to optimize the ground improvement process. The continuous measurements clearly indicate when the required levels of compaction are met or where an extra effort is needed. These maps are also excellent for presentation of the quality monitoring towards the client; they give a good overview of the achieved results at the different phases over the entire reclamation area.

6. CONCLUSION

Ground improvements are more and more needed for: support of projects constructed on poor soils or fills, construction method of hydraulic fill, mitigation of liquefaction in case of earthquake and reuse of dredging material. Indeed, ground improvements methods enable to avoid moving projects towards another site and are in most cases cost-saving alternatives to traditional deep foundations and soil substitution. Different methods can be suitable for a same project; however a solution of combined soil improvement methods can be more relevant to cope with different issues on site than only one. Technical knowledges and development of equipments evolve and progress over time as the bid growths.

Ground improvement methods are very miscellaneous and can be classified in two categories:

- The first category gathers ductile ground improvement methods, that is to say methods without material added or with granular inclusion (like drains, dynamic compaction, stone columns... as shown in Table 3). The ratio of modulus between granular columns and soil does not exceed 5 to 15.

- The second category contains ground improvement methods with semi-rigid to rigid inclusions (like CMC, Soil Mixing columns... as shown in Table 3). The columns diameter is usually more 30 cm. In the design, deformations are considered with a sufficient safety factor against failure. Design for bearing capacity of the inclusion is conducted using for example the Brinch-Hansen approach or recently ASIRI recommendations.

It has to be noted that the ground investigation campaign should be not understated. Indeed, the appraisal of soil behaviour enables to:
- determine if a soil improvement solution is whether or not feasible,
- guide towards the most suitable ground improvement technique,
- define accurately the soil parameters required to the soil improvement design.

REFERENCES

Baldi G., Belloti V., Ghionna N., Jamiolkowski M. & Pasqualini E., 1986, Interpretation of CPT's and CPTU's, 2nd Part: Drained Penetration of Sands, Proceedings of 4th International Geotechnical Seminar Field Instrumentation and In-Situ Measurements, Singapore, pp. 143- 156.

Barron R.A., 1948, Consolidation of fined-grained soils by drain wells, Transactions ASCE, vol. 113, pp. 718-754.

Brinch Hansen J., 1970, A revised and extended formula for bearing capacity, Bulletin N° 28, Danish Geotechnical Institute, Copenhagen (Denmark), pp. 5-11.

Chai J.C., Carter J.P. & Hayashi S., 2005, Ground deformation induced by vacuum consolidation, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 131, N° 12, pp. 1552-1561.

Chu J., Varaksin S., Klotz U. and Mengé P., 2009, State of the Art Report: Construction Processes, 17th International Conference on Soil Mechanics and Geotechnical Engineering, M. Hamza et al. (Eds.), IOS Press, pp. 3006-3135.

Datye K. R., 1982, Settlement and Bearing Capacity of Foundation System with Stone Columns. Proceedings of Symposium on Recent Developments in Ground Improvement Techniques, Bangkok (Vietnam), pp. 85-103.

DEME photo archive

Fookes, P.G., Baynes, F. and Hutchinson, J.N., 2000, Total geological history: a model approach to the anticipation, observation and understanding of site conditions, Proceedings of the International Conference on Geotechnical and Geological Engineer, GeoEng 2000, vol 1, Technomic, Pennsylvania, 370-421.

Ghionna V., Jamiolkowski M., 1981, Colonne di ghiaia, X Ciclo di conferenze dedicate ai problemi di meccanica dei terreni e ingegneria delle fondazioni metodi di miglioramento dei terreni, Politecnico di Torino Ingegneria, atti dell'istituto di scienza delle costruzioni, n° 507 (in italian).

Goughnour R.R. & Bayuk A.A., 1979, Analysis of stone column-soil matrix, Interaction under vertical load, Proceedings of International Conference on Soil Reinforcements, Paris (France), Vol. 1, pp. 271-277.

Hamidi B., 2011, (In)Validity of relative density for quality control of cohesionless soils, TC211 workshop 26/5/2001, 14the Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Hong Kong (available on TC211 website)

Hansbo S., 1981, Consolidation of fine-grained soils by prefabricated drains, Proceedings of 10th International Conference on Soil Mechanics and Foundation Engineering, Stockholm (Sweden), Vol. 3, pp. 677-682.

Hird C.C., Pyrah I.C. & Russell D., 1992, Finite element modelling of vertical drains beneath embankments on soft ground, Gotechnique, Vol. 42, N° 3, pp 499-511.

Indraratna B. & Redana I. W., 1997, Plane strain modelling of smear effects associated with vertical drains. J. Geotechnical Engineering, ASCE, Vol. 123, N° 5, pp. 474-478.

Indraratna B. & Redana I.W., 2000, Numerical modelling of vertical drains with smear and well resistance installed in soft clay, Canadian Geotechnical Journal, Vol. 37, N°1, pp 137-145.

Indraratna B., Sathananthan I., Rujikiatkamjorn C. & Balasubramaniam A.S., 2005e, Analytical and numerical modelling of soft soil stabilized by PVD incorporating vacuum preloading, International Journal of Geomechanics, ASCE, Vol. 5, N°2, pp. 114-124.

Institut pour la recherche appliquée et l'expérimentation en génie civil (IREX), 2012, Recommendations for the design, construction and control of rigid inclusion ground improvements, 384 p, Presse des Ponts.

Institution of Civil Engineers, 1987, Specification for ground treatment: Notes for guidance, London, Thomas Telford.

Jamiolkowski M., Ladd C.C., Germaine J.T. & Lancellotta R., 1985, New developments in field and laboratory testing of soils, Proceedings of 11th International Conference on Soil Mechanics and Foundation Engineering, San Francisco (California), Vol. 1, pp. 57-153.

Kelly, D. and Gil, J., 2012, Monitoring HEIC using Landpac CIR and CIS Technologies, Proceedings of the ISSMGE - TC 211 International Symposium on Ground Improvement, Vol I, pp. 273-285.

Kim, D.-S. Kim, Young, J.-U. & Park, H.-J., 2012, Session report: Applications of shear wave velocity on various geotechnical problems, Geotechnical and Geophysical Site Characterization 4, Coutinho, R. Q. & Mayne P. W. (eds), CRC Press, pp. 661-684.

Kitazume M., 2005, The Sand Compaction Pile method, Taylor & Francis, 232p.

Massarch K.R. & Heppel G., 1991, Deep Vibratory Compaction using the Muller Resonance Compaction System, Marburg (Germany), Report 91, N° 2, Muller Geosystems.

Masse F., Spaulding C. A., Wong I. C. & Varaksin S., 2001,Vacuum consolidation: A review of 12 years of successful development, ASCE Geo-Odyssey, VIRGINIA TECH-Blacksburg, VA USA.

Mengé, P., 2012, Louis Menard Lecture - Recent Advances and Execution Aspects in Ground Improvement in Dredging and Environmental Marine Engineering, Proceedings of the ISSMGE - TC 211 International Symposium on Ground Improvement, Vol I, pp. 189-230.

NF EN 1998-1 (P 06-030-1), Eurocode 8, Design of structures for earthquake resistance, Part 1: « General rules, seismic actions et rules for buildings ».

NF EN 1998-5 (P 06-035-1), Eurocode 8, Design of structures for earthquake resistance, Part 5: « Foundations, retaining structures and geotechnical aspects ».

Oshima A. & Takada N., 1997, Relation between compacted area and ram momentum by heavy tamping, Proceedings of 14th International Conference on Soil Mechanics and Foundation Engineering, Hamburg (Germany), Vol. 3, pp. 1641–1644.

Port and Harbour Research Institute Japan (eds), 1997, Handbook on Liquefaction Remediation of Reclaimed Land, A.A. Balkema, Rotterdam.

Priebe H.J., 1976, Abschactzung des setzungsverhaltns eiens durch stopfverdichtung verbesserten baugrundees, Die Bautechnik. Vol. 53, N° 5, pp. 160-162 (in german).

Priebe H.J., 1995, The design of Vibro replacement, Ground engineering, Technical paper GT 073--13 E.

Qian J.H., Zhao W.B., Cheung, Y.K. & Lee, P.K.K., 1992, The theory and Practice of Vacuum Preloading, Computers and Geotechnique, Vol. 13, pp. 103-118.

Raithel M. & Kempfert H.G., 2000, Calculation Models for Dam Foundations with Geotextile Coated Sand Columns, Proceedings of International Conference on Geotechnical & Geological Engineering, Melbourne (Australia).

Raithel M., Kirchner A., Schade C. & Leusink E., 2005, Foundation of Constructions on Very Soft Soils with Geotextile Encased Columns, State of the Art, Innovations in Grouting and Soil Improvement, ASCE, Austin (Texas),Geotechnical Special Publication N° 136.

Raju V.R., 2011, Ground Improvement using Vibro Techniques in Fly Ash Deposits, proceedings of National Conference on Recent Advances in Ground Improvement Techniques, CBRI Roorkee, India.

Robertson P.K., Campanella R.G., Gillespie D. & Greig J., 1986, Use of Piezometer Cone data, Proceedings of American Society of Civil Engineers, ASCE, In-Situ 86 Specialty (Use of In Situ Tests in Geotechnical Engineering), Blacksburg (Virginia), Geotechnical Special Publication GSP N° 6, pp. 1263-1280.

Rujikiatkamjorn C., 2005, Analytical and numerical modelling of soft clay foundation improvement via prefabricated vertical drains and vacuum preloading, Thesis, University of Wollongong.

Rujikiatkamjorn C. & Indraratna, B., 2007, Analytical solutions and design curves for vacuum-assisted consolidation with both vertical and horizontal drainage, Canadian Geotechnical Journal, Vol. 44, pp. 188-200.

Sathananthan I., 2005, Modelling of vertical drains with smear installed in soft clay, Thesis, University of Wollongong.

Schmertmann J.H., 1976, An Updated Correlation Between Relative Density and Fugro-type Electric Cone Bearing qc, Waterways Experiment Station, Miss. Contract report, DACW 38-76-M 6646. Vicksburg, 145 p.

Seed H.B. & Idriss I.M., 1971, Simplified Procedure for Evaluating Soil Liquefaction Potential, Journal of Soil Mechanics & Foundations Division, ASCE, Vol. 97,pp 1249-1273.

Terzaghi K., 1925, Erdbaumechanik auf bodenphysikalischer Grundlage, Franz Deutiche Publishers, Leipzig und Wien, Germany.

van 't Hoff J and Nooy van der Kolff A., 2012, Hydraulic Fill Manual, CUR and CIRIA, CRC press.

Wehr J., 2005, Influence of the carbonate content of sand on vibro compaction, 6th International Conference on Ground Improvement Techniques, Coimbra, Portugal.

Yee K., 2011, On-shore and off-shore compaction for a reclamation projects, TC211 workshop 26/5/2001, 14the Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Hong Kong (available on TC211 website),

Youd T.L., Idriss I.M., Andrus R.D., Arango I., Castro G. Christian J.T., Dobry R., Finn W.D.L., Harder L.F., Hynes M.E., Ishihara K., Koester J.P., Liao S.S.C., Marcuson W.F., Martin G.R., Mitchell J.K., Moriwaki Y., Power M.S., Robertson P.K., Seed R.B. and Stokoe K.H., 2001, Liquefaction resistance of soils: summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils, ASCE, JGTGE, Vol. 127, N° 10, October.

Zhou J., Wang J.Y. & Jia M.C., 2008, Recent development in the vibroflotation without backfill techniques, Rock and Soil Mechanics, Vol. 29, N° 1, pp 37-42 (in Chinese).

SUMMARY OF THE SHORT COURSES OF THE IS-GI 2012
LATEST ADVANCES IN DEEP MIXING

SUMMARY OF THE SHORT COURSES OF THE IS-GI 2012 LATEST ADVANCES IN DEEP MIXING

Nicolas Denies and Gust Van Lysebetten
Belgian Building Research Institute (BBRI), Geotechnical division, Belgium, nde@bbri.be and gvl@bbri.be

ABSTRACT

The present Summary highlights the significant contributions of the Short Courses on Deep Mixing coordinated under the guidance of J. Maertens, N. Denies and N. Huybrechts and organized within the framework of the IS-GI Brussels 2012. This Summary concentrates on the latest developments and current researches in the deep mixing method (DMM). The content of all the presentations is harmonized and structured as follows. Different ***execution processes*** *are summarized or classified and their mechanisms are outlined. The recent advances related to the mechanical characterization of the* ***soil mix material*** *are then discussed. Various* ***applications*** *of the technique are illustrated with the help of case histories focusing on ground improvement (GI) works but also on structural and environmental uses of the DMM: foundations and soil reinforcement, earth/water retaining structures and cut-off walls, land levees and floodwalls, in situ remediation, etc. Finally, QA/QC procedures are briefly discussed and the importance of construction monitoring is underlined. Many references on the topic of DMM are also given in the Summary.*

1. INTRODUCTION

The purpose of the present Summary is to provide an overview of the significant contributions of the Short Courses on Deep Mixing which took place during the International Symposium on Ground Improvement in Brussels in Mai 2012. These Short Courses were coordinated under the guidance of J. Maertens, N. Denies and N. Huybrechts and organized under the auspices of the ISSMGE TC211.
In total, 12 Short Courses were given, as summarized in Table 1. These presentations have been built with the aim to provide a complete overview of the Deep Mixing Method (DMM) including information over its historical development, the various soil mix equipment, the produced Deep Soil Mix (DSM) material, the areas of application illustrated with the help of case histories. But the Short Courses not only concentrate on the construction principles of the method but also on the design aspects and on the QA/QC activities related to this process. The present document completes the General Report of the Deep Mixing Session (Denies and Van Lysebetten, 2012) available in the Proceedings of the IS-GI Brussels 2012 and summarizing the content of 28 different technical papers dedicated to the DMM, as listed in Table 2.

Table 1: Presentations given within the framework of the Short Courses on Deep Mixing during TC211 IS-GI Brussels 2012, references are underlined in the present text

General Overview	Topolnicki, Keller, Poland
Innovation in Soil Mix Technology for Contaminated Land Remediation	Al-Tabbaa, University of Cambridge, UK
Overview CSM equipment	Gerressen, Bauer, Germany
Soil Mixing Equipment	Borel, Soletanche Bachy, France
Dry Soil Mixing – Liebherr Equipment	Quasthoff, Liebherr, Germany
Deep Mixing – BBRI research activities	Denies, BBRI, Belgium
Deep Mixing research – KU Leuven	Vervoort and Van Lysebetten, KU Leuven, Belgium
Mechanical behaviour of cement-treated clay	Verástegui Flores, Ghent University, Belgium
Monitoring of deep mixing structures with optical fiber technology	Huybrechts, BBRI, Belgium
Deep Mixing Support for Embankments, Levees, and Floodwalls	Filz, Virginia Tech, USA
Design of Deep Mixing for Structural Cutoff Walls for Excavation Support	Weatherby, Schnabel Foundation Company, USA, presented by Filz, Virginia Tech, USA
Mixing is the future – Case Study	Leemans, Soetaert-Soiltech, Belgium

Table 2: List of papers dealing with the DMM in the proceedings of the TC211 IS-GI Brussels 2012, referenced in italic in the present text

Partial Factor Design for a Highway Embankment Founded on Lime-cement Columns	*Al-Naqshabandy and Larsson (2012)*
Soil Mix Technology for Integrated Remediation and Ground Improvement: Field Trials	*Al-Tabbaa et al. (2012)*
Long-term performance of CSM walls in slightly overconsolidated clays	*Bellato et al. (2012)*
Geomix Caissons against liquefaction	*Benhamou and Mathieu (2012)*
Foundation Soils Improvement by "Cutter Soil Mixing"	*Bilé Serra and Mendes (2012)*
Ground improvement works for an LNG storage tank foundation	*Chapman et al. (2012)*
Quality Assurance and Quality Control for Deep Soil Mixing (DSM) in Punggol Waterway Project, Singapore	*Chew et al. (2012)*
SOIL MIX WALLS as retaining structures – Belgian practice	*Denies et al. (2012a)*
SOIL MIX WALLS as retaining structures – mechanical characterization	*Denies et al. (2012b)*
Mechanical characterization of DEEP SOIL MIX material – procedure description	*Denies et al. (2012c)*
Mechanical characterization of large scale soil mix samples and the analysis of the influence of soil inclusions	*Vervoort et al. (2012)*
Foundations reinforced by soil mixing: Physical and numerical approach	*Dhaybi et al. (2012)*
Design, Construction and Monitoring of a Test Section for the stabilization of an Active Slide Area utilizing Soil Mixed Shear Keys installed using Cutter Soil Mixing	*Gaib et al. (2012)*
CSM-Cutter Soil Mixing – Worldwide experiences of a young soil mixing method in challenging soil conditions	*Gerressen and Vohs (2012)*
Deep mixing for reinforcement of railway platforms with a spreadable tool	*Guimond-Barrett et al. (2012)*
Soil-cement columns, an alternative soil improvement method	*Lambert et al. (2012)*
Soil mixing in highly organic materials: the experience of LPV111, New Orleans, Louisiana (USA)	*Leoni and Bertero (2012)*
Stability Analyses of a Floodwall with Deep-Mixed Ground Improvement at Orleans Avenue Canal, New Orleans	*McGuire et al. (2012)*
Assessing the feasibility of a foundation treatment solution based on CSM panels at a river dock in Lisbon	*Mendes et al. (2012)*
Earth Retaining Structure using Cutter Soil Mixing technology for the "Villa Paradisio" Project at Cannes, France	*Peixoto et al. (2012a)*
Permanent Excavation Support in Urban Area using Cutter Soil Mixing technology at Cannes, France	*Peixoto et al. (2012b)*
Solutions for soil foundation improvement of an industrial building using Cutter Soil Mixing technology at Fréjus, France	*Peixoto et al. (2012c)*
Solution of earth retaining structure using Cutter Soil Mixing technology: "Parking Saint Nicolas" Project at Cannes, France	*Peixoto et al. (2012d)*
The application of Cutter Soil Mixing to an urban excavation at the riverside of Lagos, Portugal	*Peixoto et al. (2012e)*
Ground Improvement Solutions using CSM Technology	*Pinto et al. (2012)*
State of the art in "Dry Soil Mixing" – Basics and case study	*Quasthoff (2012)*
Parametric study of embankments founded on soft organic clay using numerical simulations	*Suganya and Sivapullaiah (2012)*
Design of in-situ soil mixing	*Topolnicki and Pandrea (2012)*

As related in the Short Course of Topolnicki (2012), the Deep Mixing Method (DMM) was introduced in the 1960's in Japan and in the Scandinavian countries. Indeed, after a first use in the 1950's, the method had to wait the 1980's to impose on the American market as a GI technique. In Europe, initially considered as an alternative to the jet grouting application, the DMM made its entrance in the late 1980's with the emergence of various DSM systems. After the development of various DSM column configuration systems, the market was the witness of the emergence of several systems: the mass stabilization, the trenchmixing (in the beginning of the 1990's) and the Cutter Soil Mix (CSM) in 2003. Historical development of the DMM around the world is also fully described in Bruce et al. (1998) and Topolnicki (2004). Kitazume and Terashi (2013) concentrate on the historical review of DMM in Japan.

According to the classification of GI methods adopted by the ISSMGE TC 211 Ground Improvement, formerly TC 17, DMM can be classified as ground improvement with grouting type admixtures, as illustrated in Table 3 (after Chu et al. 2009). Porbaha (1998) has notably proposed a terminology for the DSM technology, as presented in Table 4. A lot of reviews describing various deep mixing methods are available in Terashi (2003), Topolnicki (2004), Larsson (2005), Essler and Kitazume (2008) and more recently in Denies and Van Lysebetten (2012). Specialty international conferences have been held in Tokyo (1996), Stockholm (1999), Helsinki (2000), Tokyo (2002), New Orleans (2003), Stockholm (2005), Osaka (2009) and New Orleans (2012) with a large audience bear witnessing of the large worldwide success of the method. In parallel, the results of national and European research programs have been published in multiple interesting reports (such as CDIT, 2002 and Eurosoilstab, 2002), while also the European standard for the execution of deep mixing "Execution of special geotechnical works – Deep Mixing" (EN 14679) was published in 2005. Most of these research projects focused on the global stabilization of soft cohesive soils such as silt, clay, peat and gyttja (result of the digestion of the peat by bacteria). Nevertheless, as illustrated hereunder, the applicability of the method in sandy soils for structural applications can be no more put in doubt.

Table 3: Classification of GI methods adopted by TC211, formerly TC 17 (after Chu et al., 2009)

D. Ground improvement with grouting type admixtures	D1. Particulate grouting	Grout granular soil or cavities or fissures in soil or rock by injecting cement or other particulate grouts to either increase the strength or reduce the permeability of soil or ground.
	D2. Chemical grouting	Solutions of two or more chemicals react in soil pores to form a gel or a solid precipitate to either increase the strength or reduce the permeability of soil or ground.
	D3. Mixing methods (including premixing or deep mixing)	**Treat the weak soil by mixing it with cement, lime, or other binders in-situ using a mixing machine or before placement.**
	D4. Jet grouting	High speed jets at depth erode the soil and inject grout to form columns or panels.
	D5. Compaction grouting	Very stiff, mortar-like grout is injected into discrete soil zones and remains in a homogeneous mass so as to densify loose soil or lift settled ground.
	D6. Compensation grouting	Medium to high viscosity particulate suspension is injected into the ground between a subsurface excavation and a structure in order to negate or reduce settlement of the structure due to ongoing excavation.

Table 4: Terminology of the deep mixing family, after Porbaha (1998)

CCP: chemical churning pile	DeMIC: deep mixing improvement by cement stabilizer
CDM: cement deep mixing	*In situ* soil mixing
CMC: clay mixing consolidation method	JACSMAN: jet and churning system management
DCCM: deep cement continuous method	Lime-cement columns
DCM: deep chemical mixing	Mixed-in-place piles
DJM: dry jet mixing	RM: rectangular mixing method
DLM: deep lime mixing	Soil-cement columns
DMM: deep mixing method	SMW : soil mix wall
DSM : deep soil mixing	SWING: spreadable WING method

2. CONSTRUCTION PRINCIPLES AND EQUIPMENT

In the DSM process, the ground is in situ mechanically (and possibly hydraulically or pneumatically) mixed while a binder, based on cement or lime, is injected with the help of a specially made machine. DMM can be classified according to its execution process. Two types of installation methods are generally considered with regard to the way the binder is injected into the ground (with or without water addition): the wet and the dry mixing methods. In the wet mixing method, which is more frequently applied, a mixture of a binder and water with possibly sand or additives is injected and mixed with the soil. Depending on the type of soil and binder, a mortar-like mixture is created which hardens during the hydration process (Essler and Kitazume, 2008).

In the dry soil mixing process, the binder is directly mixed with the soil. The binding agents directly react with the prevailing soil and the contained water and form a soil mortar. *Quasthoff (2012)* provides a State of the Art in dry soil mixing and reviews its construction principles, its equipment and its field of applications. Interested readers can also refer to Bruce et al. (1999).

The different types of DSM systems available on the international market can be classified according to the way the mixing is performed into the ground. Such classification has been provided in the past by Bruce et al. (1998), Topolnicki (2004) and Essler and Kitazume (2008). During the Short Courses of IS-GI 2012, Topolnicki (2012) has presented an updated classification scheme as illustrated in Fig. 1. The different DSM systems are now separated according to four levels of classification taking into account the dry or wet mixing (1), the mechanical, hydraulic or hybrid way of mixing (2), the position of the mixing (3) and finally the axis of rotation of the mixing tools (4). To the best of our knowledge, this is the most well-rounded classification with regard to the recent developments in the field of DMM.

Depending on the applications, different improvement patterns can be designed with these various DMM considering soil mix columns, rectangular soil mix panels, continuous barriers or global mass stabilization.

The following paragraphs consist in a review of the DSM systems essentially used in Europe.

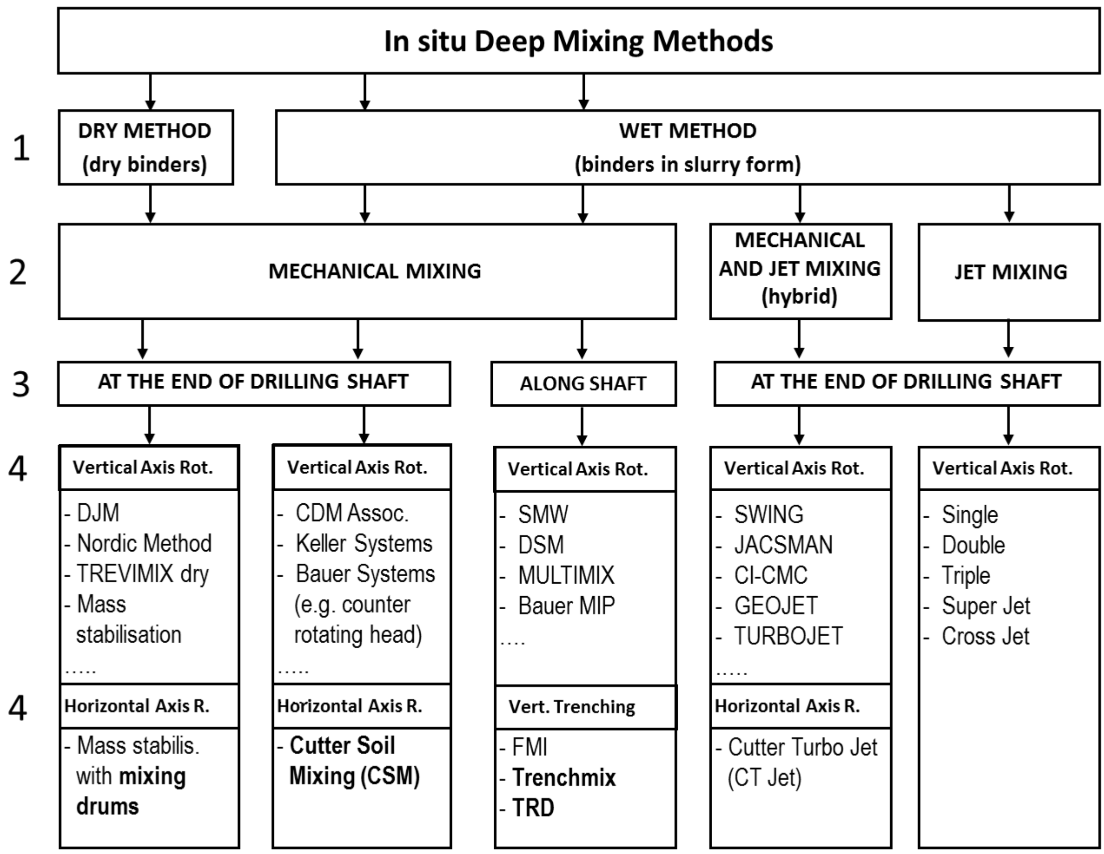


Figure 1: Updated classification scheme of DSM systems, after Topolnicki (2012)

2.1. Dry Deep Soil Mix systems

These systems are dry mixing systems wherein the mix is mechanically conducted at the end of the shaft without jet assistance.

Figure 2 illustrates the **Nordic method** including alternative mixing tools whose the choice depends on the type of encountered soil and the required diameter of the column. The injection mode is also represented. The technique was discussed in detail in the Short Course of Quasthoff (2012) who described the Liebherr equipment for dry soil mixing. Figure 3 illustrates the production steps of the method allowing the production of columns of treated soils into the ground.

Figure 2: Illustration of the Nordic method (dry deep mixing), from Topolnicki (2012), with the courtesy of Keller

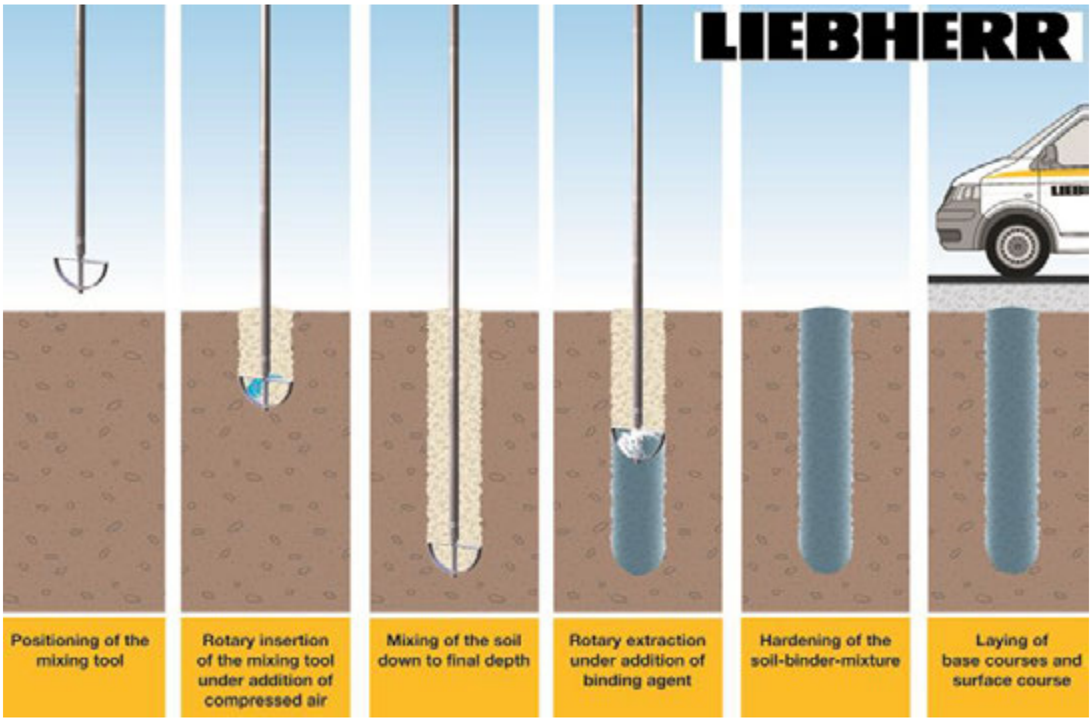


Figure 3: Production steps of the dry mixing method according to the experience of Liebherr, from Quasthoff (2012) with the courtesy of Liebherr

2.2. Global mass stabilization with dry mixing systems

Topolnicki (2012) presented the standard practice of **global mass stabilization** with dry shallow mixing method. If shallow mixing has been largely performed in the past with the help of column systems, Topolnicki (2012) brought the use of mass stabilization systems equipped with cutting mixing drums to light, such as illustrated in Fig. 4 with the ALLU mass stabilization systems (ALLU, 2010).

Figure 4: ALLU mass stabilization system with cutting mixing drums respectively mounted on an horizontal and two inclined axes of rotation, from Topolnicki (2012) with the courtesy of ALLU

2.3. Wet mixing systems in single shaft configurations

As reported in several presentations of the Short Courses, there is a large variety of wet mixing systems available in the single shaft configuration. In these **DSM column systems**, the mixing can be mechanically performed at the end of the shaft such as illustrated in Fig. 5, 6 and 7 or alternatively along the shaft. In several circumstances, hybrid mixing can be applied with the help of **jet assistance**, such as in the Trevi Turbojet system (see Fig. 8).

In a similar way, the Tubular Soil Mixing TSM technique (Smet-Boring nv) uses both mechanical and hydraulic way of mixing. Apart from the rotating mixing tool, the soil is cut by the high pressure injection (till 500 bars) of the water/binder mixture. As illustrated in Fig. 9, an external tube can be foreseen in order to obtain regular diameter and to avoid lateral soil decompression/decompaction along the boring.

Figure 5: Keller wet soil mix system in single shaft configuration (available tool diameter for single shaft ranging between 40 and 240 cm), after Topolnicki (2012)*, with the courtesy of Keller*

Figure 6: The CVR C-mix® single auger system (available tool diameter for single shaft ranging between 43 and 103 cm), after Denies (2012)*, with the courtesy of CVR nv*

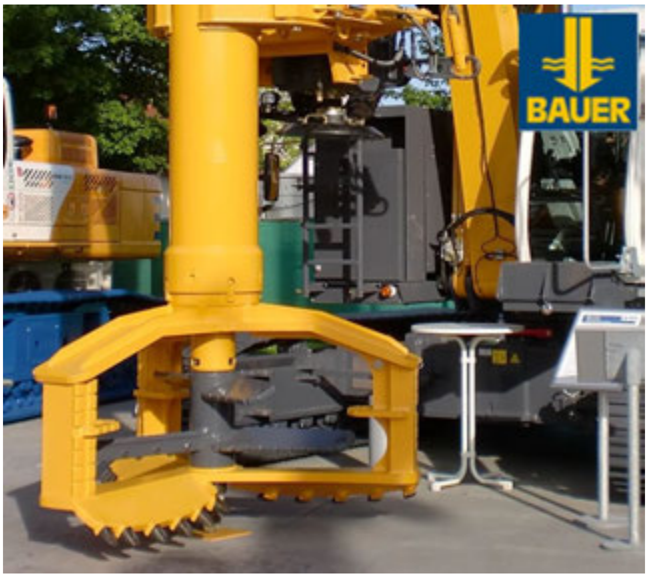


Figure 7: Bauer Single Column Mixing-Double Head, SCM-DH system (available tool diameter for single shaft ranging between 180 and 240 cm), after Topolnicki (2012) *with the courtesy of Bauer*

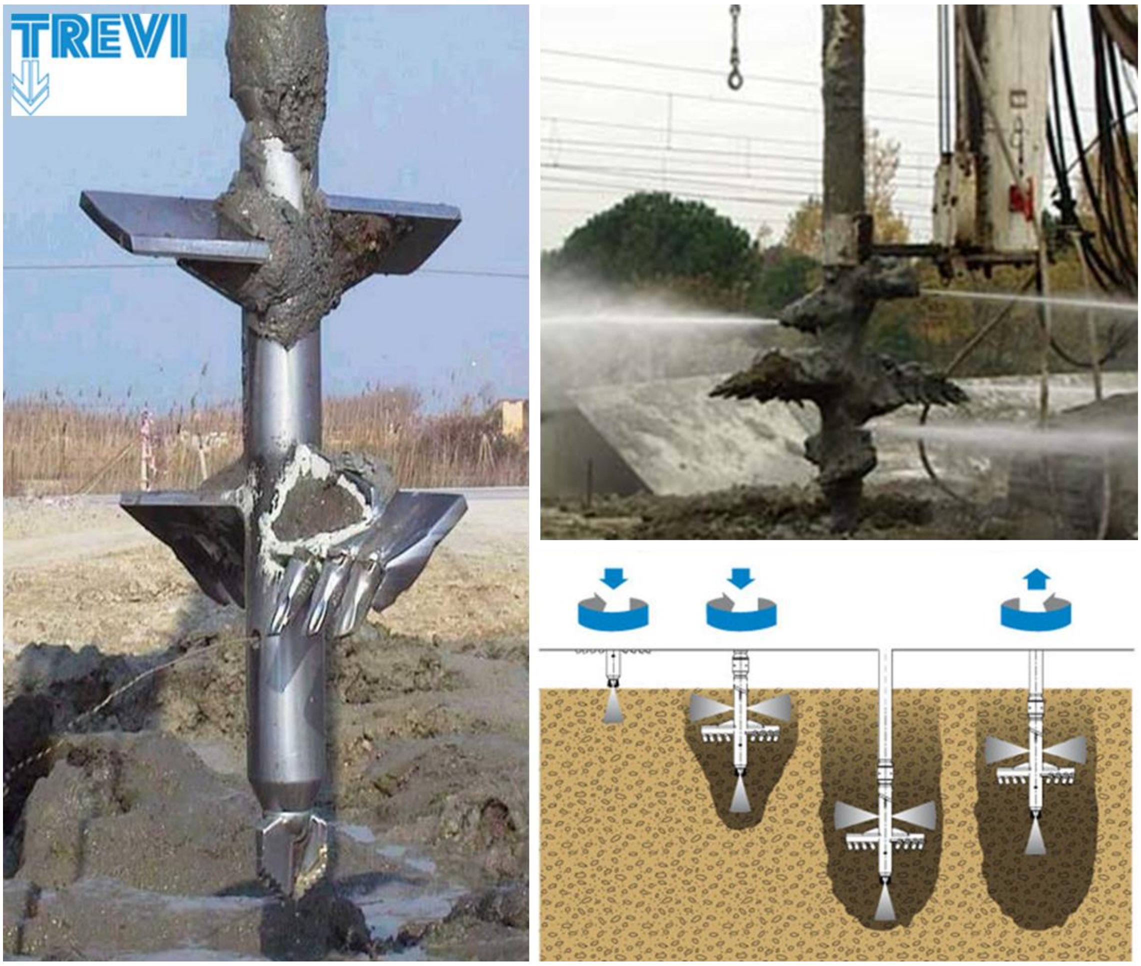


Figure 8: Trevi Turbojet technique (column diameter of 40 to 150 cm), after Topolnicki (2012) with the courtesy of Trevi

Figure 9: Smet Tubular Soil Mixing TSM system (column diameter of 38 to 73 cm), after Denies (2012) and Topolnicki (2012) with the courtesy of Smet-Boring nv

2.4. Wet mixing systems in double and triple shaft configurations

In order to increase production rate, double and triple shaft configuration systems possibly equipped with jet assistance device have been developed, such as illustrated in Fig. 10. It can be noted that other multiple shaft configuration systems are available on the international market as underlined in the "State of Practice Report – Execution, monitoring, and quality control" of Larsson (2005) who provided a full description of the DSM systems as used in Europe, in Japan and in the U.S.A.

Figure 10: Wet mixing systems in double and triple shaft configurations, after Topolnicki (2012), Denies (2012) and Borel (2012) with the courtesy of Keller, Smet-Boring nv, Bauer, CVR nv and Soletanche Bachy

2.5. Wet mixing spreadable systems

Within the framework of the Rufex project (reinforcement and re-use of railway tracks and existing foundations), soil-cement columns were installed with the help of the Soletanche Bachy ‘Springsol’ wet soil mixing tool (*Guimond-Barrett et al, 2012*). As illustrated in Fig. 11, this tool is equipped with two mixing blades that spread out under the action of springs. In its folded configuration, the tool diameter is 160 mm enabling its insertion into a temporary casing. By increasing the length of the mixing blades, the column diameter can be adapted (e.g. 40 and 60 cm as illustrated in the upper right side of Fig. 11). The main interests of the Springsol tool are the possibility to reinforce the ground under an existing railway track or an existing platform (slab and superficial isolated or continuous footings) and the opportunity to work under low headroom conditions (Borel, 2012).

Keller has also developed its own wet spreadable system. Figure 12a illustrates the first spreadable tool designed and build by Keller. The external diameter of the closed tool was 300 mm core retractable tool. Within the framework of the European Research project INNOTRACK, Keller Foundations has recently designed the FLAPWINGS system, illustrated in Fig. 12b. It consists in a 150 mm core retractable tool able to open up in order to perform the soil mixing phase on a 600 mm column. As reported in *Lambert et al. (2012)*, the FLAPWINGS tool also allows the opening and the closing of the retrieval blades. It is controlled by a two way hydraulic jack located in the mixing tool.

