Mechanical behaviour of conditioned material for EPBS tunnelling

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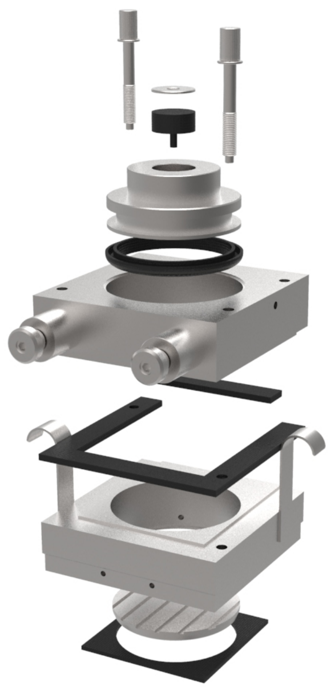

PhD Program in Environment and Territory
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Mechanical behaviour of conditioned material for EPBS tunnelling

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Summary

The rapid growth of the development of the cities all over the World, brought the necessity of bringing deeper the services and all the activities which are not strictly necessary above the ground like houses. This sudden demand of new tunnels obliged to push the excavation industry towards mechanized methods which allow to avoid settlements on the surface, where other structures and infrastructures are located. In this context EPB shield machines play a crucial role, as with a good control of this technology, a tunnel can be excavated basically everywhere, also below important structures. This implies the perfect knowledge of the geology but especially requires a precise study of the soil conditioning, in order to allow an effective counterpressure to the front.

The development of preliminary laboratory tests, which means before the tunnel project starts, allows to assess the best conditioning set for each lithotype which can be encountered during the excavation. These tests are performed at room pressure, nevertheless recently the main goal is to study the conditioned mass at pressure conditions which can be found in an excavating chamber, which might influence the state of the mass itself.

The aim of this work is the development of new techniques which can exactly reproduce this state, through the use and modification of techniques proper of the geotechnical engineering (shear and triaxial tests) and the design of new devices able to underline these aspects in detail. The new approach includes, as well as the consideration of a certain pressure condition, also the definition of an undrained condition used for testing, which allows to keep the conditioned mass in its original state for its study.

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Part I

General aspects and state of art

Chapter 1

EPB tunnelling and conditioning technology

1.1 Introduction

The globalization that we are experiencing in the last years brought people to move in urban areas, which, as a consequence, are more and more expanding. Unfortunately, the available livable space above the ground in these areas is reducing; therefore, the possibility of using underground spaces has been investigated in the last decades . The trend is to keep the open spaces for living and concentrate the services and other correlated activities in underground; in fact, its utilization offers a new approach to urban planning and infrastructure.

A crucial role in this globalization phenomenon is played by the transportation and by the public services such as aqueducts, sewages and power lines. The expansion of the cities is proportional to the need of these services, but this brought also the problem of disturbing the traffic and the activities above the ground during the construction and the development of underground facilities. A new philosophy in constructing tunnels through trenchless techniques has been developed during last years, in order to reduce and avoid problem to the ground activities. This led to an extensive use of the EPB technology in urban area, in this way the activities are prevented by any disturbance and the safety of the structures above the ground is guaranteed (Guglielmetti et al., 2007).

The excavation in urban areas is particularly complex, as numerous criticalities have to be taken into account, and above all (Kovári, 2004):

- low overburdens;
- existing structures in surface;
- presence of unknown elements along the tunnel route;
- restrictions of alignment, access to the tunnel and site investigations;
- complexity on improve the quality of the overburden soil from the surface.

The need, which rose a lot in the last 30 years, of excavating in this insidious and challenging environment brought the tunnelling community to an extensive use of mechanized shielded tunnel boring machines (TBM). This technique provides an effective system to support the tunnel: the walls by mean of the shield itself and the front by mean of a counterpressure which is able to limit (it is impossible to reset) the deformation (Anagnostou and Kovári, 1996a). In particular, the stability of the face in particular is a fundamental problem for the relevant consequences on surface subsidence, instability on the local front or, in the worst condition, for the formation of voids that can affect surface structures and the safety of the personnel in the tunnel. In addition, the pressurized excavation chamber keeps the groundwater independent from the excavation process, preventing the lowering of the water table itself and the resulting relative subsidence. The first successful tunnelling shield was developed by Sir Marc Isambard Brunel in 1806 to excavate Thames Tunnel in 1825. However, this he is known for the invention of the shield concept but he has not been involved in the construction of a complete tunnel boring machine, the digging still having to be accomplished by standard excavation methods. Although really different from the present technologies, the principle was not too much different from the modern TBM, as Brunel's machine was designed to protect the site by mean of a shield.

The shielded machines are designed to directly install the final lining, composed of precast concrete segments which are able to keep the walls stable once that the shield is advancing. The general principle of the shield is based on a cylindrical steel assembly pushed forward on the axis of the tunnel while at the same time excavating the soil. The steel assembly secures the excavated void until the preliminary or final tunnel lining is built. The shield has to withstand the pressure of the surrounding ground and, if present, prevent the inflow of groundwater.

Depending on the type of counterpressure at the front used, the shields are classified as:

- *Compressed air Shields*, which pressurize the air inside the excavating chamber, thus the counterpressure is guaranteed by a bubble of air pushing the soil ahead. The flaw of this technology is the difficulty of controlling this pressure when the soil ahead the excavating chamber is too permeable: in this case, the air is not able to push effectively the soil but tends to flow through the intergranular voids;

- *Slurry Shields* and *Hydroshields*, which excavate and keep the soil stable by mean of a bentonite suspension. Its pressure is controlled by a pressurized air bubble, as the suspension is almost liquid in the chamber. In this case, the mucking too is performed through the bentonite suspension, which is pumped away from the tunnel with pipelines;

- *Earth Pressure Balance Shields*, which excavate and use the muck, conveniently conditioned by means of conditioning agents such as water, foam and polymers, to create a front counterpressure, which keeps the front stable.

1.2 EPB machines

1.2.1 General aspects and operating principles

EPB shield technology is widely used in soil excavation, especially in urban areas, because of its effectiveness to control surface settlements and to carry out the work in safe conditions. The safety is increased if we consider the fact that the front can be stabilized by using the earth pressure generated in the excavation chamber. Moreover, with this method, the presence of water, both in saturated media and as seepage force, can be easily handled reducing risks for the manpower. For these reasons, several underground projects are realized by using this technology. An earth pressure shield worldwide was first employed in 1974 for the construction of a sewer tunnel in Tokyo, Japan (Naitoh, 1985). The ground in East Asia is often fine-grained and of soft consistency, the classic area of application for earth pressure shield machines.

In order to prevent the ground being excavated from entering the excavation chamber in an uncontrolled fashion, EPB shield machines support the standing face by the already excavated spoil, which is under pressure in the excavation chamber and is then transported by a screw conveyor out of the excavation chamber into the area of the machine under atmospheric pressure. In order to balance the pressure difference between the support pressure at the face and the atmospheric pressure in the tunnel, the appropriate pressure gradient has to be dissipated along the length of the screw conveyor. The functioning of active face support in an earth pressure shield is explained in Figure 1.1. The ground is excavated by the cutting wheel (1) and transported into the excavation chamber (2), at the back of which is the pressure bulkhead (3). The support pressure in the excavation chamber is regulated through the control of supply (advance rate of the thrust presses (4); addition of conditioning agent) and removal (volume transported by the screw conveyor (5)). After the required stroke length of the thrust presses has been bored, the erector (6) is used to install the segments (7) (Herrenknecht et al., 2011).

Basically EPB shield operation principle relies in a balance, at the front of the machine, between excavated and extracted material achieved through regulation of both the screw-conveyor rotation speed and the cutting wheel advance rate. Figure 1.2 exhibits the equilibrium condition.

As the tunnel machine advances, pressure on the front is controlled by acting on the screw extraction speed (variation in speed of rotation – rate of opening of the screw discharge gate) and the thrust monitored by pressure detectors mounted on the rear wall of the muck chamber(bulkhead). This system of regulating directly influences control over front stability. This means that the excavated material has to be kept under pressure in the muck chamber and, consequently, that the volumes of material excavated and discharged have to be kept equal at each instant; this is why the speed of rotation of the screw has to be controlled relatively to the constant advance speed of the shield, the pressure in muck chamber being kept greater by about 0.2 bar than the pressure exerted by the ground (earth pressure + the hydraulic load) (Peron and Marcheselli, 1994).

The EPB shield together with the slurry shield have been the most used technologies in the last decades, as they represent the most reliable solutions to apply

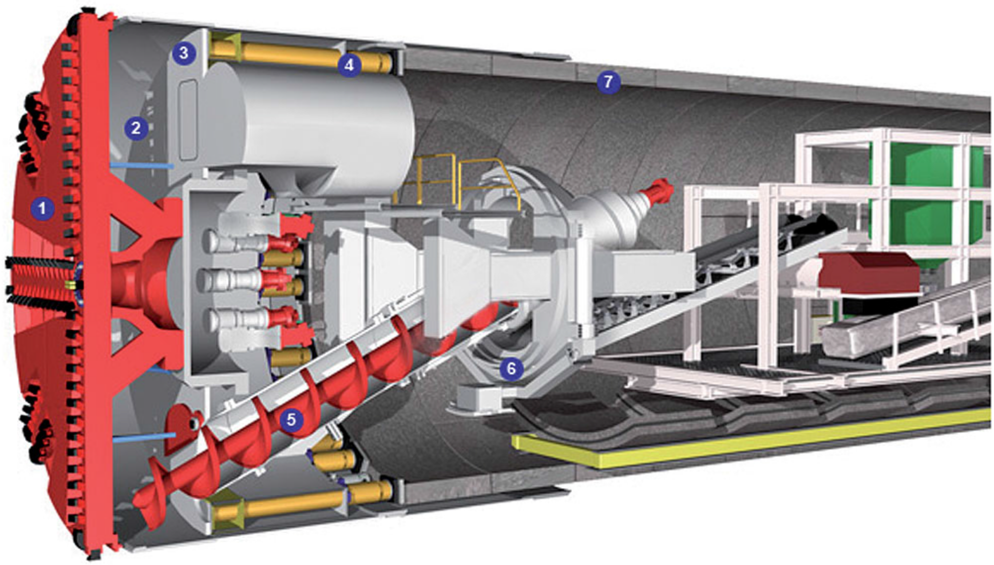


Figure 1.1: Functioning of EPB shields (Herrenknecht et al., 2011); (1) cutting wheel, (2) excavation chamber, (3) pressure bulkhead, (4) thrust cylinders, (5) screw conveyor, (6) erector, (7) lining segments

an effective counterpressure to the front (Anagnostou and Kovári, 1994). The choice between the two techniques have been analyzed among different authors (Lovat, 2006, Anagnostou and Rizos, 2009, Thewes, 2014) and it mainly depends on different factors, such as the size distribution of the soil, surface restrictions, groundwater condition and tunnel geometry. Historically EPB shields have been used in fine grained soils, especially clayey and silty soils and in average size tunnels, while on the contrary slurry shields have been chosen mostly in coarser soils and in larger tunnels. The motivations related to this choice can be summarized as follows:

- for *Earth Pressure Balance Shields*
  - finer materials are much more suitable for creating a plastic and pulpy mass which can transmit effectively a counterpressure to the front, while coarser soils are not indicated to create such conditioned mass;
  - the cutterhead, while rotating in a so plastic mass, requires high power for guaranteeing an effective torque, thus large diameters have been historically discarded;
- for *Slurry Shields*
  - finer materials such as clays are difficult to be separated from the bentonite which is used to create the slurry in this machine;
  - the cutterhead is operating in an environment with a lower viscosity for the presence of the slurry, thus even larger diameters do not require large installed powers.

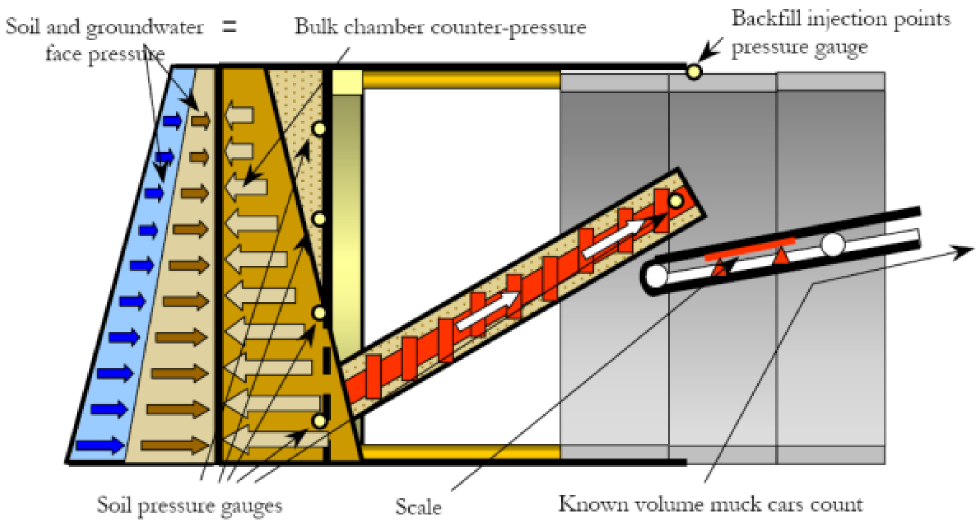


Figure 1.2: Functional scheme of an EPB TBM (Vinai, 2006)

Nowadays, the evolution of the technology based on continuous investigation of each single issue concerning the excavation process, higher installed power, the use of a proper *soil conditioning* technique (i.e. treatments executed during excavation that give to the soil the required features for the EPB tunnelling operations) and practical experience through the years allow huge machines to be constructed and applied in breakout projects (Figure 1.3).

A major advantage represented by the EPB shield is the possibility of operating in different modes (Herrenknecht et al., 2011): this is a very good aspect because the machine can modify its operating principle depending on the geological and surrounding conditions. The main operating modes (Figure 1.4) are including the closed mode, which is the standard operating mode of the EPB used in unstable ground formations under the groundwater table. This mode allows an effective transmission of counter-pressure, as the excavating chamber is full with the excavated and conditioned soil. There are other two particular type of closed operating modes are represented by the closed mode with piston pumps and slurry transport: these two are in use when the excavation proceeds in liquid and slurried fine-grained soils and coarsegrained, very permeable soils below the groundwater table, and also when the screw conveyor is very long. The use of these techniques are due to the fact that in these conditions the pressure gradient cannot be fully dissipated in standard closed mode. The other operating modes are then characterized by the partial presence of pressurized spoil material and compressed air (half open and transition mode) or, in case of particularly stable excavation fronts and when there is no risk of damages on the surface for induced settlements, for instance when the excavation is not performed in urban area, the excavating chamber can be empty and the spoil material is immediately removed by mean of the screw conveyor (no pressure control).

Figure 1.3: Martina EPB Sparvo, Italy. (from TOTO SpA web site). With a maximum excavation diameter of 15.61 m, weight 4500 t, maximum thrust force 400000 kN at 500 bar, installed power 16800 kW.

1.2.2 Range of applications

The traditional range of applications of an EPBS machine belongs to soft clayey soils and clays with a liquid to plastic consistency (Maidl, 1995). The first already cited Japanese application required soils with a size distribution with a passing of 30% at 0.075 mm, 40% at 0.25 mm and 60% at 2 mm. Lately a percentage of passing equal to 30% has been assessed as the minimum value in order to guarantee a suitable and linear functioning of the machine.

In general the permeability of the ground has to be as lowest as possible, in order to prevent the free flow of the water in the excavating chamber and in order to avoid fluctuations on the water table level with relative induced subsidences and destabilizing forces which act on the front and which may cause flows through the machine itself. Herrenknecht (1994) indicates a permeability equal to 10^{-5} m/s as limit for EPBS operations.

Summarizing, an EPBS should operate in a soil with characteristics as follows (Peila, 2009):

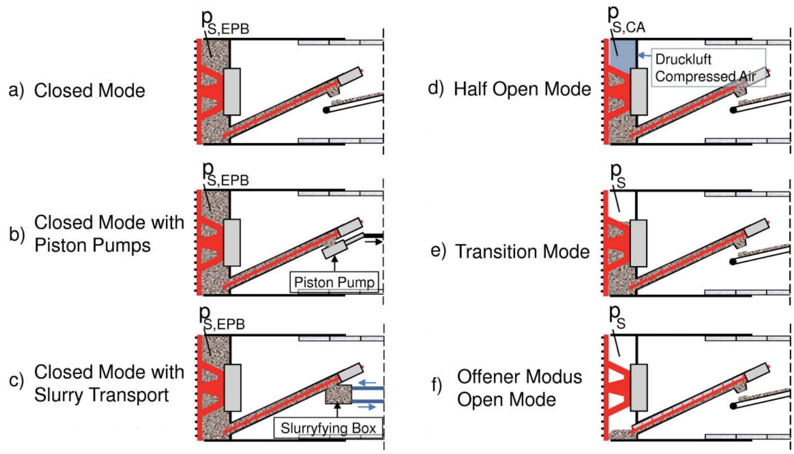


Figure 1.4: Operating modes of Earth Pressure Balance Shields (Herrenknecht et al., 2011)

- good plasticity;
- soft and pasty consistency;
- low internal frictions;
- low permability.

These features are necessary in order to guarantee that (Feng, 2004):

1. the soil, as a fluid, can transmit a pressure from the cutterhead;
2. the groundwater is adequately preserved, avoiding losses and fluctuations;
3. the torque of the cutterhead can be minimized;
4. the wear of the metallic parts can be minimized.

Recently, the range of applications of EPBS machines has been extended in sand, gravel and even in rock masses. This is possible only when the permeability of the excavated material decreases enough to reduce the risks described before. Moreover a plastic consistency must be guaranteed in order to fulfill the required counterpressure standards and to facilitate the mixing in the excavating chamber and the removal of the spoil by mean of the screw conveyor. In general the shear strength of the material must be reduced to guarantee a suitable behaviour of the soil in the excavating chamber. This is done, as in the other soils, by mean of foaming agents and water, but adding also in some cases bentonite and polymers. In this kind of coarse soils, in presence of abrasive minerals, it must be taken into account also the wear potential on the metallic parts.

Problems in overconsolidated clays are less obvious, as the reduction of the shear strength is not an easy task. This is due to the fact that in order to have a low shear

Figure 1.5: Example of clay chip sample as collected on the conveyor belt of an EPB machine excavating in a clay formation. The right picture clearly shows that the inner part of the chip is not conditioned and the conditioned mud is only outside the chip (Peila et al., 2015).

strength, in general, these clays require a certain amount of water, to obtain a cohesion of maximum 10–25 kPa. But due to their extremely low permeability this mixing is difficult. The remodeled clays require a water content close to a liquid index ranging from 0.4 to 0.75. On the contrary the overconsolidated clays usually are characterized by a water content close to their plastic limit. If a low amount of water is added to these clays, they maintain their stiffness and the cutterhead and the screw conveyor require high energy consumption and torque to attack the front and to remove the clumps of clays from the excavating chamber. On the other hand when the added water content is too high, the mass becomes a mud which cannot be handled effectively on the machine through the screw conveyor and the conveyor belt. Another important aspect to be taken into account is the clogging potential, especially in plastic clays which have the tendency to aggregate and be bound by cohesion forces (Hollmann and Thewes, 2012, Zumsteg et al., 2013b).

Studies on clay applications have been carried out by many authors, in order to investigate the behaviour of the pulverized clay (Merritt and Mair, 2006, Martinotto and Langmaack, 2007, Messerklinger et al., 2011, Hollmann and Thewes, 2012, Zumsteg et al., 2012, 2013a). More recently the study has been focused on the possible actual behaviour in an excavating chamber, by analyzing the conditioning of clamps of excavated clay (Figure 1.5 from Peila et al. (2015)). This is due to the fact that the excavation produces a muck composed by clumps of variable sizes, depending on the penetration of the cutting tools into the tunnel face, and a matrix of finer material pulverised by the action of the tools and by the continuous mixing into the bulk chamber. For this reason the extracted material appears like clay clumps surrounded by the conditioned powdered fraction, and the clay mass has already the amount of natural water which hydrates the clump. When dealing with pulverized samples the mix could be difficult and the result of the conditioning process might be unrealistic.

Another important study on clogging phenomenon, especially related to the EPBS excavation, has been done by Thewes and Burger (2005), who studied the effect of clays with different Atterberg limits in correlation with clogging potential (Figure 1.6).

From this important study, Hollmann and Thewes (2013) established a final cor-

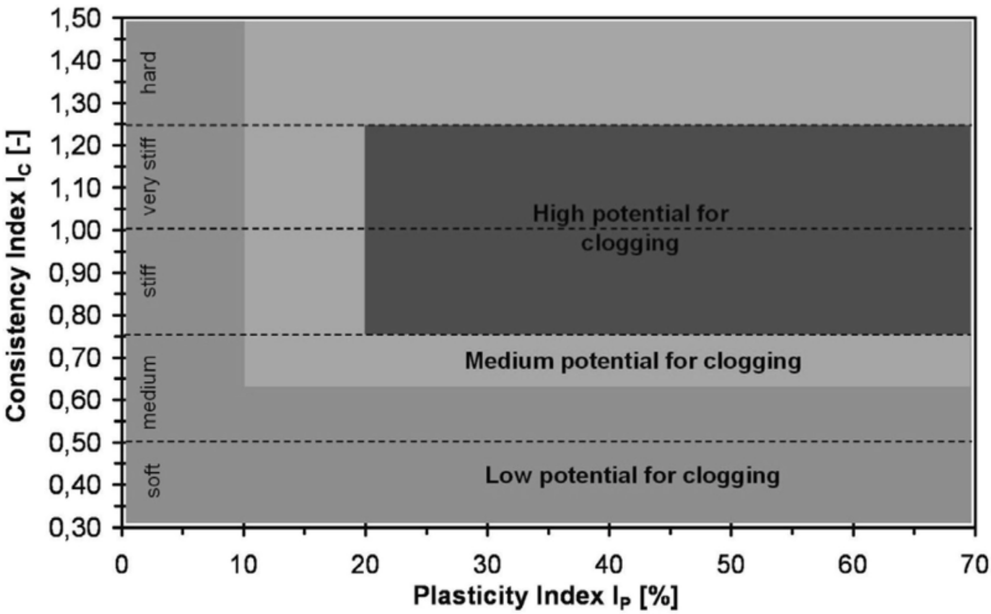


Figure 1.6: Clogging potential of clay formations regarding plasticity and consistency (Thewes and Burger, 2005).

relation of the water influence in the clogging potential assessment, underlining the importance of the water itself on the conditioning process (Figure 1.7)

Considering these aspects, the actual range of applications of EPBS technology can be summarized depending on the size distribution and the conditioning technique that has to be used, as shown in Figure 1.8. This chart is including not only the extended range of the modern EPB shields, but also the conditioning techniques necessary in order to obtain a suitable conditioned spoil material for an effective excavation process.

1.3 Conditioning principles

1.3.1 General aspects

As already stated in the previous sections, in order to have an effective application of EPBS excavation, the soil has to have precise and particular properties. First of all if it is too permeable the groundwater cannot be controlled through pressure balance, even though the excavating chamber and the screw conveyor are fully filled. If the material does not have a sufficient plastic fluidity, the flow of the spoil through the screw conveyor is not regular. Consequently there is a tendency of the material to create a space to let the water to escape. Moreover viscous-argilleous soils might increase clogging potential that can obstruct the excavating chamber and the screw conveyor and that can block the cutterhead.

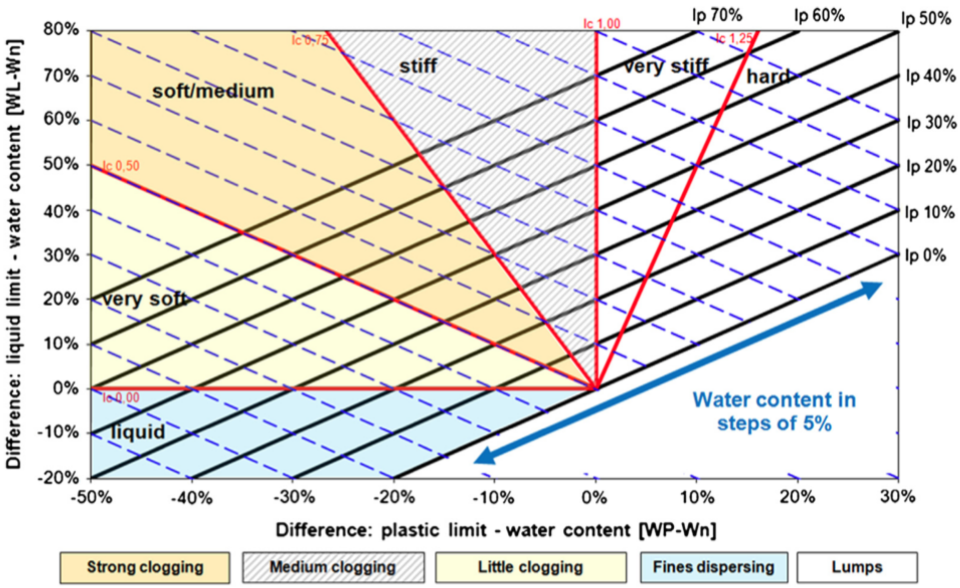


Figure 1.7: Universal classification diagram for critical consistency changes regarding clogging and dispersing (Hollmann and Thewes, 2013).

The term soil conditioning refers to the use of suitable additives (conditioning agents) in various proportions to alter the soil properties of the excavated spoil.
Conditioning agents have been used extensively in drilling and tunnelling operations for many years. Particularly in tunnelling and pipe jacking, the performance of tunnelling machines is enhanced using proper ground conditioning and lubrication agents (Milligan, 2000).
Recent experiences in mechanised tunnelling have revealed the prime importance of ground conditioning agents in the excavation process from the tunnel face to the spoil handling (Pellet-Beaucour and Kastner, 2002). Soil conditioning is applied in relatively long drives and in difficult ground conditions, where fully mechanised systems are used.

As mentioned before, for a successful EPBS application the soil must have defined properties: if too permeable, groundwater cannot be controlled by the counter-pressure applications, even if the excavation chamber and the screw conveyor are completely full. If there is no plastic fluidity in the material, there is no smooth flow to the screw conveyor. Consequently, there is a tendency for the material to arch at the entrance of the screw conveyor, allowing groundwater and material to push through, with subsequent collapse at the face (Herrenknecht, 1994). Moreover, in case of sticky clayey soils there is a high danger of cutter head, excavation chamber and screw conveyor clogging up, with difficulties in re establish the machine efficiency.
Therefore, in order to extend the original application field for EPB shields towards coarser soil, as well as towards hard and gluing clayey soils, the natural material can

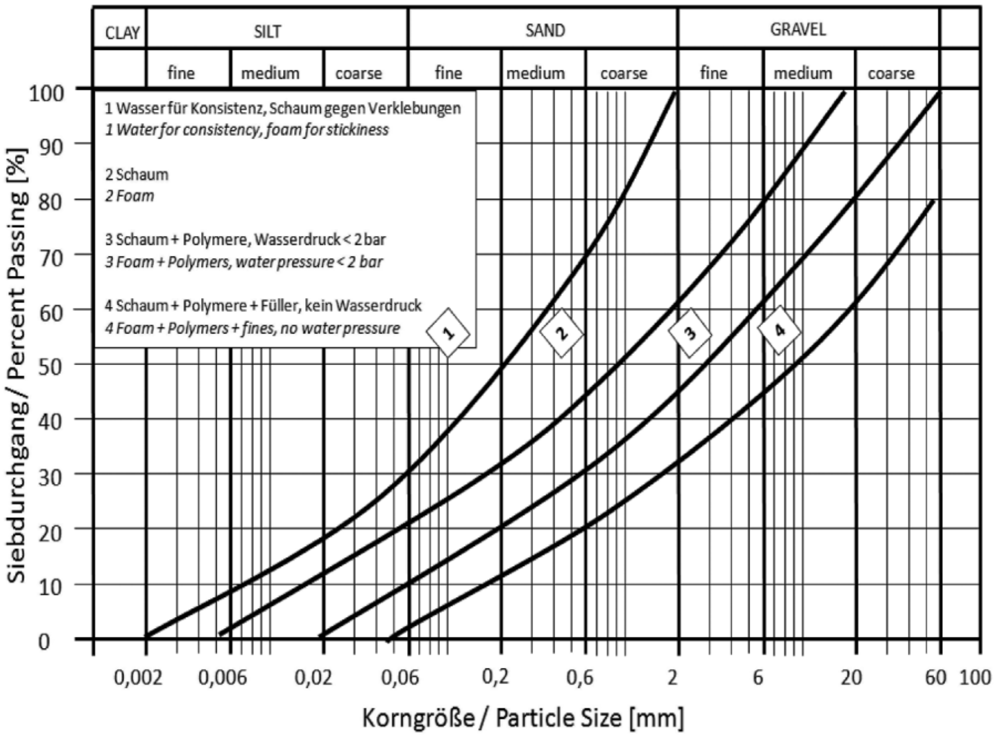


Figure 1.8: Application range for EPB shields depending on necessary soil conditioning (Thewes and Budach, 2010).

be treated by means of additives injection during the excavation step. Such additives can be of different nature and composition, and have changed deeply for the first applications to the more recent developments.

Soil conditioning agents are usually bentonite-based slurries, sometimes mixed with various types of polymer as well as foams. Foams were introduced in the tunnelling industry in the late 1970's in Japan with EPBS machines. Today, apart from the physical and chemical requirements associated with the performance of the various agents and chemical additives, environmental issues must also be taken into account. The latter means compliance with both safety-in-use and disposal regulations.

Soil conditioning agents improve the performance of several of the TBM parts and may be introduced at various points as discussed in the following sections (Psomas, 2001).

