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## **MINIMIZZAZIONE DEL MATERIALE DI REFLUO PER METODI DI MISCELAZIONE PROFONDA ATTRAVERSO L'OTTIMIZZAZIONE DEL MIX-DESIGN**

*Minimization of backflow production for deep mixing  
methods through mix design optimization*

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## INTRODUCTION

### 1.1. A brief introduction to deep soil mixing

The deep soil mixing method (DMM) is today one of the most used and technologically advanced methods for geotechnical engineering applications. It consists of an in-situ soil treatment technique based on the disintegration and mixing of the soil with binders in order to obtain a uniform and homogeneous structure with improved mechanical and hydraulic properties.

The deep mixing method originated in the late 60s and early 70s almost simultaneously in the northern European countries and in Japan. The growing demands of urban structure and infrastructure development and rehabilitation have created a very active and rapidly expanding market demand all over the world and consequently, a fast and continuous development of knowledge and innovation of the technique. Today, DMM is one of the world's most attractive and fastest growing specialty geotechnical construction processes (Bruce, D., Bruce, M., and DiMillio, A., 1998). The success of this method is because it offers various solutions in a wide range of applications; thanks to its versatility, it represents an alternative, more economic (e.g. Topolnicki and Pandrea, 2012), and environment-friendly solution with respect to the traditional methods involved in ground improvement.

The main applications concern the execution of hydraulic cut-off walls, excavation support walls, ground improvement, liquefaction mitigation, in-situ reinforcement and foundations and environmental remediation (Porbaha, 1998; Bruce, 2000).

### 1.2. Goal of the work

The aim of this study is to determine a rule on treatment of different types of soil with different deep mixing techniques in order to keep the levels of backflow material as low as possible.

The backflow material produced during a wet mixing process is due to partial replacement of the original soil with the binding slurry.

Over the years, this spoil material has become one of the most important factors that need to be taken into account while planning a deep mixing jobsite. The amount of backflow produced is usually very high and the costs for transportation, storage and disposal become extremely high and sometimes prohibitive, especially if the treatment involves contaminated

soils. For this reason, nowadays, companies involved in the realization of deep mixing structures aim to minimize or even eliminate the spoil.

Until now, the studies have been preliminary and not very detailed, but the need of development in this area is becoming fundamental. This experimental study, which deepens the knowledge about in-situ mixing processes and about mechanisms that govern the interaction between binders and soils, aims to be a first step towards optimizing the mixing procedures for improving the original ground properties at the lowest possible cost in terms of spoil management, time and binder consumption.

In particular, the objective is to find out a general rule to establish, for each specific DMM operation, the optimal mix design that minimizes the amount of backflow material, but at the same time, satisfies the prescribed requirements in terms of strength, permeability and working conditions.

### **1.3. Organization of the thesis**

This thesis is the result of an associated study of theoretical knowledge acquired from an accurate literature review and an experimental program developed and executed in the laboratory.

The first part focuses on the most important literary information used for developing the experimental program; Chapter 2 in particular shows the current state of the art in deep mixing methods with a detailed explanation of the construction procedures; the most important parameters to be taken into account are also highlighted. Chapter 3 presents a precise description regarding the main factors affecting the stabilization of treated soils. It is possible to identify four crucial variables: type of soil, type of binder, mix design and laboratory procedure. In Chapter 4, a real case study is introduced: the work of this thesis is born concurrently with the need to find answers and proposals for the development of the right approach for the complex jobsite of the Hong Kong airport. After a brief overview about the main features of this project, a detailed physical, chemical and geotechnical soil characterization is given.

The second part of the thesis reports the experimental studies and emphasizes the most relevant results obtained. Chapter 5 in particular illustrates the testing program with a detailed explanation of all the tests performed; these are subdivided in two main categories based on the features that need to be studied; a series of tests for “strength” and another one for “mixability”.

Graphs and tables show the results obtained from the experimental project in Chapter 6, whereupon considerations and future proposals can be formulated. Conclusion and suggestions for further research are presented in Chapter 7.

## IN-SITU STABILIZATION METHODS

From a legislative perspective, the topic “soil mixing” is discussed in the European standard EN 14679 (2005), where a “mixing process” is defined as “any procedure which involves mechanical disaggregation of the soil structure, dispersion of binders and filler in the soil”. The EN 14679 (2005) provides a checklist for the information needed for the execution of the work, geotechnical investigations etc. (Larsson, 2005).

### 2.1. Deep mixing methods

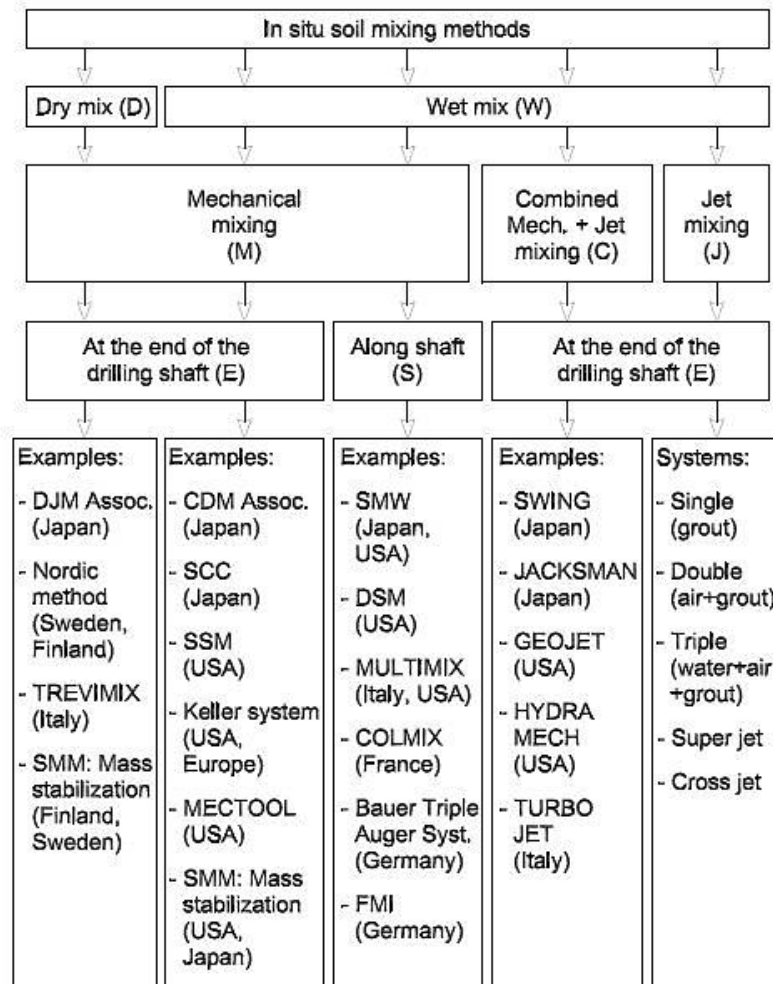
A first comparison that allows delineating the “deep mixing method” between all the mixing processes is the one with the shallow soil stabilization method: in fact, EN 14679 (2005) describes the Deep Mixing Method as “treatment of the soil to a minimum depth of 3 meters”, which involves rigs equipped with “rotating mechanical mixing tools, where the lateral support provided to the surrounding soil is not removed”. Traditionally, shallow soil stabilization is related to the shallow subgrade under *e.g.* road constructions, whereas deep mixing is related to the improvement of the whole soil deposit (Larsson, 2005).

Deep Mixing is a mechanical mixing technique where the binder is injected into the ground with relatively low pressure; mixing blades equipped to vertical mixing shafts disaggregate the soil and forcibly mix it with the binder. Thanks to the chemical reactions between the soil and the stabilizer, the treated soil or filled mass that results generally has a higher strength, lower compressibility and lower permeability than the virgin soil, although the exact properties obtained will reflect the characteristics of the native soil, of the construction techniques and the several variables that are selected (Bruce, Bruce and DiMillio, 1998; Yang et al. 1998).

A need for cataloguing was born with the rapid development of the deep mixing technique in the latest decades due to the advent of various methods founded on new technological solutions.

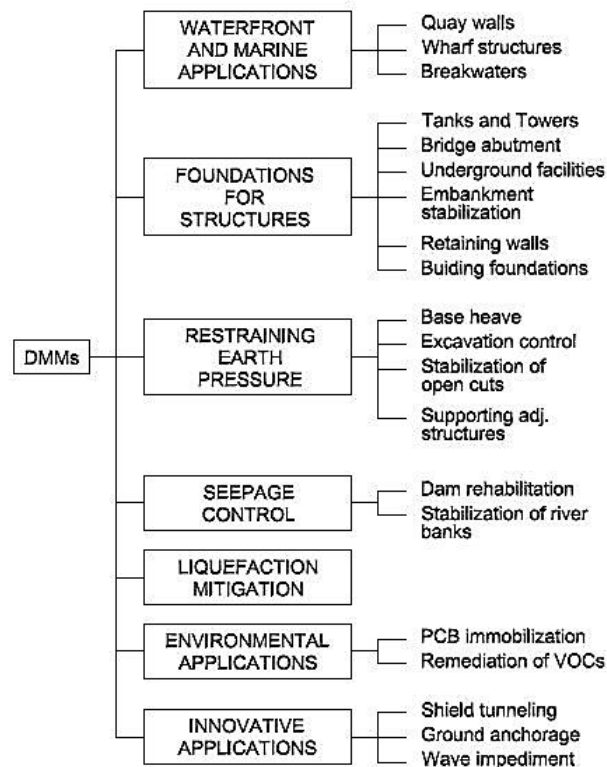


Nowadays, even if standardized classifications for deep mixing do not exist, several authors have proposed different classification systems based on different variables. The most relevant, reported in this thesis, is the one that considers different binder forms. In particular, this means that there are 2 principal ways to insert binder in the disaggregated soil: the wet mixing process and the dry mixing process (Figure 2.1).



**Figure 2.1:** General classification of deep mixing methods.

Another interesting classification is made by Porbaha et al. (1998) regarding the various possible applications of deep mixing methods to ground engineering works (Figure 2.2).



**Figure 2.2:** Classification of various applications of deep mixing technology (Porbaha et al., 1998).

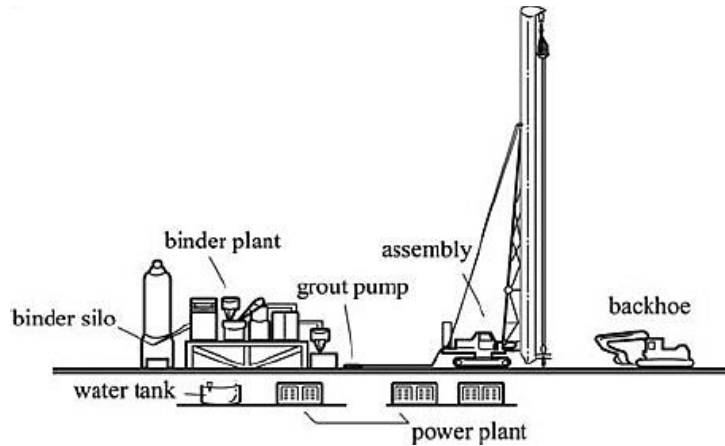
### 2.1.1. Wet deep mixing methods

The main feature of a wet mixing process is that the binder is injected into the soil through binder-water slurry. It's the most common technique used all over the world, thanks to the capability of adapting to many different combinations of variables that characterize a construction site. For this reason, a variety of deep mixing techniques and hence machines are developed by deep mixing contractors to meet the improvement requirements of each specific application.

In general, the equipment used for a deep mixing project consists of a DM machine and a binder plant. The machine is made of a mixing tool and a crawler crane with a leader that have different sizes and features depending on the base carrier, the maximum stabilization depth, etc. The mixing tool is made of a set of mixing shafts suspended along the leader. A motor and a gear box are installed on the top of the shafts. The shape and the number of mixing blades have been developed to ensure the highest possible mixing degree; these blades have various features, depending on the contractors. Binder slurry is supplied to the shafts by a pumping unit that delivers binder slurry from the mixing plant (Kitazume and Terashi; 2013), Figure 2.3.

The most commonly used binders in wet mixing treatments are cement and lime.

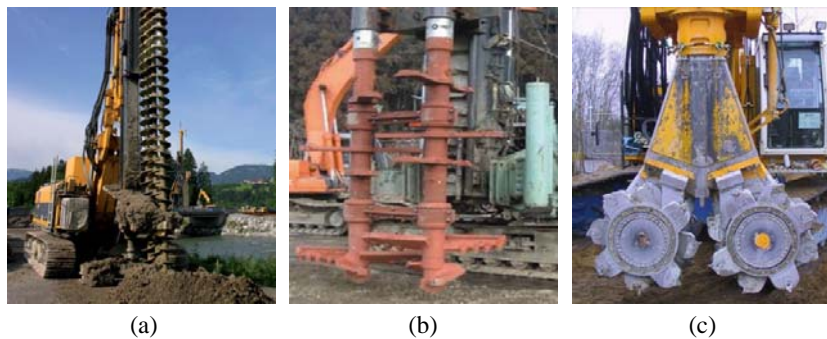
The deep mixing method was first developed in Japan in the 70s as a wet mixing technique named Cement Deep Mixing (CDM) and spread all over the world in the subsequent years.



**Figure 2.3:** Equipment for a wet mixing machine for on-land work (Kitazume and Terashi; 2013).

Today, various working machines are developed for this utilization worldwide, depending on the region.

In Europe, the installation of wet soil-mixed columns can be performed using single or multiple flight auger(s) (Figure 2.4a) or vertical shaft(s) equipped with blades, depending on the subsoil conditions and engineering applications. In Japan, wet mixing is used predominantly for marine constructions and land reclamation, even though several types of mixing rigs are available for on-land projects (Figure 2.4b).

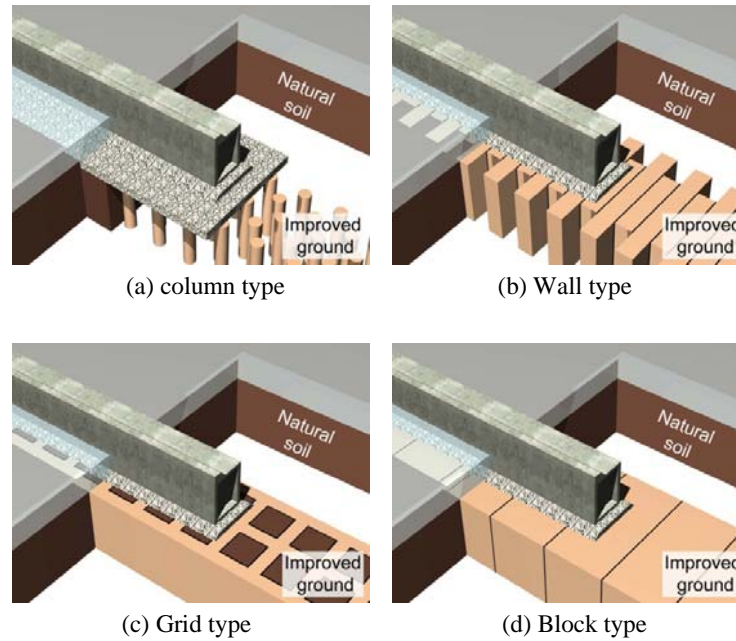


**Figure 2.4:** Wet mixing technologies: (a) Mixed-in-Place; (b) Standard CDM; (c) Innovative solution: CSM.

Mixing tools are normally equipped with fixed "anti-rotation vanes" in order to prevent the mixed material sticking to the driven blades and shaft from rotating.

Due to the liquefied state of the stabilized soil right after the treatment, steel bars, steel cages, and steel beams can be installed to obtain reinforced soil-wall structures.

Although it is the most common and least expensive application, columnar treatment it is not the only treatment that can be executed by this method; other different-shaped elements can also be produced: when the load derived from the superstructure is high or there are high deformation problems, the recommended foundation elements are the block type and the wall type; for liquefaction mitigation, a grid type treatment is often performed (Figure 2.5).



**Figure 2.5:** Possible geometries of the treated mass.

New developments are continuously proposed in the field of deep mixing applications; following are some examples.

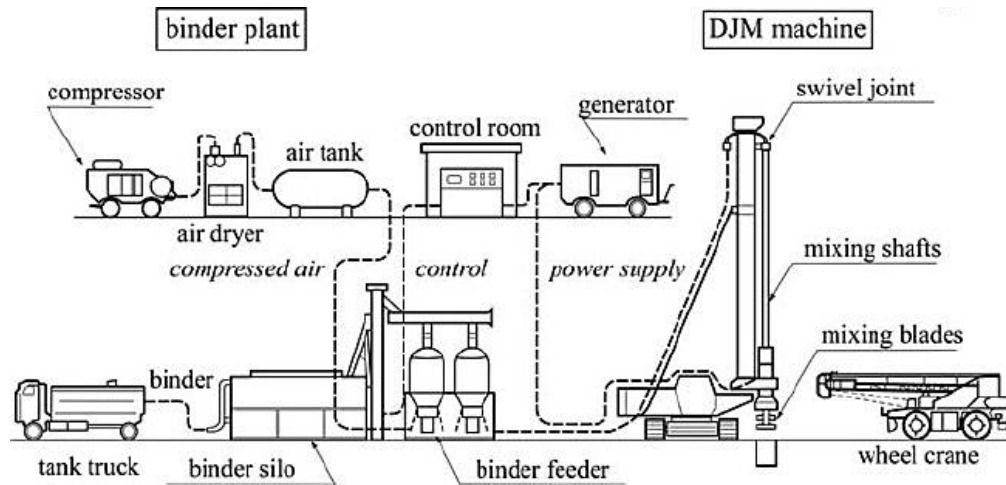
The *Colmix method*, developed by *Bachy* in the late 1980s, involves mixing the soil with a water-based or dry binder by means of a helical tool. The binder is injected as the tool penetrates the soil. Mixing and compaction take place as the tool is withdrawn. The *Trevimix* method was developed in Italy in the early 1980s and uses both dry and wet binders.

An innovative solution for rectangular mixed panels is Cutter Soil Mixing (CSM) (Fiorotto et al., 2005). Unlike the traditional wet DMMs, the CSM method uses two sets of cutting wheels rotating around a horizontal axis and driven by hydraulic motors located in a water-tight box assembled on a robust Kelly bar (Figure 2.4c) or on a wire suspended cutter frame (Bellato, 2013).

Another example is represented by the TRD (*Trench cutting Remixing Deep wall method*), which is carried out using a sort of chain-saw cutter device, ensuring the continuity of the mixed wall. Despite this advantage, the use of the TRD technique is inevitably restricted to applications requiring limited depths (Larsson, 2005).

### 2.1.2. Dry deep mixing methods

The dry deep mixing method is characterized by a different way of inserting binder into the disaggregated soil: cement or lime powder is injected in a dry state using compressed air. A mixing plant provided with a binder silo, binder feeders and an air-compressed tank is placed at the site in order to supply binder at the DJM machine (Kitazume and Terashi; 2013). Figure 2.6.



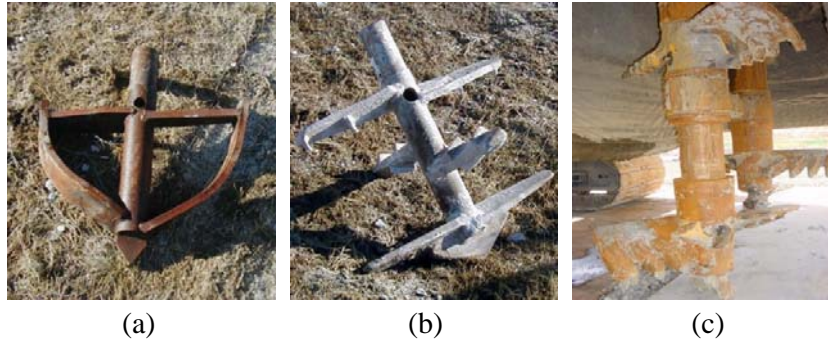
**Figure 2.6:** Equipment for DJM (Kitazume and Terashi; 2013).

The technique was first developed in the Scandinavian countries in the 1970s and was mostly used to increase the strength and reduce the compressibility of very soft clayey soils, for which a specific mixing tool in order to obtain high production capacity was developed. Originally, it is mounted on a single mixing shaft, but in further developments, for example in the Japanese technique, it was designed as a double shaft tool. The latest tools are developed studying the optimal way to inject and homogenize the powder into the soil and to improve the air collection after the treatment in order to reduce the ground displacement induced from this air jet as much as possible; for this reason, the main developments have involved the position of the holes for the outflow of the air-binder mixture. Recently, a new mixing tool where the binder is incorporated from the end of the mixing blades towards the mixing shaft has been presented (Takeda and Hioki, 2005). Another new similar technique, called Bidirectional Dry Mixing Method, was born in the last years: the modification of the drilling machine through the insertion of special transmission devices and equipment (Xie et al. 2012) has led to a better homogeneity with a consequent increase in bearing capacity of the mixed soil (Shenghua et al. 2011; Xie et al. 2012). Examples of current tools are reported in Figure 2.7.

This technique can only produce columnar treatments.

Is important to consider that since no water is added into the binder, the soft soil should possess a natural water content of at least 20% to ensure hydration reaction (Shenghua et al., 2012); sometimes, when this method is used to treat relatively dry and hard soils, water is added separately when the mixing tool is being inserted to facilitate the penetration and have a better disaggregation of the soil (Larsson, 2005).

Anyway, this method showed very limited advancements over the decades in comparison to the wet method.



**Figure 2.7:** Dry mixing tools: (a) Nordic "standard" tool; (b) Nordic dry mixing "Pinnborr"; (c) DDM standard.

Dry and wet mixing methods are based on similar mix designs and produce foundation elements used for the same objective; anyway, several comparisons based on the main features that characterize a soil treatment have been proposed to evaluate the two techniques and their best applicability field. Discussing the engineering features, such as the strength achieved, showed that higher values were reached in wet mixed columns than in dry mixed columns (Navin and Filz, 2005); On the other hand, Shenghua et al. (2011) saw a better effect of the DJM treatment on soft marine clays than the traditional Cement Deep Mixing wet method; furthermore, it was demonstrated that the columns produced with dry mixing had higher shear strength than those produced with the wet technique (Larsson and Kosche, 2005); The air injection can be a big problem in the final strength achievement in dry mixing treatments due to a complicated and less effective dispersion process of the binder and due to the possible formation of crater holes caused by bad evacuation of the air in some problematic soils (Larsson, 2005). Massarsch (2005) found that the hydraulic conductivity of dry mixed columns was higher than the permeability of the surrounding soil, while the use of the wet method gave contrary results. Other comparisons can be made from an environmental point of view: the dry mixing method, in fact, produces lower noise and lower vibrations on the surroundings and on the nearby structures with almost no production of spoil material than the wet mixing technique (Shenghua et al. 2012; Yang et al. 1998). The dry method also presents some economic advantages due to the simplicity of the technique: less binder injected, no spoil disposal, easier mixing plants and preparation of the site, etc... but has many restrictions on the applicability of the technique in general, such as the possibility of treating only a limited range of soils (soft soils with high water content) and a shallow depth of treatment (maximum 15 to 18 meters) due to the too great geostatic and water pressure (Lang et al., 1999).

However, before choosing the best method for a specific project, a careful analysis of the advantages and the disadvantages and specific field trial tests should be carried out.

## 2.2. Constructions procedures

The know-how about different construction procedures related to various techniques is very important in order to choose which one is the most suitable to reach the objective of the jobsite and to plan a working schedule.

First of all, with the aim to understand how the technique interacts with the soil concerned, it is very important to conduct some preliminary tests, in particular:

- *Laboratory tests*, to understand specifically the interaction between the soil and the binder, their mixability, and consequently, the behaviour of the treated soil in terms of strength and permeability;
- *Field trial tests*, in advance or adjacent to the construction site, to confirm the smooth execution of the treatment; all the characteristic machine parameters (like amount of binder injected, rotation speed of the mixing blades, penetration and withdrawal speed of the mixing shaft, electric and hydraulic power requested) are monitored to understand when a new soil layer is reached in order to adjust them consequently to its new features.

To plan a treatment operation and its related tests, is it also very important to know exactly how each deep mixing procedure works.

### **2.2.1. Wet mixing procedure**

After placing the machine in the right position, the mixing tool is penetrated into the ground while rotating the mixing shaft. Two basic procedures can be delineated, depending on the injection sequence of the binder:

- Injecting the binder *only during the withdrawal* of the mixing shaft;
- Injecting the binder *during the penetration and the withdrawal* of the mixing shaft;

During the penetration, the mixing blades rotate to disaggregate and disturb the soil to reduce its strength; in this way, mixing tools easily penetrate mostly by their self-weight; in the first case, in the descent phase, the water injection helps the mixing blades to disaggregate the soil and later, in the withdrawal phase, the rotation of the mixing blades is reversed and the binder slurry is injected and mixed with the soil again. In the other case, a percentage of binder slurry is directly injected during the penetration phase and the remaining amount, during the withdrawal. When the tool reaches the final layer (normally a stiff layer on which the whole foundation is based), it stays in that position or moves up and down for one meter over few minutes, continuing injection and mixing, with the aim to create an optimal contact of the structure with the base layer.

Normally, the flow rate of the slurry and the penetration and withdrawal speed are kept constant to assure that each layer is mixed with the designed amount of slurry. This is possible through a constant control of the pumping pressure (Kitazume and Terashi; 2013).

### 2.2.2. Dry mixing procedure

The procedure is basically very similar to the wet method. A slight difference is highlighted in identifying three stabilizer injection methods:

- Injecting the binder *only during the withdrawal* of the mixing shaft;
- Injecting the binder *only during the penetration* of the mixing shaft;
- Injecting the binder *during the penetration and the withdrawal* of the mixing shaft;

Of these methods, the one that gives the highest mixing efficiency is the second, when all of the binder is injected during penetration. This is because the entire amount is mixed for all the time the tool stays in the treated soil, and a better homogeneity and consequently a higher strength is achieved due to this. The disadvantage is that, if a large amount of binder is applied, the viscosity of soil increases a lot, consequently decreasing the mixing efficiency (Hayashi and Nishikawa, 1999). Anyway, the most common and less risky procedure is the first one: injecting the binder during the withdrawal phase, while the penetration injection is chosen when very sensitive soft soils have to be treated: they can occasionally cause difficulty in exhausting air. Furthermore, often, the binder is added during the initial mix but then the mixing equipment goes all the way down and back up one to three times in order to reach the best uniformity of the mix. (Liu and Hryciw, 2003).

The operative mechanism of the mixing equipment remains mostly identical to the wet procedure, even if the volume of reagent injected is smaller (Yang et al. 1998) and the machine parameters are a bit different.

The air injection is a feature that has to be monitored carefully: the minimum required air pressure is equal to the sum of the water pressure in the ground and the injection pressure at the outlet; this means that it has to increase with depth, but carefully, because exceeding a certain amount can cause the soil close to the mixing shaft to blow-out, creating big voids in the stabilized columns.

In order to understand if the properties of the built structures correspond to the design requirements of the project, a “*Quality Control/Quality Assurance*” program should be performed.

*Quality Control* consists in monitoring:

- The quality of the binder in terms of water to cement ratio and density in the binder plant before and during the treatment;
- The geometric layout of the column/wall during the production; this is possible thanks to the indications given from the machine’s sensors;
- All the other machine parameters like amount of binder injected, rotational speed, shaft’s speed etc. in order that each treated layer obtains the prescribed characteristics.



During the construction, the monitoring data is fed back real-time to the operators in a control room and to the rig operator in the cab of the machine.

*Quality Assurance* comes in after the production of the treated structure; the aim is to verify that the required quality is achieved. Geometrical features like continuity, uniformity, dimensions are checked through a coring procedure thanks to a visual observation on a full-depth core. Afterwards, the engineering properties can be determined through laboratory tests on samples selected from the continuous cores. In particular, the most common tests are the *Unconfined Compression Strength test* and the *permeability test*. The number of the core borings depends upon the amount of built elements in the project, on the contractor's conditions and on the national technical standards such as the number of laboratory tests. For each core boring, normally, three specimens are taken from three different levels to be submitted for the tests (Kitazume and Terashi; 2013). Even if the maximum strength in the stabilized soil element is reached after a long period (at least 28 days to achieve the prescribed level of permanent strength), the proper moment in which the assurance control for the engineering characteristics has to be done is quite soon after the installation, when the contract work is going on, so that the mix and other parameters can be changed if necessary. However, the early testing of the units requires a good knowledge of how their properties will develop over time. Also, in-situ tests can be performed for quality assurance, like Conventional Column Penetration Test, Reversed Column Penetration Test, Vane test, (Alxelsson and Rehnman, 1999.), Standard Penetration Test (Liu and Hryciw, 2003), but their significance is lower due to the not uncommon difficulties in properly penetrating the treated soils, especially the ones for which very high strength has to be achieved. Other quite significant and newly developed in-situ testing methods are the non-destructive tests like the Seismic methods and the Load Tests on the trial elements (Larsson, 2005).

### **2.3. Production parameters affecting the quality of soil mixed structures**

It is very important to know the parameters that govern the process for the production of a deep mixing element because it is through their optimum configuration that is possible to ensure the achievement of the design features. For this purpose, it is essential to know what they are, what their orders of magnitude and the typical values used are; in this way, it is possible to calibrate them before starting with and to adjust them during the production (for example, to ensure the continuity of the element through layers of soil with different characteristics).

The meaningful parameters are essentially the machine's mechanical variables like:

- The speed of penetration and withdrawal of the mixing shaft;
- The rotation speed of the mixing blades;
- Air pressure (Dry Mixing);

Normally, all these parameters are constantly monitored during the process through sensors and gauges placed on the machine, automated and controlled by computer systems; in this way, they can be easily set and adjusted in real-time.

Several authors proposed typical numbers that are used and should be used in construction sites, (Table 2.1). Wet mixing method and dry mixing method presents slightly different values.

<b>PRODUCTION PARAMETERS - Deep mixing methods</b>		
<b>Phase</b>	<b>Dry</b>	<b>Wet</b>
<b>Mixing shaft speed (m/min)</b>		
<i>Penetration</i>	0,7 - 2,0	0,7 - 1
<i>Withdrawal</i>	0,7 - 4,0	0,7 - 4
<b>Mixing blades rotation speed (rpm)</b>		
<i>Penetration</i>	24 - 48	20 - 25
<i>Withdrawal</i>	12 - 64	25 - 40
<b>Blade rotation number (N/m)</b>		
	274 - 284	350 - 360

**Table 2.1:** Recommended production parameters for deep mixing applications (Kitazume and Terashi; 2013, EuroSoilStab, 2002; Horpibulsuk et al., 2012; Hayashi and Nishikawa, 1999).

“*Blade rotation number*” is an important parameter: it is not a mechanical variable, but a number that describes the expected quality of the completed element.

During the years, the need to certify the mixing quality achieved was demonstrated through different indexes, but often with limitations in the applicability and large approximations. The blade rotation number or the “degree of mixing indicator” formulated by Yoshizawa (1996), however, overcomes various limitations and seems to be the best parameter for quality control during production. It is now assumed by Japan and European standards (EN 14679, 2005) on deep mixing method treatments as a common factor for the evaluation of treatment effectiveness.

The blade rotation number,  $B$ , represents the total number of rotations of the mixing blade during 1 meter of shaft movement after the stabilizer has been injected into the ground (Yoshizawa et al., 1996; Porbaha et al., 2001) and can be defined by:

$$B = \sum n_B \left( \frac{N_d}{V_d} + \frac{N_u}{V_u} \right)$$

Where:

- $B$  is the “blade rotation number” measured in (N/m);
- $\sum n_B$  is the total number of mixing blades;
- $N_d$  and  $N_u$  are the rotational speeds of the blades during penetration and withdrawal measured in (r.p.m);
- $V_d$  and  $V_u$  are the mixing blade penetration and withdrawal velocities, measured in (m/min).

For each technique, typical values that indicate the quality obtained, depending on the subsoil conditions, were defined. These are shown in table 2.1.