Figure 11: Soletanche Bachy wet spreadable mixing tool SPRINGSOL, from Borel (2012) and Guimond-Barrett (2012) with the courtesy of Soletanche Bachy

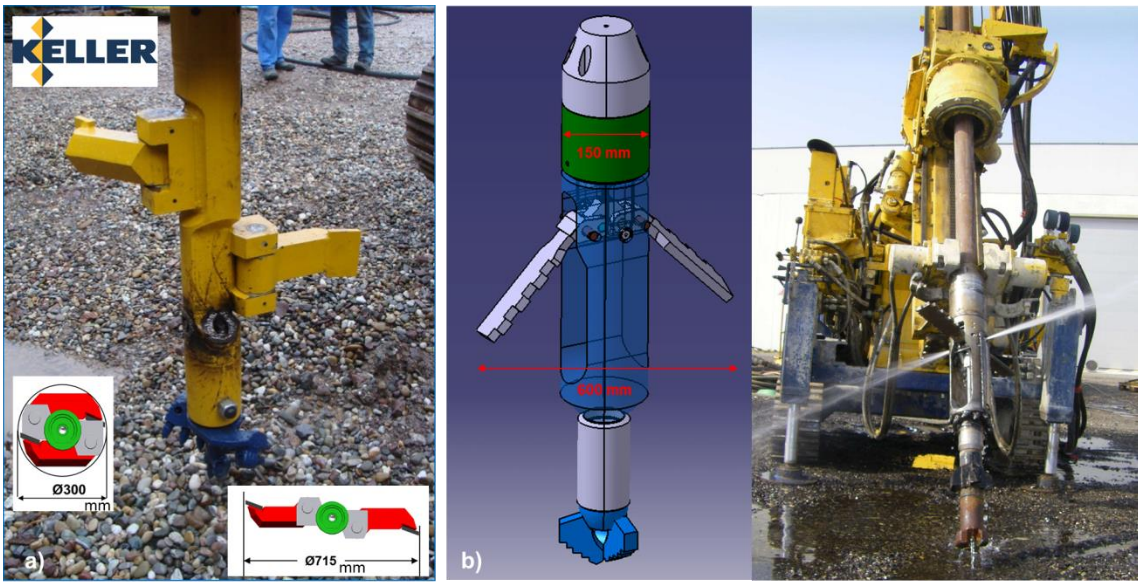


Figure 12: a) Keller wet spreadable mixing tool, after Topolnicki (2012) and b) Keller wet spreadable mixing tool FLAPWINGS, after Lambert (2012) with the courtesy of Keller

2.6. CSM panels

The execution of soil mix rectangular panels can easily be performed with the help of the Cutter Soil Mixing (CSM) system, as presented in the Short Courses by Borel (2012), Gerressen (2012) and Topolnicki (2012). The CSM is based on the principle of the trench cutter technique. Figure 13 and 14 illustrate the cutting and mixing tools of the CSM respectively for the Bauer and the Soletanche-Bachy-TEC systems. As explained by Gerressen (2012), CSM systems can be used in kelly guided or in wire rope suspended configurations depending on the applications.

Figure 13: The cutting/mixing tools of the Bauer CSM system (on left) and the QuattroCutter and SideCutter systems, from Gerressen (2012) with the courtesy of Bauer

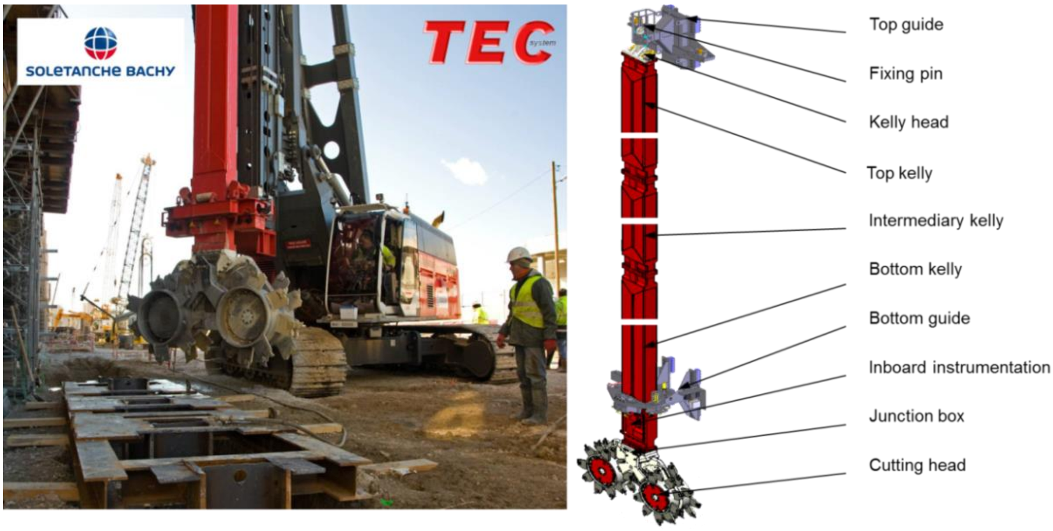


Figure 14: The cutting/mixing tools of the Soletanche-Bachy-TEC CSM system, from Borel (2012) with the courtesy of Soletanche-Bachy

2.7. Trenchmixing

As presented in Topolnicki (2012), the principle of the **trenchmixing method** is to produce a soil mix barrier in a single continuous pass which is an advantage particularly in case of water retaining function (no joints). Figure 15 shows the FMI and the Trenchmix® systems. For deep and large applications, the use of the TRD system (see Fig. 16) can also be envisaged.

Figure 15: Trenchmixing tools, from Topolnicki (2012) with the courtersy of Siedla-Schönberger and Soletanche-Bachy/Mastenbroek

Figure 16: TRD soil mix walls with the courtesy of Hayward Baker and Keller

2.8. Pros and cons of the DMM

The variety of equipment presented in this section allows the execution of DSM material in a large range of soil types with the following advantages and disadvantages (see Table 5). According to our experience, the left column can be completed as follows:
- the use of the existing soil as a construction material,
- a control of the geometry of the soil mix element with depth
- contrary to concrete secant pile walls, the execution of the soil mix walls does not suffer from delayed supply (e.g. due to traffic jams) of the fresh concrete,
- for the wet mixing method, the amount of spoil return is more limited and more controllable than for jet-grouting or slurry walls,
- dewatering is not required.

Table 5: Main advantages and limitations of the DMM, after Topolnicki (2012)

Main advantages of the DMM	Main limitations of the DMM
High productivity usually possible, hence economical for large scale projects	Depth limitations (depending on the method applied)
Can be potentially used in all types of soils and fills (without obstructions)	Not applicable in soils that are very dense, very stiff, or that may have boulders
Column's spacing and patterns highly variable, arrangements tailored to specific needs	Limited or no ability to install inclined columns (depending on the equipment applied)
Engineering properties of treated soil can be closely designed	Uniformity and quality of mixed soil may vary considerably in certain conditions
Causes minimal lateral or vertical stress that could potentially damage adjacent structures	Columns cannot be installed in close proximity to existing structures (except hybrid mixing)
No vibration, medium-low noise	Freeze/thaw degradation may occur
Very low spoil (especially for dry method)	Significant spoil produced with wet-method
Can be used for on-land, waterfront and marine projects	Weight of the equip. may be problematic for weak soils (depending on the method)
Quality of treatment verifiable during construction	Limited ability to treat isolated strata at depth

2.9. Wet or dry mixing method

If the use of the dry or wet mixing methods is often related to the available machines on the local market and to economic reasons, Topolnicki (2012) still provides an interesting tool (table 6) comparing both processes.

Table 6: The choice of the dry or wet process, from Topolnicki (2012)

Item of concern	Expectations
Initial water content of the soil to be treated	cohesive soils with moisture content w = 60 % to 200% are best suited for the dry process (lower limit w > 20%, water content below plastic limit is not fully available for hydration)
Quality of mixing	wet process usually provides better homogeneity of stabilised soil because easier distribution of slurry across the column area, prehydration of cement and longer mixing time
Compressive strength of soil-binder mix	higher strength is more reliably obtained with the wet process, except for very wet soils
Ability to penetrate through hard soil layers	much higher for the wet process due to the "lubrication" effect of the injected slurry and due to higher torque capacity of rigs
Stratified soils	wet mixing can provide more uniform strength along the column length due to partial soil exchange/movement in the vertical profile, quality control more difficult for the dry process
Spoil	dry mixing creates very little or no spoil
Use of combined binders and industrial by-products	quite frequent in dry mixing, slag cement in wet mixing, other binders and by-products very rare
Air temperature below 0º C	dry process is significantly less affected by low temperatures since compressed air is used to transport the binder
Column reinforcement	possible in combination with the wet process

3. DEEP SOIL MIX MATERIAL

3.1. Governing parameters

Several parameters have an influence on the produced DSM material. As previously made by Terashi (1997), Topolnicki (2012) proposes a new review of the main factors affecting the strength of the DSM material, such as illustrated in Table 7.

Table 7: Factors affecting the strength of the DSM material, after Topolnicki (2012)

Source	Specific items
I. Physical and chemical properties of the soil to be treated	Grain size distribution, mineralogy, natural water content, Atteberg limits, organic matter content, reactivity and pH of pore water
II. Binder, additives and process water	Type and quality of hardening agent(s), binder composition, quantity of binder and other additives, quality of mixing water
III. Installation technique and mixing conditions	Tool geometry, installation process, water/binder ratio, energy of mixing (speed and period), time lag between overlaps and working shifts
IV. Curing conditions, time	Curing time, temperature (heat of hydration in relation to treated volume), humidity, wetting/drying and freezing/thawing cycles, long-term strength gain and/or deterioration
V. Testing and sampling	Choice of testing method, type of test, sampling technique, sample size, testing conditions (stress path and drainage conditions, confining pressure, strain rate, method of strain measurement)

The DSM material quality depends on the cement type and content, on the in situ soil and on the execution process. The hardening agent is usually a mixture of cement and/or lime, water (for wet mixing), and in several cases bentonite. Sometimes ashes and gypsum are also used as additive.

The hardening agent is usually a mixture of cement, water, and in several cases bentonite. The water/binder mixture (w/b weight ratio) is also a governing parameter which plays a major role in the mechanical/durability characteristics of the material.

Moreover, the nature of the ground has a huge impact on the strength and uniformity of the material. For example, stiff cohesive soil does not allow an effective mix of the components and can lead to the presence of unmixed material in the DSM element.

The final product will be the result of a given DSM system available on the local market. There are a lot of differences between the various systems – especially with regard to the drilling/mixing tools – and the execution process influences the quality of the DSM material in terms of strength, uniformity and continuity.

The exposure conditions of the DSM elements during their lifetime will have a certain influence on the long term strength gain or on the deterioration of the DSM material.

Finally, test procedures will have an impact on the results of the characterization possibly resulting in various conclusions in function of the test method.

Within the framework of the Short Courses, Topolnicki (2012) mainly concentrates on the mechanical characterization of the DSM material with regard to the previous governing parameters. For his part, Denies (2012) presents the results of the BBRI 'Soil mix' project (2009-2013) of which the purpose is to study the use of soil mix for earth/water retaining structures (in a view of temporary or permanent function). This project concentrates on the mechanical characterization of the DSM material such as build in Belgium with the help of the CVR C-mix®, the Smet TSM and the CSM systems. In the framework of this research, numerous tests on in situ DSM material have been performed. All the results and the developments related to the BBRI "Soil Mix" project are detailed in four papers of the present symposium: *Denies et al. (2012a, b and c)* and *Vervoort et al. (2012)*.

3.2. Unconfined Compressive Strength of the soil mix material

The usual way to characterize the strength of the DSM material is to perform Unconfined Compressive Strength (UCS) tests. With regard to the aforementioned governing factors, the following paragraphs concentrate on the study of the influence of the mixing energy, the cement factor, the water/cement ratio and the curing time on the UCS of the DSM material.

3.2.1. *Influence of the mixing energy*

Topolnicki (2012) first illustrates the influence of the mixing energy on the strength of the soil mix material. The control of the homogeneity of the produced material can be performed considering the "*Blade rotation number*". This latter is defined as follows:

$$BRN = \sum M \, x \left(\frac{N_d}{V_d} + \frac{N_u}{V_u} \right) \quad (1)$$

where BRN is the Blade Rotation Number (1/m), ΣM the total number of mixing blades, N_d the rotation speed of the blades during penetration (rpm), V_d the mixing blade penetration velocity (m/min), N_u the rotational speed of the blades during withdrawal (rpm) and V_u the mixing blade withdrawal velocity (m/min). The BRN evaluates the mixing degree. It gives the total number of mixing blades passes during 1 m of shaft movement (CDIT, 2002).
Figure 17 illustrates the evolution of the coefficient of variation, *v*, of the UCS test results in function of the BRN. Additional information on these results is available in Topolnicki (2009).

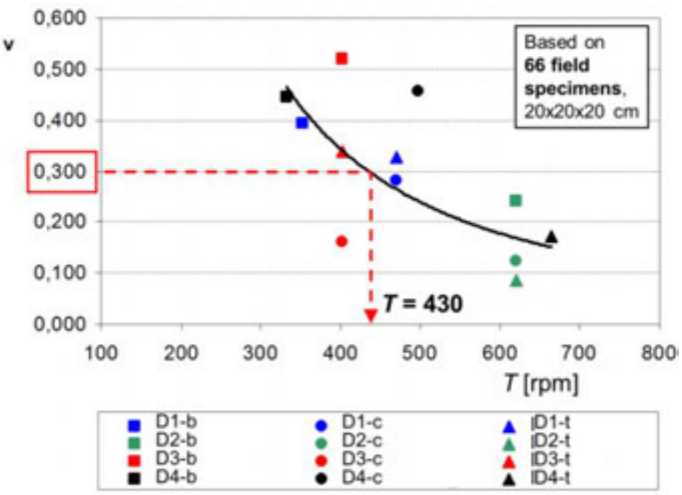


Figure 17: Variability of the soil mix material with the mixing energy, from Topolnicki (2012), more details in Topolnicki (2009)

3.2.2. *Binder factor*

Topolnicki (2012) then provides a range of UCS values in function of the binder factor (see Table 8). He defines the binder factor as the weight of injected dry binder divided by the volume of treated ground and the binder factor in place as the weight of injected dry binder divided by the total volume of treated ground and injected slurry.

Table 8: Typical field UCS ranges for different soil types and various cement factors, from Topolnicki (2012), more details in Topolnicki (2004)

Soil type	**Binder factor** cement **[kg/m³]**	**Field UCS**[†] **[MPa]**
Sludge	250 - 400	0.1 - 0.4
Peat, organic silts/clays	150 - 350	0.2 - 1.2
Soft clays	150 - 300	0.5 - 1.7
Medium/hard clays	120 - 300	0.7 - 2.5
Silts and silty sands	120 - 300	1.0 - 3.0
Fine-medium sands	120 - 300	1.5 - 5.0
Coarse sands and gravel	120 - 250	3.0 - 7.0

[†] UCS values are given in term of guaranteed compressive strength at 90% confidence

3.2.3. Water-cement content

Topolnicki (2012) also concentrated on the influence of the water/cement ratio on the strength of the soil mix material, as illustrated in Fig. 18. Details and conclusions of this study are described in Topolnicki (2009).

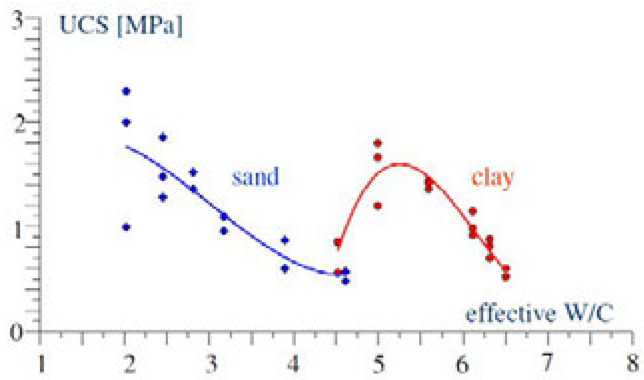


Figure 18: UCS test results for laboratory mixed sand and clay-cement samples tested with different initial water contents, after 28 days of curing (α = 200 kg/m³, density of the cement slurry ρ = 1.5 g/cm³), after Topolnicki (2012), more details in Topolnicki (2009)

3.2.4. Influence of the curing time

Finally the question of the age of the sample is discussed. Various empirical relationships are presented in the Short Courses.

For the Japanese practice of soil stabilization with the deep mixing technique, Terashi (2002) reports that the strength of the stabilized material in long-term (10 to 20 years) is 2 to 3 times the short-term value but that often concerns soil stabilization with limited cement or lime content (α≈150 kg/m³).

Based on a large review of data, Filz et al. (2012) proposes a generalized logarithmic relationship to express the hardening of the DSM material with time:

$$UCS(t) = (0.187 ln(t) + 0.375) UCS_{28curingdays} \quad (2)$$

Topolnicki (2012) provides various empirical relationships based on experience with real design cases, as illustrated in Table 9.

Table 9: Empirical relationships describing the evolution of the strength of DSM material with the time for various soil types, after Topolnicki (2012)

Soil type	Relationships describing the curing time effect on the strength of the DSM material
	$UCS_{28\ curing\ days}$ = c. 2 x $UCS_{4\ curing\ days}$
Silts and clays	$UCS_{28\ curing\ days}$ = 1.4 – 1.5 x $UCS_{7\ curing\ days}$
Sands	$UCS_{28\ curing\ days}$ = 1.5 – 2 x $UCS_{7\ curing\ days}$
Silts and clays	$UCS_{56\ curing\ days}$ = 1.4 – 1.5 x $UCS_{28\ curing\ days}$

For the Belgian practice, the question of the curing time is tackled by Denies (2012). Figure 19 illustrates the evolution of the UCS values of laboratory soil mix specimens in function of the curing time. As previously demonstrated by Ganne et al. (2010), the best fit is obtained with the help of the following equation:

$$UCS(t) = \beta_{cc}(t)\, UCS_{28days} \quad (3)$$

where β_{cc} is defined as:

$$\beta_{cc}(t) = \exp\left(s\left(1 - \sqrt{\frac{28}{t}}\right)\right) \quad (4)$$

where *s* is an empirical factor mainly depending on the type of cement and soil.

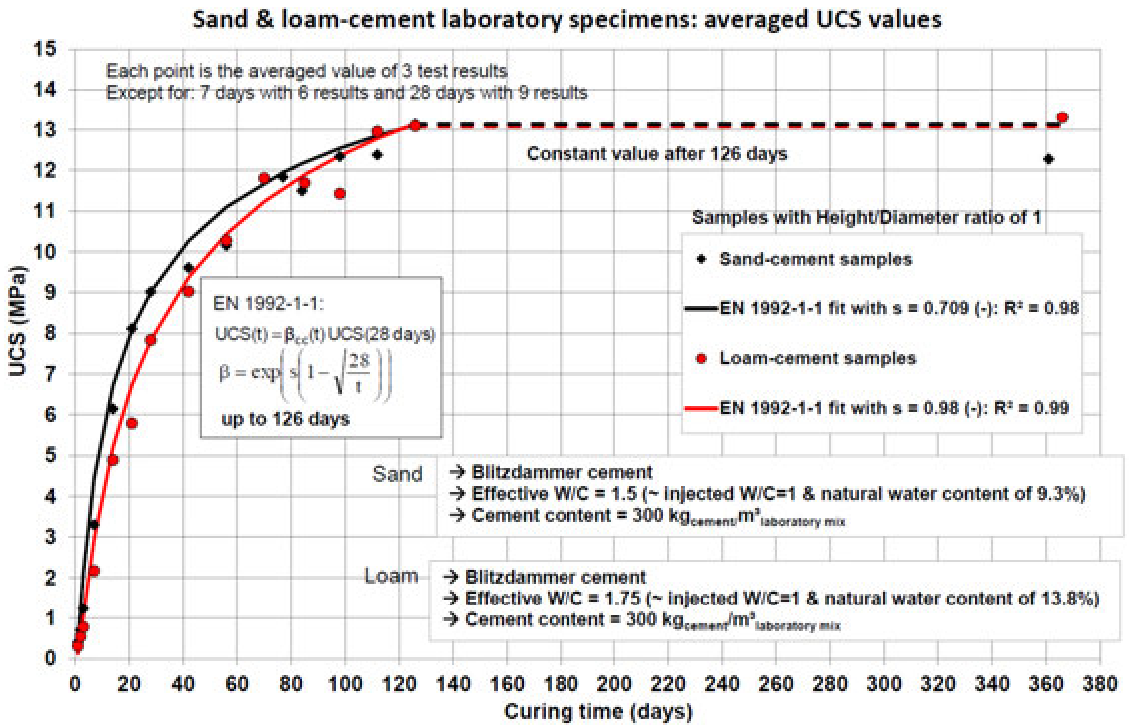


Figure 19: Evolution of the strength of DSM laboratory specimens in function of the curing time, adapted from Denies (2012)

Equation (3) comes from EN 1992-1-1 for concrete material. As shown in Fig. 19, beyond an initial growing period (126 curing days), there is no more increase of the strength.

In a general way, DSM material show delayed strength development compared to concrete, and show also a long-term increase but both phenomena are dependent on the type of soil and the type of cement. Pozzolanic reaction products should be possibly considered. They can lead to long-term strength development.

It can be noted that in the present proceedings, *Bellato et al. (2012)* have tried to fit their experimental data of CSM treated overconsolidated clays to several empirical relationships, with satisfactory results for the case of the formula (3). The best fit for a curing time larger than 3 days was found with the equation:

$$\mathrm{UCS}(t) = \ln(t) - 1 \tag{5}$$

To better represent the increase of the UCS with the curing time observed in the Bologna specimens, they proposed a new empirical equation, based on a double hyperbolic function. This function is composed of two terms. The first one describes the increase of the UCS in the first 28 curing days, whereas the second one defines the development of the long-term strength. This relationship is given by:

$$\mathrm{UCS}(t) = \frac{\mathrm{UCS}_{28\mathrm{days}} \cdot t}{t + K_1 \cdot \mathrm{UCS}_{28\mathrm{days}}} + \frac{\Delta\mathrm{UCS}_{\infty} \cdot t}{t + K_2 \cdot \Delta\mathrm{UCS}_{\infty}} \tag{6}$$

in which $\mathrm{UCS}_{28\mathrm{days}}$ can be corrected to take into account the amount of injected cement, the fine content, the mixing quality parameter and the *pH*. $\Delta\mathrm{UCS}_{\infty}$ is the strength increment due to long-term reaction products. K_1 and K_2 are two constants dependent on the type of clay and cement used in the treatment (in the case of the CSM treated Bologna overconsolidated clays $K_1 = 1$ and $K_2 = 100$).

As an alternative method to monitor the hardening of cement treated clay as a function of time, Verástegui Flores (2012) presented a nondestructive technique based on the measurement of the small-strain shear modulus (G_0) of the material. G_0 is determined by measuring the time, Δt, that a shear wave needs to travel between two bender elements (the transmitter and the receiver) at a distance, L, through the studied material (see Fig. 20). Therefore, a special test mold has been developed such as illustrated in Fig. 21. The strength increase was evaluated by conventional unconfined compression testing.

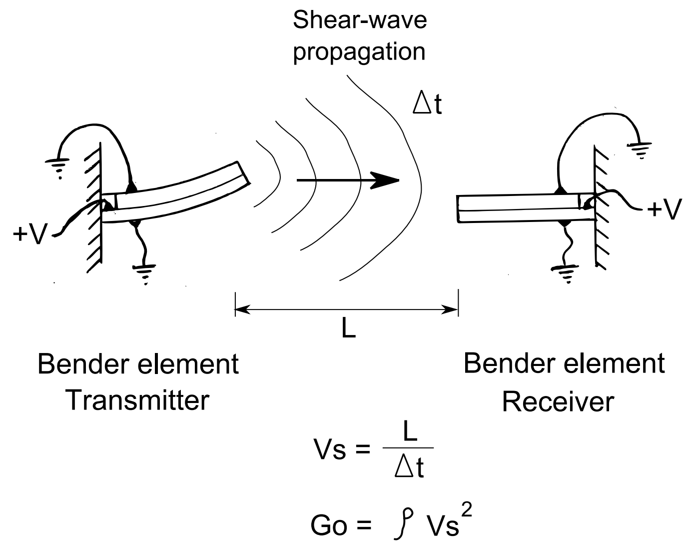


Figure 20: Principle of the determination of the small-strain shear modulus by bender element, from Verástegui Flores (2012)

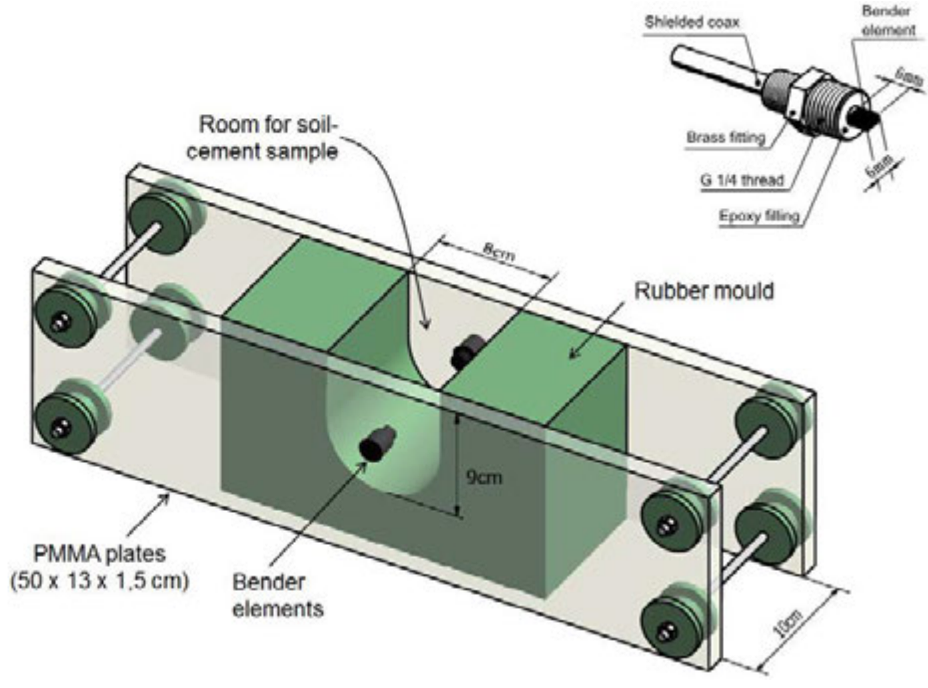
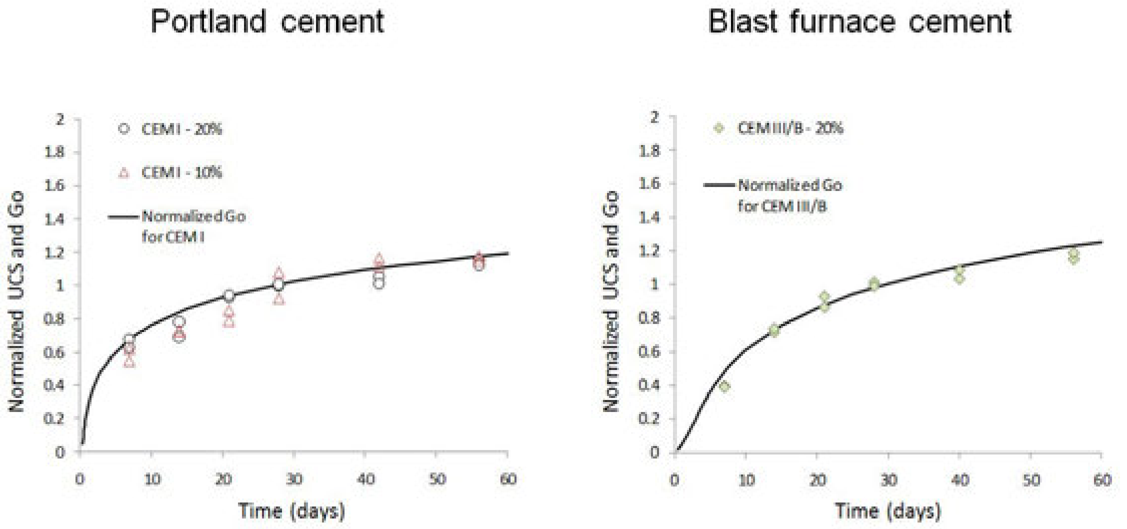


Figure 21: Test equipment for the measurement of the small-strain shear modulus of the cement treated soil samples, from Verástegui Flores (2012)

The experimental work was carried out on Kaolin clay (Rotoclay HB®) treated with Portland cement (CEM I 52.5) and blast furnace cement (CEM III/B 32.5) at different cement dosages (5, 10 and 20%).

The results demonstrate that the rates of increase of G_0 and the UCS are similar. Moreover, the increase can be approximated by logarithmic functions for soil stabilized with Portland cement and blast furnace slag cement, as illustrated in Fig. 22. A slower hardening rate is still observed for blast furnace slag cement at lower curing times. But when normalized, the hardening trend is very similar for each binder type, regardless of the cement dosage. Finally, it can be noted that the proposed hardening trends show a good agreement with results obtained on other cement treated inorganic clays found in literature (see Fig. 23 and Table 10) and may be used as a basis for strength prediction rules.

More details on this study are availbale in Verástegui Flores and Di Emidio (2011).

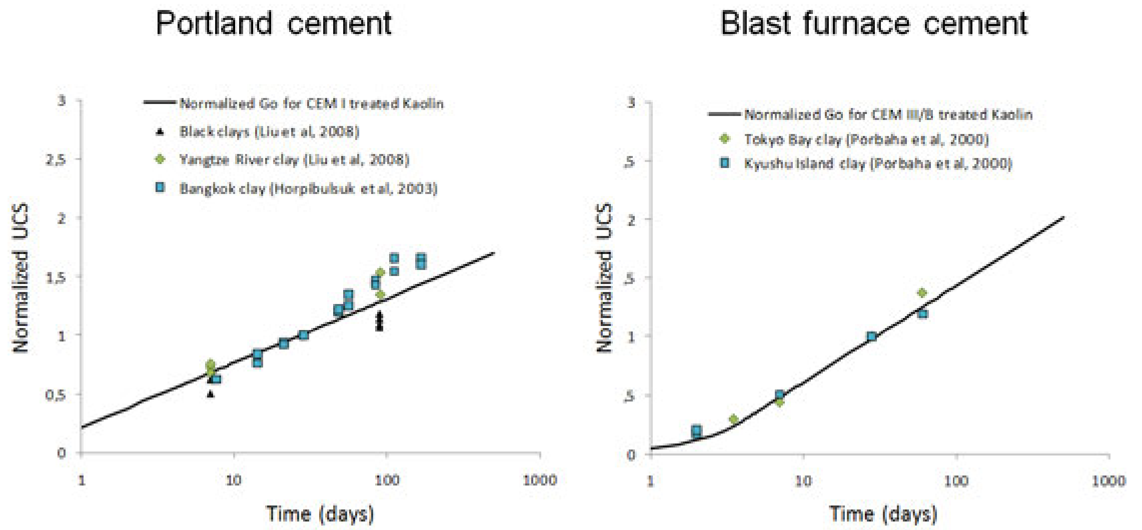


$$G_0/G_{0(28\ \text{days})} = 0.2381\ \ln(t) + 0.2145$$

$$G_0/G_{0(28\ \text{days})} = 0.0472t^{1.2866}$$

$$G_0/G_{0(28\ \text{days})} = 0.3586\ \ln(t) - 0.2159$$

Figure 22: Normalized UCS (data points) compared to the normalized small-strain shear modulus, G_0, (continuous black line) for cement treated Kaolin clay, from Verástegui Flores (2012)



$$UCS / UCS_{(28\ \text{days})} = 0.2381\ \ln(t) + 0.2145$$

$$UCS / UCS_{(28\ \text{days})} = 0.0472t^{1.2866}$$

$$UCS / UCS_{(28\ \text{days})} = 0.3586\ \ln(t) - 0.2159$$

Figure 23: Comparison of the proposed hardening functions with data from literature, from Verástegui Flores (2012)

Table 10: Comparison of the proposed hardening functions with the data of Topolnicki (2004), from Verástegui Flores (2012)

Soil	$\frac{UCS_{20\ years}}{UCS_{90\ days}}$	$\frac{UCS_{20\ years}}{UCS_{90\ days}}$
Marine clay + CEM I	2.2 (Topolnicki, 2004)	2.10 (predicted)
Volcanic soil + CEM III	≈ 3 (Topolnicki, 2004)	2.92 (predicted)

3.3. Compressive and shear behaviors of cement treated samples

Verástegui Flores (2012) also discussed the study of the strength and compressibility of Kaolin clay after treatment with binders. Aim of the study is to identify key behavior features and differences with respect to non-cemented Kaolin clay. The compression and shear strength behavior is assessed by triaxial compression testing and oedometer tests. It is observed that the behavior of cemented soils is strongly influenced by the cement content and the stress level to which a sample is subjected. Figures 24 and 25 respectively illustrate the compression and shear behaviors of the cement treated Kaolin clay. Initially, cement samples have much higher strength and stiffness than non-cemented samples. As the stress level increases, a yielding state is encountered where interparticle bounding begins to break intensively. Before yielding (at low stresses), the strength is governed by cement dosage and the one-dimensional compression is almost negligible. Beyond yielding (at high stresses), the strength is governed by the stress level just like for any non-cemented frictional material. Under one-dimensional compression, a clear collapse is observed. The compression lines tend towards the compression line of the non-cemented clay with a gradient that lightly steepens with increasing cement dosage, as illustrated in Fig. 24.

More details on this study are available in Verástegui Flores et al. (2009).

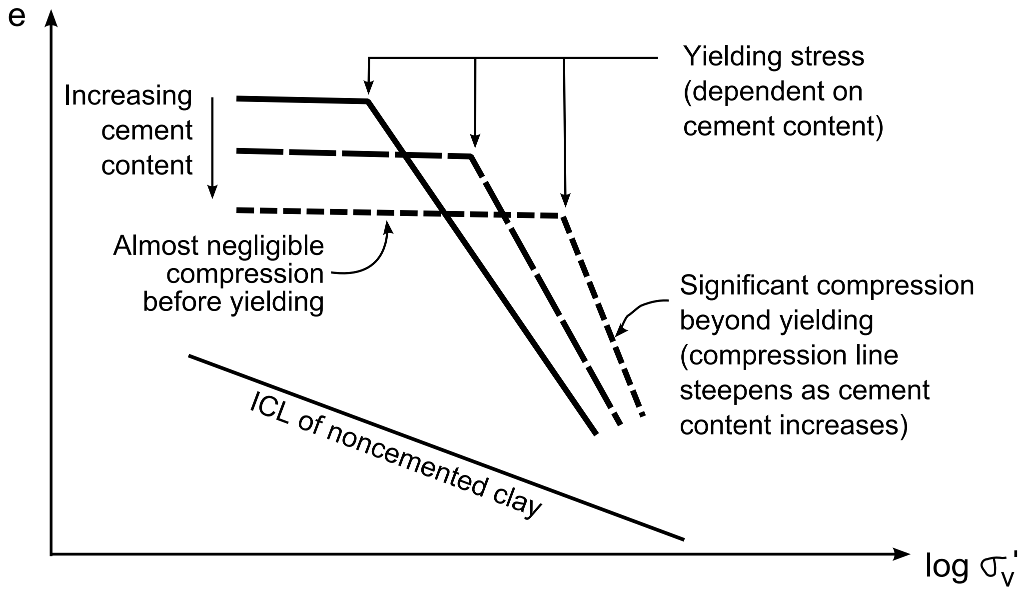


Figure 24: Compression behavior of cemented Kaolin clay, from Verástegui Flores (2012)

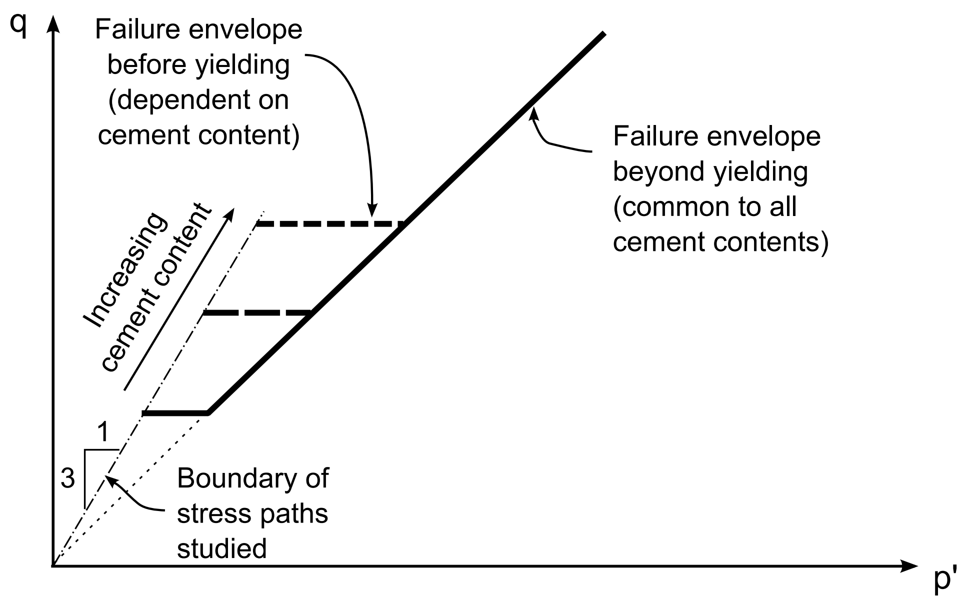


Figure 25: Shear behavior of cemented Kaolin clay, from Verástegui Flores (2012)

3.4. Correlations between mechanical properties and the UCS

For design considerations, it can be useful to include Brazilian or triaxial tests in the experimental campaign. Nevertheless, for cost or planning reasons it is not always possible to perform them. In such a way to bypass this difficulty, engineers often resort to correlations with the UCS to obtain other mechanical characteristics.

3.4.1. Shear and tensile strengths

In his Short Course, Topolnicki (2012) correlates the shear strength and the tensile strength of the DSM material to its UCS value, as illustrated in Table 11. Such results are based on field experience and experimental investigation of the material.

Table 11: Correlations between mechanical properties of DSM material, after Topolnicki (2012), more details in Topolnicki (2004)

Parameters	Expected values
Shear strength (direct shear, no normal stress)	0.4 to 0.5 ×UCS, for UCS<1 MPa 0.3 to 0.35 ×UCS, 1<UCS<4 MPa 0.2 ×UCS, for UCS > 4 MPa
Tensile strength	0.08 to 0.15 ×UCS, but not higher than 0.2 MPa
Poisson's ratio	0.3 to 0.4

In Denies (2012), the following results are presented for the tensile splitting strength (see Fig. 26). There is a good similarity with the correlation proposed by Topolnicki (2012). In Fig. 26, experimental results are compared with well-established empirical relationships for concrete (more details on these results are given in *Denies et al. 2012b*).