- **Tunnel face:** At the tunnel face, the main role of soil conditioning agents is to reduce the friction between the cuttings and the cutting tools of the cutterhead. The reduction of friction decreases wear and torque requirements for the cutting head. This, in turn, results in lower operational cost and longer life for the components of the machine.
  In slurry shields, the conditioning agent (bentonite suspension) creates a filter

cake which supports the tunnel face and makes the ground impermeable. In EPBS, soil conditioning agents can reduce the permeability of the soil in order to allow the excavation to take place in a controllable way, in coarse-grained water bearing soils. In all cases, the soil conditioning agent should be introduced at the point of cut as early as possible to allow sufficient mixing with the ground.

- **Machine head:** The main objective is to reduce friction within the machine head and to create a more homogeneous mass so that clogging is avoided. Reducing friction again results in a reduced wear within the head. It also decreases the power requirements for the cutter head to turn the excavated material into a suitably plastic mass. Creating a more plastic and homogeneous excavated mass improves its workability and consequently, allows for better control of the pressure changes at the face of the tunnel. This, in turn, appears to improve the stability of the tunnel face and provides better control of ground movements thereby contributing to safer working conditions for the personnel in the tunnel. In slurry shields, the excavated material falls into an excavation chamber filled with slurry, which is placed behind the cutting head. In EPBS, the addition of conditioning agents is necessary so that the excavated material can be transformed into an earth-slurry supporting the tunnel face. The excavation chamber is also filled with the excavated material and at this stage, can be conditioned with suitable agents. It is important to note that the interior of the excavation chamber should be designed in such a way as to provide the best possible mix of soil and additives. The ports on the cutting head must ensure early mixing of the conditioning agents with the excavated material. At the cutting head, each injection point should have its own delivery line in order to prevent blockages. This mainly concerns the face ports that are more prone to blockages.
  A high advance rate is achieved due to the improved flow characteristics of excavated material through the cutter head and the lubrication of the moving parts.

- **Spoil handling:** Soil conditioning agents also affect the handling process by reducing the wear of the parts of the spoil removal system. In slurry shield machines, the spoil is pumped out from the face to the surface separation plant. In order to transport the spoil efficiently with the minimum pipe and pump wear at the required velocity, the slurry must have the optimal flow characteristics. For example, thixotropic properties of the bentonite slurry are useful so that in case of a circulation halt, spoil remains in suspension without settling out in the pipes. The reduction in the friction losses in the pipes, valves and pumps results in lower power consumption and savings in energy.

In slurry shield machines, the last stage of the excavation process is the separation of the spoil from the slurry. The addition of conditioning agents facilitates the separation process. Adding special flocculating or deflocculating agents can also enhance separation. The final spoil contains less fine material after it has been processed in the separation plant and therefore is in a more suitable state for disposal. In slurry shields, the excavated material is removed hydraulically and is separated from the support medium (bentonite) in a separation plant. This is the major disadvantage of using slurry shield machines on account of environmental hazards and the high cost involved should the soil contain a high percentage of fines (Herrenknecht, 1994, Maidl, 1995). In EPB shields, screw conveyors achieve the removal and the transportation of the excavated material from the pressurised excavation chamber to the tunnel exit under atmospheric pressure. The removal rate is very significant because it is related to the rate of advance. Ideally, the two rates should be compatible, otherwise loss of the support pressure at the tunnel face occurs. When the soil in the excavation chamber has not reached a sufficiently low permeability, a further injection of conditioning agents in the screw conveyor prevents excessive flows of water. An important detail with regard to the design of screw conveyors is their position in the excavation chamber. When the screw conveyor is located at the bottom of the excavation chamber, it is easier to empty it. The spoil should be in a suitably plastic state in order to allow controlled extrusion through the screw conveyor without causing excessive wear or consumption of power. Sometimes the same type of conditioning agent is used to achieve different material properties. For example, in the excavation chamber of an EPBS machine, the aim is to make the soil more plastic and workable usually by adding water. However, afterwards excessive water in the screw conveyor can create problems (Milligan, 2000).

In Table 1.1 , the application of soil conditioning agents with tunnelling machines is summarised. In this table, TBMs include the open or shield mode machines suitable for rock conditions whereas slurry shields and EPBs, are principally for soft ground conditions.

Specifically, in EPBS, the addition of conditioning agents extends their range of application. The suitability of the various types of conditioning agents depends on the different ground conditions encountered. For example, in clays, when bentonite slurries are used, the addition of polymers makes them more effective. However, if polymers are used alone, they will disappear into the formation without providing any lubrication. In sands with gravels or poor rock and in sandy-silty soil, foams can be used as conditioning agents. When cobbles and gravel are encountered, polymer additive with foam (0.1 to 3 % per volume) is necessary. The addition of foam offers two major benefits: increased compressibility and reduced permeability. In fine-grained soils, foam can be enhanced with natural polymers, which prevent water absorption. This helps to prevent clogging and balling. Milligan (2000) noted that in stiff overconsolidated clays, the addition of agents makes clay more plastic. However, it is difficult to estimate how much water must be added to reduce the undrained shear strength. If too much is added, then it can turn the clay to slurry whereas insufficient water can make the clay stiffer and would then need extremely high power to remould it. In high plasticity clays, a large quantity of water is required to sufficiently change the water content and therefore, the shear strength. In this case, the danger is the creation of large chunks of clay in a softened soil matrix that will clog up the machine and the conveyor. For intermediate plasticity clays, the best practice is to create a rubble of intact clay blocks in a matrix of polymer foam, which inhibits water absorption but allows clay blocks to slide around each other.

Summarizing, the improvement due to soil conditioning may come about in a number of ways, summarised here (Milligan, 2000):

- reduced wear of machine cutter head face plate and tools, and all wear parts of the muck removal system

Table 1.1: Application of soil conditioning agents in mechanised tunnelling (Milligan, 2000)

Location	**TBMs**	**Slurry Shields**	**EPB Shields**
Tunnel face	- lubricate cutters/discs - reduce wear and power requirements	- improve slurry properties - reduce wear and power requirements	- lubricate cutters - reduce wear and power requirements - reduce water inflows
Machine head	- improve muck flow - reduce friction-wear	- prevent clogging - reduce wear with abrasive soils	- make soil more plastic - prevent clogging and recompaction
Spoil handling system	- reduce water content - improve handling	- improve dispersion of soil in slurry - reduce wear - improve performance in separation plant	- produce plastic state in spoil - reduce permeability, friction, wear and power requirements, water content of the muck - prevent excessive water inflow
Spoil tip	- improve spoil quality for easier disposal and reuse in construction	- improve spoil quality for easier disposal and reuse in construction	- improve spoil quality for easier disposal and reuse in construction
Tunnel bore	- support tunnel bore - provide lubrication	- support tunnel bore - provide lubrication	- support tunnel bore - provide lubrication

- improved stability of tunnel face, with consequently better control of ground movements
- improved flow of excavated material through the cutter head
- reduced cutter head power requirements
- reduced friction and heat build up in shield
- excavated material formed into a suitably plastic mass
- enhanced properties of soil in the pressure chamber of EPB machines, leading to:
  – more uniform pressures in the working chamber
  – better control of groundwater inflow by reducing permeability
  – reduction in clogging of machine head chamber
  – more controlled flow of soil and water through the screw conveyor
  – easier handling of excavated soil

Moreover, the soil conditioning for the tunnelling industry gives other benefits:

- support of excavated bore in pipe jacking, microtunnelling and HDD
- reduction of jacking forces in pipe jacking and microtunnelling
- reduction in the friction losses in the pipes, valves and pumps of a slurry machine system
- better separation of spoil from slurry in a slurry machine system
- more acceptable spoil for disposal
- through a number of the above, improved safety for personnel working in the tunnel, particularly during cutter changes and cutter head inspections.

1.3.2 Soil conditioning parameters

The study of the soil conditioning has to give the indication on the conditioning to be used in the tunnelling work.

- Foam Injection Ratio (FIR), representing the volume of foam added to the volume of soil:
$$\mathrm{FIR} = 100\frac{V_{\mathrm{foam}}}{V_{\mathrm{soil}}} \qquad (\%); \tag{1.1}$$
- Foam Expansion Ratio (FER), representing the ratio between the obtained volume of foam and the volume of generation fluid (water + foaming agent):
$$\mathrm{FER} = \frac{V_{\mathrm{foam}}}{V_{\mathrm{generator\ liquid}}} \qquad (-); \tag{1.2}$$

- percentage of free water added to the material ($\mathrm{w_{add}}$), represented by the ratio between the mass of free water ($\mathrm{M_w}$) and the mass of material ($\mathrm{M_s}$):

$$\mathrm{w_{add}} = 100\frac{\mathrm{M_w}}{\mathrm{M_s}} \qquad (\%); \tag{1.3}$$

- percentage of total liquid added to the material ($\mathrm{w_{tot}}$), represented by the ratio between the mass of natural water, added free water, liquid generator ($\mathrm{M_l}$) and the mass of material ($\mathrm{M_s}$):

$$\mathrm{w_{tot}} = 100\frac{\mathrm{M_l}}{\mathrm{M_s}} \qquad (\%); \tag{1.4}$$

- Polymer Injection Ratio (PIR), representing the volume of the water-polymer solution added to the soil volume:

$$\mathrm{SIR} = 100\frac{V_{\text{slurry solution}}}{V_{\text{soil}}} \qquad (\%); \tag{1.5}$$

- concentration of the foaming agent in the generation liquid ($\mathrm{c_{foam}}$) that has been kept constant at 2%.

- concentration of the bentonite in the slurry (water-bentonite) solution ($\mathrm{c_{slurry}}$) that was variable depending on the type of product following the specific setting defined by the producer.

1.4 Conditioning agents

The conditioning additives are, mainly:

- Water;

- Foam;

- Polymers;

- Fillers, like bentonite or similar fine particle materials.

1.4.1 Water as a conditioning agent

Water is responsible for the consistency of the material, described by the Attemberg limits for fine-grained soils and by slump cone fall for granular soils. Water can complete the effectiveness of other conditioning agent, like foam, by lubricating and activating the finer grain size fraction, as well as alone, in case of soft clays near to the plastic limit (Bordachar and Nicolas, 1998).

1.4.2 Foam as a conditioning agent

Foams are defined as a dispersion of gas bubbles in a liquid or solid in which at least one dimension falls within the colloid size range (1-1000 mm). Thus, foams typically contain either very small bubbles or more commonly, quite large ones separated by thin liquid films (lamellae). The detailed study of foams is beyond the scope of this thesis; here only some basic aspects of foam behaviour are presented.
The dispersed phase of the foam is usually called the internal phase, whereas the continuous phase, external. Foams can be depicted two-dimensionally as a structure in which foam is between two phases: on the bottom, there is bulk liquid and above this, in a second bulk phase, gas. The gas phase is separated from the thin liquid-film by a two-dimensional interface. The region that encompasses the thin film and the two interfaces on either side of the film is conventionally defined as lamella (Figure 1.9).

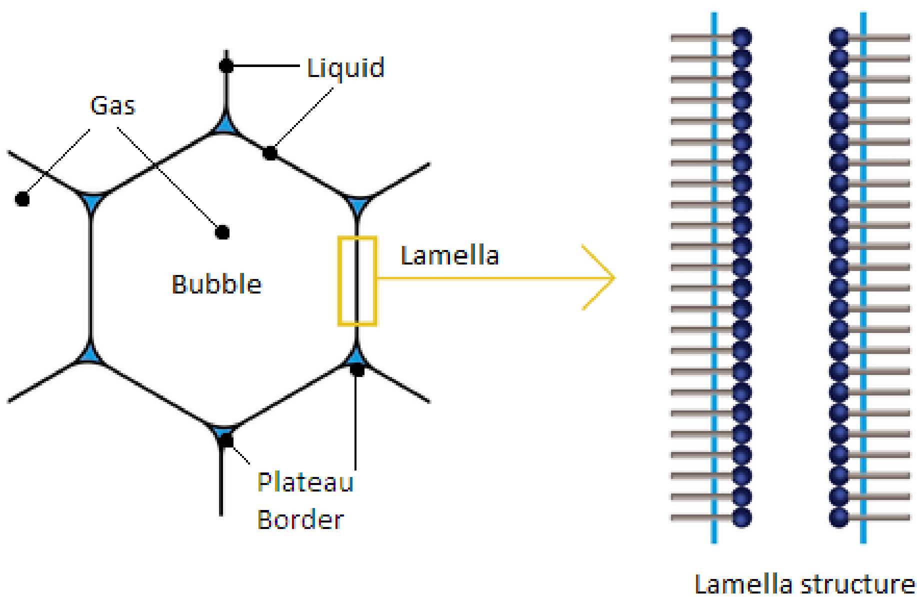


Figure 1.9: Foam structure and disposition of the surfactant molecules

Foam can be formed in a liquid, if bubbles of gas are injected and the liquid between the bubbles can drain away. In pure liquids, gas bubbles will rise and separate according to Stokes' law. The foam structure will be unstable because there will be no thin-film persistence. Persistence can be achieved by adding surfactants, which transform the bubbles into foam cells. In these cases, the foam contains gas, liquid and a foaming agent. The stability (persistence) of the foam is related to the film thinning and the coalescence process. The stability is determined by a number of factors (Efremov and Bikerman, 1973): gravity, drainage, capillary suction, surface elasticity, viscosity, electric double layer repulsion, dispersion force attraction, steric

repulsion and proper surfactants.
The interfacial properties in foams are of prime importance because the gas bubbles have a large surface area. Even a modest surface energy per unit area can become a considerable total surface energy. As the bubble size decreases, the total surface area increases and consequently energy has to be added to the system to achieve dispersion of small bubbles. The energy can be either mechanical and/or chemical by adding the proper surfactant (Schramm and Wassmuth, 1994). The role of surfactants is to reduce the surface tension. Surfactants are chemical compounds, typically short-chain fatty acids that are either amphiphilic or amphipathic. The most favourable orientation of these molecules is at surfaces or interfaces so that each part of the molecule can reside in the fluid for which they have greater affinity. In this way, they create monolayers at interfaces. The surface absorption of a surfactant at the interface acts against the normal interfacial tension. Surfactants are classified according to the nature of the polar (hydrophilic) part of the molecule (Figure 1.10). In an aqueous solution, dilute concentration of surfactants act as normal electrolytes but as their concentration increases, their behaviour alters. The surfactants behaviour can be explained in terms of the formation of organised aggregates of large numbers of molecules called micelles. In micelles the lipophilic parts of the surfactants associate in the interior of the aggregate and leave the hydrophilic parts to face the aqueous medium.

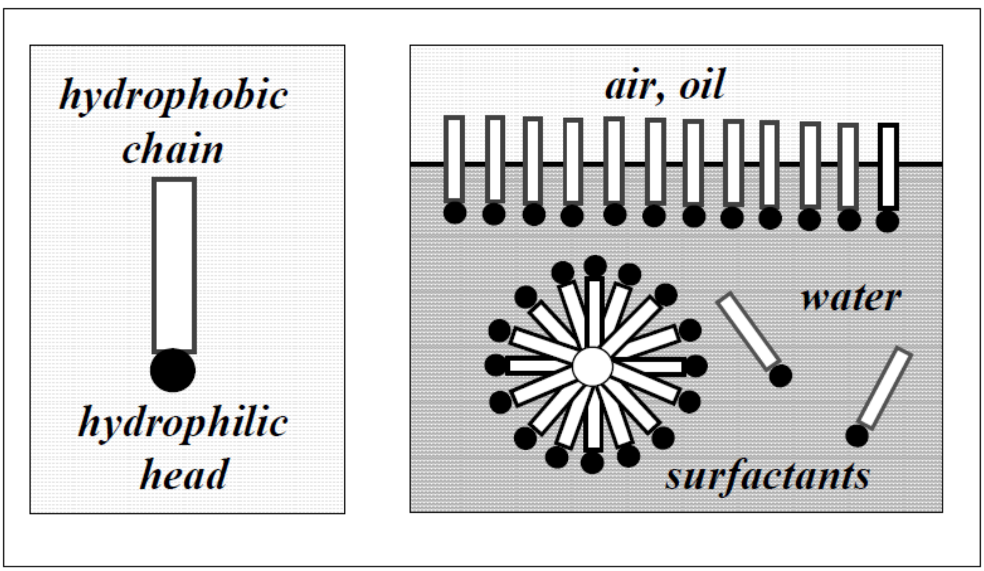


Figure 1.10: Surfactant molecule behaviour and foam formation (Jancsecz et al., 1999).

Surfactants are classified according to the nature of the polar (hydrophilic) part of the molecule, i.e. the molecule head (Bordachar and Nicolas, 1998):

- Anionic surfactants;
- Cationic surfactants;

- Non-ionic surfactants;

- Amphoter surfactants: electric features depend on the solution pH.

Immediately after foam generation, there will always be a tendency for liquid to drain due to the force of gravity. The liquid will drain by flowing downward through the existing liquid films, which constitute the interior of the lamellae. The gas bubbles will not be spherical and at this point, the capillary forces will become competitive with the forces of gravity. The pressure differences between the plateau area force the liquid towards the plateau area, initiating the thinning process, which in turn, will lead to film rupture and the collapse of the foam.

The initial requirements for foam formation are low surface tension and surface elasticity. Schramm and Wassmuth (1994) noted that greater elasticity tends to produce more stable bubbles, but a restoring force is needed in order to produce persistent foams and counteract the 'overwhelming effects' of the gravitational and capillary forces. If a surfactant-stabilised film undergoes a sudden expansion, then immediately the expanded portion of the film must have a lower degree of surfactant absorption than unexpanded portions because the surface area has increased. This local surface expansion provides surface tension, which increases the resistance for further expansions. The resisting force exists under the condition that surfactant absorption equilibrium has been established in the film. This is known as the Gibbs-Marangoni effect, where a tension force counteracts film rupture (Figure 1.11). The durability of the thin layer is dependent on the surface elasticity, which is a dynamic phenomenon; many surfactant solutions display dynamic surface tension behaviour.

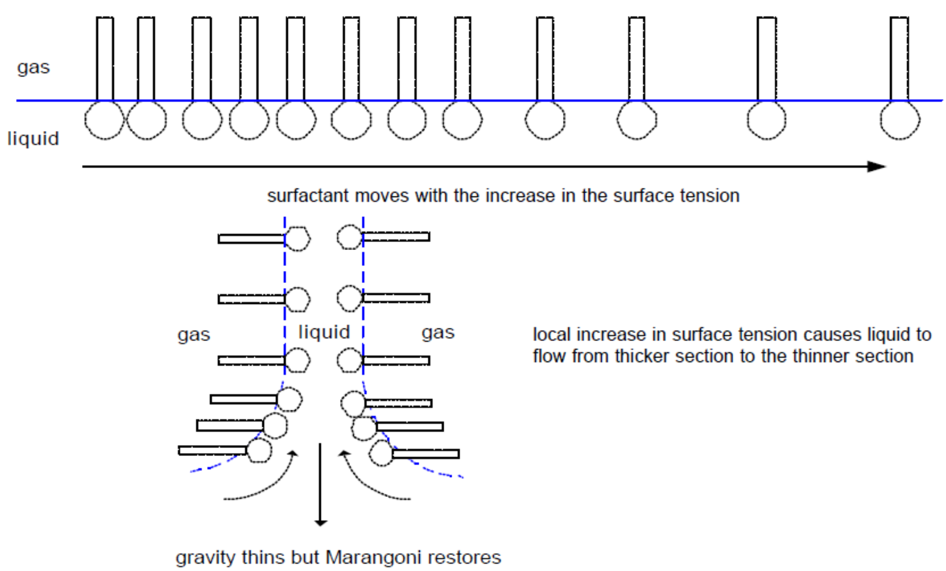


Figure 1.11: Gibbs-Marangoni effect (Psomas, 2001).

There are two basic types of foaming agents: synthetic detergents and the ones that are protein based. Typical constituents of protein foams are 20-40% protein foaming agent and 3-10% glycol-based foam booster. Typical constituents of synthetic foams are 5-30% synthetic detergent and 15-20% glycol ether foam booster. Both types may have fluorocarbon performance enhancer (<5%) and soluble polymer (<5%). They may also have various additives: preservatives, to prevent mould growth; metal salts in protein foams; anti-freeze agents; corrosion inhibitors; solvent, to reduce viscosity; substances to stabilize the foam bubbles; dyes for brand recognition. Synthetic foams are composed of a mixture of anionic hydrocarbons, solvents and stabilizers. They trend to have a low stability due to their relatively rapid drain times; the liquid drains from the walls bubble until they have insufficient strength and collapse. Protein foam agents consist on hydrolysed protein, solvent, sodium chloride, salts of iron and calcium and preservatives in an aqueous solution. The starting material for the protein may be soya beans, corn gluten, animal blood, horn and hoof meal, waste fish products or feather meal. Proteins foam are generally stiff, stable and have low draining rates. Polymers may be added to foaming agents to increase the viscosity of foam and improve its thixotropic properties. The effects of such additions tend to be more reliable with synthetic foams, for which a wide range of additives is available. Protein-based foamers are harder to modify, as the electrical charges created are unknown (Milligan, 2000).

The effects of foam on solid grains have been already studied and applied in ore dressing technology, as a flotation mean known as froth flotation. Flotation exploits the physical principle that a solid body in contact with two non-miscible fluid phases (as air/water) undergoes the effects of the superficial forces present on the phase boundaries. Roughly speaking, it is something similar to washing process, during which the dust particles on the clothes are displaced by using soap emulsions, which show higher surface affinity with grains in comparison with water, and successively pushed away in the liquid phase. The flotation theory is connected to the matter electrical structure and therefore to the related electrochemical and electrophysical phenomena. The chemical interactions are related to kind of bond that constitutes the crystal structure: covalent, ionic, metallic or residual (Var der Waals forces). The phase interactions between solid matter (crystal and grains) and fluid phases (water and air) are driven by the available bonds that can take place on the crystal surface for each molecular structure case. The surface conditions are quite different from the inner crystal reticular composition. For example, separation of covalent bond leaves very reactive superficial atoms, as the electron sharing is interrupted by the grain split. Separation of ionic crystals on the other hand leaves an un-complete electrostatic situation, hence a deformation of the molecular structure is likely to occur and a dipolar ionic condition takes place. The actual adsorption phenomena on the grain surface are therefore related to the chemical features of the liquid-gas phases according to the possible bonds allowed by the crystal structure. As far as the electrical phenomena are concerned, if the surface grain adsorption takes place in ionic conditions, an electrical charging occurs around the grain. A ionic double stratum of adsorbed ions and antagonist ions arises, and the more different the grain features are (size, mineralogy, shape, number), the more complex the global electrical condition becomes. The geometrical contact between three phases lies on a line, the so-called contact line. The features of this line can be expressed in a useful way by means of the contact angle θ introduction, that is defined as the angle formed by each two phases on a plane perpendicular to the contact line. As a rule, the usually referred contact angle is measured between the water and the solid phases. If $\theta = 0°$, this means that the mineral adheres preferentially with the water, therefore the solid-air contact is impossible. On the contrary, if $\theta = 180°$ (theoretical conditions, since not possible with any solid material), the solid prefers the air contact. If $\theta = 90°$, the solid shows no preferences between the two fluid phases (Figure 1.12).

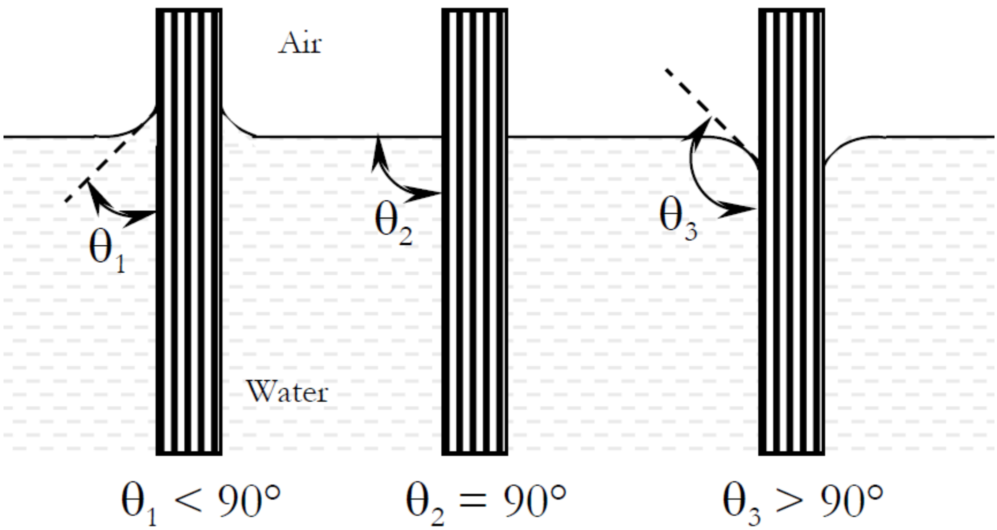


Figure 1.12: Solid matter affinity regarding the contact fluid phases.

Anyway, it must be reminded that the air bubble adhesion on a solid surface happens even if the contact angle is $< 90°$, since such adhesion modifies the phase surface conditions. The bubble-solid surface adhesion lowers the global potential energy of the considered system because the final surface extension is smaller, therefore, according to the second law of thermodynamics, such phenomenon takes place spontaneously.

Finally, it is necessary to point out that the contact angle together with the geometry and dispositions of the solid structure control the final permeability of a porous medium, because the eventual air bubbles trapped between two or more solid particles deny the liquid passage through the pores. Since usually minerals show very small contact angles, surfactants have also to modify the solid surface adsorption features in order to allow a more effective air-solid adhesion. The foam type chosen should match the properties of the soil to be excavated (EFNARC, 2005):

- **Foam type A**: high dispersing capacity (breaking clay bonds) and/or good coating capacity (reduced swelling effects). Suitable with clays and sandy clay/silt.

- **Foam type B:** general purpose, with medium stability. Suitable for sandy clay/silt, sandy clayey silts, sand.

- **Foam type C:** high stability and anti segregation properties to maintain a cohesive soil as impermeable as possible. Suitable for sand, clayey gravel, sandy gravel.

The properties of foam are related to the expansion ratio (the ratio of the foam volume to the original liquid volume) and also to the nature and concentration of the foaming agent in the liquid. In a typical application, the expansion ratio might be 5 to 20, so that 1000 litres of foam would contain 200 to 50 litres of liquid, the rest being air. The liquid in turn would typically contain 1 to 3% of foaming agent concentration , the remaining being water. Thus even if large quantities of foam are required, and the concentrated is expensive, the cost of foam may be quite modest (Milligan, 2000).

Foam with expansion ratios in the range 5-10 are usually referred as *wet* foams, whereas foams with expansion ratios in the range 10-20 are usually referred as *dry* foams. Dry foams are usually more stable than wet foam.
The foam stability when mixed with soil is totally different from the stability of foam only. Williamson et al. (1999) reported dissipation of only 25% after 7 days in confined conditions, while in non-confined conditions it is a matter of hours. The same is reported in Babendererde (1998), Bezuijen et al. (1999), Kupferroth et al. (2001) and Psomas (2001). It is to be pointed out that half-time tests for pure foam can give 3 to 30 minutes of stability averagely. Foam should be stable during injection and mixing with soil, but should become unstable as soon as possible after discharge from the screw conveyor. In the first case it is important to maintain pressure and a plug in the screw conveyor, in the second to reduce the volume of soil for transportation and land filling (Kupferroth et al., 2001). Foam effects on EPB performance are summarized in Table 1.2 (Quebaud et al., 1998).

The amount of foam that is added to the soil is measured through the so-called foam injection ratio (FIR), that is the ratio of the injected foam volume over the volume of the excavated material.

Since the foam is made up by roughly 90% of air, its volume depends strongly from the reference pressure. Moreover, the soil density changes from the in-situ conditions, the chamber condition and the extraction conditions, therefore its volume is not unique. For such reasons in technical literature the definition of volume condition can be somehow confusing. It would be much simple to refer to the mass quantities of additives and soil (kg of additive over kg of soil), but this does not seem to be used by now. The proper amount of foam that should be added has to be carefully defined through tests on natural soil. Based on experience accumulated over ten years, Kusakabe et al. (1999) give the required foam injection ratio (from the Japanese Foam Shield Method Association) for coarser soil as:

$$FIR = \frac{a}{2}\left[\left(60 - 4.0X^{0.8}\right) + \left(80 - 3.3Y^{0.8}\right) + \left(90 - 2.7Z^{0.8}\right]\right) \tag{1.6}$$

Where:

- X is the percentage of soil passing 0.074mm
- Y is the percentage of soil passing 0.25mm
- Z is the percentage of soil passing 2.0mm

Table 1.2: Foam influence on EPBS operation (Quebaud et al., 1998).

Parameter	**Foam influence**
Soil consistency	- to give to the soil a pseudo-hydraulic state, that is to say able to transmit pressure - to give to the soil a pseudo-plastic state, able to create a pressure gradient along the extraction screw
Ground permeability	- to make the excavated soil almost impermeable
Homogeneity	- to make the mixing between soil and foam easier, in order to obtain a paste with the required quality
Sticking compaction (fine soils)	- to avoid the compaction of muck due to the sticking of the clay - to avoid an over-consolidation of the fine elements under the mechanical action of the cutterhead and the screw
Friction	- to help the muck circulation from its excavation to its storage
	- to reduce the abrasive behaviour of some soils
Confining regulation	- to fill the empty spaces of the expanded soil in order to reduce the pressure variations
Impregnation (granular soils)	- to give an apparent cohesion to the soil at the head front through its impregnation in a limited depth

The coefficient *a* depends on the uniformity coefficient of the soil (Uc):

- a = 1.0 for Uc > 15

- a = 1.2 for 15 ≥ Uc >4

- a = 1.6 for Uc ≤4

However, this formulation does not take into account other aspects, that are fundamental in the correct FIR definition, like water content or porosity of the soil, nor specifically identify the applicability of the foam chemistry (Boone et al., 2005). (Bezuijen et al., 1999) provide an alternative concept, which gives rather different required foam volumes, whereby the quantity of foam mixed with the soil must produce a porosity of the soil greater than the maximum porosity of the soil alone at the pressures obtaining in the head chamber. As the pressure drops through the screw conveyor, the porosity will increase further as the air in the foam expand. The vane shear strength of a sand-water-foam mixture was measured at less than 2 kPa.