There are a few characteristics that lower the accuracy and limit the utility of the Blade rotation number: first of all, this factor is closely connected with the specific geometry of the selected mixing tool. Therefore, it becomes difficult to identify a general range of good quality levels; the applicability of the factor is limited to deep mixing machines that operate with mixing blades that rotates around a vertical axis (and not a horizontal one as in the case of CSM). If the mixing blade penetration or withdrawal velocity is very low, or if the machine has to stop during the procedure, the blade rotation number loses significance. Finally, changes in speed due to the machine adapting to crossing layers of soil with different characteristics are not taken into account (Bellato, 2013).

## **2.4. The problem of the backflow material**

As introduced in chapter 1.2, the spoil material produced during a wet mixing treatment, in both economical and disposal terms, is becoming a significant problem. It affects several components of a building project, from the project planning to the steps after completion of the entire structure. In Europe, the average cost for disposal of inert construction material and waste soil starts from 5€t (excluding shipping costs), rising considerably when the waste is contaminated.

The production of spoil can be seen as partial substitution of the soil with the binding slurry; the volume of spoil produced during construction varies with the construction technique used. Data on the spoil production of differing wet methods is not available, but the spoil production can normally be quite high: around 50% to 60% of the original soil volume or even up to 100% for clayey soils. The dry mixing method, on the contrary, creates almost no spoil (Larsson and Kosche, 2005; Shenghua et al., 2012). Considering the two different DM techniques, it can be observed that the parameter that makes the difference in terms of backflow is the water content. The more water inserted into the ground after the voids have been filled, the more of the original soil volume it will replace. The result is that the volume of mixed soil is larger than the volume of the original soil, and the additional quantity is pushed out i.e. it becomes the backflow. From this point of view, the most first solution that comes to mind for overcoming this problem is acting on the water content of the slurry injected.

Depending on the type of soil that has to be treated, the main task of the water inserted with the slurry is to guarantee a certain degree of mixability of the soil in order to reach, from the mechanical point of view, the optimal working conditions for the machine that assure a fluid movement for penetration and withdrawal; and from the geotechnical point of view, a certain degree of homogeneity during the mixing that allows the structure to reach the highest strength with a predetermined cement content.

Hence, the goal is to reduce the percentage of spoil material by lowering the water content, but still providing the optimal working conditions and a high level of homogeneity.

The research described in this thesis provides a solution for achieving this goal. Two main approaches are proposed:

- To reduce the water content to a minimum level and, after a very detailed study on the soil characteristics, optimally combining all other relevant components of the mix-design that affect its mixability and homogeneity;
- Alternatively, to reduce the water content but guaranteeing mixability and homogeneity through the use of additives.

While this kind of procedures acts on preventing the production of spoil material, other economic and environmental solutions have been proposed to solve this problem during the past years. The European Directive 2008/98/EC says: “the recovery of waste and the use of recovered materials should be encouraged in order to conserve natural resources”, more effort has to be put in "reducing the environmental impacts of waste generation and waste management, thereby strengthening the economic value of waste”. According to the latter, the recycling of the spoil material can indeed be optimized and sometimes even become convenient for reuse, for example, in the field of earth constructions (dikes, mechanically stabilized earth, slope profiling..), transportation engineering (aggregates and fillers for pavements’ basements), civil engineering (as aggregate in the production of mortar, in the production of fluid fill and for the preparation of prefabricated elements used for structural applications), there is a good demand for these kind of things due to the increasing cost of gravel and other natural resources used for those purposes.



## MAIN FACTORS AFFECTING THE STABILIZATION OF SOILS

### 3.1. General

Since that the quality assurance is reflected mainly in the engineering properties obtained in the completed structure, is important to analyse the most important parameters from which these properties are influenced. Kitazume and Terashi (2013), Yoshizawa et al. (1996) proposed a general classification of those in four main categories:

- I. *Characteristics of binder;*
- II. *Characteristics and conditions of soil;*
- III. *Mixing conditions;*
- IV. *Curing conditions;*

#### 3.1.1. *Characteristics of binder*

This feature strongly influences the strength parameter in a soil treated structure.

While planning a project, once is known the resistance value required, three main decisions have to be taken in order to achieve it in the field:

1. The type of binder to be used: the choice normally is between cement and lime. The first one is the most used but both have advantages and disadvantages depending on all the other features that characterize a jobsite, especially on the interaction with soil. This topic was studied extensively from the engineers during the decades and some interesting comments and general guidelines were delineated. This topic is deepened in paragraph 3.2.
2. Quality of binder: it can be considered principally from two points of view:
  - The *choice of the fineness* of the binder, normally the finest it is the highest quality is reached;
  - The *storage* of the binder: to have good quality in the applications, the binder has to be storage properly to avoid for example the start of the hydration.

3. Mixing water and additives: is well known that an increase in the quantity of binder injected gives a higher final strength, but this doesn't mean, for reaching a predetermined strength, that this has to be the only parameter to play with. Several authors like Liu et al. (2008), Horpibulsuk et al. (2011) in fact say that the most important parameter that has to be taken into account in deep mixing applications is the water to cement ratio. Water have an important role both on the feasibility of the work, going to guarantee a certain degree of mixability during the execution of the element which leads to optimize and facilitate the work of the machine, and even directly on the strength: the highest the water content, the lowest the strength gained. (Porbaha et al., 2000; Kitazume, 2005)

The use of additives is not very common in deep mixing treatments but they have been used in a few jobsites mainly with the role of fluidizers (Miura et al., 1987; Min, 1996). Nowadays, the research regarding additives in soil mixing is lacking and rough, but it should be deepened because additives probably represent, as for concrete, good alternative solutions with respect to the classical procedures.

### ***3.1.2. Characteristics and conditions of soil***

The characteristics and the condition of the soil have to be deeply studied while planning the mix design for a deep mixing application, because they heavily influence the interaction mechanisms with the binder. In particular the most relevant features that affect the final result are:

1. Physical, chemical, and mineralogical properties of soil, that will be analysed in detail in paragraph 3.2;
2. Organic content: the presence in the soil of contaminated or organic parts, especially those with a low pH value, often can have a bad influence on the final mechanical characteristics of the ground improved construction; in fact they act chemically, limiting the interaction of the binder with the soil (Bellato, 2013).
3. Water content: as already described in the prior chapters, water has a relevant role in the achievement of the final properties of the treated element; not only on the "mixability and homogeneity" point of view but also, for example, chemically, in particular is the factor that allows the hydration reactions responsible of the hardening of the soil-cement mixture.

### ***3.1.3. Mixing conditions***

The mixing conditions that concern the choice of the mix design, of the mixing tool, of the mixing time and of the mixing parameters are important characteristics that have to be taken into account both in a real deep mixing application and in a laboratory procedure. Their effect

cannot be seen directly but, as shown from several authors, is influencing massively the final result (e.g. Larsson, 2005).

The mixing parameters were already discussed in chapter 2.3, while a detailed analysis around the mixing tool and the mixing time will follow in paragraph 3.4.

#### **3.1.4. Curing conditions**

Curing conditions are important aspects that have to be monitored after a deep mixing treatment and even during a testing procedure in the lab. The principal parameters are: temperature, curing time, humidity and confining pressure.

Normally, except from the curing time, the other parameters are hardly measured in-situ, while in a laboratory procedure they can be controlled much more effectively, in fact they are illustrated specifically in paragraph 3.6.

All of these characteristics are relevant for real deep mixing applications and laboratory procedures; the two are mostly dependent from each other for several reasons and normally laboratory procedures are conducted:

- on samples of natural soil coming from the site, to do preliminary tests for better understanding the characteristics of the natural soil to be treated;
- on treated soil coming from the jobsite, to test the engineering properties achieved by the soil;
- on artificially reconstituted soil and soil-mixtures before the starting of a construction site, to understand and delineate which could be the most suitable mix design for a deep mixing application.

Since this thesis is based on an extensive program of laboratory tests conducted on artificially reconstituted soil and soil-mixtures, in the following chapters a highly detailed description of each parameter affecting fundamentally, not only the final characteristics of a real treatment in a jobsite, but also the mechanical and rheological properties of the laboratory treated soils, is presented.

## **3.2. Materials**

The main aspects regarding the most important materials that affect a deep mixing treatment are presented in a detailed description in the following paragraphs.

### **3.2.1. Soil**

Soil can generally be defined as a complex, heterogeneous, and polyphase material composed of solid particles with different dimensions (also known as solid skeleton) and of a system of voids or pores, filled with a fluid substance, gas or liquid, like air or water or a



mixture of the two. Soil constitutes the most superficial part of the earth's crust, and their physical and mechanical properties and their distribution in the ground, are strictly connected to their geological origins: basically they are the result of physical and chemical alteration of the rock, and of the subsequent transport, due to climatic and environmental factors. In geotechnical engineering, the nature of soil is described through two main properties: particle size and plasticity.

The soil is catalogued through its particle size composition, described in the granulometric curve, drawn for each soil: after the passing through a sieve or in an aerometer, the curve is editing a cumulative frequency diagram of the percentages by weight of the grains of a certain diameter. Five principal sorts of soil are identified, with increasing diameter:

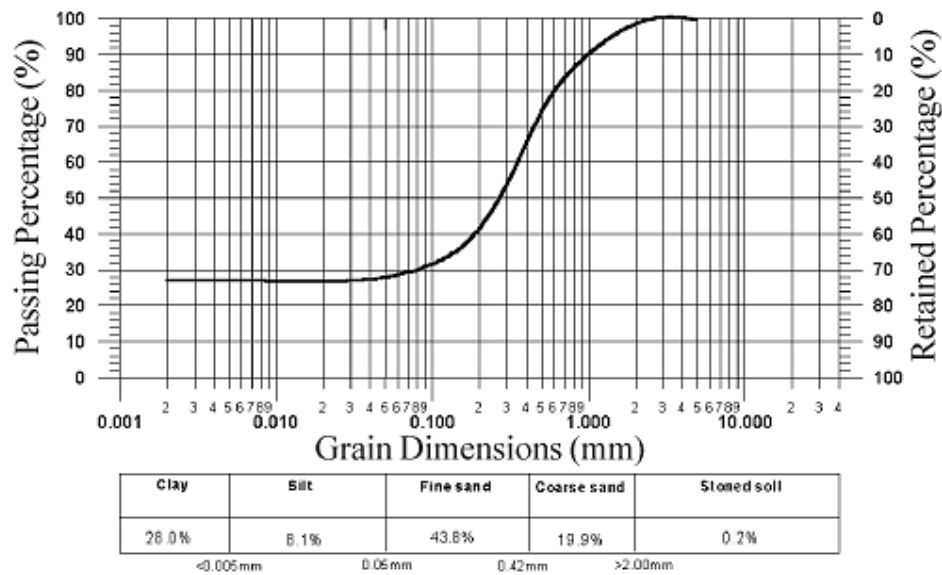
*Fine-grained soils:*

- *Clay* ( $d < 0,002$  mm);
- *Silt* ( $0,002 < d < 0,06$  mm);

*Coarse-grained soils:*

- *Sand* ( $0,06 < d < 2$  mm);
- *Gravel* ( $2 < d < 60$  mm);
- *Cobbles* ( $d > 60$  mm).

A typical curve is shown in Figure 3.1. If in a soil there are more parts, it is called with the name of the prevalent one followed by the others.



**Figure 3.1:** Granulometric curve of a soil mixture.

A second classification is needed for the fine-grained soils, in order to classify their mineralogical composition: this is made through the plasticity chart.

Plasticity is described empirically through the Atterberg limits that represent characteristic values of water content to which the soil changes behaviour and they are determined in laboratory analyses through standardized procedures. The plasticity chart is shown in Figure 3.2 and allows classifying different types of fine-grained soil based on where this is

positioned in. Soil belonging to the same place, typically is positioned along the line “A” (Viggiani, 2003).

The Atterberg limits, Liquid Limit ( $w_L$ ), Plastic Limit ( $w_P$ ), Plasticity index (PI), will be fundamental reference variables for all the experimental work carried out for this thesis.

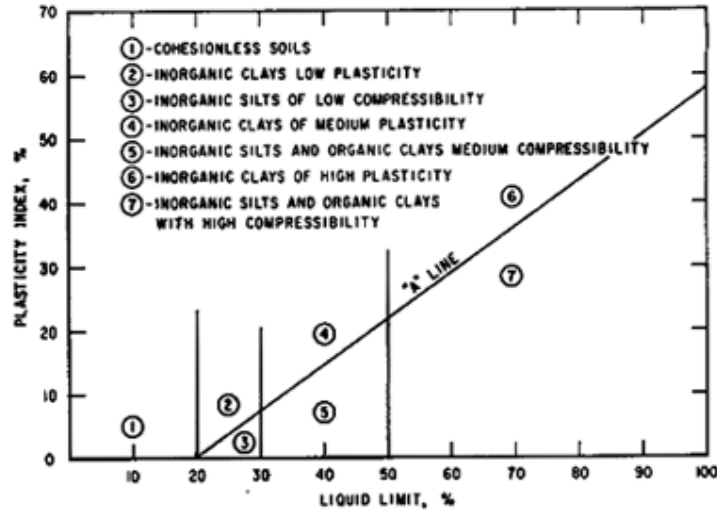


Figure 3.2: Casagrande's plasticity chart.

Another very important index used to describe the mechanical behaviour of the fine-grained soil is the consistency index; it is influenced by the value of the real water content with respect to the limits:

$$I_c = \frac{w_L - w}{I_p}$$

Consistency largely depends on soil minerals and the water content and provides a means of describing the degree and the kind of cohesion and adhesion between the soil particles as related to the resistance of the soil to deform or to rupture and hence to the workability of the soil. It is commonly described as soft, stiff or firm, and hard. The knowledge of the soil's consistency is important in defining or classifying a soil type or predicting soil performance when used as a construction material.

Following, a detailed analysis of the two main classes of soils is presented, according to Mitchell and Soga (2005).

### 3.2.1.1. Cohesive soils

Cohesive soils are mainly identified by clay minerals belonging to the phyllosilicates family, composed by a multi-layered structure that may contain silicate sheets of various types. They have a small particle size and their unit cell has a residual negative charge on the surface balanced by the absorption of cations from a solution in which they are immersed. The configuration of the most common layer silicates are made up of combinations of four simple structural units: the *silica tetrahedron* (silica tetrahedra arranged in hexagonal net) or the

*aluminium or magnesium octahedron* (sheet structure composed of magnesium and aluminium in octahedral coordination with oxygens and hydroxyls); *dioctahedral sheets* or *gibbsite sheets* (in which are present only trivalent cations, prevalently  $Al_{3+}$ ) and *trioctahedral sheets* or *brucite sheets* (in which are present only divalent cations, prevalently  $Mg_{2+}$ ).

Different clay mineral groups are characterized by the stacking arrangements of sheets (sometimes chains) of these units and the manner in which two successive two- or three-sheet layers are held together. Differences among clay mineral groups result primarily from differences in the type and amount of isomorphous substitution within the crystal structure. Possible substitutions are nearly endless in number, and the crystal layout arrangement may range from very poor to nearly perfect. The isomorphous substitution is the process through which some cations in the basic, ideal clay configuration are changed with other mineral particles without causing any change in the crystal structure. All the clay minerals are organized in particles having a net negative charge due to isomorphous substitutions. This charge imbalance provokes the attraction of exchangeable cations that are held between the layers and on the surfaces and edges of the particles.

In most clay minerals, the silica tetrahedral is interconnected in a sheet system. Three of the four oxygens in each tetrahedron are shared to form a hexagonal net. The fabric has the composition  $(Si_4O_{10})^{4-}$  and can repeat indefinitely. The octahedral sheet shape is composed of magnesium or aluminium in octahedral coordination with oxygens or hydroxyls. In some cases, other cations are present in place of  $Al_3$  and  $Mg_2$ , such as  $Fe_2$ ,  $Fe_3$ ,  $Mn_2$ ,  $Ti_4$ ,  $Ni_2$ ,  $Cr_3$ , and  $Li$ .

A single plane of atoms that are common to both tetrahedral and octahedral sheets forms a part of the clay mineral layers. Bonding between these sheets is of the primary valence type and is very strong. However, these connections holding the layers together can be of several types, some sufficiently weak that the physical and chemical behaviour of the clay is influenced by the response of these bonds to changes in environmental conditions. Principally five types of interlayer bonding for silicates are identified:

- *Van der Waals forces;*
- *Hydrogen and van der Waals bondings;*
- *Hydrogen bondings;*
- *Cationic bondings;*
- *Polar bondings.*

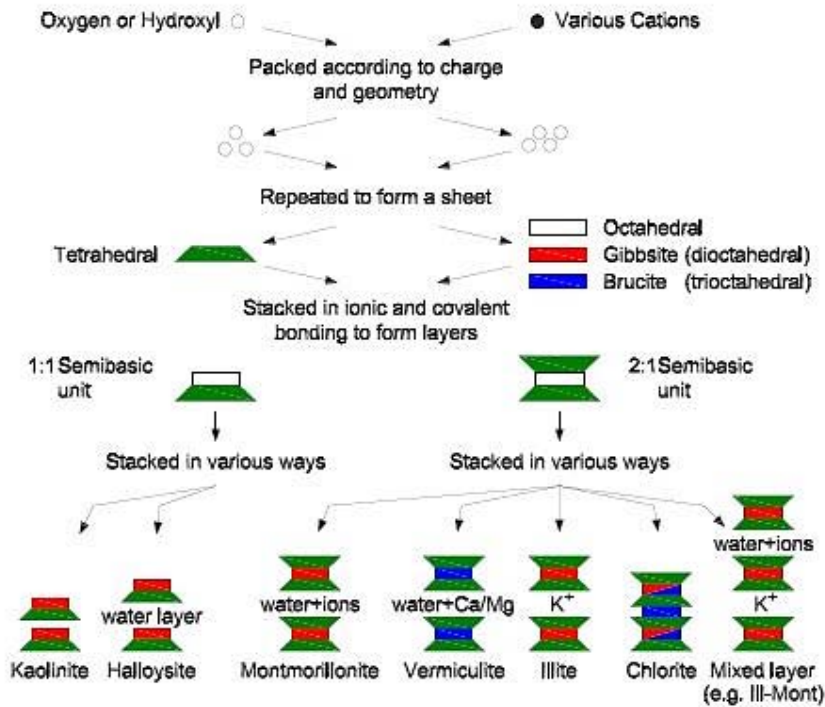
The way in which the various atoms are assembled to form different basic types of clay are shown in Figure 3.3. Clay minerals are grouped according to their crystal structure and stacking sequence.

This is appropriate since members of the same group show similar engineering characteristics. Knowledge on the mineralogical and chemical properties of the clays is fundamental, since crucial changes that are going to influence the mechanical and physical behaviour of soil and consequently of a soil-binder mixture, take place from this microscopic level.

The three most common types of clay studied for this thesis are described below.

**Illite**

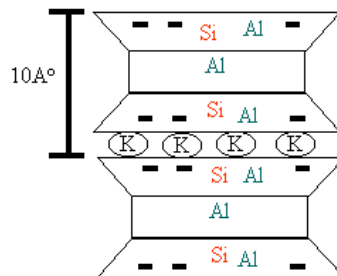
Is a very common mineral in nature and one of the most common encountered in the engineering practice. The basic structure is made of octahedral layers sandwiched between two silica sheets, but the substitution of about one-fourth of the silicon positions in favour of aluminium produces a charge deficiency balanced by potassium between the units (Figure 3.4).



**Figure 3.3:** Simplified pattern for predominant mineral clays (modified from Mitchell and Soga 2005).

The presence of non-exchangeable potassium atoms between the layers leads to a very strong interlayer bonding.

There is extensive isomorphous substitution in illite.



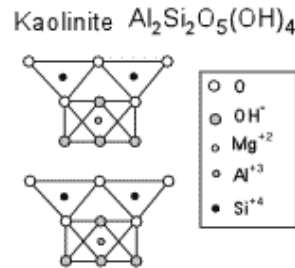
**Figure 3.4:** Detail of the basic structure of illite.

### ***Kaolinite***

Kaolinite mineral is composed of alternating silica and dioctahedral (gibbsite) sheets, which share the same interplane of atoms (Figure 3.5).

The bonding between two consecutive layers may be both van der Waals forces and hydrogen bonds, which are sufficiently strong to prevent interlayer swelling when clay particles come in contact with water.

Whether or not measurable isomorphous substitution exists within the structure of the kaolinite minerals is uncertain.



**Figure 3.5:** Detail of the basic structure of kaolinite.

### ***Smectite***

The minerals of the smectite group have a structure characterized by an octahedral layer sheet sandwiched between two silica sheets with whom is shearing the oxygen atoms on their tips.

Bonding between following layers is by Van der Waals forces and by cations that balance charge deficiencies in the structure. These bonds are weak and easily separated by cleavage or adsorption of water or other polar liquids.

An extensive isomorphous substitution takes place in smectite crystals in particular for silicon and aluminium. In the octahedral sheet, aluminium for example can be replaced by magnesium, iron, zinc, lithium and other cations and can replace up to 15% of the silicon ions in the tetrahedral sheet.

A particular and common type of smectite coming from isomorphous substitution is *montmorillonite*. There is a particular type of montmorillonite very used in civil-ground engineering for a wide range of applications: *bentonite*. It is well known for its thixotropic properties and high expansive behaviour when exposed to water. It is a high colloidal, expansive alteration product of volcanic ash and can have liquid limit even up to more than 500%.

Several other types of clay can be identified in nature, some as variations of the types described above and other as completely new types, *e. g.* chlorite minerals, chain structures clay materials, non-crystalline clay minerals and mixed-layer clays. (Mitchell and Soga, 2005)

Since clays, unlike other materials, are (chemically) very complex, it was necessary to describe them from a microscopic point of view to be able to fully understand how they can influence the mechanism of interaction with the binder and then the final properties of the treated element. But now it is equally important to consider their influence from a geotechnical point of view.

Different group of clay minerals exhibit different ranges of engineering properties. Within any one group, the range of property values may also be great. This is a function of their chemical and mineralogical properties such as particle size, degree of crystallinity, type of adsorbed cations, pH, the presence of organic matter, etc.

The first main engineering property that characterizes a clay is the plasticity, influenced by mineralogical constitution and described through the Atterberg limits. The following table 3.1 reports the values of the three kinds of soils studied and used in the laboratory for this thesis. It is worth to notice that the value of the limits vary consistently between different clay types and according to the Casagrande plasticity chart (Figure 3.2) it can be said that while illite and kaolinite are clays of medium/medium-high plasticity, bentonite is a very highly plastic clay: for this reason it presents also a very different engineering behavior, as shown in the next chapters.

<b>ATTERBERG LIMITS - natural soil</b>			
<b>Limits</b>	<b>ILLITE</b>	<b>KAOLINITE</b>	<b>BENTONITE</b>
$w_L$	<b>54,9%</b>	<b>48,2%</b>	<b>160,9%</b>
$w_P$	<b>41,7%</b>	<b>34,8%</b>	<b>57,7%</b>
IP	<b>13,1%</b>	<b>13,4%</b>	<b>103,2%</b>

**Table 3.1:** Typical values for the Atterberg limits of clays used in the testing program.

Even particle size and particle shape are features influencing the engineering behavior, normally particles of kaolinite are relatively large, thick, and stiff, smectites are composed of small, very thin, and filmy particles, illites are intermediate between kaolinite and smectite and are often terraced and thin at the edges.

These parameters, associated with the mineralogical composition, the void ratio, fabric and the pore fluid characteristics, influence the hydraulic conductivity and the strength of the soil. In particular, regarding the permeability, the usual measured range for natural clay soils is about  $1 \times 10^{-8}$  to  $1 \times 10^{-10}$  m/s.

For clay minerals compared at the same water content, the hydraulic conductivities are in the order montmorillonite < illite < kaolinite.

Regarding the strength, it can be said that the latter is given from the sum of two parts: a component of cohesion that depends on void ratio (water content), and a frictional contribution, dependent on normal effective stress. A number of studies (Mitchell and Soga, 2005) highlighted that an increasing cohesion and a decreasing friction was shown with the increasing of the plasticity of the clay.

### **3.2.1.2. Granular soils**

Gravel, sand and non-plastic silts are identified as cohesionless soils. Their physical characteristics are determined primarily by particle size, shape, surface texture, and size distribution. The mineral composition affects hardness, cleavage, and resistance to physical and chemical breakdown.

The gravel, sand, and most of the silt fraction in a soil are composed of bulky, non-clay particles. As most soils, they are the products of the breakdown and weathering of preexisting rocks (mostly igneous) and soils.

The prevalent minerals are feldspars (about 60 %), pyroxenes, amphiboles (about 17 %) quartz accounts for about 12 % of these rocks, micas for 4 %, and other minerals for about 8 %. However, in most soils, quartz is the most copious mineral, with lower amounts of feldspar and mica present. Pyroxenes and amphiboles are seldom found in significant amounts.

The abundance of quartz mineral in the soil is due to his highly stable structure: it is composed by silica tetrahedra grouped to form spirals which lead to the absence of cleavage planes, furthermore there are no weakly bonds in the fabric. Feldspars have an open structure with low bond strengths between units caused by the incorporation of large cations in the tridimensional framework structure, this leads to cleavage planes and moderate hardness. Micas have structures formed by octahedral and tetrahedral staked sheets held together by potassium ions that provide moderate strength electrostatic bonds. These mineralogical features are responsible for the lack of abundance of feldspar and micas in soils and for the high compressibility/swelling behavior of sands and silts containing only few percent of mica.

In many cases, however, the non-clay particles are treated as relatively inert, their interactions that are predominantly physical in nature.

Regarding the engineering characteristic of cohesionless soils, their mechanical behavior is governed mainly by their structure and the applied effective stresses. Structure depends mostly on the arrangement of particles, particle sizes, shapes, strength and distributions, then on density, anisotropy, arrangement of grains and grain contacts comprise the soil fabric.

### **3.2.2. Binders**

In this paragraph, are presented the chemical and the production aspects of the most used hydraulic binders in deep mixing treatments and their interaction mechanisms with soil.

#### **3.2.2.1. Cement**

Ordinary Portland Cement (OPC) is the most common hydraulic binder used in civil engineering applications and hence in the deep mixing field.

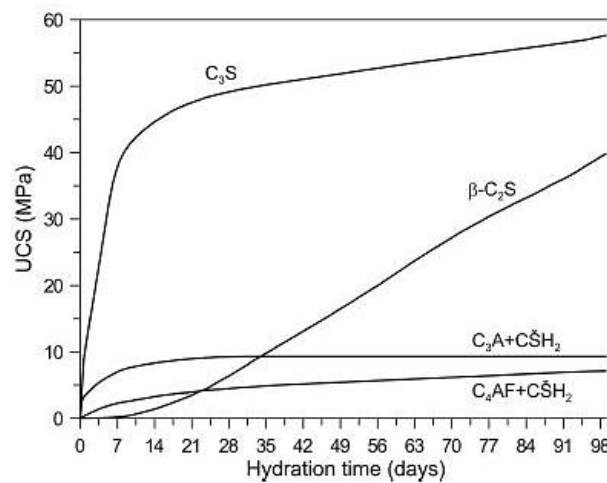
According to the norm EN 197-1 (2011), cement is defined as a finely ground inorganic material which, when mixed with water, forms a paste which sets and hardens by means of hydration reactions and processes and which, after hardening, retains its strength and stability even under water. When combined with aggregates and water, concrete and mortars which have to achieve specified strength level and volume stability are produced.

Cements consist of different materials, statistically homogeneous in composition, resulting from quality assured production and material handling processes. The main constituent of the Portland cement is the Portland cement clinker, defined from the norm as a “*hydraulic material which shall consist of at least two-thirds by mass of calcium silicates ( $3CaO\_SiO_2$  and  $2CaO\_SiO_2$ ), the remainder consisting of aluminium and iron containing clinker phases*”

and other compounds. The ratio of  $\text{CaO}$  to  $\text{SiO}_2$  shall not be less than 2.0. The magnesium oxide content ( $\text{MgO}$ ) shall not exceed 5.0% by mass.”

Hydraulic hardening of cement is a very complex chemical procedure, primarily due to the hydration of calcium silicates; other chemical compounds like aluminium may also participate in the hardening processes. All the chemical reactions involved in the hydration process are elaborate to be described and an easier two-phase “physical” proceeding is proposed (Bellato, 2013): the “setting” process consist on a change from concentrated suspension of flocculated particles to a visco-elastic skeletal solid capable of supporting limited stress. It is monitored by rheological measurements and typically occurs within a few hours. The “hardening” phase involves the continuous formation of the solid framework by means of physico-chemical processes, and leads to the development of the final mechanical strength of cement paste. The hydration is very fast in a first moment but to end is very slow and can go ahead for years until one of the two reagents is completely consumed or the space to deposit new hydration products is totally filled.

It is worth to understand the phenomena of cement hydration also from a chemical point of view. In the anhydrous state, four main types of minerals are normally present: alite, belite, (silicate phases), aluminate ( $\text{C}_3\text{A}$ ) and a ferrite phase ( $\text{C}_4\text{AF}$ ). When the cement comes in contact with water, the exothermic reaction that occurs, takes to the formation of the hydration products, especially, the hydration of clinker silicates (alite and belite) produces two main phases, calcium hydroxide (also known as portlandite,  $\text{CH}$ ) and a nearly amorphous calcium silicate hydrate ( $\text{C-S-H}$ ) having the aspect of a rigid gel. The hydration of the aluminate and ferrite phase give birth to the  $\text{AFm}$  and  $\text{AFt}$  phases: One of the most common  $\text{AFm}$  phases is monosulfate and the most common  $\text{AFt}$  phase is ettringite. While  $\text{CSH}$  products provide the most important contribution to the strength gain of clinker cement at short curing times, aluminate and ferrite phases do not contribute notably to the early strength development and, due to their high reactivity, cause an undesired rapid setting of cement accompanied by strong exothermic reactions (Figure 3.6).



**Figure 3.6:** Contribution to the strength development of cement pastes due to the most important clinker phases.

The hydration of ordinary Portland cements has been historically divided into four or five stages based on the experimental results obtained from calorimetric curves (Figure 3.7)



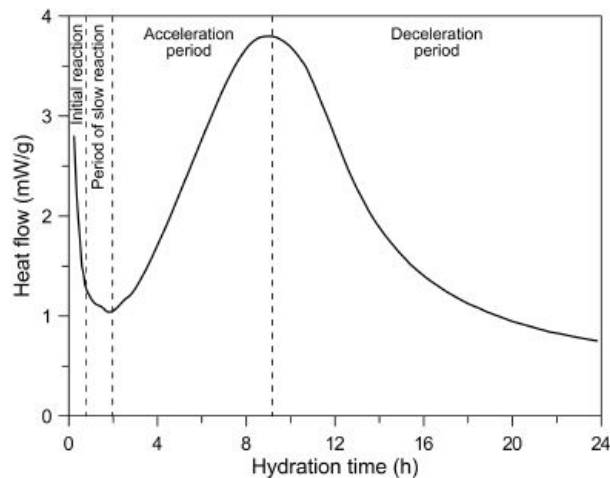
which, generally, confirm that the process is mainly controlled by the reactions of silicate phases (Gartner et al. 2002).

1 - *Initial reaction*: first minutes, corresponding to wetting and mixing. A rapid dissolution of free lime, sulphates, and aluminates occurs with an immediate formation of AFt phases (initially as a gel layer), while C3S grains start to hydrate on their surface. A large initial burst of heat is produced mainly due to the dissolution of C3A and C4AF and, secondarily, of C3S and CaO.

2. *Period of slow reaction*: often referred to as "dormant" or "induction period". A gel made of C-S-H nucleates and the concentrations of Si and Al in the pore solution decrease to very low levels. The nucleation of portlandite also takes place. During this stage a low rate of heat evolution is measured. Slow formation of early C-S-H and more AFt phases leads to an increase in viscosity.

3. *Acceleration period*: corresponding to the setting and early hardening. The hydration of C3S to form C-S-H and CH accelerates and reaches a maximum with an increasingly heat flow. The rapid growth of hydrates creates a reduction in porosity and the solidification of the paste (early strength development).