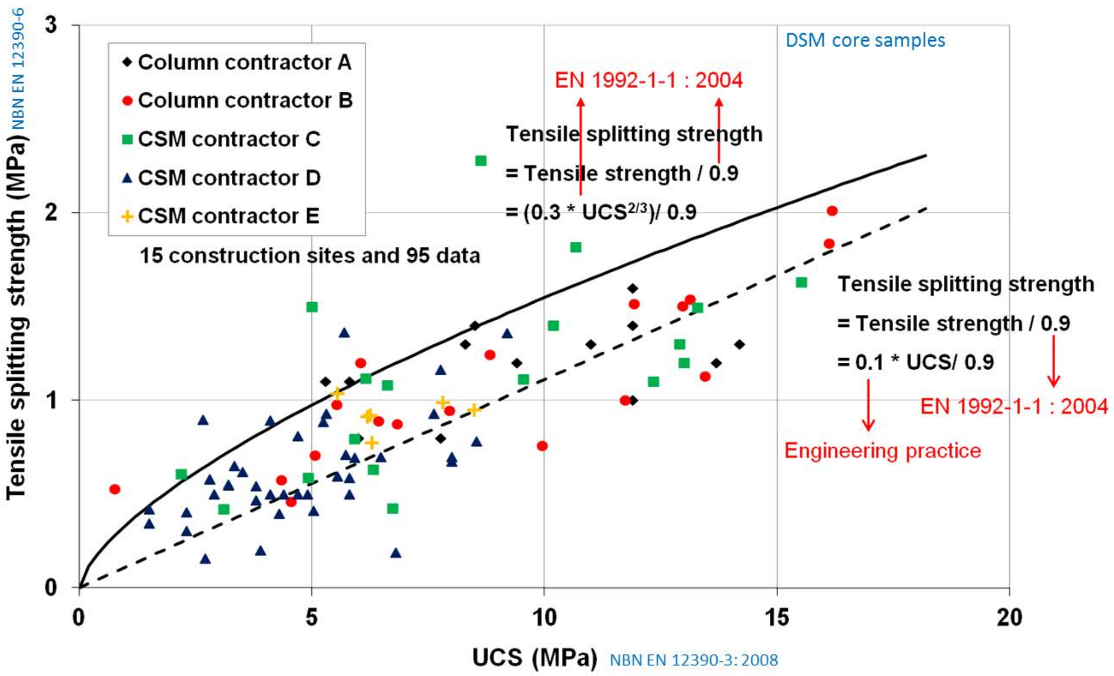


Figure 26: Correlations between the tensile splitting strength and the UCS of DSM core samples, from Denies (2012)

3.4.2. Modulus of elasticity

Interesting results concern the relationships between the modulus of elasticity and the UCS of the DSM material, such as proposed in Fig. 27 and 28. Considering both sources, the differences can be related to the test procedures and to the definition of the modulus of elasticity. In the study of Topolnicki (2012), the secant modulus (at 50% of the UCS value) is considered while in the study of Denies (2012) the modulus of elasticity is determined in a tangent way varying the applied load between 10% ($\sigma_{10\%UCS}$) and 30% ($\sigma_{30\%UCS}$) of the estimated UCS. For the determination of the modulus of elasticity; the test procedure has definitely a major influence on the test results.

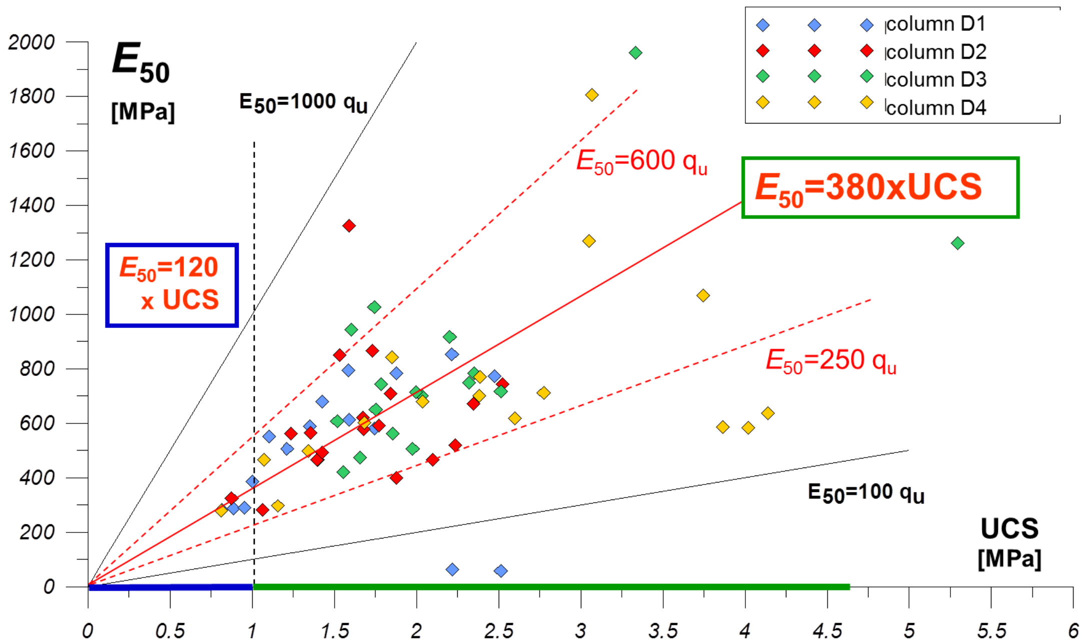


Figure 27: Correlations between the modulus of elasticity and the UCS, from Topolnicki (2012)

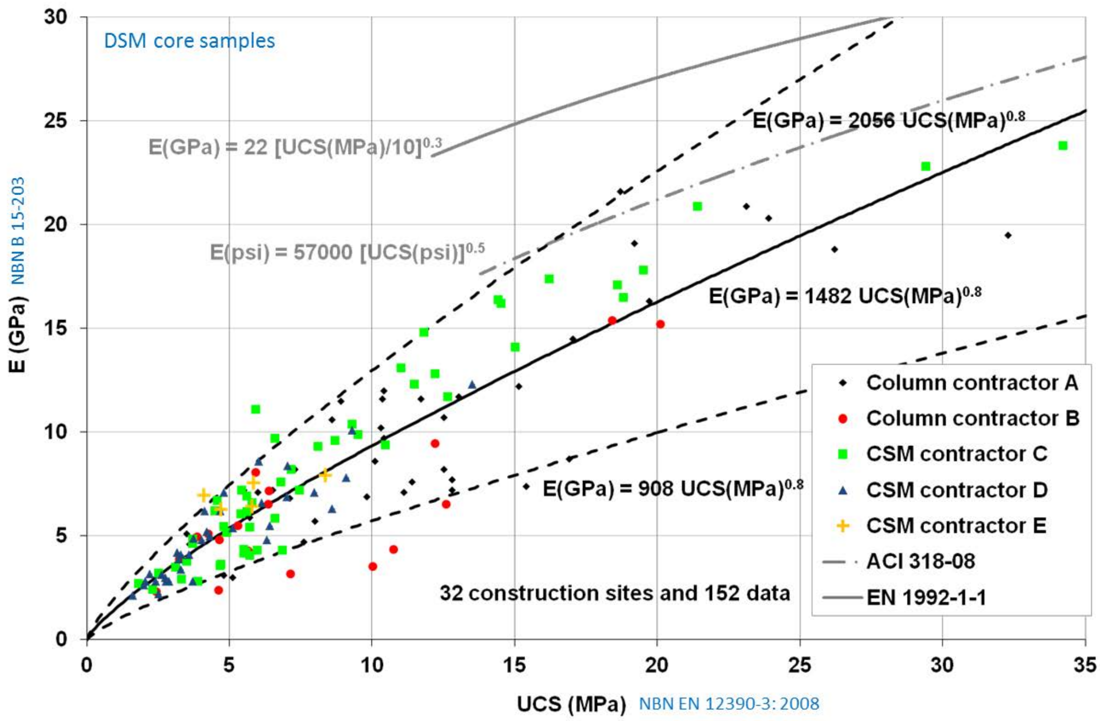


Figure 28: Correlations between the modulus of elasticity and the UCS, from Denies (2012) , more details in Denies et al. (2012b)

3.5. 'Steel-soil mix' adherence

To investigate the adhesion between DSM material and various steel profiles, in situ pull-out tests were conducted within the framework of the BBRI 'Soil mix' project (2009-2013). These results were presented by Denies (2012). Figure 29a presents the test setup used for these tests. After the execution of the soil mix, steel reinforcement was suspended from the guidance device and vertically installed into the fresh DSM material. As illustrated in Fig. 29b, the top part of the steel profile is made frictionless (over 1 m) using a flexible protection tube in order to eliminate the influence of the first non-representative meter on the results.

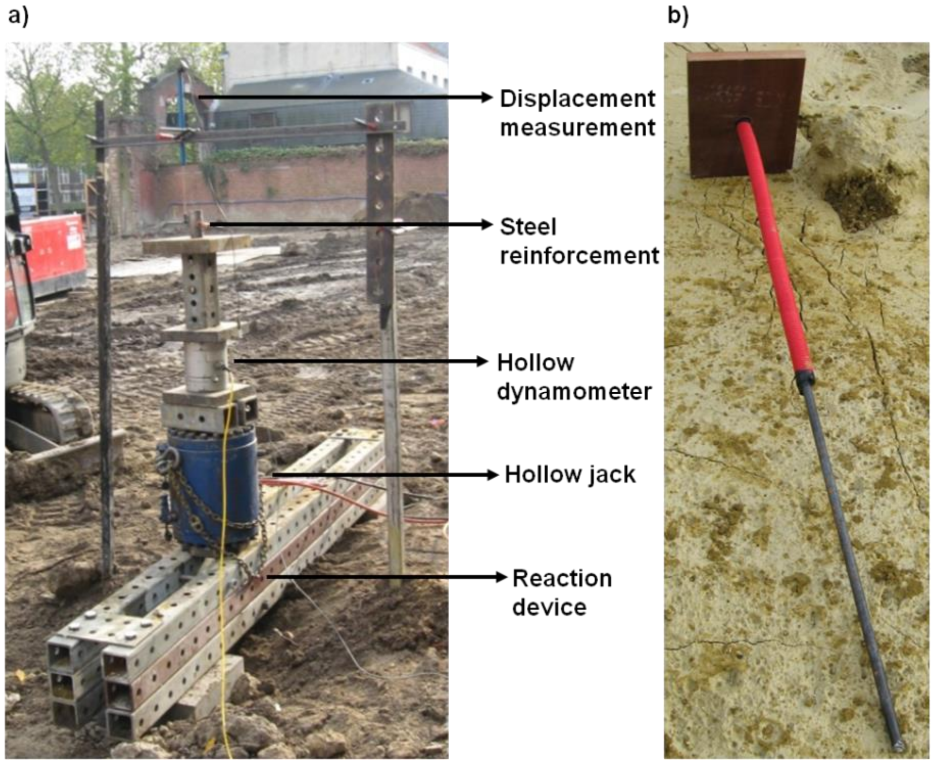


Figure 29: In situ investigation of the 'steel-soil mix' adherence a) Pull-out test set-up and b) steel profile with protecting tube, from Denies (2012), more details in Denies et al. (2012b)

Figure 30 presents the peak extraction resistance in function of the UCS of DSM cores, for different types of steel reinforcements.

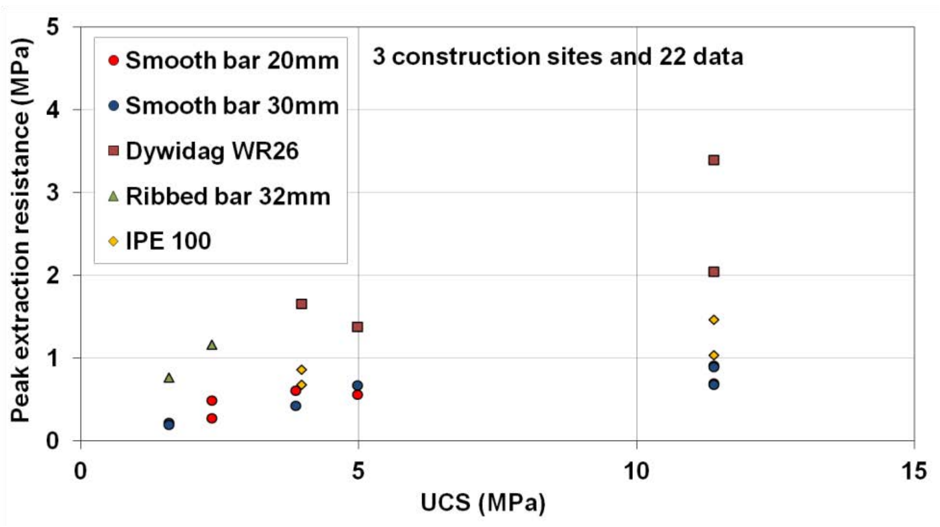


Figure 30: Peak pull-out resistance in function of the UCS of cored DSM material, from Denies (2012), more details in Denies et al. (2012b)

3.6. Porosity and permeability of the soil mix material

Within the framework of the BBRI 'Soil mix' project (2009-2013), porosity values were measured and vary between 25 and 65% for all soil types, as illustrated in Fig. 31. In order to explain this high range of values, a petrographic analysis is conducted on samples from two construction sites (in silty and sandy soils) with the help of image processing techniques (IPT) and thin section technology. The results of this study are presented in *Denies et al. (2012b)*.

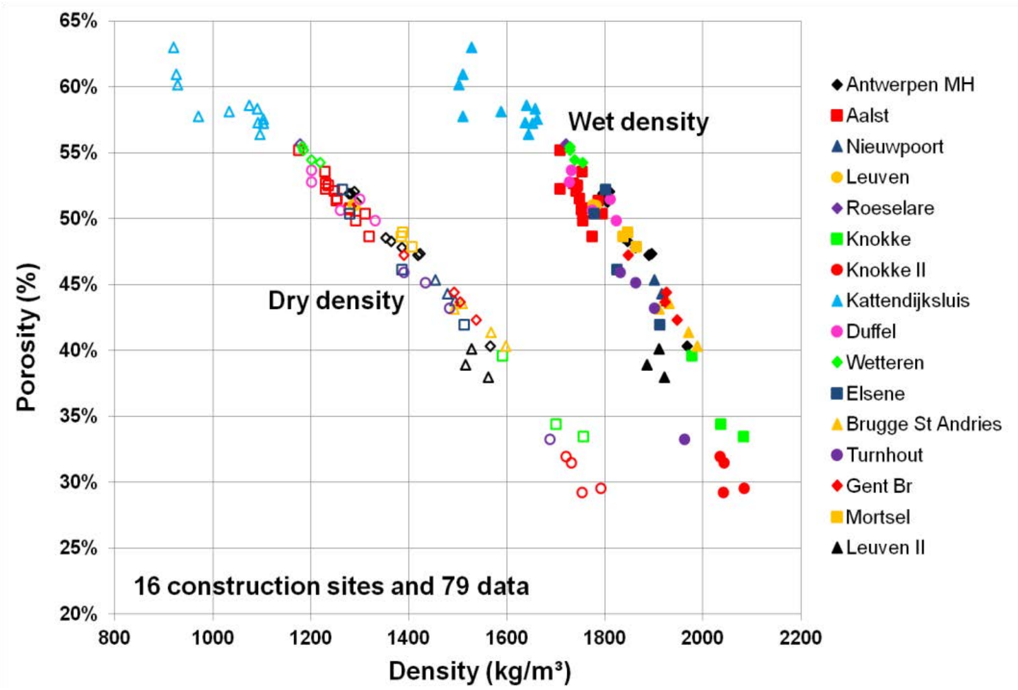


Figure 31: Relationship between dry and wet density and porosity for DSM cores, from Denies (2012), more details in Denies et al. (2012b)

Within the framework of the BBRI soil mix project (2009-2013), permeability tests were also performed on DSM samples. The samples were cored from real SMW's. The coefficient of hydraulic conductivity varied between 10^{-8} and 10^{-12} m/s, regardless of the execution process and the soil conditions, as illustrated in Fig. 32. No correlation was observed between porosity and permeability for the BBRI data. It can be noted that similar range of permeability is described in the Short Course of Weatherby (2012) with a coefficient of permeability varying between 10^{-8} and 10^{-9} m/s.

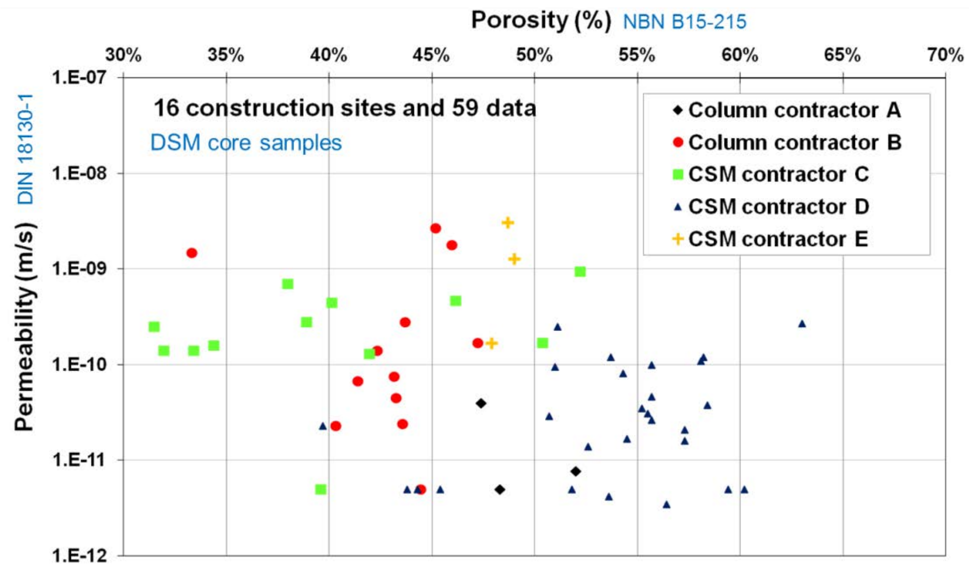


Figure 32: Relationship between permeability and porosity for DSM cores in function of the execution process, from Denies (2012), more details in Denies et al. (2012b)

3.7. Testing and sampling

The quality control (QC) of the produced soil mix material is generally based on laboratory tests performed on cored material. Each sample is still characterized by its own history influencing the test result and its interpretation. Beyond the question of the representativeness of the core samples with regard to the in situ executed material, Denies (2012) discusses over the sampling, the transport, the storage, the handling and the preparation of the DSM test specimens and proposes test procedures in the continuity of the content of the European standard EN 14679 (2005) for deep mixing. Table 12 illustrates the timeline of the DSM sample life with regard to the standards or test methodologies supporting its several stages, as used during the BBRI 'Soil Mix' project (2009-2013). As underlined in Denies (2012), special attention has been given to the presence of unmixed soft soil inclusion into the soil mix material. In addition, a handling procedure for the preparation of the test specimens has been established.

Two methodologies to quantify the volume of unmixed soft soil inclusions have been developed: the line and the surface methodologies (see Ganne et al., 2011, Ganne et al., 2012 and *Denies et al., 2012c*). Figure 33 gives an overview of the results for 27 Belgian construction sites. The amount of unmixed soft soil inclusions into the soil mix material mainly depends on the nature of the soil:

- in quaternary or tertiary sands, it is less than 3.5%,
- in silty (or loamy) soils and alluvial clays, it ranges between 3 and 10%,
- in clayey soils with high organic content (such as peat) or in tertiary (overconsolidated) stiff clays, it can amount up to 35% and higher.

One major issue concerns the representativeness of the core samples with regard to the in situ executed soil mix material. On the one hand, there is the question of the scale effect and on the other hand, the question of the influence of the unmixed soft soil inclusions. Both have an influence on the UCS test results. To investigate these topics, an experimental, as well as a numerical simulation research programme has been initiated at KU Leuven. Results are presented within the framework of the Short Course of Vervoort and Van Lysebetten (2012).

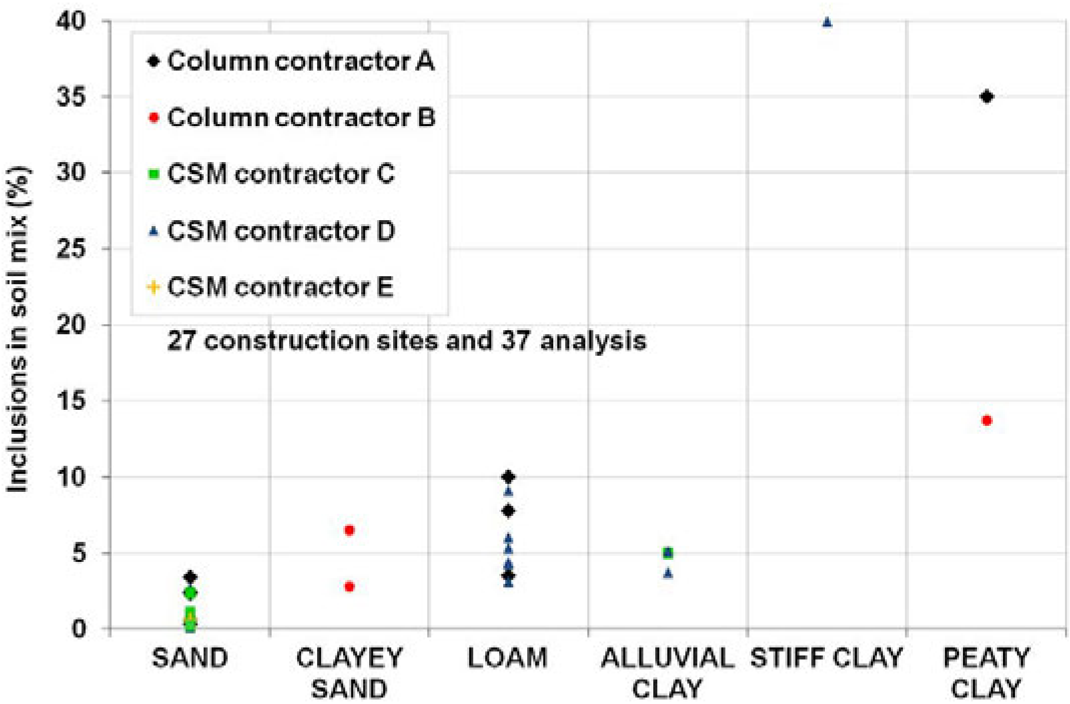


Figure 33: Percentage of unmixed soft soil inclusions into the soil mix material, from Denies (2012), more details in Denies et al. (2012b and c)

Table 12: Timeline of the soil mix samples: procedures followed within the framework of the BBRI 'Soil Mix' project (2009-2013), from Denies (2012), more details in Denies et al. (2012c)

Sampling	Transportation	Preserving/Storage	Handling/Preparation of test specimens/Test and report	
In situ	In situ	In laboratory	In laboratory	
EN 14679: 2005 Execution of special geotechnical works – Deep mixing **EN ISO 22475-1: 2007** Geotechnical investigation and testing - Sampling methods and groundwater measurements - Part 1: Technical principles for execution **ASTM D 420 – 93** Standard Guide to Site characterization for Engineering, Design, and Construction Purposes				
EN 12504-1: 2009 Testing concrete in structures - Part 1: Cored specimens - Taking, examining and testing in compression **ASTM D 2113 - 83** Diamond Core Drilling for Site Investigation	**ASTM D 4220 - 89** Standard Practices for Preserving and Transporting Soil Samples **ASTM D 5079 - 90** Standard Practices for Preserving and Transporting Rock Core Samples	**EN 12390-2: 2009** Testing hardened concrete - Part 2: Making and curing specimens for strength tests **ASTM D 1632 - 87** Standard Practice for Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory	Visual analysis and quantification of soft soil inclusions	**Section 5.2 of Denies et al. (2012c), after Ganne et al. (2011 and 2012)**
			Handling procedure for the preparation of DSM test specimens	**Section 5.3 of Denies et al. (2012c)**
			Density	**EN 12390-7: 2009** Testing hardened concrete - Part 7: Density of hardened concrete
			Unconfined compressive strength, UCS	**EN 12390-3: 2009** Testing hardened concrete - Part 3: Compressive strength of test specimens
			Modulus of elasticity, E	**NBN B 15-203: 1990** Concrete testing - Statical module of elasticity with compression **ISO/FDIS 1920-10: 2010** Testing of concrete - Part 10: Determination of static modulus of elasticity in compression
			Tensile splitting strength, T	**EN 12390-6: 2010** Testing hardened concrete - Part 6: Tensile splitting strength of test specimens
			Ultrasonic pulse velocity, V_p	**ASTM C 597 - 09** Standard Test Method for Pulse Velocity Through Concrete **EN 12504-4: 2004** Testing concrete in structures - Part 4: Determination of ultrasonic pulse velocity
			Porosity	**NBN B 15-215: 1989** Concrete testing - Absorption of water by immersion
			Hydraulic conductivity	**DIN 18130-1: 1998** Laboratory tests for determining the coefficient of permeability of soil

3.8. Influence of the unmixed soft soil inclusions

Within the framework of the Short Courses, Vervoort and Van Lysebetten (2012) presented the results of discrete simulations of uniaxial compression tests on DSM samples.

Numerical simulations are often limited to a continuum approach, assuming a linear elastic or elasto-plastic behavior of the material. However, failure is more complicated than only plastic deformation. Fractures are induced and propagate through the sample either in shear or in tension. Apart from the maximum strength, the stiffness and the stress-strain curve, discrete simulations also are able to simulate fracturing initiation and growth through the DSM material. By simulating the fracture process as realistic as possible, the study of the effect of heterogeneities, such as the unmixed soft soil inclusions remaining into the soil mix, is broadened and facilitated.

As described in Van Lysebetten et al. (2013), the discrete simulations are performed in UDEC, a 2D discrete element software package. The simulated models are based on a real sample (120 × 240 mm) taken from a cylindrical DSM column (see Fig. 34). The model is built up by a triangular mesh of discrete blocks as illustrated in Fig. 35. These blocks can only deform elastically and are interconnected by contacts which bound them tightly together. Therefore, the contacts are modeled with a certain strength and stiffness in normal and tangential direction, giving a specific strength and stiffness to the sample. Moreover, the contacts act as potential fracture paths, or in other words, they do not represent a physical crack as long as they are not activated. Note that the contact parameters are not physically measurable.

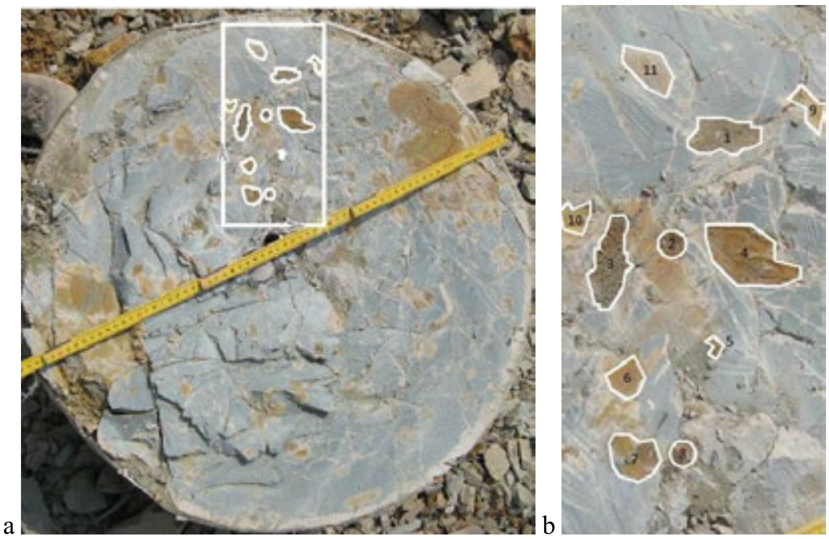


Figure 34: (a) Top view of a cylindrical soil mix column. (b) Sample (120 × 240 mm) based on a real soil mix section, from Vervoort and Van Lysebetten (2012), more details in Vervoort et al. (2012)

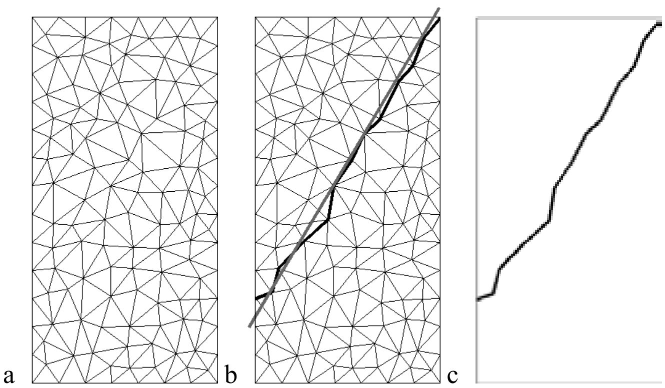


Figure 35: (a) Simplified example of adjacent triangular blocks that form the sample. A contact only represents a physical crack when it is activated. (b) Fracture along activated contacts with a global dip of 60° (straight line). (c) Apart from the activated contacts the sample is still intact, from Van Lysebetten et al. (2013)

First, the contact parameters for a homogeneous sample (without unmixed soft soil inclusion) are calibrated based on the sample strength, stiffness and stress-strain behavior and the observed fracture pattern (see Fig. 36).

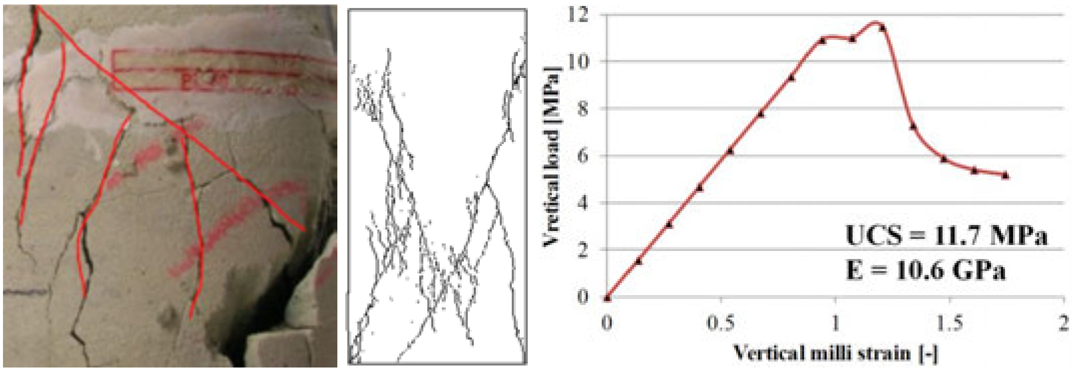


Figure 36: Calibration of model of homogeneous material on the basis of laboratory experimental results, from Vervoort and Van Lysebetten (2012), more details in Vervoort et al. (2012)

Concerning the influence of the unmixed soft soil inclusions onto the mechanical characteristics of the soil mix, the simulations show that the reduction of strength and stiffness is considerably larger than percentage of soil inclusions. This is illustrated by the figure 37 which presents the strength and stiffness in function of the percentage of unmixed material. A mere 1% of weak inclusions reduce strength and stiffness with on average 13 and 3%, respectively. For 10% of weak inclusions more than half of the strength disappears and the stiffness is reduced with 32% on average. Moreover, for the strength there is an overlap between the considered percentages of unmixed material. For example, the maximum strength of samples with 10 and 20% of unmixed material is larger than the minimum strength of samples with respectively 5 and 10% of unmixed material. This means that other parameters than the percentage of weak inclusions have an important influence.

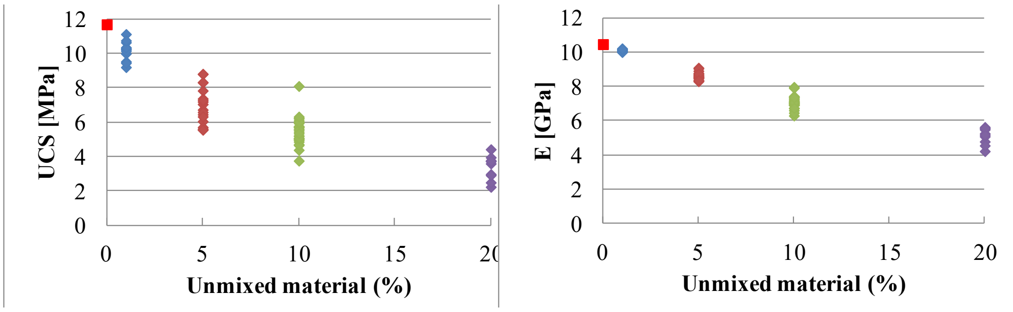


Figure 37: Variation of (a) strength and (b) stiffness as a function of the percentage of unmixed material, from Vervoort and Van Lysebetten (2012), more details in Vervoort et al. (2012)

Figure 38 shows the effect of the number of inclusions and their shape on strength and stiffness for 30 models with 10% of weak inclusions. It is observed that sharp-ended inclusions reduce strength and stiffness more than rounded inclusions, at least for the same number and size of inclusions. Moreover, strength is reduced more by larger inclusions (i.e. less inclusions), at least for the same shape and percentage of inclusions.

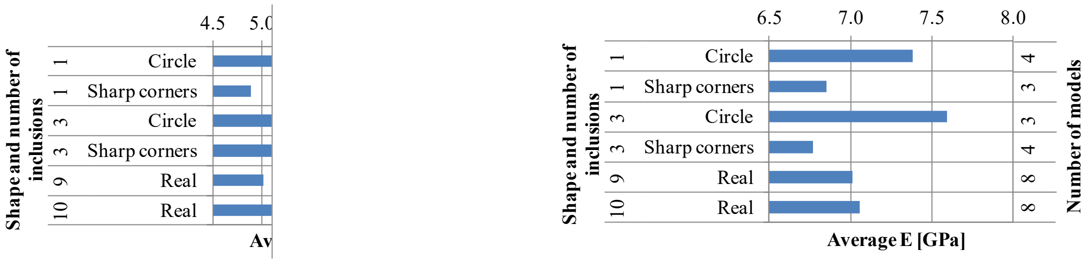


Figure 38: Effect of the number of inclusions and their shape on (a) strength and (b) stiffness for 30 numerical models with 10% of unmixed material, from Vervoort and Van Lysebetten (2012), more details in Vervoort et al. (2012)

3.9. Influence of the scale effect

Apart from the results of discrete simulations, Vervoort and Van Lysebetten (2012) also report on a study about the scale effect of the strength and stiffness of soil mix material. Therefore, the results of large scale testing of soil mix blocks (dimensions of approx. 0.5 × 1.2 m) are compared with the results of UCS tests on samples cored from the same CSM panel (dimensions of approx. 0.1 × 0.2 m), such as illustrated in Fig. 39.

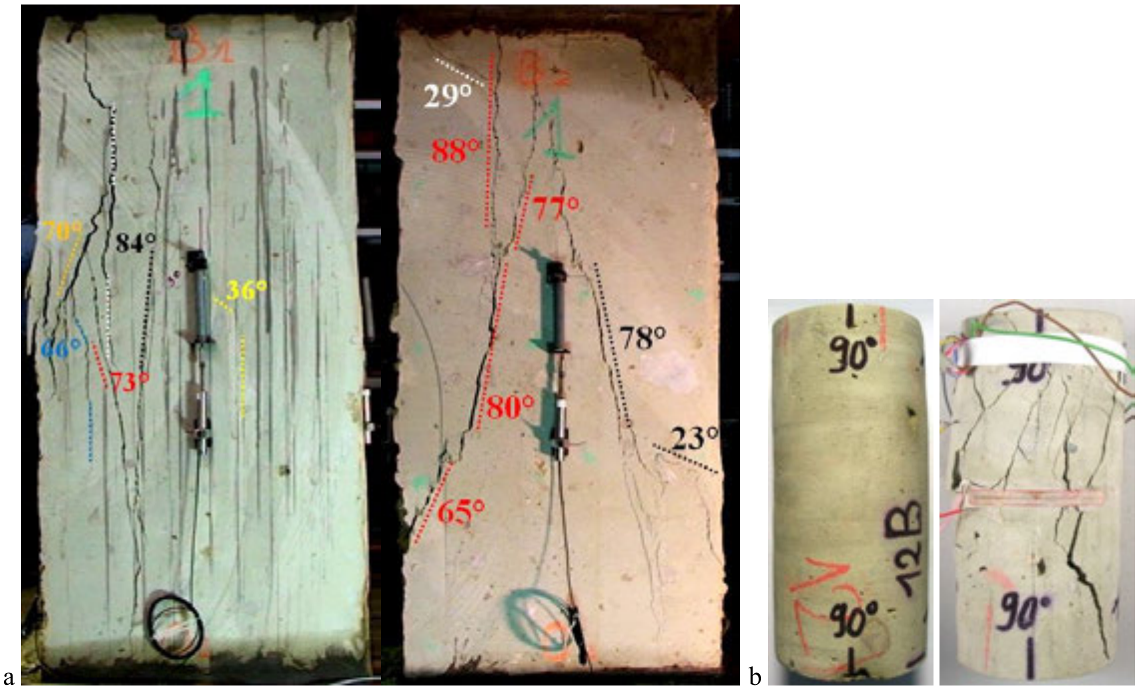


Figure 39: (a) Picture of the observed fracture pattern on the face perpendicular to the soil-wall contact of blocks 1 and 2 executed in tertiary sands after the uniaxial compression test. (b) Pictures of soil mix samples before and after UCS testing (with a diameter of 94 mm and a length of 200 mm), from Vervoort and Van Lysebetten (2012), more details in Vervoort et al. (2012)

The results of four tested blocks were presented, originating from 3 different Belgian construction sites in different soil types (quaternary sand, mixed material with construction debris and tertiary sand). The CSM panels executed in the homogeneous quaternary and tertiary sands clearly contained lower percentages of inclusions, while the panel executed in the heterogeneous soil with construction debris resulted in much higher percentages of inclusions. The details of the tested samples and their results are summarized in Table 13.

A first observation is that the block and core samples coming from the heterogeneous soil resulted in the lowest maximum strength and stiffness (see Table 13 and Fig. 40). Second, it is observed that for homogeneous soils the strength of the blocks is about 30% smaller than the UCS values of the corresponding small scale samples. In the heterogeneous soil, the strength of the block is about 50% smaller than the core samples. With regard to the Young's modulus, no clear effect has been observed.

Results of extra real-scale compression tests have been recently published in Denies et al. (2013). Additional information on scale effect will also be given in the proceedings of the DFI-EFFC conference (Denies et al. 2014).

Table 13: Overview of large scale tests and cored samples for the three different sites studied, from Vervoort and Van Lysebetten (2012), more details in Vervoort et al. (2012)

Site	**Knokke**	**Wetteren**	**Leuven (IMEC)**	
Soil type	Quaternary sand	Mixed soil and construction waste	Tertiary sand	
Dimensions (cm)	61 × 53 × 124	55 × 48 × 90	57×75×119	58×53×120
Maximum strength	**8.3 MPa**	**2.1 MPa**	**4.8 MPa**	**4.2 MPa**
E_{tg}	**13.6 GPa**	**2.9 GPa**	**5.6 GPa**	**5.5 GPa**
Diameter cores (mm)	114	113	94	
Height cores (mm)	230	230	200	
UCS (MPa)	**11.1-12.4 MPa**	**3.4-4.9 MPa**	**5.0-7.6 MPa**	
E_{tg} (based on SG)	**12.5-13.2 GPa**	**1.3-2.7 GPa**	**5.6-6.9 GPa**	

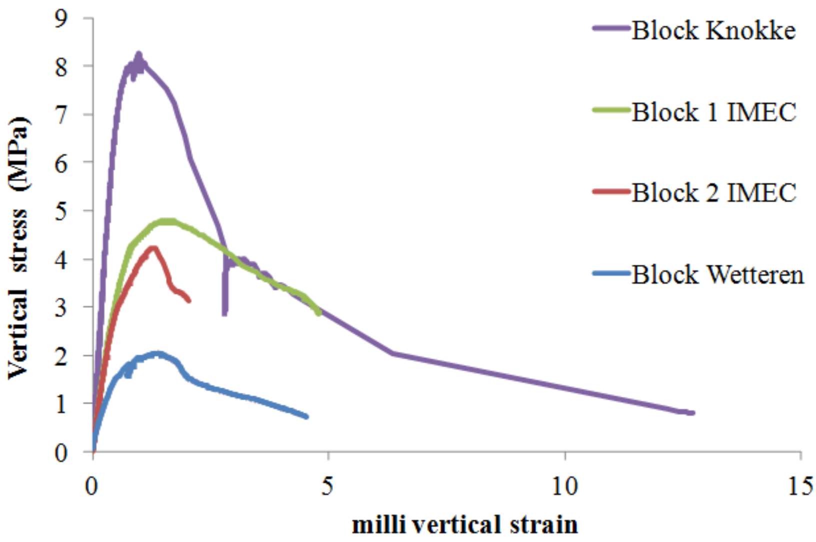


Figure 40: Stress-strain curves of the uniaxially compressed blocks from 3 different construction sites in Belgium, from Vervoort and Van Lysebetten (2012)

4. FIELD OF APPLICATIONS AND CASE HISTORIES

4.1. DMM as an alternative to traditional foundation solutions

With regard to the world population growth and in response to the expansion of our society, there is an increasing need of establishing new constructions on soils of poor quality and especially in alluvial area. That need is generally coupled with a challenging time schedule and economic criteria. In this respect, DMM constitutes an interesting **alternative to the traditional foundation solutions** allowing construction on soft/weak/alluvial/compressible soils.