This interpretation in Bezuijen et al. (1999) is said to come from Maidl (1995) and it has been confirmed by the laboratory tests of Peña (2003), where the relative density of foamed soil samples undergoing shear box tests was negative, i.e. the void ratio was higher than the maximum one. This implies that the effective stress is very low since the pore pressure, and this has been indicated as the reason for the shear strength reduction in case of foam injections. Further experimental confirmations can be found in Bezuijen et al. (2005), concerning the measurements of the pore pressure and the total pressure during excavation steps at the Botlek rail tunnel project: almost no differences were found between the two pressures, therefore effective pressures are said to be negligible. Last, laboratory tests carried out by Merritt et al. (2003) and Merritt and Mair (2006) with the experimental screw conveyor apparatus at the Oxford University showed almost no differences between normal pressure acting on the screw case and pore pressure in the same screw section during tests that gave good results in terms of pressure transmission along the screw conveyor. This is an evidence of the negligible effective pressure when foam increases the sand porosity of a stressed sample. However, some authors disagree with this schematization: Maidl and Hintz (2003) points out that, if by means of conditioning, the density of the ground is below the loosest packing grade, no grain to grain forces can really be transmitted (effective stresses). This situation is critical since the tunnel face is then practically only supported by the foam and the solid fraction consolidates in short standstill. Moreover, Anagnostou and Kovári (1996b) carried out the face stability calculation in terms of effective stress, assuming the pore pressure inside the chamber responsible for the eventual prevention of the seepage effects. It is therefore evident that the volume of foam required is not yet well defined. For stiff clays it appears from limited evidence to be around 30%. Cash and Vine-Lott (1996) suggest that the foam flow rate should approximately equal the void content of the cut material after allowing for pressure effects. For example, a cutter head excavating 1 m^3/min that bulks up to 29% after excavation will need 200 litre/min of foam at the face pressure. Thus if the absolute face pressure is 2 bar, the volume at atmosphere pressure is about 400 litre/min (Milligan, 2000).

1.4.3 Polymers as a conditioning agent

Polymers are essentially large, long-chain molecules formed by the linking together of large numbers of small chemical building blocks or monomers. Homopolymers are achieved by polymerization of a single basic monomer unit, copolymers by two or more different monomers. A polymer material may exist in many different forms, depending on the lengths of the polymer chains (measured by the molecular weight), the presence and nature of any linking between polymer chains, and the existence or not of structured (crystalline) groups of molecules.

Polymers exist widely in nature, and natural polymers that may be used in tunnelling include starches, celluloses and proteins. Some of the man-made polymers that have found applications in tunnelling are polyacyclamides and polyacrylates, partially hydrolysed polycylamides (PHPA), carboxymethyl cellulose (CMC), and polyanionic cellulose (PAC) (Bordachar and Nicolas, 1998). Such new developments are the so-called biopolymers. On the basis of hydrocarbon chains and produced with the help of fermentation , the material is biodegradable pollution free and poses no disposal problem later on (Babendererde, 1998). One of the most important groups of polymers used for soil conditioning and lubrication are the polyacrylamides (PA) and their derivatives, which have been extensively developed for the mineral processing industry, where they have largely replaced natural products such as starch. Lambe (1953) discussed the conditioning of soil for strength and compactability using sodium polyacrylate more than 60 years ago. High molecular weight (>10x106) water-soluble polyacrylamides have been used for many years to aid separation of solids from liquids by acting as flocculant. Polyacrylamides of lower molecular weight (<300,000) may be used as dispersants, while cross-linked versions produce water-absorbent polymers. A wide range of polyacrylamides is possible, with molecular weights ranging from several thousand to over 10 million, and with ionic character from 100% cationic to 100% anionic (monomer units having positive or negative charges respectively). They retain their water solubility due to their high degree of linearity, the introduction of high degrees of cross-linking produces materials that do not dissolve but swell and absorb large quantities of water.

Flocculation is thought to occur mainly by bridging, whereby the polymer molecules attach themselves to the surface of the mineral particles, leaving projecting loops of varying lengths. As particles collide these loops entangle with each other and lock the particles together. In addition to use synthetic PA-based polymers, a number of natural products have been used as flocculating agents. The majority are polysaccharides such as starches and sugars; starches have been most widely used, notably for the flocculation of bauxite in the alumina industry. PA polymers of low molecular weight act as dispersing agents by increasing the overall negative surface charge on solid particles to which they become attached, reducing the natural tendency of particles to flocculate as a result of the variable distribution of charge on particles, and hence maintaining a dispersed structure of lower viscosity. Water-absorbing polymers are Pas of very high molecular weight (>107) and a high level of cross-linking. As a result, they crease to be soluble in water, despite the presence of polar amide and acrylate groups, but merely absorb water and swell. Water-absorbing capacities of 500-600 times the weight of the polymer are possible with pure water, but 100 times is more realistic for water containing dissolved solids. Polymers generally find use as

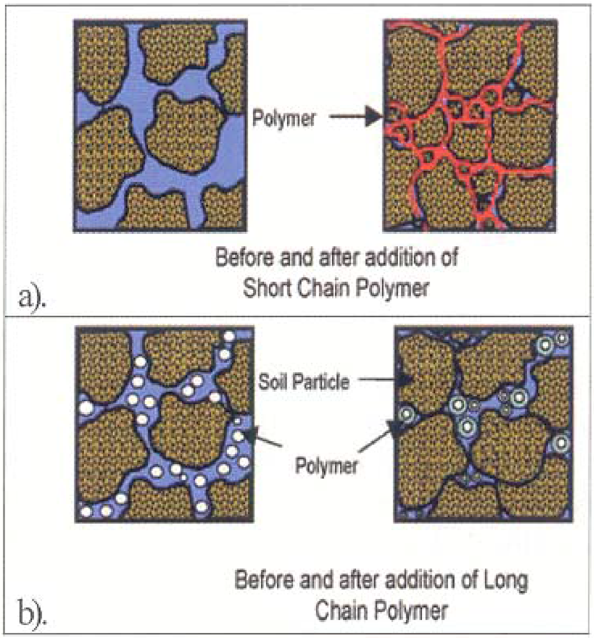


Figure 1.13: Short and long chain polymers action on the soil.

additives to enhance the performance of bentonite or clay slurries or pastes. They may act to improve the ability to form a filter cake, increase lubricity, or maintain a dispersed structure. Alternatively, polymers on their own or in foam formulations may be introduced in relatively small proportions to lubricate and give plasticity to coarse soils (Milligan, 2000). Polymers are available in a variety of forms, but most commonly either as solid beads, dissolved or dispersed in water or oils. Dry polymer beads may be difficult to dissolve quickly as they tend to lump and form a gel layer on the outside, which inhibits the further ingress of water. It helps to provide agitation, but high shear mixers should not be used as they tend to break up the large molecules. Liquid supply would be the best option (Obladen et al., 2003, Kupferroth et al., 2001).

Polymers have the following basic functions in EPBS machines (Kupferroth et al., 2001):

- to increase the viscosity of water in the soil near the face
- to bind fine particles of silt and sand to reduce permeability
- to increase stiffness of the ground and help form a plug in the screw conveyor
- to stiffen or strengthen the foam to help prevent breakdown (for instance during stoppages)
- to lubricate the soil to assist travel through the working chamber and screw conveyor

- to prevent or reduce adhesion to face plates, tools and other metal surfaces
- to reduce cutter head torque for lower maintenance and increased speed of advance

Water-absorbing polymers, such as cross-linked Pas, may be added to wet excavated material to render it dry enough for normal handling to tip. The same effect may be achieved more cheaply with lime or cement, depending on the conditions controlling disposal (Milligan, 2000). When a water soluble or partly water soluble polymer is introduced into the system water is taken from the surrounding soil. The polymer swells with the uptake of water and the soil particles are able to move closer together, increasing the inter-particle friction and so giving the impression of stiffening or drying. In fact the actual moisture content remains the same and some polymers will even release the trapped water with time (Kupferroth et al., 2001).

1.4.4 Fillers and bentonite as a conditioning agent

The name bentonite is popularly used for a range of natural clay minerals, principally potassium, calcium and sodium montmorillonites. The term smectite is also used for the group of minerals that includes montmorillonites. Because of the chemistry and microstructure of the clay particles, they have a strong ability to absorb water and are able to hold up ten times their dry volume by absorption of water. Montmorillonite consists of very thin flat crystalline sheets of clay minerals that are negatively charged and are held together in stacks by positively charged sodium or calcium ions in a layer of absorbed water. In particular the particles of sodium montmorillonite are extremely small and thin, being typically of the order of 1.0μm or less in the length and 0.001μm thick. The ability to absorb water comes from the relatively low bonding energy of the sheets, which allows water molecules to be adsorbed onto the internal and external sheet surfaces. Calcium ions provide a stronger bond than sodium, so the calcium montmorillonite. Potassium ions provide much stronger bonding between clay sheets as the potassium ion is exactly the right diameter to fit into the clay structure with negligible gap between the clay sheets. A similar material to montmorillonite, but with potassium bonding is the non swelling clay mineral known as illite. The substitution of sodium by calcium or potassium ions in montmorillonite greatly reduces the ability of the clay structure to hold water. Bentonite slurries are made by adding bentonite to fresh water and mixing in a high-shear mixer (to ensure proper dispersal of the clay particles), and then leaving the slurry for a recommended time to ensure sufficient hydration of the clay. Slurries are usually formed from a few percent of bentonite in water. Bentonite slurries are thixotropic and typically form a gel concentrations above about 5% by weight in water. The use of bentonite as additive to the head chamber in EPB shields comes to confer or increase the plasticity and reduce the permeability of coarse-grained soils (Milligan, 2000, Bordachar and Nicolas, 1998). Other fillers can be used instead of bentonite slurries for high density slurry EPB operations: for example, the excavation of Lot 5 of the first track of the Turin subway was been carried out adding a fine crusched limestone (grain size $< 20\mu$m) with injections ratio between 5 and 10% of the excavated volume, to provide the necessary fine contents to the natural soil gradation (Boone et al., 2005).

1.5 Summary of the effects of the conditioned soil properties on EPB performance

In the previous sections the properties of various soil conditioning agents and their role in modifying the ground properties in tunnelling applications were presented. In this section, the effect of the foamed soil properties, particularly on the improvement of the EPBS performance, is summarised. The three fundamental properties of the ground examined are compressibility, permeability and shear strength. In the ensuing paragraphs the influence of each soil property on the operation of EPBS is discussed.

1.5.1 Compressibility

The increase of the compressibility of the soil in the pressure chamber through the addition and mixing of conditioning agents, improves the workability and the homogeneity of the spoil. A more compressible and plastic material in the pressure chamber enables the bulkhead to be responsive to pressure fluctuations, resulting in a better control of the stability of the face. The main benefit is that if the material in the pressure chamber is very incompressible then small fluctuations in extraction rate cause large pressure changes. Increase in compressibility causes a "softer" response in which the pressure in the chamber can be more easily kept constant.

1.5.2 Permeability

Reduction of soil permeability at the face minimises the possibility of face collapse due to water inflow. Successful control of the permeability of the spoil in the pressure chamber allows a suitably plastic consistency to be achieved. This is also important in the spoil removal stage where an effectively impermeable spoil can be remoulded in the pressure chamber and extruded through the screw conveyor without allowing inflow of ground water. Particularly in stiff clays, the aim is to form a rubble of intact blocks, in a matrix of foam which inhibits uptake of water by the clay.

1.5.3 Shear strength

The shear strength of the soil affects the wear of moving parts and cutting tools. Decreasing the angle of shearing resistance of the soil at the face results in a reduction in wear due to the reduced resistance to cutting. Reduced resistance results in reduction of wear and torque and consequently, significant savings in energy. Another function of the conditioning agents is the lubrication of the cutting parts which in turn reduces the working temperatures and extends the life of movin g parts such as the cutter-head, cutters and screw conveyor. Reduced resistance in shear improves the workability of the spoil once it enters in the pressure chamber and the screw conveyor. However, if the shear strength is reduced too much than it may not be possible to sustain the necessary pressure gradient in the screw conveyor.

1.5.4 Ecological and environmental aspects

According to Langmaack and Feng (2005), the risk to human being and environment is mainly determined by the following four points:

1. the amount of substance entering the environment;

2. the toxicity of a substance for the environment, respectively the toxicity towards aquatic organisms, and for humans;

3. the chemical and physical properties of a substance, which determine the distribution in the environment. In most cases this is leachability into ground water. In addition, bioaccumulation has also to be taken into account.

4. The elimination process known as degradation and/or immobilization also determines the distribution in the environment. Organic substances can be degraded in two separate ways: either by organisms which are already existing in the soil or added separately (biodegradation) or by abiotic processes (hydrolysis in presence of water; photolysis under influence of light).

Suitable soil conditioning products should only be those that show the desired functional properties and in the same time are as save as possible for the workers and the environment. This implicates a judgement of the acute aquatic toxicity, potential of bioaccumulation, biodegradation and chronic aquatic toxicity by risk assessment. Some useful definitions, given hereafter, allow understanding the main issues for the environmental analysis of conditioning additives.
Toxicity is the intrinsic capacity or substances to cause negative effects to organisms. Toxic effects depend on the amount of a substance, which is available to the organism. Toxicity tests carried out in the laboratory are used to predict the so-called safe concentrations at which no negative impact on the organism is expected. The acute toxicity differentiates toxicity to mammal organisms and toxicity to aquatic organisms. For mammals the lethal oral dose of 50% of the population (LD50) is listed in mg substance for kg of organism weight, for aquatic organisms the lethal concentration for 50% of the population (LC50 or EC50) is listed in mg substance per litre of water.
Ecotoxicology Jancsecz et al. (1999) takes into account the behaviour of substances in the environment (biotic and abiotic degradation phenomena) and on the other hand toxic effects or the toxicity of substances. The potential toxic effects produced by chemicals on organisms have to be detected. There are two ways to assess the ecotoxicity of chemicals:

- Field studies which are very complex (interaction between numerous parameters), long and costly

- Laboratory studies, which represent a simplified approach by the choice of species to chemical laboratories and the assessment of effects.

The laboratory tests are standardized for the choice of species, the criteria studied, the exposure duration and conditions. This allows the chemicals to be compared each other. The aquatic ecotoxicity tests include acute or short-terms, chronic or long-term

tests and microcosm tests. For the TBM additives foam and polymer acute ecotoxicity, a test that can be carried out is the short exposure (few days) of species to tested substances and mortality measurements, for example Daphnids (Acute Toxicity tests on Daphnids). As far as conditioning additives are concerned, the most expensive area is the acute aquatic toxicity. Test should be done according to OECD Guidelines 201 to 203. Generally, the LD50 and L(E)C50 product shall be as high as possible. For all type of polymers the L(E)C50 data for Daphnids and Algues shall be preferably >100 mg/l water in order to be not classified for acute toxicity. Foams, due to their reduction of surface tension, should reach LC50 data of >10 mg/l concerning fish (class acute III) (Langmaack and Feng, 2005).
Bioaccumulation is a process by which organisms concentrate chemicals within themselves. This can result either from their food or directly from the surrounding environment.
Biodegradation is the breakdown of an organic substance by the action of microorganisms. Before degrading completely to water and CO2, substances may degrade to smaller intermediates. Persistence is the ability of substances to resist degradation. The following mentioned tests are referring only to aerobic biodegradation (Jancsecz et al., 1999):

- Ready biodegradation (carbon dioxide evolution, OECD 301 B): biodegradation of organic substances to CO2 and H2O substance as only carbon source and addition of only a low content of microbiotic organisms
- Inherent biodegradation (Zahn-Wellens, OECD 302 B): biodegradation of organic substances to CO2 and H2O testing of the substance in favorable conditions of biodegradation (internal reserve of carbon, higher content of microbiotic organisms)

The ecological properties of a product are judged by biodegradation data, using the above-mentioned OECD tests with a defined amount of starting bacteria. Generally, soil conditioning products shall be either (a) readily biodegradable or (b) not biodegradable (inert material) and non-toxic. Both the possibilities guarantee the lowest possible impact to the surrounding ecology (Langmaack and Feng, 2005).

Water Pollution Class is a German classification, also used in other countries. The existing ranking is the following (Jancsecz et al., 1999):

- WGK 0: non water hazardous
- WGK 1: slightly water hazardous
- WGK 2: water hazardous
- WGK 3: strongly water hazardous

This water classification is changed regarding the European harmonization. The WGK 0 will not exist anymore, substances are now called non water hazardous (mostly for non soluble substances). In consequence all surfactants are at least ranked in WGK1: slightly water hazardous. Regarding their use during TBM boring, only a water-based solution of surfactants (preparation) is used. Typically, the surfactant

concentration during boring is 2-4% of a 20% water based solution. The real used surfactant concentration which is added to the soil can be calculated to 0,4-0,8% in water. Preparations of less than 3% of WGK1 substances (total weight) can be classified as non water hazardous. It is the concentration which plays a key role on the toxicity of substances.
In the case of soil conditioning operation with EPB it is therefore important to:

- Choose products with minimum toxicity and eco-toxicity values

- Choose products with high biodegradation or inert components (if the bioaccumulation risk is low)

- Minimize the quantity of injection materials

In order to evaluate the general risk of the injected substances it is strongly recommended to carry out a general risk assessment prior to their intended use. The goal of the risk assessment is to summarize the technical data concerning the environmental hazards. Some aspects of risk analysis to be pointed out are here presented (Langmaack and Feng, 2005):

- Predicted environmental concentrations should be compared with predicted non-effect concentrations and recommended exposure limits. A collection of various physicochemical transformations and toxicological data as well as the product quantities that are typically used are required.

- The environmental risk assessment has to be carried out for each of different ingredients of one product and the predicted environmental concentrations have to be compared individually to the respective predicted non-effect concentrations.

- For the predicted non-effect concentrations (PNEC) for surface water, the European Technical guidance document in support of commission directive 93/67/EEC and commission regulation (EC) No1488/94 should be used

- For the identification of the relevant emissions during application, loading and disposal, a relevance matrix should be prepared. For every specific emission the potential human exposures should be evaluated.

- The calculation of the biodegradation rates and the predicted environmental concentrations (PEC) (based on the assumed infiltration into underground, groundwater widths, depth and flow; also evaporation has to be considered) should be produced.

- The expected impact on the environment should generally be low if the substances are adequately handled and the recommendations of the Material Safety Data Sheets are implemented.

- Risk for the workers: ideally the concentrations in the air are, even under worst case assumption, more 1000 times below the respective occupational exposure limits (hazard index = 1).

- Risk for the environment : no risks to surface water from emission due to pumped tunnel water or run-off water should be expected, providing that the water is drained into the public sewage system for treatment. The potential infiltration of ingredients into the ground water during the product application should not cause any relevant risk for the environment. Based on the available information of the ingredient concentration in the treated soil, it should be able to disposed on an appropriate landfill site without any special pre-treatment.

Chapter 2

Soil conditioning laboratory tests

2.1 Foam generation

The quality and reproducibility of testing is totally reliant on the quality of the foam produced for the tests. In a poorly produced foam the bubbles combine and break more rapidly and the foam quality varies from test to test. EFNARC (2003) gives recommendations on the production of the foam in a laboratory: while the production of foam solution with a high speed stirrer and propeller is not highly recommended because the quality of the foam is not reaching a standard which is needed for testing, the use of a lab scale foam generator is more suitable.

The equipment in use at TUSC laboratory is following these indications and it has been designed and manufactured by Spoilmaster Ltd. In Figure 2.1 a photo of the equipment is shown, while in Figure 2.2 a scheme of the generation process is illustrated.

2.2 Half-life test on the foam

2.2.1 Introduction and equipment

This test is used to assess the stability of the foam mixture and it is based on the guidelines given by EFNARC (2003). The equipment used for the test is composed of the following elements:

- a filter-funnel of 1 litres capacity with a non-absorbent filter;

- a graduated container of 1 or 2 litres capacity made from plastic or non-breakable material;

- a 50 ml graduated cylinder;

- retort stand;

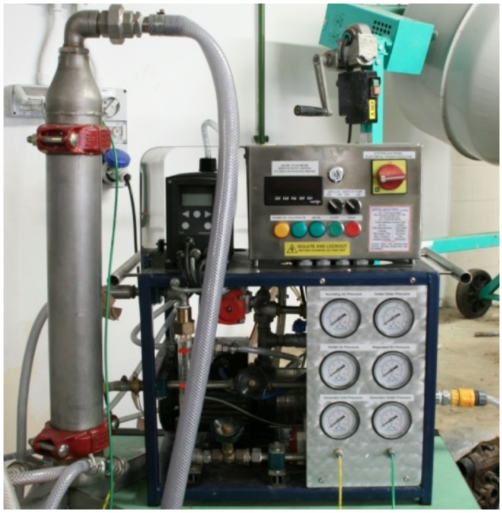


Figure 2.1: Lab scale foam generator at TUSC laboratory.

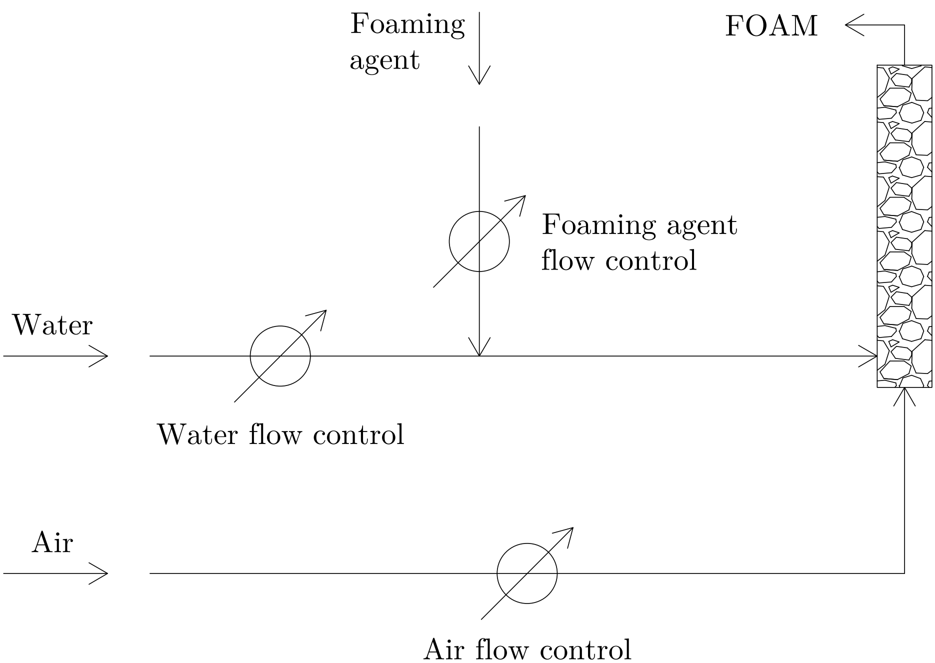


Figure 2.2: Scheme of the foam generation process.

- means of making foam with a known expansion ratio (FER) such as a foam generator (as described in Section 2.1);
- weighing balance accurate to 0.1 g;
- stopwatch.

2.2.2 Test procedure

1. prepare a water-foaming agent solution with the desired concentration;
2. prepare a foam using the laboratory scale foam generator to the required expansion (FER);
3. fill the filter-funnel with 80 g of the foam;
4. measure the time for 40 ml of liquid to collect in the lower cylinder (i.e 50% of the liquid content of the mixture);
5. record the results of the test.

2.3 Slump test

The slump test is a widespread method used mostly to verify the so called workability of the concrete (ASTM, 2015b). The measured parameter is the fall of the material in a standardized metal cone (Figure 2.3), in order to assess the fluidity and estimate the strength of the mass. The cone, at the beginning rigidly linked to a metal plate, is filled with the material to be tested, and when the cone itself is full of material is lifted up and the fall of the material represents the slump (Figure 2.4)

Figure 2.3: A standardized slump cone used for slump testing

This test has been in the recent years borrowed from the mechanized tunnel industry to estimate the suitability of the conditioned soil for the excavation with EPB

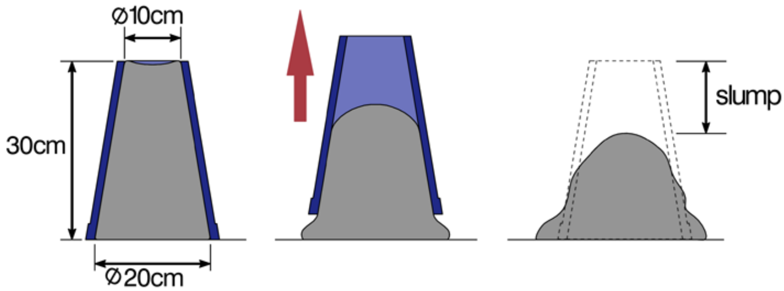


Figure 2.4: Scheme of the slump test with the parameter to be assessed

shields. The first application for this testing procedure is represented by the case by Peron and Marcheselli (1994), which assessed the suitability of the conditioned mass during the construction of a railway twin tunnel in Milan (Italy), driven in an alluvial sandy and gravely ground by mean of a EPB shield. In that case the values obtained by the testing were ranging between 50 to 100 mm.

Other good applications of this method was described some years later, with the first actual research on this field, by Quebaud (1996) and Quebaud et al. (1998). In this case tests on homogeneous fine sand with with a grain size distribution curve ranging from 0.2 mm to 0.4 mm (soil Q1) and an average sand with a grain size distribution curve ranging from 0 mm to 4 mm (soil Q2) have been carried out by varying the water content and the FIR, in order to obtain a value equal to 120 mm, which has been considered as an optimal value according to the authors of the research.

After few years the first research about slump testing in coarser soils was carried out by Jancsecz et al. (1999). The soil used was represented by sandy gravel (50% of sand and 50% of gravel) and by sandy gravel with clay (44% of sand, 43% of gravel and 13% of clay) obtained from the LRTS tunnel project in Izmir (Turkey). In this case the slump values obtained by the authors have been higher than the previous applications (up to 200 mm), due to the fact that the coarser part does not interact effectively with the foam. Similar values of slump (maximum value 216 mm) were obtained later by Williamson et al. (1999), but in this case the soil used for the testing was a sand.

Leinala et al. (1999) carried out tests on the different types of soil encountered during excavation of the Toronto Sheppard Subway Project. With reference to their tests on the soil described as sand/silt, using a initial water content of the samples ranging from 8% to 11% and a FER between 1-6, they observed that it was necessary to have a FIR larger than 50% to obtain a slump larger than 100 mm.

Later on, the researches expanded in a size distribution field finer. Peña (2003) compared the effects of different foaming agents to condition a reference sand with a grain size distribution curve ranging from 0.002 mm to 2 mm. He observed that with a water content of 22% and a concentration of foaming agent ranging from 1.5% to 2.5% for the various products and with a FIR of 65%, the slump was 100–150 mm (which he considered the correct range for EBPS application) while with a FIR of 80%, the slump rose up to 150–200 mm.

Important applications in the slump test research come also from Politecnico di Torino, first with Vinai (2006) and Vinai et al. (2007a) and then with deeper studies on cohesionless soils by Peila et al. (2009) and Borio and Peila (2011). These studies allowed to find that there was a close correlation between the water content and the FIR, and that the optimal combination was with a water content of 10% and a FIR = 40%. Moreover the analysis of different cases brought the research group to finalize during the year the slump assessment and to create a slump test quality diagram in Peila et al. (2009), which was then updated in its recent version in Martinelli et al. (2015b) (Figure 2.5).

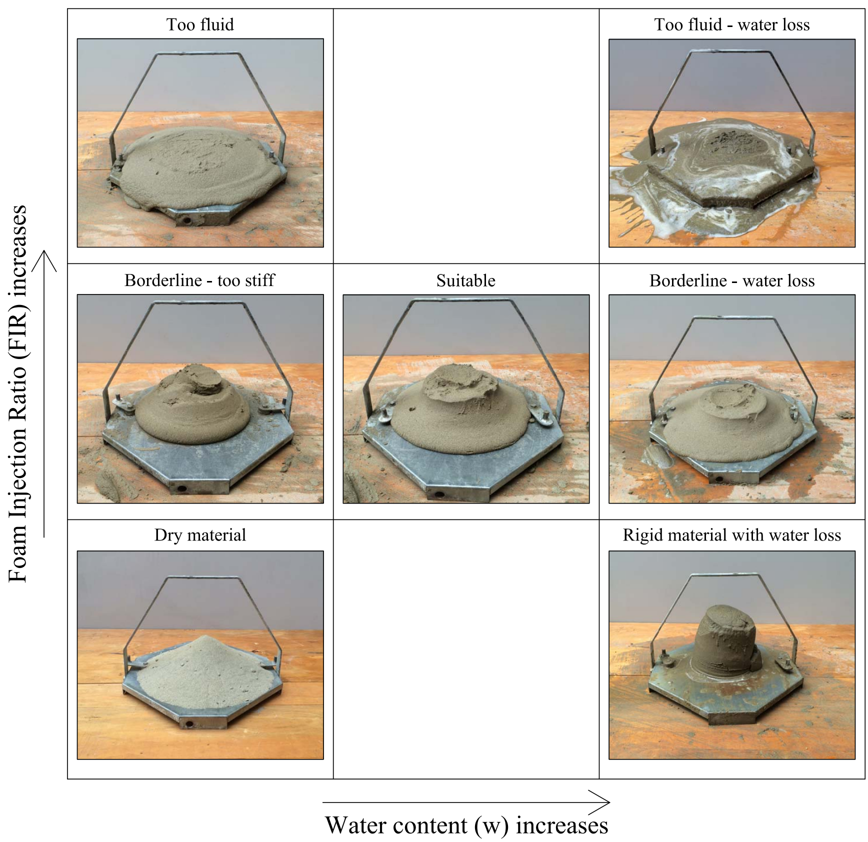


Figure 2.5: Assessed diagram of slump test quality

More recently it is important to mention the research by Hanamura et al. (2007) who carried out mini-slump tests (with a cone of reduced size compared to the standard concrete one) on five different soils (sand with gravel, sandy gravel, silty gravel, silty sand, sand with gravel and clay) conditioned with different polymer types.