4. *Deceleration period*: The rate of production of C-S-H and CH from the silica phases decreases. A renewed hydration of aluminates generates AFm phases and AFt may dissolve or re-crystallize. The continuous reduction in porosity due to the growth of hydration phases yields to a gradual enhance in strength. The rate of reaction progressively slows down but hydration can last for years, as long as reagents and a sufficient pore space for the deposition of reaction products are available.



**Figure 3.7:** Isothermal calorimetry curve of the rate of alite hydration as a function of time.

Other inorganic additions, such as waste materials or industrial products can be ground together with the clinker and take part in the hydration reactions, the addition of

Supplementary Cementitious Materials (SCMs) is convenient not only from an environmental and cost-effective point of views, but also allows to manufacture binder characterized by specific properties necessary for particular purposes: for example composite cements can be produced for slowing and decreasing heat evolution, improving durability, or highering strength, as well as for ground improvement applications.

In accordance with EN 197-1 (2011), cements are subdivided in five main classes depending on their composition:

- **CEM I: ORDINARY PORTLAND CEMENT**; is composed of clinker and up to 5% of minor additional constituents.
- **CEM II: PORTLAND-COMPOSITE CEMENT**; it has a clinker content of at least 65%. The remaining constituents determine several subclasses:
  - *Portland slag cement (S)*;
  - *Portland silica fume cement (D)*;
  - *Portland pozzolana cement*: two types of pozzolanas can be mixed with the Portland clinker, i.e. natural (P) or calcined (Q);
  - *Portland fly ash cement*: two types of fly ash can be mixed with the Portland clinker, i.e. siliceous (V) or calcareous (W);
  - *Portland burnt shale cement (T)*;
  - *Portland limestone cement (L,LL)*;
  - *Portland composite cement (M)*.

Secondary constituents, such as fillers, are allowed up to 5% by mass.

- **CEM III: BLASTFURNACE CEMENT**; composed of clinker and higher percentages of blastfurnace slag, ranging between 36-65% (III/A), 66-80% (III/B), and 81-95% (III/C).
- **CEM IV: POZZOLANIC CEMENT**; in which up to 55% is composed of pozzolanic constituents (silica fume, pozzolanas, or fly ash). Two designations are provided: IV/A (11-35% of mineral additions) and IV/B (36-55% of mineral additions).
- **CEM V: COMPOSITE CEMENT** obtained from mixing Portland clinker (20-64%) with blastfurnace slag (18-50%) and pozzolanas (18-50%).

### 3.2.2.2. Lime

The original rocks, from which lime for engineering applications is derived, are typically limestone or chalk, prevalently composed of calcium carbonate (calcite –  $\text{CaCO}_3$ ). The limestone extracted from quarries or mines may be cut, crushed or pulverized and chemically altered. Part of this material is then selected accordingly to its chemical composition and granulometry and subjected to calcination (heating) at temperatures somewhat over 1000 C°. The procedure converts the natural stone into highly caustic material named "quicklime" or "burnt lime" (calcium oxide –  $\text{CaO}$ ).

Important properties of burnt lime that can affect its effectiveness as binder are the degree of calcination and the particle size distribution. The degree of calcination depends on the amount of CO<sub>2</sub> removed from the limestone during the heating process. The reactivity of CaO is strongly influenced by its particle size distribution, with finer-grained particle reacting more quickly with water. A typical lime composition, expressed in term of oxides, appears as follows: 94% total CaO (90% available for hydration reactions), 1.5% SiO<sub>2</sub>, 0.8% Al<sub>2</sub>O<sub>3</sub>, 0.4% Fe<sub>2</sub>O<sub>3</sub>, 1.7% MnO and 1.6% of other components (mainly K<sub>2</sub>O and Na<sub>2</sub>O).

Quicklime is most frequently used for lime stabilization in Europe, while slaked lime is more often used in the United States (Bell, 1988). Both can be used in combination with mineral additions to serve specific applications.

Hydration of quicklime is denoted as a strong exothermic reaction which results in the production of slaked lime. Slaked lime alone does not develop any strengthening mechanism and, therefore, no setting and no hardening are expected. A significant increase in strength and the formation of the typical cementing products take place only when lime comes in contact with pozzolanic materials (e.g. PFA, natural pozzolanas, etc.) or in the case of ion exchange. Mineral dissolution, which strongly depends on the pH value of the pore solution containing soluble calcium, occurs initially at the edges of the crystalline structure, producing soluble aluminium and silicon. Thus, the soluble calcium is consumed by the reaction with the newly dissolved ions to give cementitious materials. Slaked lime, if exposed to air, reacts with carbon dioxide to reform calcium carbonate. This process, named "carbonation", produces a strength gain which is independent of pozzolanic reactions and which is typical of mortars (Janz and Johansson, 2002). Because of its properties, lime can be considered an effective binder for stabilization/solidification treatments of soils partly composed of minerals having pozzolanic properties, e.g. active clays. No useful purposes can be served in granular soils or in those containing small amount of cohesive materials.

Silica and calcium carbonate in the form of quartz and calcite, in fact, are much less reactive than silicate minerals due to their more crystalline structure. In these types of soils, therefore, an addition of supplementary cementitious materials is required (Rajasekaran and Narasimha Rao, 2002).

Lime reacts with all clay groups, even if much rapid reactions involve minerals with higher proportions of available silica surfaces. Due to its mineral structure, the greater the surface area, the higher cation exchange capacity: montmorillonitic clays are expected to be more reactive than illite and chlorite, which, in turn, react quicker than kaolinite (Boardman et al., 2001).

The formation of hydration products last until water is present and pH value is sufficiently high. Generally, four basic physico-chemical mechanisms are regarded as responsible for the modification of soil-lime mixtures (Bell, 1988; Prusinski and Bhattacharja, 1999):

1. *Cation exchange;*
2. *Flocculation/agglomeration;*
3. *Carbonation;*
4. *Pozzolanic reactions.*

The first two reactions, along with the drying action of lime slaking (Rogers et al., 1997), lead to an immediate improvement in soil plasticity, workability, uncured strength, and load

deformation properties (Locat et al., 1990; Bell, 1996). The third reaction forms weak cementing agents and its role in changing the soil properties is still not clear (Sivapullaiah, 2000).

Finally, pozzolanic reactions yield to the gradual formation of cementitious hydrated calcium silicate of variable composition and to the corresponding long-term increase in strength and improved deformation behavior of treated soils.

It is worth noticing that a certain percentage of quicklime is required to saturate the pore solution of calcium ions, necessary for the satisfaction of the ion exchange affinity of clay particles. Cations are therefore adsorbed by clay minerals until this affinity is achieved and, consequently, they are not available for other reactions.

Due to this sensitivity to atmospheric moisture and carbon dioxide, special care must be taken during the transport and storage of lime. In this respect, Rogers et al. (1997) showed that the freshness of quicklime is an important factor affecting the performance of treated soil.

### 3.2.2.3. *Supplementary Cementitious Materials (SCMs)*

As already mentioned in chapter 3.2.2.1, SCMs as inorganic products, waste materials or industrial products, are widely used in the manufacture of various cements, for their environmental and cost-effective advantages and for their influencing properties. It is therefore very important to understand how they are produced and then how they interact with the binder material in the engineering applications.

#### ➤ *Blastfurnace slag (GGBF)*

Is a by-product of the iron industry formed from rapid cooling of the liquid at 1350-1550 C° composed of a combination of the siliceous constituents of the iron ore with the limestone flux used for smelting iron (Sherwood, 1993). A rapid cooling of it, below 800 C°, gives a wet, sandy, vitreous material (often containing over 95% of glass), which is a highly reactive latent hydraulic binder with compositions (depending on that of the ore) broadly intermediate between pozzolanic materials and Portland cement. For these reasons, ground granulated blastfurnace slag (GGBF slag) is generally used as mineral addition in combination with other binders.

When combined with ordinary Portland cement, slag reacts considerably slower than alite, and strength gain is therefore slower, especially when high proportions of slag are used. Generally, if compared with pure OPC pastes, lower strengths are achieved after 28 curing days in slag cements, but an increase in strength can be observed in longer times.

The addition of GGBF slag may be useful to obtain a better durability of cement-based structures (e.g. slag inhibits swelling in clays containing sulphates), but a slower formation of early-age hydration products is expected when higher amounts are used (Jegandan et al., 2010). Moreover, GGBF slag is very effective for the treatment of contaminated soils as it has the ability to absorb heavy metals (except for nickel) and avoid in this way their diffusion in the nearby through filtration (Lind, 2005).

➤ ***Fly ash (PFA)***

Fly ash is the filter dust separated from the flue gas of power and heating plants fuelled with pulverized coal. The chemical and phase composition vary widely with the combustion process and the minerals associated with the coal used. In particular, two types of pulverized fly ash (PFA) can be obtained depending on the crystalline content and composition: anthracitic or bituminous coals give ashes high in glass and low in CaO, whereas lignites and sub-bituminous coals provide ashes higher in CaO and also in crystalline phases. The main factors determining the suitability of a PFA to be used as supplementary cement materials are primarily, its pozzolanic activity, the utilization of the amount of unburned carbon and the ability to decrease the water demand. The degree of pozzolanic activity is an important factor for the evolution of long term strength and progressive decrease in permeability and an enhanced workability of concrete can be attained if good quality Class F-PFA is added to Portland cement, due to the smooth and spherical particles composing the ash and to the slower rate of formation of hydration products observed. Furthermore, a reduction in the water content of the mixture is expected since ashes can contain hollow spheres and spongy material. The effect of fly ash is to be a retarder in the formation of alite hydration products at early stage, which, however, is accelerated in the middle stages. Even if the reaction rate of aluminate and ferrite seems to increase in the presence of PFA, a marked reduction of belite consumption occurs afterwards. The reactivity of PFA is mostly based on the glass content, specific surface area, glass composition, and possible crystalline inclusions.

PFA used as mineral addition, has the ability to prevent sulphate heave in the stabilization of sulphate bearing clays.

➤ ***Natural pozzolanas***

Natural pozzolanas are prevalently volcanic materials showing cementitious properties when properly activated. Volcanic deposits can be found in both consolidated (tuff) and unconsolidated state and can be characterized by various chemical alteration.

In order to be used as pozzolanic materials, they have to contain active constituents, i.e. glass high in silica or zeolites, or both. Inactive minerals and organic substances interfering with the setting or strength gain may also be present.

Heat-treated clays, including crushed bricks or tiles, can further be used as pozzolanas and are referred to as "artificial pozzolanas".

➤ ***Silica fume***

Silica fume (also known as "microsilica") is a by-product of the silicon and silicon alloys production composed of an amorphous polymorph of SiO<sub>2</sub>.

During the reduction of quartz in an electric furnace, some SiO is lost as a gas and ionized by the air, giving spherical particles of glass, typically around 100 nm in diameter. Reactivity of silica fume depends mostly on the type and nature of

impurities rather than on the fineness or SiO<sub>2</sub> content. Good-quality silica fume has small particle size and high pozzolanic activity (Taylor, 1997). Due to the high specific surface of SiO<sub>2</sub> spheres, the addition of silica fume is limited by the high water demand. More flexibility is possible when superplasticizers are also used.

Silica fume are usually incorporated in high performance concrete and may provide several effects like strong pozzolanic reaction, accelerated reaction of clinker phases, filling of spaces between clinker grains and, with the consequently production of a denser paste etc. All these physico-chemical processes result in an increased strength and in a reduction in permeability.

➤ ***Cement kiln dust (CKD)***

Cement kiln dust (CKD) is a by-product created during the manufacture of Portland cement. CKD particles are collected from cement kiln exhausted gases and are composed of entrained grains of clinker, unreacted and partially calcined raw materials and fuel ash enriched with alkali sulphates, halides, and other volatiles (Corish and Coleman, 1995).

Despite some of this dust is recycled and added to the cement manufacturing, some has to be disposed in hazardous landfills.

A wide range of chemical composition can be observed in CKD, depending on the proportions of the raw materials, type of kiln, fuel, processing efficiency, and dust collection equipment. Due to the high alkaline content, CKD can be potentially used to activate latent hydraulic binder such as GGBF slag (Konsta-Gdoutos and Shah, 2003).

### **3.3. Mix design**

Every mixing process performed for deep mixing applications is calibrated on a specific mix design, defined by preliminary laboratory suitability tests and validated by the execution of field trials as illustrated in chapter 2. Especially in the latter phase, small variation and calibration of several basic quantities can be operated. The most important parameters that characterize the mix-design of deep-mixing treatments are the set type and amount of binder and the correspondent water content and hence the water-to-cement ratio.

#### ***3.3.1. Type and Amount of binder***

The choice of the most suitable binder and the exact quantity for stabilization purposes is affected by technical, economic and environmental aspects. A precise rule does not exist to find the right combination for stabilizing a specific soil type, because the effect of different binders can vary considerably accordingly to the type of soil to be treated. Some general guidelines are presented from several studies but, usually, both composition and amount of binder mixtures have to be customized for the soil types of each new site basing the choice mostly on the constructor experience on the field related to the specific technology to be used;

hence the mix-design defined for one soil is not directly applicable to another one. Furthermore it is worth noticing that the optimal binder combination is not necessarily the combination yielding the highest possible strength in the shortest possible time. The fact that more moderate strength levels often permit a higher degree of interaction with the surrounding soil is an important perspective to be considered, as well as the possible benefits of a long-term strength increase (Åhnberg et al., 2003).

The experimental evidence has shown that, as a rule, strengths of stabilized soils are considerably lower when treated with lime than with cement (Petry and Kelly, 1988).

In clays, usually, both lime (up to a threshold content of about 4-8% depending of the type of minerals contained - Bell 1996) and cement (Prusinski and Bhattacharja, 1999) may provide satisfactory performances.

In soils containing high fraction of organic matter, especially humic acid, and or heavy metals, GGBF slag in combination with cement has typically proved to be very effective (Hernandez-Martinez and Al-Tabbaa, 2005; Porbaha, 2005; Lind, 2005), even if in specific contexts it may be considered inappropriate (Hayashi and Nishimoto, 2005). Also when using GGBF slag, both in combination with lime or cement, significant improvements have further been observed in inorganic and non-cohesive soils (Jegandan et al., 2010; Wilkinson et al., 2010).

Kuno et al. (1989) proposed guidelines (Table 3.2) for selecting admixtures on the starting point of a soil classification based on natural water content and humic acid content (Figure).

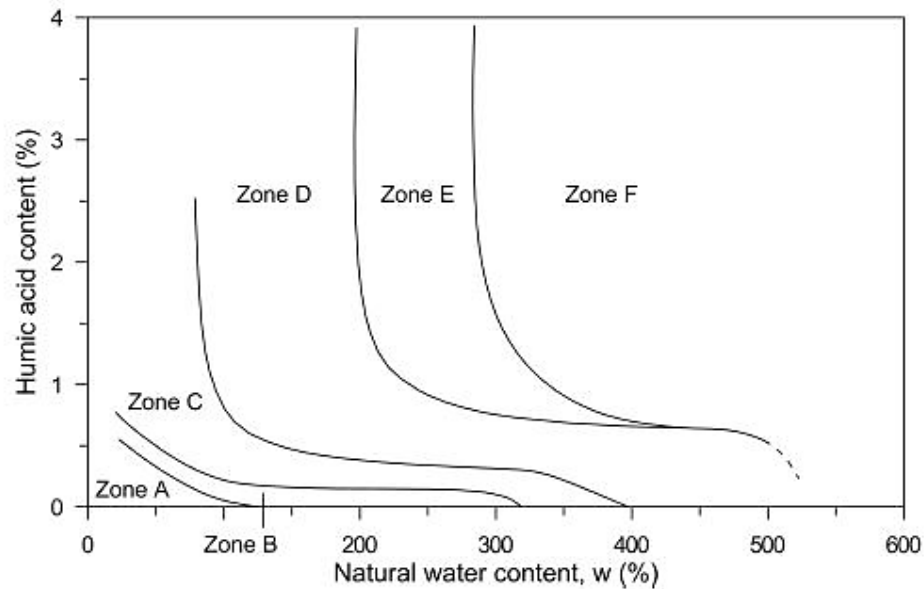
**Table 3.2:** Guidelines for selecting admixtures in organic soils (according to Kuno et al. 1989).

Soil Class	Type of binder				
	Slaked Lime	Quick lime	Slaked Lime + Gypsum	OPC	OPC + Gypsum
Zone A	●	●	●	●	●
Zone B	○	○	●	●	●
Zone C	×	○	○	●	●
Zone D	×	×	×	●	●
Zone E	×	×	×	×	○
Zone F	×	×	×	×	×

●: the short-term increase is excellent.  
○: the short-term increase is modest, but the long-term is significant.  
×: even at long-terms the strength is not significant.

According to Prusinski and Bhattacharja (1999) cement produces higher amounts of cementitious materials due to hydration than lime: this reduces permeability and inhibits leaching in a treated structure. Miura et al. (1997) show that treating marine clays with lime yields to good quality in the improved soil thanks to the presence of salt that accelerate the improving effects.

In dry treatments the lime is able to adsorb more water than the cement to hydrate, this giving bigger benefit in the final strength achieved (Esrig, 1999).



**Figure 3.8:** Soil classification zones based on the natural water content and the humic acid content (modified from Kuno et al. 1989).

Recently, the EuroSoilStab research project (EuroSoilStab, 2002), which was carried out by 17 partners and which was funded by the EU, provided useful recommendations for the stabilization of soft organic soils (Table 3.3).

**Table 3.3:** Guidelines for selecting admixtures in soft and organic soils (according to EuroSoilStab 2002).

Binder	Type of soil			
	Silt	Clay	Organic soils	
			(e.g. Gytja, organic clay)	Peat
				OM
OM 0-2%	OM 0-2%	OM 2-30%	OM 50-100%	
Cement	xx	x	x	xx
Cement + gypsum	x	x	xx	xx
Cement + furnace slag	xx	xx	xx	xxx
Lime + cement	xx	xx	x	-
Lime + gypsum	xx	xx	xx	-
Lime + furnace slag	x	x	x	-
Lime + gypsum + slag	xx	xx	xx	-
Lime + gypsum + cement	xx	xx	xx	-
Lime	-	xx	-	-

- : not suitable

x : good in some cases

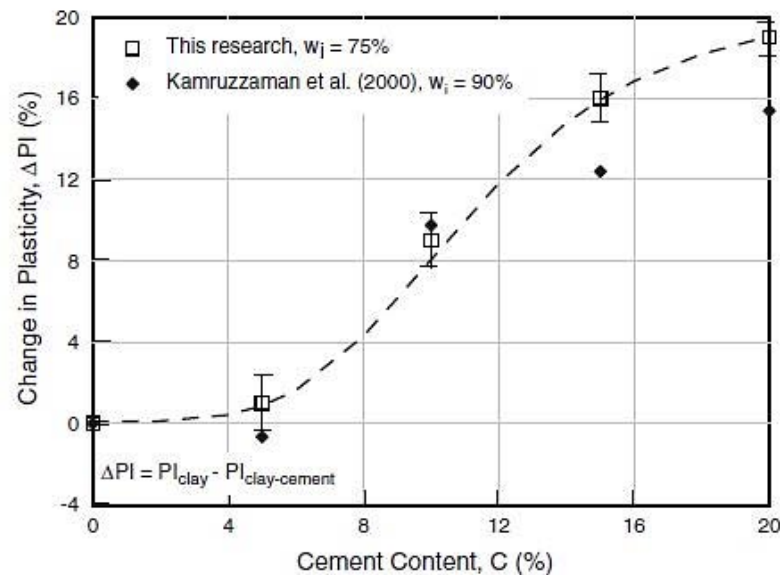
xx : good in many cases

xxx: very good binder in many cases.



In wet mixing methods, an amount of binder of about 100-150 kg/m<sup>3</sup>, for problematic conditions (e.g. presence of organics, contaminants) to 200 kg/m<sup>3</sup> and further beyond, should be considered suitable in most engineering applications. When particular design specifications require somewhat higher strength and lower permeabilities, the cement factor may be increased up to 400-500kg/m<sup>3</sup> of natural soil. For dry mixing treatments, the quantities to be injected are normally lower than the wet ones, around 50 to 80 kg/m<sup>3</sup>. Several authors suggests values within this ranges e.g. Chun and Kim (2003), Hayashi (2005), Horpibulsuk et al. (2011), Horpibulsuk et al. (2012). For lime treatments in dry mixing normally the quantities of lime injected are lower, starting from 40 to 70 kg/m<sup>3</sup> (Nishida et al., 1996).

According to Saadeldin (2013), the percentage of cement to be inserted in the soil with respect to the dry weight of the natural soil to be treated is between 5 to 15% that is the range in which the treated soil has the highest improvement in its characteristics (measured in the “Change in Plasticity”), as shown in (Figure 3.9), with an optimum cement content of 15%. The percentages to be considered in lime treatments are between 3% to 8% (Esrig, 1999).



**Figure 3.9:** Change in plasticity of a clay–cement mix at different cement contents (Saadeldin, 2013).

Despite these general indications, standard strength and permeability tests on samples of mixed soil expressly prepared in the laboratory or collected from field trials at the site should always be employed to determine the most appropriate stabilizing agent and the quantity required for the specific condition (Åhnberg et al., 2003; Al-Tabbaa, 2005). In this way, any special mixing, waste handling, or chemical reaction problem can be identified and resolved before mobilization to the field. Furthermore, since the suitability of a soil for stabilization is controlled by its chemical and mineralogical composition as well as its texture, an understanding of these basic properties is fundamental to reject many unsuitable materials without recourse to testing (Croft, 1967). Notwithstanding, the influence of the construction method, economical efficiency, and other conditions must be considered in determining the final choice.

Two interesting variables used for describing the amount of binder are:

- “cement factor”  $a = W_{c,slurry}/V_{soil}$  (expressed in kg/m<sup>3</sup> of natural soil), in which  $V_{soil}$  represents the volume of natural soil to which  $W_{c,slurry}$  is added;
- “cement content on a dry-weight basis”  $a_w = W_c/W_{d,soil}$  (percent), where  $W_{d,soil}$  is the weight of dry soil mixed with an amount of cement corresponding to  $W_c$ ;

### 3.3.2. Water content and water to cement ratio

One of the most important parameter controlling the efficacy of mixing treatments is the water content of the soil-binder mixture. It may be associated with the natural water content of the same soil when dry mixing methods are employed, or with the total-water-to-cement ratio in wet mixing applications. A limited increase in the degree of liquidity (up to a specific value) of the soil being mixed causes both an increase in strength and uniformity of the blended material (Bell, 1988). Because of this fact, many new mixing technologies rely upon wetting up the soil formations encountered at the site and fluidizing them sufficiently for a much more effective workability and homogenization of the resulting mix (Lebon, 2005). The achievement of the correct degree of fluidization is absolutely a critical task (must enable the achievement both the strength and a certain degree of workability required in the mix-design) and the ideal consistency for the optimum mixing lies slightly beyond the liquid limit content of the original soil (Kitazume, 2005; Szymkiewicz et al. 2013).

Different responses are generally expected by treating materials of distinct mineralogy and crystalline structure, e.g. montmorillonite, illite, kaolinite, etc.

Because of the strong exothermic nature of the hydration process (Section 3.2.2.1), the considerable amount of heat generated immediately after binder addition contributes to the decrease of the natural water content,  $w_n$ , of the soil. This reduction is further enhanced, especially when dry binders are added, by the introduction of a large quantity of solid particles and, at longer curing stages, by the chemical reactions taking place between the stabilizing agent and the natural material.

According to Åhnberg et al. (2003), the total water content of the stabilized soil can be roughly computed by:

$$w_T = \frac{\rho_{soil} \left( \frac{w_n}{w_n + 1} \right) - w_e \alpha}{\rho_{soil} \left( \frac{1}{w_n + 1} \right) + (1 + w_e) \alpha}$$

Where  $\rho_{soil}$  is the bulk density of the unstabilized soil (kg/m<sup>3</sup>),  $w_n$  is the natural water content of the unstabilized soil,  $\alpha$  is the amount of dry binder added to the soil (kg/m<sup>3</sup>), and  $w_e$  is the content of non-evaporable water of the hydration product with respect to dry binder weight, which can be assumed to be approximately 0.23 in a hydrated cement and 0.30 in lime. This decrease in water content is accompanied by an increase in strength and workability.

Other researchers proposed similar relationships to estimate the amount of water of the treated soil mass in the case of wet mixing. In particular, Saitoh et al. (1996) suggested determining the amount of water for cement slurries by:

$$w_T = w_n + \left( \frac{1}{\rho_{\text{soil}}} + \frac{w_n}{100 \cdot \rho_w} \right) \cdot w/c \cdot \alpha \cdot 100 \quad \text{in \%}$$

Where  $\rho_w$  is the bulk density of water. Bergado et al. (2005) used the following equation for the estimation of  $w_T$ :

$$w_T = w_n + w/c \cdot \alpha_w$$

Where  $w/c$  is the water-to-cement ratio of the binding slurry. The above relations, however, disregard the amount of water lost in the hydration of cement during slurry preparation and show that an increased amount of water is present in the wet mixed soil due to the contribution provided by the injected cementitious suspension. The total amount of water present in a treated mass should be at least around 20% of its own weight to ensure the complete hydration of the cement (Sherwood, 1993).

Moreover, the previous estimations are only rough approximation of the initial water content of the soil-binder mixture in its fresh state and do not take into account possible reduction of  $w_n$  due to the ongoing formation of hydration products and corresponding consumption of water (Hernandez-Martinez and Al-Tabbaa, 2005).

The water-to-cement ratio is a key parameter in wet mixing applications. Several authors highlight how this is a fundamental relation to be taken into account while planning a mix-design as it is significant in illustrating the influence of these two parameters on the engineering properties of the treated element and that has to be calibrated to achieve predefined levels of strength and/or permeability required by design specifications. It is defined as: “water-to-cement ratio of the binding slurry introduced deep into the ground”

$$w/c = W_{w,\text{slurry}}/W_{c,\text{slurry}} \text{ (dimensionless),}$$

Where  $W_{w,\text{slurry}}$  and  $W_{c,\text{slurry}}$  are, respectively, the weight of water and cement in the slurry;

In order to take into account the effective amount of water in the mixture, different authors (e.g. Horpibulsuk et al., 2003; Liu, 2008) have suggested taking in consideration also the parameter “total-water-to-cement ratio” which is defined as:

$$w_T/c = W_{w,\text{mix}}/W_{c,\text{mix}} \text{ (dimensionless),}$$

In which  $W_{w,\text{mix}}$  and  $W_{c,\text{mix}}$  are, respectively, the weight of water and cement in the slurry-soil mix.

This parameter is generally much more important than the original water-to-cement ratio of the binder slurry, since it allows taking into account both the real amounts of water (water injected with the slurry plus the natural water content of the untreated soil) and cement present in the treated structure. This avoids the definition of unreliable correlations between

strength and the natural water content of the soil. (Åhnberg et al. (2003), in fact, underlined how there is no unique correlation between the natural water content and the strength of stabilized soils, as this varies with the soil as well as with the type of slurry used.

However, a general result shows (Figure) that the maximum effect is attained at around the liquid limit of the original soil and that the improvement effect considerably decreases with increasing the water content in excess of this threshold (Porbaha et al., 2000; Kitazume, 2005).

The water-to-cement ratio seems to affect also the hydraulic performance of stabilized soils, with higher permeabilities obtained for higher w/c (Marzano et al., 2009).

Water-to-cement ratios ranging from 0.6 to 1.3 are customarily used for cement slurry (Min, 1996; Horpibulsuk et al. 2011), even if, in particular contexts, w/c ratios of 2 and 3 have been adopted (Lebon, 2005). Even if the total-water-to-cement parameter seems to be more significant, normally the parameter water-to-cement ratio is more used and representative in the practice and easier calculated and controlled.

### 3.3.3. Presence of additives

Several researches in the past have been carried out in order to study the effects on the overall response of treated soils with different types of admixtures, e.g. sodium and calcium chloride/sulphate, bentonite and plasticizers.

The addition of *chlorides* (NaCl, CaCl<sub>2</sub>) in soft clays produces the following effects:

- they decrease the liquid limit and reduce the spacing between clay particles thus increasing workability (Modmoltin and Voottipruex, 2009);
- they improve flocculation processes and accelerate the formation of aggregates (Rajasekaran and Narasimha Rao, 1998) with a gradual shifting of the grain size distribution towards larger diameters;
- they increase the strength properties when added to stabilized soils (according to Rajasekaran and Narasimha Rao 2000, maximum increase in strength from 8 to 10 times with respect to that of the original clay can be expected), even if Ca admixtures seem to be more beneficial than Na compounds (Modmoltin and Voottipruex, 2009).

Nonetheless, chlorides may be detrimental at higher proportions, especially when steel reinforcements are installed in the ground improved structures.

Although at short curing after mixing, *sulphates* lead to somewhat similar effects than those obtained by chlorides (the same phenomena, in fact, govern the initial stages - Rajasekaran and Narasimha Rao, 1998). However the use of calcium sulphate and, in particular, of sodium sulphate in combination with lime for clay stabilization is not encouraged (Rajasekaran and Narasimha Rao, 2000).

In cement treatments, *bentonite* can be used to stabilize more effectively organic soils: reduces permeability of treated mixtures (Porbaha et al., 2000; Marzano et al., 2009) and prevent segregation of suspended particles (Lebon, 2005), thus enhancing homogeneity of ground-mixed constructions. However, a slight reduction in strength can customarily be detected.

**Plasticizers** or dispersants or water reducers are additives that increase the plasticity or fluidity of a material. The properties of materials like plastic, concrete, clays, and related products are improved when blended with plasticizers.

They are chemical admixtures that, when added to concrete mixtures or soil mixtures, improve their workability. Unless the mix is "starved" of water, the strength of the element is inversely proportional to the amount of water added or water-cement (w/c) ratio. In order to produce stronger mixtures, less water is added, which makes the concrete mixture less workable and difficult to mix, necessitating the use of plasticizers.

Adding 1-2% plasticizer per unit weight of cement is usually sufficient. Depending on the particular chemical used, use of too much plasticizer may result in a retarding effect. Water-reducing admixtures usually reduce the required water content about 5 to 10 percent. Consequently, mixtures containing a water-reducing admixture need less water to reach a required slump than untreated ones and can have a lower water-cement ratio. This usually indicates that a higher strength product can be produced also without increasing the amount of cement.

The effectiveness of an admixture depends on several factors including: type of soil, type and amount of cement, water content, mixing time, slump, and temperature.

Plasticizers are commonly manufactured from pop lignosulfonates, a by-product from the paper industry, (Figure 3.10). Other common plasticizers used in the construction field are derived from plastic substances.



**Figure 3.10:** Sodium-lignosulphonate powder.

Traditional lignosulfonate-based plasticizers disperse the flocculated cement particles through a mechanism of electrostatic repulsion. In normal plasticizers, the active substances are adsorbed on to the cement or the soil particles, giving them a negative charge, which leads to repulsion between particles.

In this thesis, as described in chapter 2.4, two types of plasticizers have been used for the experimental program, with the aim of reducing the water content of the soil mixture:

- *Sodium-lignosulphonate*: already introduced above, it is an economical additive. Its main performances when applied to cement mixtures are reducing at least to 10% of the mixture's water content, improving the mixability, the compression strength is

improved equally on 3-28 days, reducing the initial stage of the cement heat of hydration by a large margin and having no function of corroding any reinforcing bar. It can be found in powder or in liquid form.

- *Bentocryl 86*: is a water soluble acrylic polymer specially designed to control rheology in self-hardening, cement-bentonite slurries as well as contaminated fresh and salt water-based mud. Secondary functions are fluid-loss control and shale inhibiting properties. It has dispersive and deflocculating properties and its main effects are decreasing of viscosity and shear strength of a mixture, improving homogeneity and having less water losses, reduction of fluid loss down to 4 times depending on percentage of BENTOCRYL-86 thus reducing and thickening the volume of required slurry due to water filtration through permeable formations and finally it acts as a cement retarder. It is cost effective if low amounts are required and environmental friendly.