As illustrated from Fig. 41 to 43, a lot of case histories or examples were presented within the framework of the Short Courses in order to demonstrate the possibilities related to the permanent use of soil mix material for foundation of roads, bridges, multi-storey structures, linear structures, pipes and even for power plants and wind turbines (see Topolnicki and Sołtys, 2012 for more details). Finally, as previously discussed in Section 2.5, the development of adapted soil mix spreadable systems expands the use of the deep mixing method to the underpinning works and to the reinforcement of existing railway platforms (see Fig. 11 and 12).

4.2. Soil mix walls as excavation support: earth and water retaining structures

If originally, the deep mixing method was developed for ground improvement applications, over time it was progressively dedicated to various structural and environmental applications. In Belgium, for example, the use of the soil mix as building material for the construction of **earth/water retaining structure for an excavation** is become very common. It can equally be supported by anchors or shoring systems and really represents an economic solution for the realization of retaining structures. Figures 44 and 45 illustrate some examples of soil mix walls. Retaining walls can also be designed as silo structure or pit for the entrance and exit of **(micro)tunneling activities**.

The Short Course of Leemans (2012) highlights an interesting case study of composite retaining wall in the downtown of Aalst in Belgium for the construction of a 3-storey car park below grout level (12m depth). The dewatering of the region was not allowed because of settlement risks. The top soil layers presented a large amount of peat and soft loamy clay. Horizontal permeability inferior to 10^{-8} m/s was required in the project specifications and the lateral displacement of the retaining walls was limited to 6 cm. For the design solution a combination of techniques was envisaged with the realization of a composite retaining wall.
A CSM wall was first executed making easier the installation of sheet piles (without vibration or impact, as required in the specifications of the project). The CSM wall was installed in the impermeable clay layer at a depth of 21 m (to allow the dewatering of the excavation). Then the sheet piles were sunk into the fresh mix until a depth of 15 m.
Figure 46 presents the situation of the construction site and figure 47 illustrates the cross section of stability calculation (without representation of the CSM wall).
The CSM wall has a temporary water retaining function (during construction stages).
The sheet piles have a double role. During the construction, they are already part of the stability of the excavation but at long term they not only ensure this stability but also play the role of watertight barrier.
The interlock of the sheet piles was welded before they were inserted into the fresh soil mix material. In a similar way, the anchorage lock and shoe were also welded before placing the sheet piles. Figure 48 illustrates the installation of the sheet piles into the fresh soil mix material and figure 49 the progressive excavation and anchoring of the composite retaining wall. Specific measures were taken into account to ensure the waterproof qualities of the wall, as mentioned in Leemans (2012).
The soil mix material of the CSM wall was investigated within the framework of the BBRI 'Soil Mix' project (2009-2013). Extra CSM panels executed on the same site with similar execution parameters and slurry properties were excavated (see Fig. 50), transported in laboratory and characterized (refer to Table 14 for test results). As described in this table, the soil mix material was studied with the help of classical tests performed on core samples but also with real-scale tests conducted on large soil mix elements. Indeed, three extra CSM panels had been executed. The first one was cored and cut to obtain core samples and large soil mix blocks for large compression tests such as described in *Vervoort et al. (2012)*. The two others were dedicated to four real-scale bending tests on half CSM panels. The results of these real-scale tests will be published in the proceedings of the DFI-EFFC conference (Denies et al., 2014).

Figure 41: Examples of various structures supported by DSM elements – first part, after Topolnicki (2012) and Leemans (2012)

Figure 42: Examples of various structures supported by DSM elements – second part, after Topolnicki (2012)

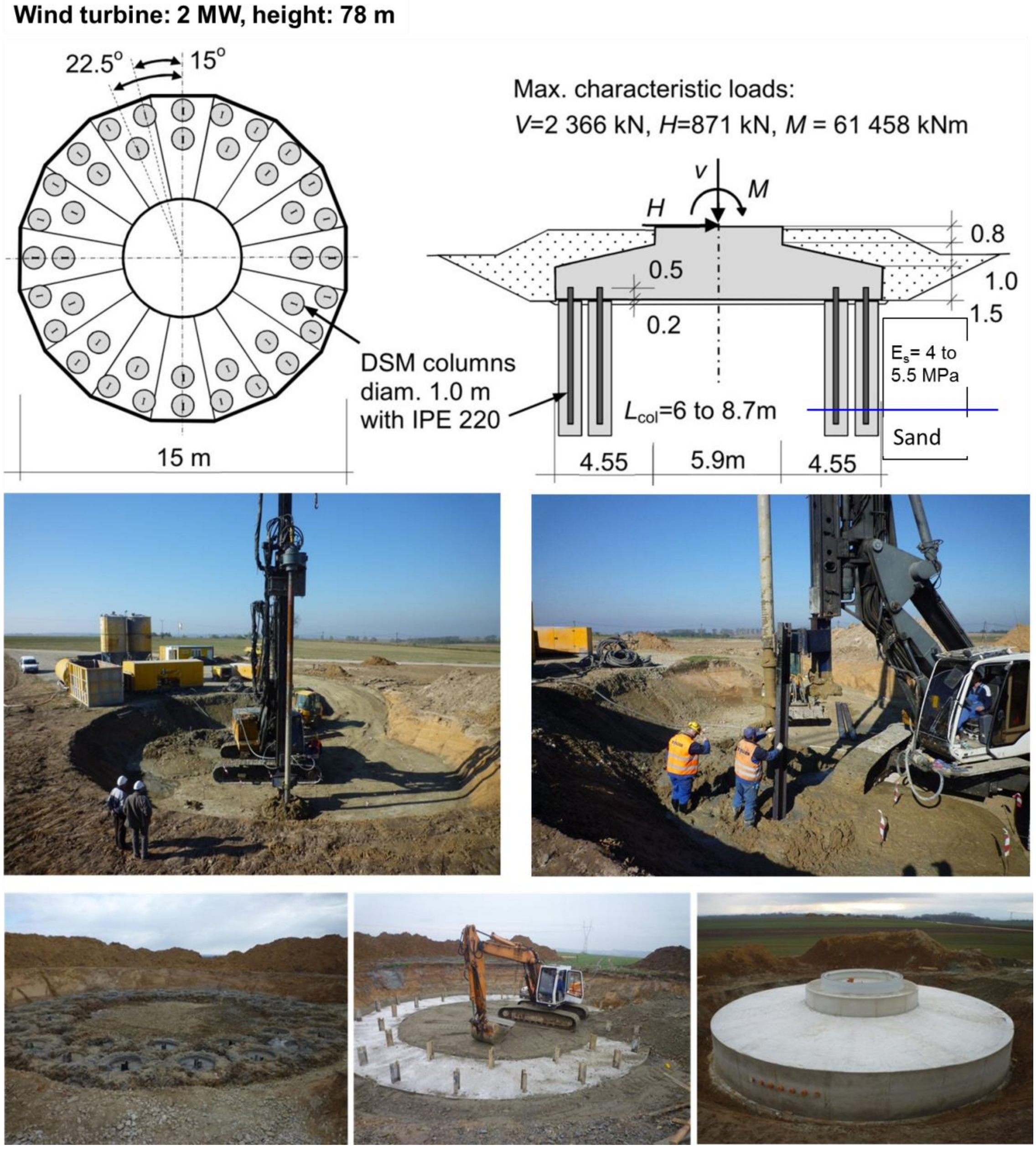


Figure 43: Examples of various structures supported by DSM elements – third part, after Topolnicki (2012)

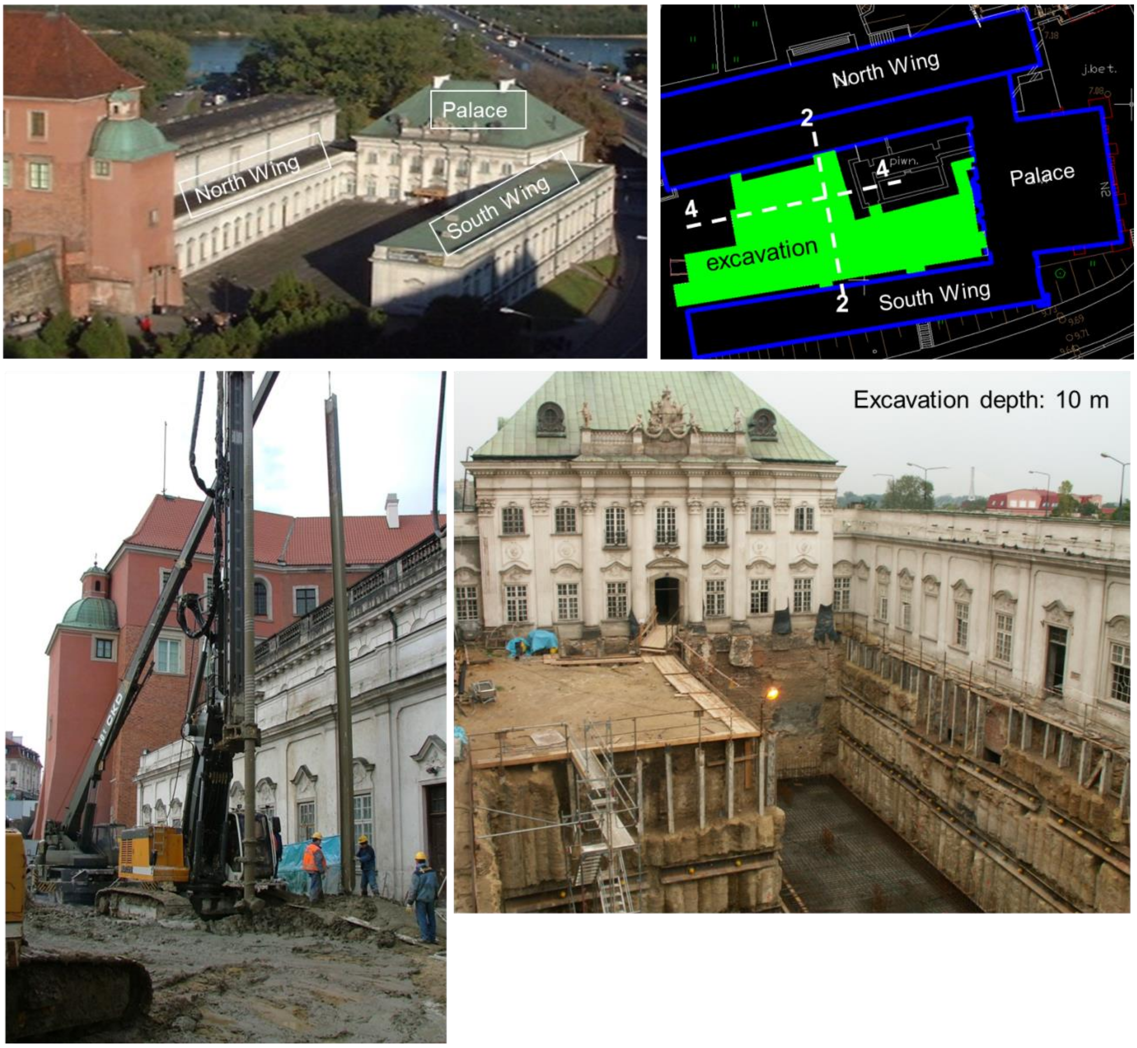


Figure 44: Anchored soil mix walls for an excavation near the Royal Castle courtyard in Warsaw (Poland), after Topolnicki (2012)

Figure 45: Shored CSM walls in Belgium, after Leemans (2012)

Figure 46: Excavation in the downtown of Aalst (Belgium), after Leemans (2012)

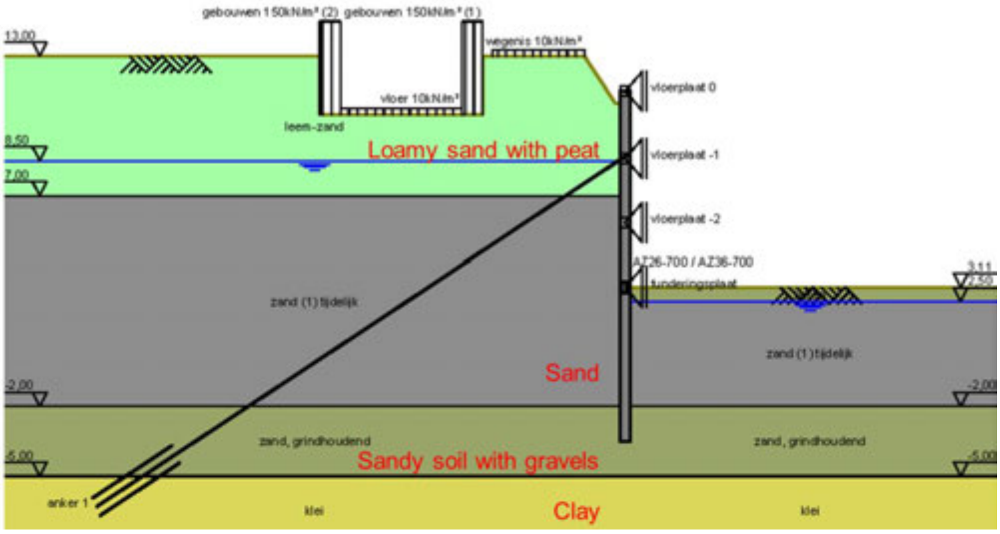


Figure 47: Cross section of stability calculation, after Leemans (2012)

Figure 48: Installation of the sheet piles into the fresh soil mix material, after Leemans (2012)

Figure 49: Progressive excavation and anchoring of the composite retaining wall, after Leemans (2012)

Figure 50: Excavated extra CSM panels for mechanical characterization, after Leemans (2012)

Table 14: Mechanical characterization of the soil mix material executed in Aalst, from BBRI 'Soil Mix' project (2009-2013)

Test results obtained from in situ core samples	
Main UCS value	**7.31 MPa**
Main value of the modulus of elasticity, E	8.40 GPa
Main value of the tensile splitting strength	1.31 MPa
Line percentage of unmixed soft soil inclusions into the soil mix material (see *Denies et al. 2012c*)	2.6%
Porosity	47.2%
Coefficient of permeability	< 8 10^{-11} m/s
Test results obtained from 2 real-scale compression tests on large soil mix samples	
UCS values for large soil mix samples	5.2 and 4.1 MPa
E values for large soil mix samples	6.0 and 6.0 GPa

Alternately the soil mix material can be used for the construction of temporary or permanent **cut-off walls**, used as seepage barriers to limit the flow of water (including or not contaminants). Figure 51 presents the construction of a cut-off wall performed with the trenchmix® system for the improvement of a dike along the Rhone river in Aigle (Switzerland). The dike ensures the protection of an industrial site against flooding.

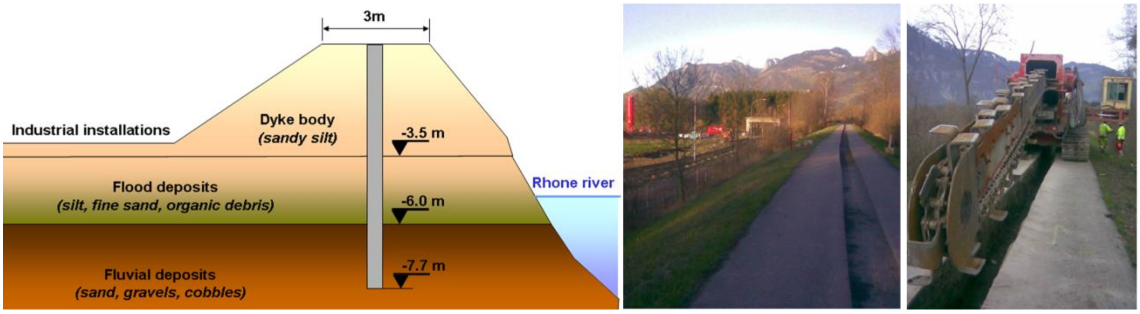


Figure 51: Improvement of a protection dike with a cut-off wall performed with the trenchmix® system, after Borel (2012)

4.3. DMM for land levees and floodwalls

In the recent years, the use of the deep mixing method for the realization or the reconstruction of embankments seems to become the favorite alternative in USA. In the case of land levees or floodwalls, the soil mix walls play not only the role of watertight barrier but take part in the design of the foundation of the embankment (see Fig. 52) and provide added stability regarding to the potential shearing (circular) failure (see Fig. 53). Within the framework of the Short Courses, Filz (2012) concentrates on the design of **land levee reinforced with soil-cement columns**.

A design and construction flow chart to illustrate the importance of realistic analyses was first presented (see Figure 54). It consists of four main project phases: (1) information collection; (2) analysis and design; (3) contractor procurement; and (4) construction with continuous quality control and quality assurance, which are all extensively discussed in Filz et al. (2012). The key outputs of the design process are the geometry and strength requirements for the DSM ground, which are communicated to the contractor through the plans and specifications for construction and verified during the construction process by QA/QC activities. Of course, this loop only provides assurance if the analyses at the heart of the design process really represent the behavior of the system. Therefore, two particularly important factors must be taken into account: multiple potential failure modes, such as illustrated in Fig. 55, and the relatively high variability of DSM material.

According to Filz (2012), a limit equilibrium slope stability analysis only takes into account shear failure. In order to capture other failure mechanisms, numerical analyses should be performed. When other failure mechanisms (such as column bending and tilting) are allowed, the calculations lead to lower and more realistic values of safety factors, especially for isolated DSM columns. The difference is smaller for continuous shear panels, which perform much better than isolated columns for resisting lateral loads (note that the column overlap can be important also to avoid vertical shearing).

The strength of DSM material has a relatively high variability. According to Filz (2012), the strength of DSM material is about twice as variable as the strength of natural clay deposits. Of course, this variability has implications for the selection of appropriate design strengths. It can be taken into account by performing reliability analyses. Alternatively, if design is based on deterministic calculations, the specified strength of DSM material should be adjusted to obtain a design value that accounts for its variability (Filz et al., 2012).

For routine design work, simplified analyses that capture multiple failure modes and the variability of DSM material properties may be useful. Such simplified analyses for each of the critical failure modes were presented by Filz (2012). They are in large part based on the procedures proposed in CDIT (2002), although some adaptations have been made. In order to estimate the shear strength of DSM material to be used in simplified stability analyses, Filz (2012) proposes the following formulation:

$$s_{dm} = \frac{1}{2} f_r f_c f_v q_{dm} \tag{7}$$

where s_{dm} is the design shear strength of the DSM material, f_r the factor for residual strength equal to 0.8 according to CDIT (2002), f_c the curing factor, f_v the variability factor, and q_{dm} the contract specified UCS value of the DSM material. These factors can be calculated as explained in Filz et al. (2012).

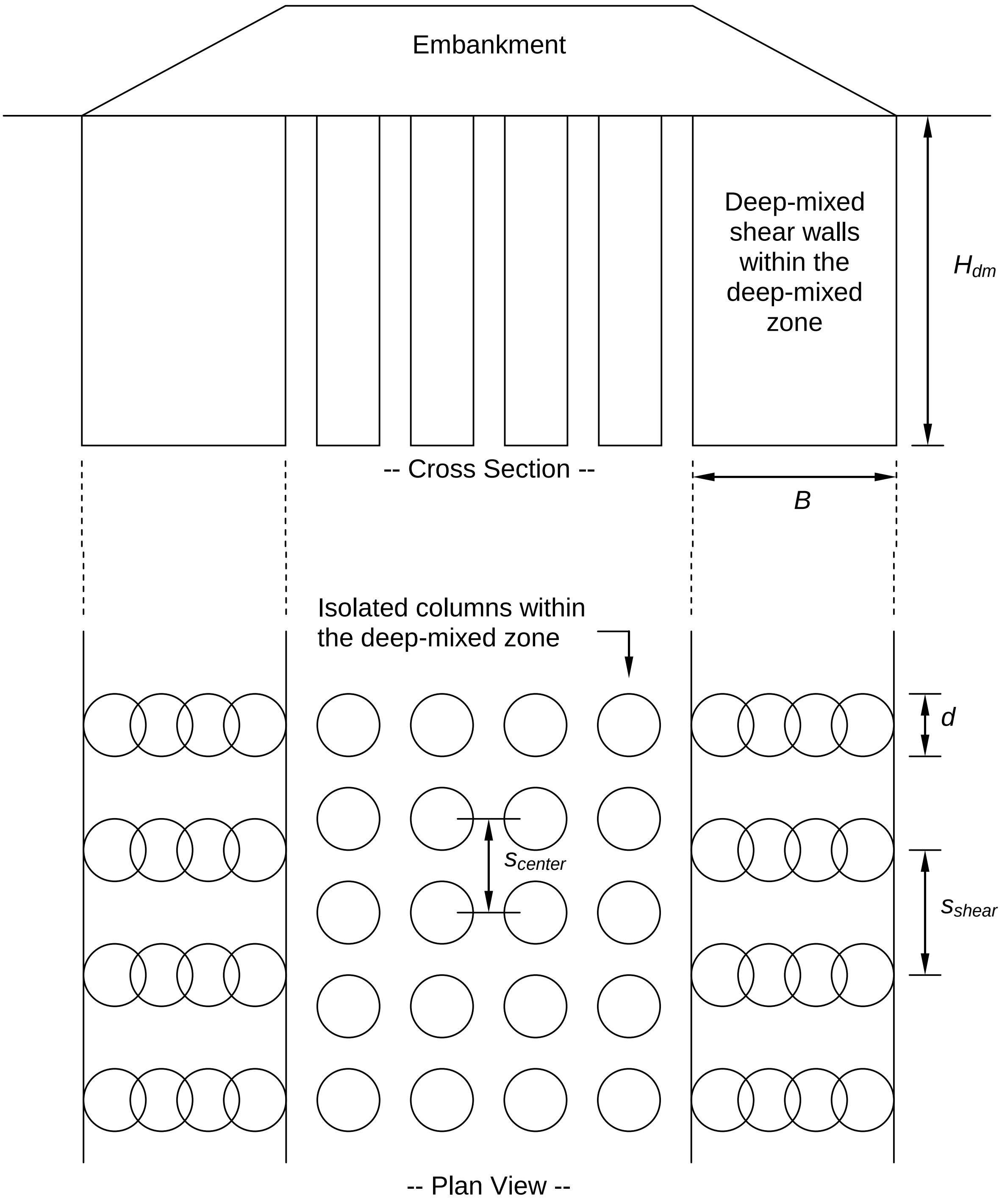


Figure 52: Illustration of the design of an embankment supported by DSM columns, after Filz (2012)

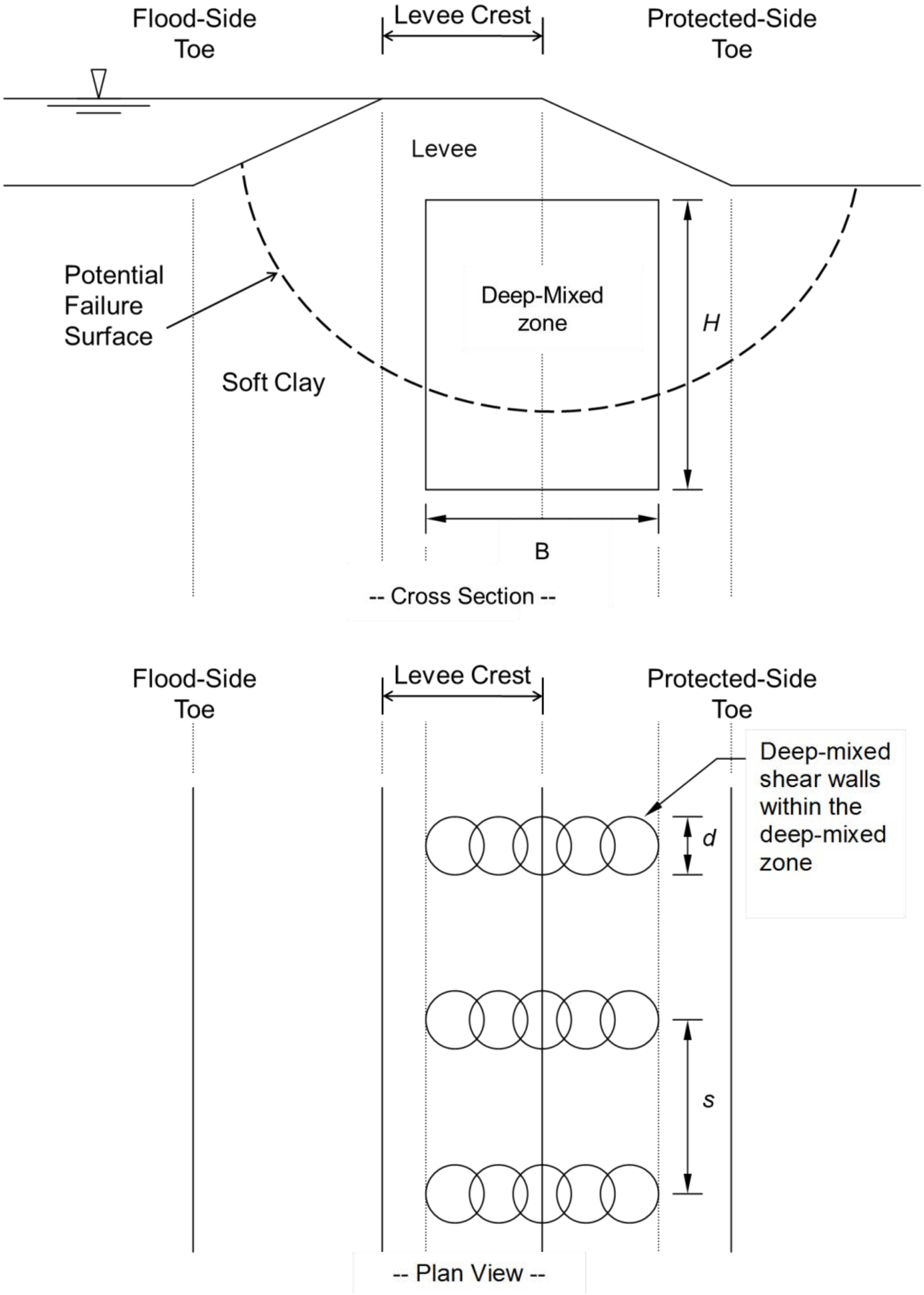


Figure 53: Reinforcement of land levee with soil-cement columns, after Filz (2012)

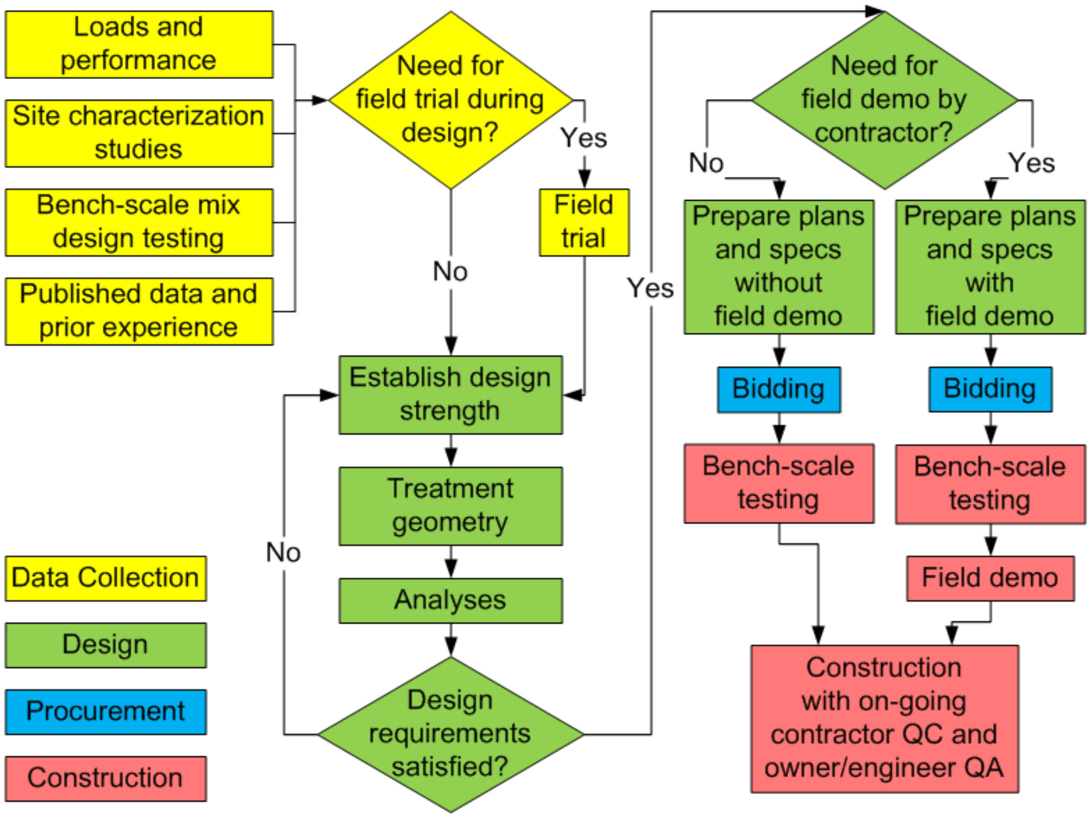


Figure 54: Flow chart for design and construction of DSM support systems for embankments and levee, from Filz (2012), more details in Filz et al. (2012)

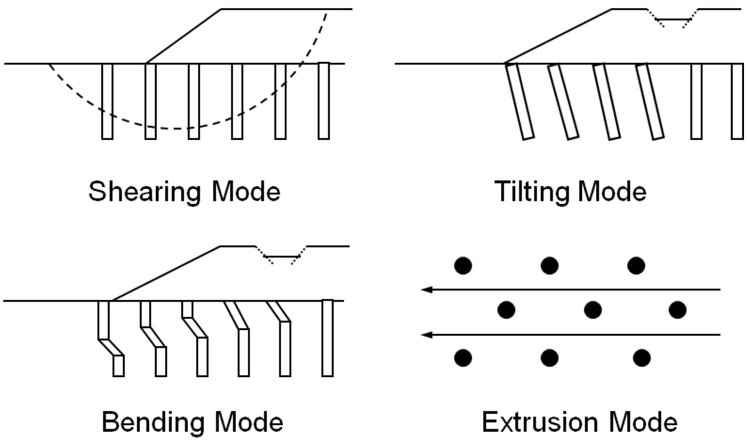


Figure 55: Stability failure modes for embankments supported on DSM columns, from Filz (2012)

4.4. Soil mix remediation technology

As another interesting application of the DMM, Al-Tabbaa (2012) presented the Soil Mix Remediation Technology (SMiRT) R&D project (2008-2011) and in particular the field trials that took place within the framework of this project. The project aimed to achieve significant technical advancement and cost-savings by developing innovative soil mix systems for integrated remediation and ground improvement. The investigated applications include 3 main contaminated land remediation techniques:

- **Cut-off containment walls:** in-ground, low permeability barriers which encapsulate the contaminated area and isolate it from the surrounding environment. Most commonly, cement-bentonite slurry trench walls (with geomembranes) are used;
- **Stabilisation/solidification (S/S) treatments:** the physical encapsulation and chemical fixation of contaminants in place through a range of processes including sorption, precipitation, lattice

incorporation, complexation and encapsulation (*Al-Tabbaa et al., 2012*). Commonly, a range of backhoe systems, mixers and blenders is used;
- **Permeable reactive barriers (PRBs):** permeable walls installed in the ground to intersect the flow of contaminated groundwater. Reactive material placed in the barrier is designed to remove the contaminants by one or more processes, including sorption, precipitation, oxidation, biodegradation and encapsulation.

According to *Al-Tabbaa et al. (2012*), the soil mix technology is able to perform all three contaminated land remediation techniques with numerous technical and environmental advantages in comparison to alternative technologies. Moreover, its application produces very little spoil and thus reduces off-site disposal problems. In addition to this, it reduces the surface exposure and emissions of the contaminated soil. Soil mix technique can deal with sites of any size and with multiple contaminants. This made the DMM a promising and timely contender to lead the market place in offering a cost-effective, efficient and low risk solution to contaminated soil and groundwater remediation.

The field trials took place in a previous chemical works site in the north of England. The site consisted of up to 4 meters made ground, 0 to 1 meters of silts and clay and 3 to 4 meters of natural sand and gravel deposits, before bedrock was found at about 8 meters depth. The contamination consisted of high levels of heavy metals: Pb, Zn, As, Cr, Cu and Ni and significant organic contamination including VOCs, SVOCs, TPHs and PAHs. The groundwater was also heavily contaminated, mainly with the organics above and some limited metal contamination.

A number of soil mixing systems were used in the field trials: a triple auger system, the ALLU mass stabilization system, a standard single auger system mounted at the end of a CFA pile shaft and a double rotary head auger developed by Eco Foundations (for more details see *Al Tabbaa et al., 2012*). A large number of stabilizing materials were also employed, namely Portland cement, Ground-granulated blast-furnace slag GGBS, Pulverised Fuel Ash PFA, pre-bagged PC-GGBS CEM III cement, MgO, zeolite, organoclay and natural bentonite and chemically modified bentonite slurries.

The layout of the various field trial activities are shown in Fig. 56.

The hexagon system (upper left corner) is shown in more detail in Fig. 57. As described in *Al-Tabbaa et al. (2012)*, the inner hexagon is a permeable reactive barrier system and was constructed using the triple auger system. In each side of the hexagon different reactive materials were introduced in slurry form and mixed with the soil down to a depth of 8 meters, keyed into the bedrock layer. The outer hexagon and radial wall sections are low permeability sections and were constructed using cementitious binders. They serve as a reactive low permeability S/S system as well as to hydraulically isolate the individual PRB sections and the whole system from the surrounding environment. In the centre of the hexagon, in the middle of each of the 6 outer sections and around 20 meters away from the hexagon, wells were installed to create certain groundwater flow in order to assess the performance of the reactive barrier and cut-off walls.

The triple auger system was also used to install individual S/S sections in which a wide range of binders and binder contents were tested as well as a number of different installation variables including speed of rotation, penetration and withdrawal rates and a number of mixing tools.

The ALLU mass stablisation system was used for the improvement of the soft made ground soils in the areas of very low or no contamination. Again, different binders, binder compositions and installation variables were tested.

The single auger and double rotary auger were used to install 5 columns each.

The performance of the executed contaminated land remediation systems is currently still being investigated based on groundwater samples and soil samples. However, the field trials have so far demonstrated the versatility and ease of application of the soil mixing systems in remediation applications.

4.5. Overview of the soil mix applications

Finally, table 15 presents a summary of all the soil mix applications discussed within the framework of the Short Courses.

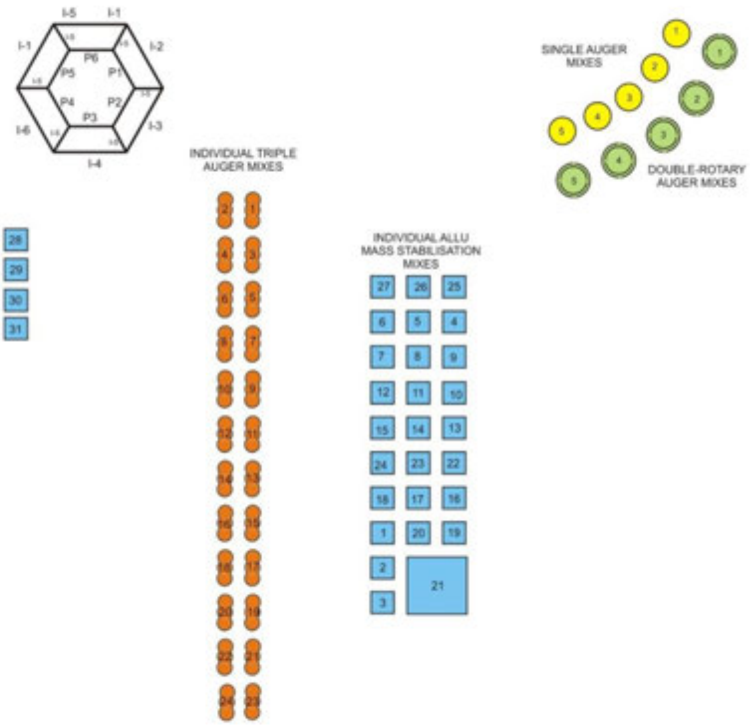


Figure 56: Plan of field trial treatments, from Al-Tabbaa (2012)

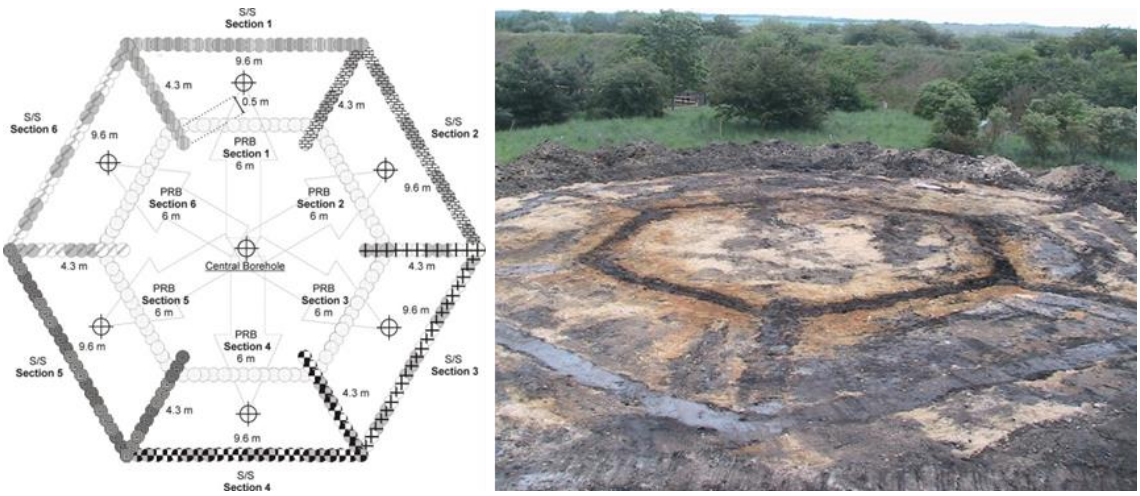


Figure 57: Plan of field trial treatments – details and photography of the hexagon, from Al-Tabbaa (2012)

Table 15: Field of application of the soil mix technology

Soil reinforcement and foundations (as an alternative to classical foundation, for underpinning, for railway tracks reinforcement)
Earth/water retaining wall (for excavation, as silo structure, as pit for (micro)tunnelling activities)
Cut-off wall
Floodwall
Reinforcement of land levee and embankment
Slope stabilization
In situ remediation (permeable reactive in-ground barrier PRB, containment walls and 'hot-spot' soil treatment by stabilization/solidification S/S)
Mass stabilization
Barrier against liquefaction
Land reclamation

5. QA/QC PROCEDURES AND MONITORING

An important part of a deep mixing project concerns the quality control and quality assurance (QA/QC) activities. If this topic was not specifically treated during the Short Courses, it was actually discussed in various papers of the symposium. Denies and Van Lysebetten (2012) have summarized the major aspects in this regard in their General Report considering: the QA/QC activities, the workflow of deep mixing project, the deep mixing process design in practice, the execution monitoring, the different lab and field characterization tests and their procedures and the interest of a global monitoring plan. Moreover, one can note that for the US practice QA/QC procedures have been recently published on the website of GeoTech Tools (©Iowa State University, 2010-2013) and the 'FHWA Design Manual for Deep Mixing for Embankment and Foundation Support' is currently in press. The recent advances in Japanese QA/QC activities are reported in Kitazume and Terashi (2013).