2.3.1 Test procedure

The test procedure is based on the literature and on the past experience of the laboratory on soil conditioning testing. The idea is to standardize the method in order to be able to compare the results between the laboratories and different soil type.

The standard procedure is as follows:

1. a quantity of soil equal to 8–10 kg is placed in the mixing device;
2. if required in the set to be tested, a quantity of filler/polymer/bentonite suspension (previously well mixed with a mechanical stirrer) and water is added to the soil;
3. the foam is generated with the FER fixed beforehand (which is always verified);
4. the foam produced with the planned FIR for the test is mixed with the soil;
5. the material obtained is then placed in a standard Abrams cone which is lifted immediately. The subsidence of the material is then measured and the essential technical aspects described before are checked in order to verify the actual result of the test.

The material is usually then inspected in order to check the quality of the mix, for checking the uniformity and the consistency of the material, as just the result of the slump alone is not sufficient. The final purpose is to compare the obtained result with the slump matrix already described and shown in Figure 2.5.

2.4 Extraction test

This test has been created to simulate the situation which occurs in an excavating chamber of an EPB machine. The apparatus used at TUSC laboratory is based to similar devices used in other studies (Merritt et al., 2003, Merritt and Mair, 2006) and it is able to simulate the complex EPB operation that involves both the conditioning of the soil but also the interaction with machine-member for the extraction operation would be really significant for the mix (soil + conditioning agents) behaviour understanding (Peila et al., 2007, Vinai et al., 2007a,b, Borio, 2010).

2.4.1 Apparatus

The apparatus (Figure 2.6) consists of a steel tank 800 mm height with a nominal diameter of 600 mm, on which an aluminum plate 40 mm thick fits. The plate is then connected to a hydraulic cylinder, with a stroke of 500 mm, bore 100 mm, ram 50 mm diameter, driven by a hydraulic pump for the downward movement and pressure application. All the system has been designed in order to allow a working pressure in the tank up to 1.5 bar.

A 1500 mm long screw conveyor coupled with an inclination of 30° on the horizontal is installed at the base of the tank. The inner diameter of the screw case in 162 mm, the flights have a pitch of 100 mm with a screw shaft of 60 mm diameter.

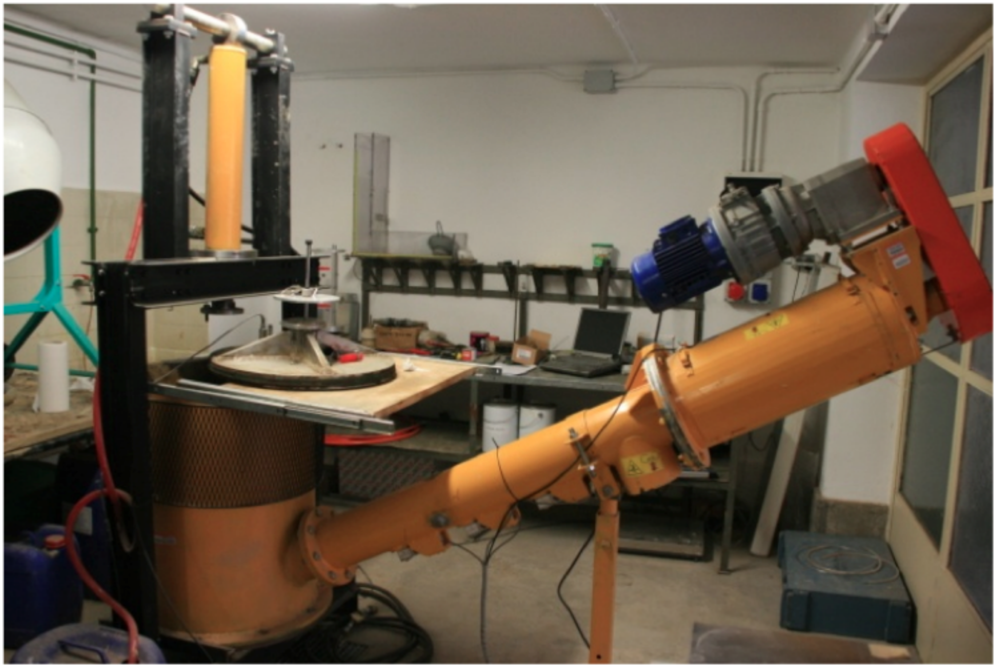


Figure 2.6: Extraction test apparatus (Borio, 2010)

The screw conveyor has been specially designed and constructed modifying a conventional product for grain transport in order to fulfill the request for the laboratory application, that were mainly: the seat for three pressure cells along the screw case, the coupling with a torque meter in line with the screw shaft, the low screw revolving speed, an excurrent unsupported end of the screw 300 mm long.

The screw is equipped with a three-phase motor with an installed power of 1.1 kW, coupled with a mechanical gear that gives screw rotation speeds between 6 and 15 rpm.

Following safety considerations, in order to avoid the possibility for an operator to be injured by accidental plate movement while working around the charging point of the tank, a iron mesh has been placed around the tank, that interrupts via a microswitch the power supply of the hydraulic pump when not in the up-right position.

Furthermore, in order to prevent unexpected extra-pressures in the hydraulic cylinder, and consequently in the tank, an overpressure valve has been installed at the oil inlet of the cylinder.

In order to gather functional parameters as well as physical values coming from the test and record them for subsequently analysis, a dedicated Data Acquisition System has been set up (Vinai et al., 2008). It consists of eight sensors placed on the Apparatus in order to monitor:

- the plate movement

- the pressure under the plate
- the pressure on the bottom of the tank
- the pressure along the screw conveyor (in three points)
- the required torque
- the weight of the extracted material

2.4.2 Test procedure

The tests are carried out with the following procedure: a certain quantity of material (approx. 30 kg), with defined or natural moisture, is placed into a concrete mixer and the desired (with reference to the tests) amount of foam is added into the mixer bowl. As soon as the mix is ready (according to the operator check) it is poured into the tank and this operation is repeated until the tank is full. Subsequently the upper plate is positioned and pushed down by the jack until the test pressure in the tank is reached.

The screw conveyor is then started and the material is collected and weighted at the discharging point of the screw. The test is stopped when the piston reaches its maximum stroke. Slump tests on the material coming out from the screw as well as on soil sampled in the tank at the end of the test are carried out in order to check to final plasticity of the material.

In general this test is valid for standard applications, and it can give good indications about the conditioning parameters in particular conditions. A standard laboratory testing includes the comparison between different conditioning sets obtained with different producers through the slump test. Figure 2.7 shows an example from the study by Martinelli et al. (2015a), where the difference between the top and bottom pressure is investigated. The graph shows perfectly that in case of Set B, where the FIR and concentration of liquid generator was much lower, the pressure is not transmitted effectively along the tank during the all test.

2.5 Permeability test

As already stated, the conditioning process is usually reducing dramatically the permeability of the soil. Thus the permeability is one of the key factors to influence the behaviour and the performance of an EPB shield. This has been clearly evaluated by Anagnostou and Kovári (1996b): if there is a flux between the ground and the excavation chamber, the filtration forces can reduce the stability of the tunnel. The permeability of the soil influences also its capacity of transmitting a suitable counterpressure to the front, moreover with a low permeability material the pressure drop along the screw conveyor can be controlled much better and constantly, as the air and the fluid cannot escape and we have a sort of cap-effect which not allow a flow which is not mechanically induced by the screw conveyor.

The permeability limit has been defined during the years and it must be below the theoretical limit of 10^{-5} m/s, but quite often the actual value found after the

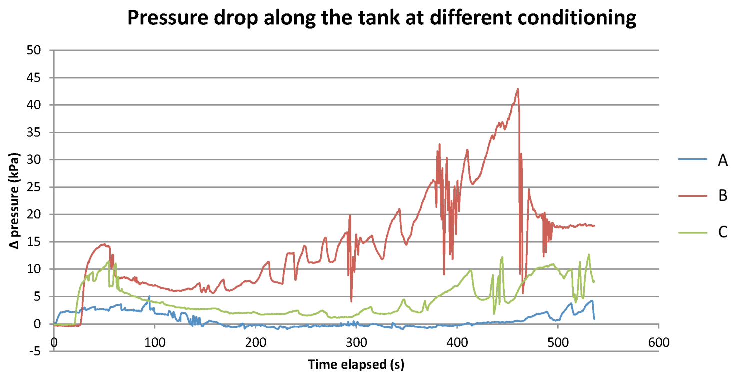


Figure 2.7: Example of pressure drop ($p_{top} - p_{bottom}$) along the tank with different conditioning sets (Martinelli et al., 2015a)

conditioning process has been found even lower than 10^{-7} m/s. The assessment of this limit is really complex, considering that the standard permeability test procedure cannot be used because it is not possible to keep the flux until this is not constant. For this reason a new testing procedure has been designed and developed (Borio et al., 2010).

In general the reduction of permeability is linked to the soil type and the conditioning technique used. In some cases the foam is sufficient to fill the intergranular voids and reduce the flow of the water through those voids. This is valid for a soil type that perfectly fulfills the required standard for EPB (as shown in Figure 1.8); in case there is presence of coarse fraction such as gravel in the soil, it is necessary to add polymers or bentonite to the mass to obtain a suitable conditioning. This leads to a low permeability soil, but for the assessment of the shear strength this testing method is not giving effective indications.

2.5.1 Testing device

For this test, a constant head permeameter is used (according to (ASTM, 2006). The main components are as follows (Figure 2.8):

- 75 mm high permeability testing cell with constant water head;
- water tank with a constant volume of water of 2 l;
- connection pipe ($\varnothing = 8$ mm) between tank and permeameter;
- clamps to control the flow of the water in the pipes;
- graduated cylinder with a capacity of 1 l.

The used water head for this test was usually equal to 0.1 bar (1 m of water), but for testing the conditioned material and compare with the dry soil a value of 1 bar (10 m of water) has been used.

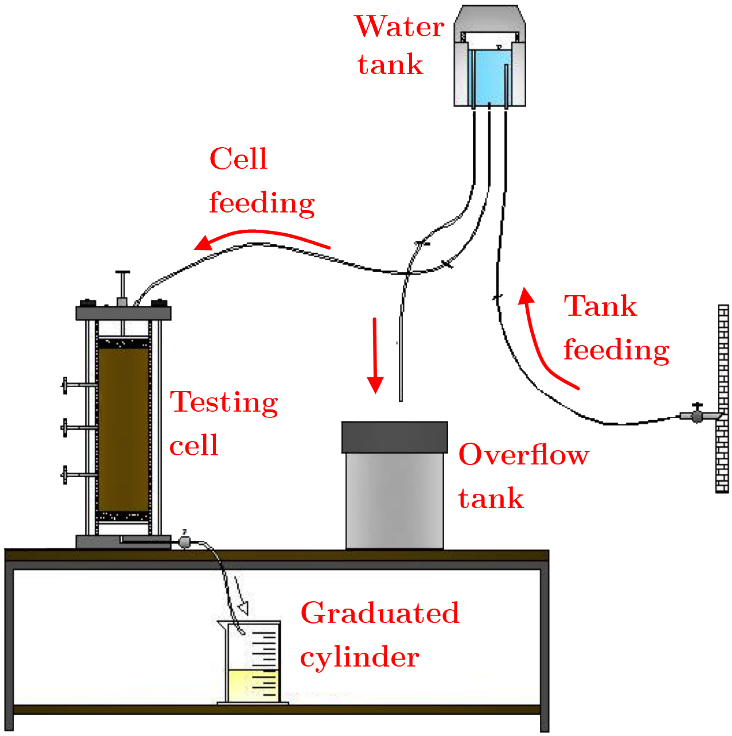


Figure 2.8: Scheme of the permeability test with the indication of the water flows (red arrows)

2.6 Shear and vane shear test

2.6.1 Direct shear test

This is the standard method to verify the shear strength of a soil (ASTM, 2011b). During the direct shear test the specimen is confined inside an upper and a lower rigid box and is subjected to the normal load N and the shear force T, while both the horizontal and vertical displacements are measured using dial gauges or linear variable displacement transducers (LVDTs). A scheme of the apparatus is shown in Figure 2.9.

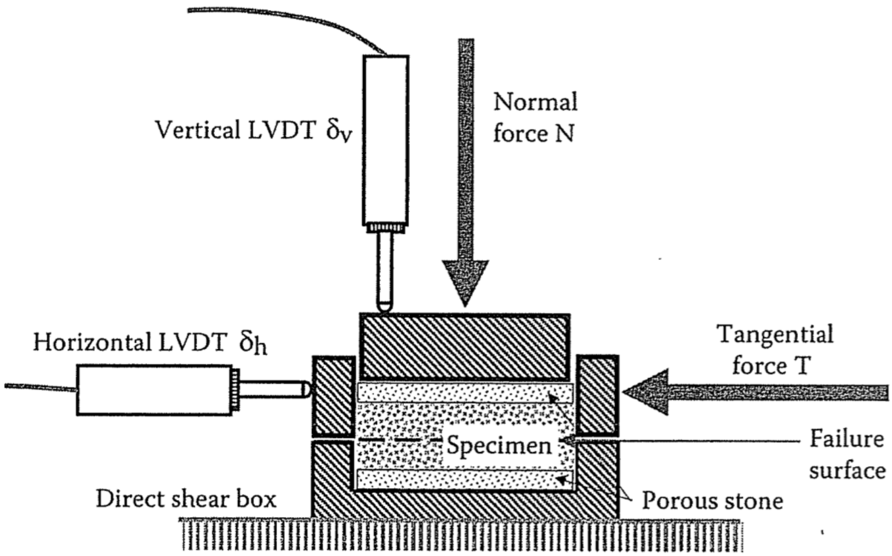


Figure 2.9: Cross-sectional diagram of the direct shear apparatus (Fratta et al., 2007)

Although this is a very common test for assessing the friction angle, the undrained shear strength and dilative and contractive tendencies of soils, the sample requires to have a predetermined failure surface. This cannot be completely fulfilled in a conditioned material that is too fluid, as the mass cannot fail along a surface, but the test is giving a good indication on the reduction of the total shear strength of the conditioned mass.

2.6.2 Vane shear test

This test is based on the field vane test in saturated clay and silt soils for determination of undrained shear strength (ASTM, 2015a) and on the miniature vane test in very soft to stiff saturated fine-grained clayey soils (ASTM, 2016, Pamukcu and Suhayda, 1988). The miniature vane shear test consists of inserting a four-bladed vane in the end of an undisturbed tube sample or remolded sample and rotating it at a constant rate to determine the torque required to cause a cylindrical surface to be sheared by the vane.

This torque is then converted to a unit shearing resistance of the cylindrical surface area. The torque is measured by a calibrated torque spring or torque transducer that is attached directly to the vane. The miniature vane shear test may be used to obtain estimates of the undrained shear strength of fine-grained soils. The test provides a rapid determination of the shear strength on undisturbed, or remolded or reconstituted soils. In the recent years this test has been extensively used also in soil conditioning assessment (Martinelli et al., 2015b, Peila et al., 2015).

2.6.3 Past applications

As already stated in the previous sections, the shear strength of the conditioned samples is of high interest, since the power consumption and the machine wear are directly related to this entity. Several authors proposed shear strength measurements, with vane test or box shear test. Psomas (2001) found a high decrease of shear strength for foamed samples using a large shear box. However Peña (2003), using a similar apparatus, observed that the decrease of shear strength is related to the decrease of effective pressure due to the high pore pressure present during the test. Friction angle has not been found dissimilar from the critical angle of the investigated sand. Jancsecz et al. (1999) report a significant reduction in cohesion and friction for conditioned sand clay. Mair et al. (2003) carried out several vane shear tests on London Clay conditioned with different polymers and foaming agents, finding high decreases in both cases.

EFNARC (2005) indicates vane shear and shear box tests for characterizing the conditioned material. Bezuijen et al. (1999) findings are in agreements with Peña observations, stating that the shear strength, measured with vane tests, is related to the final porosity of the sand-water-foam mixture. A lower final porosity gives higher shear strength.

Messerklinger et al. (2011) studied, realized and tested a modified vane shear apparatus (Figure 2.10) able to evaluate the undrained shear strength with the possibility to change the pressure applied to the soil sample. The device is composed of the sample container with a squared shape in order to avoid the slippage of the sample, a frame composed of a bottom plate and two pillars to which the hut is fixed via the top plate. The hut is movable in vertical direction along the pillars. A level mechanism composed of two springs and two handholds with an X-shaped boring in the rod of the handhold at the contact with the pillar, permit to the upper part to be fixed at any height along the pillars. Before the test performance the hut is moved down on the sample container. Rubber rings located at the base of the hut seal the contact area and convert the hut to a tight pressure chamber. Two fixing screws ensure that the hut remains in position during the test performance. The pressure is controlled via a pressure regulating valve, and the vane is activated with a Brushless DC electric motor. The vane has a diameter of 20 mm and the other dimensions according with the ASTM (2016). The undrained shear strength is calculated by the measured torque applied to the vane. This measure is carried on by a torque sensor directly connected to the vane through a clamp and on the other side to the stepping motor, outside the hut, through a coupling connected to the ball bearing who transmit the motion through the upper plate.

Another recent study with the vane shear test was carried out by Merritt et al.

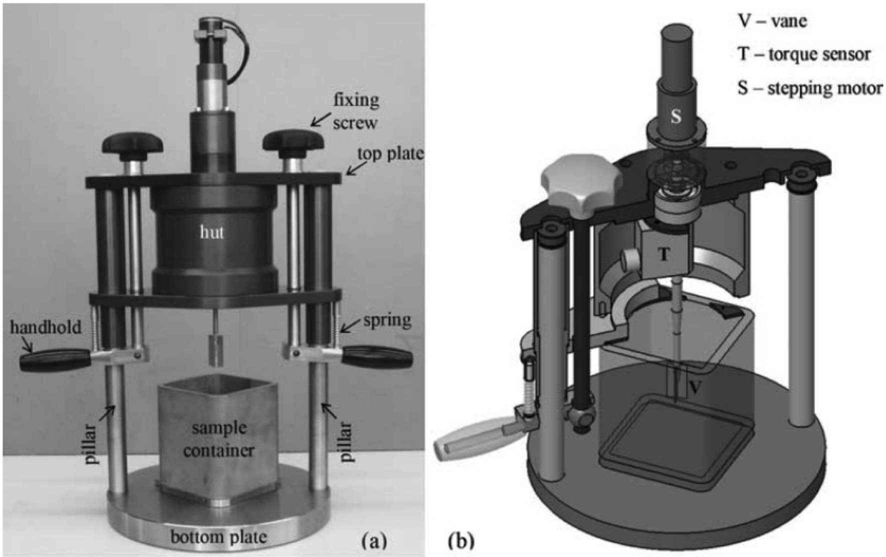


Figure 2.10: Modified vane shear strength apparatus (Messerklinger et al., 2011)

(2013), where the tested conditioned mass was verified and studied also by using a standard vane shear tester. In this research the authors studied the correlation existing between the vane shear strength (undrained shear strength) and the slump test results, obtaining an almost linear correlation (Figure 2.11.

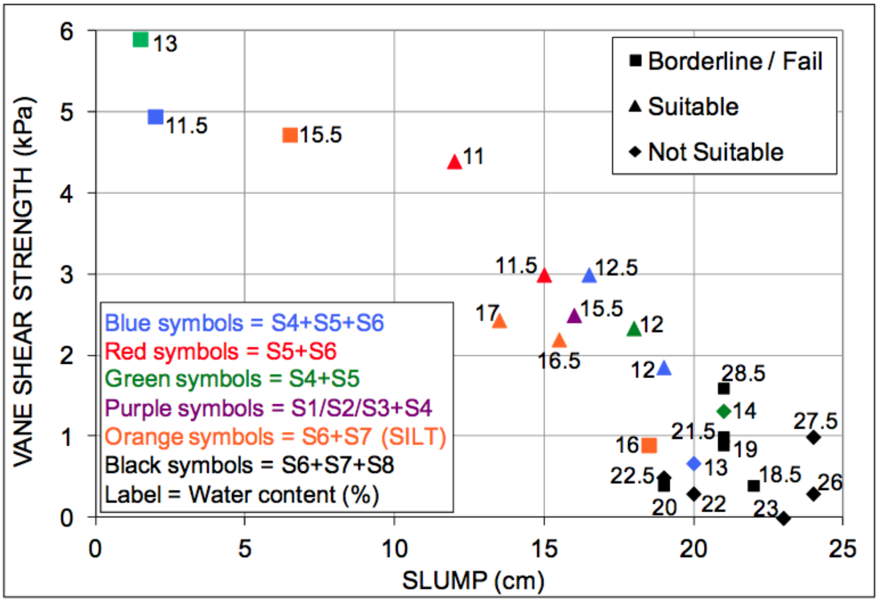


Figure 2.11: Correlation between measured slump and vane shear strengths (Merritt et al., 2013)

Part II

Preliminary study of granular soils

Chapter 3

Preliminary studies based on common testing

3.1 Conditioning agent

For the study only one conditioning agent has been used, in order to uniform the comparisons between the tests. The agent is the Polyfoamer ECO/100 from Mapei SpA, which is a liquid foaming agent based on 100% biodegradable anionic surfactant combined with natural polymer. The use of this conditioning agent is due to the extreme stability of the foam with a high biodegradability of the agent. The stability is demonstrated by the half-life test carried out on the foam generated with the above mentioned Polyfoamer ECO/100, with a concentration of 2% and a FER of 10, which returned a half-life time of 270 s, much higher than other similar conditioning agents that with the same parameters returned values 50% lower (max. 180 s). This result is really important, because by using a stable foam, the best result of the tests on the time is guaranteed.

3.2 Samples studied

For the testing campaign, five different granular soils have been taken into account, in order to cover a wide range of materials depending on the grain size (Figure 3.1). The investigated soils are sands, with variable content of fines.

The characterization campaign included the most common geotechnical laboratory tests, in order to characterize the different materials from the geotechnical point of view, and soil conditioning laboratory tests, to find the most suitable conditioning set for the five materials. For the geotechnical characterization of the Soil A, Soil B, Soil C and Soil D, a direct shear test apparatus has been used, in order to obtain easily the parameters of the Mohr-Coulomb criterion. This is due to the fact that the soils are granular and non-cohesive, thus by mean of this test it is possible to linearize the peak values of each test result in a $\tau - \sigma$ plane. The tests have been carried out by using a standard circular Casagrande shear box with a inner diameter of 60 mm. The

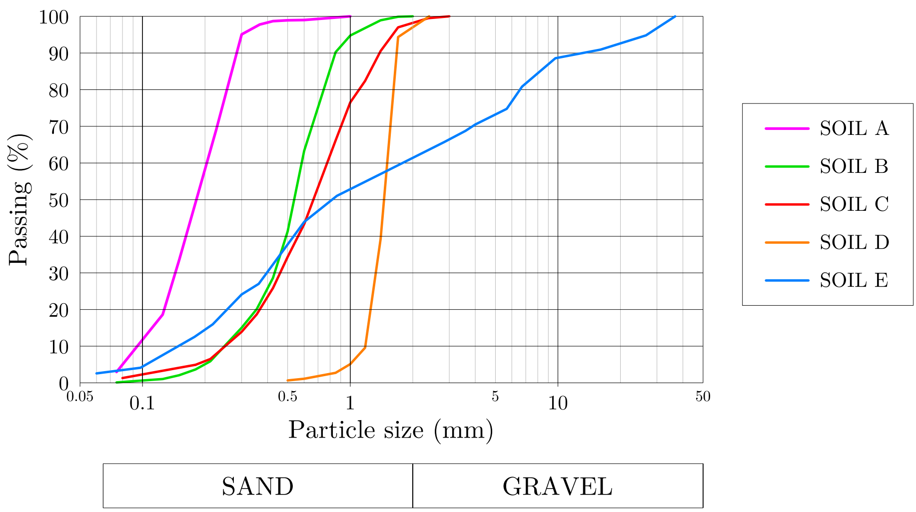


Figure 3.1: Grain size distribution of the five sample soils used for the testing campaign

normal loads chosen for the testing campaign are listed in Table 3.1.

Table 3.1: Normal loads and pressures used in the direct shear test campaign

Normal load (N)	Resulting normal pressure (kPa)
140	49.52
240	84.89
440	155.63
740	261.74
1240	438.59

The soil E has a more complex size distribution, as it is composed also of a relevant part of gravel with size too large to fit in the Casagrande shear box. In this case, the Mohr-Coulomb parameters have been obtained by using a triaxial testing cell.

For the soil conditioning characterization the slump test has been chosen as the most suitable, especially considering the extended literature references on the suitability of the conditioning based on the slump results. The suitability is based on the matrix in Figure 2.5, described in Chapter 2.

3.2.1 Soil A

This soil (Figure 3.2) represents the finer material used for this study and, according to Thewes et al. (2010), should represent the most suitable material to be conditioned, as it is composed of grains which are big enough to avoid clogging problems but small enough to interact with the foam.

Figure 3.2: Soil A

Geotechnical properties

The soil has a friction angle, based on the linear interpolation of the direct shear tests, of around 36°, in dry and natural conditions. Figure 3.3 shows the linearization of the failure envelope, with the value of apparent cohesion equal to 33.7 kPa. The unit weight of the dry loose material is 16 kN/m^3.

Optimal conditioning properties

The soil has a natural attitude to absorb a relevant quantity of water, thus in order to obtain a good mix to be added with the foam, it is necessary to add a lot of water compared to other materials. This lead to a high quantity of liquid to be added, both water and foam. The grain size distribution reveals the good attitude to be conditioned, so no polymers or fine material (such as bentonite slurry) have to be added to the conditioned mass. Figure 3.4 shows the slump of the optimal conditioning set, while in Table 3.2 the parameters of this optimal conditioning set are listed.

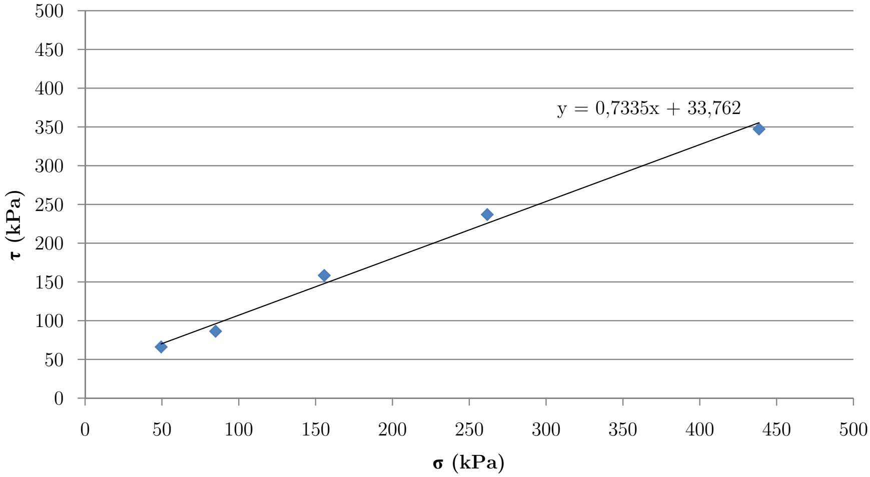


Figure 3.3: Mohr-Coulomb failure envelope for Soil A based on direct shear test results

Figure 3.4: Optimal conditioning slump for Soil A

Table 3.2: Optimal conditioning parameters for Soil A

Parameter	Value
Total water content, w (%)	20
Concentration of the foaming agent, c (%)	2
Foam Expansion Ratio, FER (-)	15
Foam Injection Ratio, FIR (%)	80
Concentration of the slurry (%)	-
Slurry Injection Ratio, SIR (%)	-
Slump (cm)	16

3.2.2 Soil B

This soil (Figure 3.5) represents the second finer material used for this study. According to Thewes et al. (2010), it is suitable for the conditioning process.

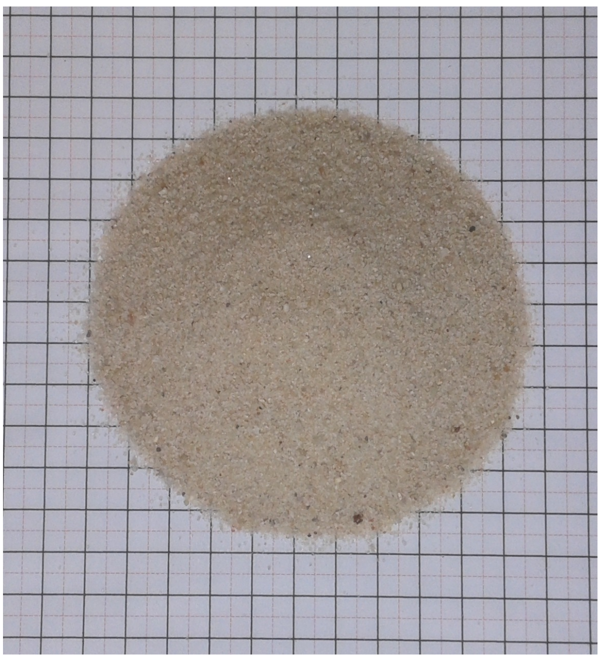


Figure 3.5: Soil B

Geotechnical properties

The soil has a friction angle, based on the linear interpolation of the direct shear tests, of around 33°, in dry and natural conditions. Figure 3.6 shows the linearization of the failure envelope, with the value of apparent cohesion equal to 16.2 kPa. The unit weight of the dry loose material is 16.5 kN/m^3.