Most admixtures are supplied in ready-to-use liquid or pulverized form and are added to the mixture at the plant or at the jobsite. Other types of admixtures can be used for specific goals, like retarding admixtures, which slow the setting rate of concrete, accelerating admixtures which increase the rate of early strength development, reduce the time required for proper curing and protection, superplasticizers, also known as plasticizers or high-range water reducers (HRWR), reduce water content by 12 to 30 percent; corrosion-inhibiting admixtures fall into the specialty admixture category and are used to slow corrosion of reinforcing steel in concrete.

### **3.4. Mixing process**

As already introduced at the beginning of the chapter, all the elements characterizing a mixing process have to be taken into account while planning a mix-design for a specific project because their influence on the final result is not negligible, and this has to be known even when laboratory prequalification results have to be compared with the real field conditions.

#### **3.4.1. Mixing tool**

According to Larsson (2005) the mixing tool is the parameter which affects mostly the mixing quality of a treatment; a list describing the significance of every single mixing parameter on the overall efficiency of ground improvement treatments is reported in Table 3.4. The number of shafts, as well as the number of blades mounted on each shaft, has been found to improve performances (Porbaha et al., 2001; Larsson, 2005).

The use of flight augers may be sufficient for predominantly granular soils, but with increasing fineness and stiffness more complicated mixing tools provided with mixing and cutting blades of different shapes and arrangements are required (Massarsch, 2005).

**Table 3.4:** Factors influencing strength variability of DM structures (according to Larsson 2005).

Factor	Strength variability
Mixing tool geometry related to incorporation and spreading of binder	+++
Rheological properties	+++
Compaction / consolidation	+++
Retrieval rate	++
Number of blades	++
Binder content	++
Amount of air	++
Mixing tool geometry	+
Type of binder	+
Rotation speed	-
Air pressure	-

+++ Significant and major influence.  
 ++ Significant influence.  
 + Diverged results.

The incorporation of a set of anti-rotation vanes to prevent simultaneous rotation of the excavated soil with the mixing blades is essential to ensure a good homogenization at the site (Yoshizawa et al., 1996).

Thin blades show, in general, higher strengths than those obtained with thicker elements (Dong et al., 1996).

Experimental findings confirm the significant effect of blade orientation and geometry on the quality achieved, in particular in the case of very soft ground. However, no extensive scientific studies have been published on the shape of mixing tools and its effect on the mixing process of deep mixing structures. It is therefore not possible to give general recommendations concerning the mixing tool geometry, as well as the diameter and position of the outlet hole; the amount of air injected has been reported to have considerable effects on the strength properties of stabilized soils (Larsson, 2005).

### 3.4.2. *Mixing time*

The retrieval rate (along with the number of blades) seems to be the most affecting parameter controlling the results attained from in-situ soil treatments. It is indeed inversely correlated to the mixing time and, consequently, to the energy transferred to the ground (Aalto, 2001). Lower withdrawal speeds, in particular, are suitable when a high strength level and a good homogenization are required. Furthermore Lower extraction speeds in combination with anti-rotating vanes avoid the lifting of blended material which could possibly get stuck on the tool during the mechanical mixing action. High rotation speed is

desirable since the intensity of mixing increases and the time for installation concurrently reduces (Dong et al., 1996; Yoshizawa et al., 1996; Porbaha et al., 2001).

In the Scandinavian countries, the retrieval rate (mm/rev) of the mixing tool is used as a measure of the mixing time. In Japan, on the other hand, the mixing time is measured in terms of the penetration speed (m/min), the retrieval speed, and the rotation speed.

### **3.4.3. Mixing energy**

The mixing work may be defined as a combination of the effective strain in the mixture (indirectly the retrieval rate of the mixing tool) and the intensity of mixing or agitation (indirectly the rotation speed of the mixing tool).

The type of soil and its rheological behavior have a considerable impact on the efficiency of a mixing process and on the effort necessary to mix it up. Clays, normally, require more mixing energy than sands to reach an adequate homogenization, even if, in the current practice, there is a risk that much of the energy used in the dispersion process is expended on remoulding the cohesive material.

Until now, there are still no simple and validated models for the prediction of strength properties of improved soils able to take into account, besides other factors (e.g. curing conditions, type of soil and binder, etc.), mixing energy concepts. On the other hand, the present knowledge is mainly based on laboratory and model tests, which are deemed to be not representative of the real in situ conditions (e.g. Rogers et al., 1997; Terashi, 1997; Bruce et al., 1998). Furthermore, only a few outcomes from field tests are available. The mixing energy is seldom measured.

It is a well-known fact that the mixing work has an influence on the mixing process and its results.

Many studies have come to this conclusion on the basis of laboratory-prepared specimens; however, it is still uncertain how the mixing work affects the mixing process in different soil types with different binders in the field.

More efficient and powerful monitoring systems mounted on recently designed DM machines have been continuously developing and allow the collection of several informations about the mixing process and of the production data which can subsequently be correlated to laboratory outcomes (Babasaki et al., 1996).

## **3.5. Laboratory procedures**

To determine the level of effectiveness for a proposed mix-design, mechanical and permeability tests are usually performed on laboratory prepared samples of soil-binder mixtures created for suitability purposes preliminary to a jobsite activity or to an experimental investigation for development intents.

Other tests are then customarily carried out during preparatory field trials and throughout the real site production. In these cases, samples have to be retrieved at different depths and curing times directly from the improved ground paying special care during sampling operations, in



order to cause the minimal (inevitable) alteration on the hydro-mechanical properties of the collected material.

Because of the quite different blending and curing processes, laboratory and field mixed samples are generally characterized by different results.

In this respect, new-developed DM machines have been recently equipped with high technological instrumentation and monitoring systems able to provide continuous information about the most important parameters affecting the quality and homogenization of the mixed structures and, possibly, explain variations from the expected (laboratory) performance. Furthermore, to better understand the significance of laboratory tests depending on the values of each feature achieved or to be achieved in the site, many correlations derived from several studies are proposed (Van Impe and Veràstegui Flores, 2006; Miura et al., 1987; Liu et al., 2008; Horpibulsuk et al., 2011; EuroSoilStab, 2002).

As different construction procedures and tools influence the attainable outcomes from a real jobsite, also laboratory devices and testing conditions are relevant in influencing the final result; the following description highlights the most important parameters influencing a laboratory procedure.

### ***3.5.1. Mixing***

As in the field, the mixing energy used in laboratory test affects decisively the mechanical characteristics of a soil-mixture specimen and due to this consideration, is fundamental knowing that a key characteristic of the procedure is that a high level of laboratory mixing energy have to be employed; the idea behind this decision is to establish a baseline to which field mixing can be compared, rather than attempting to simulate field mixing in the laboratory. During design-phase testing, the contractor's equipment and techniques are not generally known. Also, for materials that are especially sensitive to mixing energy, laboratory testing in a region of low mixing energy is likely to increase scatter in the test results, and this may make test interpretation more difficult. For example a high level of mixing energy is obtained mixing for a long time (e.g. 10 min) a batch in a kitchen dough mixer or in mixers especially created for testing soil or slurry mixtures.

Nevertheless, it may be useful in some circumstances to prepare several laboratory batches using the same mix proportions but varying the mixing times to investigate the influence of mixing energy on strength for the soils from a particular project site (Filz et al., 2005).

### ***3.5.2. Size of samples***

Customarily, international standard on testing procedure on improved soils (EuroSoil-Stab, 2002) recommend the use of specimens with a diameter comprised between 50 and 100 mm. This suggestion derived from the experimental evidence that larger samples are more representative than smaller volumes of the soil-binder mixture, as they statistically contain more inclusions and inhomogeneities that provoke discontinuities in the structure of the cemented matrix, leading to a reduction in the measured strength.

Hence, smaller samples are believed to overestimate the actual strength (Larsson, 2005).

It should be emphasized that the sample size ought to be chosen according to the magnitude of a soil inclusion: for example a soil inclusion of 20 mm or less will certainly not influence the behavior of the whole soil mix structure, but, conversely, it could notably affect the results in a test sample of 100 mm diameter. In this respect, a scale of scrutiny can be defined to identify the minimum size of the regions of segregation in the mixture that would cause it to be regarded as imperfectly mixed for a specific purpose.

An adequate size of sample would yield to a more reliable estimate of the strength properties of DM structures.

Some stress – strain curves relative to a series of unconfined compression tests carried out on specimens with a diameter ranging from 60 mm to 400 mm are shown in (Figure). They confirm that differences between 100 and 400 mm diameter specimens are negligible.

From this observation, Hosoya et al. (1996) proposed the following empirical relationship:

$$UCS_{d=400} = 0.87 UCS_{d=60}$$

According to (Figure) (Porbaha et al., 2000), small variations in UCS can also be observed between specimens with diameter ranging from 50 to 300 mm.

An aspect ratio of 2 is further suggested for compression tests. This specification can be found in many international standards (EuroSoilStab, 2002; EN 12390-3, 2003) and should be adopted in order to largely eliminate the undesired confining effect provided by the steel loading platens of the testing machine in restraining the lateral expansion of the specimen's bases. If lower aspect ratios are used, an increased strength is expected. In this case, the resulting value has to be reduced by the application of specific slenderness correction factors listed in Table 3.5.

**Table 3.5:** Strength correction factors for length-to diameter ratio (according to ACI Committee 214-2010).

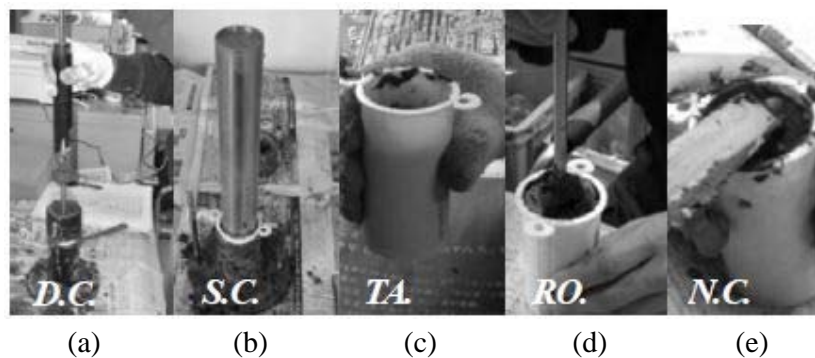
l/d	ASTM C 42/C 42M	BS 1881
2.00	1.00	1.00
1.75	0.98	0.97
1.50	0.96	0.92
1.25	0.93	0.87
1.00	0.87	0.80

### 3.5.3. Sample preparation and moulding

When, in a laboratory test, the soil specimens have to be remolded, in order to ensure the production of a test specimen with the lowest number of defects in the way of representing as much as possible the real treated structure in the field, EuroSoilStab, 2002 and other authors (Kitazume, 2012; Grisolia et al., 2012; Grisolia, 2013) suggest a similar procedure for sample preparation and molding.

After mixing a quantity of soil sufficient to prepare the required number of stabilized soil samples for the given soil and a given stabilizer at a given dosage, the compaction should be performed directly after mixing. The time from mixing to finished sample should be kept low. The entire batch of mixed soil must be formed into samples within 45 minutes of mixing to reduce the effect of the time of rest between the hydration of binder and completion of molding on the specimen's properties. If many samples are to be prepared with the same dosage it can be advisable to split them into two or three batches.

Layer of mixed soil have to be placed in the mould to a thickness of approx. 25 mm thick (aspect ratio 0.5 in case of differing sample diameter), prod it and press it in place with a fork. Take care to eliminate bubbles of liquid or air. Then the compaction process has to take place, choosing between several molding techniques, according to the type and consistency of the material to be compacted (figure 3.11):



**Figure 3.11:** Compaction techniques: (a) Dynamic Compaction; (b) Static compaction; (c) Tapping; (d) Rodding; (e) No compaction.

- *Dynamic Compaction (D.C.):* Each layer is compacted by a falling weight using a special apparatus for a predefined number of blows;
- *Static Compaction (S.C.25 and S.C.50):* Each layer is statically compressed for 10 seconds by using a heavy rod and applying two different pressures (e.g. 25 kPa and 50 kPa);
- *Tapping (TA.50 and TA.100):* For each layer, the mold is tapped against floor 50 times (standard value);
- *Rodding (RO.):* it consists in tamping the mixture with a steel rod for each layer; typical numbers of poking are between 30 and 60.
- *No Compaction (N.C.):* simply consisted in filling the mold with a spoon by either pouring or placing in the case of higher consistency mixture. For example for some high water content peats, very loose and liquid after mixing with stabilizer, there is no need of compaction.

These techniques are those currently used in most of the laboratory all over the world (EuroSoilStab, 2002). The choice of the best technique is dependent on the mixture consistency: Grisolia et al., 2012, showed in a trial test on soil-cement mixtures with different consistencies that, for example, the tapping technique is the most suitable for softer mixtures while the rodding technique is the most applicable for higher consistency mixtures. For very high consistency mixtures an appropriate moulding technique is not identified.

The sample is filled up to slightly above the upper rim of the mould, and trim off excess material above the rim, leaving the upper surface entirely flat. If the mould has a length of more than 100 mm the compaction will have to be done in more than 4 layers.

To prevent water evaporation from the specimen each mould was covered with the sealant and stored in special curing tanks at 95% relative humidity.

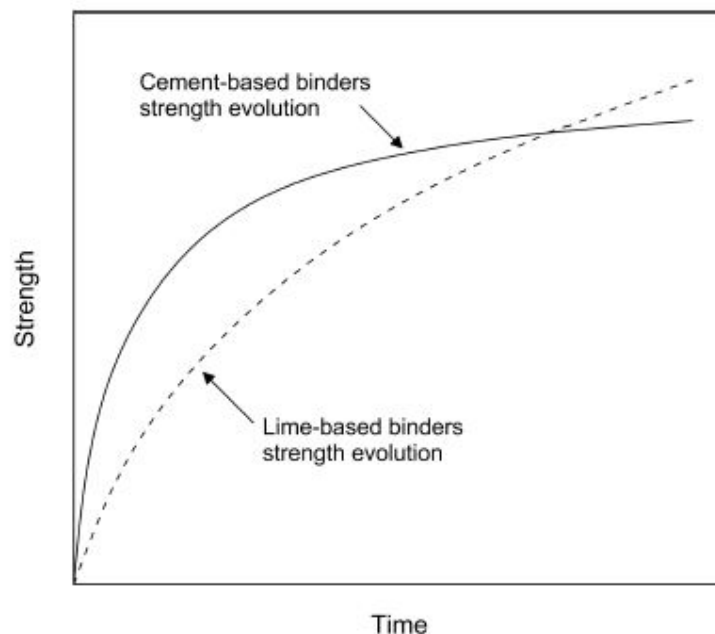
### 3.6. Curing conditions

Curing affects strongly the strength development and the hydraulic and mechanical behavior of DM structures and its influence is a function of time, temperature and relative humidity (Bell, 1996). Generally, strength is expected to increase with curing time, with a more pronounced rapid evolution observed in the initial stages after mixing. The effect of temperature is to increase the early-term strength of stabilized materials, but, at the same time, to possibly reduce their long-term strength. A moist environment is always desirable as allows complete hydration of the binders used for stabilization purposes (Croft, 1967; Bell, 1988).

Time and temperature are the most important parameters that influence the strength gain of treated soils during curing.

The effect of curing time on the hydro-mechanical behavior of stabilized soils is mostly attributable to the progressive precipitation of cementitious products formed by hydration of the binder particles and to the subsequent pozzolanic reactions between primary hydration phases and soil minerals. The chemical composition of these compounds and the time required to be generated vary considerably in function of the type of soil to be treated, of the stabilizing agent used at the site and, particularly, of the binder content adopted for production (Bellato, 2013).

A schematic trend of strength gain with curing time is reported in Figure 3.12 for cement and lime treatment.



**Figure 3.12:** General strength evolution with time using lime- and cement-based binders.

Cement is expected to generate larger amounts of cementing phases at shorter time after mixing than lime, due to its hydraulic nature and self-hardening properties. On the contrary, lime, still reacting rapidly with water, cannot provide by itself any contribution to the strength of DM structures. In order to produce such improvement, lime has to be mixed with reactive materials, like clay minerals, especially expandable clays, or alkali-activated sands. This is the reason why lime, in dry form, is often used for stabilizing only soft and very wet soils. However, as the slow rate of pozzolanic processes is normally not acceptable for engineering applications, lime is customarily used in combination with cement.

In their experimental study on the behavior of treated clays, peats, and Swedish gyttja (organic soil), Åhnberg and Johansson (2005) observed that a more rapid stabilization occurred in samples mixed with binders containing cement. Notwithstanding, at longer curing times the strengths obtained from these samples tended to level off to a fairly constant value or showed clearly reduced rates of long-term increase with respect to the specimens containing lime, which exhibited a steadier pronounced long-term increase in strength.

Similar results were described by many researchers in the literature and allow to conclude that a considerable factor of safety could be assumed when DM constructions are designed based on strengths measured at 7, 14, or 28 days (Bell, 1988).

From the permeability point of view, the continuous precipitation of new cementitious compounds produces a gradual infilling of pores present in the stabilized matrix and a progressive reduction in the hydraulic conductivity.

Several empirical equations for the prediction of strength development of cemented soils have been proposed (Bellato, 2013). Many of them are simple correlations between unconfined compressive strengths at different curing times; the experimental evidence shows that strength data can be customarily well approximated by a natural logarithmic function of the curing time (Kitazume, 2005). Significant statistical correlation has also been observed between measured strength, age after mixing, and water-to-cement ratio in accordance with Filz et al. (2005).

Curing temperature is another meaningful variable acting in the achievement of the proper strength characteristics of a treated soil.

Several studies have confirmed that the hydration and pozzolanic reaction processes significantly depend on temperature (Bell, 1996). Elevated temperatures, in fact, promote an increase in the rate of strength development (Porbaha et al., 2000) and, consequently, the achievement of better mechanical properties especially at short terms after treatment (Kitazume, 2005) (Figure). On the other hand, low temperatures, below 4 °C, retard and in many cases completely stop any hydration or pozzolanic reaction (Bell, 1988).

Due to its direct relation with the chemical mechanisms occurring during the hydration process, the effect of temperature is intrinsically connected with the type of binder and soil to be treated and with the actual amount of stabilizing agent adopted for production (Åhnberg et al., 1989).

In lime a considerable amount of heat, much greater than that observed in cement stabilization, is generated right after mixing. Despite this initial difference, the heat produced by lime slaking rapidly drop off, whereas that derived by cement hydration continues for a longer period (Åhnberg and Johansson, 2005).

The final temperature influencing the curing of DM structures embedded into the ground is also dependent on the temperature variation in the subsoil, on its thermal properties, and on

the boundary conditions between the mixed material and the surrounding media, i.e. size and shape of the improved soil mass (Babasaki et al., 1996).

In this respect, high temperatures may be detected for longer periods within larger volumes of treated soil due to the slower heat dissipation.

As more elevated temperatures lead to a corresponding increase in strength, a more rapid enhancement of strength probably characterize in-situ DM constructions rather than laboratory improved soil samples (Van Impe and Veràstegui Flores, 2006). In order to compare directly the heat and temperatures affecting the curing process of laboratory mixed specimens with that produced at the site, adiabatic instrumentation should be employed (Åhnberg et al., 1989).





# 4

## **A CASE STUDY: HONG KONG MARINE CLAY**

### **4.1. General**

A real case study is presented to illustrate how such investigations can be used as an initial approach for identifying solutions to specific problems encountered during the planning of a jobsite. In particular, as described below, this jobsite was a perfect example because it has to take into account critical working conditions due to the fact that the soil treatment has to be performed from the sea.

### **4.2. Deep soil mixing for land reclamation purposes: the Hong Kong project**

The Hong Kong project is regarding the Hong Kong International Airport (HKIA). It is one of the world's busiest airports, connecting to about 180 destinations worldwide. Air traffic at (HKIA) has increased steadily and the ambition is to turn the airport into a major aviation hub in the Asia-Pacific region. In order to meet this growing demand, the airport must enhance its capacity and to meet future air traffic growth, the Airport Authority has been planning to expand HKIA into a three-runway system. The three-runway system involves the formation of around 650 hectares of land to the north of the existing airport island, about 40% of which falls on an area of contaminated mud pits.

The project entails the construction of a new runway, of related taxiway systems and airfield infrastructure, the construction of a new passenger concourse, the expansion of Terminal 2, the improvement of the road network in the passenger and cargo areas, as well as new landside transportation and car park facilities and any other modification, reconfiguration, and/or improvement of the facilities on the existing airport island as a result of the third runway.

The proposed Airport expansion requires land reclamation in the north of the existing airport island, to provide a platform for the development of the runway and all associated airport infrastructures and facilities. This site has historically been part of the open waters off the northern coast of Lantau Island, where there has been no previous land formation. However, part of the site was used for disposal of contaminated mud by the Government since early 1990s. The contaminated mud pit (CMP) within the site boundary have been filled and capped.

The proposed layout of the Project is shown in Figure 4.1.



Environmental constraints and opportunities were also identified at the outset and considered qualitatively as part of the initial screening process. These include impacts on marine ecology including Chinese White Dolphins, water quality, future aircraft noise, and proximity to the Marine Park.

To form a strong foundation basis for further works of filling for land reclamation, the use of Deep Cement Mixing (DCM) is proposed for the area over the contaminated mud pits (CMP) to prevent the potential of leakage of contaminants from the CMP and eliminate the need for dredging works especially within the CMP area, in order to avoid and/or minimize environmental impacts from reclamation activities.



Figure 4.1: The proposed layout of the Three Runway System.

With the CDM technique in fact, cement is injected and mixed into columns of the soft mud below a sand capping layer is placed over the seabed to increase its strength and stiffness, allowing land formation above without the release of contaminated sediment. For non-CMP area, DCM will be also considered to be applied underneath the runway and conventional vertical drains will be used for the remaining areas. A non-dredged method will also be adopted for seawalls construction.

This method has been widely used in Japan and Asia but has not previously been implemented in Hong Kong, thus the environmental acceptability of this new technique has been tested via field trials before recommending large scale implementation. A comprehensive environmental monitoring program was conducted before, during and after the DCM trial. Potential impacts on the environment during the construction and operation phases of the project have been identified based on a preliminary environmental assessment exercise and are summarized below.

Compared to other construction methods, the adoption of DCM method for construction over CMP area and non-dredge method using vertical wick drains over the remaining area will largely avoid potential adverse impact on water quality and the associated marine ecology and fishery resources due to site formation and reclamation. Generation of underwater noise will be minimized by adopting construction technologies with low underwater noise (e.g. DCM) for site formation/ reclamation and avoiding the use of percussive piling.

However, marine works in general, such as land formation and filling, would possibly generate increased level of suspended solids, generation of sediment plume, re-deposition of sediment and subsequent deterioration of water quality which might indirectly cause disturbance impact on marine habitat and wildlife.

To mitigate contamination by runoff generated within the construction site, runoff will be collected and treated prior to discharge.

The current recommended third runway option set out in this project profile is thus a result of the combined optimization of environmental as well as engineering and other important considerations for expansion of the HKIA (Airport Authority Hong Kong, 2012).

As already introduced above, this jobsite is considered critical for the application of deep mixing technologies. The work for the construction of the foundation layer is done from the sea, this implies that:

- There is an absolute need to produce the minimal spoil material as possible to prevent the pollution of the marine environment; furthermore, the costs for an eventual disposal of the whole amount would be extremely high;
  
- For this application one of the possible alternative is to use a deep mixing machine hold by suspended technology due to strict height restrictions in the area to be treated close to the existing airport. In the use of a suspended technology (Figure 4.2), big cables are driving the mixing tool during the execution of the treatment from a barge. In this case, it is very important to know quite precisely and in advance the load that the equipment needs to lift during the withdrawal phase to calibrate properly the system.

This thesis will attempt to provide a guideline from which it is possible to find an initial answer to both of these requirements.



**Figure 4.2:** CSM (Cutter Soil Mixing) equipment mounted on freely suspended system (Courtesy of BAUER Group).

### **4.3. Main properties of the Hong Kong marine sediments**

As already mentioned previously, in addition to the study of the pure soils there was the interest of facing a real case. This was possible with a prior study on the main properties of the Hong Kong soil: the most important details are reported in this paragraph.

The characteristics of the Hong Kong clay in the area assigned for the construction for the new runway were extensively studied through geotechnical investigations in-situ (boreholes, piezocones, field vane shear tests, permeability tests, etc.) plus laboratory tests (Atterberg limits, water contents, density, triaxial tests). However, this information was not made available for the present thesis investigations.

Thus, the information reported below, to which this thesis refers, is taken from the investigations performed for the construction of the first airport, most of which are reported in the article by Koutsoftas et al. (1987). Nonetheless, the results described by Chack-Fan (1970) indicate that the characteristics of the soils in the two locations are in all likelihood quite similar.

#### 4.3.1. Geotechnical properties

The investigations show a typical seabed made of six main strata that is extended across most of the site. In particular the layers recognized can be described as following:

- “*Upper marine clay*”: The seabed was found to be blanketed by a layer of very soft and compressible marine clay (a seabed mud) up to 15-m thick.
- “*Upper alluvium*”: layer of very stiff desiccated and fissured clay, up to 8-m thick, with frequent dense sand lenses; It is preconsolidated with OCRs in the range of 4-8.
- “*Lower marine clay*”: a layer of medium-stiff, light grey clay up to 15-m thick contains lenses of dense sand and layers of very stiff, rust brown (apparently due to oxidation) clay; is also preconsolidated, but less heavily than the upper alluvium, with OCRs ranging from 2-3.
- “*Stiff clayey layers*” within the lower marine clay generally have significantly higher preconsolidation pressures than the rest of the lower marine clay with overconsolidation ratios ranging up to 5.
- “*Lower alluvium*”: The lower marine clay is in turn underlain by a stratum of very dense coarse to fine sand, up to 10-m thick. At many locations this stratum becomes very silty, with occasional lenses of hard clay. It frequently grades into gravels and cobbles.
- “*Completely decomposed granite (CDG)*” apparent as a very dense, reddish-brown, yellow, or grey coarse to fine sand with little to some silt. It is a residual soil derived from weathering of the granitic bedrock which underlies the CDG.

A typical seabed section is shown in Figure 4.3.

Now the properties of the three main seabed strata identified previously are summarized.

##### *Upper Marine Clay*

It is a mud stratum made of a very homogeneous clay deposit, mostly composed of material from the same characteristic sources and free of varves, stratification, and granular inclusions. It has a thickness up to 15 meters.

The Atterberg limits show a material with a plasticity index of 45-65% corresponding to a highly plastic clay on Casagrande's plasticity chart. Natural moisture contents are typically at or above the liquid limit, indicating a sensitive material. Field vane measurements of remoulded strength showed sensitivities ranging from 7-15 with an average value of 10. Maximum past pressure data from consolidation tests show clearly that the mud is preconsolidated with OCRs in the range 1.5-2 and presumably resulting from aging. The normalized shear strength ratio  $S_u/\sigma_{v0}$ , is of 0,42 from field vane test and of 0,28 from laboratory UU tests. There is a general trend for increasing value of compression ratio CR (defined as the slope of the strain versus log stress curve in the virgin compression zone from consolidation tests) with increasing water content for all soil types, and the mud has the highest values ranging from 0.3-0.5. Recompression ratios (RR, defined as the average slope of the recompression strain versus log of stress curve in consolidation tests) for the mud were typically 5-10% of the corresponding CR values. Representative values for the coefficient of vertical consolidation ( $C_v$ ) for the mud are in the range  $1.3 \pm 0.5$  m<sup>2</sup>/yr. The

consolidation test values of  $C_v$  were also used to compute values of the coefficient of vertical permeability,  $k_v$ , from the relationship  $k_v = C_v m_v \gamma_w$ , where  $m_v$  is the coefficient of volume change and  $\gamma_w$  is the unit weight of water. Coefficients of secondary consolidation ( $C_\alpha$ ) were evaluated for the three clay strata; the mud has the highest  $C_\alpha$  values of  $1.8 \pm 0.5\%$ /log cycle of time, consistent with its highest water content and higher compressibility.

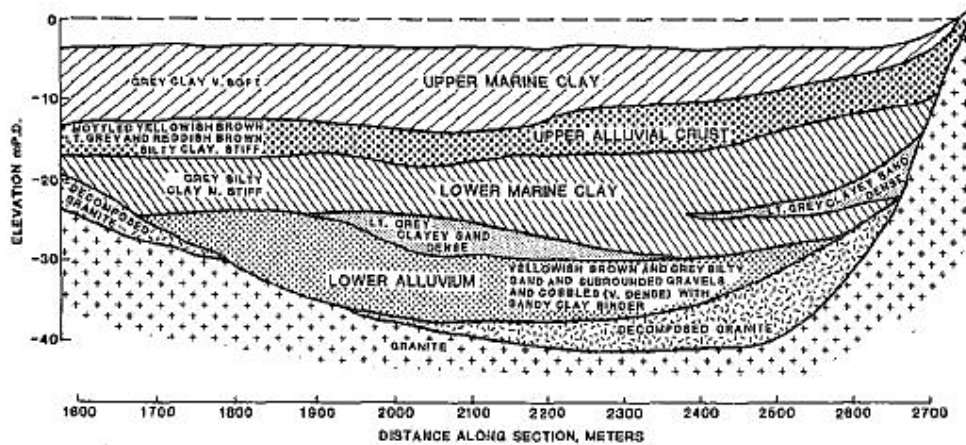


Figure 4.3: Typical subsurface stratigraphy, Hong Kong.

### *Upper alluvium*

Its features are very similar to the lower alluvium ones. The upper alluvium is primarily a very strong clay of relatively low compressibility. It appears to have been highly desiccated and has preconsolidation pressures of 300 kPa or more. Its CR values range from 0.10-0.25, and RR values of 0.015-0.04 were indicated by the consolidation tests, with  $C_\alpha$  values of  $0.6 \pm 0.3\%$ /log cycle of time in the virgin range. Normally consolidated  $C_v$  values were  $3.0 \pm 1.5$  m<sup>2</sup>/yr with recompression values several times higher. Undrained shear strengths of 70 kPa or higher were indicated from field vane and UU tests. The upper alluvium is present throughout the area, with thickness varying from 2 to 8 meters. The stratum includes sand layers, probably resulting from slope wash, and are most common near the islands. The sand layers are typically less than 2 m in thickness. They are of limited areal extent and do not appear to constitute a dependable drainage boundary for incorporation into the reclamation design.

### *Lower marine clay*

The lower marine clay is a much more variable deposit than the mud, ranging from 5-15 m in overall thickness and including granular layers and lenses and zones of stiffer clay material. On Casagrande's plasticity chart it results as a medium-to-high plasticity (CL/CH) material with a liquid limits range from 40-65%, with plasticity indices in the range 20-40%. Natural water contents were found to be from 30-60%, and field vane tests indicated a sensitivity of 4-7. Preconsolidation stresses from consolidation tests indicated a variable pre-stress, with OCRs most frequently in the range of 2-3. Corresponding

normalized strength values from direct simple shear tests were found to be 0.35 and 0.48, respectively, while field vane strength data normalized using the overburden effective stress at each test location resulted in values in the typical range of 0.4-0.65. CR values range from 0.15-0.35, RR values were 0.02-0.06, and C<sub>v</sub> values were  $2.0 \pm 1.0$  m<sup>2</sup>/yr.

Some other general geotechnical characteristics of the Hong Kong soil are reported from Chack-fan (1970): in the natural state marine soils are always fully saturated; the natural voids ratio may be as high as 2 at the surface and its value decreases with increasing depth due to consolidation by the overburden pressure; the density can vary between about 1600 kg/m<sup>3</sup> at the surface to about 1900 kg/m<sup>3</sup> at a depth of 6 to 9,5 m; the permeability of marine soils varies from  $1 \times 10^{-8}$  cm/sec to  $1 \times 10^{-6}$  cm/sec.

Because of its geotechnical properties, the lower marine clay layer appears to be the most difficult soil to be treated with a deep-mixing procedure. In fact it is an overconsolidated clay that shows a certain resistance, requiring a greater effort to be broken up and mixed. For these reasons, this was the type of clay that was decided to be reproduced in the laboratory.

#### ***4.3.2. Mineralogical properties***

To reconstruct as precisely as possible the Hong Kong clay, it was necessary to know its composition since the mineralogical point of view.