If QC and QA are often misunderstood and used interchangeably, the following definitions (adapted from ©Iowa State University, 2010-2013) can still distinguish both.
By definition, quality control (QC) refers to procedures, measurements, and observations performed by the (deep mixing) contractor to monitor and control the construction quality such that all applicable requirements defined in the project specifications are satisfied. The monitoring during execution is then part of the QC activities.
QA refers to measures, measurements and observations performed by the owner or its representative to provide assurance that the construction has been realized in agreement with the project plans and specifications.

According to Maswoswe (2001), the critical factor in the execution of soil mix walls is to maintain an auger withdrawal rate consistent with the grout flow rate. One way to control the success of the procedure and its efficiency is to estimate the cement factor or cement content (the cement mass per cubic meter of soil mix material) at different locations. The cement factor can be estimated considering the grout flow rate, the auger withdrawal rate and the assumed percentage of grout loss during the process.
Beyond the mechanical characterization of the soil mix material, the continuity and the overlapping of the soil mix wall elements (columns or panels) must be proved with regard to the execution tolerances. Locations and verticality of the soil mix elements should be controlled during execution. **Hence, the best way to ensure QC during execution is not only by monitoring and adjusting the execution parameters but also recording and reporting them**.

5.1. Quality control by execution monitoring

Within the framework of the Short Courses, the manufacturers of deep mixing machines have highlighted the possibility of their equipment to perform continuous monitoring during execution. Control devices were first presented (see for example the figure 58 illustrating the control devices of the CSM equipment). The monitored data are controlled in real time during execution by a monitor display (see Fig. 59) and production logs are finally produced and provided to the engineers responsible for the QA (see Fig. 60).

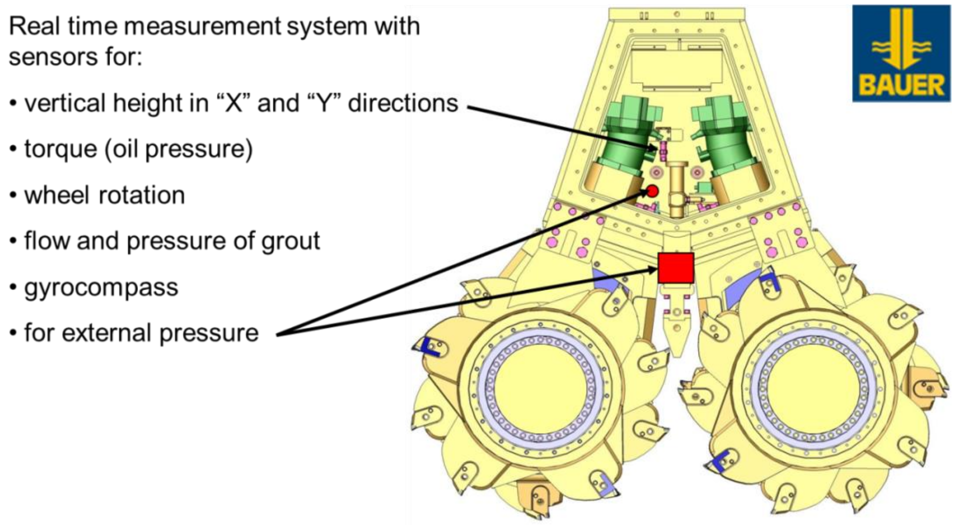


Figure 58: Equipment overview of the CSM – presentation of the control devices, from Gerressen (2012) with the courtesy of Bauer

Figure 59: Equipment overview of the CSM – real time representation of the monitored data, from Gerressen (2012) with the courtesy of Bauer

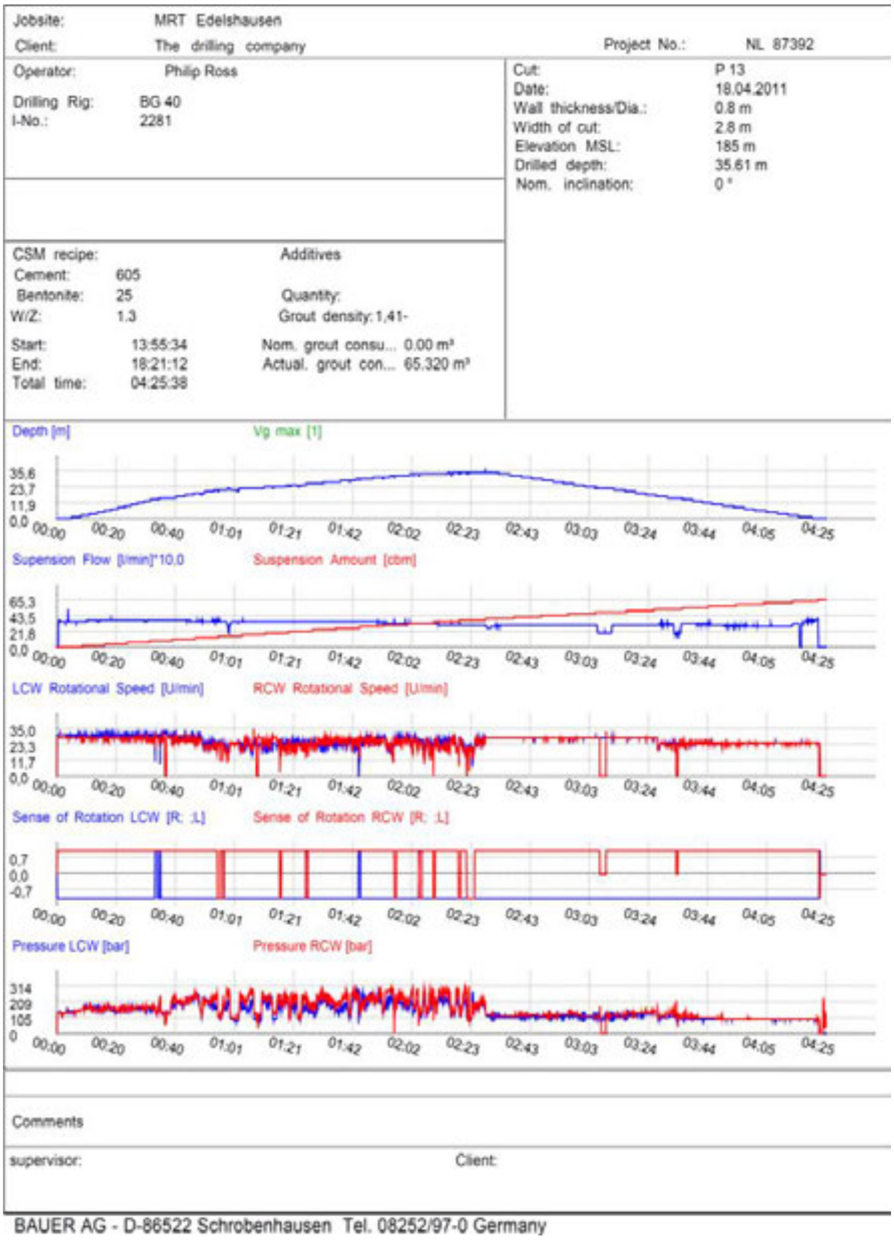


Figure 60: Example of production log of the monitored data for the realization of a CSM panel, from Gerressen (2012) with the courtesy of Bauer

5.2. Structural monitoring with optical fibers

After realization of the soil mix elements, the implementation of a monitoring plan during the construction of the structure (or permanently – during all its lifetime) allows the reduction of the risk for the structure itself but also for the neighboring constructions. Preventive or rescue measures (if necessary) can then be applied. In the proceedings of the IS-GI 2012, several authors report temporary or long-term monitoring plan for temporary and permanent soil mix structures in the cases of earth/water retaining wall, foundation and slope stabilization applications.

Within the framework of the Short Courses, an interesting development for monitoring field performance of soil mix structures was presented by Huybrechts (2012): the optical fiber technology. The applied technology is known as the FBG/DTG optical fiber technology. A 195 µm diameter optical fiber is locally treated (Bragg grating, FBG). At the local FBG zone incident light waves (in a band of 1520 to 1600 nm) are reflected at a predefined wavelength, such as illustrated in Fig. 61. This reflected wavelength also depends linearly on temperature and deformation of the FBG zone. Once installed (in situ or in laboratory) inside or along the investigated element, it works as an extensometer with the possibility to obtain measurements of the local deformation of the element. Wavelengths are converted into deformations. Accuracies smaller than 5 microstrains or smaller than 0.5°C can typically be obtained. Up to 20 or more FBG sensors can be installed on one optical fiber. When installing the optical fiber according to the extensometer principle, average deformations of a certain measurement base (e.g. 0.5 to 1 m) can be monitored. In this way, local heterogeneities and anomalies do not influence the results. Because of the specific Draw Tower Grating technology, the optical fiber has a tensile strength up to a deformation of 6% (60 000 microstrains) or 1 to 2% for long term service.

As for other fiber optic technologies (e.g. Brillouin scattering), the measuring system has a long term stability and it is not influenced by electromagnetic radiation or stray currents. Moreover, it is not sensible to corrosion and short circuits (due to contact with water) and it has very small dimensions. Disadvantages are the fragility of the fiber and its sensitivity to temperature changes. Special attention should be paid to the fiber protection in alkaline environments (e.g. concrete).

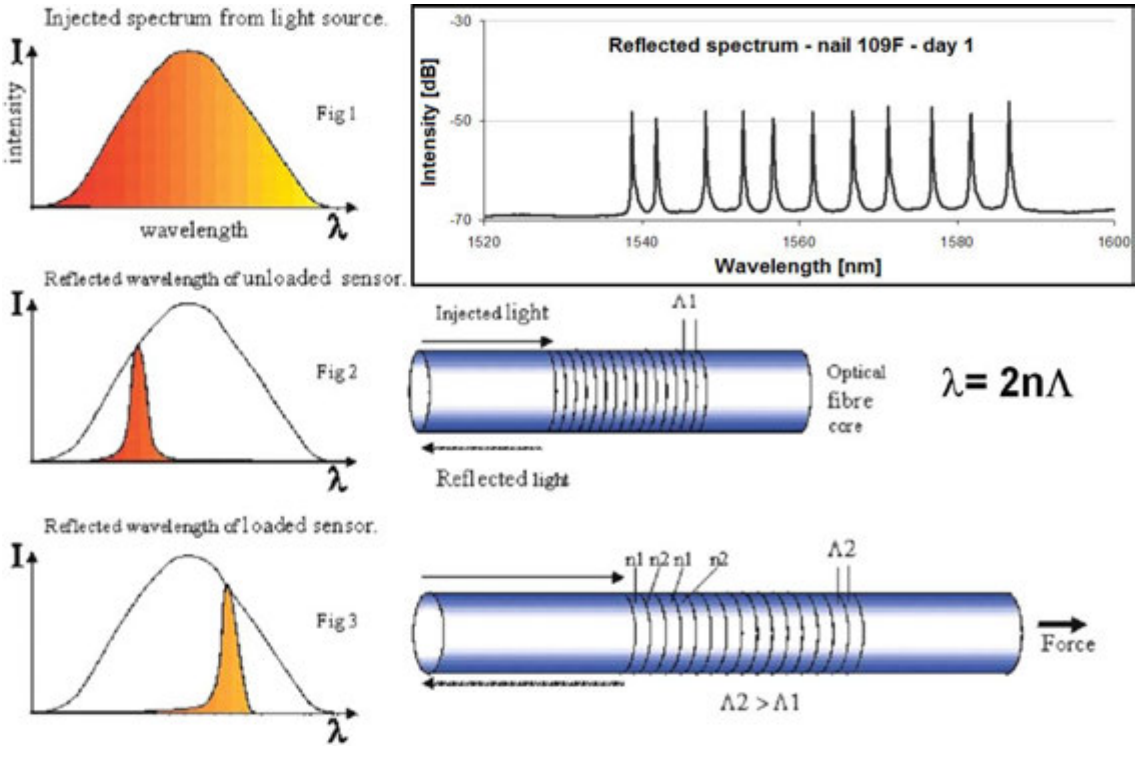


Figure 61: Illustration of the working principle of the FBG/DTG optical fiber technology. In the black box at the upper right corner: an example of the reflected spectrum of an optical fiber installed in a nail with 11 Fiber Bragg Gratings (FBGs). All 11 FBG sensors reflect the light at a certain wavelength, dependent on the production properties and the deformation and temperature of the FBG, from Huybrechts (2012).

As an example of application, this technology makes it possible to monitor the load distribution in the anchorage elements of an excavation (e.g. anchors, tension piles or nails). Figure 62a illustrates the strain distribution in a pre-stressed anchor measured with an optical fiber device with 16 FBG zones. The influence of increasing stress levels on the strain distribution is obviously demonstrated. The transition of bounded and unbounded length is also clearly visible. The compressive behavior of a bearing element (classical pile or soil cement column) could be also highlighted, such as illustrated in Fig. 62b presenting the results of a static load test (SLT) on a foundation pile. The evolution of the deformation along the pile depth can be observed for increasing load steps.

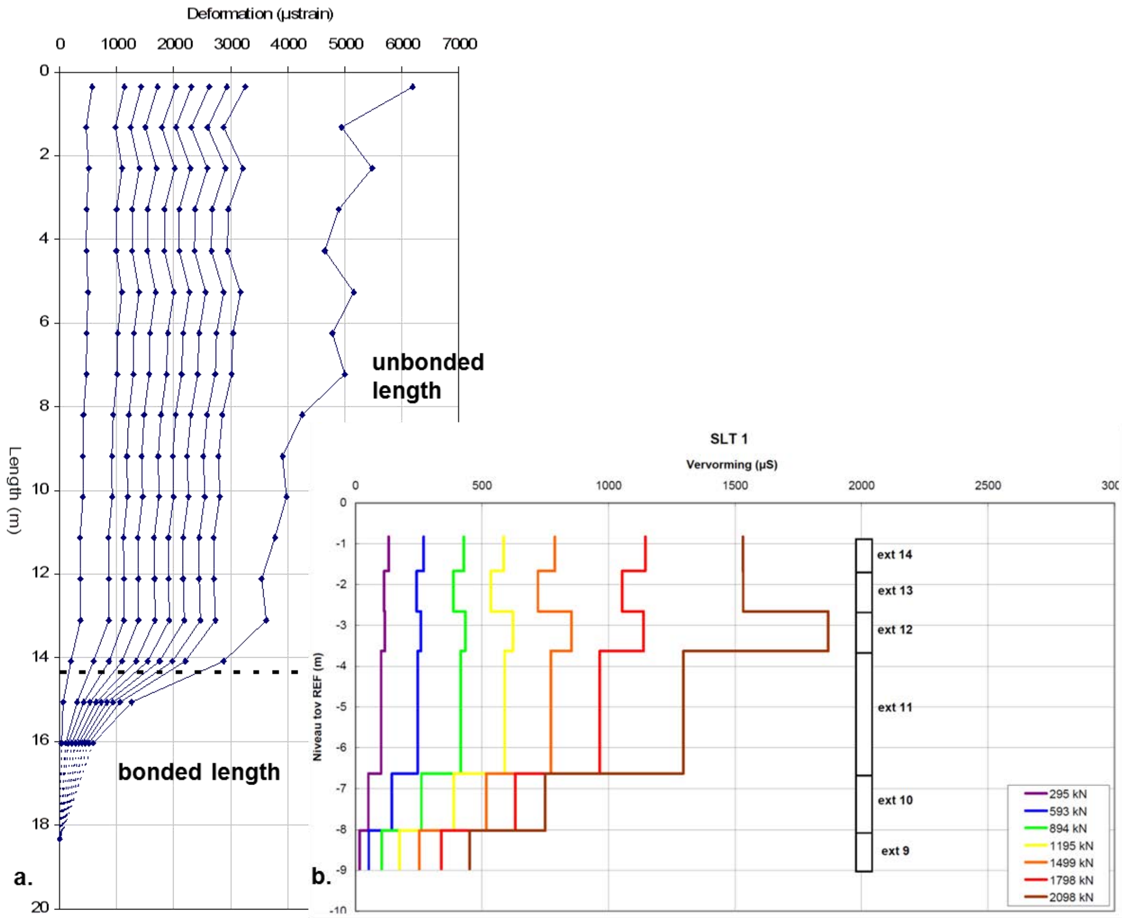


Figure 62: Examples of the results of optical fiber instrumentation (extensometer principle): (a) strain distribution in a pre-stressed anchor and (b) deformations measured during static load test (SLT) of a pile at successive load steps, from Huybrechts (2012)

The optical fiber system can be integrated in the DSM element as lost or retrievable system.

For lost systems, the instrumentation is connected to the steel reinforcement element or another carrier that is integrated in the soil mix element during installation (see Fig. 63). Alternatively, the lost instrumentation can be installed after installation of the soil mix element with the help of a reservation tube either directly installed into the soil mix or connected to the reinforcement element (see Fig. 64). The minimum internal diameter of the reservation tube is 16 mm in order to allow grouting after the installation of the lost instrumented line into the tube.
As illustrated in Fig. 65, retrievable systems also use a reservation tube, but with a minimum internal diameter of 30 mm.

Huybrechts (2012) emphasizes the expected high added value of optical fiber monitoring systems in particular with regard to the soil mix elements. It will contribute to a better knowledge of the real scale behavior of soil mix structures and it will help to optimize design methods. In this view, results of real-scale bending tests performed on soil mix columns and panels equipped with lost optical fiber systems (connected to steel reinforcement) will be published in Denies et al. (2014).

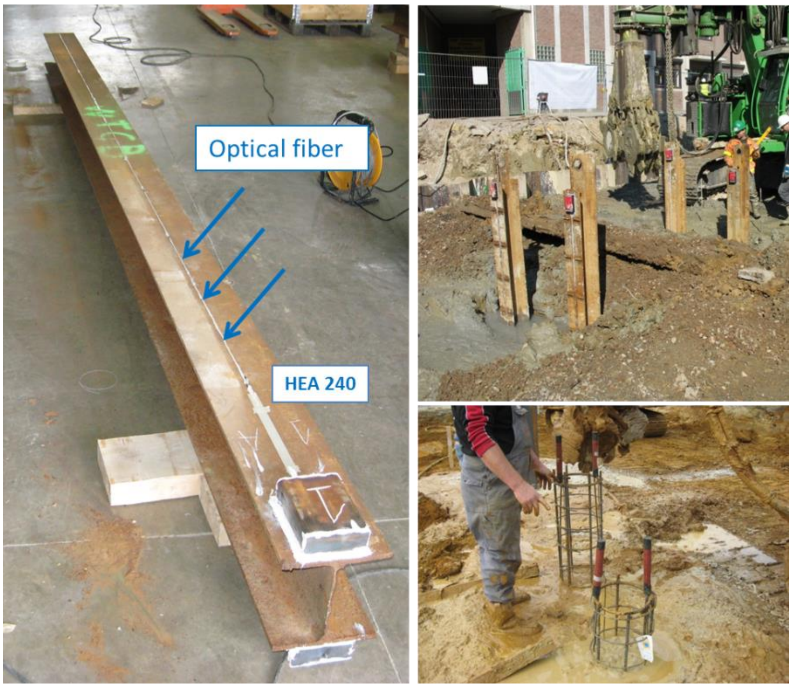


Figure 63: Examples of lost instrumentation systems connected to the reinforcement element integrated into the fresh soil mix material during installation, from Huybrechts (2012)

Figure 64: Examples of lost instrumentation systems integrated in a reservation tube after the intallation of (a) an anchor and (b) a reinforced soil cement column, from Huybrechts (2012)

Figure 65: Installation of a retrievable instrumentation system in a reservation tube installed in a vertical anchor, from Huybrechts (2012)

6. CONCLUSIONS

Ground improvement (GI) is one of the major topics in geotechnical engineering. It has become a fast growing discipline in civil engineering as an alternative allowing construction on soft/weak/compressible soils. Various specialized ground improvement conferences have been frequently held in the past and recent years such as the International Symposium on Ground Improvement (IS-GI Brussels 2012). Within the framework of the Short Courses organized for this occasion, 12 presentations have been given on the topic of the deep mixing method (DMM) which is nowadays a worldwide accepted GI technology. The different stakeholders have highlighted the huge potential of this method and have illustrated its rapid development over the years, particularly with regard to its range of applicability, cost effectiveness and environmental advantages. This is a versatile technology characterized by a continuous innovation of its equipment. There is obviously a growing private and public market for its application. At present considerable experience has been acquired by the deep mixing contractors and by the design engineers especially resulting in the publication of the European Standard EN 14679 (2005) elaborated under the umbrella of CEN TC 288 "Execution of special geotechnical works". Nevertheless, if progress has been made in standardization, from practice it seems that it remains a real need to develop guidelines for pragmatic aspects especially regarding QA/QC procedures. Moreover, the design process of some particular soil mix structures, such as the retaining walls, remains vague in most countries. Flexible QA programs complying with the variable character of the soil mix material should be elaborated in parallel with specific design requirements adapted to the function of the soil mix elements. During the Short Courses, the manufacturers of soil mix equipment have largely illustrated the possibilities of execution monitoring of their machines. But beyond the question of the characterization of the soil mix material, the analysis of the global behavior of the soil mix structure remains essential. Within the Short Courses, Huybrechts (2012) illustrates this topic presenting the use of optical fibers as a very interesting perspective for structural monitoring. If Filz (2012) has proposed a design approach for the reinforcement of embankment with soil mix, design aspects related to various soil mix applications were not really discussed within the framework of the short courses. Interested readers can refer to Denies and Van Lysebetten (2012) in order to obtain an overview of the recent advances in this topic.

REFERENCES

References underlined in the text refer to the Short Courses (see Table 1).
References in italic in the text refer to papers of the present proceedings (see Table 2).

Other references:

ACI 318-08. 2008. American Concrete Institute. Building Code Requirements for Structural Concrete (ACI 318-08) and Commentary. An ACI Standard. Fourth printing January 2011.

ALLU. 2010. Mass Stabilisation Manual, ALLU Finland Oy.

BBRI soil mix project (2009-2013). IWT 080736 soil mix project: SOIL MIX in constructieve en permanente toepassingen – Karakterisatie van het material en ontwikkeling van nieuwe mechanische wetmatigheden. For information : www.bbri.be.

Bruce, D. A., Bruce, M. E. C. and DiMillio, A. F. 1998. Deep mixing method: a global perspective. ASCE, Geotechnical special publication, n°81, pp. 1-26.

Bruce, D. A., Bruce, M. E. C. and DiMillio, A. F. 1999. Dry Mix Method: A Brief Overview of International Practice. Dry Mixing Methods for Deep Soil Stabilization, Bredenberg, H., Holm, G. and Broms, B.B. (Eds.), Balkema, The Netherlands, pp. 15-25.

CDIT. Coastal Development Institute of Technology. 2002. The Deep Mixing Method – Principle, Design and Construction. Edited by CDIT, Japan. A. A. Balkema Publishers/Lisse/Abingdon/Exton (PA)/Tokyo.

Chu, J., Varaksin, S., Klotz, U. and Mengé, P. 2009. Construction Processes, Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, 5-9 October 2009, Alexandria, Egypt, M. Hamza et al. (Eds.), IOS Press, Amsterdam, Vol. 4, pp. 3006-3135.

Denies, N. and Van Lysebetten, G. 2012. General Report – Session 4 – SOIL MIXING 2 – DEEP MIXING. International Symposium of ISSMGE - TC211. Recent research, advances & execution aspects of ground improvement works. N. Denies and N. Huybrechts (eds.). 31 May-1 June 2012, Brussels, Belgium, Vol. I, pp. 87-124.

Denies, N., Van Lysebetten, G., Huybrechts, N., De Cock, F., Lameire, B., Maertens, J. and Vervoort, A. 2013. Design of Deep Soil Mix Structures: considerations on the UCS characteristic value. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris, France, 2013.

Denies, N., Van Lysebetten, G., Huybrechts, N., De Cock, F., Lameire, B., Maertens, J. and Vervoort, A. 2014. Real-scale tests on SOIL MIX ELEMENTS. Proceedings of the International Conference on Piling and Deep Foundations, Stokholm, Sweden, 21-23 May 2014 (to be published).

EN 14679 – 2005. Execution of special geotechnical works - Deep mixing. European Standard.

EN 1992-1-1 – 2004. Eurocode 2: Design of concrete structures - Part 1-1: General rules and rules for buildings. European Standard.

Eurosoilstab. 2002. Development of design and construction methods to stabilise soft organic soils. Design Guide Soft Soil Stabilisation. EC project BE 96-3177.

Essler, R. and Kitazume, M. 2008. Application of Ground Improvement: Deep Mixing. TC17 website: www.tc211.be.

Eurosoilstab. 2002. Development of design and construction methods to stabilise soft organic soils. Design Guide Soft Soil Stabilisation. EC project BE 96-3177.

Filz, G., Adams, T., Navin, M. and Templeton, A. E. 2012.Design of Deep Mixing for Support of Levees and Floodwalls. Grouting and Deep Mixing 2012. ASCE Geotechnical Special Publication no 228, Vol. 1, pp. 89-133.

Ganne, P., Huybrechts, N., De Cock, F., Lameire, B. and Maertens, J. 2010. Soil mix walls as retaining structures – critical analysis of the material design parameters, International conference on geotechnical challenges in megacities, June 07-10, 2010, Moscow, Russia, pp. 991-998.

Ganne, P., Denies, N., Huybrechts, N., Vervoort, A., Tavallali, A., Maertens, J, Lameire, B. and De Cock, F. 2011. Soil mix: influence of soil inclusions on structural behavior. Proceedings of the XV European conference on soil mechanics and geotechnical engineering, Sept. 12-15, 2011, Athens, Greece, pp. 977-982.

Ganne, P., Denies, N., Huybrechts, N., Vervoort, A., Tavallali, A., Maertens, J., Lameire, B. and De Cock, F. 2012. Deep Soil Mix technology in Belgium: Effect of inclusions on design properties. Grouting and Deep Mixing 2012. ASCE Geotechnical Special Publication no 228, Vol. 1, pp. 357-366.

Iowa State University, 2013. Geotech Tools: Geo-construction Information & Technology Selection Guidance for Geotechnical, Structural, & Pavement Engineers. 2013. http://geotechtools.org/ Copyright © 2010–2013 Iowa State University.

Kitazume M. and Terashi M. 2013. The Deep Mixing Method. CRC Press, Taylor & Francis Group, London, UK.

Larsson, S.M. 2005. State of practice report - Execution, monitoring and quality control, International Conference on Deep Mixing, pp. 732-785.

Maswoswe, J. J. G. 2001. QA/QC for CA/T Deep Soil-Cement. ASCE, Geotechnical special publication, N°113, pp. 610-624.

Porbaha, A. 1998. State of the art in deep mixing technology: part I. Basic concepts and overview. Ground Improvement, Vol. 2, pp. 81-92.

Terashi M. 1997. Theme lecture: Deep mixing method – Brief state of the art. Proceedings of the 14th International Conference of Soil Mechanics and Foundation Engineering, Hambourg, 6-12 September 1997. A. A. Balkema/Rotterdam/Brookfield/1999. Vol. 4, pp. 2475-2478.

Terashi, M. 2002. Long-term strength gain vs. deterioration of soils treated by lime and cement, Proceedings of the Tokyo Workshop 2002 on Deep Mixing, pp. 39-57.

Terashi, M. 2003. The State of Practice in Deep Mixing Methods. Grouting and Ground Treatment (GSP 120), 3rd International Specialty Conference on Grouting and Ground Treatment New Orleans, Louisiana, USA, pp. 25-49.

Topolnicki, M. 2004. In situ soil mixing. In M. P. Moseley & K. Kirsch (Eds.), Ground improvement, 2nd ed., Spon Press.

Topolnicki, M. 2009. Design and execution practice of wet Soil Mixing in Poland. International Symposium on Deep Mixing & Admixture Stabilization, Okinawa, 19-21 May, 2009, pp. 195-202.

Topolnicki, M. and Sołtys, G. 2012. Novel Application of Wet Deep Soil Mixing for Foundation of Modern Wind Turbines. Grouting and Deep Mixing 2012. ASCE Geotechnical Special Publication no 228, Vol. 1, pp. 533-542.

Van Lysebetten G., Vervoort A., Denies, N., Huybrechts, N., Maertens, J., De Cock, F. and Lameire B. 2013. Numerical modeling of fracturing in soil mix material. Proceedings of the International Conference on Installation Effects in Geotechnical Engineering. March 24 – 27, 2013. Rotterdam. The Netherlands.

Verastegui Flores, R., Van Impe, W. 2009. Stress-strain behavior of artificially cemented Kaolin clay. Proceedings of the 17th International Conference on Soil Mechanics and Geotechnical Engineering, 5-9 October 2009, Alexandria, Egypt, M. Hamza et al. (Eds.), IOS Press, Amsterdam, Vol. 1, pp. 283-286.

Verastegui Flores, R., Di Emidio, G., 2011. Hardening of kaolin clay mixed with different binders. Proceedings of the Fifth International Symposium on Deformation Characteristics of Geomaterials, Seoul 2011, 1-3 September 2011, Seoul, Korea, Vol. 2, p. 669-676.

SUMMARY OF THE SHORT COURSES OF THE IS-GI 2012
LATEST ADVANCES IN RIGID INCLUSIONS AND SOIL REINFORCEMENT

SUMMARY OF THE SHORT COURSES OF THE IS-GI 2012 LATEST ADVANCES IN RIGID INCLUSIONS AND SOIL REINFORCEMENT

Jan Verstraelen, TUC Rail, Belgium, jve@tucrail.be
Fahd Cuira, Terrasol, France f.cuira@terrasol.com

This report aims to present an extensive summary of the short course 3 regarding rigid inclusions and soil reinforcement, presented during the IS-GI international symposium in Brussels on 30-05-2012.

1. RIGID INCLUSIONS

The present section deals with the first part of the short course 3: Rigid Inclusions. It is based on the lectures by L. Thorel, V. Eekelen, M.Walker, B. Simon and J. Racinais and combines the different topics that were presented.

1.1. Introduction

Different names are used to describe the reinforcement by rigid inclusions (RI), a technique meeting increased use in many countries: piled-embankment, column-supported embankment, geosynthetic reinforced pile supported (GRPS) embankment, pile-supported earth platform or soil column reinforcement. Rigid inclusions are also referred to as columns, pile-like inclusions or non-contact settlement-reducing piles in a generic sense, deep mixed columns, lime columns, or jet grouting columns in reference to some of the installation techniques commonly used; and Controlled Modulus Columns (CMC) or Vibro Concrete Columns (VCCs) in reference to proprietary names.

The applied technique consists in combining of an array of vertical rigid columns and a granular mattress (load transfer layer or LTP) in order to obtain load transfer from an embankment or a slab to a deep bearing stratum (Figure 1). The columns may have enlarged heads or caps. The presence of a transition layer, ensuring a load transfer function on top of the column heads, is a primary attribute of rigid inclusions (RI) ground improvement techniques. The fact that there is no connection between piles and the superstructure clearly distinguishes rigid inclusions ground improvements from piled raft foundations.

The load transfer platform (LTP) should ideally consist of high grade granular material. Under an embankment, this layer may simply comprise the bottom fill layer if it is of good enough quality. The LTP may be reinforced by one or more high-strength geotextile or geogrid reinforcements or even by a wire mesh. Hydraulically stabilized soils are sometimes used to build the transition layer when mineral resources are scarce or costly.

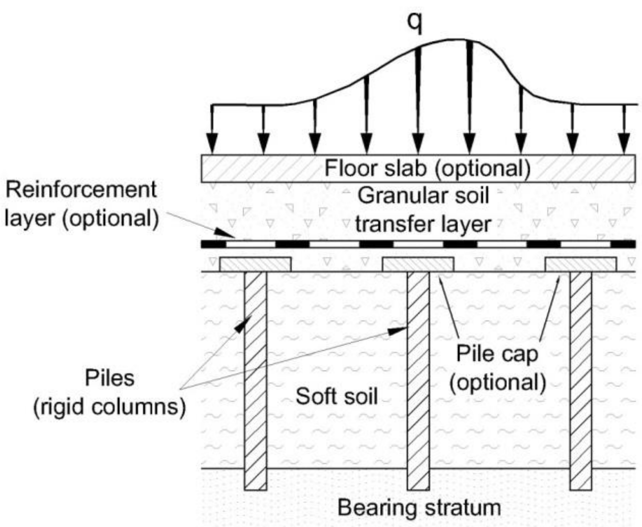
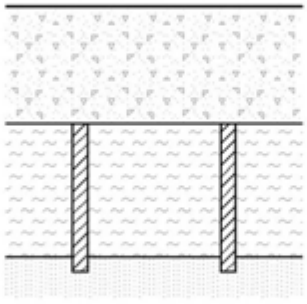


Figure 1: Constituents of the rigid inclusion ground improvement concept

1.2. Operating mechanisms

1.2.1. General concepts

Shear mechanisms that develop within the transfer layer and around the inclusion shafts are essential for this technique and are activated by differential settlements that arise between the "rigid" inclusion with low compressibility and the surrounding soil (Figure 2). Differential settlements, and therefore the shear stress, extend to the layers above the inclusion heads, between the column centrelines and the mesh centreline, meaning that some load transfer already takes place at a distance above the inclusion heads. If one considers the average soil settlement over one mesh and the settlement at the column centreline at any given elevation, it appears that these values are equal only in some horizontal planes: the upper plane located in the LTP or the fill (and all planes above), the plane located along the column length and the plane located at a certain depth below the column tip (and all planes below). The existence of these "equal settlement planes" is characteristic of RI ground improvements.



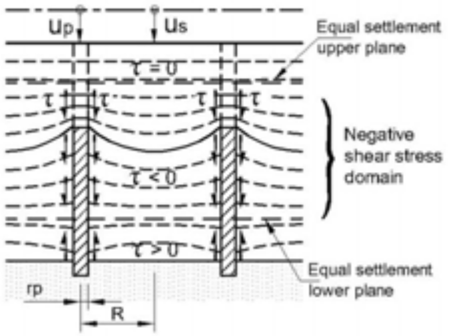


Figure 2: Operating mechanisms in the rigid inclusions ground improvement system

1.2.2. Above inclusion heads

Whether it is under an embankment or a concrete slab, shear stress develops in the load transfer platform as an outcome of differential settlements, generated by loading, between the column of low compressibility ("rigid") and the surrounding soil. Numerical models lead generally to what is summarized in Figure 3. In a piled embankment (Figure 3, left), soil settlements at the column head elevation reveal a significant gradient close to the pile shaft that levels out further away. Significant shear stress can develop along the vertical cylindrical surfaces, causing directions of principal stresses to rotate, as seen on the right. When a 90° rotation is reached, the major principal stress becomes horizontal: the stress field reveals an analogy with that of an arch extending in the granular fill and bearing on the pile heads. Arching is the basis of the load transfer models established by Hewlett and Randolph (1988) and Zaeske and Kempfert (2002). According to these models, it is understood that surface settlements become uniform ("equal settlement plane") when the height of the embankment over the column heads exceeds a given ratio of the column's clear span along the diagonal mesh, called the critical embankment height (Mc Guire et al, 2012).

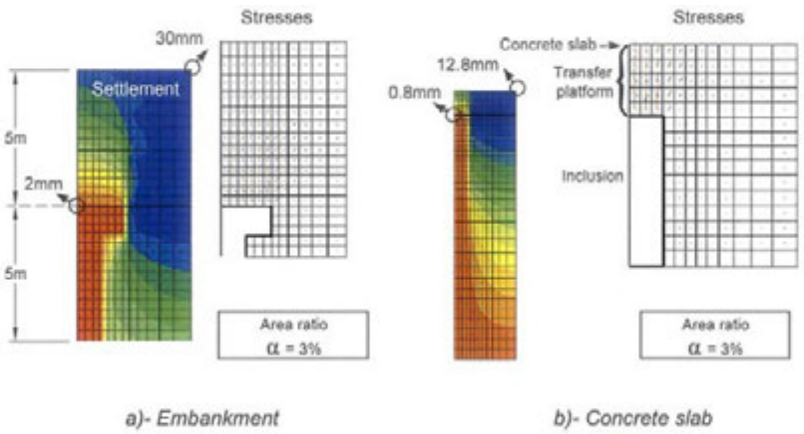


Figure 3: Comparison of load transfer mechanisms underneath a) an embankment and b) a concrete slab over rigid-inclusion-improved ground (Flac 3D models) (ASIRI, 2012)

Under a concrete slab (Figure 3, right), the plane of equal settlement is "forced" to coincide with the slab's lower face. In that case, the flexural strength of the slab has to resist non-uniform fill reactions. Due to the limited thickness of the LTP, the stress rotation is limited, and the stress field within the granular fill shows no arching pattern, contrary to what is observed under a piled embankment. The arch design approach is thus ineffective for slab-on-grade applications.

Other numerical modelling using discrete elements (Chevalier et al, 2011) has shown that when the load transfer platform is not overlaid by a concrete slab, the zone within the LTP that reveals slight particle displacements takes the shape of an inverted pyramid lying on every inclusion head (Figure 4). This shape is similar to the load transfer model proposed by Carlsson (Nordic Handbook, 2005), however the angle β defining the pyramidal shape is close to the peak friction angle by Chevallier et al (2011), while Carlsson's model assumes β = 15°. When the load transfer platform is covered by a slab, the zone with very slight displacement is primarily restricted to the cylindrical LTP volume between the column head and the concrete slab (Figure 4). Most of the load is transferred onto the inclusion head through the bending of the concrete slab and compression of this volume. The total deformation in this volume controls slab settlement. As seen also in Figure 4, a concrete slab on a LTP has an enhanced efficacy compared to the case of a LTP of equal thickness without slab. This difference between efficacy values obtained with or without a slab tends to disappear when the LTP thickness increases.