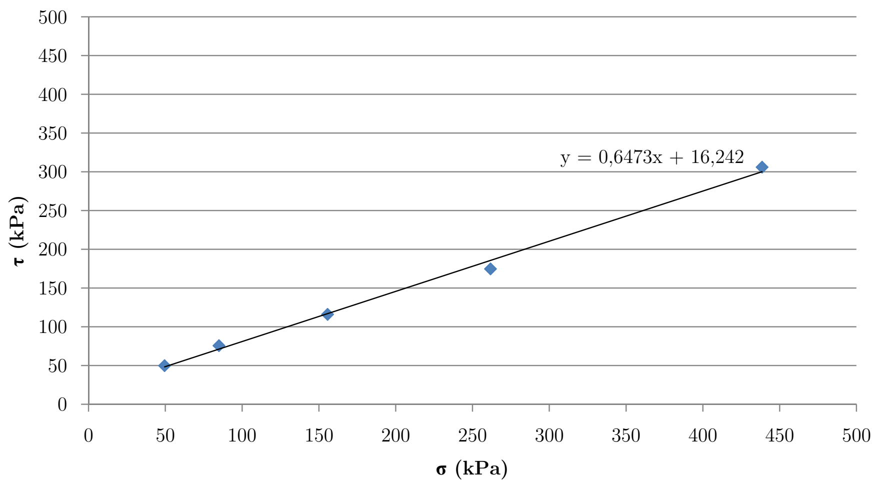


Figure 3.6: Mohr-Coulomb failure envelope for Soil B based on direct shear test results

Optimal conditioning properties

The soil does not have a good attitude to absorb water, probably due to the mineearalogy of the grains which are almost waterproof. In this case a low quantity of water is needed (the minimum necessary to hydrate the grains to improve the action of the foam. The grain size distribution reveals the good attitude to be conditioned, so no polymers or fine material (such as bentonite slurry) have to be added to the conditioned mass. Figure 3.7 shows the slump of the optimal conditioning set, while in Table 3.3 the parameters of this optimal conditioning set are listed.

Table 3.3: Optimal conditioning parameters for Soil B

Parameter	Value
Total water content, w (%)	2
Concentration of the foaming agent, c (%)	2
Foam Expansion Ratio, FER (-)	15
Foam Injection Ratio, FIR (%)	40
Concentration of the slurry (%)	-
Slurry Injection Ratio, SIR (%)	-
Slump (cm)	22

Figure 3.7: Optimal conditioning slump for Soil B

3.2.3 Soil C

This soil (Figure 3.8) represents the third finer material used for this study. According to Thewes et al. (2010), it is still suitable for the conditioning process.

Geotechnical properties

The soil has a friction angle, based on the linear interpolation of the direct shear tests, of around 39°, in dry and natural conditions. Figure 3.9 shows the linearization of the failure envelope, with the value of apparent cohesion equal to 31.7 kPa. The unit weight of the dry loose material is 14 kN/m^3.

Optimal conditioning properties

The soil does not have a good attitude to absorb water, probably due to the minearalogy of the grains which are almost waterproof. In this case a low quantity of water is needed (the minimum necessary to hydrate the grains to improve the action of the foam. The grain size distribution reveals the good attitude to be conditioned, so no polymers or fine material (such as bentonite slurry) have to be added to the conditioned mass. Figure 3.10 shows the slump of the optimal conditioning set, while in Table 3.4 the parameters of this optimal conditioning set are listed.

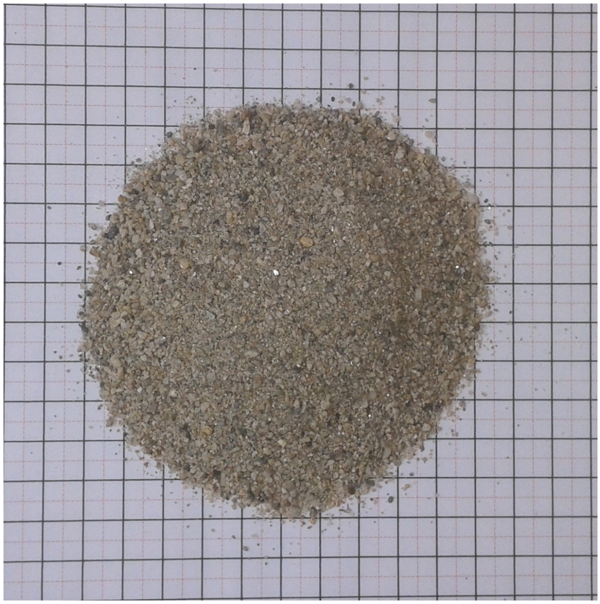


Figure 3.8: Soil C

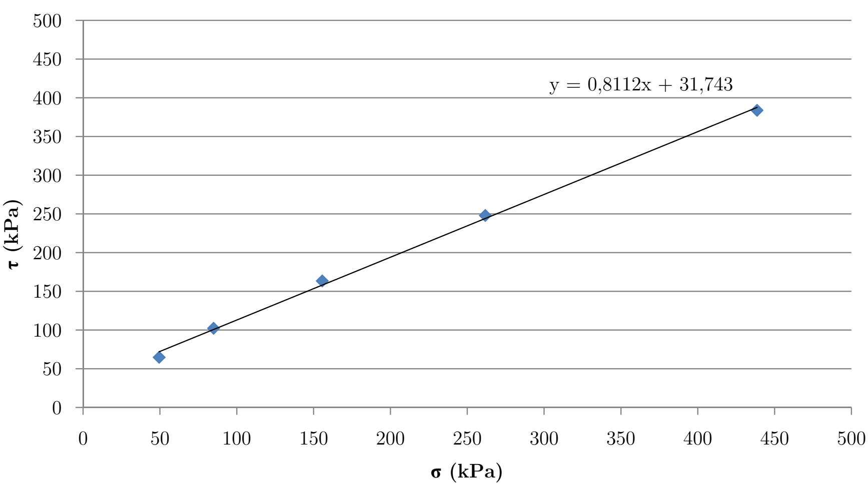


Figure 3.9: Mohr-Coulomb failure envelope for Soil C based on direct shear test results

Figure 3.10: Optimal conditioning slump for Soil C

Table 3.4: Optimal conditioning parameters for Soil C

Parameter	Value
Total water content, w (%)	2
Concentration of the foaming agent, c (%)	2
Foam Expansion Ratio, FER (-)	15
Foam Injection Ratio, FIR (%)	60
Concentration of the slurry (%)	-
Slurry Injection Ratio, SIR (%)	-
Slump (cm)	21

3.2.4 Soil D

This soil (Figure 3.11) represents the second finer material used for this study. According to Thewes et al. (2010), it is suitable for the conditioning process.

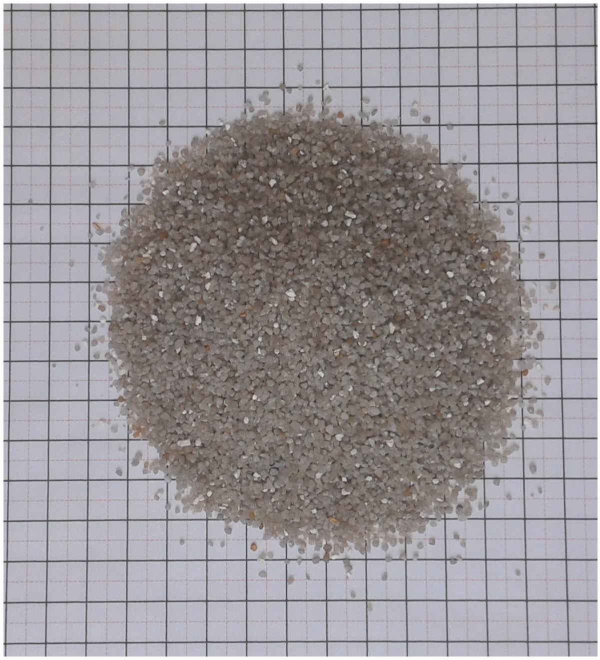


Figure 3.11: Soil D

Geotechnical properties

The soil has a friction angle, based on the linear interpolation of the direct shear tests, of around 44°, in dry and natural conditions. Figure 3.12 shows the linearization of the failure envelope, with the value of apparent cohesion equal to 33.8 kPa. The unit weight of the dry loose material is 15 kN/m^3.

Optimal conditioning properties

The soil does not have a good attitude to absorb water, probably due to the minearalogy of the grains which are almost waterproof. In this case a low quantity of water is needed (the minimum necessary to hydrate the grains to improve the action of the foam. The grain size distribution reveals a weak attitude to be conditioned, so fine material (such as bentonite slurry) has to be added to the conditioned mass. Figure 3.13 shows the slump without using the fine material, resulting in a liquid material like a slurry, which is not suitable according to the slump matrix (Figure 2.5). Figure 3.14

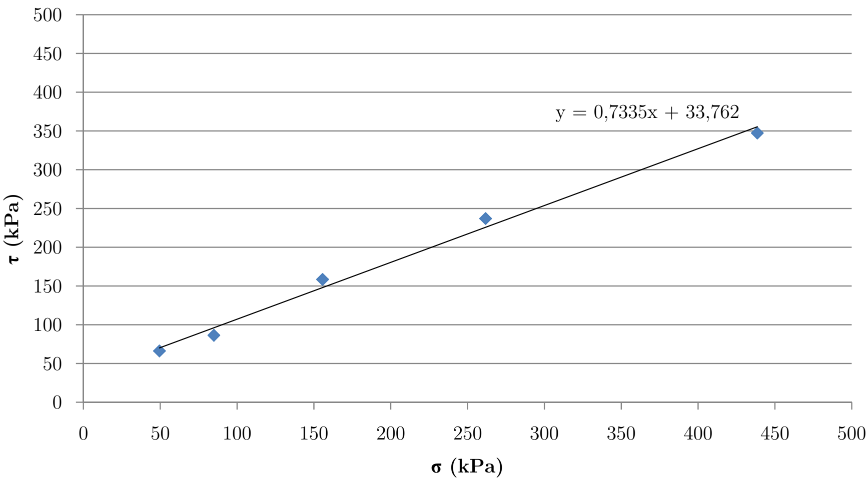


Figure 3.12: Mohr-Coulomb failure envelope for Soil D based on direct shear test results

shows the slump of the optimal conditioning set, while in Table 3.5 the parameters of this optimal conditioning set are listed.

Figure 3.13: Slump for Soil D without adding fine material

3.2.5 Soil E

This soil (Figure 3.15) represents the most heterogenous material used for this study. This is actually coming from a real job site. According to Thewes et al. (2010), it is

Figure 3.14: Optimal conditioning slump for Soil D

Table 3.5: Optimal conditioning parameters for Soil D

Parameter	Value
Total water content, w (%)	2
Concentration of the foaming agent, c (%)	2
Foam Expansion Ratio, FER (-)	15
Foam Injection Ratio, FIR (%)	40
Concentration of the slurry (%)	20
Slurry Injection Ratio, SIR (%)	15
Slump (cm)	23

still suitable for the conditioning process.

Geotechnical properties

The geotechnical parameters of this soil have been obtained by using the triaxial test and the results are given in Section 7.3.3, summarized in Figure 7.62. The unit weight of the dry loose material is 14 kN/m^3.

Optimal conditioning properties

The soil has a good attitude to absorb water in the fine part, while the coarse part should remain took over. In this case a low quantity of water is needed (the minimum necessary to hydrate the grains to improve the action of the foam. The grain size

Figure 3.15: Soil E

distribution reveals the good attitude to be conditioned, so no polymers or fine material (such as bentonite slurry) have to be added to the conditioned mass. Figure 3.16 shows the slump of the optimal conditioning set, while in Table 3.6 the parameters of this optimal conditioning set are listed.

Table 3.6: Optimal conditioning parameters for Soil E

Parameter	Value
Total water content, w (%)	10
Concentration of the foaming agent, c (%)	2
Foam Expansion Ratio, FER (-)	12
Foam Injection Ratio, FIR (%)	50
Concentration of the slurry (%)	-
Slurry Injection Ratio, SIR (%)	-
Slump (cm)	18

Figure 3.16: Optimal conditioning slump for Soil E

3.3 Extraction tests

Based on the results obtained through the slump testing campaign, extraction tests have been carried out on two representative soils, the soil A and E.

3.3.1 Soil A

The soil A has been studied through the extraction test in the last years by Vinai (2006) and Borio (2010) and thus they have not been repeated for this research.

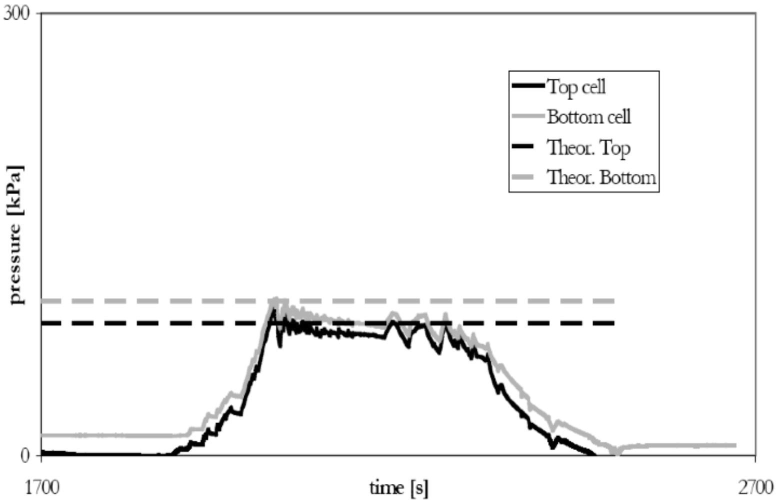


Figure 3.17: Sand A. Recorded pressures at the top and the bottom of the tank.

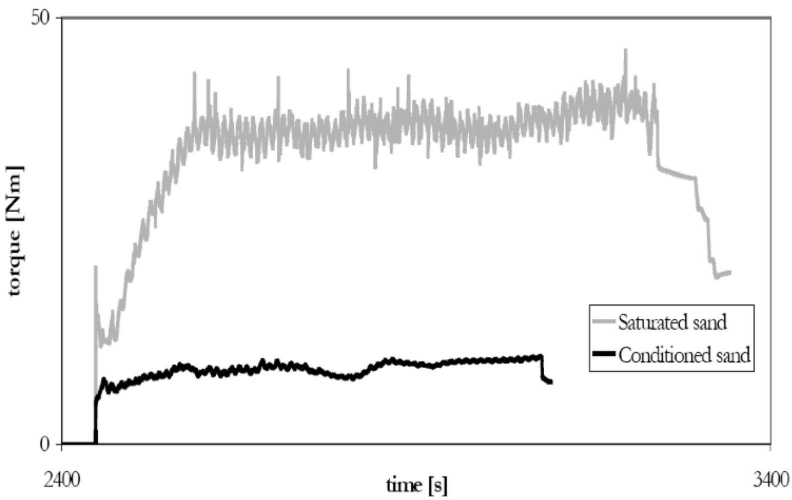


Figure 3.18: Sand A. Recorded torque.

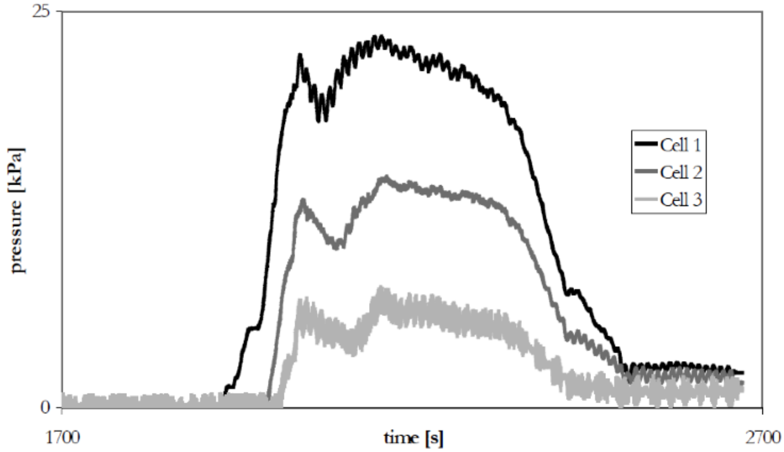


Figure 3.19: Sand A. Pressures recorded by the sensors installed along the screw conveyor.

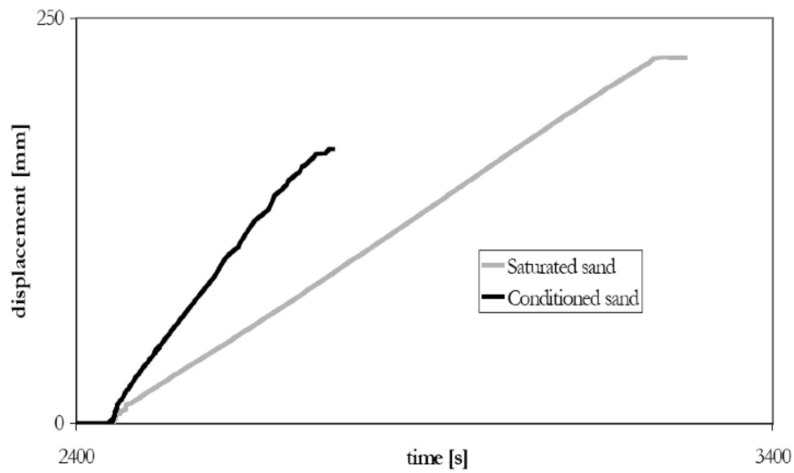


Figure 3.20: Sand A. Top plate displacement.

3.3.2 Soil E

For this test, carried out for a research contract for Mapei SpA, the client asked to study the material in two cases, dry material (Set 1, as used for the research of this thesis) and saturated soil (Set 2). This was due to the necessity of the client of assess the suitability of the material in case that the EPB machine would need to cross areas below the water table.

Set 1

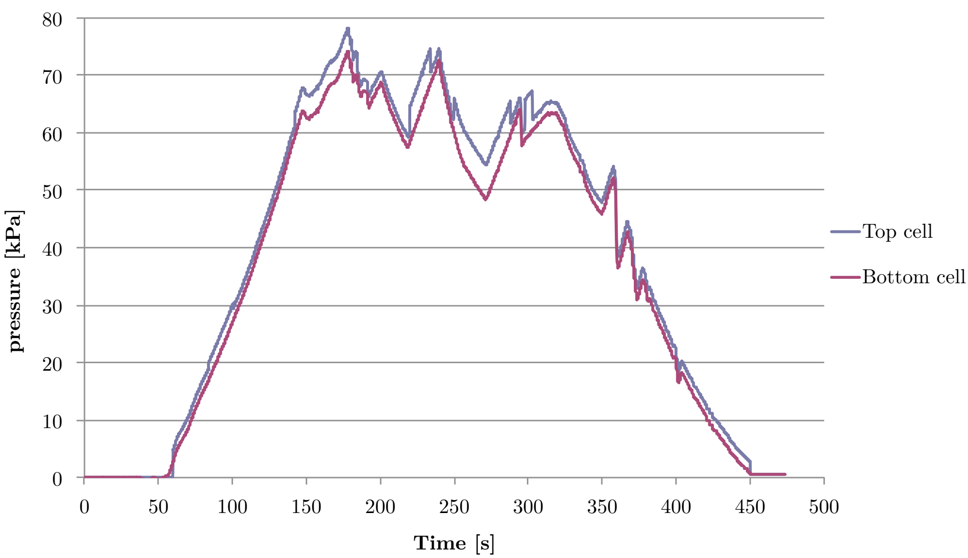


Figure 3.21: Sand E (Set 1). Recorded pressures at the top and the bottom of the tank.

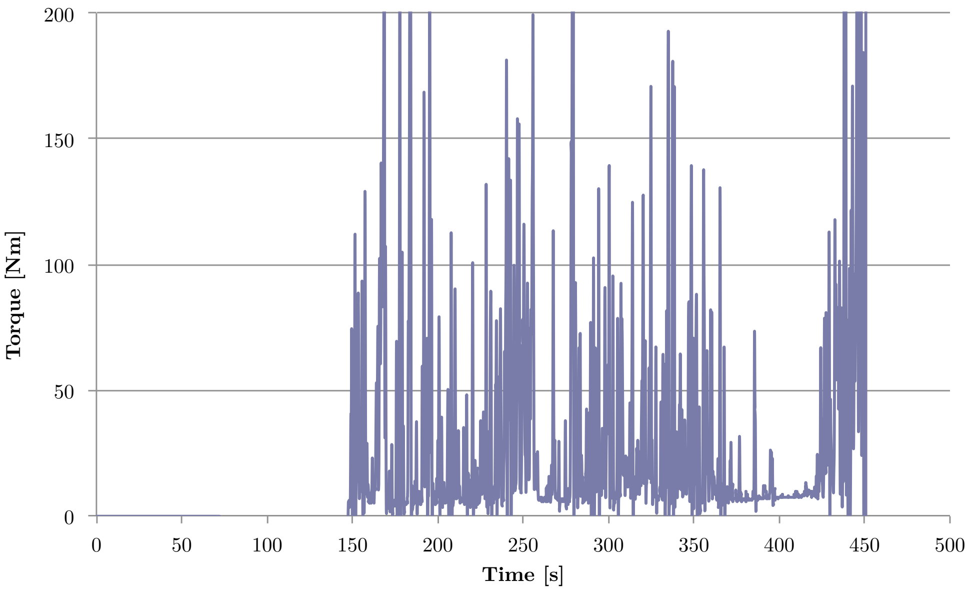


Figure 3.22: Sand E (Set 1). Recorded torque.

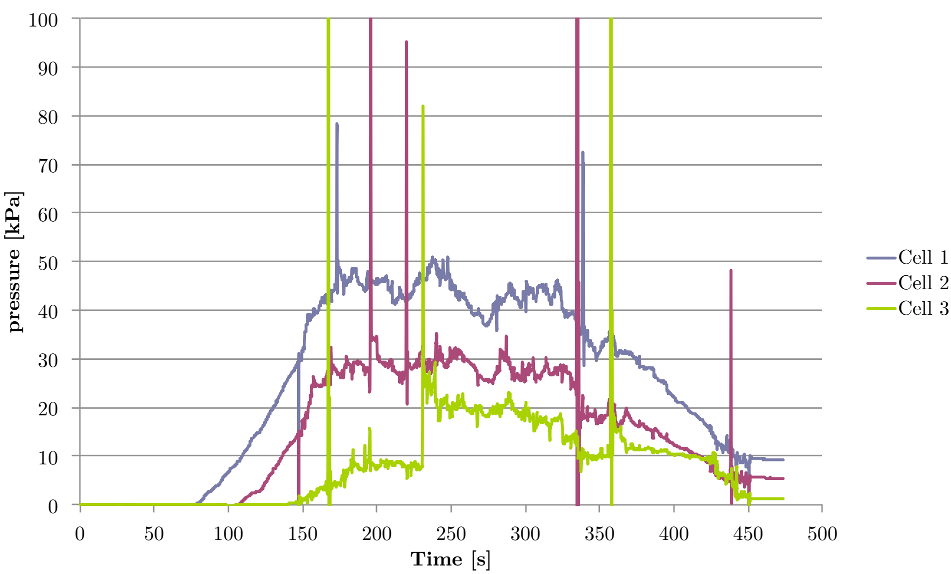


Figure 3.23: Sand E (Set 1). Pressures recorded by the sensors installed along the screw conveyor.

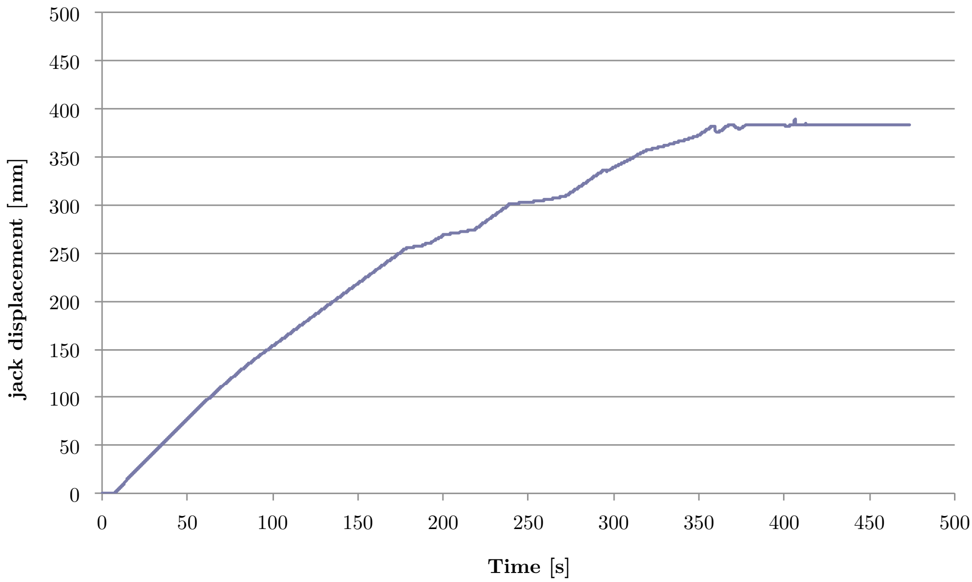


Figure 3.24: Sand E (Set 1). Top plate displacement.

Set 2

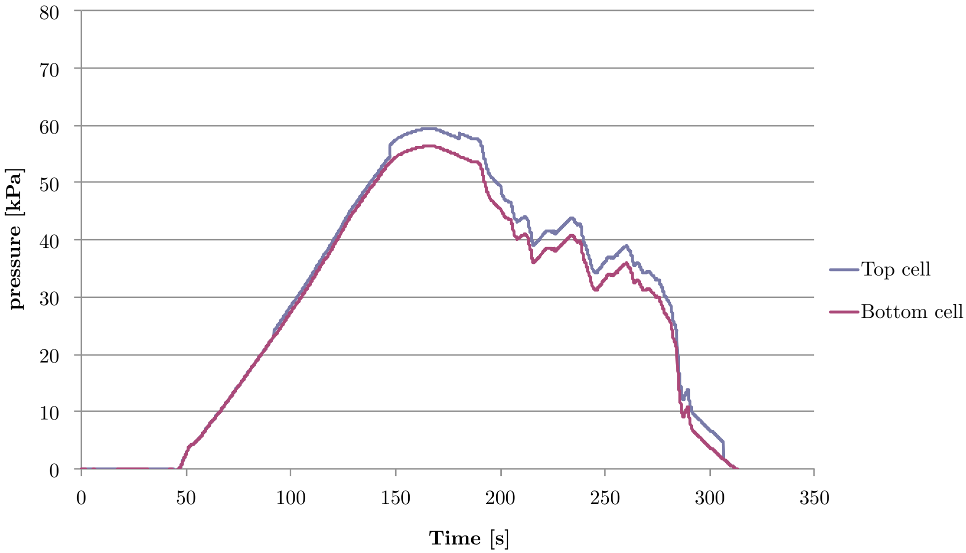


Figure 3.25: Sand E (Set 2). Recorded pressures at the top and the bottom of the tank.

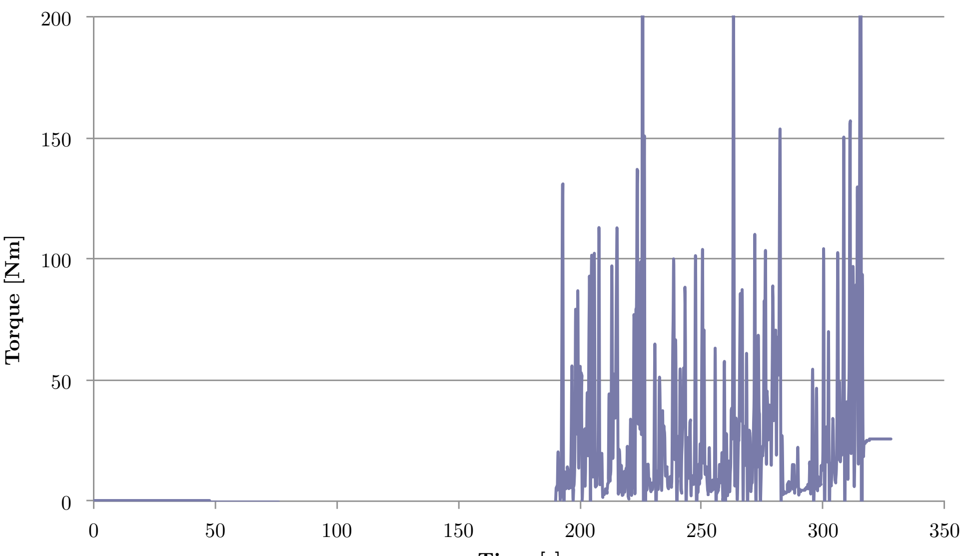


Figure 3.26: Sand E (Set 2). Recorded torque.

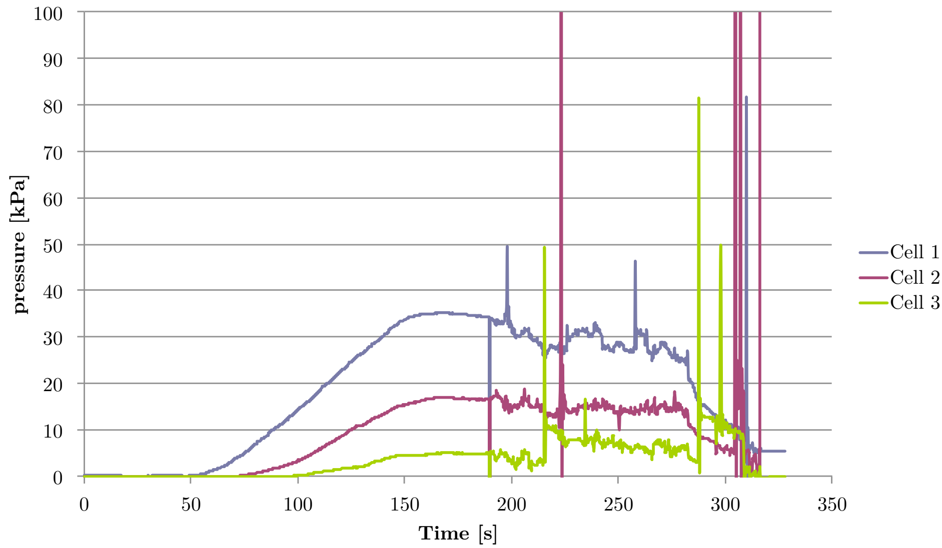


Figure 3.27: Sand E (Set 1). Pressures recorded by the sensors installed along the screw conveyor.