In the paper of Chack-fan (1970), interesting information was found about Hong Kong marine soils. A research was in fact conducted on 115 clay soil samples where 31 were collected from construction sites in various parts of Hong Kong. There was primary focus on marine soils which showed good uniformity in their properties along the vicinities of the main islands.

Illite was found to be the major clay mineral in the marine soils in the coastal waters of Hong Kong; its presence is being attributed to the diagenetic transformation of kaolin minerals in the marine environment. The average marine soil near the seabed has an illite content of 80% to 90% in the clay fraction. Kaolin minerals and carbonaceous materials are usually present in minor quantities. Those high percentages of illite are found in samples collected in coastal water within 10 meters below the seabed. Naturally, the illite content varies with the distance offshore, the thickness of the sediments, and the environmental conditions. In general, it can be noted from the coring investigation that there was a progressive decrease in illite content and a corresponding increase in kaolin content with depth. There is also a sharp decrease in both the illite content and the kaolin content going from the clay fraction to the silt fraction, accompanied by a notable increase in the proportion of organic matter and calcium carbonate.

From these considerations the percentages of the distinct pure clays to be used for the reconstitution of the Hong Kong clay were decided. In particular, it was decided to reconstruct the clay with:

- 20% of silt fraction;
- 80% of clay fraction made of: 70% illite (as the predominant clay), 20% of kaolinite, 10% of bentonite (to represent the possible presence of organic matter).

Also the main geotechnical characteristics needed for the reconstitution were decided on the basis of what reported in chapter 4.3.1. For example considering the water content varying from 30% to 60% it was chosen to prepare the samples with a water content of 45%.

## TESTING PROGRAM

A detailed description of the testing program carried out is presented in the following section. As introduced in chapter 2.4, the aim of the tests was to identify guidelines to use when planning a mix-design for a specific jobsite, in order to minimize the backflow produced by soil treatments.

### 5.1. General

The approach used to attain the objectives presented in the thesis, was to act on the water content existing in the soil-mixture in order to maximally reduce the production of spoil material. Two possible approaches were studied:

- Reducing the water content to a minimum level and combining optimally the other relevant features affecting the mixability and the homogeneity, such as binder characteristics, mixing conditions, and production parameters after a very detailed study on the soil characteristics;
- Reducing the water content while guaranteeing mixability and homogeneity through the use of additives.

Once these parameters are defined, tests are needed to confirm or deny their effectiveness. The two main features with which the efficacy of the treatment was evaluated are:

- ❖ The achievement of a predetermined level of strength and permeability in order to meet the engineering requirements of the project;
- ❖ The attainment of a certain degree of mixability and flowability of the soil-mixture in order to allow for a smoothly and accurate execution of the work by the machines. This also led to a high level of homogeneity of the mixture that enabled the development of the optimum engineering parameters described above.

The testing program is subdivided into two parts according to these two different goals: the first part was based on investigating strength development of different soil-binder mixtures with different water contents and with the use of additives. In the second part, a new testing



apparatus was created to simulate the working of a deep mixing machine in different kinds of soil with different water contents. Three types of trials were performed to understand the behaviour of the material induced to flow, so as to obtain an indication of the easiness and quality with which the work of deep mixing can be executed. After two months of literature review and documentation, a testing program was designed: the first month was devoted to the preparation of samples for the unconfined compression test while the next month was dedicated to mixability tests.

The testing program was concentrated on analysing the behaviour of clayey soils. The behaviours of clays are the most difficult to predict during an on-site treatment due to the complexity of their chemical and mechanical properties. Three types of pure clay, those most commonly found in nature, were analysed: illite, kaolinite and bentonite (already described in depth in chapter 3). Then a reconstituted clay soil was examined: the Hong Kong clay soil. A detailed description of each test is presented below.

## **5.2. Strength tests**

The strength test was the first performed at the laboratory. As already introduced, a daily program lasting about a month was planned for building soil-binder samples to break for the unconfined compression strength test after 28 days of curing. In particular, a first series of soil samples was created to analyse the pure clay that was mixed with a pre-set type and amount of cement with the water-content of the slurry as the only variable that was changed. Three different water-to-cement ratios were fixed to characterize the slurry to be mixed with the natural soil and for each W/C ratio, three samples were built. A second series of tests was created with a sand-cement mixture starting with two different natural water contents of the sand, while adding the same amount and type of cement. The aim of also testing a non-cohesive material was twofold: on the one hand, it provides a benchmark against which the clay-cement mixtures can be assessed because the sand-cement mixture should give the highest strength values among all kinds of soil; on the other hand, it completes the research overview by also evaluating the behaviour of non-cohesive materials. A third series of specimens was designed to analyse the Hong Kong clay. This test differed from the previous because three new variables were introduced: while the water-to-cement ratio of the slurry to be mixed with the soil was kept constant, two types of cement in two different quantities were used giving four sets of different samples. Other four sets of samples were built with these same features but included a fixed amount of additive to the slurry. A fourth trial was created with the Hong Kong clay at a high cement content (to reach the already planned amount to be injected in the real project, see paragraph 5.2.3) of a fixed W/C ratio and fixed type of binder, but with the inclusion of two different additives (table 5.1).

Serie	TYPE OF SOIL	Combination No	Material				wn	Cement		w/c ratio	Additive		
			Ill	Kao	Bent	Silt		Sand	CEMI		CEM III	Bentocryl	S-Ligno
1	Illite	1	100%	-	-	-	-	-	5%	-	Dry	-	-
		2	100%	-	-	-	-	-	5%	-	1	-	-
		3	100%	-	-	-	-	-	5%	-	1,8	-	-
	Kaolinite	1	-	100%	-	-	-	-	5%	-	Dry	-	-
		2	-	100%	-	-	-	-	5%	-	1	-	-
		3	-	100%	-	-	-	-	5%	-	1,8	-	-
	Bentonite	1	-	-	100%	-	-	-	5%	-	Dry	-	-
		2	-	-	100%	-	-	-	5%	-	1	-	-
		3	-	-	100%	-	-	-	5%	-	1,8	-	-
2	Sand	1	-	-	-	100%	-	5%	-	Dry	-	-	
		2	-	-	-	100%	-	5%	-	Dry	-	-	
3	Hong Kong Clay	1	16%	56%	8%	20%	-	5%	-	1	-	-	
		2	16%	56%	8%	20%	-	10%	-	1	-	-	
		3	16%	56%	8%	20%	-	-	10%	1	-	-	
		4	16%	56%	8%	20%	-	-	5%	1	-	-	
		5	16%	56%	8%	20%	-	5%	-	1	-	6%	
		6	16%	56%	8%	20%	-	10%	-	1	-	6%	
		7	16%	56%	8%	20%	-	-	10%	1	-	6%	
		8	16%	56%	8%	20%	-	-	5%	1	-	6%	
4		9	16%	56%	8%	20%	-	-	30%	-	1,2	-	-
		10	16%	56%	8%	20%	-	-	30%	-	1,2	-	1,5%
		11	16%	56%	8%	20%	-	-	30%	-	1,2	1,5%	-
		12	16%	56%	8%	20%	-	-	30%	-	1,2	1,5%	-

Table 5.1: Composition of mixes used for the testing program.

### 5.2.1. Brief description of the test procedures

Many laboratory procedures have been proposed over the years in order to have a reference to get a reliable test that describes as closely possible the conditions characterizing the site. The procedure used in this laboratory trial was based mostly on the EuroSoilStab (2002) guidelines.

Taking into account the parameters that affect laboratory procedure (chapter 3.5), this laboratory approach was based on the following choices:

- ❖ The mixer and the mixing time: in order to achieve a high level of mixing energy, a special forced-action mixer was chosen, the Collomix XM2 – 650 (Figure 5.1). It is a mixer created for the production of quite large batches of concrete, mortars, soil mixtures etc. (up to 50 litres). It is provided with two sets of blades rotating on a vertical axis and it is able to mix from very stiff to very soft materials. The time set to mix the final mixture was 10 minutes, as suggested by various guidelines. In particular, the mixture was created by first introducing the right amount of water in the bucket in order to obtain a soil with a precise water content. Then to obtain better homogeneity, the soil was added in two to four aliquots, mixing for fewer minutes than the initial times and then 10 minutes for the final mixture. Finally, the cement slurry was added and the mixture was mixed again for 10 min. Once the mixing was finished, the preparation of the specimens started (Figure 5.2).



**Figure 5.1:** Collomix XM2 - 650.



**Figure 5.2:** Steps in the mixing procedure used to prepare the soil mixture samples.

- ❖ Size of samples: as described in chapter 3.5.2, a sample size between 50 mm and 100 mm diameter is suggested by EuroSoilStab (2002) and a sample with a diameter of 100 mm was chosen. As indicated above, larger samples are more representative than smaller volumes of the soil-binder mixture and in this case, there was no risk of having large inclusions (for example of non-mixed soil or binder) which compromise the result. As recommended, an aspect ratio of 2 was chosen. Hence the height of the samples was 200 mm, in order to obtain the maximum significance from the results. (Figure 5.3).
- ❖ Sample preparation and moulding: after mixing, the moulds were filled with the mixture following the general guidelines mentioned above. The filling and the compaction were made in the shortest time possible (around 30 to 45 min) in order to finish before the binder started the hydration process. The material was placed in the mould in 8 to 10 layers (20 – 25mm thick). Then, based on the observation of Grisolia et al. (2012), a test was performed on the first sample of each mixture to determine the optimal compacting procedure. For the first and the third trial, the rodding compaction procedure was confirmed to be the most suitable; for the second and the last trials, the tapping technique was the best one. For the rodding compaction a steel rod was used; its weight was 600 g and it was covered with a plastic film. This latter approach has been interesting as the film permitted a lower adhesion and suction of the clay on the rod, thus allowing for better and easier compaction. The number of pokes was between 50 and 60. For the tapping procedure, the sample was tapped in a fast way on the floor for 50 to 60 times (Figure 5.3)  
The sample was filled to slightly above the upper rim of the mould, and after the curing time, the top was trimmed off, leaving the upper surface entirely flat.



**Figure 5.3:** Moulds and steel rod used for the testing program.

- ❖ **Curing conditions:** to prevent water evaporation from the specimen, each mould was covered with plastic film as a sealant and stored under water in big buckets placed in a storage room at a constant temperature of 19 °C for 28 days (Figure 5.4).



**Figure 5.4:** Bucket containing water for sample storage.

#### **5.2.1.1. Unconfined compression strength test**

An unconfined compression test is a simple laboratory testing method to assess the mechanical response of concrete, rocks, and fine-grained soils. It gives a quantification of the unconfined compressive strength (UCS) and the stress – strain relationship of the inspected materials. Unconfined compression tests are often included in laboratory geotechnical testing programs, especially when dealing with high strength samples. The main advantage of this

technique is related to the relatively short duration of the loading sequence, which allows for rapid measurements of UCS. In geotechnical practice, this test is used to calculate the undrained shear strength of clays under unconfined conditions, but in the context of deep mixing applications this parameter partly loses its physical meaning. Regardless, it can be a good reference when comparing strength characteristics of different soil-mixtures.

In an unconfined compression test, the sample is placed in the loading machine between the lower and the upper steel plates. Before starting the loading, the upper plate is adjusted to be in contact with the top base of the sample and the deformation is set to zero. The test then starts by applying a constant axial strain of about 0.5-2% per minute. The load and deformation values are recorded as needed for tracing a reasonably complete load – deformation curve. The loading is continued until the load values decrease or remain constant with increasing strain, or until large axial strains are reached. At this state, the sample is considered to be at failure.

Two of the three samples of each type described above were tested for the UCS, according to the DIN 18136. Unconfined compression tests were carried out under a displacement control procedure at a displacement rate of 2 mm/min (Figure 5.5).



**Figure 5.5:** Apparatus for unconfined compression strength test on a soil-mixture sample.

#### **5.2.1.2. Water content**

The “water content” was a fundamental feature characterizing all of the tests performed for this thesis. Several values had to be investigated and monitored during the preparation of the samples.

At the beginning of the laboratory period, a series of pre-tests were carried out to identify the Atterberg limits of the different pure clays, of the Hong Kong clay and for the soil-binder mixtures.



The procedures followed to determine the limits are written in the norm ISO/TS 17892-12:2004. After mixing and homogenizing by hand the natural dry soil with a certain content of water, previously hypothesized to be near the liquid limit, the soil was placed in the Casagrande spoon. After some trials, the moisture content at which it took 25 jolts of the cup to cause the groove generated in the middle to close by 13.5 millimetres was defined as the liquid limit (Figure 5.6).



**Figure 5.6:** Casagrande spoon with illite at its liquid limit.

The plastic limit was determined by rolling out the soil into a rod on a flat, non-porous surface. As the moisture content falls due to evaporation, the rod will begin to break apart at larger diameters. The plastic limit is defined as the moisture content where the rod breaks apart at a diameter of 3.2 mm (Figure 5.7).



**Figure 5.7:** Plastic limit test showing bentonite rods.

After performing all these operations (and for measuring the water content of a soil mix, in general) the water content of the soil mixture was measured as follows: the mixture was spread out on a small pre-weighed (tare,  $w_T$ ) ceramic dish, and re-weighed (wet weight,  $w_W$ ).

This was dried in a microwave at its maximum power for 7,5 min. After the microwave the weight was measured again (dry weight,  $w_D$ ). The water content ( $w_C$ ) was calculated through this formula:

$$w_C(\%) = \frac{w_W - w_D}{w_D - w_T} * 100$$

All the relevant data collected in the laboratory on the water content, for example, the typical limits of the clays studied, are reported in the next chapter.

### 5.2.2. Prequalification tests

All the tests on the pure soils will be considered under the caption “prequalification tests”. These tests are a starting point and provide an indication of the general behaviour of different types of clays found in nature. Their analysis provides an initial idea, of the actual conditions at a job-site, and of the final performance of the soil subjected to treatment once its composition is known. In particular, the predominant amount of a certain component and the presence of other components that may have a considerable influence on the behaviour of the complete soil can be evaluated.

To attain the aim of the thesis, which was to reduce the volume of spoil material by altering the water content, it was decided to begin each test, from a reconstituted soil with a fixed natural water content while varying the characteristics of the binder injected immediately after, as described in the procedure presented above. In particular:

- For clays, the natural water content of the soil to be treated was set at its liquid limit, while two low typical natural water content was chosen for the sand.
- The variable of the test was the water to cement ratio of the slurry that was mixed with the natural soil. Three significant different values were adopted for each kind of clay:
  1. Binder added in dry form, no water;
  2. W/C=1;
  3. W/C=1,8.

For the sand, the binder was inserted only in the dry state.

The amount of cement to be mixed was fixed at 5% of the soil weight while the water content was varied according to the values defined by the W/C ratio.

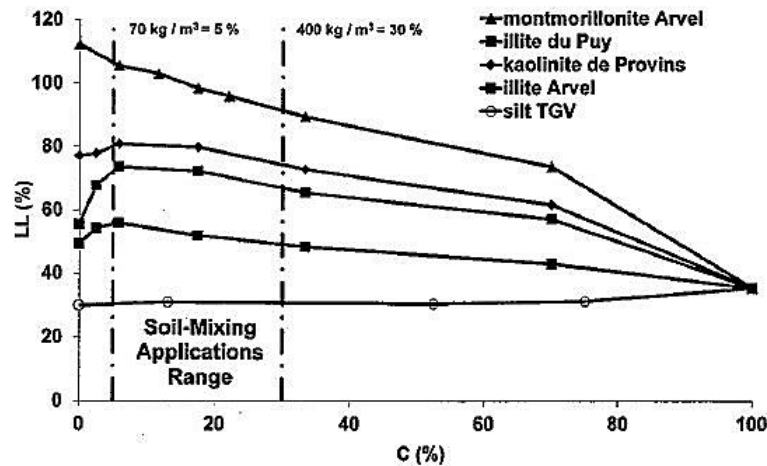
The reason for the choice of these steps is presented in the following description.

First, the easiest way of intervening to minimize backflow is to get an idea of the ranges of water contents over which the dry deep mixing technique (which does not produce any spoil material; as described in chapter 2.2.2), is applicable. Because it is already known that this method can be performed only in very soft and high water content soils, in order to reach a good degree of homogeneity and to have a smooth treatment for less wet soils, the minimum



amount of water in the slurry mixture to inject into the ground must be defined for each kind of soil with a defined natural water content. For this reason, one of the variables used for the test was a dry binder. The specific rationale for the other two variables that concern the water addition, was as follows: condition 2 was chosen because it is a value that is very frequently used for slurries in deep mixing treatments; condition 3 was calculated from a formula suggested by the paper of Szymkiewicz et al. (2013) which was a fundamental reference for the set-up of the entire laboratory procedure for this thesis.

This article focuses on defining a workability limit based on an optimum water content in order to optimize the homogeneity and hence the strength gain of a deep mixing material. The experimental study was carried out on three different types of clay (illite, kaolinite and montmorillonite), on sand, on silt, and on mixed soils. It is proposed in this article that deep mixing is a method that is implemented without compaction of the material. This implies that the material should be self-compacting, meaning that it should be fluid enough to flow under its own weight. Water is, therefore, an essential component, and from the definition of Norman (1958) of the liquid limit of clays as “the water content at which the material flows at a stress level of 2 kPa”, it can be assumed that above this limit, the material is self-compacting. Therefore, in the case of the deep mixing material, the workability limit can be assumed to be equal to the liquid limit. An additional factor to consider is that after the treatment with cement (adding a percentage of between 5% and 30% for deep mixing treatments), the liquid limit of the soil mixture differs from that of the natural soil. However, among soil mixtures this will vary depending on the clay type present. This is shown in (Figure 5.8) and it was also confirmed (chapter 6.2.1) in pre-test studies on the water content performed for this thesis, as described above.

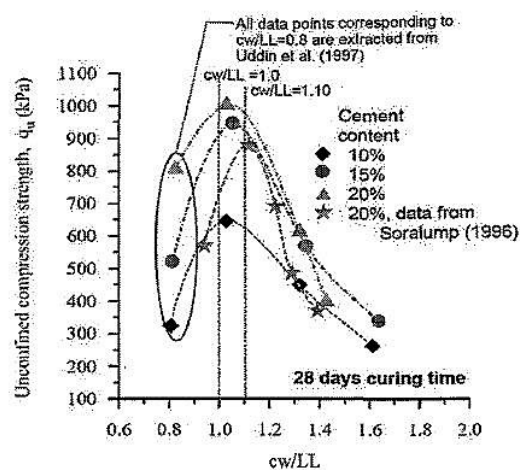


**Figure 5.8:** Evolution of the fresh mixes' liquid limit as a function of cement content for all soils (Szymkiewicz et al., 2013).

On the basis of this, and because the liquid limit of the mixture varies by only a few percentage points, for this research it was decided to treat the natural soil prepared with a water content equal to its liquid limit.

The data of Szymkiewicz et al. (2013) also indicated that being close to the liquid limit means that the material is still very viscous and dense. In these conditions the shearing of the paddles will have an optimal effect on dispersion of the particles, by destroying aggregates of

untreated soil and allowing for better dispersion of cement in the mix. This effect will be less pronounced if the material has higher water content, especially when it is close to or over the flocculation limit and, furthermore, adding too much water means having higher porosity and therefore a loss in strength. Hence, as far as workability and strength are concerned, the existence of an optimum water content, defined as the total water content of the material that would yield the highest possible strength, is observed for distinct soils and is correlated to their plasticity index. This allows one to propose an optimized domain of workability within which the homogeneity and the strength of the material would be the highest achievable. Due to these considerations, the domain can be positioned between the liquid limit and the flocculation limit; the optimum water content is in between. Furthermore Bergado and Lorenzo (2005) showed that this optimum does not vary with the cement content (Figure 5.9).



**Figure 5.9:** The optimum water content does not vary with cement content (Szymkiewicz et al., 2013).

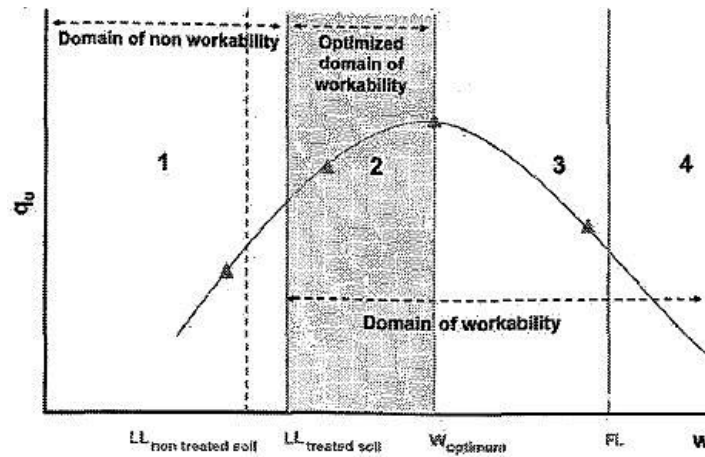
Hence, Szymkiewicz et al. (2013), considered that the plasticity index (PI) was a significant parameter that distinguished different soil types, and found a linear correlation between the optimum water content and the PI of low plasticity clays (illite and kaolinite):

$$\text{Optimum } w/LL = 0.0464 \cdot (PI) + 0.5621$$

This relationship is valid for every cement content and W/C ratio. In this way it is now possible, by knowing only the plasticity index of the soil to be treated, to estimate the optimum amount of water to add to the mix, after having previously determined the dosage of cement to be used.

For this thesis, this calculation was made for each type of low plasticity soil and the result suggested that the optimum water content, according to this theory, would have been reached with an addition of water corresponding to the injection of a slurry with a water to cement ratio of about 1.8. From this result, the third point of the analysis was fixed.

The final idea, therefore, was to investigate the behaviour of the treated clay soil in this domain of workability, and in particular, in the optimized domain of workability as shown in (Figure 5.10) in the graph created by the authors. This optimized domain of workability, in fact, shows which configuration of soil mixture is the best to reach the highest strength values possible, while still having a workable and thus homogeneous material.



**Figure 5.10:** Domain and optimized domain of workability according to Szymkiewicz et al. (2013).

This study of Szymkiewicz et al. (2013) was relevant to the work proposed for this thesis because it provides a potential point of reference as it is probably physically impossible to treat soils that do not have a water content of at least their liquid limit without water injection. Moreover, considering the liquid limit of a soil as a lower limit, the range investigated by this theory and, hence in this thesis, was in the vicinity of this limit and then in accordance with the idea of limiting as much as possible the water injection in the mixture, with the ultimate purpose of reducing the backflow.

The test with the sand also was considered under the “prequalification tests”. As described above, two different series of samples of the sand with two different water contents were built and a fixed quantity of dry cement was added. Because sand is a cohesionless soil, it can be easily mixed at lower water contents due to its mechanical and physical properties. However, a minimum amount of water must be present in order to enhance the complete hydration of the binder injected. Even if sand is more easily mixed than cohesive soils, and high homogeneities are achieved, at the construction site, a deep mixing process in a sandy soil must be made smooth to make the material flowable through the machine’s blades by adding water. For this reason, spoil material is also produced with sandy soils and should be minimized.

### 5.2.3. Hong Kong soil

As already addressed above, the case of the Hong Kong soil was analyzed as a possible application of this experimental study to a real construction site. For this reason and because of the unavailability of the authentic soil, the Hong Kong clay was reconstituted from the description obtained from jobsite reports on the geotechnical characterization of the Hong Kong soil (chapter 4). Because Hong Kong subsoil has different types of soil layers with different characteristics, it was chosen to represent one of the predominant soil types and the most problematic with respect to a deep mixing treatment.

While the features of the real subsoil are reported in the previous chapter, an explanation for how the reconstituted soil was designed is described.

According to the jobsite reports and to other papers describing the Hong Kong soil, the aim was to represent the “lower marine clay” layer, as already illustrated in the previous chapter. The clay fraction was described as being predominantly illite, with the presence of some kaolinite and a little amount of organic matter.

The composition of the reconstituted clay was:

- a) Clay fraction: 80%; composed of 70% illite + 20% kaolinite + 10% bentonite.
- b) Silt fraction: 20%
- c) Natural water content: 45%.

For this type of soil several samples were prepared. As already mentioned above, different variables were taken into account in this test.

- In order to perform a study for a real deep mixing treatment of the Hong Kong clay, the starting soil in which the slurry was inserted was prepared, at its natural water content as was fixed previously, not at its liquid limit.
- The hypothesis to treat the soil with only dry binder was immediately excluded, since the soil has a water content lower than the liquid limit. This decision was confirmed after an initial visual assessment of its consistency which, with this water content, was very stiff. Then it was decided to treat the soil with a binder-slurry of a fixed water-to-cement ratio equal to 1 in order to minimize as much as possible water injection.
- Three variables were tested:
  - Type of cement: CEM I, CEM III;
  - Quantity of cement: 5%, 10%;
  - Use of additives.

The rationale for these was to understand how the behaviour of the clay treated with different binders was altered, and to identify the one that would be most suitable for the real on-site application. In particular, it was necessary to establish the levels of strength achieved with each binder and the increment in strength achieved by an increase in the percentage of cement added. The scope of this kind of study was to understand the capacity of the Hong Kong soil mixture to develop strength. In fact, it is essential to reach the required strength of the project with the least possible cement content to minimize the spoil material. Thus, given that the W/C ratio of the slurry is kept low, the quantity of binder will be less, and the amount of water injected can be reduced. A second series of samples with these same varying features was made with the addition to each sample of 6% of the binder weight of *sodium-lignosulphonate*. The aim of including the additive was to improve the mixability of the mixture without adding more water. The percentage of additive included was decided after a series of tests in which the percentage of additive that produced a visual improvement of the mixture’s mixability had been determined. The strength test on these samples investigated if there would be any effect on the strength gain due to the presence of the additive.

- Another series of samples made with Hong Kong clay but with different characteristics were produced to study the case in more detail. In particular, in this set of tests, different additives were introduced. The initial decision was to use in the mixture the cement content already planned for use on-site (assuming that a normal

wet mixing treatment would be done), i.e. 30%. Since this percentage was very high, during the preparation of the first bucket of treated soil it was decided to add a slurry with a W/C ratio of 1,2 to reach a better workability and homogeneity of the mixture. CEM III was the only binder used and two types of additives were tested: four groups of specimens were created with the following characteristics:

- The first group of three samples was made with the Hong Kong soil mixed with a slurry made of CEM III (at a cement content of 30% of the soil weight) with a W/C ratio of 1,2 and without additives
- The second group was made with the same characteristics as the first but adding to the binder slurry the *sodium-lignosulphonate* additive at a percentage equal to 1,5% of the cement content; this percentage was determined after a trial, and was the first percentage that led to a visual improvement of the mixture's mixability;
- The third group was prepared with the same characteristics of the first as well, but adding to the binder slurry the *bentocryl-86* additive at a percentage of 1,5% of the cement content;
- The fourth group had exactly the same features as the third with the difference that the additive was added to the pure soil before the introduction of the slurry.

The objective of these tests was to determine the efficacy of the additive in terms of strength, its effect on the workability of the soil and the conditions in which it worked best.

### 5.3. Mixability tests

The mixability tests were fundamental for understanding the behaviour of the soil to be subjected to a deep mixing treatment. A smooth procedure and a certain degree of workability have to be guaranteed to reach the development of the optimal engineering properties of the treated volume, to optimize the time required for the execution of the project, and to facilitate the working of the machine. In order to achieve these demands, a basic requirement for deep mixing treatments in general is that the soil has to flow when the machine is moving inside the soil allowing it to exert the least amount of effort possible.

No existing test was identified that would permit these studies to be performed in the laboratory before an on-site application. Thus, in accordance with the objective of this thesis, it was necessary to develop and build a new apparatus (Figure 5.11).

#### 5.3.1. Laboratory equipment used for the tests

The instrument is made of a metal supporting structure which rests on a horizontal base. Bolted in front of this structure is a hollow steel pipe with a square section of 91x91 mm that is filled with the material to be tested. Within this material, an element is positioned with a cross section that perfectly fits the pipe, and that is lifted (Figure 5.12).



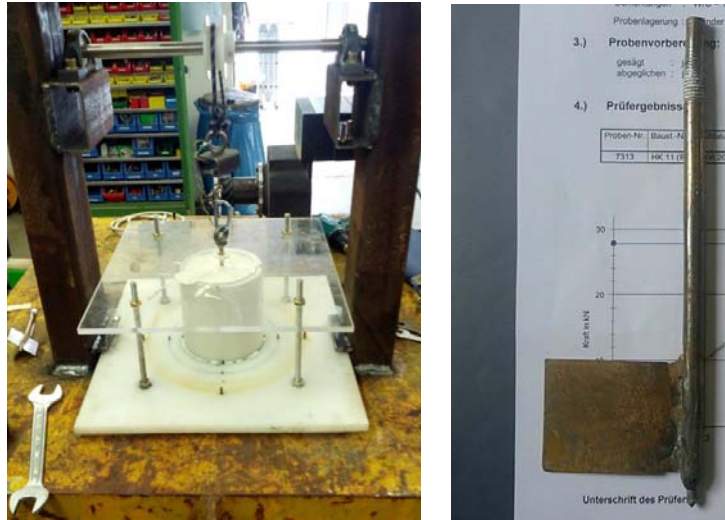
**Figure 5.11:** New test apparatus for mixability tests.

On the base of the apparatus, under the steel structure, a cylindrical plastic cell is placed which is filled with the material to be tested. Two other tests can be carried out with this device through the use of a small blade fixed on its metallic support (Figure 5.13).



**Figure 5.12:** New test apparatus for mixability tests – steel pipe (left) and cross-section element (right).

An engine placed on the horizontal base is connected to a reel that permits the instrument to be lifted with a cable. A load cell placed at the end of the cable measures the force and converts it into an electrical signal sent to a control unit for data recording (Figure 5.13).



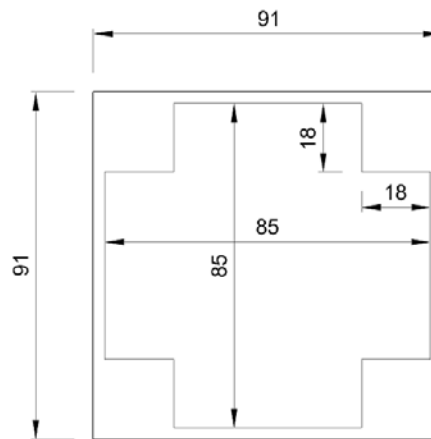
**Figure 5.13:** New test apparatus for mixability tests – cylindrical plastic cell (left) and the blade element (right).

With this new test apparatus, it is possible to perform three different tests. Like the strength tests, four kinds of soil were tested: three different kinds of clays (illite, kaolinite, and bentonite) plus the Hong Kong clay. No binder was added to the soil because the time within which the hydration of the cement begins would not have permitted the test to be concluded under the same initial conditions. However, this condition is not dissimilar to that which occurs in real deep mixing applications in which the most common procedure is to inject the binder only during the withdrawal phase (this phase is the most critical from the point of view of effort). When the tool is being extracted the treated soil stays at the bottom whereas it is the untreated soil above the tool that influences its withdrawal.

Each clay was tested with four separate water contents.

- ❖ TEST 1 consisted of lifting the cross section element through the soil placed and compacted inside the column. The column was sealed at the top with the aim of containing the material and to force it to flow through the instrument (Figure 5.12). The idea behind this test was to simulate the behaviour of the whole machine-soil complex during the withdrawal phase of a deep mixing treatment. The measurement determined the force needed to lift the tool, from which the propensity for the soil to be flowable, and the effort that has to be exerted by the machine to extract the tool from a given mixed soil, without lifting up the soil itself can be evaluated. In order to meet the requirement of producing low quantities of spoil material, the soil must not exit the treated volume and the work has to be performed with the lowest water content possible, but enough to guarantee the workability of the soil. The dimensions of the complete pipe-element were designed to recreate the same percentage of free space where the soil flows through that is present in the excavation area not occupied by the machine's mixing tool. For reference, a percentage value of 71%, was calculated based on an excavation site with the CSM technology

(figure 5.14). The weight of the cross section element (660,18 g) was calibrated in order to overcome the buoyancy component.