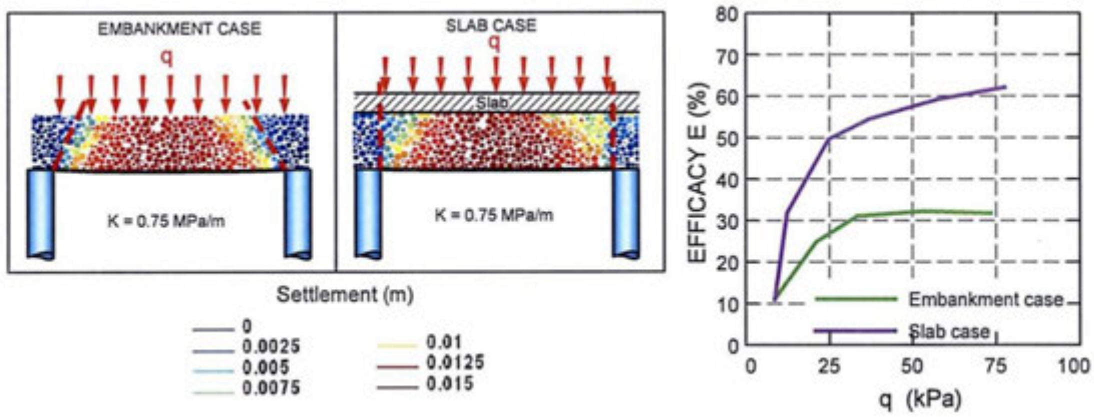


Figure 4: Comparison of the domains in LTP showing slight particle displacement and the efficacy obtained, with or without a slab (Chevalier et al, 2011)

1.2.3. Along inclusion shaft

Inclusions "attract" load as a result of the negative skin friction that is developed in zones where soft soil settles more than the inclusion. Unlike deep foundations, this negative skin friction is beneficial because it contributes to the load transfer towards the column. Inclusions being much stiffer than the surrounding soil, the relative settlement become negative at some depth along the inclusion. Below that depth, the inclusion settles more than the soil, and the skin friction becomes positive. At equilibrium, the axial load in the inclusions increases in proportion to the negative skin friction developed between the head and the neutral point (the intersection with the "equal settlement" intermediate plane). A maximum value Q_{max} is reached at the neutral point (Figure 5). The elevation of the equal settlement intermediate plane may vary with the intensity of the loading. Safety against bearing capacity failure depends on the comparison of the maximum axial load in the inclusion (Q_{max}) with the design bearing resistance of the lower part of the inclusion, between the neutral point and inclusion's tip.

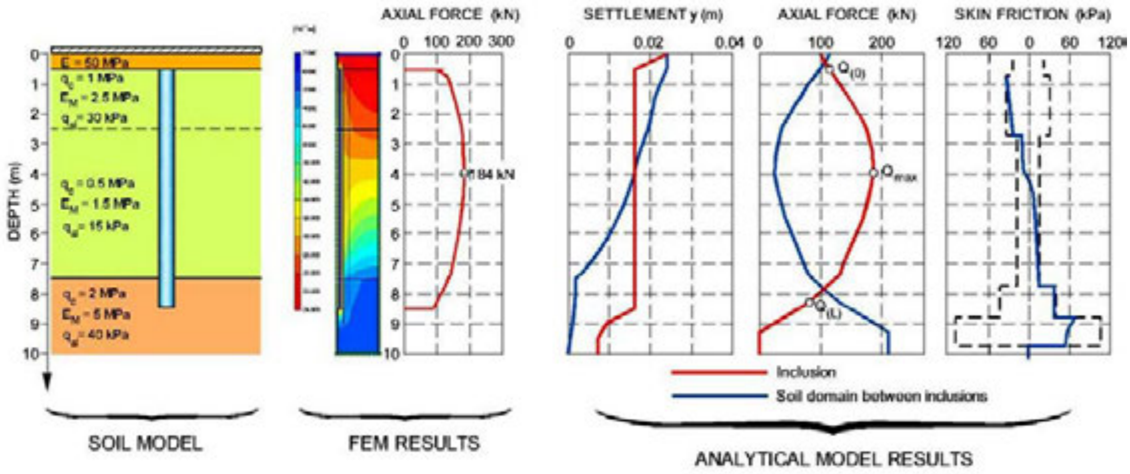


Figure 5: Simulation of a full-scale load test of a slab over an RI ground improvement (ASIRI, 2012)

1.3. US practice in LTP Design

The content of this paragraph was presented by M. Walker from US and focuses on common methods used for LTP design in US. Figure 6 presents the typical layout of studied situations.

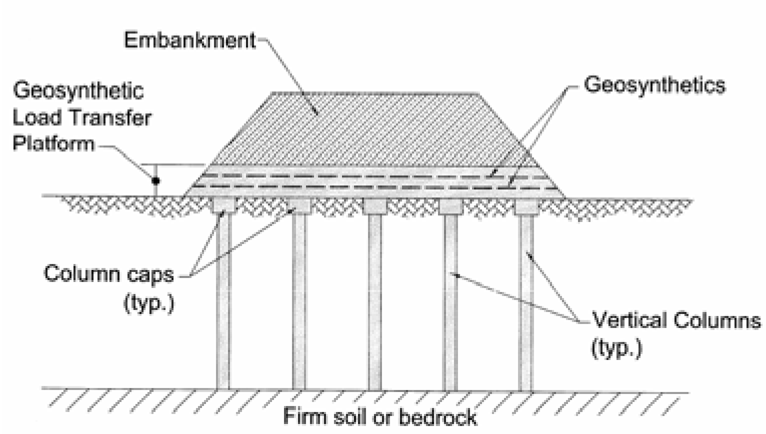


Figure 6: Typical layout of studied situations

1.3.1. Failure mechanisms

For a piled embankment project, the following failure mechanisms have to be studied (Figure 7):

- **Limit state failure criteria** (Figure 7, left)
  (a) Vertical load capacity (ULS);
  (b) Lateral extent of the columns (ULS);
  (c) Vertical load transfer (ULS);
  (d) Lateral sliding of the embankment (ULS);
  (e) Global stability of the system (ULS).

- **Serviceability state design** (Figure 7, right)
  (a) Strain within the geosynthetic reinforcement (SLS);
  (b) Settlement of the columns (SLS).

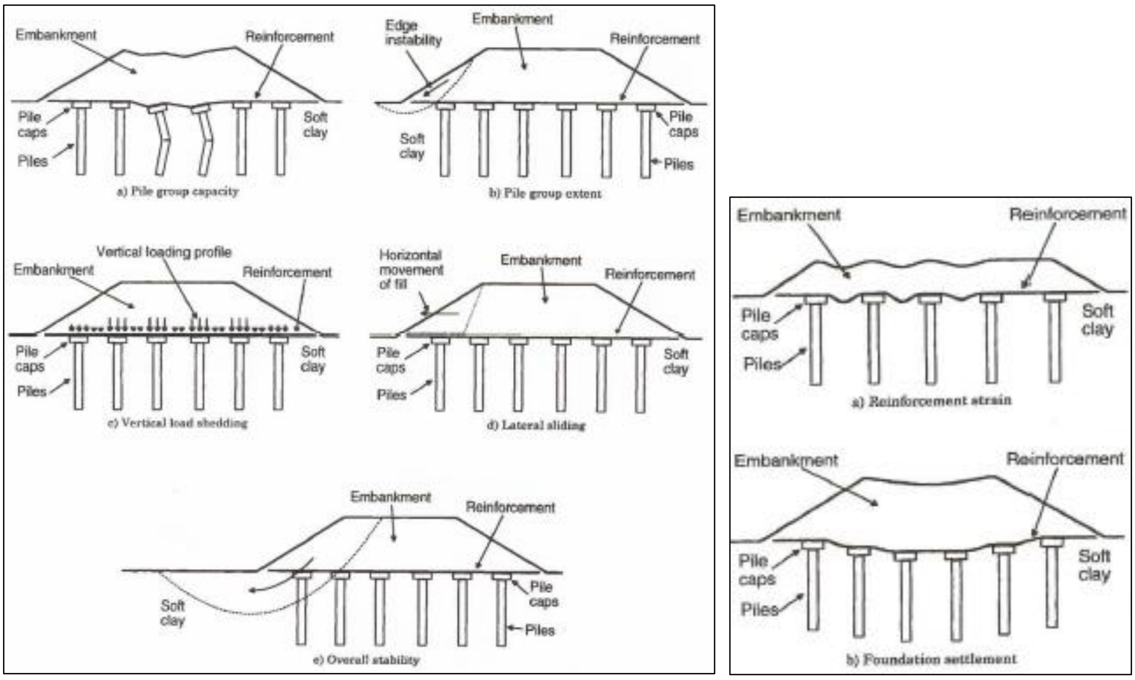


Figure 7: Failure mechanisms to be studied for a piled embankment project

1.3.2. Design methods

Three calculation models are generally used for estimating the vertical stress on the geosynthetic reinforcement:

- **Catenary model** (Figure 8, left): load transfer through catenary tension in the reinforcement. Benefits of composite reinforced soil are neglected. The following primary assumptions are considered :
  - Soil arch forms in embankment ;
  - Reinforcement is formed during loading ;
  - One layer of reinforcement is used.

  This method typically requires higher strength reinforcement than beam method.

- **Beam model** (Figure 8, right): stiff reinforced soil mass using multiple layers of reinforcement (also called the Refined Guido Method). In this model, beam action transfers the embankment load to the columns. The model allows typically larger column-to-column spacing than with Catenary Method. The following primary assumptions are considered:
  - A minimum of three layers of reinforcement used ;
  - Platform thickness is superior than ≥ ½ the clear span between columns ;
  - Soil arch is fully developed within the depth (thickness) of the platform.

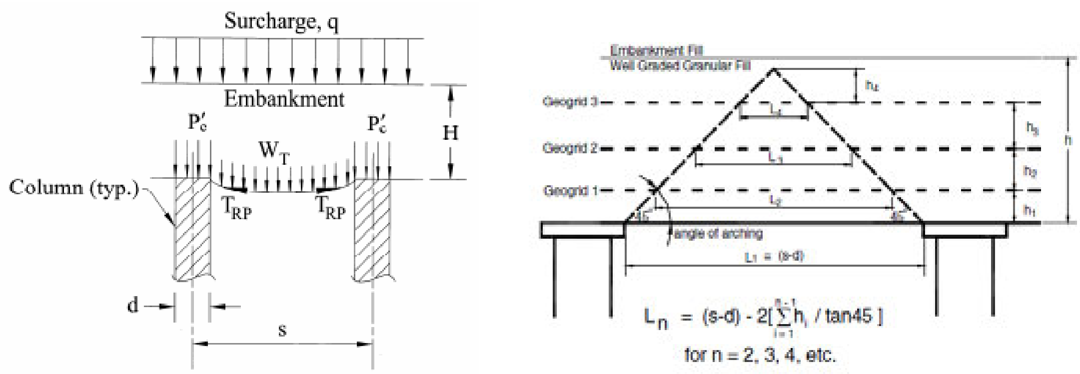


Figure 8: Principle of Catenary (left) and Beam (right) methods

- **Filz and Smith model**: vertical load equilibrium and displacement compatibility assumed at the level of the geogrid reinforcement. This model best represents behaviour with small strain in geogrid. The calculation is based on an axisymmetric approximation of a unit cell and parabolic deformation pattern for the geogrid tension. It assumes a linear stress-strain response of the geogrid and thus requires iterative calculation to approximate the non-linearity of the real response. The model is completed by incorporating nonlinear response of the embankment and nonlinear compressibility of soil between columns. Moreover, the slippage is allowed between the soil and the column when the interface shear strength is exceeded

Filz and Smith developed a spreadsheet which may be used iteratively to calculate the vertical load in the element and the geogrid tension.

The tension in the geogrid reinforcement can be evaluated by several analytical methods: parabolic method, tensioned membrane method, Kempfert method... Strain is typically assumed to be around 5% in the geogrid. These techniques prove however insufficient for applications of ground improvement which targets low settlement (2 to 5 cm) of load transfer platform. This results in reduced effectiveness of geogrid and needs another evaluation technique (numerical modelling) to determine load transfer behaviour.

Numerical modelling can be carried out using geotechnical software based on the finite element method with various degrees of complexity:
- Axisymmetric model (Figure 9, left) : valid only for interior columns under uniform vertical loading ;
- 2D Plane strain model (Figure 9, right): Column modelled as a “wall” and the dimensions and properties (axial stiffness, skin friction, end bearing...) must be adjusted to approximate actual characteristics. This model generally over-predicts load transferred to columns and is not valid for evaluation of lateral deformation ;
- 3D complete model: enabling evaluation of lateral deformation and edge effects. This model however requires a prohibitive computation time and more effort to set up and validate the calculation.

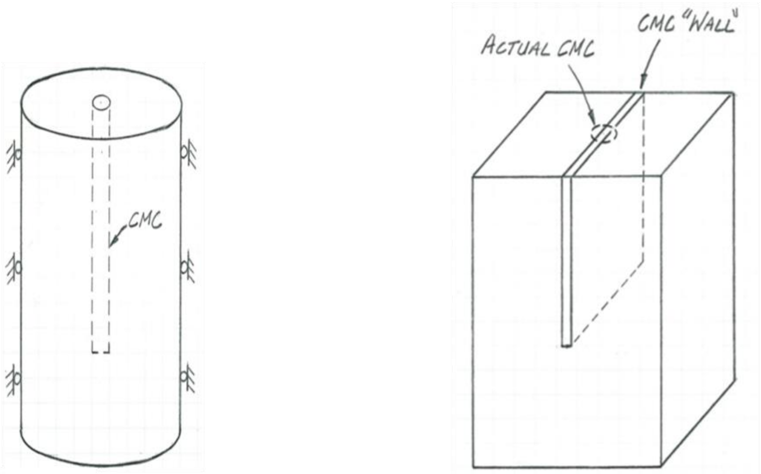


Figure 9: Principle of axisymmetric (left) and 2D Plane strain (right) models

1.4. Lessons from the Dutch research program

This paragraph summarizes the results of the Dutch research program which was carried out for further optimizing the design of piled embankments reinforced with Geosynthetics. The content of this paragraph was presented by S. Van Eekelen from The Netherlands.

1.4.1. Purpose

The presented work, which focuses on vertical loading, evaluates how the fill load can be divided into a part “A”, which is directly transmitted to the pile heads by shear (or arching) within the fill, and a part “B+C”, where part “B” can also be transferred through the geosynthetic reinforcement (GR) to the pile, while the remaining part “C” could be carried by the subsoil (Figure 10). The evaluation of the part “B+C” differs significantly between design national rules: BS8006 assumes an equally distributed load without any allowance of soil support, and CUR and EBGEO assume a triangular distribution of load on the GR and allow for some reaction of the subsoil, which is expressed by a coefficient of subgrade reaction, to be input in the analysis.

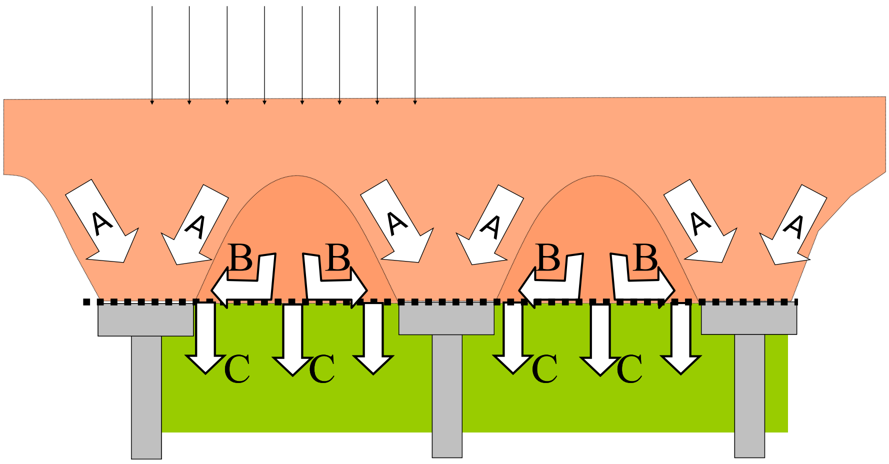


Figure 10: Load distribution in a piled embankment (Van Eekelen and Bezuijen, 2012)

1.4.2. Monitoring data

Valuable monitoring data were analysed for two piled embankment projects: a railway in Houten (*a*) and a highway's exit near Woerden (*b*) (geometry and properties are given in table 1). Results (Figure 11) show that the measured part "A" is higher than that predicted using EBGEO/CUR. Strains in the GR were also found to be considerably smaller than those predicted using the model of Zaeske (2001), confirming the overestimation of part "B" by the current rules.

Table 1: Overview of three Dutch field tests

		Houten Railway		**Woerden Highway's exit**
	in use since	November 2008		June 2009
		location 1	location 2	
Geometry and properties	soil conditions	1 m sand, 3 m soft clay, 20 m sand		17 m soft clay
	pile foundation	High Speed Piles (HSP), pile shafts ø0.22 m, cast in situ pile heads ø0.40 m.		prefab piles 0.29x0.29 m^2, smooth square prefab pile caps 0.75x0.75m^2
	centre-to-centre distance piles	1.25x1.40 m^2	1.45x1.90 m^2	2.25x2.22 m^2
	height embankment[a]	2.60 m	2.60 m	1.53-1.89 m
	reinforcement across (bottom layer)	Fortrac M 450/50 (PVA)	Fortrac R 600/50 T (PET)	Stabilenka 600/50 (PET)
	reinforcement along (top layer)	Fortrac M 450/50 (PVA)		Fortrac R 600/50 T (PET)
	load distribution	A, A+B		traffic weight, A, A+B, locally C

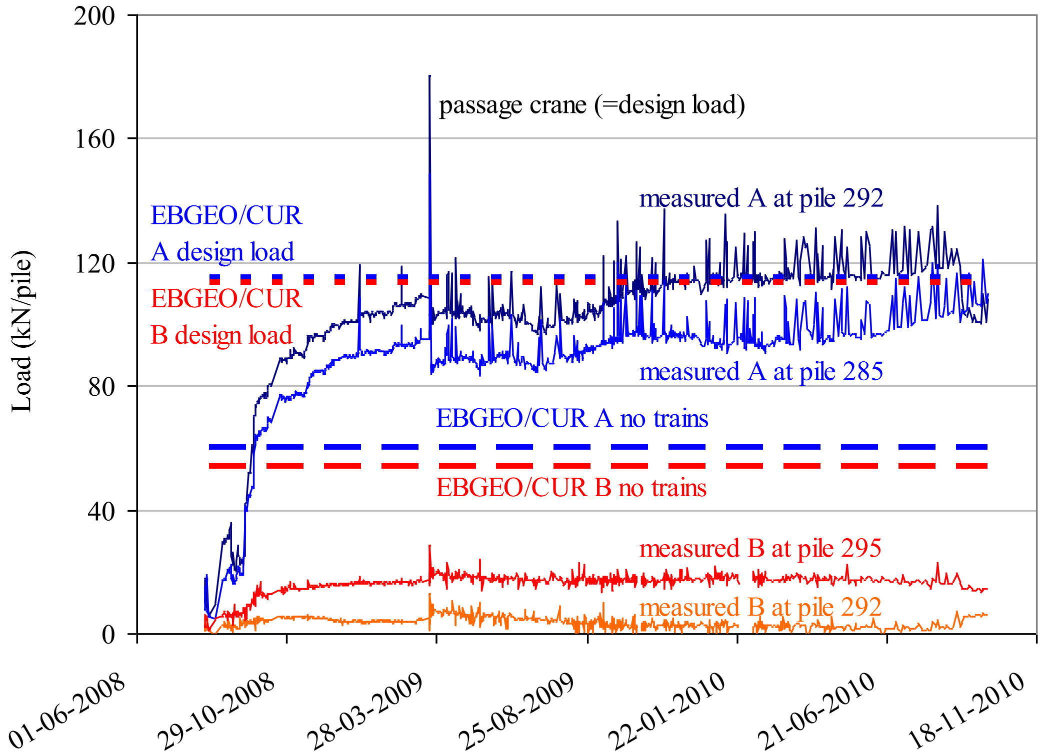


Figure 11: Results of field measurements at the Houten railway (Van Duijnen, 2010)

The results obtained above are also supported by a series of 19 piled embankment model experiments that were carried out in the Deltares laboratory. The main purpose of the experiments was to understand why the predicted GR strains are larger than the GR strains that were measured in the field. Starting point was that it had to be possible to measure load parts A, B, C and the GR strain separately. Furthermore, GR was to be included, and the fill had to be as realistic as possible, that implies that most tests were carried out with a granular fill of crushed recycled construction material.

1.4.3. Lessons learned

Analysis of obtained results shows that consolidation of the subsoil results in an increased part “A” in the fill, indicating that arching was not independent from the subsoil consolidation (unlike the assumption implicitly made by EBGEO/CUR or BS8006). An additional finding from these small-scale tests was that the GR strains occur mainly in the GR strips between two adjacent piles (more than in the GR strips across the diagonals of the mesh). It was also found that the load distribution on the GR agrees better with an inverted triangular shape than with the uniform distribution assumed in BS8006 (giving a parabolic GR profile) or the triangular shape adopter by EBGEO/CUR. This is a first improvement to be added to the EBGEO/CUR calculation model.

A second improvement would be to increase the supporting subsoil area to the entire available area below the GR, which is described in more detail in Lodder et al. (2012). Figure 12 shows the results of modifying EBGEO/CUR by improving both the subsoil support and the load distribution. This leads to a good agreement with the measurements and 19-26% less GR strain than the EBGEO/CUR assumptions.

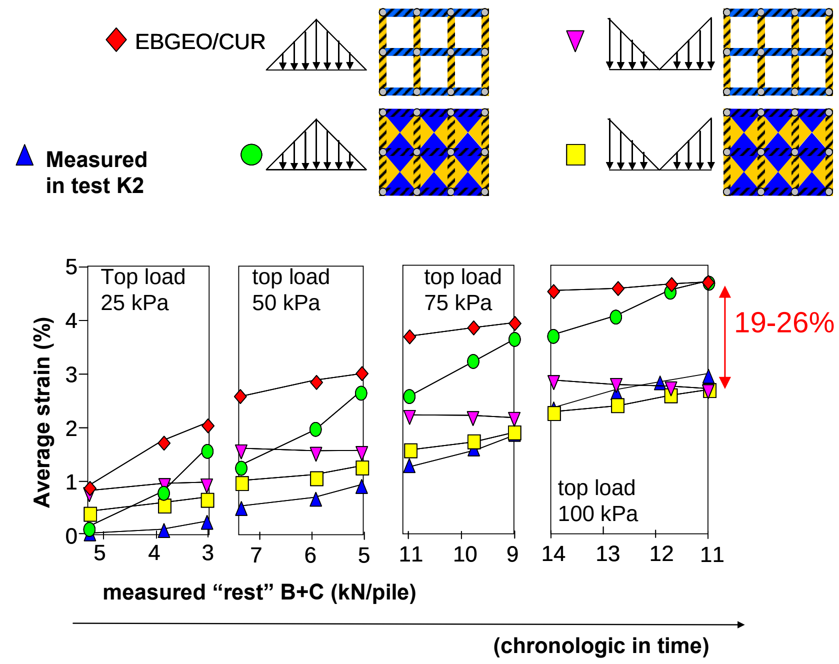


Figure 12: Average strain in geosynthetics – Calculation vs. measurements (Van Eekelen, 2012)

1.5. Lessons from ASIRI research program

The content of this paragraph was presented by B. Simon and L. Thorel from France.

1.5.1. Full-scale field tests

The first full scale experiment was carried out for the case of a piled embankment. Four test areas were laid out, including the unreinforced reference one. Coverage ratio was 2.9%. The thickness of the soft soil layer was about 9 m. The three reinforced units (2 R, 3R and 4R) differ by the use or not of a granular transfer layer at the bottom of the fill and reinforcement of this layer by either one geotextile or two geogrids. Fill reached a maximum height of 5 m.

Figure 13 illustrates the vertical stress at the top of one central inclusion plotted versus time for each test section. Embankments 3R and 4R, where a high quality granular layer was put in place, clearly led to a high amount of load transfer onto inclusions. Stress on inclusion head is five to six times higher than in section 2R. Stress on top of the 4R transfer layer (which incorporates two geogrids) is quite lower and close to the overburden pressure. This shows that when using geogrids, load transfer is mostly operated in the granular layer itself and not in the overlying ordinary fill.

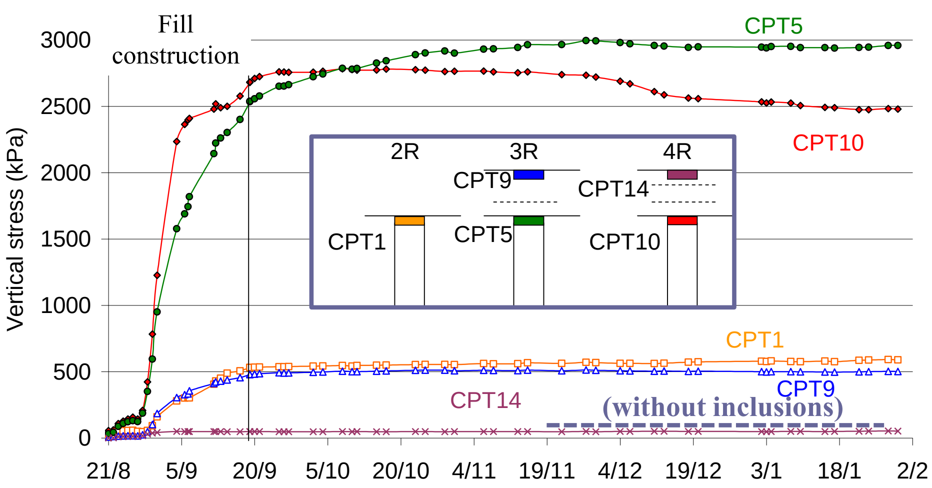


Figure 13: Measurement of load transfer onto inclusions heads – Chelles site (ASIRI, 2012)

Results of load-settlement curves monitored at ground level between the columns (2R, 3R and 4R) compared to the unreinforced reference section (1R) are presented in Figure 14. Results show that a stress reduction ratio as low as approximately 10% (2R) led to a settlement reduction factor of 40% (settlement efficacy of 60%) and that a stress reduction of approximately 50% (3R or 4R) led to a settlement reduction factor of approximately 80% (settlement efficacy of 80%). This means that there is no linear relationship between load efficacy (or stress reduction ration, SRR) and settlement efficacy (SRF). Thus, a small gain in SRR can lead to much higher gain in SRF.

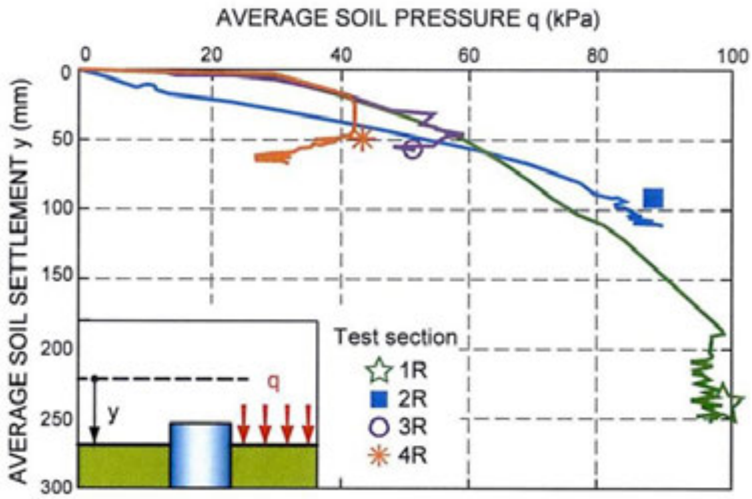


Figure 14: Non-proportionality between settlement and vertical stress (ASIRI, 2012)

Another full scale experiment concerns the case of ground slabs over reinforced soil. Four test areas were laid out, including the unreinforced reference one. Coverage ratio was 2.2%. The three reinforced units (2D, 3D and 4D) differ by the use or not of a concrete slab covering the LTP and the used technique for building inclusions (with or without soil displacement). A 4m fill load was applied on all four areas, in two successive stages. The Figure 15 presents the cell pressure measurements made in test section 3D. One can notice that during the first load stage, development of shear stress within the transfer layer makes the vertical stress increase from the slab bottom to the pile head in the axis centre-line. No similar load increase between LTP top and bottom is observed during next loading stage which proves that load transfer is mainly operated by the concrete slab itself. Vertical stress on soft soil between inclusion heads amounts to less than 10% of that on inclusions. Soil reaction at the slab under-face is thus non uniform: this should be taken into account for the concrete slab design.

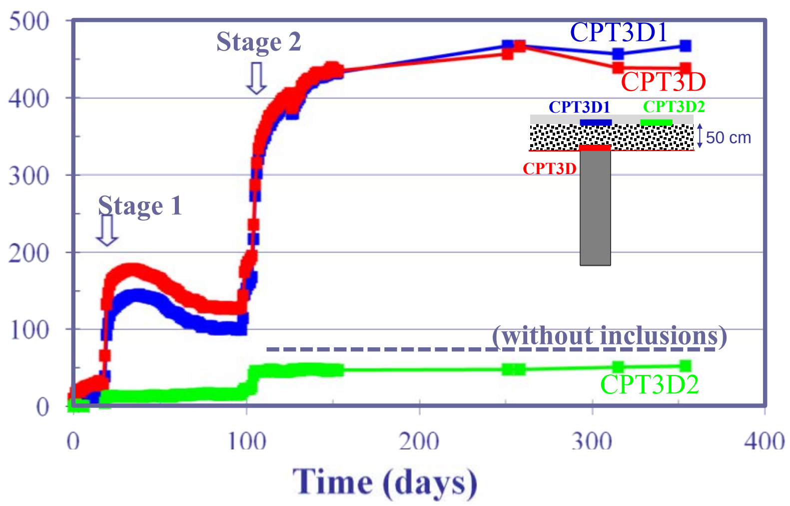


Figure 15: Measurements of load transfer onto inclusions heads – Saint Ouen site (ASIRI, 2012)

Settlement at the base of the granular layer was monitored with a special inclinometer probe travelling in horizontal tubes and linking two fixed references on either side of the test area. Obtained measurements show a quite interesting result: the flat shape of the settlement profile between inclusions revealing a quite distinctive U-shape (Figure 16). Similar U-shape has also been observed on the other full scale experiments as well as in the physical or numerical models.

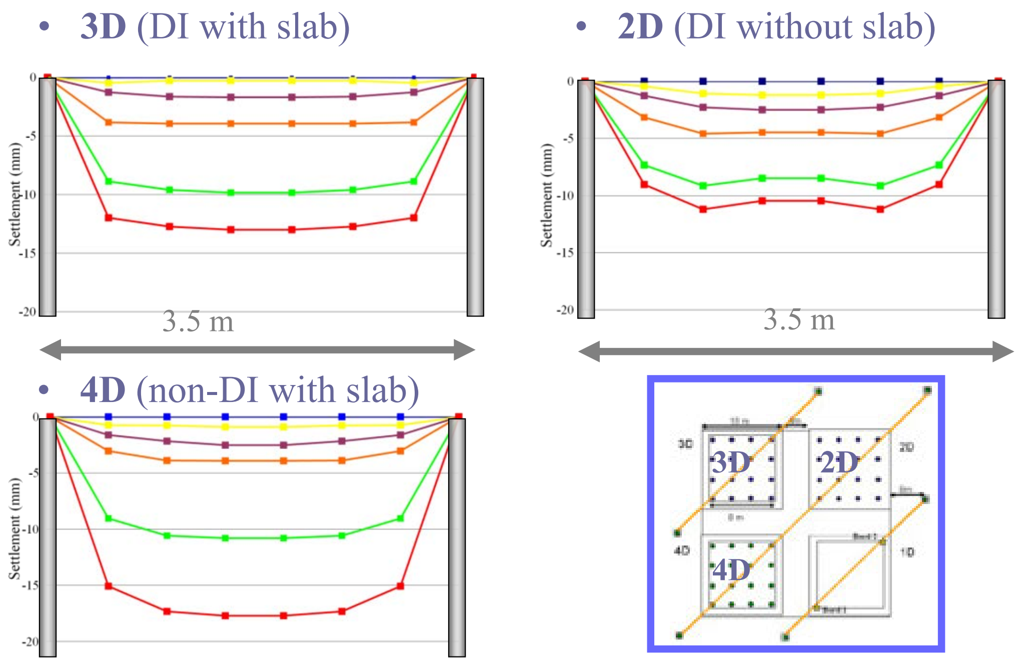


Figure 16: Settlements between inclusions – Saint Ouen site (ASIRI, 2012)

Moreover, on both test sites, axial test loading was carried out on isolated inclusions located outside the grid layout in order to determine the load-displacement curves for each type of inclusion (Figure 17). Due to the use of removable extensometers, these tests also provided the load-displacement curves at the pile tip. It is quite interesting to note that in the slab case, the relationship between stress and settlement at the top of one inclusion from the central part of the grid duplicates the load-displacement curve at the tip of an isolated and axially loaded inclusion. The same observation holds for the embankment case. This result demonstrates that the behaviour of an inclusion underneath the structure (slab or embankment) is the same as the one of the tip of a single rigid inclusion subjected to axial loading. These results have an important implication for design. If reliable results at the top of the inclusion are really expected, the capacity of the numerical model to give a realistic simulation of the behaviour of soil under loading exerted at the tip of inclusions should be checked.

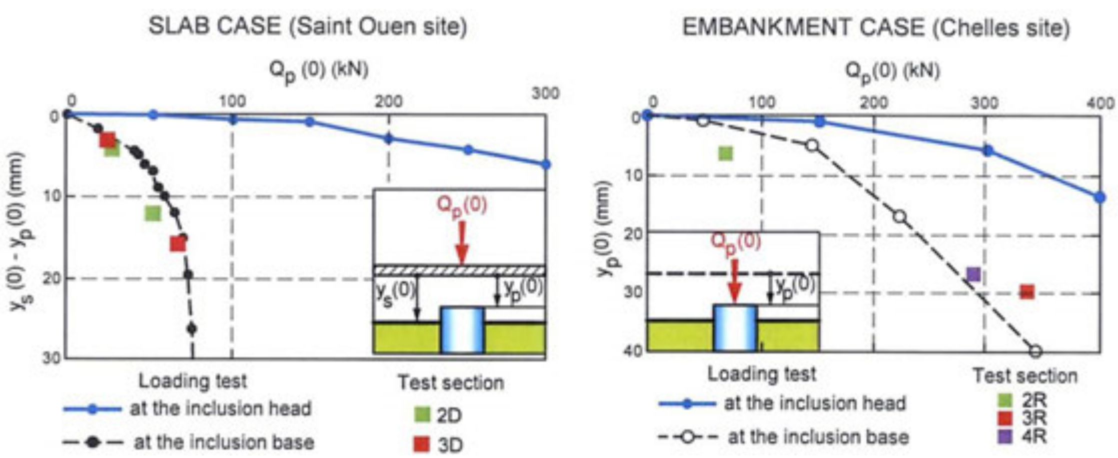


Figure 17: Load-displacement curves at head and base of an axially loaded isolated inclusion, compared to the monitoring data at the head of one inclusion from the central part of the grid (ASIRI, 2012)

1.5.2. Small-scale physical models (centrifuge testing)

An important centrifuge testing program was carried out at the IFSTTAR facility in Nantes (France) with a big part dedicated to the study of the arching effect in the LTP.

Centrifuge physical modelling of the arching effect in the LTP was simulated by displacing a mobile tray with respect to model inclusions. This modelling has provided interesting information about the efficacy and its variation relative to soil-column displacement (Okyay et al., 2012).

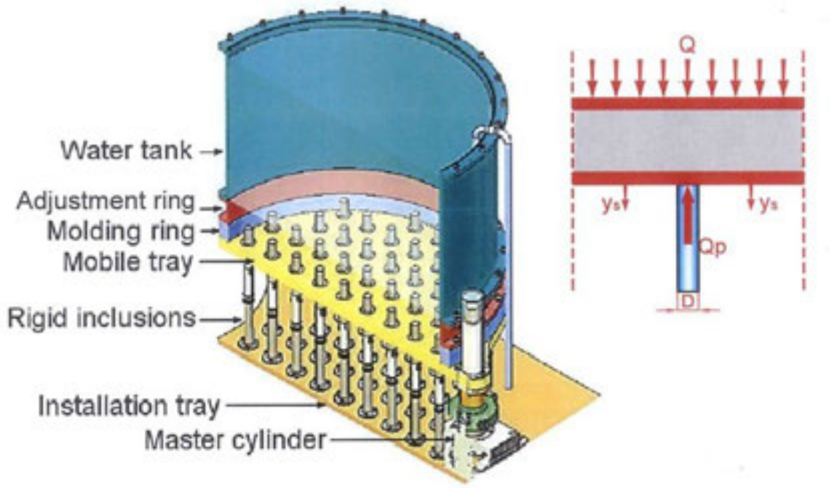


Figure 18: Simulation of the arching effect by displacing a mobile tray – centrifuge testing (ASIRI, 2012)

The tests demonstrated that the efficacy reaches an ultimate value after some amount of displacement, and for any given LTP thickness, this value depends on the shear strength of the LTP material and on whether the boundary conditions at the top of the LTP imitate those of a uniformly distributed load ("embankment case") or of a rigid plate load ("slab case"). In the slab case, the measured ultimate values of efficacy agree with the vertical stress values given by the Prandtl failure mechanism shown in Figure 19: $q_p^+ = Nq\ q_s^+$, where q_p^+ and q_s^+ are the average vertical stresses on the inclusion head and the soil, respectively, and N_q is calculated for the critical state friction angle.

The ultimate efficacy value is therefore $E = \alpha N_q/[1 + \alpha(N_q-1)]$, where α is the coverage ratio. In the absence of slab overlying the LTP, both the Prandtl failure mode and the inverted pyramid mode may be critical (Figure 20). The ultimate efficacy value is given by the minimum value yielded by either of the models (ASIRI, 2012).

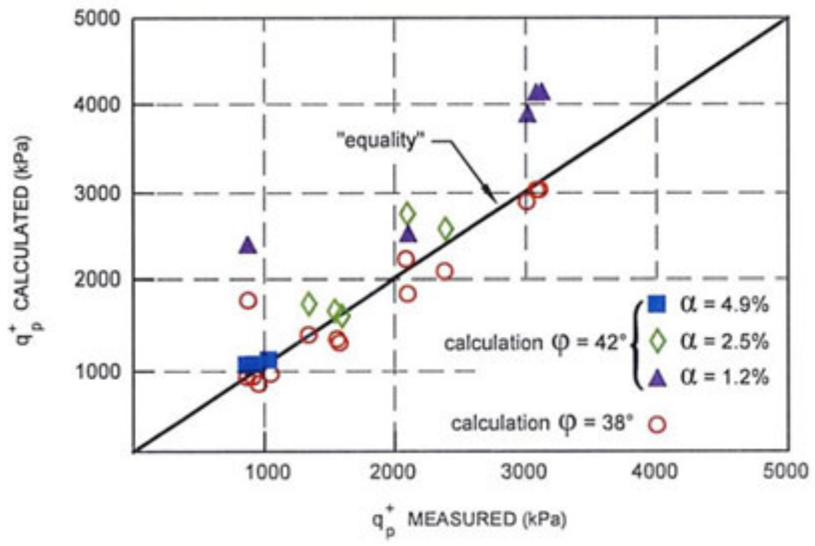


Figure 19: Comparison of calculated and measured values of ultimate values of vertical stress on the heads of inclusions (ASIRI, 2012)

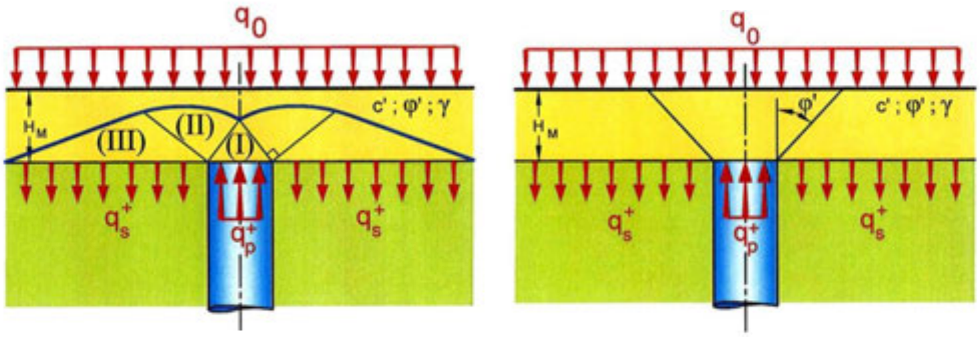


Figure 20: Two different failure modes in Load Transfer Platform. (ASIRI, 2012)

1.6. Bending moments in a slab on grade supported with RI reinforced soil

The content of this paragraph was presented by J. Racinais from France, and focuses on a simplified method for evaluating bending moments for slabs on grade supported with RI reinforced soil.