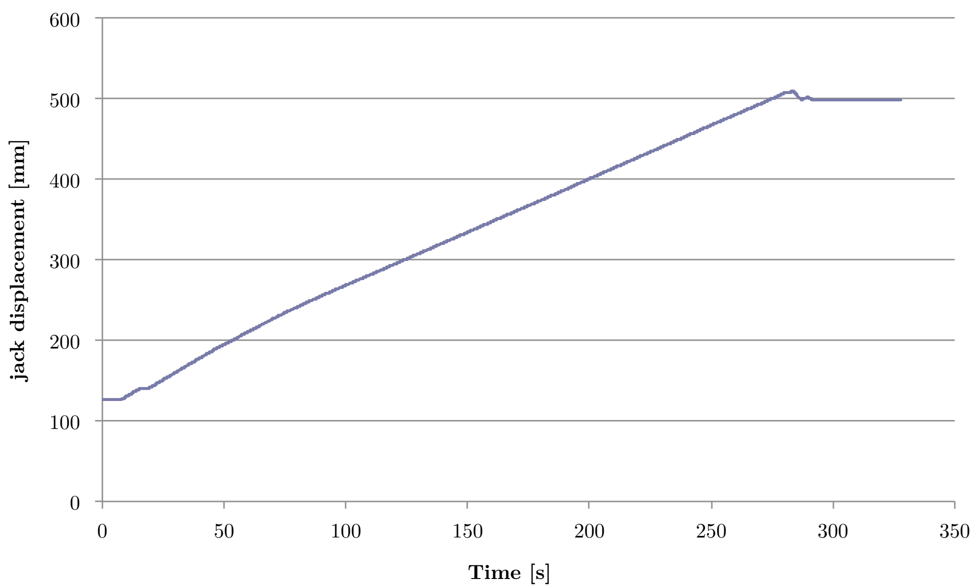


Figure 3.28: Sand E (Set 2). Top plate displacement.

3.3.3 Outcome of the tests

The tests carried out on Soil A and Soil E showed a good attitude of the conditioned material for the excavation in a EPB tunnel. This is underlined from the effective transmission of the pressures, which is visible from the good matching of the values of the top and bottom pressure cells, and also by the good flow of material through the screw conveyor, as shown from the torque graphs and as shown in Figure 3.29, where the material is regularly flowing from the screw conveyor at room pressure.

Figure 3.29: Flow of the material from the screw conveyor.

Chapter 4

Main issues concerning the common testing

4.1 Main aspects

In the previous section of the thesis (Chapter 2 and Chapter 3), the main testing procedures used on assessing the suitability of conditioning agents have been first introduced and described, and then applied for the study of different soil types.

The results obtained by the testing methods analyzed are giving just a qualitative assessment of the suitability of the conditioning process. Thus the common testing procedure for assessing the most suitable conditioning set for a soil is not exactly as the one used for instance for geotechnical engineering, where the usual tests are giving results which can be directly correlated each other by mean for example of constitutive models. Unfortunately, the complexity of such material as it is a conditioned soil, with the solid fraction trapped in the foam, does not allow in an easy way to use the same procedure.

4.2 Shear strength assessment with slump testing

As already stated in Chapter 2, the slump test is used mostly in the construction industry to assess the workability of the fresh concrete. For the soil conditioning study is useful to verify the applicability of the conditioning parameter set in different soils, the mass must be fluid but with a certain consistency. The effectiveness of this method, in comparison with the cost, the applicability and the rapidity of its use in the common practice, allowed to state that the slump test fits very well for the preliminary study of a conditioned mass. But in some cases the test is giving just partial answers and does not allow to assess correctly the effectiveness of the conditioning set for the excavation with EPB.

4.2.1 Shear strength verification for different conditioning sets

In order to verify if the result of the slump test is always giving a good indication of the shear strength of the conditioning mass, which is an important aspect for having an effective counterpressure at the front (this issue will be analyzed in detail in Chapter 5), a series of conventional shear tests have been carried on Soil A, by using different dosages and conditioning techniques, as well as on the natural and saturated sample. Table 4.1 is resuming the conditioning sets used for the tests.

The tests have been carried out also in this case with a Casagrande shear box. The values obtained are just indicating a overall total shear strength, without considering the effective stress and the pore pressure which eventually will be generated in the conditioned mass. Also this aspect, really important in this research, will be discussed in Chapter 5. The results of this campaign are summarized in Figure 4.1 and Table 4.2.

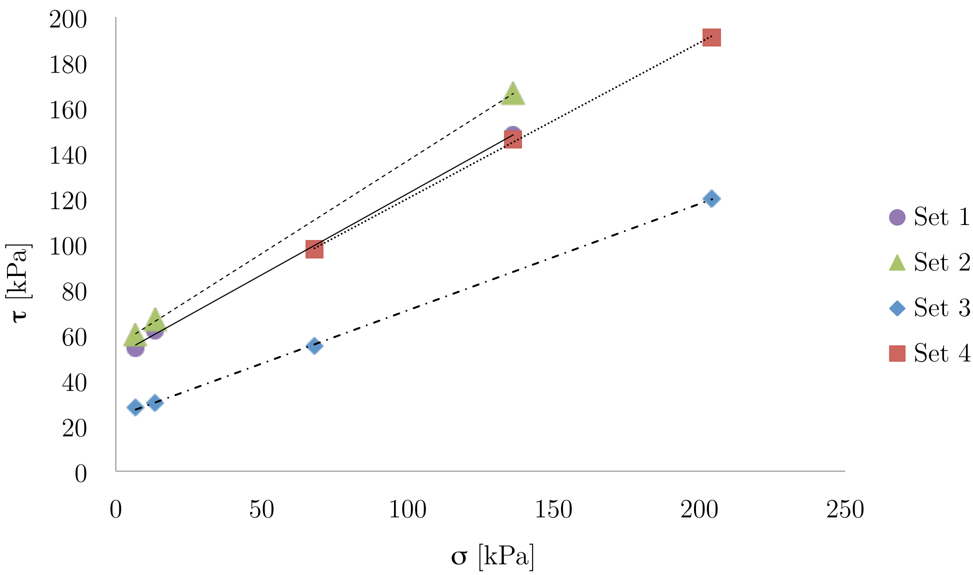


Figure 4.1: Results of the shear testing on Soil A with the 4 conditioning sets

As shown in the graph and in the table, the simple shear test is returning certain values of friction angle for the mass, depending on the conditioning set. Of course in case the material has no conditioning, thus in the dry and wet condition, the friction angle reaches the highest value (around 36°–37°). This value is reducing dramatically in Set 3, where the foam effect is giving a good fluidity to the mass. On the contrary in Set 4, the talc slurry effect is not reducing so much the total shear strength of the mass.

The results of this simple test, carried out in the same conditions for all the samples, are crucial for this research. First of all it is important to underline that the slump test results in Set 3 and Set 4 are equal (16 cm), while the shear strength assessment returned totally different scenario. This aspect is generating an important consideration to be taken into account: in some cases the results obtained from the two tests are not matching. This means that there is not always a direct correlation between the two methods.

Table 4.1: Conditioning sets used for the shear testing campaign

Set	Conditioning products	Parameter	Picture
1)	None	FER = - FIR = - SIR = - w_{add} = 0% w_{tot} = 0% Slump = -	
2)	Only water	FER = - FIR = - SIR = - w_{add} = 40% w_{tot} = 40% Slump = 3 cm	
3)	Foam + water	FER = 15 FIR = 80% SIR = - w_{add} = 20% w_{tot} = 25.3% Slump = 16 cm	
4)	Talc slurry c = 15%	FER = - FIR = - SIR = 6% w_{add} = 40% w_{tot} = 46% Slump = 16 cm	

Table 4.2: Shear parameters of the Soil A depending on the 4 conditioning sets obtained from standard direct shear testing

Set	Friction angle (°)
1	36.6
2	36.4
3	25.6
4	34.5

Another important issue about the results of this simple test is, in case of testing with the Set 3, that the points representing the peaks at different normal pressure are not linearly correlated, thus the Mohr-Coulomb envelope does not perfectly match. This is especially visible at lower normal pressures: the first point (normal pressure equal to 6.8 kPa) is clearly above the trend line. If just the two first point are considered (normal pressure 6.8 kPa and 13.6 kPa) for all the Sets, the results found are really interesting: with the dry sample (Set 1), the value is comparable, returning a slightly higher value of friction angle (around 40° compared to 36° obtained with the linearization of all the results) which is quite normal in loose samples; on the contrary with the sample conditioned with foam (Set 3) the friction angle drops dramatically (from 25.6° to 12°), which is uncommon in shear testing. This is probably due to the bubbles of the foam which are trapped between the grains are not breaking and transforming into liquid. In this way if the pressure is low, the foaming agent is working in bubble form, and it is leaking as liquid; once higher pressures are reached the bubbles are breaking and becoming liquid which is free to flow out from the sample.

4.3 Outcome of the extraction test

The extraction test represents at the moment the most important test to assess the suitability of a conditioned mass. The sensors are able to return crucial parameters to understand if the conditioning applied to a certain soil could work effectively in a certain tunnel project. Despite this, there are still some aspects which have to be taken into account.

First of all the device cannot study the soil in its initial state (dry or wet), because the torque on the screw conveyor would be too high. Surely this is not so crucial aspect, as in general with this test it is important to verify if it is suitable for a EPBS application. Nevertheless the torque obtained is not a value directly comparable to the one expected on a real machine, it is just a general indicator of the quality of the flow along the screw. For a better study of the shear strength of the material, additional testing is required.

The most important aspect to be taken into account with this device is the impossibility of mixing the material during the test. This is the most critical feature of the extraction test, as it probably represents the only main difference from the real machine mechanism, where the material is continuously kept in rotation and thus mixed. This characteristics leads to the possibility that the conditioned material is drying out during the first phases of the test, with a flow of liquid along the screw due to the pressure pushing the whole material in the screw. This usually happens more on less homogenous and on permeable materials, which have a stronger trend to segregate from the conditioning, while on homogeneous and more fluid material this could also not appear (Figure 4.2). Although this, after several tests carried out during the years, this feature is used as additional parameter for assessing the good homogeneity of a conditioned mass. The most used indicator for assessing this characteristics is the creation of a crater during the extraction (Figure 4.3); in this case the material is not following a regular flow and is segregating more clearly.

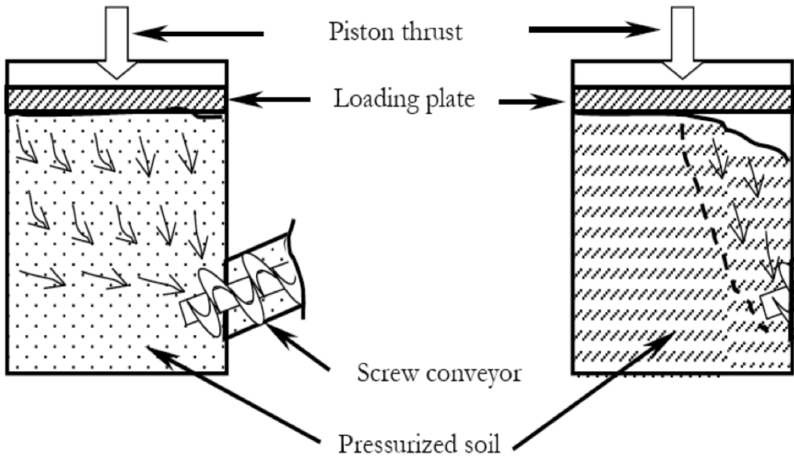


Figure 4.2: Scheme of the extraction mechanism. (Left) Homogeneous material. (Right) Non-homogeneous material where only a portion of the soil volume is involved in the extraction phase (Borio, 2010).

Figure 4.3: Creation of a crater over the screw conveyor, as visible at the end of the test when removing the top plate.

Part III

New approach on studying conditioned granular soils

Chapter 5

New approach for testing conditioned masses

5.1 General aspects

Before starting the description of the new testing methods, it is necessary to introduce and describe the issue concerning the new approach. The most important aspect regarding the study of the conditioned soil is to understand clearly the mechanical behaviour when the mass is stressed by a pressure. This is crucial in order to assess the suitability of a conditioned mass during an excavation. As already said in Chapter 1, the mass has to be fluid enough to apply effectively the counterpressure to the front but with a consistency pulpy enough to be extracted from the screw conveyor. The second aspect can be efficaciously studied by mean of tests such as slump test, which can give good indications on the consistency of the conditioned mass. The first aspect is on the contrary a bit more difficult to be assessed with standard tests, as no clear indications on the pressure transmissivity can be studied. This is a key aspect for the present work, and this new testing approach is an attempt to investigate it.

First of all it is necessary to imagine a model of the problem to be studied. In a EPB tunnelling project in cohesionless soil, we are dealing with a soil which is excavated, mixed with a conditioning agent (usually water and foam) and then strained with an external stress. In the excavating chamber this stress is represented by the compression of the conditioned material with other material up to the needed counterpressure. The stress is ideally hydrostatic, that is the reason why the conditioning has to bring to the mass sufficient fluidity. The best way to represent in the laboratory such situation is to apply a confinement to the conditioned mass, in order to reproduce the excavating chamber as a cylindrical pressurized tank with one of the bases which represent the cutterhead and thus which can be able to move and virtually apply a pressure.

Of course the best material to apply the pressure in such situation is a fluid like the water or the air, because by definition when a pressure is applied on these, it is transmitted immediately in all the direction hydrostatically. On the contrary a soil is not able to transmit the pressure is such way, for example in a natural deposit close to the surface the vertical stress is given usually by the weight of the soil itself, but the horizontal resulting stress is usually lower and is function of the friction angle ϕ. Considering a normally consolidated granular deposit, the K_0, which is called at rest lateral earth pressure, representing the in-situ lateral pressure coefficient, is equal to (Jaky, 1948):

$$K_0 = 1 - \sin\phi' \tag{5.1}$$

As clearly achievable from the Equation 5.1, in order to obtain the hydrostatic condition and therefore a K_0 equal to 1, the effective angle of friction must be equal to 0°. This drop of friction angle can be obtained by mixing conditioning agents with the cohesionless mass.

In order to establish the true mechanical behaviour of the soil after conditioning, it is crucial to maintain the foam and the liquid trapped inside the mass, otherwise the actual behaviour cannot be established. By applying the pressure without a confinement, for instance a piston in a tank which is not completely sealed, we can observe a substantial fluid loss (water and foaming agent). This will cause a wrong assessment of the actual mechanical behaviour of the mass, as the intergranular voids will loose the presence of the bubbles and the trend will be to have less space between the grains.

5.2 Undrained condition

In order to avoid this fluid loss in the conditioned mass, the best solution would be to seal all the possible gaps where the liquid could flow away. For this reason, the situation that has to be studied is similar to the undrained condition used usually in geotechnics performing triaxial tests. In that case the water in the intergranular voids, which is produced by the external pressurization when the sample is saturated, is creating a pore pressure. In general in geotechnics the definition of undrained condition is directly linked to the pore pressure, and this condition is encountered when the rate of loading is high relative to the soil hydraulic conductivity, so that water cannot escape from the pores during loading (Lancellotta, 2009). This condition, performing a triaxial test, indicates the circumstance in which a soil element (i.e. locally) cannot exchange water mass with the surrounding ambient. If soil is saturated and both particles and water are assumed to be incompressibile, the above definition means that the undrained condition is a constant volume condition. Because of this constraint, an excess pore pressure develops and increments of effective and total stresses do not coincide.

Considering the above mentioned definitions, it is immediately clear that the undrained condition used for testing conditioned soils cannot strictly coincide with the geotechnical one. This is mainly due to the fact that the conditioned sample is not saturated with water, it is usually in a condition close to the saturation but most of the pores are filled with foam bubbles. In this scenario it is thus clear that compared to a sample saturated with water, the conditioned sample is compressible, therefore the constant volume condition is not fulfilled. The saturation of the conditioned sample with water and foam is crucial in order to transmit effectively the pressure. If this is not happening, the material once compressed does not immediately transfer the pressure in all the directions, as the fluids are first absorbed by the drier mass. Thus the condition we are considering for testing the conditioned samples is just partially equal to the undrained condition used in geotechnics; in the samples studied in this research the medium is compressible and the pore pressures develop from a mixture of water, foaming agent and mostly air.

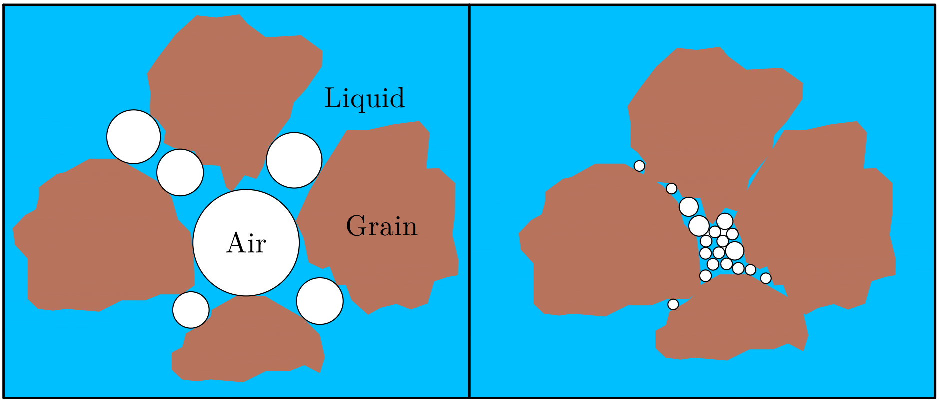
5.3 Pressure contribution

As already stated in Chapter 1, the mechanical behaviour of the conditioned material in certain pressure conditions is not clear and the past studies are not clearly clarifying the concept. The pressures acting on the material are, according to Terzaghi's theory (Terzaghi, 1923, 1936). The theory states that the stress in any point of a section through a mass of soil can be computed from the total principal stresses σ_1, σ_2 and σ_3 which act at this point. If the voids of the soil are filled with water under a stress u the total principal stresses consist of two parts. One part u acts in the water and in the solid in every direction with equal intensity. It is called the neutral stress (or the pore pressure). The balance $\sigma_1' = \sigma_1 - u$, $\sigma_2' = \sigma_2 - u$ and $\sigma_3' = \sigma_3 - u$ represents an excess over the neutral stress u and it has its seat exclusively in the solid phase of the soil. This fraction of the total principal stress will be called the effective principal stress. The theory of the effective pressure in EPB tunnelling has been treated especially by Anagnostou and Kovári (1996b), where a distinction is given between fluid-pressure and effective pressure in the chamber. In this case, the effective pressure can be visualized as a grain to grain contact pressure between the muck and the ground at the face. The water pressure in the chamber reduces the hydraulic head gradient in the ground and, consequently, the seepage forces acting in front of the face. Considering the front stability, the face is thus stabilized both by direct support of the pressurized muck and by the reduction of the seepage forces in the ground. The difficult point which has to be better studied, and that is object of this research work, is the influence and the contribution of the foam bubbles inside this theory.

In EPB tunnelling, the material is usually conditioned under a certain stress condition, which is not zero. Thus the studies carried out on the conditioned soil should be performed under particular pressure conditions, like in the excavating chamber in operation. This issue is quite complex to be taken into account: the addition of foam and other conditioning agents usually done by the laboratory operator is done in room conditions, and the representation of the pressurized conditions is difficult to do.

The aim of this aspect is to verify the behaviour of the conditioned material in different pressure conditions. If in one side the slump test is generally giving a good answer and response on the quality of the soil material for EPB tunnelling, on the other hand it cannot give indications on its behaviour under different stress conditions. This is crucial, especially considering that a key parameter of the conditioning is the FER, which is representing the expanding ratio and which is strictly linked to the pressure. Considering that the conditioned soil is a multiphase medium, composed of different material with different compressibilities, thus the study cannot be performed easily.

In this context new testing procedures would solve this important issue. Theoretically, the application of the stress on the conditioned mass would cause a large deformation in the first phase, as the bubbles of the foam are the first to be strained due to the higher compressibility of the air; in the second phase, once the intergranular voids between the grains are small enough to allow again the contact of the soil (as it usually happens in the natural soil), the deformability is different and also the stress application behaviour of the soil itself. In this second stage it is normal to think that the deformability of the medium will decrease and the hydrostatic transmission of the pressure would be much more difficult. Figure 5.1 shows the mechanics of the conditioned mass when the pressure is applied: at room pressure the grains are not in contact (Figure 5.1a), after the application of the pressure the grains are moving closer when the bubbles are deforming much more compared to the soil (Figure 5.1b).



(a) (b)

Figure 5.1: Conditioned mass before (a) and after (b) the pressure application.

This consideration explains why this study is crucial in the future laboratory testing procedure, as the rigidity of the conditioned spoil is dependent on the stress. A material that is too stiff in the excavating chamber can cause an increase of torque and temperature, with possible faults and severe damages to the machine.

An important and recent study which is going on in parallel to this research is represented by the work by Mori et al. (2015) and Mooney et al. (2016). Their research illustrates how pressure influences conditioned soil behaviour and how atmospheric test results must be viewed in the context of expected chamber pressures taking into account, through digital image analysis, the influence of pressure on bubble-soil interaction (including with time). The study is aimed to assess the compressibility, shear strength, and abrasivity of conditioned soil under pressure explained in terms of density, soil and air compressibility and porosity.

Chapter 6

Design of experimental devices and procedures

6.1 Foam compressibility test

An experimental set up was designed at TUSC laboratory in order to understand the behaviour of pressurized foam, studying the variation of the expansion ratio in a confined environment. The former idea was the comprehension of the compressibility law of the foam, because even if this works mostly under pressure in EPB applications, the definition of the conditioning dosage and testing of the conditioned soil is principally done under atmospheric pressure. The test consists on a transparent plexiglass tube sealed in both extremes with a tap on each side and a manometer that allow the monitoring of the pressure inside the set up (Figure 6.1).

Test procedure

The scope of this test is the measurement of the Foam Expansion Ratio under a pressure similar to that registered in the excavation chamber, and its comparison with the theoretical compressibility law of the gases defined before as:

$$FER_p = \frac{Pa}{P}(FER_0 - 1) + 1 \tag{6.1}$$

With FER_0 the expansion ratio a measured under the atmospheric pressure P_0 and FER_p the ratio at p pressure. The testing sequence have been established as follows:

1. a foam specimen is sampled following step 1-4 from the FER definition procedure;
2. once the FER_0 is determined, the foam hose is connected to the pressurizing chamber;
3. foam is pumped until a steady flow is verified through the cylinder;
4. the outlet valve is then closed;

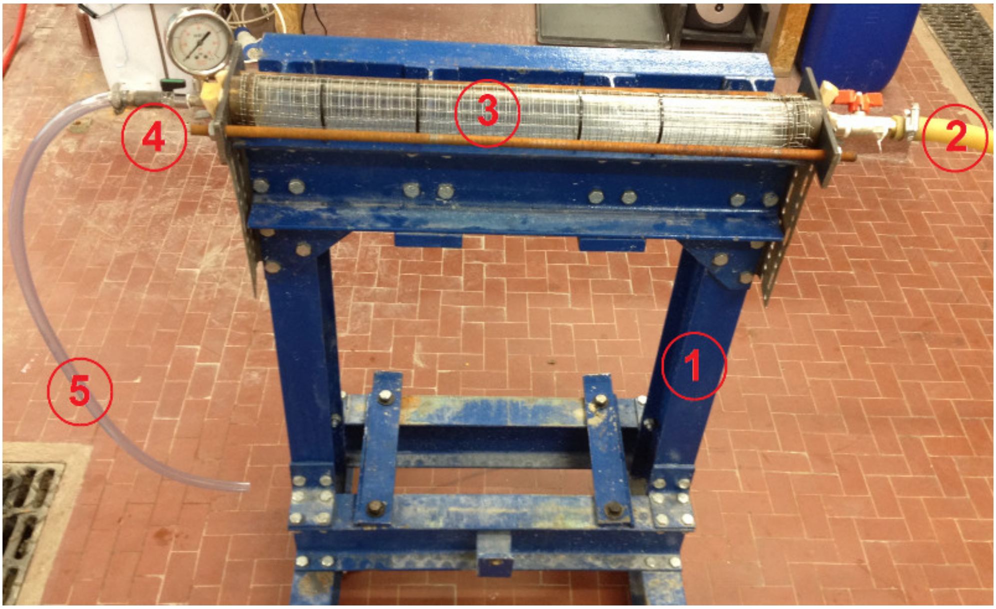


Figure 6.1: Foam compressibility experimental device (TUSC laboratory) main parts: 1) fixing frame, 2) foam inlet hose with its corresponding valve, 3) cylindrical pressure chamber with protective frame, 4) foam outlet provided with a valve and a manometer, 5) outlet hose.

5. foam pressure is measured continuously through the manometers installed at the foam generator unit and in the chamber until the desired value is reached (p) when;

6. the inlet valve is closed;

7. the foam hose is disconnected, the foam generator is turned off and the whole set up (cylinder full of pressurized foam) is weighted;

8. FER_p is determined as:

$$FER_p = \frac{Cylinder\ volume}{Total\ weight - Cylinder\ weight} \tag{6.2}$$

9. the outlet valve is re-opened and the chamber returns to atmospheric pressure but still full of foam;

10. the set up is weighted another time in order to determine the residual expansion ratio FER_r.

6.2 Undrained direct shear test

The apparatus is the modification of an existing Casagrande shear box located at the geotechnical laboratory of the Department of Structural, Geotechnical and Building Engineering (DISEG).

6.2.1 Design of the new apparatus

The design and modification of the shear box have been a trial and error process that started from brain-storming and graphical schemes from the gained experience while evolving the tests before described. Three main critical leakage points were identified during the tests (Figure 6.2):

- Sliding surface: gap between upper and lower portions of the box;
- Loading pad: space between the vertical load application device and the soil specimen; chamber.
- Bottom cap of the lower half.

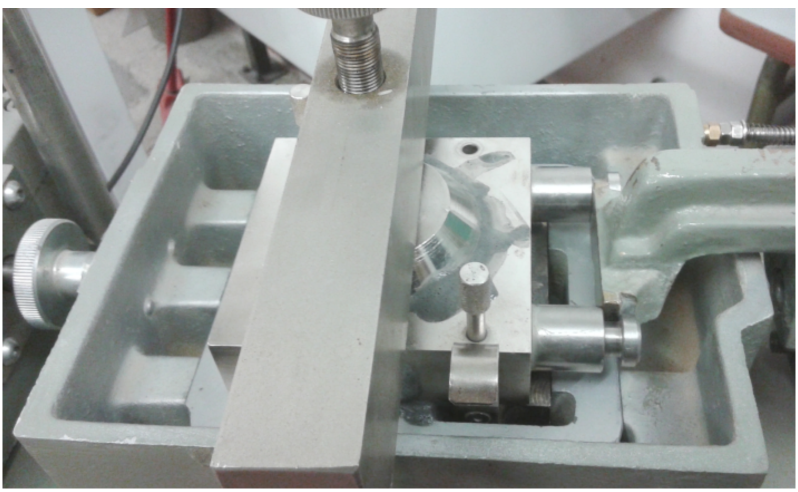


Figure 6.2: Fluid loss from several points of the shear box during shear stage.

Sliding surface sealing

In such a way to seal the remaining gap between the two portions of the assembly, after some failed attempts, the final decision was installing a C shaped rubber joint bonded to the lower part and a closure joint at the upper portion achieving this way a smooth relative displacement. Tests carried out with pressurized water still presented some losses even at low vertical load values. A further addition was the use of high viscosity grease, with a strong adhesive consistence that bounded both surfaces with a negligible shear resistance. This combination produced satisfactory results under water pressure even for values up to 35 kPa approximately. Figure 6.3 depicts the two parts of the box before the test, with rubber seals and two kind of grease. High viscosity grease (black one) was used in the outermost of the surface over the joints and a regular mechanical grease was employed to fill the remaining spaces and to create a regular sliding surface.

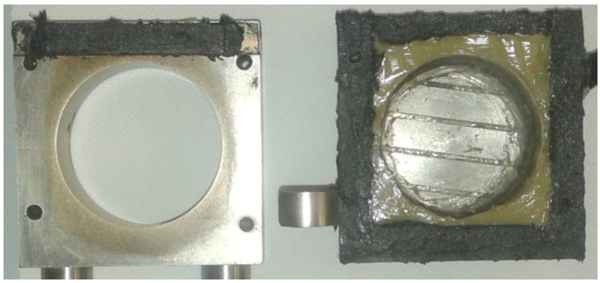


Figure 6.3: Undrained shear box assembly before the test. Left: Upper portion with closure seal greased. Right: Lower part with C shaped joint greased and regular grease used to level the sliding surface (Winderholler, 2015).

Loading pad sealing

This problem have been solved by manufacturing a whole new pad (Figure 6.4) from an aluminum cylinder, providing the space for a special lip gasket usually used for hydraulic pistons, a high viscosity grease was required also in this case. In order to allow the air outlet during the insertion of the pad, a 3 mm perforation have been done (Figure 6.5). Once the soil specimen is reached, a special cap is positioned and the set up is ready for the test.

Bottom cap sealing

This issue have been solved trivially by glueing this component to the lower half of the box by silicone adhesive. There was not necessary removing this device during tests and leakages problem were not found any more (Figure 6.6).