**Figure 5.14:** Diagram showing the dimensions of the cross section element and of the steel pipe for test 1.

- ❖ TEST 2 involved the use of the blade placed inside the cylindrical cell filled with clay. After the box was filled, it was sealed and the metallic support of the blade was connected to the load cell and the lifting was started (Figure 5.13). In this case, the force measured by the load cell represents the effect caused by the adhesion of the clay. In fact, unlike test 1, where the load cell also measured the weight of the soil column above the cross-section element, in this test, where the blade has a negligible thickness, the cell measures only the effect of the clay's adhesion to the blade. In this way the significance of the adhesion effect of each kind of clay at different water contents was demonstrated. The blade is 50 mm wide and 40 mm in height with a thickness of 1 mm; its pedestal is 175 mm long and the total weight is 73,5 g (Figure 5.13). These dimensions were chosen to ensure a good torque and adhesion measurement and its weight was calibrated in order to overcome the buoyancy component.
  
- ❖ TEST 3 was also performed in the cylindrical cell. The blade was replaced in the box and on the top of its support the instrument for the torque measurement used for the vane test was applied (Figure 5.15). The blade was turned slowly by 180° transmitting a torque value (Figure 5.18). This test provides a measure of the propensity of the soil to flow when it is moved. Test one and test three investigate the same property of the clays; the main differences between the two are that a torque instead of a force is measured but in this test other factors that influence the measurement, such as the effective weight of the tool and the weight of the column of the overlying soil are not included.

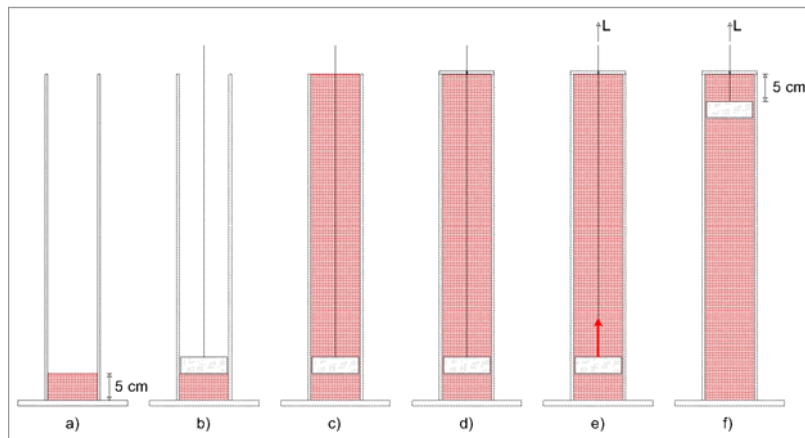




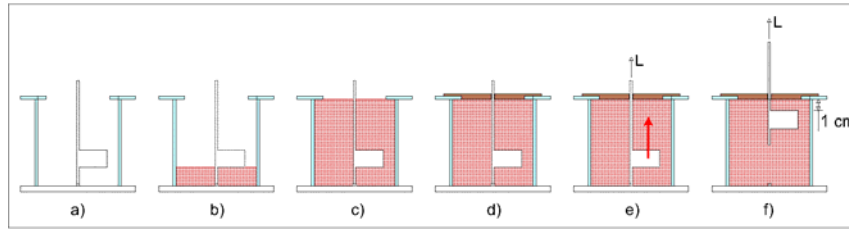
**Figure 5.15:** Vane test apparatus.

The procedure used to perform the test was the following: firstly the material was prepared in the mixer with the set water content. Then the steel pipe was filled with the material. Initially a small layer was placed at the bottom of the pipe. The element was then placed, and then the rest of the pipe was filled to the top and sealed with a cover fixed with screws. The cable with the load cell was connected to the steel cable hooked to the element and, after setting the tare, the first test was started (Figure 5.16). After the end of the test, the tube was emptied into the bucket and the material was poured into the cylindrical cell. The cell was sealed with a cover, and the blade's support was connected to the load cell and lifted after setting the tare. The speed for the two lifting tests was set to 0,5 cm/sec (Figure 5.17). Finally, the load cell was removed and the blade was connected to the torque measurement instrument. Then the blade was placed in the middle of the box on an appropriate pedestal and was turned and the torque measurement was recorded (Figure 5.18).

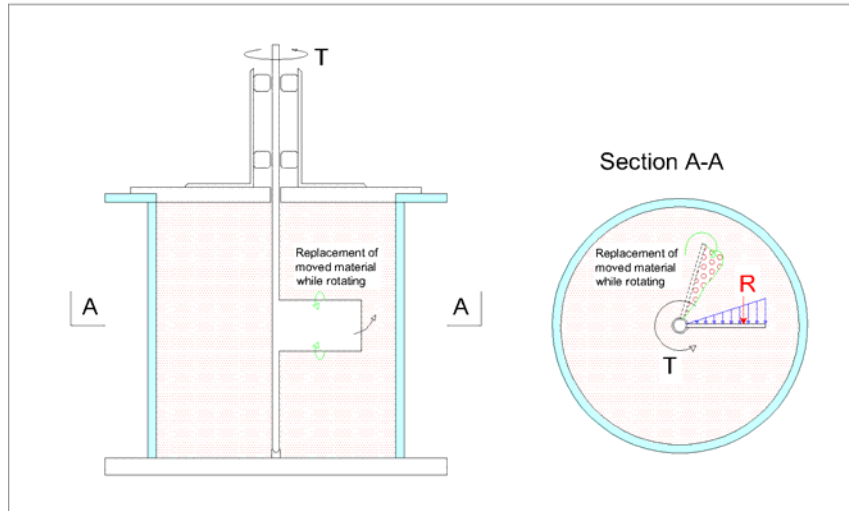
Before starting the tests with the soil, a trial was carried out with only water. As expected, the collected values for all the three tests with water were nearly zero. It was also necessary to determine the buoyancy component on the tests. It was calculated that the buoyancy component was approximately equal to the weight of the two instruments used. This also provided an indication of a good functioning system.



**Figure 5.16:** Diagram showing procedure for the execution of test 1. (a) soil layer at the bottom of the pipe; (b) the element is inserted into the pipe; (c) the pipe is backfilled with soil; (d) the pipe is sealed; (e) the element is raised; (f) the test is complete when the element reaches 5cm from the top of the pipe.



**Figure 5.17:** Diagram showing procedure for the execution of test 2. (a) the blade is inserted in the cell; (b) a layer of soil is placed; (c) the cell is backfilled with soil; (d) the cell is sealed; (e) the blade is raised; (f) the test is complete when the blade reaches 1 cm from the top of the cell.



**Figure 5.18:** Diagram showing the principles of test 3. Vertical (left) and cross section (right) of the cell.

The “*Haegermann table*” test also was performed on all the clay types to evaluate the reliability of this new test apparatus and to determine if there was a correlation between the two instruments (Figure 5.19). The “*Haegermann table*” is an instrument that is usually used for measuring the consistency of a mortar or a concrete, according to DIN 1048. The tests are executed, for example, with a specified amount of material filled into a cone-shaped mould. Subsequently, this cone is pulled off in an upward direction, and the diameter of the spreading material is measured. Then the table top with the material is jolted 15 times. The material spreads out, and after 15 jolts the final dimensions of the material are measured. This method provides information about the rheological behaviour, the tendency for sedimentation, and possibly the existing thixotropic properties of a mortar. The idea of adapting this test to a soil mixture was made to try to classify the soil according to its rheological properties (consistency, flowability) using a standardized tool. However, this instrument works over a limited range of soil water contents and for very liquid soils, the spreading is not homogeneous and the soil flows out of the table, whereas for very stiff and consistent soils there is no modification in the shape after the 15 jolts.



**Figure 5.19:** Haegermann table.

### ***5.3.2. Determination of mixability***

From TEST 1 and TEST 3 it was anticipated that the results would provide an estimate of the ideal mixability of the soil through a graphic function that expresses the effort required to make soil mixtures with varying water contents flow through the spaces surrounding the machine's tool. It was anticipated that a decrease in water content would result in a significant increase in the effort, and hence the mixability behaviour would exhibit a hyperbolic trend. The aim was to identify minimum water content, for each kind of clay, above which the machine can work in optimal conditions.



## TEST RESULTS

### 6.1. General

The results obtained from the laboratory tests are presented in this chapter. The combined results derived from the strength and mixability tests provide novel and meaningful information that confirms the initial hypotheses. The most relevant observations are described below:

### 6.2. Strength tests

After 28 days the samples prepared in the laboratory were broken to perform the unconfined compression strength tests. In the following paragraphs the graphs that describe the results for each series of tests are reported and commented on.

#### 6.2.1. Water content

The determination of water content was the first fundamental test performed. Before continuing with other tests, it was necessary to first define the characteristics of the soil-treated and to be treated. As reported in the previous chapter, one of the main variables on which the entire research is based is the water content. The measurement of water content was necessary for two purposes: to determine the Atterberg limits and to check the value of the water content reached during a test. In particular, it was essential to determine the Atterberg limits for the pre-qualification tests; for the Hong Kong clay it was needed to verify that it was at its natural water content after reconstitution of the soil, whereas for the mixability tests it was important to confirm the water content for each soil at each step of the analysis.

In the following tables (table 6.1 and 6.2) the characteristic limits for each soil and treated soil are reported.

<b>ATTERBERG LIMITS</b>			
<i>Limits</i>	ILLITE	KAOLINITE	BENTONITE
w <sub>L</sub>	<b>54,9%</b>	<b>48,2%</b>	<b>160,9%</b>
w <sub>P</sub>	<b>41,7%</b>	<b>34,8%</b>	<b>57,7%</b>
IP	<b>13,1%</b>	<b>13,4%</b>	<b>103,2%</b>

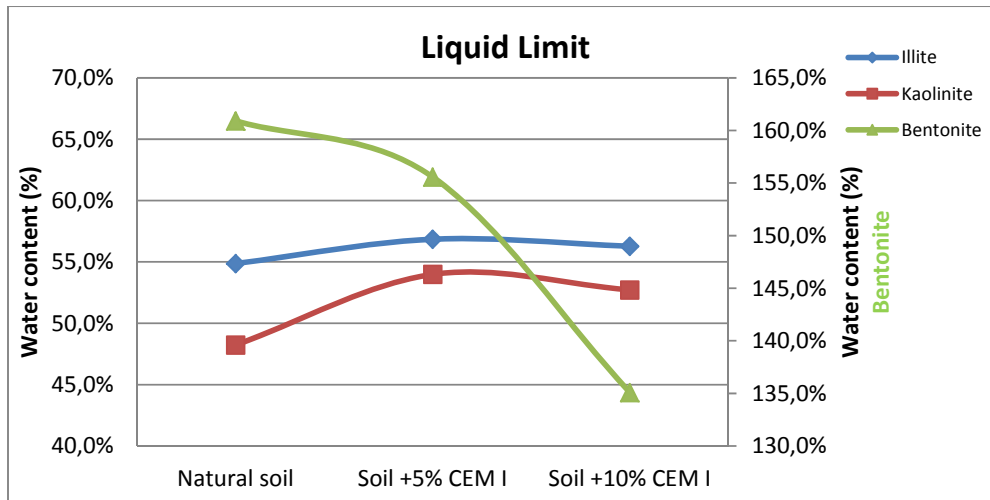
**Table 6.1:** Atterberg limits of pure clays.

<b>LIQUID LIMIT</b>			
$w_L$	ILLITE	KAOLINITE	BENTONITE
Natural soil	<b>54,9%</b>	<b>48,2%</b>	<b>160,9%</b>
+5% CEM I	<b>56,8%</b>	<b>54,0%</b>	<b>155,6%</b>
+10% CEM I	<b>56,3%</b>	<b>52,7%</b>	<b>135,1%</b>

**Table 6.2:** Liquid limits of treated clays.

From these two tables it can be observed that there are significant differences in the liquid limits of illite, kaolinite and bentonite. While the first two have a similar limit, the third has a much higher limit. This result confirms the differences in the chemical and mineralogical properties of low plasticity and high plasticity soils, as addressed in chapter 3. Consequently, it is possible to obtain an initial idea of the magnitude of the differences in their behaviour from the geotechnical point of view.

The second table (table 6.2) associated with the following graph shows how the liquid limit varies with the addition of the binder.



**Figure 6.1:** Liquid limits of treated clays.

This data confirm the observations of Szymkiewicz et al. (2013) reported in chapter 5.2.2 (Figure 5.8). After the treatment with cement, the liquid limit of the soil mixture differed from the natural soil's liquid limit, although this varied depending on the clay type. The liquid limit for low-plasticity clays like illite and kaolinite increased from 0 to 5% of binder addition, but then decreased slightly with further addition. However, for high plasticity clays, like bentonite, a continuous decrease with binder addition was observed. This is a further verification of the diverse behaviours among clay types.

### 6.2.2. Prequalification tests

The results from the prequalification tests are summarized in the graphs presented below. For each test, several relevant observations can be noted:

#### 6.2.2.1. Illite

Illite at its liquid limit was mixed with a dry binder or a slurry with a cement content equal to 5%. The graph below (Figure 6.2) below shows that the highest strength was achieved when the binder was added in the dry state, whereas increasing the water content (with the slurry injection of W/C=1 and W/C=1,8) led to a decrease in the maximum strength achieved. It also should be noted that all the soil mixes broke down under compression, demonstrating a brittle behaviour as expected from a cemented material.

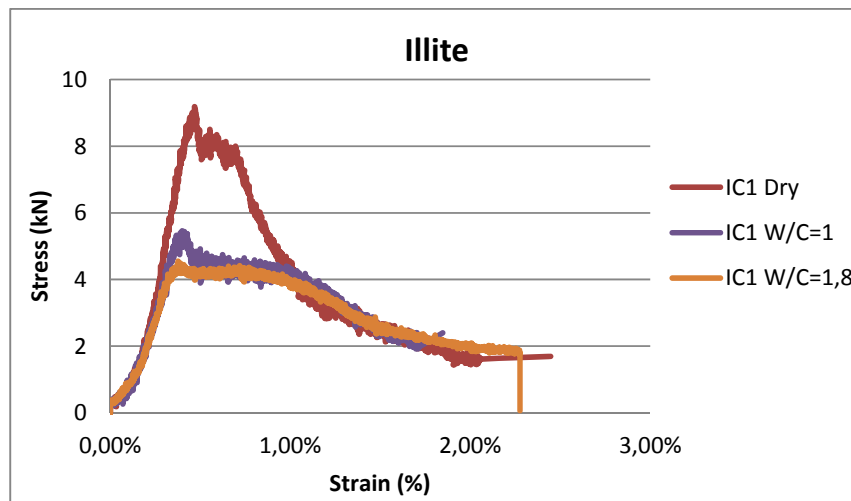


Figure 6.2: Strength tests using dry binder or slurries with different W/C ratios - Illite.

#### 6.2.2.2. Kaolinite

As for illite, kaolinite also was prepared at its liquid limit and then mixed with the dry binder, or with the slurries at two different water-to-cement ratios (1 and 1,8). Even for kaolinite, the highest strength was reached when the binder was added in dry state, while increasing the water content led to a decrease in the maximum strength achieved (Figure 6.3). These samples also broke down under compression, demonstrating the brittle behaviour typical of cemented materials.

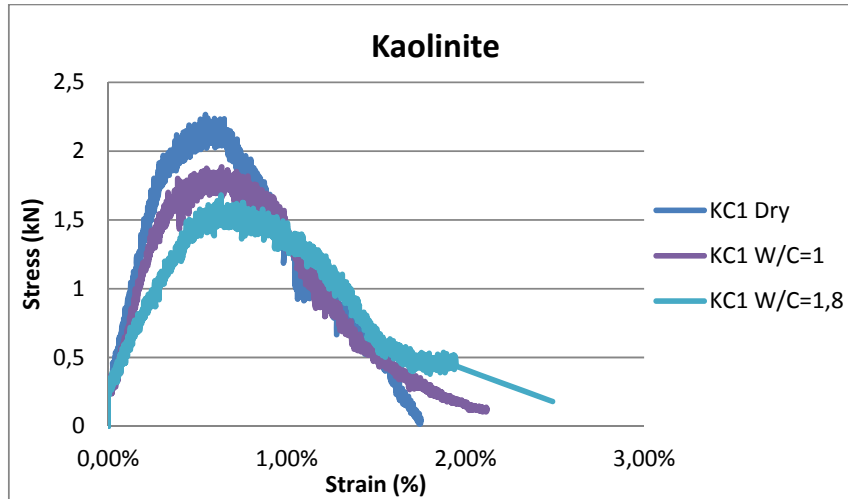


Figure 6.3: Strength tests using dry binder or slurries with different W/C ratios - Kaolinite.

#### 6.2.2.3. Bentonite

Bentonite, was also mixed at its liquid limit and tested with dry binder and two slurries as described for the other two clays. The bentonite specimens in all three combinations exerted little resistance to compression, as if it contained no binder. The specimens immediately deformed under the increasing weight and demonstrated a diminishing load-bearing capacity. Of the 3 mixes, the bentonite mixed with dry cement was the soil mixture that provided the highest resistance (Figure 6.4). The reason for these responses is that bentonite has a higher starting water content (because of its liquid limit) and the percentage of binder added (5% CEM I) was not sufficient to develop any additional resistance.

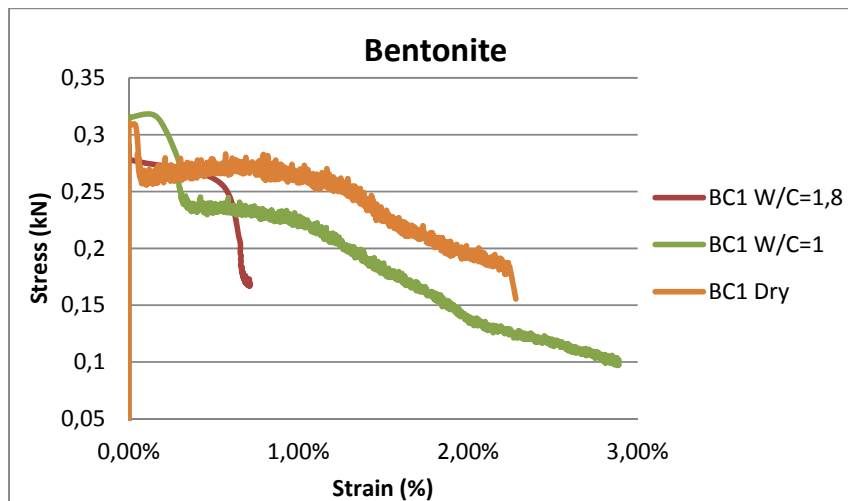


Figure 6.4: Strength tests using dry binder or slurries with different W/C ratios - Bentonite.

After evaluating the results for each kind of pure soil tested under the same conditions of percent cement content and slurry composition, the performance of the different clay types was compared. As shown in the graph in the Figure 6.5, Illite is the clay that exhibited the highest strength even though its water content, for example, was higher than for the kaolinite (its liquid limit is higher). On the other hand, bentonite, with its much higher water content, produced the smallest resistance.

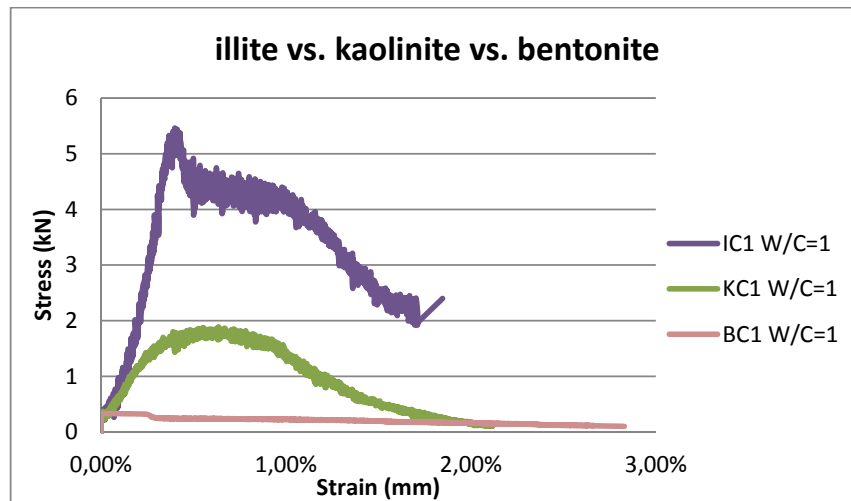


Figure 6.5: Comparison of strength tests among clay types.

A comparison of these results with the observations of Szymkiewicz et al. (2013), show some discordance in the derived conclusions. While he highlights an optimum domain of workability starting from the liquid limit of the soil and going to an optimum value of water content where the maximum strength is achieved at the optimum water content (as already described in chapter 5.2.2), the results of the present research show that the maximum strength is not obtained in the soils mixed at their optimum water content calculated with the method presented by Szymkiewicz et al. (2013). Rather, the maximum strength occurs in the soil mixtures containing the absolute lowest water content. Although these two points of view appear to contrast from one another, they could both be valid with the apparent difference resulting from the way the soil samples were compacted. If the compaction technique is good, as was the one used in the present research, the highest level of strength is reached at water contents around the liquid limit, as explained in chapter 3 and shown by several other authors (Porbaha et al., 2000; Kitazume, 2005). However, because soils, especially low-plasticity soils, at their liquid limit are very stiff and have little flowability, if no compaction or a bad compaction technique is used, the soil will not compact well under its own weight leaving many internal voids and resulting in a lower resistance. Hence, soils with higher water contents can ensure better self-compaction and consequently achieve higher strength.



#### 6.2.2.4. Sand

As can be seen in figure 6.6, as expected, the sand mixture with the lowest water content developed the greatest strength despite the inclusion of equivalent amount of cement. In comparison to clays, the results confirm that non-cohesive materials generate greater strength than cohesive ones.

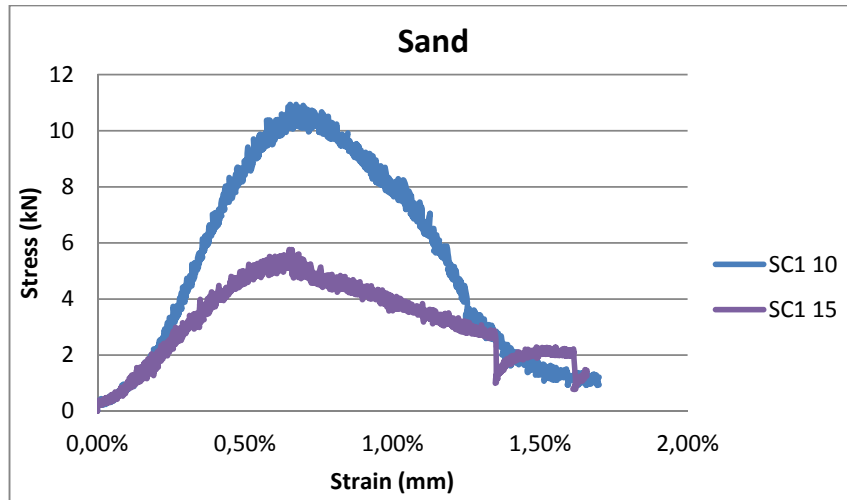


Figure 6.6: Strength test on sand mixture at two different water contents.

#### 6.2.3. Hong Kong soil

The strength tests also were performed on the reconstituted Hong Kong clay, but in this case other variables like the cement type and quantity and the use of additives was altered as described in the previous chapter. Several significant observations were made. First, comparing the two types of cement: it can be noted from the graph below (Figure 6.7) that CEM III mixed with this type of clay develops a much lower strength than CEM I after 28 days, even lower than when the smallest percentage of CEM I was used.

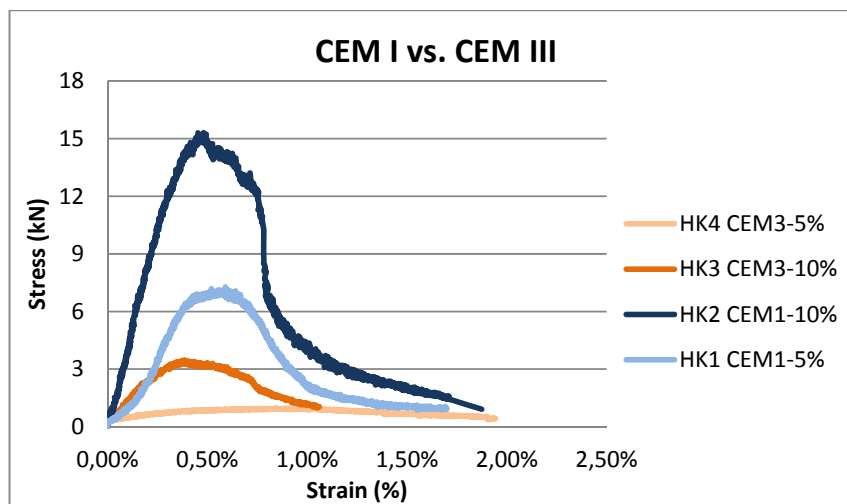


Figure 6.7: Comparison between CEM I and CEM III.

On the other hand, good levels of resistance were achieved with a relatively low percentage of CEM I; the maximum values reached are reported in Table 6.3 in units of resistance [MPa]. The reason for the large difference between the two types of cement could be related to the high percentage of slag in CEM III. Slag is known to demonstrate a delay in the development of resistance (as described in chapter 3). Therefore, it is likely that if the tests had also been carried after periods longer than 28 days, a considerable increase in strength would have been noted. It also can be observed that a doubling in the amount of cement used resulted in approximately a doubling of the maximum resistance reached for CEM I, while for CEM III there was a threefold increase. This can be viewed as a potential advantage of CEM III. Slag is a latent binder that needs to be activated. Typically, there is a specific pH (basic) that activates it, which is generated by the presence of lime present in Portland cement (OPC). However, CEM III contains a high percentage of slag and the amount of OPC is small. When added to the soil, the pH of the soil raises the pH of the mixture. If there is little lime, the pH is raised only slightly for the same soil, while if much more lime is present due to a higher percentage of CEM III, this raises the pH more and thereby enhances the activation of the slag, which significantly increases its resistance. Finally, it can be concluded that the most suitable binder for the Hong Kong project should be CEM I because it gave the highest strength values after a standard curing time of 28 days. Thus, once the required strength value that must be achieved for the on-site project is known, one can be certain that the selection of the best cement will ensure that the minimum amount necessary is added and hence that the minimum amount of water is used. Consequently the volume of spoil material generated also will be minimized. However, if for a real case higher percentages than those investigated need to be added, it is advisable to run another test using the relevant percentages, to determine if CEM I is still the most efficient binder since, as just mentioned, increasing the addition of CEM III has a more amplifying effect on strength than CEM I.

<b>q<sub>u</sub> max [MPa]</b>		<b>q<sub>u</sub> max [MPa]</b>	
HK clay + 10% CEM I	1,95	HK clay + 10% CEM I + ADD	1,57
HK clay + 5% CEM I	0,93	HK clay + 5% CEM I + ADD	0,89
HK clay + 10% CEM III	0,44	HK clay + 10% CEM III + ADD	0,40
HK clay + 5% CEM III	0,12	HK clay + 5% CEM III + ADD	0,07

**Table 6.3:** Maximum values of resistance attained for HK clays in the context of various mix-designs.

A second interesting comparison is between the samples prepared with and without the additive. The purpose of the strength test on these samples was to investigate if there is any effect on the strength gained in the presence of the additive, and if so, by how much. As expected, the presence of the additive slightly compromised the strength gain. The impact though is not very pronounced, except for CEM I at 10% (Table 6.3, Figure 6.8). In conclusion, these results show that at 6% *sodium-lignosulphonate* can be a good additive because it does not significantly influence strength gain. However, because this added

percentage led to a minimum improvement in mixability, it is likely that in an on-site application much higher percentages will be required. Even in this case, therefore, it is recommended that other pre-tests using the relevant percentages have to be performed.

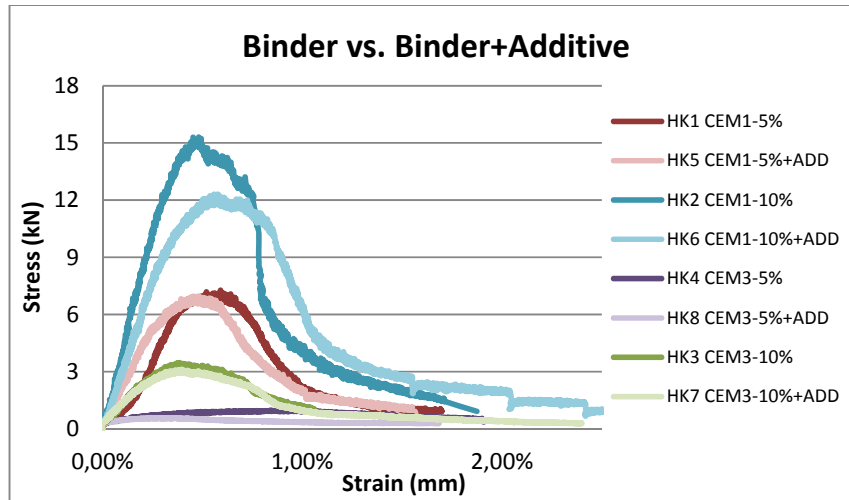


Figure 6.8: Effect of additive inclusion on the strength of Hong Kong clay.

The third series of tests on the Hong Kong clay were performed on a series of samples with different characteristics, as described in the previous chapter. Specifically:

- Two types of additives were tested;
- The cement content in the mixture was the one already planned for use for the site (assuming a normal wet mixing treatment) at a percentage of 30%.

For these tests, the water to cement ratio of the slurry was fixed at 1,2 and the cement used was CEM III. Strength tests were performed to provide an indication of the strength attained with the high percentages of added cement and with a fixed minimum addition of water. The choice of the amount of water to add was based on visual considerations: that is, the water content was the minimum necessary to guarantee a good degree of mixability leading to a smooth treatment and a good degree of homogeneity on-site. In fact, as it is not possible to perform any compacting technique on site, the soil-mixture has to be sufficiently flowable in order to be self-compacting under its own weight to reach good levels of homogeneity.

Furthermore, to enhance the mixability properties of the soil mixture without any supplementary addition of water, two types of additive were introduced, *Sodium-lignosulphonate* and *Bentocryl86*. Thus, their effect on the development of strength by the soil mixture was investigated.

As shown in the graph (Figure 6.9), the highest strength was reached without mixing any additive with the slurry. As expected, the strength value reached was approximately nine times higher than the value reached with 10% CEM III (table 6.3). This response is consistent with the previous tests which demonstrated that doubling the amount of CEM III, enhanced the strength approximately threefold.

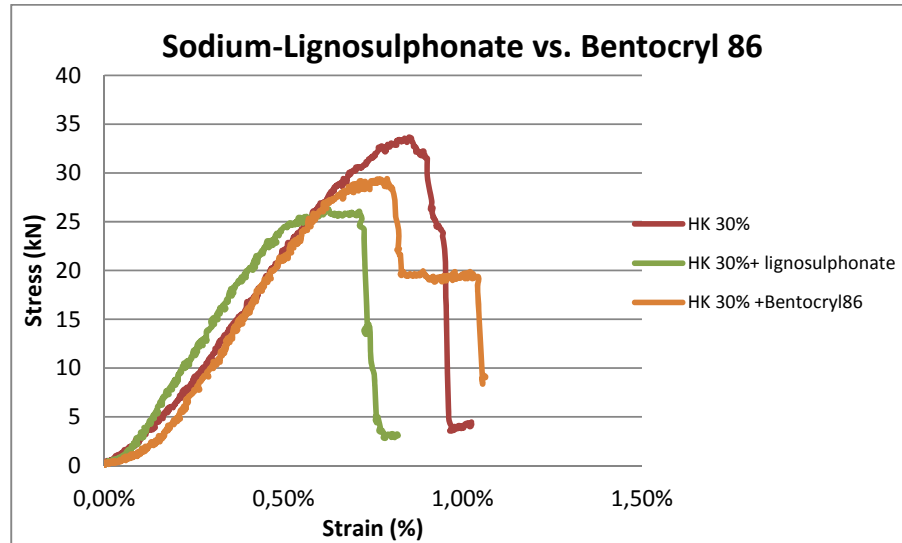


Figure 6.9: Comparison on the effect of two different additives on the strength of Hong Kong clay.