The design of industrial and logistic building's slab-on-grades is a complex exercise. The design needs to consider the different loading types and configurations (uniform or alternated loading, racks, live loadings…) together with the relative positions from the hinged constructions joints to the loads, whose position and intensity can vary during the life of the structure. The non-uniform stress reaction distribution in the soil reinforced with rigid inclusions creates an additional stress in the slab with a different pattern than the ones of the loads and of the joints. The optimization of the design of the slab becomes a complex problem with three different intertwined patterns (loading, joints, and rigid inclusions) that can move relative to one another with usually no typical symmetry conditions. Existing codes of practice dedicated to slab-on-grades are only able to consider uniform soil conditions and the typical size of those structures forbids the modelling of the full extent of the slab. The proposed method is a powerful solution which is easy to use while allowing for the precise optimization of the design of slabs on grade. The approach has been validated and calibrated with an extensive number of finite element calculations and has been integrated in ASIRI (2012).

The main principle of the proposed method, called "the additional bending moment method", consists in adding to parameter [ma] which is generally calculated by the structural engineer, two correcting terms [mb] and [mc] : M = [ma] + [mb] + [mc].

The parameter [ma] represents the impact of the loadings configuration on a slab with joints, without any impact from the non-uniform reaction of the inclusions. This parameter is calculated by the structural engineer for all the configurations and relative positions between the loads and the hinged construction joints according to applicable codes of practice and regulation. In this calculation, the reinforced soil is represented by an equivalent soil profile characterised by an equivalent Young modulus E* (see Figure 21). This equivalent soil profile can be deducted from an elementary axisymmetric calculation, centred on one single inclusion, where an equivalent average uniform load is applied.

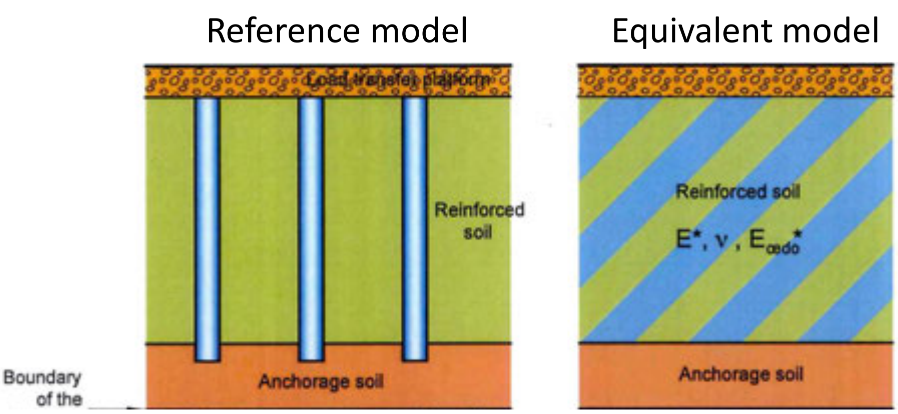


Figure 21: Definition of an equivalent soil profile for the calculation of parameter [ma]

Parameter [mb] represents the impact of the rigid inclusions on a slab without joints and doesn't depend on the loading distribution. This parameter only depends on the equivalent average loading and doesn't depend on its type and configuration (uniform, alternate, punctual). As a consequence, [mb] can be estimated from an elementary axisymmetric calculation, centred on one single inclusion (Figure 22), where an equivalent average uniform load is applied. In term of bending moment in the slab, this calculation results in a positive bending moment +Msup at the vertical of the inclusion and a negative bending moment −Minf in the middle of the grid. The parameter [mb] is taken in the interval [Msup+, −Minf].

Parameter [mc] represents the interaction of the rigid inclusions with the joints, without any impact from the distribution of loading or from the non-uniform reaction of the inclusions. The value of this parameter is the same for any loading configuration with same surface average value and depends only on the geometry of the joints and of the inclusions. By construction, hinged joints cannot transmit bending moments but can only transmit shear forces. The effect of the joints is thus to bring the bending moment in the slab to zero at position of the joint and to "shift" the bending moment curve around the joint by the corresponding value. Thus, the maximum impact of parameter [mc] representing the interaction between joints and inclusions is [mc] = - [mb] = [+Minf ; -Msup].

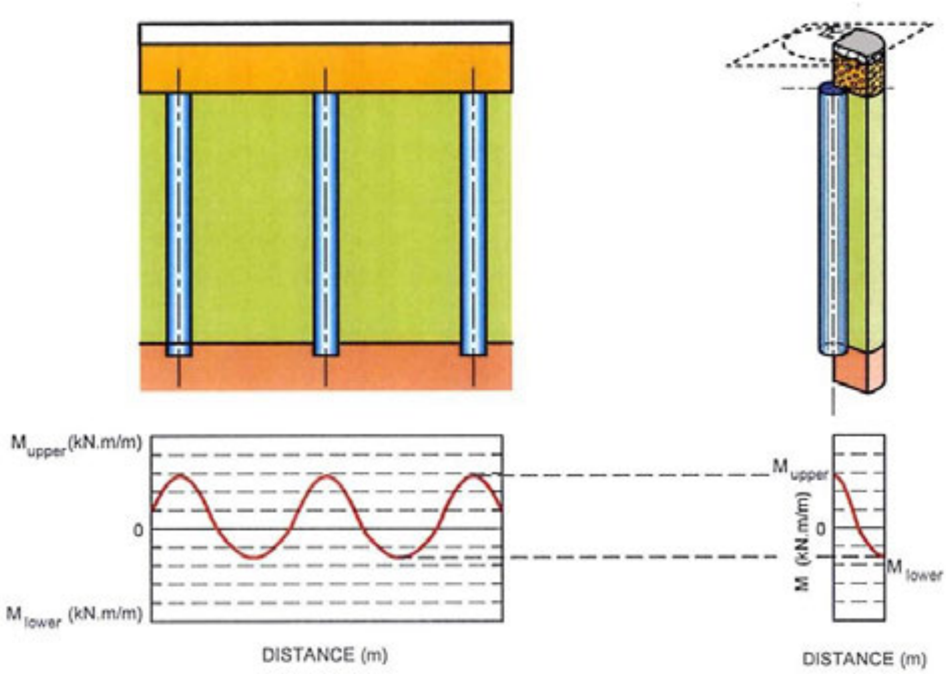


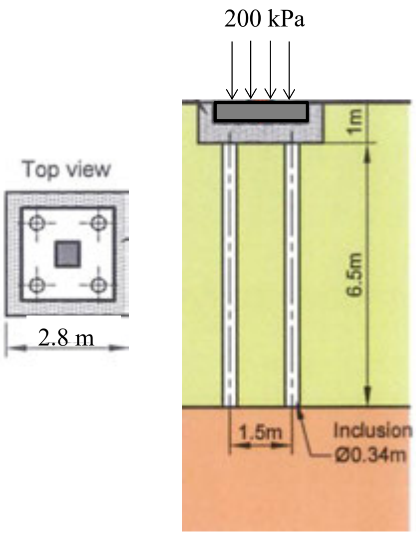
Figure 22: Description of the calculation model used for evaluating parameter [mb]

1.7. Spread footing on RI Reinforced soil

Slabs submitted to uniform lading can be designed with an axisymmetric model of the elementary reinforcement cell as shown in the Figure 22. Such simple models are reliable and easy to use, but they cannot be applied to the case of spread footings. The limited number of inclusions no longer makes it possible to satisfy the symmetry assumptions. The interaction of the reinforced soil block below the footing with the surrounding non reinforced soil domain must be taken into account. 3D models can be used but they require strong computational effort. They are out of the scope of basic project design.

This paragraph focuses on a simplified analytical method for designing a spread footing on RI reinforced soil (B. Simon, France).

The studied typical case here is shown in the Figure below.



- **Load transfer layer**
  - $\phi' = 35°$
  - E = 50 MPa
- **Soft soil**
  - $E_M = 2.5$ MPa
  - $c_c/(1+e_0) = 0.065$
  - $\sigma'_c - \sigma'_0 = 10$ kPa
  - $q_s = 30$ kPa
- **Bearing layer**
  - E = 120 MPa
  - $E_M = 8.0$ MPa
  - $q_p = 7.2$ MPa

Figure 23: Spread footing on RI reinforced soil – studied case (Simon, 2012)

1.7.1. Under axial loading

The proposed method entails 3 successive steps which are carried out using ordinary tools for deep foundation design, i.e. study of an isolated pile or of a pile located at the centre of a reinforcement element mesh, under vertical loading, through use of transfer functions characterizing shaft friction and point pressure mobilization around the pile (Cuira and Simon, 2010). In the reinforcement mesh case, the analytical model is of a biphasic type (i. e. associating a pile domain and a soil domain) where interaction forces between both domains are expressed by the same transfer functions as an isolated pile, just replacing the absolute pile-displacement by the relative soil-pile displacement.

Step 1 : A study of the behaviour, under distributed vertical load, of a basic cell without any interaction with the external domain serves to establish the horizontal plane position underneath the inclusion tip where soil settlement is uniform (lower neutral plane). The average settlement derived between the upper cell face (below the footing) and this lower plane allows evaluating the apparent modulus of deformation E* of the cell under vertical loading (Figure 24).

Step 2 : A study of the vertical monolith with modulus E* assimilated to an isolated pile interacting with the exterior (non-reinforced) soil domain, exposed to vertical force Q, determines the profile $y_s(z)$ of the average monolith settlement, in accounting for shaft friction mobilization on the monolith perimeter. The settlement recorded at the head y_s remains less than the settlement of the cell studied during Step 1, as a result of the load diffusion by means of shaft friction towards the surrounding soil block (Figure 24).

Step 3: The load-displacement curve of an inclusion assumed to be isolated (including the granular pad prism displaying the same cross-section as inclusion) in a soil block subjected to an imposed settlement profile as calculated in step 2 makes it possible to establish the load value to apply at the head of this column in order to obtain the same settlement as previously calculated at top of the model. This load value then determines the distribution of axial forces $Q_p(z)$ in the actual inclusion.

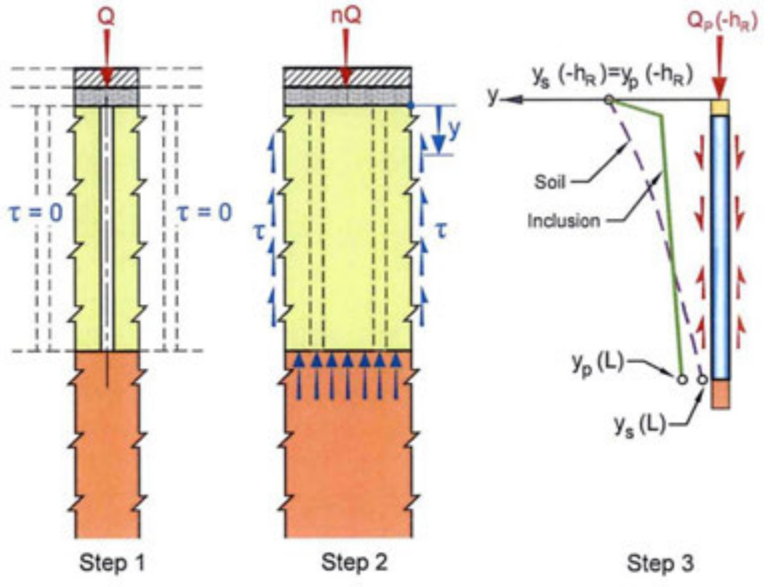


Figure 24: Principle of the simplified method under vertical loading (Simon, 2012)

The comparison of results concerning the inclusion axial load or the pile and average soil settlement has shown fair agreement between numerical modelling with Flac 3D calculation and the simplified method (Figure 25).

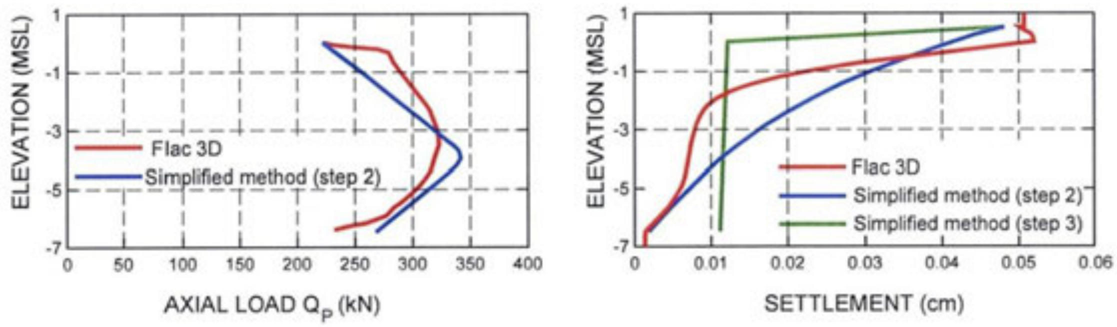


Figure 25: Comparison between simplified analytical method and numerical 3D modelling (Simon, 2012)

1.7.2. Under transverse loading

Under transverse loading, the simplified method is composed of two successive additional steps. Both make use of an ordinary tool for pile foundation design, i. e. study of an isolated pile bearing on elastic-plastic springs and subjected to transverse loading.

Step 4: The monolith with an equivalent modulus of E* (as established during step 1) is assimilated with a transversely-loaded pile interacting with the external unreinforced soil block via elastic-plastic springs The calculation establishes a lateral displacement profile g(z) for the monolith under action of the horizontal force T and bending moment M loading applied to the footing (Figure 26). The limited monolith length-to-width ratio and its orthotropic nature however necessitate taking shear deformations of the pile into account, in addition to bending deformations. The simple model of a slender beam, commonly used for piles, tends to be inappropriate. These shear deformations are controlled by the G*A' factor (with G* being the equivalent shear modulus of the monolith and A' the reduced shear cross-section).The equivalent shear modulus G* may be assimilated with the shear modulus G_{sol} of the soil on its own (since the contribution of inclusions to shear strength in effect remains negligible). Bending deformations depend from the factor E*I (where E* is the monolith's apparent equivalent modulus - established during step 1- and I the monolith flexural rigidity).

Step 5: A subgrade reaction pile model, limited to the inclusion alone and assumed subjected to the previous displacement field g(z), enables to calculate the shear force and bending moment distributions in the inclusion for any given set of boundary conditions at the inclusion head and tip.

A horizontal force can develop at any inclusion head by friction exerted by the granular pad. A limiting value of this force can thus be found by considering the concomitant axial load in the same case. One can nevertheless observe that this force cannot induce a displacement of the inclusion head that exceeds displacement of the surrounding soil. Therefore a quite conservative assumption consists of selecting for the boundary condition $T_p(0)$ a value that "reduces" inclusion head displacement to that of the surrounding soil. The associated axial forces in the inclusion placed at the centre of this cell can then be estimated by assimilating them with the axial forces found under a uniform vertical loading of the cell that yields the same settlement. This step is performed by means of a specific calculation linking Steps 2 and 3. The values of the corresponding axial force, shear force and bending moment obtained according to the vertical and transverse load cases must be combined in order to verify stresses in the inclusions.

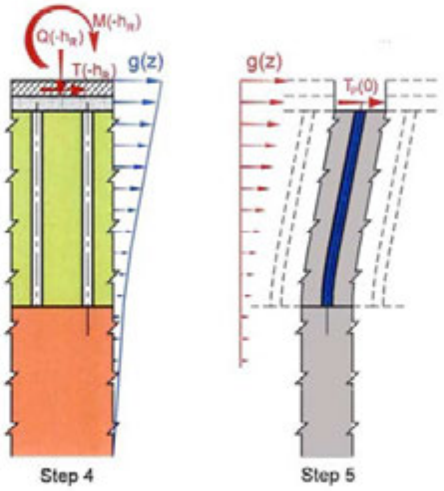


Figure 26: Principle of the simplified method under horizontal loading (Simon, 2012)

Figure 27 plots the different inclusion displacement fields which are obtained when the shear force boundary condition $T_p(0)$ value is varied between 0 and the one giving equal soil and inclusion displacements at head. They can also be compared to the front inclusion displacement field as calculated by Flac 3D. This latter one reveals a strong similarity with the simplified method curve for $T_p(0) = 0$. This suggests that the granular pad shear strength was fully mobilized under the vertical load component, leaving no residual friction capacity in reaction to any soil-pile horizontal displacement; this could also be stated: "*the vertical axis remains a principal stress direction in the vicinity of the inclusion head during transverse loading*". Further evaluation of the simplified method is planned using the results of an on-going dedicated centrifuge testing program also funded by the ASIRI project.

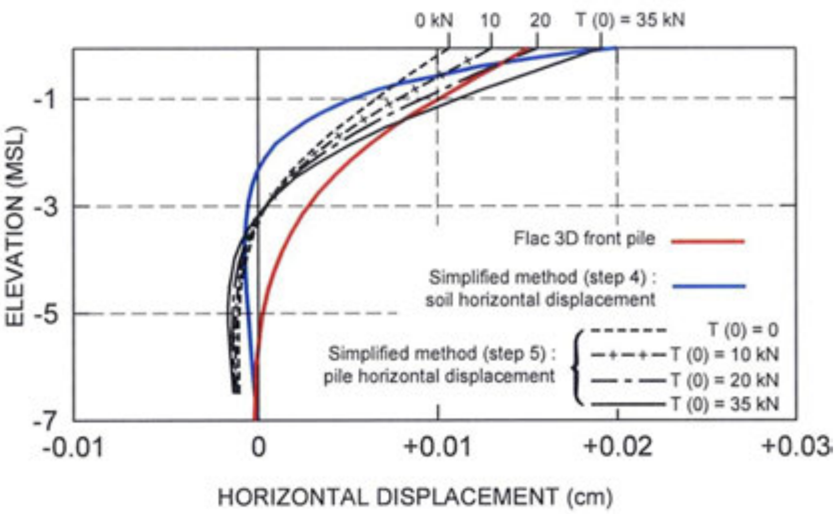


Figure 27: Comparison of the horizontal displacement profile calculated by Flac 3D model and the simplified method for a range of shear force boundary conditions T(0) at the inclusion head (Simon, 2012)

1.8. References of Section 1

Thorel L., 2012, Lessons from centrifuge model tests on piled embankments and foundation over soil reinforced by inclusions, short course rigid inclusions, IS-GI Brussels 2012.

Eekelen S. V., 2012, Piled embankments reinforced with geosynthetics: model experiments and analytical model developments, short course rigid inclusions, IS-GI Brussels 2012.

Walker M., 2012, US practice in Load Transfer Platform design: Lessons learned, short course rigid inclusions, IS-GI Brussels 2012.

Simon B., 2012, A review of the ASIRI research project findings about behaviour and design of foundations on soil reinforced by rigid inclusions, short course rigid inclusions, IS-GI Brussels 2012.

Simon B., 2012, Design of spread footing on soil reinforced by rigid inclusions, short course rigid inclusions, IS-GI Brussels 2012.

Racinais J., 2012, Design of slabs on grade supported with reinforced soil by rigid inclusions, short course rigid inclusions, IS-GI Brussels 2012.

Simon B., 2012, General report about ground improvement using stone columns and rigid inclusions - International Symposium on Ground Improvement IS-GI Brussels May 31st & June 1st 2012

ASIRI, 2012, Amélioration des sols par inclusions rigides, (with English translation on CD: Rigid Inclusions Ground Improvement) /Presses des Ponts/ISBN 978-2-85978-462-1

Eekelen, S.J.M. van, Bezuijen, A., 2012, Basal reinforced piled embankments in the Netherlands, field studies and laboratory tests, Proceedings of the international symposium IS-GI, Brussels

Dias, D., Simon, B., 2012, Spread foundations on rigid inclusions subjected to complex loading: Comparison of 3D numerical and simplified analytical modeling, Proceedings of the international symposium IS-GI, Brussels

Okyay, U. S., Dias, D. , Thorel, L. & Rault, G., 2012, Centrifuge modeling of a pile-supported granular earthplatform over soft soils, submitted for publication in ASCE, Journal of Geotechnical and Geoenvironmental Engineering.

Racinais J., Plomteux C.,2011, Design of slab-on-grades supported with soil reinforced by rigid inclusions, 21st EYGEC, Rotterdam.

2. SOIL REINFORCEMENT

The Section 2 is based on the lectures by J. Sankey, J.Wynn, N. Freitag and T. Durgunoglu and combines the different topics that were presented. During these lectures, the origins, the design and future applications of reinforced soil were discussed. Special attention was given to the different national standards that exist, as well as the impact of the choice between different types of facing and reinforcements on the design.

2.1. Introduction

2.1.1. Origins

Reinforcing soil is a concept that was adopted as early as the Persians, who used pressed paper to create steep embankments. Later, also the Romans used wicker mats to reinforce their roads, of which some have even survived till today. Only more recently, during the last 100 years, the concept was updated, with the invention of steel and later polymer reinforcements.

Most famous and internationally successful is the concept of "Terre Armée", originally patented in 1965 by Henri Vidal. Legend goes he developed the concept while building sand castles on a beach, with reinforcements consisting of pine needles. This simple idea led to a new commercial concept, which was originally used in France but soon spread over the rest of the world. Since then, reinforced soil is a globally accepted technique to create steep embankments or provide greater bearing capacity for roads, railways, dykes …

Figure 28: Concept of original Terre Armée

The original concept consisted of steel reinforcement strips and a metallic or concrete facing (Figure 28). The strips were smooth and coated with 30 µm of galvanisation. Stresses were calculated with active soil pressure from top to bottom, and service life was the lifetime of the steel reinforcements.

2.1.2. Further developments

When the concept was used more and more, the design was updated with new empirical and theoretical design models (coherent gravity, tieback wedge ...) and new types of reinforcement (ribbed steel, metallic grids, polymers ...) and facings (concrete, gabions ...). As a result, the more generic term of Mechanically Stabilized Earth or MSE was adopted.

Due to this large variation in types of reinforcements and facings, the design has become more and more complex, and the designers have to be aware of the different design methods and national codes. The basic components of an MSE-wall however, always remain the same (Figure 29): a reinforcing element is placed within a selected backfill. The facing element at the front of the wall only has to take a limited part of the load and is therefore usually small and light.

Figure 29: Basic components of an MSE-wall

Different types of MSE-walls can be distinguished by the use of different materials for these basic components. All of these can be combined to create numerous technical solutions, each with their specific design requirements, aesthetics and durability.

2.2. Types of MSE-walls

2.2.1. Different types of facing

The most visible component of an MSE-wall is the facing element. All of the different types have the following in common:
- Ease in installation (lightweight),
- Versatile (small modular blocks),
- Tolerate settlement well (flexible),
- Economical,
- Dependable and sustainable.

Usually, facing types are distinguished based on the flexibility of the element, since this also relates to the design methodology that must be followed. The flexibility of the facing has to be taken into account in the design and has to be compatible with the flexibility of the reinforcement.

Rigid facing

Although all types of MSE-walls are relatively flexible (compared to cantilever or gravity walls), some facings are called rigid since the individual elements are unable to take up deformations. Rigid facings consist of full height concrete panels, steel (sheet) piles and some types of modular concrete panels or concrete blocks. Full height panels can provide a smooth finish from top to bottom of the wall (photo 1).

Photo 1: Back view of full height concrete facing with polymer geogrid attachments

Alternatively, cast-in-place or shotcrete facing can also be applied. Since these are placed (some time) after construction of the MSE-wall, they do not suffer restrictions from deformations during construction. This is the reason why this technique is the favored construction method in some countries (Koerner, 2005). Similarly, a screen wall can be placed in front but not in direct contact with a wraparound facing, to avoid an impact of post-construction deformations on the (rigid) facing element.

Semi-flexible facing

Sometimes, joints or openings are placed between the otherwise rigid facing elements, creating so-called semi flexible systems, e.g. modular panel walls.

These modular panels are prefabricated and can thus easily be provided with architectural texturing or printing (photo 2).

Photo 2: Bicycle artwork on prefabricated concrete facing

Alternatively, gabions can also be used. In this case, the gabions themselves take up some of the deformations.

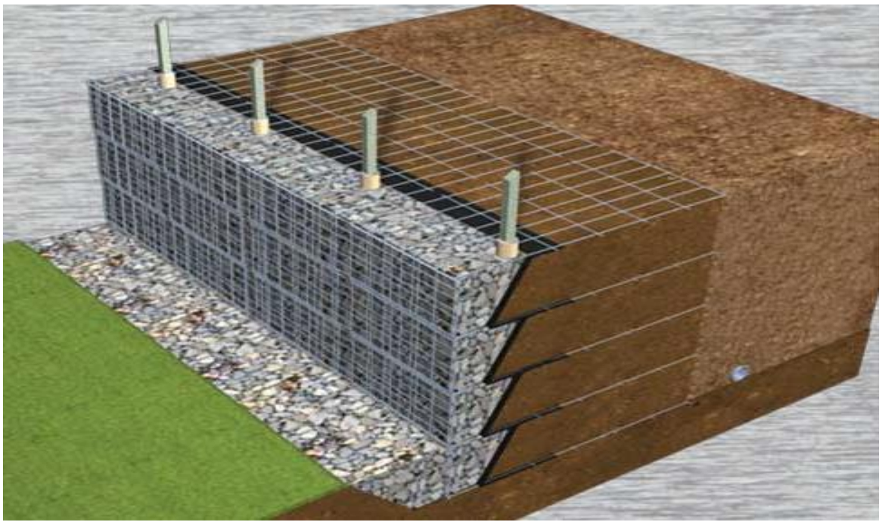


Figure 30: Gabion facing

In some cases, block walls can also be placed in this category (depending on joint-type).

Flexible facing

Finally, flexible facing can be used to create a finished green or natural slope. Flexible walls are typically battered wraparound walls with polymeric reinforcements or consist of a welded wire-mesh without a stone fill.

Photo 3: Wrap-around MSE wall with geosynthetic reinforcement

2.2.2. *Different types of reinforcement*

Similarly to the different types of facing, reinforcements can be classified in two categories based on their rigidity. The rigidity of the reinforcement must be compatible with that of the facing, and must be taken into account in the design of the system.

Inextensible reinforcement

Inextensible or rigid reinforcement usually consists of steel strips, ladders, or wire meshes, but glass products can be used as well.

Photo 4: Inextensible reinforcement, e.g. steel strips

Inextensible reinforcements may be preferred to provide a consistent and aesthetic finish, since they will lead to the smallest deformations during and after construction. Also, handling and placement is very simple.

The use of steel should however be avoided in difficult electro-chemical conditions. The corrosion rates and lifetime of preservatives such as galvanization remains to be considered by the designer, especially in such difficult conditions. Also, since the use of geosynthetic reinforcement has quickly become more widespread, the use of steel reinforcement is unfamiliar to some contractors or in some countries.

Extensible reinforcement

Extensible reinforcement consists of polymers or plastic, usually geosynthetics such as geogrids or geotextiles (photo 5). They are preferably used in difficult electro-chemical conditions, related to the lifetime of steel materials, and when the backfill contains larger aggregates.

Photo 5: Extensible reinforcement, e.g. polymer strips

However, similar to steel products in difficult electro-chemical conditions, the right polymer must also be selected based on specific soil conditions (e.g. hydrolysis of polyester fibers in an alkaline environment). Furthermore, all polymers exhibit creep and thus time dependent deformations which may lead to unwanted post-construction deformations. Once again, different types of polymers have a different resistance against creep.

Also, stiffer and stiffer types of polymers such as PVA can be used (Verstraelen, 2011 & 2012) and the design can also be made to take into account these deformations (e.g. overdesign of tensile strength to limit construction strain).

2.2.3. Different types of functionality

Finally, MSE-walls can be distinguished by their application (Figure 31). However, it must be noted that every practical material as discussed in the previous paragraphs can be used for virtually every application in Figure 31.

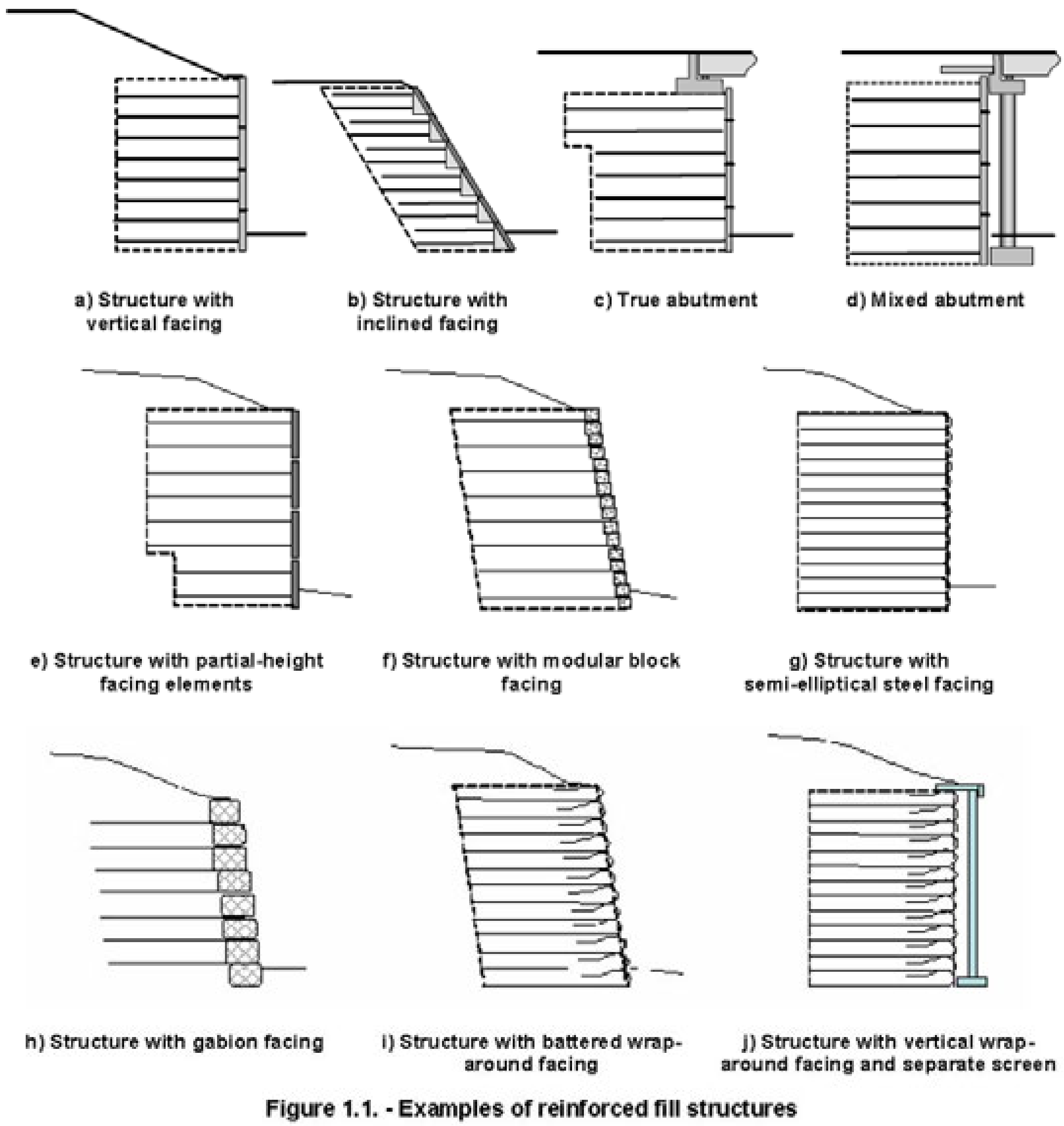


Figure 31: Different structures using MSE-walls

2.3. Design of MSE-walls

The design of MSE-walls is covered by different national standards. These standards do not all cover the same scope and use different methodologies to calculate the same properties. Most common are the AASHTO LRFD, NF P94-270, BS 8006 and EBGEO.

		Reinforcements								
		Steel			Geosynthetic					
Country	Standard	strips	welded wire mesh	woven mesh	strips	sheets	walls	steep slopes	bridge abutments	Soil nailing
USA	Aashto LRFD 2012	x	x			x	x	x	x	
France	NF P 94270:2009	x	x	mentioned	x	x	x	x	x	x
UK	BS 8006:2010	x	x		x (BBA)	x (BBA)	x	x	x	
Germany	EBGEO					x	x	x		

		SLS	ULS				
Country	Standard	Deformations	External stability	Internal stability	Site general stability	Compound stability	Seismic design
USA	Aashto LRFD 2012	rule of thumb	x	x	x		x
France	NF P 94270:2009	FEM / FDM	x	x	x	x except for simple cases	x with EN 1998
UK	BS 8006:2010		x	x	x	x	x only indicative
Germany	EBGEO	rule of thumb	x	cf compound		x	

Figure 32: Scope and applications of different national standards

Although these standards differ in scope and methodology (Figure 32), the general aspects are the same:

- Load and resistance factoring is used to create a safe interval between maximum load and minimum resistance,
- Design in ULS, to a lesser extent in SLS,
- Different failure mechanisms must be investigated (Figure 33):
    - General site stability,
    - External stability,
    - Internal stability,
    - Compound stability.

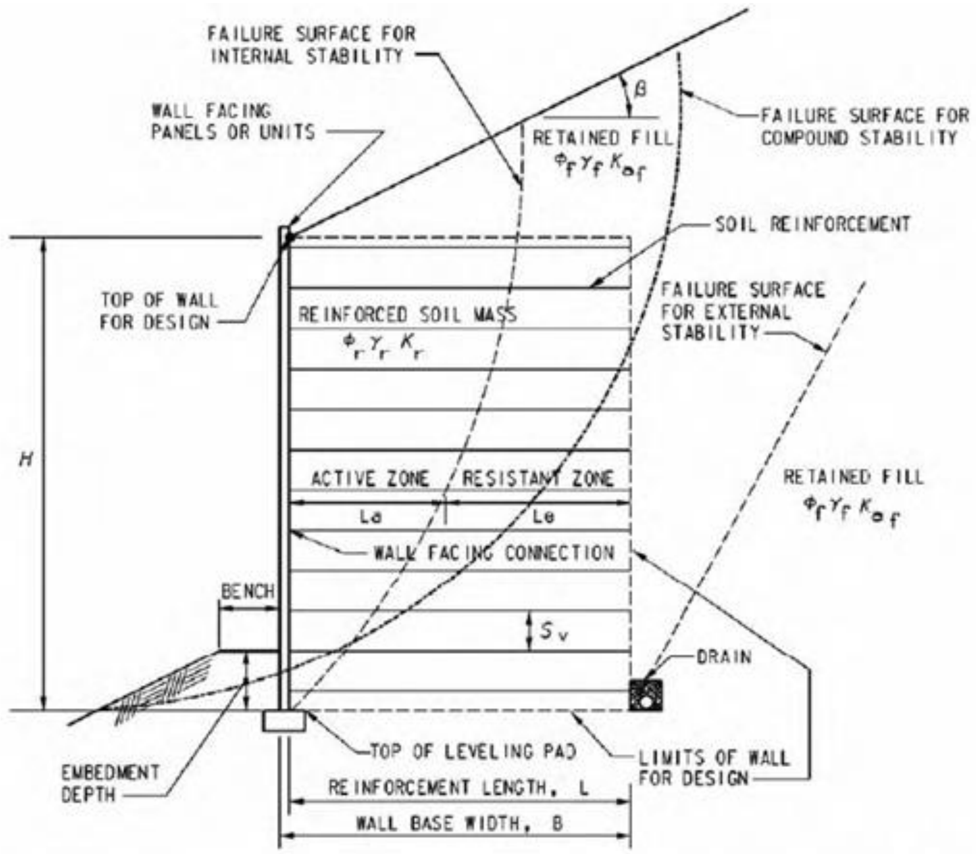


Figure 33: Different failure mechanisms (from AASHTO LRFD bridge design, 2010)

Among these different failure mechanisms, the internal and external mechanisms can be studied with a structural approach, the general and compound stability can be investigated with a geotechnical approach.

In a structural approach, the reinforced soil is treated as a gravity wall subjected to self-weight, soil pressure, live loads etc. The reinforced mass is treated as a composite material with an equivalent anisotropic cohesion. Forces within the reinforcements are calculated with empirical models (e.g. coherent gravity method) or extrapolations of scale models.

In a geotechnical approach, the reinforced soil is investigated with a theoretical model which is an extension of conventional slope stability theory, e.g. Bishop slice method, logarithmic spirals...

2.3.1. General site stability

General site stability consists of deep sliding of the entire MSE-wall, considered as a monolithic block. In this analysis, the type or properties of the reinforced soil do not influence the outcome of the analysis. Typically, this type of failure may be dominant when different structures, such as MSE-walls, are combined (Figure 34). General site stability or overall stability can be investigated using conventional techniques.

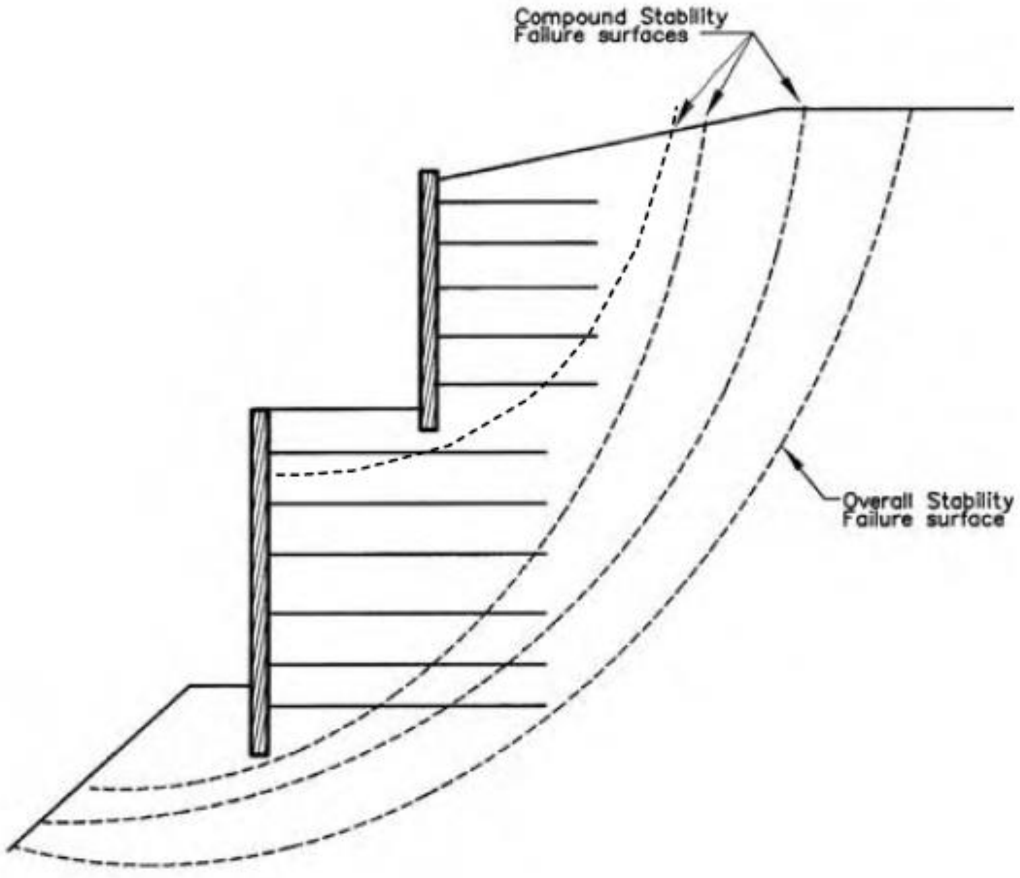


Figure 34: Overall stability (from AASHTO LRFD bridge design, 2010)

2.3.2. External stability

External stability, similar to general stability, is independent of the properties of the reinforced soil or reinforcing elements. When studying external stability, the following different failure mechanisms have to be considered (Figure 35):

- Bearing capacity, taking into account load eccentricity and inclination,
- Sliding,
- In some codes: overturning (or limit on eccentricity).