6.2.2 Final assembly of the undrained direct shear box

Finally after the before detailed modifications have been done, the shear box was ready for the undrained characterization campaign that will be described in the following chapters. Even though some fluid loss were verified anyway at high pressures, this apparatus consent to test conditioned materials in the range of 0–35 kPa of normal stress and for a horizontal displacement up to 6 mm ($\epsilon = 0.1$). The results obtained differed greatly from those found before the transformation process recording shear

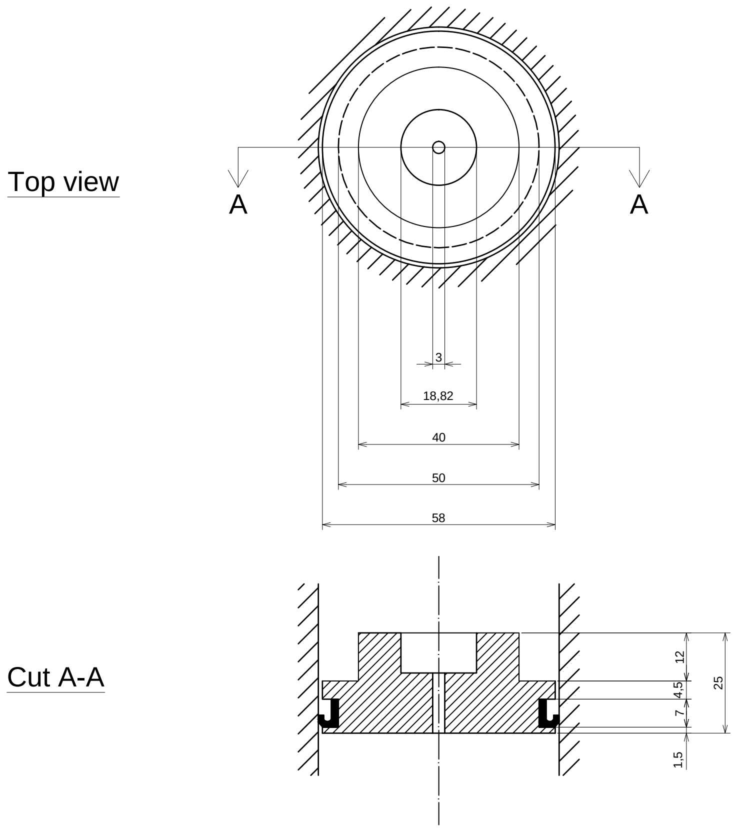


Figure 6.4: Design of the watertight loading pad.

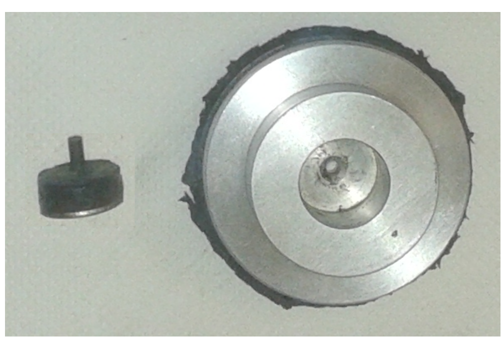


Figure 6.5: Loading pad with greased o-ring and air outlet cap before the test.

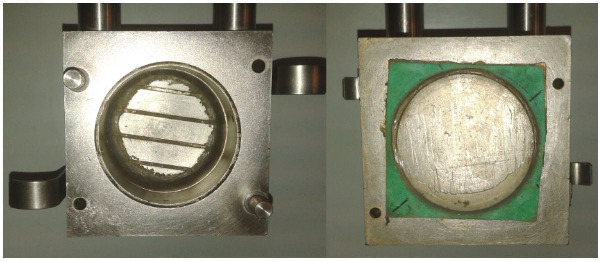


Figure 6.6: Detail of the bottom cap sealing solution. Left: perimeter seal with silicone glue (top view); Right: closure of the bottom surface with a thickness joint and silicone.

stresses definitely lower. The material after the test was still conditioned and pseudo-fluid, so it is possible to say that the compression-bursting process of the bubbles diminished considerably. A proper image of the final assembly design is depicted in (Figure 6.7).

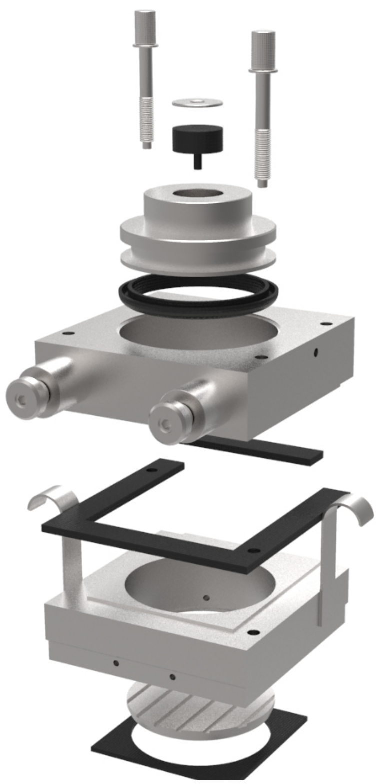


Figure 6.7: Exploded view of the Undrained direct shear apparatus.

6.2.3 Empty run tests

Once the whole undrained apparatus was mounted, a empty run campaign have been evolved in order to register the residual friction added by the parts running in contact during the stroke of the test machine without filling the shear box with the soil specimen. From a number of about 10 tests (Figure 6.8) the representative relation for the shear force provided by the empty box have been found as the average shear force–horizontal displacement registered.

In such a way to subtract the friction generated between the rubber joints greased and the metal parts, the linear function of the representative registered force have been found (Figure 6.9). For each test carried out this quantity is then deducted from the total registered force and this way the soil specimen shear strength (τ) is calculated by 6.3 where r is the radius of the soil specimen (30 mm in this case).

$$\tau = \frac{\text{Shear force}}{\pi \cdot r^2} \tag{6.3}$$

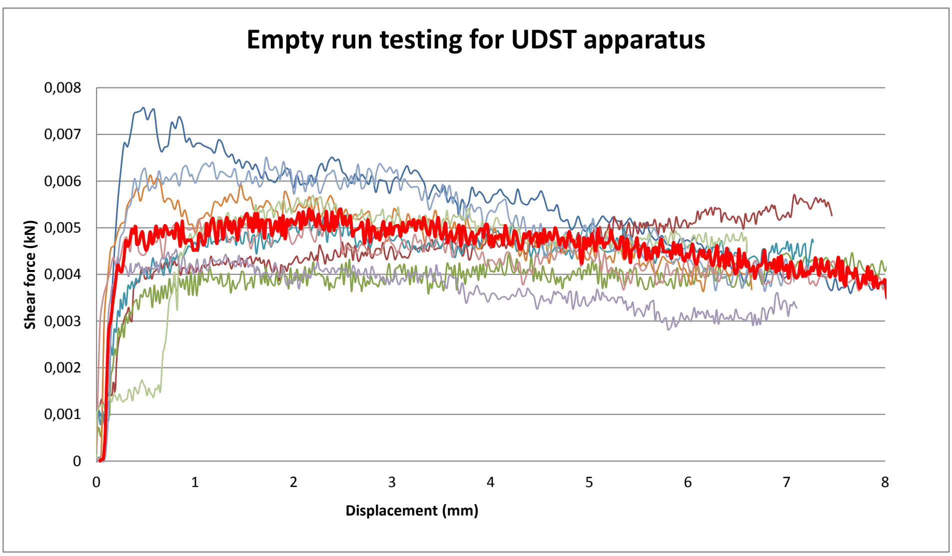


Figure 6.8: Empty run test campaign and representative relation (in red) for the shear force–horizontal displacement of the undrained direct shear test (UDST).

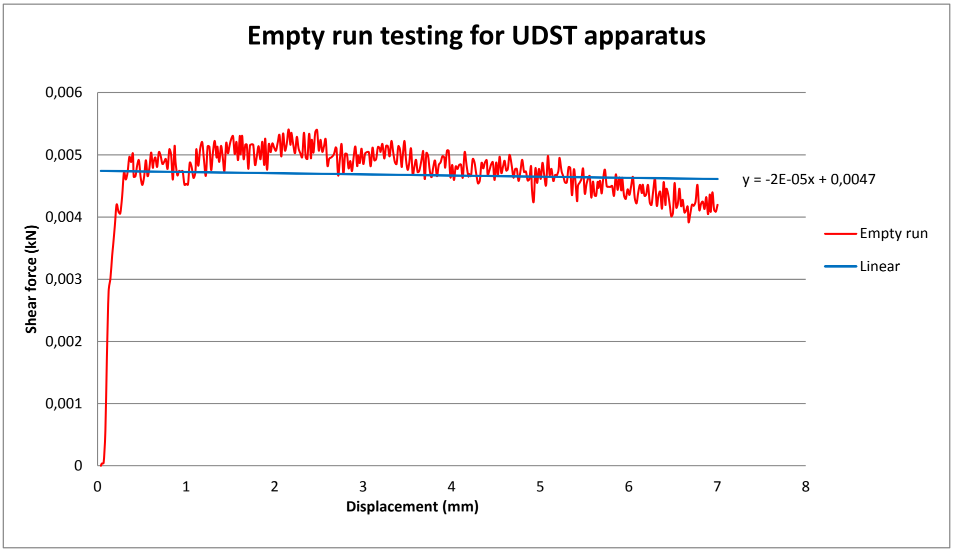


Figure 6.9: Representative registered shear force (red) function and its linear trend (blue).

6.3 Modified large diameter triaxial test

The idea of using a triaxial testing cell starts with the considerations listed in Chapter 5, as in order to study the conditioned material it is necessary to reproduce the conditions in the machine. This device allows to stress the material in a similar way than in an excavating chamber, that is mostly a hydrostatic pressure. Moreover, it is possible to apply to the material different stress paths to measure the behaviour of the conditioned mass in varying stress conditions.

Based on the above cited issues, also the diameter of the cell plays an important role: with a larger size of the sample, the contribution of the foam in the conditioned mass is more clear and visible, moreover a larger volume of soil can be verified before and after the testing by mean for example of a slump test, in this way the result can be compared easily and the condition of the mass can be assessed also after been stressed and after a certain amount of time. This aspect is really important, because it is crucial in order to verify the suitability of the material after a certain period, which in the site can represent the stop time due to unexpected events or long segment lining installation.

6.3.1 Triaxial testing

The triaxial test is an important test mostly used in geotechnical engineering for testing basically all kind of soils, from clays to sand, and also for rocks (ASTM, 2011a). This test is used to evaluate the shear strength, strain–stress behaviour, contractive and dilative response, and generation of pore-pressure of soils under axisymmetric state of stress and controlled drainage conditions (Fratta et al., 2007). As already test is applicable to any type of soil, both dry and saturated, and it is used not only to obtain design parameters for geotechnical engineering projects, but also to measure parameters used in geotechnical engineering research and modelling.

Ideally the triaxial test should allow independent control of the three principal stresses, so that generalized states of stress, including the important special case corresponding to plane strain. However, the relatively high compressibility of the soil skeleton and the magnitude of the shear strains required to cause failure lead to mechanical difficulties which make independent control too complicated for other than special research tests. The type of triaxial test most commonly used in research work and in routine testing is the cylindrical compression test.

In this test, shown diagrammatically in Figure 6.10, the cylindrical specimen is sealed in water-tight rubber membrane and enclosed in a cell in which it can be subjected to fluid pressure. A load applied axially, through a ram acting on the top cap, is used to control the deviatoric stress. Under these conditions the axial stress is the major principal stress σ_1; the intermediate and minor principal stresses (σ_2 and σ_3, respectively) are both equal to the cell pressure.

Connections to the ends of the sample allow either the drainage of water and air from the voids in the soil or, alternatively, the measurement of the pore pressure under conditions of no drainage.

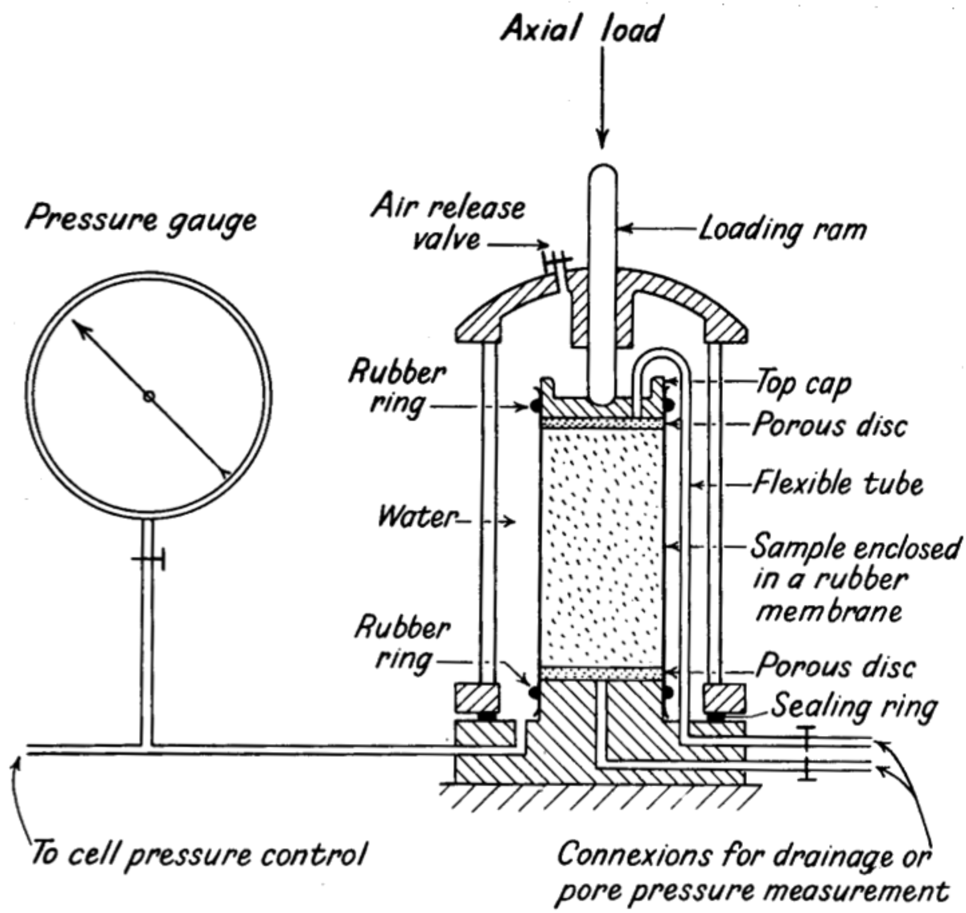


Figure 6.10: Diagrammatic layout of the triaxial test (Bishop and Henkel, 1967)

Generally the application of the all-round pressure and of the deviatoric stress form two separate stages of the test; tests are therefore classified according to the conditions of drainage obtaining during each stage:

- *Undrained tests.* No drainage, and hence no dissipation of pore pressure, is permitted during the application of the all-round stress. No drainage is allowed during the application of the deviatoric stress.

- *Consolidated-undrained tests.* Drainage is permitted during the application of the all-round stress, so that the sample is fully consolidated under this pressure. No drainage is allowed during the application of the deviatoric stress.

- *Drained tests.* Drainage is permitted throughout the test, so that full consolidation occurs under the all-round stress and no excess pore pressure is set up during the application of the deviatoric stress.

These classification may be further qualified, for special tests, by indicating, for example, whether failure is caused by increasing σ_1 or by decreasing σ_3. The state of stress during the consolidation stage may also be modified to give a principal stress ratio σ_1/σ_3 greater than 1.

Usually the mechanical parameters of a soil can be obtained through testing almost 3 samples with the triaxial test. The first part of the test is usually the consolidation part of the test, in which the sample is brought to the needed conditions.

As shown in Figure 6.10, the standard apparatus includes a loading ram which is used to transmit the axial pressure σ_a to the sample, thus is mechanically induced by the displacement of the top cap. On the contrary the radial pressure σ_r is hydraulically applied to the sample through a fluid (usually water and/or air) which is pressurized depending on the needed pressure. As the sample is cylindrical, the radial pressure is representing two of the three main components of the stress tensor, for this reason the test is in reality a biaxial test, where two components are equal. Depending on the stress path chosen for the test, the two pressures σ_a and σ_r can increase or decrease independently. Figure 6.11 illustrates the most common total stress paths, with the aim of clarifying terms such compression, extension, loading and unloading, depending on whether σ_a and σ_r is increasing or decreasing while the other is held constant. As in Figure 6.11, the stress paths are usually indicated in a p-q plane, with these two components p, also called mean stress, and q, also called deviatoric or shear stress, defined as follows:

$$p = \frac{\sigma_a + 2\sigma_r}{3} \tag{6.4}$$

$$q = \sigma_a - \sigma_r \tag{6.5}$$

The stress path is usually starting with the component q equal to 0, which is clearly indicating the isotropic state of stress at the beginning.

6.3.2 Apparatus

The apparatus used for this research (Figure 6.12) had been initially designed for testing undisturbed and disturbed samples of coarse soils, such as gravel and cobbles. The original design included the possibility of testing loose soils and cores of undisturbed samples obtained by using the freezing technique. The apparatus used has been designed by the staff of the geotechnical laboratory of the Department of Structural, Geotechnical and Building Engineering (DISEG) and the original design details have been introduced originally by Fiorio (2003). The main internal parts of the cell are shown in Figure 6.13.

The development of this apparatus, when it has been realized, had a double objective: on the one hand the realization of a mechanic structure characterized by a small deformability (rigid structure) able to reduce the measurement errors and on the other hand the development of a measurement system characterized by a high reliability on the small deformation field.

The main key mechanical characteristics of the new apparatus are:

- high stiffness of the structure;

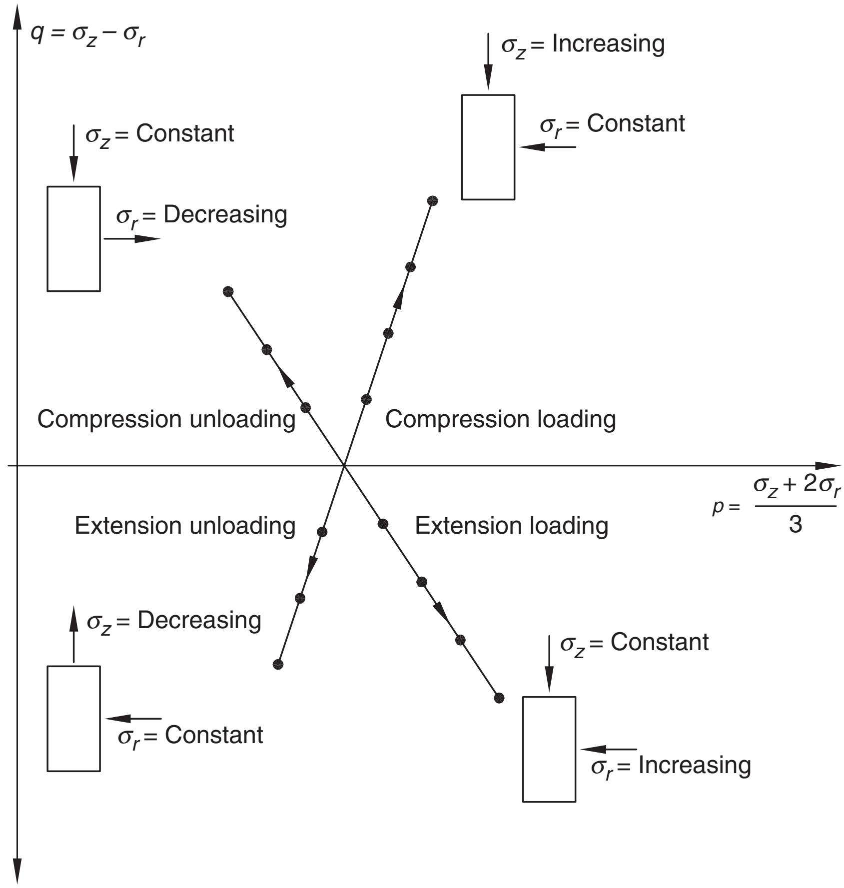


Figure 6.11: Typical stress paths performed with triaxial tests (Lancellotta, 2009)

Figure 6.12: Large diameter triaxial cell apparatus used for the research (Pirone, 2016)

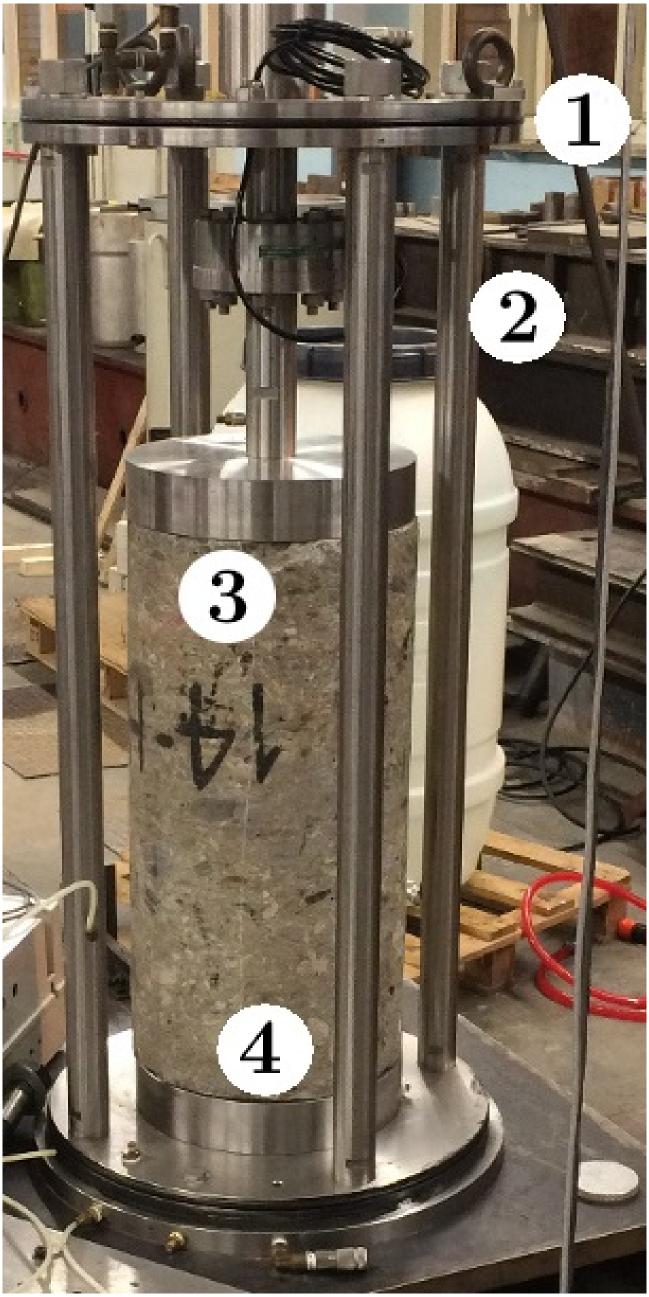


Figure 6.13: Main internal parts of the cell: 1) top plate, 2) steel bar, 3) top cap, 4) bottom cap

- the loading ram and the top are rigidly connected to the sample;
- the bottom and top plates are connected by 4 steel bars;
- the pressurized cell is outside the steel bars;
- the guiding system is virtually without friction;
- the correct alignment between loading ram, top cap of the sample and sample itself is guaranteed by precision mechanics production;
- the effect of the self weight of the system loading ram and top cap is undone by mean of a counterweight.

The structure is realized in stainless steel which guarantee as said the necessary stiffness to reduce possible errors of measurement caused by the deformability of the structure of the cell. These appear when the measurement of the axial deformation is carried out externally to the pressure cell. In some cases the external measure of the deformation is necessary in order to obtain a continuous sampling of the deformations from the 1% to the failure (usually up to 20%); in this condition the use of a transducer with dimensions which can allow the installation inside the pressure cell is necessary.

The 4 steel bars connecting the bottom and top plates, which are all inside the cell, are able to facilitate the assembly operations and moreover they allow to be able to realize a rigid connection between the top cap of the sample and the loading ram; this last expedient permits to carried out tests characterized by any stress path.

The cell is composed of a bottom cap (Figure 6.14) of 300 mm in diameter, which allows to accommodate samples 600 mm high. The sample is placed on the bottom stainless steel cap with the same diameter as the sample and with a thickness of 60 mm. On the upper face the cap has a deeper cross groove which collect the liquid passing through the porous stone which is inserted at the top of the bottom cap. This connection between the sample and the external part of the cell allow the possibility of drainage of water or measurement of pore pressure. For this research, the hole used for this purpose has been sealed to avoid the drainage of the foam along the tubes. In the upper part a similar plate is closing the sample, and also in this case there is a hole for the drainage. This hole has been left open both for creating the depression at the beginning which allows to close the triaxial cell, and moreover to measure the pore pressure generated by the conditioned mass strained in the cell.

This bottom cap element is fixed to a base plate (Figure 6.15) made of stainless steel, with a diameter of 530 mm and 60 mm thick. This element has been designed in order to:

- allow the passage of the drainage system, which is mostly composed of the line coming from the bottom cap and from the top cap, both connected through flexible pipes, from inside the pressure cell to the exterior;
- guarantee the sealing on the contact of the cell with the base itself, through a o-ring which is inserted in a groove;
- rigidly fasten the 4 steel bars.

Figure 6.14: Bottom cap

Figure 6.15: Base plate of the cell

Figure 6.16: Top plate of the cell with the ram and the load cell

The 4 stainless steel bars (diameter 45 mm) are 1025 mm high and they are equipped with o-rings in the upper part in order to guarantee the sealing in the upper plate.

In the upper part of the cell, a similar cap (same size than the one on the bottom) which is rigidly linked to the ram is placed in contact with the sample. As already stated the drainage circuit in this case has been kept in operation. Last important element of the top part is the top plate (Figure 6.16) which has the same thickness of the bottom one but a smaller diameter (489 mm) needed to allocate the pressure cell. Also in this case the design has been done in order to:

- rigidly fasten the 4 steel bars and the loading ram with a pressurized airtight seal;

- allow the passage, also in this case with a pressurized airtight seal, of the connecting cables for the transducers and the load cell from inside the pressure cell and the data acquisition device.

The cylindrical pressure cell, as the fluid used for pressurizing the sample is compressed air, is made of steel 1091 mm high, inner diameter of 491 mm and thickness of 16 mm. The bottom and top extremities are thicker to guarantee the tightness with the o-rings.

Even though the dimension of the apparatus is much larger than a common triaxial cell, the accuracy of the load transmission has the same importance, and thus also the loading ram needs a perfect alignment with the sample. This is especially complex due to the actuator which is providing the axial force, which is a large MTS hydraulic actuator located at the MASTRLAB laboratory of the Department of Structural, Geotechnical and Building Engineering (DISEG). This device, that is really precise on providing even small loads, has the problem of connecting the piston to the ram in order to have a perfectly axial load. To obtain this result, a swivel has been connected to a rigid steel frame which is holding the actuator and moreover the connection between the piston and the ram is done by using a radial spherical plain bearing, which is transmitting effectively the thrust axially to the ram. The ram is inserted in a guide (Figure 6.17) with two ball bearings which guarantee the perfectly straight direction on transmitting the load to the sample. Moreover, between the two bearings there is a length of 50 mm in which the ram is moving in a guide with a maximum tolerance on the diameter of 0.2 mm, guaranteeing the minimum loss of pressure which can be easily counterbalanced with the flow of air. At the top of the guide a system to lock the ram has been provided.

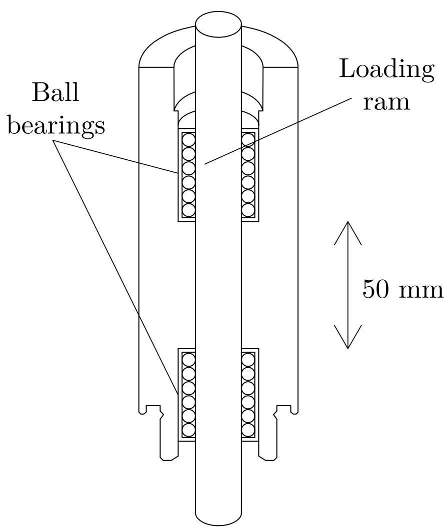


Figure 6.17: Guide of the loading ram

The loading ram, where is also located the loading cell, has a variable length depending on the test needed (compression or extension). The maximum stroke of the loading ram is around 150 mm,larger than the 20% of the sample height, so more than the recommended deformation needed for the triaxial test.

With such dimensions, the estimated volume of the sample is around 42 dm^3, much larger than a slump cone (its internal volume is around 5.5 dm^3). In this way it is possible to verify the state of the conditioned soil after the testing through a slump test, which can be performed for example on the soil at the top and and the bottom of the sample.

6.3.3 Concerns about the tests

The study of conditioned soil through undrained triaxial tests brought to a series of considerations about the applicability of tests which are mostly used in geotechnical engineering to materials which are not typical in this science. In fact from the past studies of conditioned soils it is known that this material cannot really be considered neither a granular material nor a fluid. The aim of this study is thus the possibility of verifying the total shear strength of the soil before and after the conditioning process with similar procedure, in order to obtain parameters which are directly comparable.

Nevertheless it is important to be careful when performing a standard geotechnical test on a conditioned soil, and especially it is necessary to consider two fundamental aspects:

1. the test procedure, the positioning of the sample and the drainage condition could modify the intrinsic nature of the conditioned material. This aspect was particularly visible in the loss of liquid in the direct shear test (Figure 6.2), which resulted in a reduction of content and dimension of the bubbles, and thus a modification of the conditioning parameters.

2. the constitutive laws which usually are applied on evaluating the results of a triaxial test have been obtained under particular hypothesis, with assumptions regarding the variation of volume or compressibility of the different phases. These issues might not be applicable for the conditioned soil, thus it is crucial to verify for each equation which one can be used.