The additives lowered the maximum strength attained but did not modify the breaking properties of the sample which still behaved like a brittle material. In particular, it was noted that the additive with the greatest effect on strength gain was *sodium-lignosulphonate*, while the effect of *bentocryl-86* was smaller (table 6.4).

<b>q<sub>u</sub> max [MPa]</b>	
HK clay + 30% CEM III	4,29
HK clay + 30% CEM III + lignosulphonate	3,36
HK clay + 30% CEM III + bentocryl	3,75

Table 6.4: Comparison on maximum values of resistance attained with the inclusion of two different additives.

Another relevant observation that was made concerned the correct time at which to include the additive. Two series of tests were made with *bentocryl 86* to further characterize the behaviour of the additive. In the first, all the water (natural water content + the water content of the slurry) was added when the bucket of clay was prepared. Then the additive was included, and finally the cement in dry state was introduced. In the second test, the clay was prepared at its natural water content, then the slurry made of water, cement, and additive was included.

In the first test the inclusion of the additive led to a complete liquefaction of the soil which resulted in a total loss of consistency. After the cement addition, however, the final soil mixture was in a state similar to that in the test without the additive. In the second case, the mixing of the slurry with the soil seemed to be easier for the first few minutes, but the final result was the same as the test without the additive.

An interesting idea emerged as a result of this test: if in an on-site application, the additive was to be injected with water during the penetration phase causing the liquefaction of the soil,

while the binder was added in the withdrawal phase, the liquid soil lying above the treated layer would be very flowable and would allow for a smooth lifting of the machine's tool and good homogenization.

The two different procedures also showed a slight difference in the final strength achieved, with the first procedure leading to higher strength (table 6.5). A possible explanation of this is that the soil in a very fluid state permitted a better mixing of the cement and thus the mixture attained better homogeneity.

<b>Hong Kong clay + CEM III + Bentocryl 86</b>	
<b><math>q_u</math> max [MPa]</b>	
1st procedure	3,75
2nd procedure	3,22

**Table 6.5:** Comparison on maximum values of resistance achieved inserting additive in two different times of the mixing procedure.

### 6.3. Mixability tests

As already introduced in chapter 5, the mixability test was developed and performed with the aim of understanding and obtaining an initial assessment of the optimal conditions with which a deep mixing treatment should be performed on site. In this researcher's opinion, in order to optimize the entire mixing process to reduce backflow, it is necessary that the mix design is optimized in accordance with the best working conditions of the machine. These conditions, analysed from the perspective of the soil, are studied with these mixability tests.

#### 6.3.1. Main results of mixability tests

The main results for each test performed with the new apparatus are reported below, as presented in chapter 5.3. At the end, a general conclusion comparing the three tests is presented.

Each clay was tested at four distinct water contents. Specifically:

- $W_n = 70\% - 80\% - 90\% - 110\%$  for Illite, Kaolinite and Hong Kong clay;
- $W_n = 155\% - 175\% - 195\% - 235\%$  for Bentonite.

This range of values was selected because it provides a reasonable range over which to evaluate the variability in the consistency of the material (from very low flowability to very fluid). However, because soils, especially low-plasticity soils at their liquid limit, are very stiff and are poorly flowable, if no compaction or a bad compaction technique is used, the soil will not compact well under its own weight leaving many internal voids that will lower the resistance. This finding contrast with the assertions of Szymkiewicz et al. (2013) that soils at

their liquid limit are self-compacting. If there is no external compaction, only soils with higher water contents can undergo better self-compaction and achieve higher strength.

Although in the laboratory it is possible to homogenize well even very stiff materials and thereby reach higher levels of compaction, in situ it is not possible to reach the same levels of efficiency. In fact, even in a laboratory mixer, the soil at its liquid limit, especially the low-plasticity soils, were very stiff with very little flowability. These characteristics do not satisfy the on-site mixability requirements, especially the one of being a self-compacting material. As a result of these considerations made during the initial part of the laboratory tests, it was decided that for the mixability tests, soils with much higher water contents would be investigated. Bentonite is an exception because, having a high liquid limit, it demonstrated the ability to flow even at water contents slightly lower than its liquid limit.

#### ***6.3.1.1. Test 1 - flowability test inside a confined space: steel pipe***

The test was performed on the three pure soils and on the Hong Kong clay at the four different water contents as described above.

#### ***Data processing***

An initial evaluation was performed in which a graph that represented the course of the test was generated (Figure. 6.10, 6.12, 6.14, 6.16). For each soil tested, how the weight required to lift the instrument varied during the time of the trial was monitored. The trends were very similar, demonstrating that the reliability of the test apparatus was good.

In some instances the trend showed a peak in weight which can be related to a sort of suction effect that occurred when the conditions of the soils were flowable, but which disappeared when the consistency changed at lower water contents or at very high water contents.

After analysing the trends for the tests, the relationship between the effort generated by the engine to lift the instrument and the varying water content of the soil was evaluated. The procedure used to find this correlation was the following:

- Firstly, because the effort required to lift up the cross-section element was quite constant, for every test, an average value of weight was calculated starting from when the values had settled around a constant range;
- Each weight value [kg] was converted to a resistance value [kPa] and plotted against water content. This represents the resistance exerted by the soil as the machine is withdrawn from it. The choice to convert the units to kPa, which is dimensionless with respect to the treatment's surface, was made to enable the graph to be adapted to any deep-mixing, on site application where the relationship between the cross-sectional area occupied by the tool and the cross-sectional area of the bore are the same as for the test situation.
- In processing the values, the weight of the lifted element was subtracted from the total weight to take into account only the component of effort due to the resistance of the

soil. Furthermore, for the soils with the highest water content, the buoyancy component was taken into consideration. Buoyancy most likely does not occur in instances where the water content is low because the material is too viscous and does not maintain continuous contact with the lifted element.

- An interpolation line for the reported points was drawn and a fifth point, corresponding to no effort (i.e. the mixture is mainly composed of water), was fixed to better interpolate the whole;
- The asymptotes of the two branches of the hyperbolic interpolation curve, starting from the two outer points was plotted; they intersect at a specific point corresponding to a defined water content;
- In the calculation of the hyperbolic interpolation, the value of the water content at which the effort goes to infinity was considered the plastic limit of the soil. This water content represents a physical state transition of the material and, at that limit, it is not physically possible to make the soil flowable.

Three interesting observations can be made regarding this data processing procedure:

1. The hyperbolic function is a good fit for the data points (Figure 6.11, 6.13, 6.15, 6.17). This means that for high water content soils, the effort to make the soil flowable through the machine is low and increases slightly as the water content decreases. When the curve changes tilt, the relationship is reversed; that is even a slight reduction in water content can significantly increase the effort required.

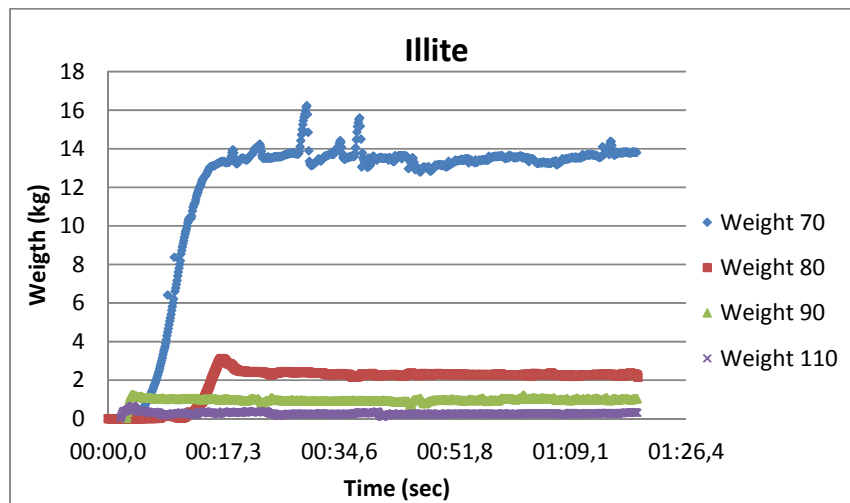
This model demonstrates that there is a limit of convenience, beyond which it makes no sense to decrease the water content otherwise the effort to make the soil flowable during the withdrawal of the machine would become too high and unsustainable.

2. The point corresponding to the intersection of the two asymptotes found through this graphic procedure has practical relevance; this point can be identified as the “*limit of convenience*”. Specifically, it represents the lowest water content for each kind of soil at which it is possible to execute a smooth deep-mixing treatment, while guaranteeing a self-compacting behaviour of the soil-mixture and, hence, good levels of homogenization can be achieved. It is the point at which the deep-mixing procedure can be considered as optimized.
3. A red line was drawn on the graph which represents a calculated limit of water content when the amount of effort required is so high that the withdrawal of the tool can be considered to occur by the pull-out of the soil above it rather than through the exploitation of the soil’s flowability. In particular, it corresponds to a limit where it is almost impossible make the soil flowable, unless it is constrained (in the laboratory test, in fact, the sealed cover prevented the soil from exiting from pull-out). The effort exerted is calculated as the sum of the weight of the soil column above the tool plus the friction stress of the column on the walls of the excavation area due to the pull-out.

A second graph for each soil (Figure 6.11, 6.13, 6.15, 6.17) shows this correlation and the graphic procedure to find the “limit of convenience” as just described.

### *Illite*

The graph in Figure 6.10 shows the trends recorded for the illite clay. It can be noted that after the initial seconds, a relatively constant effort was needed to lift up the cross-section element. While the effort made for the high water content soils was quite low and increased more or less constantly as the water content decreased, when the water content was reduced at the last step, the weight required to lift the instrument increased more than six times. This last test was also the most variable, because the effort required made it more difficult to slide the instrument.



**Figure 6.10:** Measurements obtained from the steel-pipe test for illite at different water contents.

The graph in Figure 6.11 shows the trend for the average values of the steel pipe test. The hyperbolic curve that best fit the points and the red line were drawn, and the “limit of convenience” was determined. It was determined using this criterion that for illite, a minimum water content of the soil of 81,4% is the suggested level necessary to perform a smooth and optimized deep-mixing treatment.



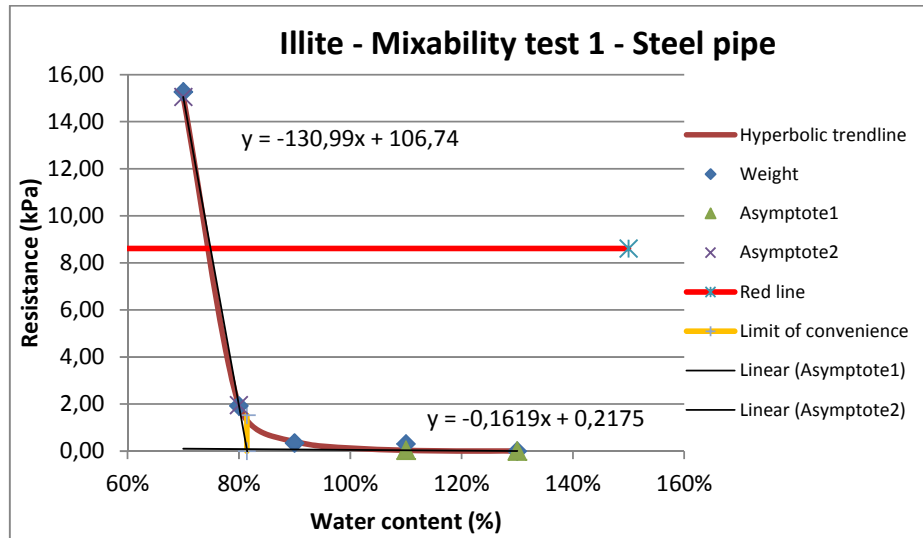


Figure 6.11: Data processing results from test 1 for illite.

**Kaolinite**

The graph in Figure 6.12 shows the trends recorded for the kaolinite clay. Also for this test, the effort required to lift the cross-section element up after the first seconds was quite constant. Whereas the effort required for the high water content soils was quite low, when the water content was reduced step by step, the weight required to lift the instrument doubled. The last two tests are also the most variable, indicating that the effort required makes the sliding of the instrument less easy. A difference that can be observed in a comparison with the illite clay was that, while for illite a water content of 80% still guaranteed a low effort for the lifting of the tool, for kaolinite, the effort required at 80% water content was already quite high (approximately ten times that for illite). Even the step with 70% water content required greater effort. This difference is meaningful because it is yet another demonstration of how two distinct types of clay (even if with similar properties of low-plasticity and low liquid limit) can have a diverse influence in the behaviour of the whole mix.

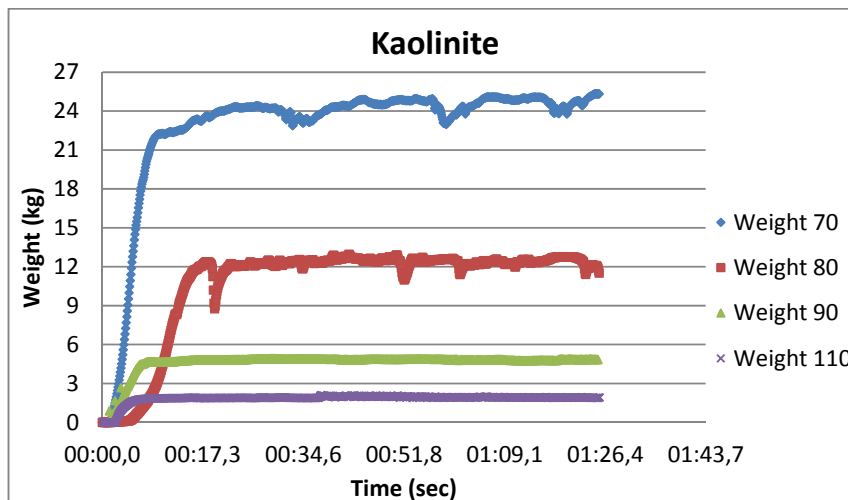


Figure 6.12: Measurements obtained from the steel-pipe test for kaolinite at different water contents.

The graph in Figure 6.13 shows the trend for the average values of the steel pipe test. The hyperbolic curve that best fit the points and the red line were drawn and the “limit of convenience” was found. It was determined using this criterion that for kaolinite, a minimum water content of 85% is required to perform a smooth and optimized deep-mixing treatment. As expected, the “limit of convenience” is higher than the limit for illite.

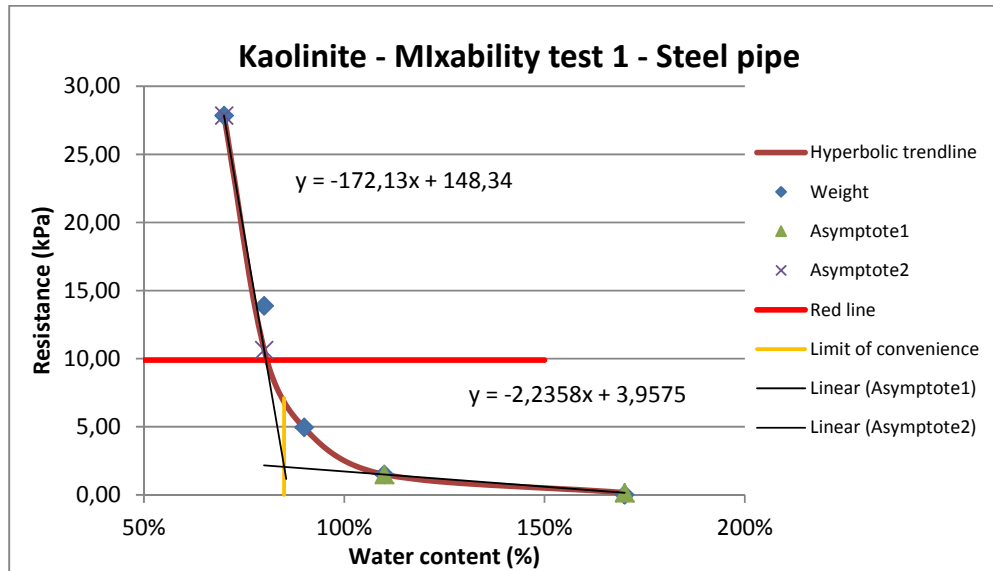


Figure 6.13: Data processing results from test 1 for kaolinite.

**Bentonite**

The graph in Figure 6.14 shows the trends recorded for the bentonite clay. Even in this case it can be noted that after the first few seconds from the start of the test, the effort required to lift the cross-section element is relatively constant. While the effort required for the high water content soils is again quite low, when the water content was reduced stepwise, the weight required to lift the instrument doubled. Similarly, the test with the lowest water content was also the most variable, because the effort required made the sliding of the instrument more difficult. These tests show how the bentonite requires almost double the water content to have properties similar in mixability to the other two clays. This is another example of the diversity in the behaviour among different clay types.

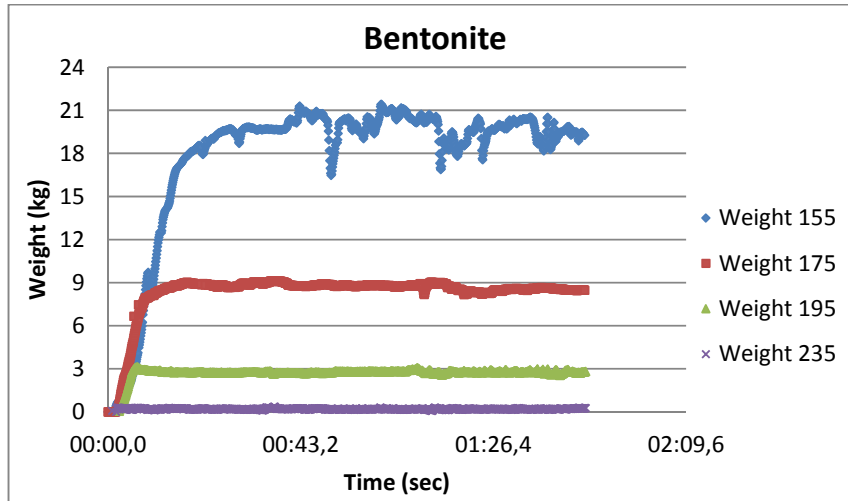


Figure 6.14: Measurements obtained from the steel-pipe test for bentonite at different water contents.

The graph in Figure 6.15 shows the trend for the average values of the steel pipe test. The hyperbolic curve that best fit the points and the red line were drawn, and the “limit of convenience” was determined. According to this criterion, for bentonite, a minimum water content of 182,5% is required to perform a smooth and optimized deep-mixing treatment. As expected, the “limit of convenience” is much higher than for the other clays, but relative to the range of values, it is similarly positioned.

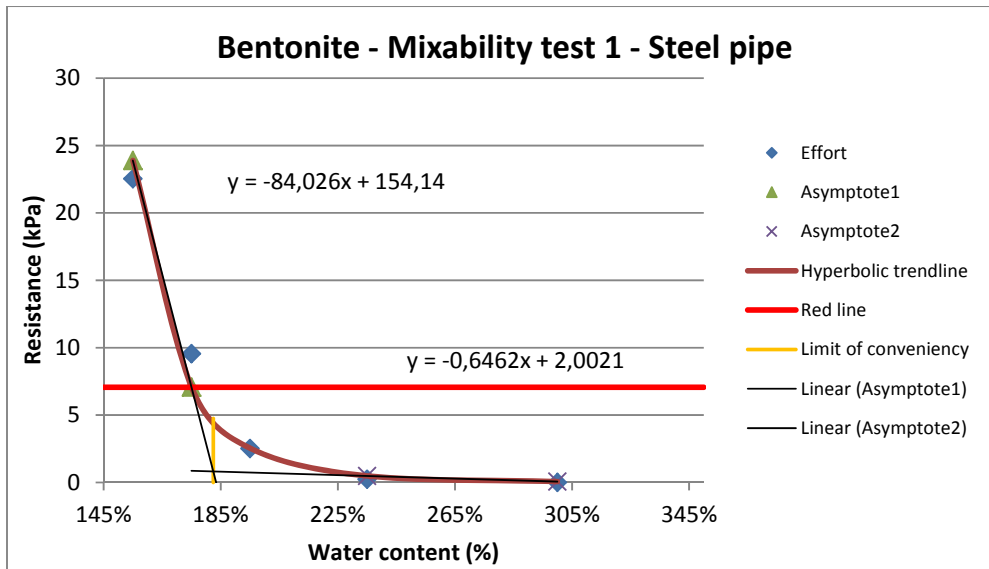


Figure 6.15: Data processing results from test 1 for bentonite.

### Hong Kong clay

The graph in Figure 6.16 shows the trends determined for the Hong Kong clay. As for the other test, after the initial few seconds from the start, the effort required to lift the cross-section element was relatively constant. While the effort required for the high water content soils was quite low, when the water content was reduced stepwise, the weight required to lift the instrument nearly tripled. Again the last test was also the most variable, due to the effort required which makes the sliding of the instrument less easy. It can be observed that both the trend, which shows a tendency for strength to increase by more than twice with each reduction of water content, and the values around which each test settles, are similar to those for illite, confirming the fact that illite is the predominant material present in the Hong Kong clay.

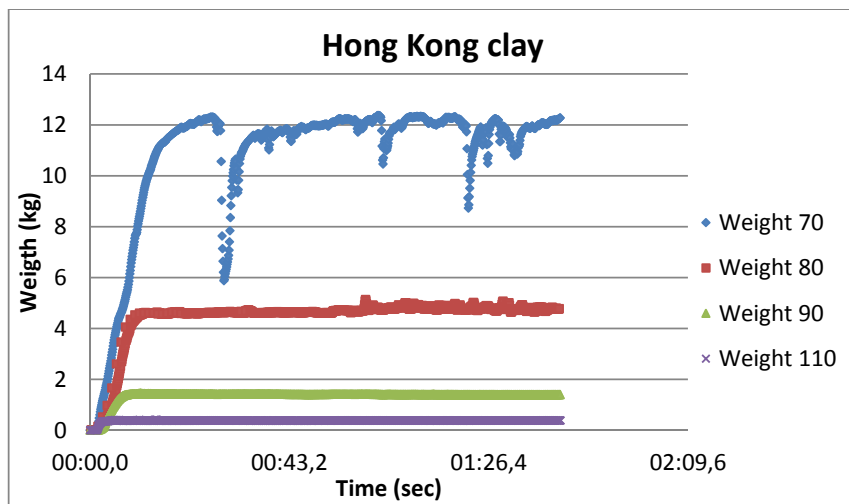


Figure 6.16: Measurements obtained from the steel-pipe test for Hong Kong clay at different water contents.

The graph in Figure 6.17 shows the trend for the average values of the steel pipe test. The hyperbolic curve that best fit the points and the red line were drawn and the “limit of convenience” was determined. According to this criterion, for Hong Kong clay, a minimum water content of the soil of 83,2% is required for performing a smooth and optimized deep-mixing treatment. As expected, this value also was near the typical limit for illite, although a little higher due to the presence in the soil mixture of a not negligible quantity of kaolinite.

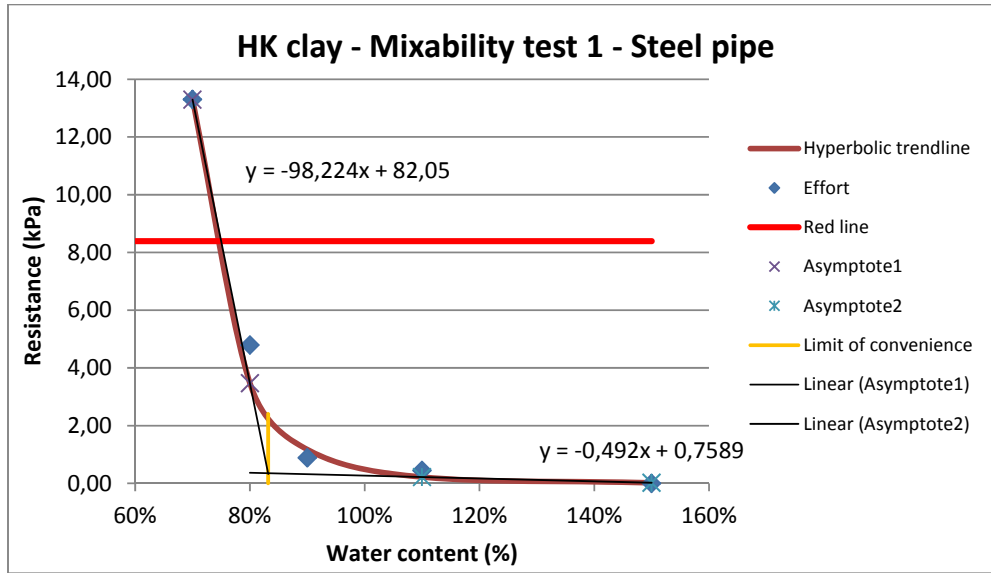


Figure 6.17: Data processing results from test 1 for Hong Kong clay.

### 6.3.1.2. Test 2 – flowability test inside a confined space: adhesion test inside a cell

The test was performed on the three pure soils and on the Hong Kong clay at the four different water contents as described above. However, because it was a little bit more difficult to guarantee a certain level of accuracy with this test due to the very short run time for the test, only the two most meaningful tests (kaolinite and Hong Kong clay) are reported in the thesis.

#### Data processing

The data processing for this test was identical to test 1. Also in this case an initial evaluation was performed in which a graph that represented the course of the test was generated: for each soil tested the weight required to lift the instrument over time of the trial was determined. A good reliability of the test apparatus was shown even for this test, as indicated by the very similar trends obtained among the tests executed.

Even for this test in some cases the trend shows a peak in the weight which can be related to a sort of suction effect that occurred when the conditions of the soils were flowable, but which disappeared when the consistency changed either at lower or very high water contents.

After analysing the trends for the tests, the relationship between the effort generated by the engine to lift the blade and the water content of the soils was determined. The procedure used to find the correlation was similar to that described previously:

- Firstly, an average value for weight was calculated starting from when the values had settled around a constant range.
- Each weight value [kg] was converted to an adhesion value [kPa] and graphed against water content to represent the adhesion offered by the soil to the tool. In this test it was possible to isolate the adhesion feature because the blade did not provide any significant surface for lifting the soil above it. The blade was cutting the soil and the only component responsible for creating resistance from the soil was the adhesion. Thus, to generate its measuring unit [kPa], the weight measured was distributed over the surface area of the blade and its support. It was then possible to have a dimensionless value with respect to any tool's surface which makes the graph applicable to any deep-mixing, on site application.
- In processing the values, the weight of the lifted element was subtracted from the weight measurements to take into account only the component of effort due to the adhesion of the soil. Furthermore, for the soils with the highest water content, the buoyancy component was taken into consideration.
- An interpolation line for the reported points was drawn and a fifth point, corresponding to no effort (i.e. the mixture is mainly composed of water), was fixed to better interpolate the whole.
- The asymptotes of the two branches of the hyperbolic interpolation curve, starting from the two outer points were plotted. They intersect at a specific point that corresponds to a defined water content.
- In the calculation of the hyperbolic interpolation, the value of the water content at which the effort goes to infinity was considered the plastic limit of the soil. This water content represents a physical state transition of the material and at that limit it is not physically possible to make the soil flowable.

Three observations can be made concerning this data:

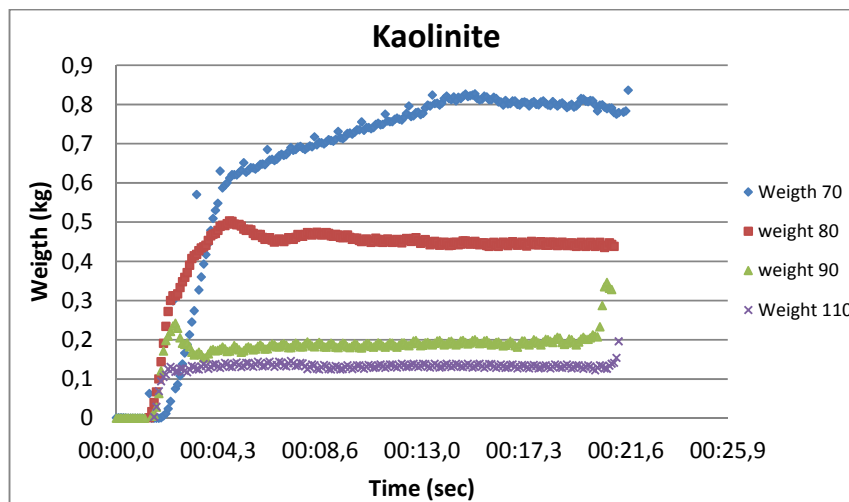
1. The hyperbolic curve also was a good function for interpolation of the points (Figure 6.18, 6.20). This means that high water content soils did not offer high resistance to the lifting of the tool, whereas when the curve changes tilt, at low water contents even a slight reduction in content significantly increased the amount of effort required.  
Consequently, this model also is suitable for determining the “limit of convenience”, beyond which there is no advantage in decreasing the water content further because the resistance offered by the soil becomes so high that the effort required during the withdrawal of the machine would become too great and be unsustainable.
2. The point that represents the “limit of convenience” and that corresponds to the intersection of the two asymptotes was found through the same graphic procedure described for test 1. As for the steel pipe test, it represents the lowest water content for each kind of soil at which it is possible to execute a smooth and easy deep-mixing treatment; it is the point at which the procedure can be considered optimized.

3. No red line was drawn in the graph. As already introduced, in this kind of test the only variable measured was the adhesion; no surface was presented to the soil during the lifting of the element. Thus, for this measurement there was no component that represented the weight of the soil column overlying the tool. The scope was only to understand how large and influent the adhesion variable could be during a deep-mixing procedure.

A second graph for each soil is shown which depicts the graphic procedure described for finding the “limit of convenience”.

### *Kaolinite*

The graph in Figure 6.18 shows the trends recorded for the kaolinite clay. After the initial seconds, a relatively constant effort was required to lift the blade. While the effort required for the high water content soils was quite low, when the water content was reduced stepwise, the weight required to lift the instrument doubled. For the test with the lower water content, it seems that the resistance offered by the soil increased slightly and constantly.



**Figure 6.18:** Measurements obtained from the adhesion test for Kaolinite at different water contents.

The graph in Figure 6.19 shows the trend for the values of the cell test. The hyperbolic curve that best fit the points was drawn, and the “limit of convenience” was determined through the asymptotes. This test suggests that for kaolinite, a minimum water content of the soil of 85,4% is required to perform a smooth and optimized deep-mixing treatment. It can be noted that this value is very close to the value of the “limit of convenience” found with the steel pipe test.

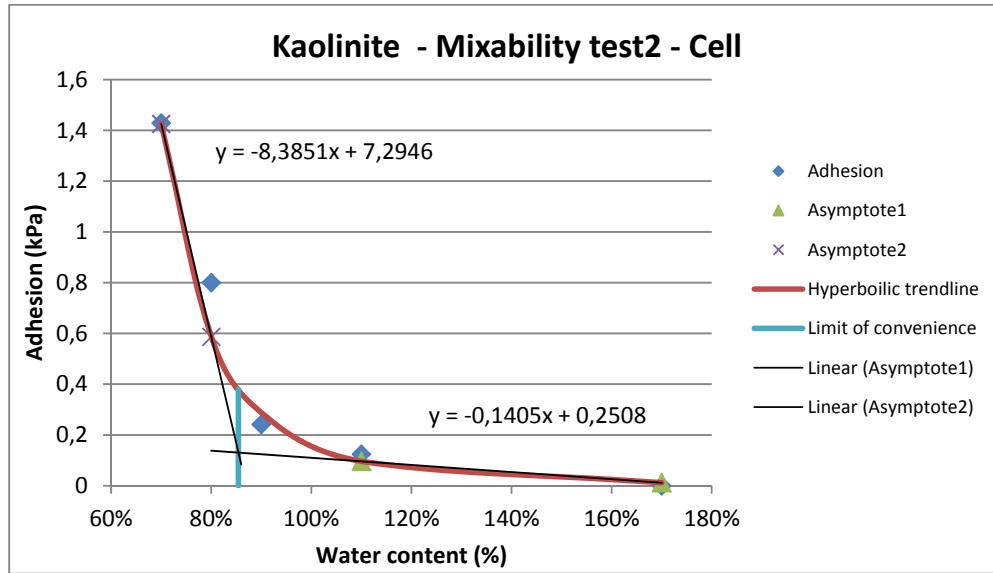


Figure 6.19: Data processing results from test 2 for Kaolinite.