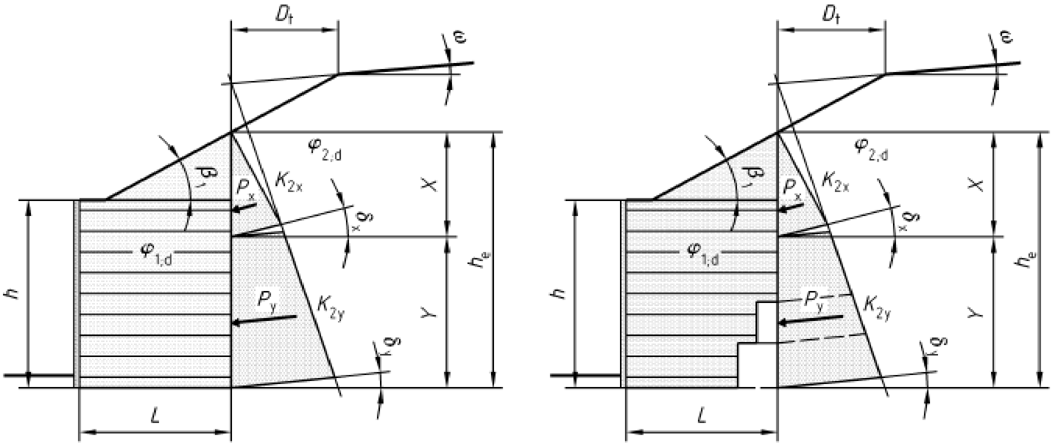


Figure 35: External stability (from NF P94-270)

The different national standards may provide details about each different type of analysis. These standards may however differ in the partial factoring and methodology, e.g. which value for δ should be used to calculate the active earth pressure behind the reinforced soil (ranging from 0° to 2/3 φ', depending on the chosen standard).

2.3.3. *Internal stability*

Internal stability addresses the loads and resistance of the reinforcing elements and connections. The design consists of calculating the loads in the reinforcements, and checking for:
- Rupture of the reinforcement,
- Pull-out of the reinforcement,
- Rupture of the connection between facing and reinforcement.

Different guidelines exist for determining the load in the reinforcement, all of which are based on the concept of a so called line of maximum tension. This line determines the position of the maximum tensile force in the reinforcement and divides the reinforced soil in an active and resistive zone. However, the form and shape of this line may differ between standards, as well as the value of earth pressure coefficient that should be adopted. An example based on the French NF P94-270 is presented in Figure 36.

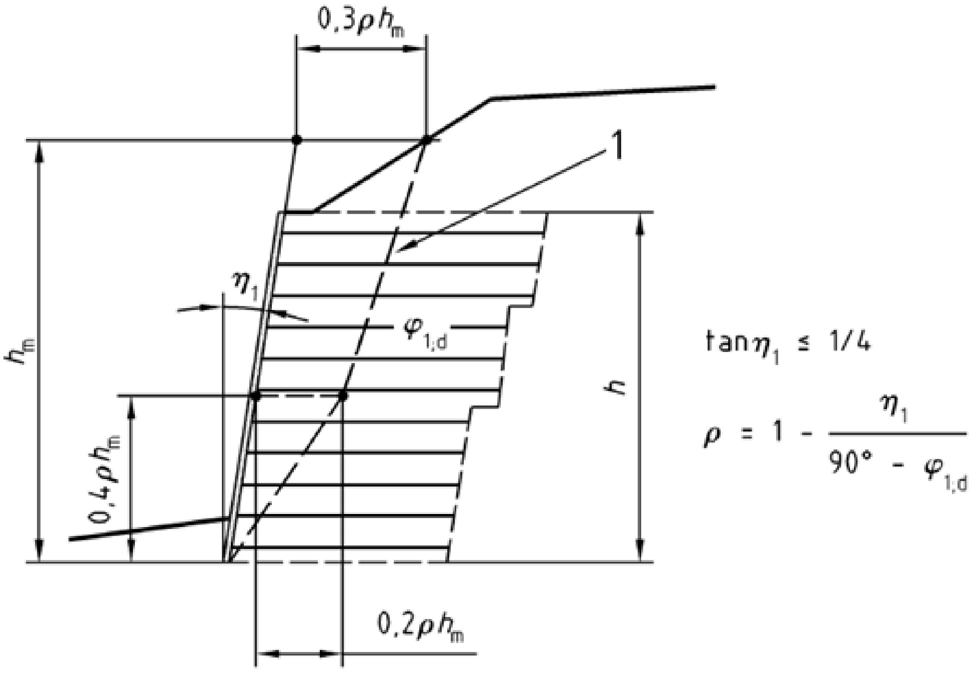


Figure 36: Line of maximum tension (from NF P94-270)

The line of maximum tension in Figure 36 is similar to that of the coherent gravity method (as used in BS 8006). The original coherent gravity method was based on empirical measurements and was valid for inextensible reinforcements. In these measurements, it was noted that the earth pressure near the surface was higher than the active earth pressure. Only at a depth of about 6m, an active earth pressure was noted. The value near the surface was near to K0. The turnover depth of 6 m was adopted in most codes, but the value of the earth pressure coefficient near the surface depends on the standard (e.g. 1.6xKa for NF P94-270, K0 for BS 8006 and dependent on the type of reinforcing element for AASHTO, see Figure 37).

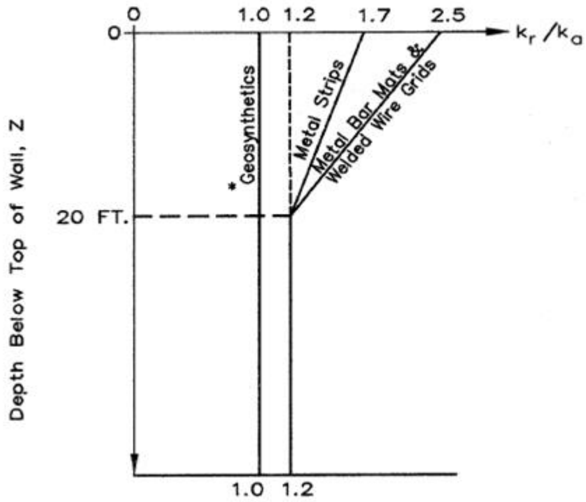


Figure 37: Earth pressure coefficient (from AASHTO LRFD bridge design, 2010)

When considering inextensible reinforcements, it is considered that the bond length develops over the full length of the reinforcement. The mobilized shear strength will have to remain smaller than the soil's peak shear strength in order to avoid pull-out. The unused shear strength provides the factor of safety against pull-out.

For extensible reinforcements, the bond length develops over a part of the length of the reinforcement. Along this partial length, peak shear strength may be developed (localized). The remaining unused bond length provides the factor of safety against pull-out. The failure plane according to this method, the tieback wedge method, is a straight line at an active angle.

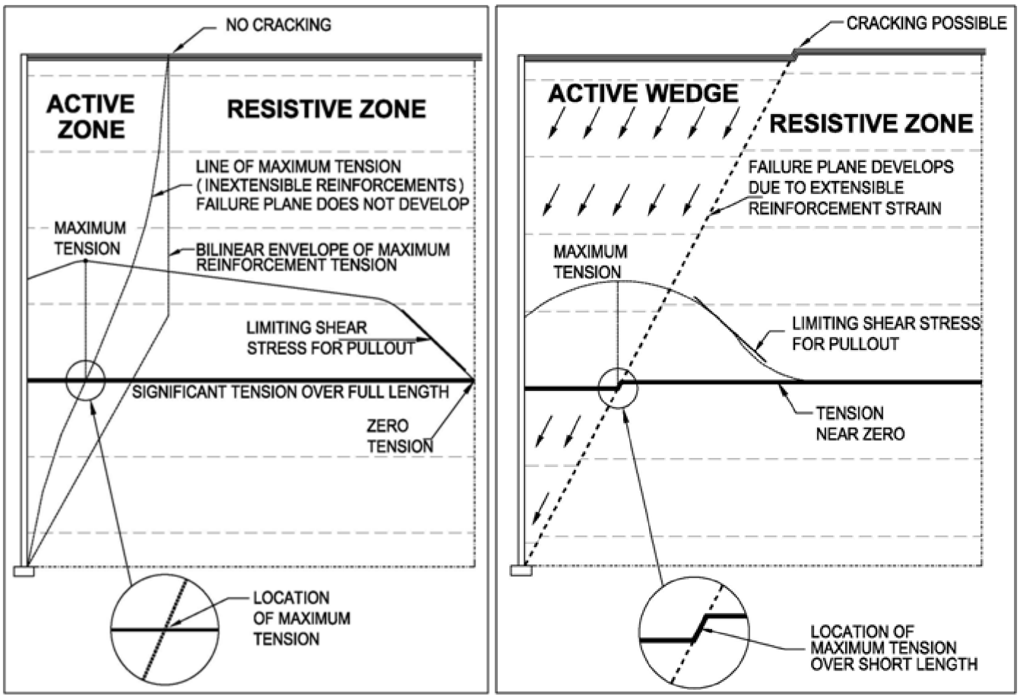


Figure 38: Tensile load along reinforcement (inextensible versus extensible reinforcement).

The design against pull-out must also take into account interaction coefficients, which describe the (frictional) interaction between the reinforcing element and the surrounding soil. These will depend on:

- The material type of the reinforcements (steel/polymer),
- The shape of the reinforcement (strip, sheet, ...)
- The confinement level.

The connection with the facing can be designed with a specified reduction on the maximum tensile load. The extent of this reduction once again depends on the flexibility of the facing and on the chosen standard or code, with values ranging from 1 (AASHTO, independent of flexibility of the facing) to 0.75 (NF P94-270, for flexible facing).

Finally, the rupture strength of the reinforcement can be designed. The necessary strength is largely dependent on the type of reinforcement (steel/polymer), see also paragraph 4.

2.3.4. Compound stability

Finally, compound or mixed stability investigates critical failure mechanisms in which only part of the reinforcement is involved in the analysis. This typically consists of a slip circle analysis in which only part of the reinforcement is crossed or the slip circle runs along the reinforcement (see also Figure 34).

2.3.5. Deformations in SLS

Deformations in SLS are more difficult to determine, but are sometimes the decisive factor in accepting an MSE-wall or not. The national codes however, only provide a rule-of-thumb to estimate the deformations or refer to FEM/FDM calculations. Not only the calculation, but the limiting value is sometimes difficult to determine, since this will rely on the project, type of structure ... and is thus not an absolute value or specified in codes. This limit to settlement or deformation must also be compatible with the chosen type of facing element. Some codes (e.g. CUR 198) provide allowable settlements for different types of facing elements.

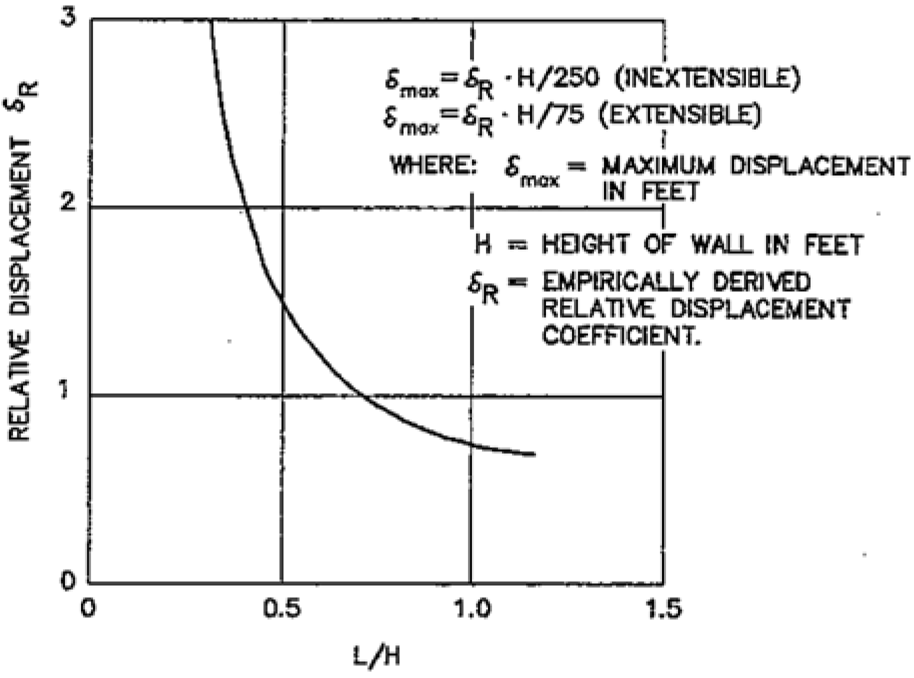


Figure 39: Estimation of wall displacement (from AASHTO LRFD bridge design, 2010)

2.4. Design of tensile strength and durability

The design of the tensile strength and the durability of the reinforcement is a main issue when designing an MSE-wall. The design is very dependent on the type of reinforcement, and a distinction is made between steel and polymer reinforcements.

2.4.1. Design of polymer reinforcement

The necessary design strength for polymer reinforcements is calculated from the short term characteristic tensile strength $R_{t,k}$ with appropriate reduction factors as in equation 1.

$$R_{t;d} = \rho_{end}\rho_{flu}\rho_{deg}\frac{R_{t;k}}{\gamma_{M;t}} \qquad (1)$$

The following reductions are applied:
- ρ_{end} : reduction for installation damage,
- ρ_{flu} : reduction for creep,
- ρ_{deg} : reduction for chemical degradation (e.g. hydrolysis for polyester),
- $\gamma_{M,t}$: partial material factor (1.25 in NF P 94-270).

The reduction factors for installation damage, creep and chemical attack are mostly determined by the type of polymer and the general circumstances of their application. The national codes provide methodologies for testing or determining these reduction factors and values can be obtained from manufacturers. General reduction factors are presented by Koerner, 2005.

2.4.2. Design of steel reinforcement

The design of steel reinforcement must take into account the effect of corrosion. In most codes, this is specified as a loss of thickness with time, for which tables of corrosion rates can be used. The standard NF P94-270 however, uses equation (1) with:
- $\rho_{end} = \rho_{flu} = 1$,
- ρ_{deg} : reduction for corrosion,

and calculates the design strength at yield and rupture (for which different material factors apply).

2.4.3. Durability and monitoring

Durability is always a main concern when proposing the use of MSE-walls. This could be easily resolved by monitoring. Also, monitoring would provide an answer about concerns for the lack of an early warning in case of imminent failure. However, monitoring is not clearly developed in current standards.

It must be noted that this is the case for both polymer and steel reinforcements, although the cause for concern is different (corrosion versus chemical degradation). There is a need for the development of chemical tracers, which is on the way for chemical ageing of polyester.

2.5. New applications

Although the use of MSE-walls is relatively new, this type of wall is more and more being used in new applications or in combination with other techniques. Since the design of MSE-walls is only recently being developed in different national codes, these new applications are not covered.

One of such new applications is the combination MSE-walls with soil reinforcement. Such soil reinforcement may consist of stone columns, soilmix, controlled modulus columns or rigid inclusions, ... Depending on the type and thickness of the load transfer platform (if present), the reinforcement of the MSE-wall may be influenced by the load transfer mechanism that develops between the soil reinforcement elements. This transfer mechanism may work in other directions compared to the MSE-wall‘s principal reinforcement direction (usually uniaxial).

Figure 40: Combination of MSE-wall with soil reinforcement

Another new application is the combination between MSE-walls and soil nailing. Both types of walls lie within the same geotechnical domain and are covered by the same standards and codes. While soil nailing is used to make cuts in existing embankments or slopes, MSE-walls are used to create new steep slopes or walls. In many cases, such as the extension of existing embankments, large cuts would be necessary to create enough width to use a conventional MSE-wall. When combining both, the cuts can be limited and also the width of the new MSE-wall is limited.

Figure 41: Combination of MSE-wall with soil nailing

The design of such a combined wall can be carried out using the existing guidelines for both separate systems, and example of an analysis of an overall (when looking from the MSE-wall point of view) or internal (when looking from the soil nailing point of view) failure mechanism is shown in Figure 42.

A specific point of interest for this application is the connection or load transfer between both reinforcing elements, i.e. the soil nails and the MSE-wall reinforcements. Distinction can be made between systems

using a direct link, where both are connected, and a friction link, where the forces are transmitted as frictional forces in the reinforced soil with an overlap length between both systems.
A direct link is a simple but vulnerable system when differential settlements may occur. Also, compaction loads are difficult to estimate.

A friction link provides a flexible solution and is easy to implement. Primary reinforcement is the traditional MSE-wall reinforcement that extends to the soil nail wall. Secondary reinforcement is attached to the nail heads (ladders, strips). A sufficient overlap between both is designed to transfer all loads through the reinforcement soil mass.

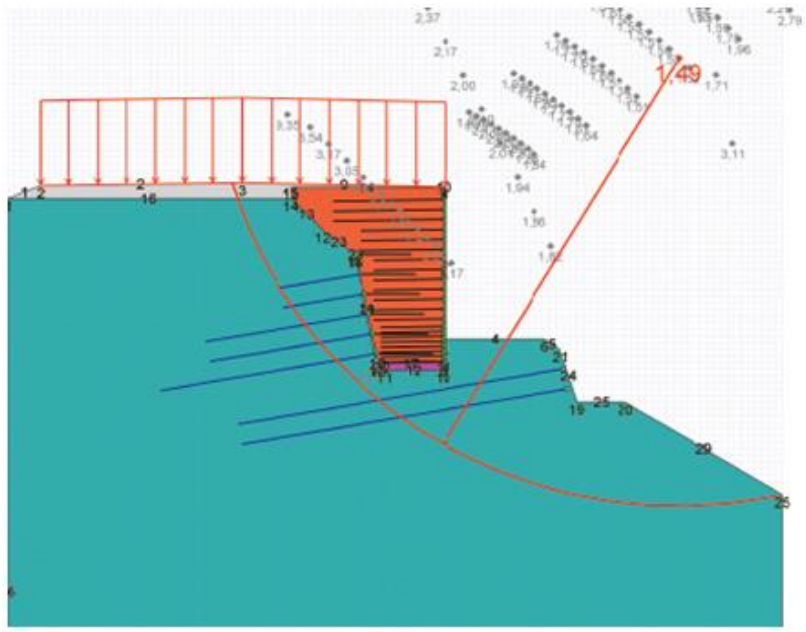


Figure 42: Stability analysis of MSE-wall with soil nailing

2.6. Conclusions

During the short course regarding soil reinforcement, the different speakers clearly presented the different aspects and possibilities of using reinforced soil. The technique remains very economical, versatile and durable, but the design is still in a developing phase. The most recent codes and their different design methodologies where presented, showing their similarities and differences. Finally, although the conventional design is still debated, new applications already emerge and create the need to extend current codes and practices.

2.7. References of Section 2

J. Sankey, 2012, Introduction to mechanically stabilized earth wall technology, <u>short course soil reinforcement</u>, IS-GI Brussels 2012

J. Wynn,2012, Review of inextensible and extensible MSE reinforcement applications, <u>short course soil reinforcement</u>, IS-GI Brussels 2012

N. Freitag, 2012, Application of Local Country Codes for MSE wall design, <u>short course soil reinforcement</u>, IS-GI Brussels 2012

T. Durgunoglu, 2012, Combined MSE wall applications with ground improvement technologies, <u>short course soil reinforcement</u>, IS-GI Brussels 2012

NF P94-270, Ouvrages de soutènement, remblais renforcés et massifs en sol cloué

AASHTO, 2010, LRFD Bride design specifications

BS 8006-1:2010, Code of practice for strengthened/reinforced soils and other fills
CUR 198, 2000, Kerende constructies in gewapende grond, taludhelling steiler dan 70°

EBGEO, 2011, Recommandations for Design and Analysis of Earth Structures using Geosynthetic Reinforcements, Ernst & Sohn

R. Koerner, 2005, Desiging with Geosynthetics, Pearson prentice Hall

J. Verstraelen, E. De Clercq, 2011, Green terraces for the Regional Express Network in Brussels, Proceedings of the 21st European Young Geotechnical Engineers' Conference, Rotterdam

J. Verstraelen , C. Lejeune, E. De Clercq., 2012, Realisation of integrated steep landscape slopes within existing railway embankment, Proceedings of the international symposium IS-GI, Brussels

SPONSORS

PLATINUM SPONSORS

On 26 October 2000, 7 foundation companies decided to represent the interests of the foundation sector through the creation of a non-profit association under the name ABEF. Meanwhile, the association already consists of 17 members.

The association aims to create healthy relationships in the foundation sector and to represent and defend the economical and moral interests of its members, as long as it concerns matters that are related to the foundation business.

It seeks to achieve this aim with all legal resources by:

- promoting a constructive relation between the members, associations, companies and institutions who come into contact with the sector;
- adopting positions regarding the preparation of rules, regulations, documents, etc. which are applicable on foundation works,
- attracting employees and promoting professional competence;
- organizing training in deep foundation techniques for workers and site-managers;
- the pursuit of a high quality label in all aspects of the foundation sector;
- delivering opinions to the members in possible conflicts;
- the connection to and the participation in the European Federation EFFC (European Federation of Foundation Contractors;
- the participation in working groups and committees in the framework of National and European standards, directives, documents and research programs;
- undertaking all other legal interventions that are in line with the objectives of the association.

ABEF
Lombardstraat 34-42 – B-1000 BRUSSEL
T: 00 32 (0)2 545 57 58 - F: 00 32 (0)2 513 24 16
info@abef.be
www.abef.be

The non-profit association ABEF is since 2012 part of FEDECOM, - the federation of complementary companies of the construction sector, that is on its turn part of the federation of Belgian building contractors.

The members of ABEF are:

OLIVIER SOLID FOUNDATIONS
FONDEDILE
EUR DRILL
Smet-Boring
de waal PALEN - PIEUX
VOTQUENNE FOUNDATIONS
CVR
SOILTECH SOIL & PILING CONTRACTORS
ProFFUND N.V.
SOLETANCHE BACHY
Franki Foundations Belgium
FONTEC
LAMEIRE future foundation technics
HEVI BVBA
FUNDEX
VOTQUENNE
WIG PALEN

ABEF
Lombardstraat 34-42 – B-1000 BRUSSEL
T: 00 32 (0)2 545 57 58 - F: 00 32 (0)2 513 24 16
info@abef.be
www.abef.be

BESIX

Beyond Challenges

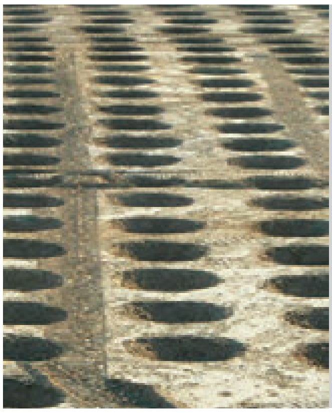
Port and marine projects have been one of BESIX's spearheads since the company was first established. BESIX focuses on the design and implementation of on-shore and offshore projects: locks, breakwaters, seawalls, seawater intakes, quay walls, gas or ore terminals, and LNG (liquefied natural gas) jetties.

Investors are attracted by BESIX's international experience, know-how, its technical skills and the innovatory solutions it offers, as well as by the marine equipment at its disposal, enabling work to get started quickly.


www.besix.com

Cofra

Soil improvement

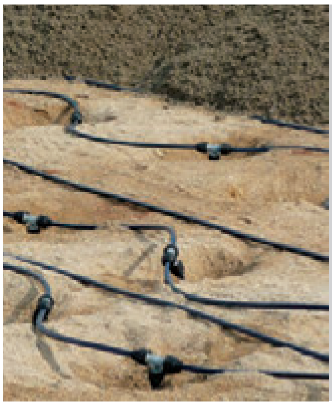


Our history

Cofra is an experienced and proven partner in soil improvement techniques, founded in 1923. Originally an Amsterdam based contractor, with local operations, Cofra has evolved into an international specialist in soil improvement. Cofra has international offices and an extensive network of agents. Since 2006 Cofra has been a division of Royal Boskalis Westminster, a dredging conglomerate with international operations. This has further strengthened Cofra's position worldwide.

Our core values

Innovation, reliability and professionalism are Cofra's primary core values. Its proactive approach and advanced equipment designed in-house, among other things, have earned Cofra a leading position as a soil improvement specialist. Its core values have made Cofra a reliable international knowledge partner in civil engineering.



Our techniques

Cofra has a wide range of techniques for soil improvement. With state-of-the-art equipment and its own geotechnical engineering department, Cofra specialises in soil improvement techniques and environmentally-protective liner techniques. Cofra's consolidation and compacting techniques provide the solution for construction site preparation in international infrastructure projects and land reclamation projects.

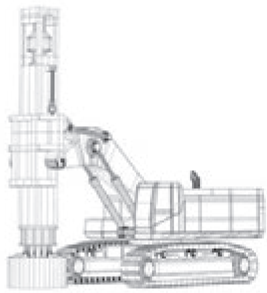
Cofra

T +31 (0)20 693 45 96, F +31 (0)20 694 14 57
www.cofra.com, mail@cofra.com
Cofra BV, P.O. Box 20694, 1001 NR Amsterdam
The Netherlands

Amsterdam Bratislava Londen Singapore Stockholm

Cofra

CDC compaction



Applications of the CDC technique include:
> Improvement of land reclamation projects
> Preparation for tank terminals
> Preparation for major infrastructure projects in granular soils
> Densification of embankments

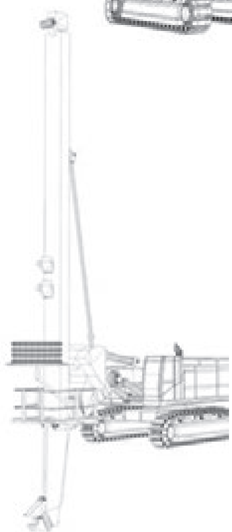
Advantages of CDC:
> Cost-effective
> Compaction impact down to depths of up to 9 metres
> Real-time GPS monitoring
> Flexibility

MebraDrain vertical drainage

Applications of vertical drainage include:
> Accelerated preparation of construction sites
> Site preparation for different types of infrastructure projects
> Construction of embankments
> Soil improvement for land reclamation projects

Advantages of MebraDrain vertical drainage:
> Sophisticated equipment - light to heavy duty
> Short consolidation periods
> Quick installation
> Installation to a drain depth of 65 m

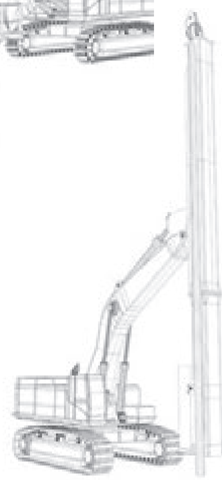
BeauDrain(-S) vacuum consolidation



Applications of BeauDrain include:
> Accelerated preparation of construction sites
> Site preparation for different types of infrastructure projects
> Soil improvement for land reclamation projects
> Expedited construction of embankments

Advantages of BeauDrain:
> Short installation period
> Clean work area after installation (BeauDrain)
> Very large working depths possible (BeauDrain-S)
> Suitable for installation through thick sand layers (BeauDrain-S)

AuGeo embankment



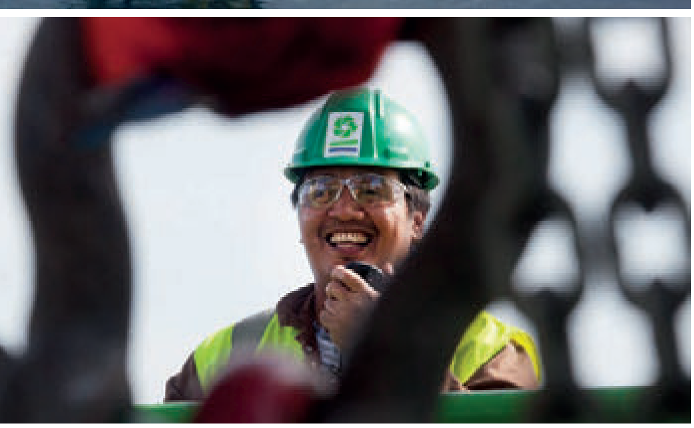
Applications of AuGeo include:
> Construction and widening of embankments
> Construction and widening of various types of infrastructure
> Foundation of roads in urban development areas
> Foundation of industrial flooring systems

Advantages of AuGeo:
> Quick, vibration-free and low-noise installation
> No settlement period and no risk of instability
> No impact on the surrounding area
> Guaranteed pile diameter in peat soil

Building worldwide on our strength

A multidisciplinary approach for new horizons




As a specialist geotechnical contractor, Menard delivers complete design-build foundation systems using ground improvement techniques. Our solutions are creative and cost-saving alternatives to the deep foundations traditionally used to support buildings and structures.

A Comprehensive Service

As a result of the geotechnical know-how accumulated through the completion of thousands of projects, MENARD consistently provides the essentials for success: a good price, a reliable schedule, well managed construction techniques and a commitment to excellence in safety and environmental protection.

A Complete Solution

Often involved from the project conception onwards, Menard works closely with clients in analyzing their objectives. Based on the geotechnical investigation report and the description of the structures to be built, MENARD will recommend the most appropriate soil improvement technique, depending on the analysis of the soil, the structure and the environmental constraints. MENARD engineers will then produce a detailed design, and the subsequent construction will be carried out by dedicated teams based in more than 50 locations worldwide. Finally, our quality control procedures will allow us to complement our design and construct package with a long-term guarantee.

State of the Art Technology

Over the last 50 years, MENARD has improved and developed a complete range of innovative and sustainable soil improvement techniques. A fundamental objective of the company has always been to remain at the forefront of technological advances in this field and to offer more efficient, reliable and economical solutions. Our leading expertise is built on our field experience, our numerical modelling capabilities and the development and optimization of specialised construction equipment.

Siège Social :
2 rue gutenberg
91620 NOZAY
Tel : 01 69 01 37 38
Fax : 01 69 01 75 05

www.menard-web.com

First application of the Geomix® process as ground improvement in a seismic area

Fort de France prefectural headquarters/Martinique

The major earthquake suffered by Martinique in 2007 showed once more the urgency, for the prefectural authorities, to construct two new administrative buildings as a replacement for one older building. The call for tenders issued for the construction of the foundations sought two technical solutions: one, very traditional solution, using piles and barrettes; and the other, on the face of it less expensive, using ballasted columns and rigid inclusions. The objective, in both cases, was to transfer the load from the buildings onto "good" ground through the layer of alluvial sands, at a depth of 12 to 19m on the site.

"Although, obviously, there were no feasibility problems with the technique of piles and barrettes," explains Emmanuel Ollier, the head of Bachy Fondaco Caraïbes, "the envisaged ground improvement solution did not seem to us to be suitable, bearing in mind the great thickness and the high potential for liquefaction of the ground in Fort de France. In the event of an earthquake, at the final stage of deterioration, the ground actually 'flows', depending on the angle of inclination of the upper surface of the substrate."


A grid of Geomix® retaining walls will provide earthquake protection for the new buildings

In this case, the difference in level of the top of the good ground is substantial, as it is up to 7m over a length of only 40m. Rigid inclusions and/or ballasted columns would not therefore be strong enough to withstand the intensity of the forces created by an earthquake.

Not wishing to waste the opportunity of putting forward a cost-effective alternative to the basic solution, Bachy Fondaco Caraïbes proposed a grid of Geomix®* walls 0.50m thick under the whole building, on a layout of approximately 4.30m by 4m. This unusual alternative has a number of advantages: it blocks liquefaction of the ground in the event of an earthquake and, during the works phase, it causes less disturbance (vibrations) for neighbours. It also reduces to a minimum the need to use machinery on the site, as the process generates very little spoil and only requires a limited supply of cement and water.

Although it was not the least expensive bid submitted, Bachy Fondaco Caraïbes' tender was accepted as being the best for the construction of the foundations for the two buildings. One is a four-storey and the other a five-storey, with a total ground area of approximately 600m² each (i.e. a total of 1,115m²).

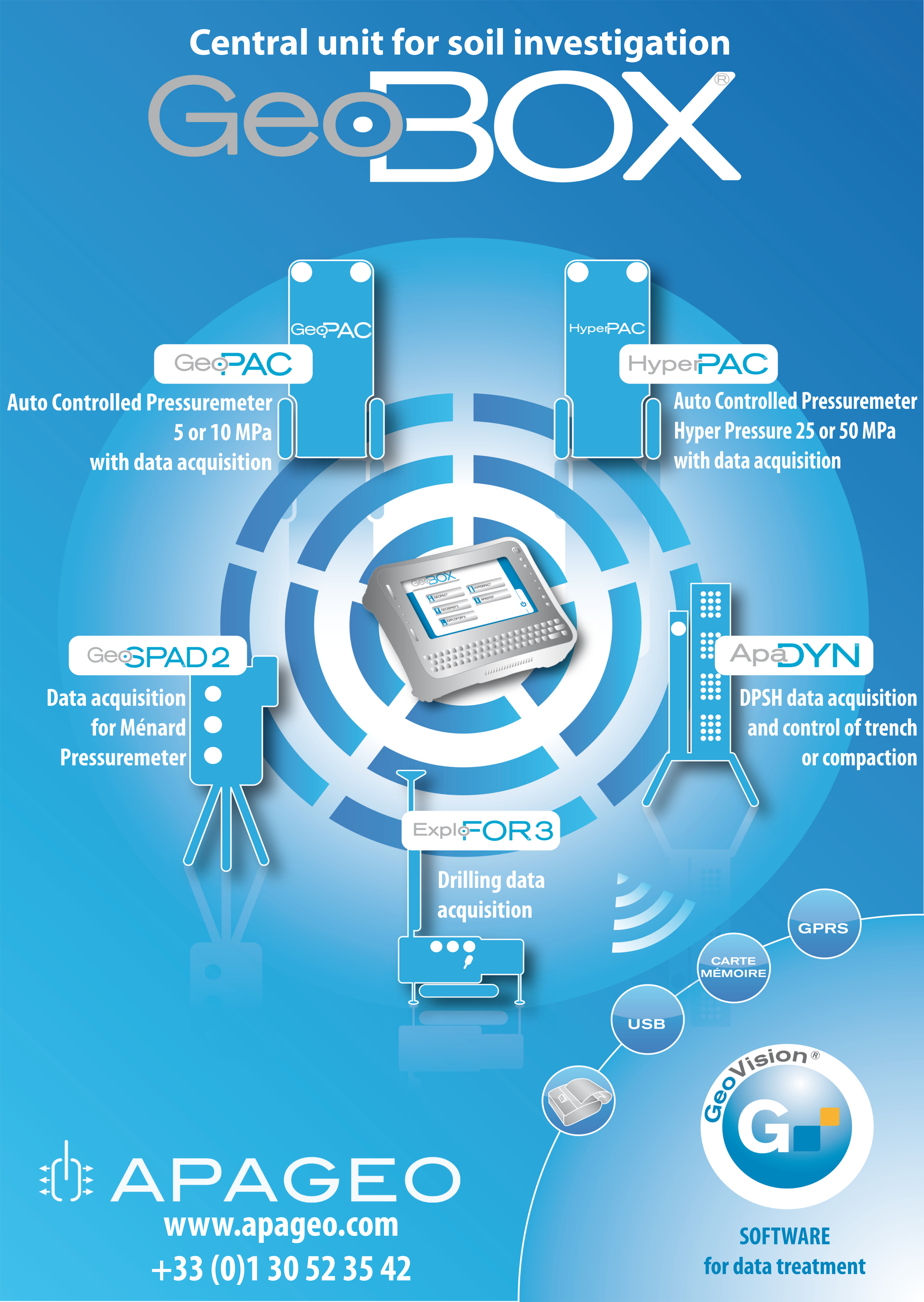
"This new technique, implemented over a very short period of time between mid-October 2010 and mid-January 2011, represents a big step forward in the approach to foundations in a seismic zone," considers Emmanuel Ollier. "If generalised, it could create a revolution for us, encouraging us to obtain the appropriate plant and develop this new expertise."

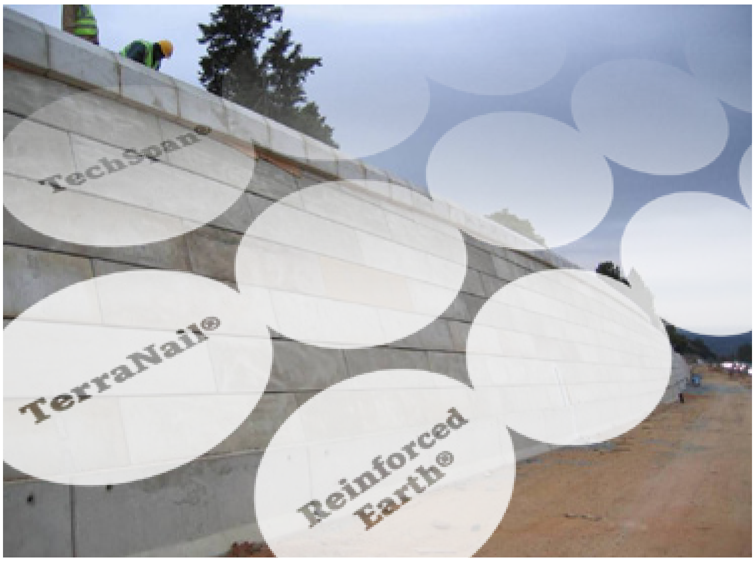
** Geomix®: a retaining wall and foundation process combining Hydrofraise trench cutting with deep soil mixing.*

Participants
Client: French Ministry of the Interior
Client representative: Direction Départementale de l'Équipement
Project manager: DHA joint venture (Arch'Îles Concept, CIEC Engineering)
Deep foundations: Bachy Fondaco Caraïbes

SOLETANCHE BACHY

GOLD SPONSORS


Central unit for soil investigation
GeoBOX®
GeoPAC
Auto Controlled Pressuremeter
5 or 10 MPa
with data acquisition
HyperPAC
Auto Controlled Pressuremeter
Hyper Pressure 25 or 50 MPa
with data acquisition
GeoSPAD2
Data acquisition
for Ménard
Pressuremeter
ApaDYN
DPSH data acquisition
and control of trench
or compaction
ExploFOR3
Drilling data
acquisition
GPRS
CARTE MÉMOIRE
USB
GeoVision®
SOFTWARE
for data treatment
APAGEO
www.apageo.com
+33 (0)1 30 52 35 42



TechSpan®
TerraNail®
Reinforced Earth®

SILVER SPONSORS

PLATINUM SPONSORS

ABEF

BESIX

Cofra

DEME
Dredging, Environmental & Marine Engineering

menard

SOLETANCHE BACHY

GOLD SPONSORS

APAGEO

DENYS

EIFFAGE

FUGRO

Holcim

LIEBHERR

Tensar

TERRASOL

TERRE ARMEE

CATHIE ASSOCIATES

SILVER SPONSORS

ISCHEBECK TITAN

Wicks

KELLER

ORGANIZING SECRETARIAT

BBRI

Belgian Building Research Institute

With the support of TIS-SFT

TIS
Speciale funderingstechnieken