*Hong Kong clay*

The graph in Figure 6.20 shows the trends recorded for the Hong Kong clay. After the initial few seconds from the start of the test, a relatively constant effort was required to lift the blade. While the effort made for the high water content soils was quite low, when the water content was reduced to the last step, the weight required to lift the instrument doubled.

Also in this case, for the test with the lower water content the resistance offered by the soil seemed to increase slightly and constantly.

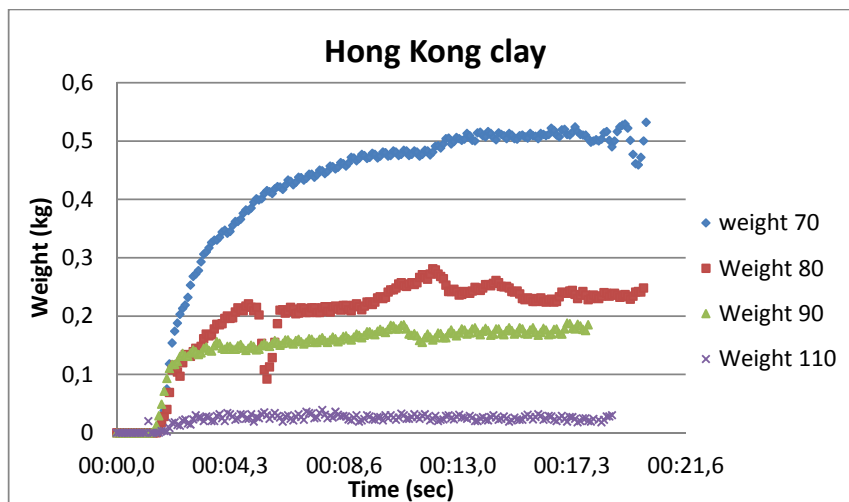


Figure 6.20: Measurements obtained from the adhesion test for Hong Kong clay at different water contents.



The graph in Figure 6.21 shows the trend for the average values of the cell test. The hyperbolic curve that best fit the points was drawn and, through the asymptotes, the “limit of convenience” was determined. This test suggests that for Hong Kong clay, a minimum water content of 84,8% is required for performing a smooth and optimized deep-mixing treatment. It can be noted that this value is very close to the value for the “limit of convenience” determined with the steel pipe test.

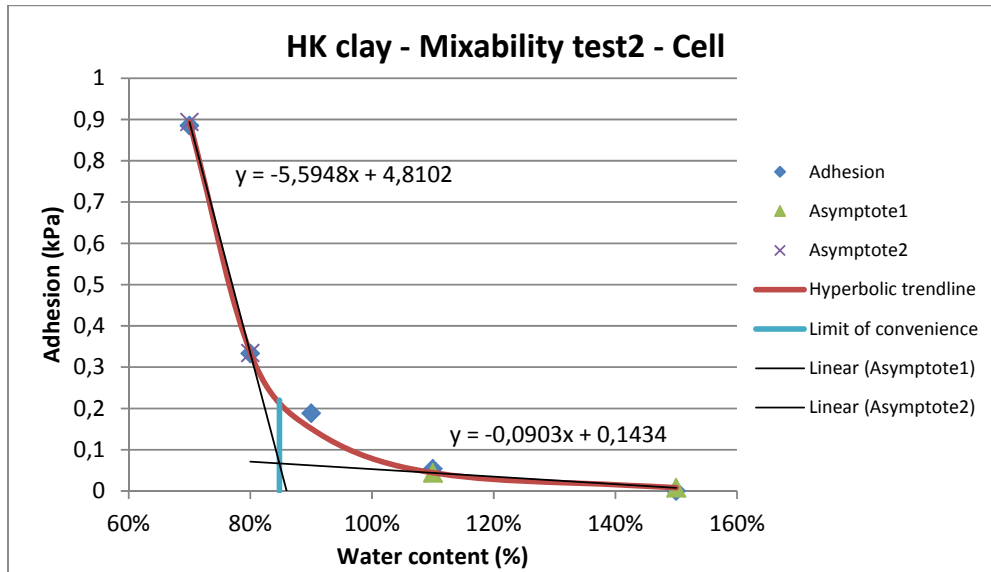


Figure 6.21: Data processing results from test 2 for Hong Kong clay.

### 6.3.1.3. Test 3 – flowability test inside a confined space: torque test inside a cell

The test was performed on the three pure soils and on the Hong Kong clay at the four different water contents as described above.

#### Data processing

The data processing of this test was similar to that for test 1. As already described above, the test was performed with no engines and recording data systems but only with the same blade used for the previous tests which was turned with the apparatus used for the vane test. All values were recorded manually. The objective of these tests was to find a correlation between the torque measurement and varying soil water contents. The procedure used to find the correlation was similar to that described previously:

- Initially, each recorded value was plotted against water content, The measure (taken as  $[t/m^2]$ ) was converted to a torque measurement in  $[Nm]$  and adapted for the special blade used for these tests (because the instrument is calibrated on its standardized blades).

- A hyperbolic curve was fitted to the points and a fifth point, corresponding to no torque (i.e. the mixture is mainly comprised of water), was fixed to better interpolate the whole;
- The asymptotes of the two branches of the hyperbolic interpolation curve, starting from the two outer points were plotted, intersecting at a specific point corresponding to a defined water content.
- In the calculation of the hyperbolic interpolation, the value of the water content at which the effort goes to infinity was considered the plastic limit of the soil. This water content represents a physical state transition of the material and at that limit it is not physically possible to make the soil flowable.

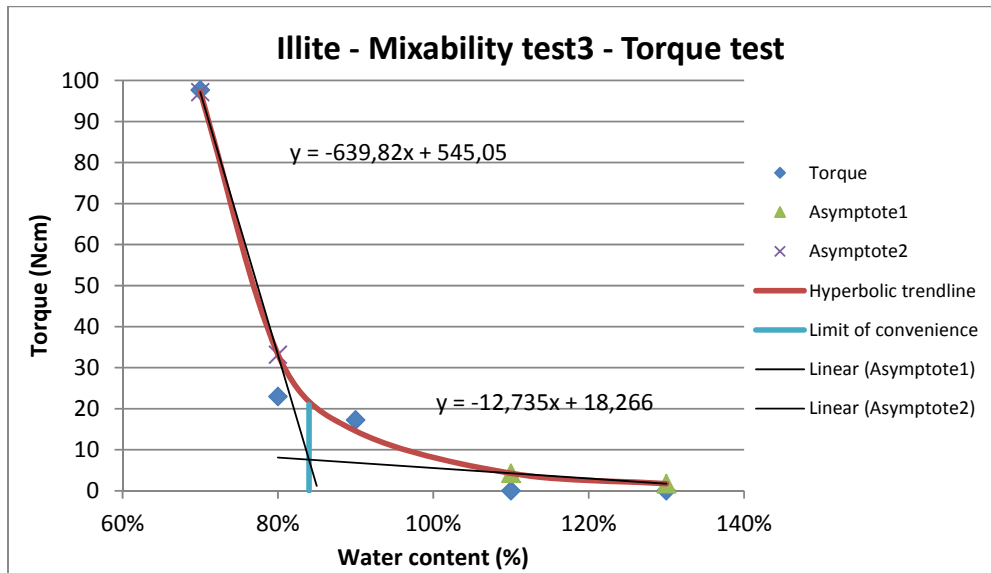
Three relevant observations can be made regarding these data:

1. The torque measurement was performed with a single blade mounted on a support, installed in a sealed cylindrical box full of soil. The aim of the test was to understand, through the rotation of the single blade inside the soil, the propensity of the soil to be displaced and to replace the void left immediately behind the displacement. The soil cannot be lifted and exit from the cell, it can only replace the void and the effort calculated from the torque measurement is typical of the resistance offered from the soil to this movement. This investigated behaviour is representative of the ability of the soil to flow and, hence to the predisposition of the soil to be mixable.
2. Even in this case, there was a tendency for the point to have a hyperbolic distribution (Figure 6.22, 6.23, 6.24, 6.25). This means that high water content soils, did not offer high resistance to the rotation of the tool, whereas when the curve tilt changes, at low water contents even a slight reduction in water content can significantly increase the effort. Thus, this test is also appropriate for determining a “limit of convenience” beyond which it is not reasonable to decrease the water content further because the soil becomes too stiff to be properly mixable.
3. The point that represents the “limit of convenience” that corresponds to the intersection of the two asymptotes was found through the same graphing procedure used for test 1. As for the steel pipe test, it represents the lowest water content for each kind of soil at which it is possible to execute a smooth and easy deep-mixing treatment. It is the point at which the soil is considered adequately mixable and hence the procedure can be considered optimized.

A graph for each soil demonstrates the procedure to find the “limit of convenience” with the torque test as just described.

### *Illite*

The graph in Figure 6.22 shows the results of the torque test measurements. The hyperbolic curve that best fit the points was drawn, and the “limit of convenience” was determined through the asymptotes. According to this criterion, for illite, a minimum water content of the soil of 84% is required for performing a smooth and optimized deep-mixing treatment. It should be noted that this quantity is very similar to the value of the “limit of convenience” determined by the steel pipe test.



**Figure 6.22:** Data processing results from test 3 for illite.

### *Kaolinite*

The graph in Figure 6.23 shows the results for the torque test measurement. The hyperbolic curve that best fit the points was drawn and, through the asymptotes, the “limit of convenience” was determined. According to this criterion, for kaolinite, a minimum water content of the soil of 87% is the suggested lowest value suitable for performing a smooth and optimized deep-mixing treatment. It can be noted that this value also is very similar to the value of the “limit of convenience” found with the steel pipe test.

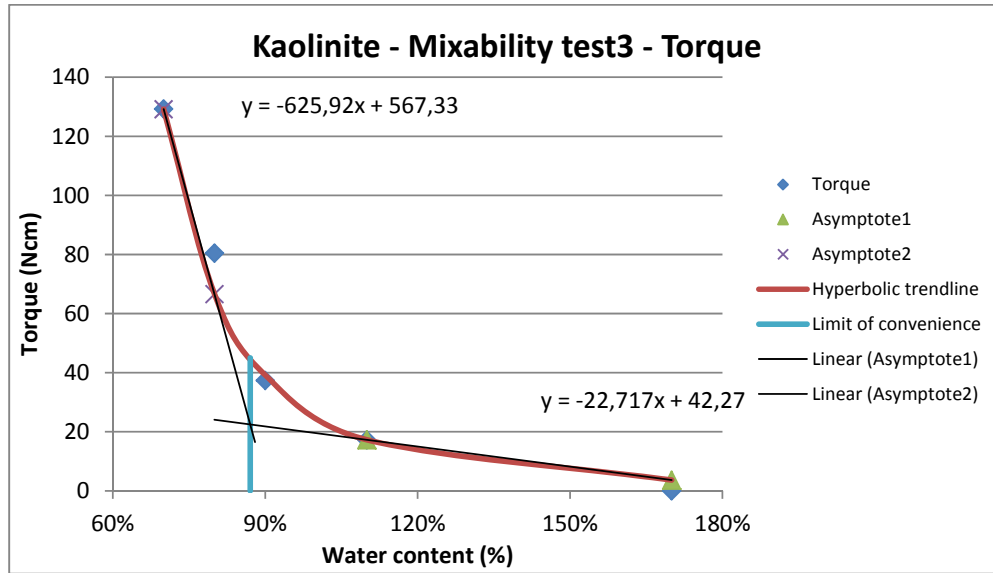


Figure 6.23: Data processing results from test 3 for kaolinite.

**Bentonite**

The graph in Figure 6.24 shows the results of the torque test measurements. The hyperbolic curve that best fit the points was drawn and, through the asymptotes, the “limit of convenience” was determined. For bentonite, a minimum water content of the soil of 185,3% is the suggested lowest value suitable for performing a smooth and optimized deep-mixing treatment. This value also is very similar to the value of the “limit of convenience” found with the steel pipe test.

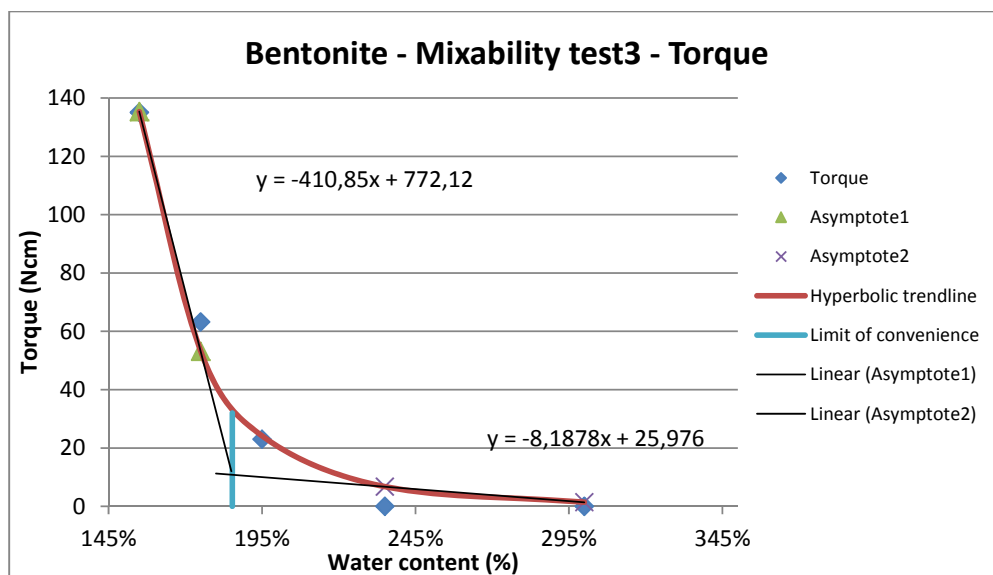


Figure 6.24: Data processing results from test 3 for bentonite.

### Hong Kong clay

The graph in Figure 6.25 shows the results of the torque test measurement for the Hong Kong clay. The hyperbolic curve that best fit the points was drawn and, through the asymptotes, the “limit of convenience” was determined. According to this test, even for Hong Kong clay, a minimum soil water content of 86,2% is the recommended for performing a smooth and optimized deep-mixing treatment. It can be noted that this value also is very similar to the value of the “limit of convenience” found with the steel pipe test and similarly this value is near the typical limit for illite, although a little higher due to the presence in the soil mixture of a not negligible quantity of kaolinite and other materials.

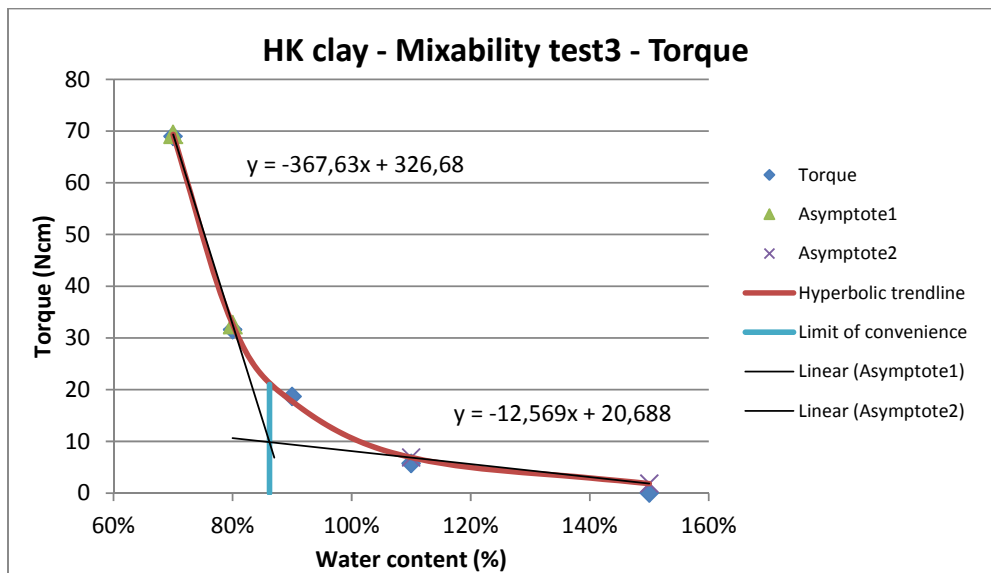


Figure 6.25: Data processing results from test 3 for Hong Kong clay.

### Haegermann Table

As already described in chapter 5, for application of the new apparatus, a comparison was made to the analysis of a soil mixture using a standardized tool; in this case, the *Haegermann table* that classifies soils according to their rheological properties (consistency, flowability) was compared. However, as reported in the results below (Table 6.6), this instrument works over a limited range of soil water contents and it is probably not the best instrument for classifying soils. For example, spreading was not homogeneous for very liquid soils, and the soil flowed out of the table. Indeed, as can be seen in the Table 6.6, spreading of the material after 15 jolts was not constant; the differences in diameter and in height of the shape of soil before and after 15 jolts were too variable and did not increase or decrease consistently with the water content. This behaviour is probably governed by the different properties of plasticity and adhesion of each soil at different water contents. Regardless, these results also demonstrate that the behaviour of soils is very diverse.

<b>HEGERMANN TABLE – DIFFERENCE VALUES</b>									
W <sub>c</sub>	Bentonite		W <sub>c</sub>	Illite		Kaolinite		HK clay	
	$\Delta\phi$ [cm]	$\Delta h$ [cm]		$\Delta\phi$ [cm]	$\Delta h$ [cm]	$\Delta\phi$ [cm]	$\Delta h$ [cm]	$\Delta\phi$ [cm]	$\Delta h$ [cm]
155%	0,6	0,2	70%	3,75	0,8	3,1	1	3,3	0,8
175%	2,2	0,6	80%	6	2,8	4,4	1,9	6,4	2,8
195%	5,5	1,8	90%	3,7	1	5,8	1,7	5,9	1
235%	-	-	110%	-	-	7,7	1,3	-	-

**Table 6.6:** Diameter and height differences in the values of the Haegermann table test performed on each clay.

### 6.3.2. Further considerations and future proposals

Regarding the strength tests:

- ❖ These research tests focused on developing the best mix-design that would yield the highest values of strength gain. However, permeability, another important engineering characteristic that is often required in deep-mixing applications was not considered. Future studies will need to evaluate this variable in the study of the mix-design.
- ❖ The number of tests performed, and the number and the manner in which the variables were analysed was limited by the time available for using the laboratory. In spite of this limitation, the testing program was designed to be as thorough as possible to meet the objectives of this research. However, in order to further this type of research it would be useful to perform additional tests in the future in order to compare different mix-designs using multiple variables.

Concerning the mixability tests:

As shown in Table 6.7, for each soil, the three tests gave a very similar value for the “limit of convenience”.

<b>LIMIT OF CONVENIENCE</b>				
Test	Illite	Kaolinite	Bentonite	HK clay
Test 1 - Steel pipe	81,4%	85,0%	182,5%	83,2%
Test 2 - Cell	-	85,4%	-	84,8%
Test 3 - Torque	84%	87%	185,3%	86,2%

**Table 6.7:** Diameter and height differences in the values of the Haegermann table test performed on each clay.

Table 6.7 shows that the maximum difference in the values is within a range of a water content variation of 3% (Hong Kong clay). This range is very tight suggesting that the measurement is realistic despite the fact that:

- the criterion used to find the “limit of convenience” is based on a mean of recorded values;
- the values generated include variability due to measurement error;
- the interpolation performed to determine the “limit of convenience” also includes measurement error associated with the graphing approach.

Hence, because the range of variability of the “limit of convenience” for each soil is very narrow, the results from the three tests performed with the new test apparatus can be considered to be equivalent. Thus, for further applications, the execution of only one of the three tests will be sufficient to provide an indication of the water content for mixability properties of a given soil.

Finally, the internal consistency of the results obtained for each test for each soil, suggests that reliability of the test apparatus is acceptable.

- ❖ The graph in Figure 6.26 illustrates the characteristics of the hyperbolic curve used to interpolate the data for each soil tested. These values were derived from the steel pipe test but the trends of the hyperboles for the other tests were similar.

The equation that describes the hyperbole used for these interpolations is the following:

$$y = \frac{1}{(x - x_0)^\alpha} \cdot C$$

Where:

- $y$  is the value corresponding to a resistance measurement;
- $x$  corresponds to the water content of the soil ( $w_c$ );
- $x_0$  represents the water content corresponding to the plastic limit (LP);
- $\alpha$  is the exponent;
- $C$  is the constant of the curve.

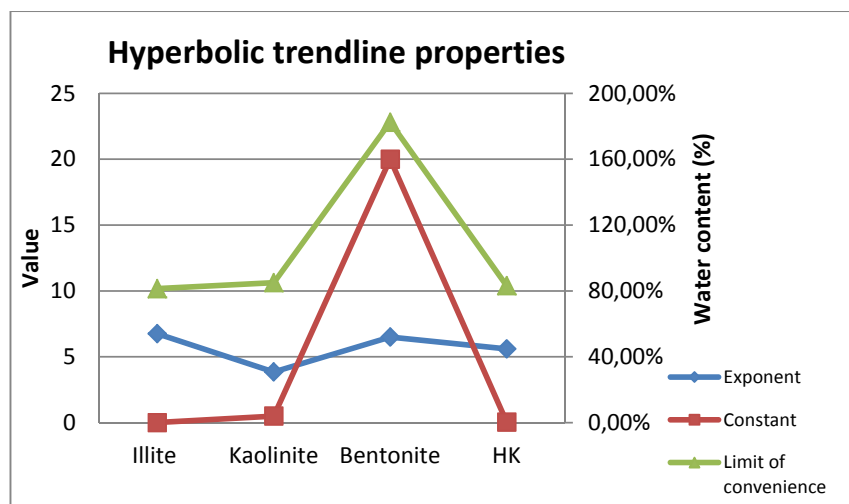


Figure 6.26: Properties of the hyperbolic curve used for interpolation of the data from the various tests.

As shown in Figure 6.26, the “limit of convenience” is strongly influenced by the value of the constant and that the typical variables for the Hong Kong clay are consistently similar to those for illite, its predominant component.

- ❖ An important factor that must be evaluated is the influence on the results of the tests’ speed. The tests were conducted at only one, predefined, lifting speed. The assumed speed (0,5 cm/sec) was chosen because it is a typical withdrawal speed used for deep-mixing applications (Kitazume, 2013). Hence, it has to be considered that the speed is another fundamental variable should be accounted for when planning a deep mixing treatment. Undoubtedly lower speeds would result in lower resistance values while higher speeds would lead to higher resistance values.

A low speed during a deep mixing application allows for the use of a lower water content and also permits the blades to remain for longer in the same soil layer for a better mixing. This can be a relevant consideration when the aim is to reduce spoil material production. On the other hand, a lower speed increases the time required to perform the work which inevitably increases the total costs.

- ❖ Further developments are proposed for the new test apparatus:
  - The structure of the whole apparatus can be developed to make it easily portable thereby making it possible to perform the tests directly on-site;
  - The torque measurement can be improved by automating the test like the other two. This requires the installation of a motor that permits a constant rotation of the blade in the cell and the use of an electronic apparatus specific for the torque measurement.







## CONCLUSIONS

Spoil material has become one of the most critical factors that must be evaluated when planning deep mixing at a jobsite. The amount of spoil material produced is usually very high and the costs for its transportation, storage, and disposal have become prohibitive, especially if the treatment involves contaminated soils. Currently, spoil production is high, from approximately 50% to 60% or even up to 100% for clayey soils. In the past, solutions based on economic and environmental considerations have been proposed to deal with the problem of disposal. In contrast, the present research has developed a solution for preventing the formation of backflow. After a very detailed study on soil characteristics, the solution was derived through a research program that combines reducing the slurry's water content to a minimum level, with optimization of the mix design and the features affecting the mixability of the deep-mixing structure.

A brief summary on the most important results from the testing program is reported here.

### *Strength tests*

Strength tests have been very useful for understanding how to optimize the mix design, with the goal of reducing the volume of spoil material, for any deep-mixing application. Since these tests were conducted mostly on pure soils and on a reconstituted soil as an example, the results can provide good guidelines for on-site conditions where the soil is composed of different materials. Indeed, it is helpful, to think that the predominant material of a given site will be the one that most influences the performance of the complete soil mix. This research, therefore, provides an approach to predict how the soil will tend to behave in situ. Nonetheless, it is advisable to perform laboratory tests on the soil to be treated before the beginning of each construction site, in order to validate the effectiveness of the mix design chosen for the specific deep-mixing procedure.

The role of each investigated variable can be summarized as follows:

- **Soil.** The type of soil is a fundamental variable; it has been demonstrated in this research, for example, that two kinds of clay can have very different features and, hence, can radically affect the behaviour of a complete soil mixture. In particular, the distinction is very marked between cohesive and non-cohesive soils, as well as between high-plasticity and low plasticity soils. It is interesting that a difference in

behaviour can be observed even between soils with similar properties, such as illite and kaolinite. This means that before an on-site deep-mixing application, while planning an appropriate mix design, the characteristics of the soil to be treated, from the microscopic level to its chemical composition, must be studied in depth and well-understood. Finally, given a constant type and quantity of cement, sand is the soil that gives the highest strength gain while, among the clays, illite is the best. Bentonite provides the worst conditions for strength development and probably needs more cement than the amount injected in the test to develop any resistance.

- **Water.** The research performed has demonstrated that the highest strength gain is obtained from the soil mixed with the lowest possible amount of water. However, this test was conducted using high mixing energy and a good compaction technique which produced a very homogeneous mixture. As already explained previously, if these two features are lacking, this theory would partially lose its significance and the water content would need to be enlarged to guarantee mixability and self-compacting properties of the soil mixture in order to reach the required strength values of the treated element.
- **Binders.** Soil mixtures that contained one of two types of binders in different quantities were examined. As expected, for both binders, the higher the quantity of binder included, the greater the strength achieved. CEM I is the binder that gave the greatest strength values after 28 days. CEM III, however, showed a proportionately greater increase in strength gain than CEM I when their quantities were doubled. Additionally, CEM III is known to be retardant in strength development and, thus, it would be useful to investigate its behaviour after more than 28 days.
- **Additives.** Two types of additives were considered. The aim was to find an alternative solution to increase the soil mixability without increasing its water content. As expected, the inclusion of additives led to lower strength. *Bentocryl 86* seems to be a better additive than *Sodium-lignosulphonate* to use for deep-mixing applications because it had the best effects in terms of gain of mixability and resulted in a lower loss in strength of the cemented product. However, only two kinds of additives with a single fixed quantity were tested. These results provide a preliminary idea, but as the use of additives seems to be a promising solution for the mix-design of deep-mixing works, it would be desirable to run more specific tests in the future to determine how, in this kind of applications, the best efficiency can be attained from the use of additives.

Additional research in the future would further advance developments in this field. First, it would be useful to conduct additional tests to compare different mix-designs with multiple variables; for example, the evaluation of more types of binders and additives and, hence, more combinations with different water contents could be productive. It also would be interesting and fulfilling, to add some triaxial tests to study the soil's behaviour taking into consideration the confinement component, as well as tests to take into account variations in permeability that sometimes represents a fundamental requirement of such work projects.

### ***Mixability tests***

The mixability tests were fundamental for completing the characterization of a deep mixing process. The research examined the most suitable mix-design, while guaranteeing the smoothness and easiness of the mixing procedure in order to have an optimized deep-mixing treatment and minimizing the amount of backflow.

The new test apparatus was developed during the time dedicated to this research with the idea of introducing a new testing procedure specifically created for soil mixing applications. In particular, this study wanted to simulate the mixability conditions and, especially, the effort required to make the soil able to flow through the spaces surrounding a mixing tool during the withdrawal phase of a deep-mixing treatment. The final aim was to estimate the ideal mixing conditions in clayey soils. This was possible through identification of the point of minimum water content at which the optimal working process for the machine is still guaranteed.

Three kinds of tests were performed to investigate three different aspects of mixability:

- ***Test 1.*** The idea behind this test was to simulate the behaviour of the complete machine-soil complex during the withdrawal phase of a deep mixing treatment. Through the measurement of the force developed to lift the tool, one can evaluate both the propensity of the soil to be flowable and the effort that has to be carried out by the machine to extract the tool from a specific mixed soil, without lifting up the soil itself.
- ***Test 2.*** This measures the effect of the clay's adhesion on the instrument. In this way the significance of the adhesion effect for each kind of clay at different water contents can be demonstrated.
- ***Test 3.*** The idea behind this test was to understand the propensity of the soil to flow when moved. Test 1 and Test 3 investigate the same property of the clays. The main differences between the two are that a torque instead of a force is measured. However, in Test 3 the measurement is not influenced by the effective weight of the tool and the weight of the overlying column of soil unlike Test 1.

After the execution of the tests, a graphic procedure for data processing was elaborated to find meaningful correlations. The most important results obtained can be summarized as follows:

1. A hyperbolic curve describes the trend for the effort necessary to mix soils each with varying water content. It provides an immediate first assessment on how, beyond a certain limit, even a minimal reduction of water content requires a considerable increase in the effort that must be exerted.

2. In accordance with the goal of reducing the volume of spoil material, the objective was to obtain an initial indication of the ideal conditions of mixability. From the analysis, a meaningful point can be defined. This point, termed the “limit of convenience”, represents the lowest water content for each kind of soil at which it is possible to execute a smooth deep-mixing treatment, guaranteeing a self-compacting behaviour of the soil-mixture and, hence, achieving good levels of homogenization. It is the point at which the deep-mixing procedure can be considered optimized.
3. Each of the three tests represents one or more properties of mixability. When the same graphic procedure was applied to the data collected from each test, the results were essentially equivalent: each test identified for each soil a very similar “limit of convenience”. Hence, the execution of only one of the three tests would be sufficient to provide an indication of the water content for mixability properties of a certain soil.

A final deliberation regarding the importance of water content should be addressed. This fundamental variable needs to be calibrated on the basis of two features: it needs to be lowered as much as possible to enhance the strength gain and to reduce the generation of spoil material, whereas it should not be kept too low so as to guarantee the soil’s mixability conditions. The “limit of convenience” perfectly represents the alignment between these two features.

Additionally, further developments are proposed for the new test apparatus:

- The structure of the whole apparatus can be developed in order to make it easily portable thereby generating the opportunity to perform tests directly on-site. Although three tests were performed, the results indicate that developing the apparatus to perform only one of these tests would be sufficient and more practical. Additionally, the torque measurement can be improved by making the test automated like the other two tests. The idea would be to install a motor that permits a constant rotation speed of the blade in the cell and to use a specific electronic apparatus for the torque measurement.
- Another possibility would be to perform additional tests with this new apparatus, in which not only the behaviour of pure soils is analysed, but also the behaviour of soils treated with only binder, soils treated with only additive, and finally soils treated with binder and additive. Additionally, the tests should be repeated varying the lifting speed. The objective of expanding the testing program is to complete the analysis of different mixability conditions taking into consideration how the “limit of convenience” varies with different mix-designs and varying the deep-mixing procedures.

### ***Hong Kong Project***

The idea behind the Hong Kong project was to apply the complete testing program to a complex real case, reconstituting the Hong Kong soil in the laboratory. This kind of research is easily applicable to a pre-qualification testing program to create guidelines for planning a specific jobsite. In the future it will be important to make this procedure more applicable and reliable. This will be possible in two ways: by broadening the range of tests to real, treated soils and by making the new test apparatus portable with the possibility of running tests directly on-site.

Research until now has developed only a few preliminary studies, but the need for development of approaches to minimize the generation of spoil material is becoming fundamental. The results obtained from this detailed testing program, have introduced an innovative perspective on which to base the planning of a deep mixing application. In particular, this research can be seen as providing new guidelines for the reduction of backflow through an optimization of the entire process in order to obtain suitable benefits from both economic and environmental points of view.



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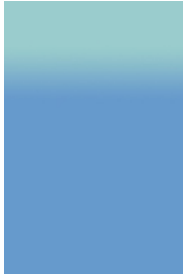
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