During the testing campaign, started by using a compression loading stress path, which is regularly used in geotechnical engineering tests, the tests on the dry material brought satisfactory results. On the contrary, the use of this type of stress path in conditioned material did not allow to perform the test. This is mostly due to the fact that after the mold is placed to confine mechanically the sample, in order to remove the mold itself from the cell it is necessary to apply a negative pressure through the drainage pipe (usually 20–30 kPa are enough): for the dry sample (Figure 6.18) everything worked fine as the sample has no liquid; for the conditioned material it did not work due to the presence of liquid under the form of bubbles, which saturated the sample just partially with a relevant part of air trapped between the grains of soil. In this case the grains are not directly in contact, so when a depression is applied through the drainage pipe, the air trapped between the grains starts to flow outside the sample, bringing the foam with it (Figure 6.19) and changing the volume and the state of the sample.

This problem brought to a necessary adjustment of the test procedure for conditioned material by using a different approach, with the extension unloading approach. This stress path allow to avoid to use the depression, as the mold can be left in place because the sample is reducing its width during the test and the top cap is moving upwards.

These two tests have two different molds: the one for the compression loading tests is made of two half pipe thick steel elements, linked each other with bolts; the one for extension unloading tests is a polyvinyl chloride pipe which is less stiff. This mold is

Figure 6.18: Dry sand sample with the external depression applied

rigidly linked at the base with a lashing strap which also dovetails the membrane with the bottom cap.

Another type of test used in the conditioned soil a non-conventional test. This test is performed by leaving the loading ram rigid (no axial displacement Δ_a) and increasing in steps the radial pressure. This type of test would allow to study the behaviour of the conditioned material at different pressures, especially it is important to verify the difference between the applied radial pressure (confining pressure p_c) and the pressure induced by the fluids trapped in the soil in the sample (interstitial pressure p_i).

6.3.4 Testing procedure

The main use of the apparatus in its original configuration is the testing of granular soils with compressive stress paths, performed load or deformation controlled, both drained or undrained.

The testing procedure has been proposed both for the dry and the conditioned ma-

Figure 6.19: Liquid generator flowing in the tank of the depression circuit

terial, but after the first test in the conditioned soil the procedure has been changed, as the compression loading stress path was not possible due to the difficulties on creating the necessary void needed for removing the mold. For this reason the conditioned material has been tested by using an extension unloading stress path, which would not require the removing of the mold, as the sample is reducing its width during the test.

The tests performed during this research have been of three types, depending on the material and the needs:

1. compression loading test (σ_a and Δ_a increasing, σ_r constant);
2. extension unloading test (σ_a and Δ_a decreasing, σ_r constant);
3. lateral confinement increase test (Δ_a constant, σ_r increasing in steps);

In this research the modified triaxial test consists mainly on these operations:

1. *preparation of the sample.* The material is placed inside the rubber membrane which is rigidly linked to the mold. The dry material is inserted in layer, and as it is a non-cohesive dry material, it was placed in its natural state, without pressing it. Also in the conditioned material case, the material appears so fluid that it flows almost like the water in the mold. The most critical part for the conditioned soil testing is the time, as the foam is naturally degrading in the time. The test was attempted to be carried out within 90–120 minutes, in any case this parameter has been always registered;

2. *assembly of the apparatus.* This phase regards all those operations concerning the assembling of all the mechanical components, the disposition and connection of all the sensors and to the configuration of the system actuator;

3. *consolidation.* This phase allows to apply any initial stress condition to the sample, in this research the initial condition applied to the sample has been always isotropic, in order to reproduce as much as it is possible the hydrostatic conditions. This phase is usually performed, as in this research, load controlled.

4. *test execution.* This phase completes the performance of the test. The actuator is moving the loading ram in order to apply the load. This phase is usually performed as in this research, deformation controlled. The test is ending at the limit stroke of the piston, usually after reaching the peak strength and during the post-peak phase.

Preparation of the sample

The phases of preparation depend on the material to be studied and the testing type. The common operations are as follows:

1. the bottom cap is cleaned and its lateral surface is covered with a layer of silicone greased;

2. the porous stone and the filter paper are placed over the bottom cap;

3. the rubber membrane is inserted in the bottom cap, in contact with the grease;

4. the mold is placed on the bottom cap and the membrane is turned over it in the upper part;

5. the membrane is filled with the material to be tested. The dry material is inserted in layer, and as it is a non-cohesive dry material, it was placed in its natural state, without pressing it. Also in the conditioned material case, the material appears so fluid that it flows almost like the water in the mold;

6. once the material fills the membrane and the mold for the necessary height (600 mm), the material is leveled off in order to obtain a perfectly straight and uniform surface (Figure 6.20). Over this surface a filter paper is placed in contact with the porous stone which is embedded in the top cap;

7. connection of the membrane with the top cap, greased in the same way of the bottom cap, and application of o-rings to fasten the membrane over the cap.

Consolidation

This stage represents the first actual part of testing of the sample and it is performed by removing the depression and applying the radial pressure (compression loading tests) or just by applying the radial pressure (extension unloading tests). The most important issue regarding this phase is the perfect combination between radial pressure

Figure 6.20: Leveled surface of the sample before the connection of the top cap with the membrane

applied by the compressed air and the the axial load applied with the loading ram: these values in fact must be kept equal in order to fulfill the initial isotropic condition of the sample. Once the desired confinement pressure is reached and the axial load is balanced to obtain the isotropic condition, this state is usually kept for several minutes because, especially in clays, viscous deformation can occurs; nevertheless in the case of conditioned material the presence of bubbles which are naturally degrading this operation should be neglected in order to keep the material as much as possible in the initial state.

6.4 Pressurized rotating mixer

This is a totally new device, designed in order to verify different parameters and assess the behaviour of the conditioned mass in comparison with the natural material. As this equipment is not based on any existing geotechnical testing technique, the results are giving indications on the differences between the natural and conditioned material, without returning an absolute value, but mostly relative results.

Based on this principle, the aim of the device is to assess the resistance of the sample toward mixing blades, in order to simulate a similar mechanism which can happen in the excavating chamber. In this case, the best feature regarding this testing machine is the possibility of applying hydrostatic pressure to the sample in the tank, by pressurizing it by means of compressed air. This, as in the case of the modified triaxial testing, allows to study the behaviour of the masses, both natural and conditioned or in other conditions, at different pressures.

6.4.1 Needs of the new apparatus

The need of this apparatus started from the necessity of studying the workability of the conditioned material under different pressure conditions, in order to simulate the actual behaviour which can be encountered during the excavation with EPBS machines.

The device (Figure 6.21) has been designed by the TUSC laboratory group especially for this research. It is able to reproduce the pressure conditions in which the material can be conditioned in an EPBS excavating chamber. The apparatus can mix the material under defined pressure conditions and meanwhile can record the torque values.

Another possible issue which can be studied by using this mixer is the evolution of the wear of the metallic parts, important aspect during last years not only in hard rock TBMs but also in EPBS machines. This aspect is part of a research which is going on in TUSC laboratory of Politecnico di Torino (Barbero et al., 2012, Hedayatzadeh et al., 2013, Onãte Salazar et al., 2016) and it will not be taken into account in this thesis work.

The good and strong point on using this apparatus, compared to the triaxial testing cell, is the simplicity of performing the test. First of all the set up is much faster, and the test can be performed in a shorter period, which means a better representativeness of the conditioned soil studied. Then the possibility of adding and mixing the foam and water with the soil directly in the tank, and eventually in pressure, represents a very good point, as it simulates much better the behaviour of the soil conditioned in a excavating chamber.

Another important need of this apparatus is the possibility of studying soils with very heterogenous grain size distribution, and also of studying really coarse materials which usually are difficult to study (especially gravel and small cobbles).

Figure 6.21: Pressurized rotating mixer

6.4.2 Design of the new apparatus

According to the needs requested by this new test, the new apparatus has been designed in order to fulfill the requirements.

First of all the tank has been conceived in stainless steel, in this way the corrosion due to the use of water or chemical aggressive agents for standard metals (such as surfactants) can be avoided. Moreover the tank is designed to resist to an internal peak pressure of 10 bar, thus more than the necessary pressure which is used usually in EPB shields (around 4 bar). The size of the tank has been also taken into account, as a too small diameter would induce side effects and the conditioned mass could appear stiffer than in reality. The tank has an internal cylindrical shape and it is reinforced with external steel elements to increase the strength and counterbalance the internal applied pressure up to 10 bar. The tank is opened in the top part to allow the filling of the material, a removable closing plate is fastened by using 20 M20 steel bolts. The top removable closing plate (Figure 6.22), which is thicker than the tank body, has several apertures: one porthole, with an internal diameter of 154 mm, used for inspection purposes; one fast connection for inserting compressed air; one needle valve to eventually counterbalance the internal pressure, when needed, one pressure transducer.

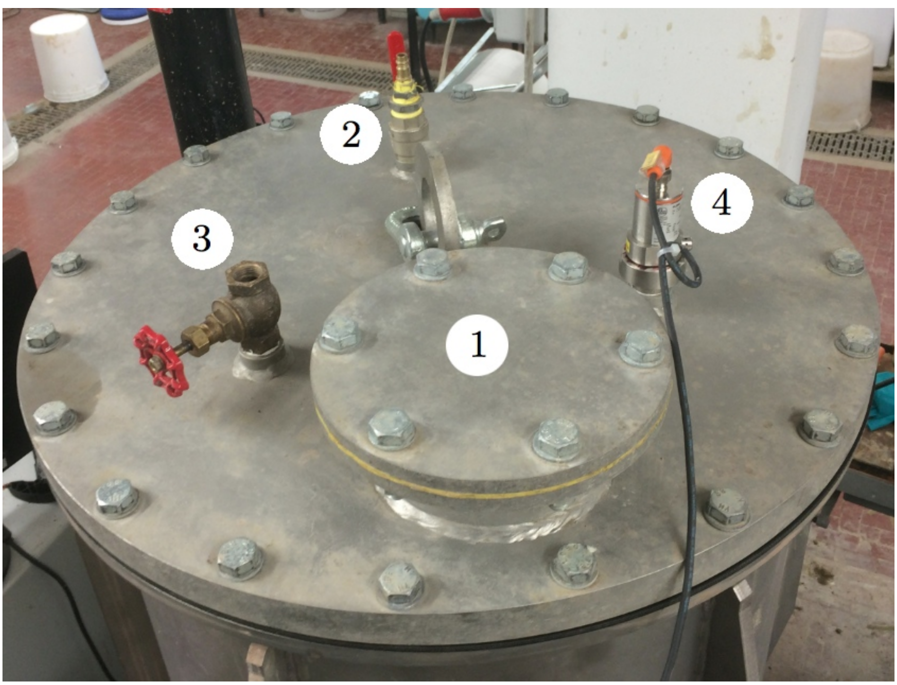


Figure 6.22: Top removable closing plate with the inspection porthole (1), compressed air connection (2), needle valve (3) and pressure transducer (4).

The essential characteristics of the tank are as follows:

- internal height: 613 mm
- internal diameter: 600 mm
- internal capacity: 0.13 m^3
- thickness of the tank: 15 mm
- thickness of the top closing plate: 25 mm
- peak internal pressure: 10 bar

The total geometry of the device is shown in Figure 6.23 and Figure 6.24)

For the mixing part, a system of warped blades (Figure 6.25) has been chosen, for using the same principle of the plane propeller. In this way the clockwise rotation produces a downwards flow of the material, on the contrary the counterclockwise rotation causes an upwards flow. This is really useful, as it is possible to mix more properly the soil with the foam.

The mixing is provided by an electric engine allowing a maximum rotating speed, which can be obtained both in clockwise and counterclockwise direction, is 32 rpm with a peak reachable torque during the rotation of 500 Nm.

The software which has been created for this device is able not only to register the various parameters, but also to control the movement of the device itself, as it is possible to change the speed and the direction of the rotation. Moreover the software has the possibility to sample the values of torque belonging to the apparatus itself when no material is inserted, and remove them from the sampling during the test execution. In this way only the net torque, thus the one induced only by the resistance and friction of the material object of test. The software has several screens, the most important are the main panel (Figure 6.26), where the engines are controlled and the operating parameters are plotted in real time, and the sampling panel (Figure 6.27), where the operator can perform the sampling of the torque with the empty tank and set a program which can be used in each test (speed and direction of the blades).

The tank has been designed with different female hydraulics threaded connections, which are used both to allocate pressure transducers on the top and the bottom of the tank and to eventually include different connections with hydraulics lines such as air, water and foam. This characteristics is crucial and unique for this kind of apparatus, as it would is possible to condition the raw material directly inside the tank at the required pressure condition.

The apparatus is designed to measure different parameter during the mixing phase:

- **torque of the rotating blades**. This parameter is measured through a load cell which is linked through a beam from the engine to the rigid frame of the apparatus. The torque is then calculated by the software developed by multiplying the load on the cell times the lever arm;
- **top and bottom internal tank pressure**. These physical quantities are measured through two differential pressure transducers which can monitor the pressure on the top, which usually indicates just the applied pressure by the air

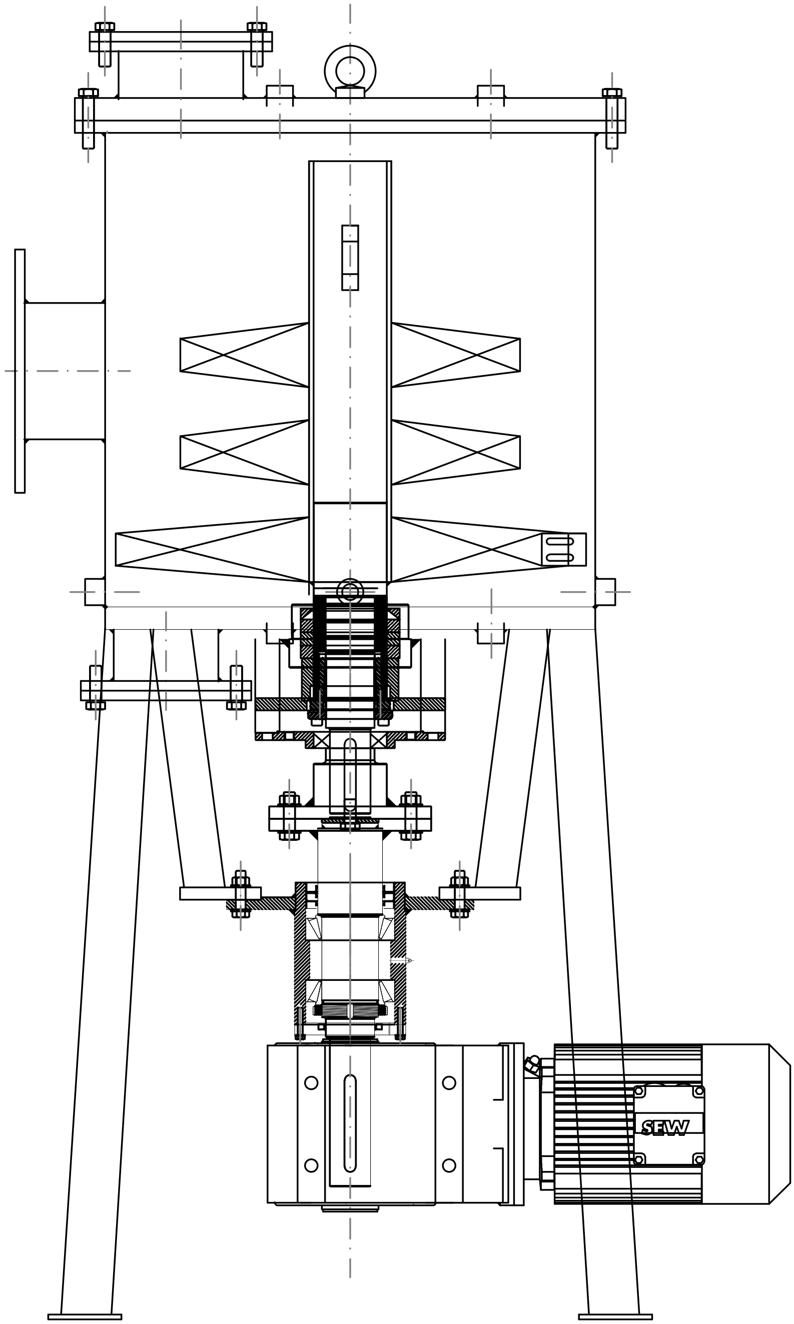


Figure 6.23: Pressurized rotating mixer (lateral view)

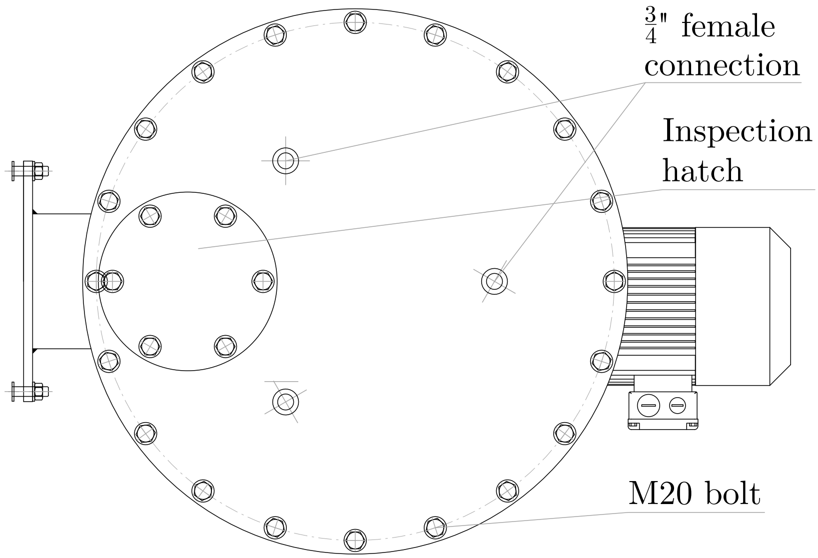


Figure 6.24: Pressurized rotating mixer (top view)

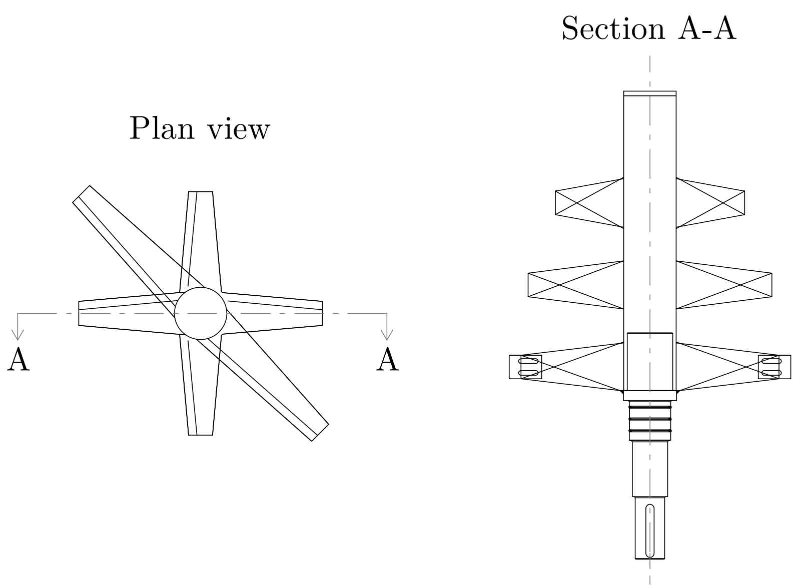


Figure 6.25: Technical drawing of the warped blades

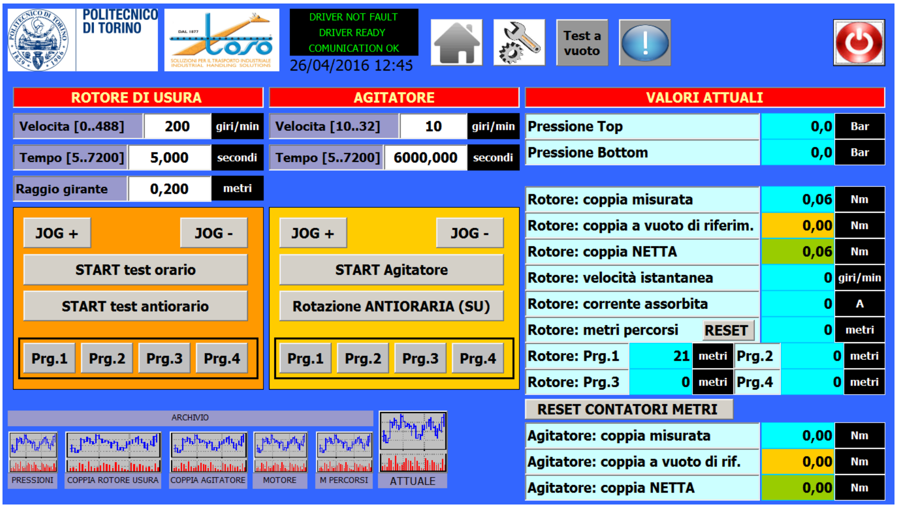


Figure 6.26: Main control panel of the software.

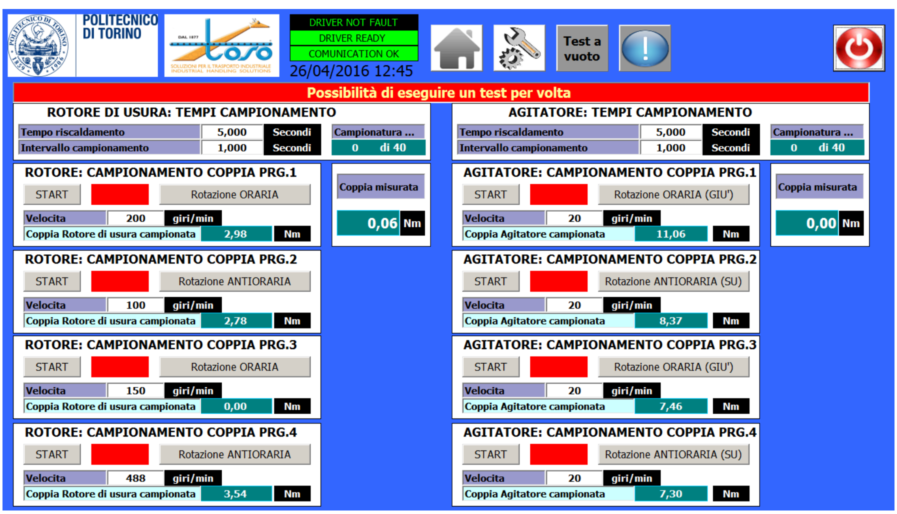


Figure 6.27: Sampling panel of the software.

bubble in the top, and on the bottom, which is indicating the pressure given by the air in case of permeable tested soils or directly by the pushed material, in case of really fluid and homogeneous conditioned masses. In case of impermeable but heterogenous soils the bottom sensor could return null values, so also this result could represents a good indication of the material.

6.4.3 Testing procedure

The testing procedure has been perfected after a calibration testing campaign, which highlighted some important aspects to be taken into account.

First of all the amount of material to be tested: as this test is very competitive due to its simplicity and to the possibility of comparing each others soils in all the different states (dry, wet, conditioned etc.), it is important to establish a common rule for the quantity to be used. The chosen parameter has been chosen as the 60% of the tank volume, represented by a height of 360 mm of dry material. In this way, once the unit weight has been established during the first stages of the characterization campaign, a constant mass of dry and raw material can be obtained and used. This allows to consider the water or the foam just as conditioning agents and to study directly only the soil in its different states. The volume has been chosen equal to the 60% in order to consider the fact that the soils, after the conditioning process, have a lower unit weight, thus the occupied volume of the same mass of raw material is larger.

The preliminary tests of the apparatus, carried out already at the workshop where it has been assembled and then at the TUSC laboratory, have underlined some aspects which have to be taken into account. First of all the speed of rotation, it should not be to fast, because the material in a EPB shield excavation chamber is moved slowly, but it cannot be too slow otherwise the mixing phase might result incomplete. The optimal speed of rotation has been chosen equal to 20 rpm.

Another crucial aspect is the rotation direction: as already said depending of the revolution direction, the material moves upwards or downwards, as the blades have been designed warped. Several preliminary tests have been carried out on this issue and the result is that when the rotation is clockwise, so the material is flowing downwards, the registered torque is much higher than the one registered in the other direction (Figure 6.28). This might be explained by the fact that when the material is flowing downwards the blades are compacting and crushing the material. Moreover once the material is compacted is starting to follow the rotation. In this last case the material is not free to move depending on the blade rotation, but it has a relevant component of friction at the bottom. This is visible on the test conducted on Soil A, but it is even more relevant in the test carried out with Soil E, where the large gravel grains have been crushed by the blades after pressing the single grain on the stiff bottom plate. Thus this means that the most realistic test to assess the internal resistance of the material through the mixing is the one carried out in clockwise direction.

The final configuration of the test is characterized by the following phases:

1. preparation of the raw material;
2. execution of the calibration program (void run) to calculate the average torque of the rotating blades induced by the mechanical internal frictions;

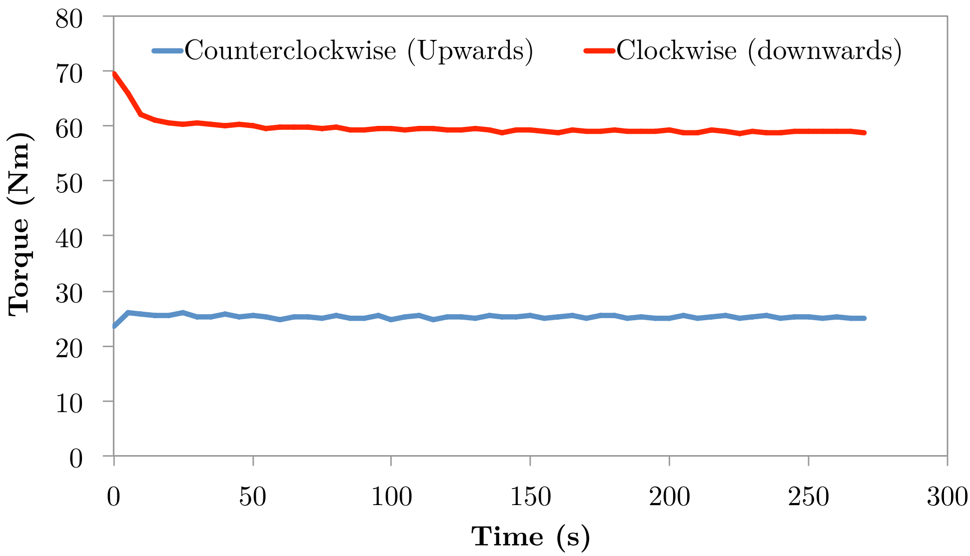


Figure 6.28: Comparison of the obtained resisting torques for the different rotation directions at 20 rpm with the dry Soil A

3. loading of the material into the tank up to the level representing the volume of 60% of the tank;
4. sealing of the top of the tank through the fastening of the closing plate and linking of the required hydraulics lines (air, water and foam);
5. execution of the desired program, depending on the required needs.

The tests conceived in order to investigate the different parameters are as follows:

- *pressure step test*. In this type of test the program is running with a fixed time (300 s) and it calculates the average torque in the period. Once the program ends, the blades stop the rotation and the pressure can be increased in step (usually 1 bar steps, but can be done also for smaller/larger values) up to the desired pressure value. This test is good for checking the quality of the material on the time, if its characteristics are stable and constant upon the time;
- *constant pressure increase/decrease test*. In this type of test the program is running continuously, until the operator is stopping the rotation of the blades. The program starts, and once the torque is stabilized the pressure is slowly constantly increased by using a pressure regulator up to the desired pressure (pressure increase cycle). Once the peak pressure is reached, the pressure is reduced by closing the entrance flow of the compressed air and by removing the compressed air through the needle valve (pressure decrease cycle). Compared to the previous configuration the mass is continuously in movement during the all test.

Chapter 7

Results of the new campaign

7.1 Foam compressibility test

Three tests were carried out, for the first two experiments a FER value has been set and the applied pressure increased in order to register the evolution of the expansion ratio; as for the third test a higher FER value have been chosen (dryer foam) and the same pressure was applied. A FER decrease equal to 5 was registered for an increase of 1 bar pressure at the cylinder as seen in Figure 7.1. This phenomenon may be explained as a reduction of the size of the bubbles due to the increase of the pressure of the air contained inside the lamella structure. Afterwards, the FER value has been set higher in such a way to depict the relation between set FER_0 and pressurized FER_p (Figure 7.2). A higher FER reduction was registered for a dryer foam and it is possible that in this case as the gas phase is higher, the foam behaves an increase of the compressibility. Finally FER_r value found was lower to FER_0 set, possibly due to a destruction of the bubbles during the compression–decompression, resulting in a more wet foam.

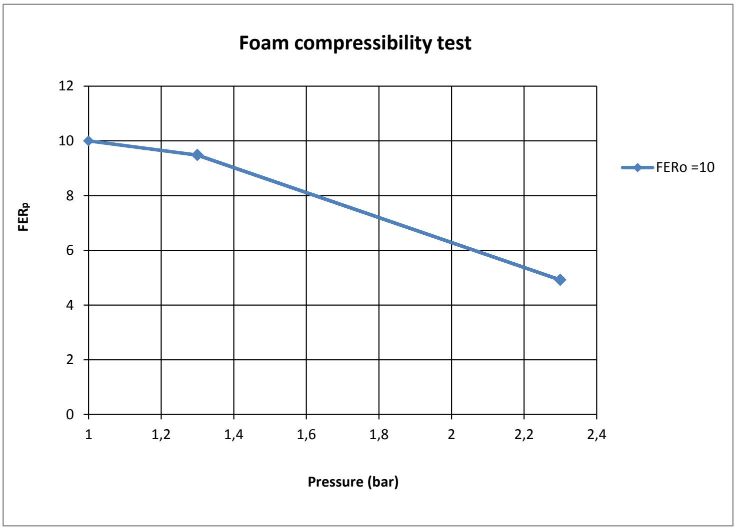


Figure 7.1: Foam compressibility test, evolution of FER with the increase of pressure.

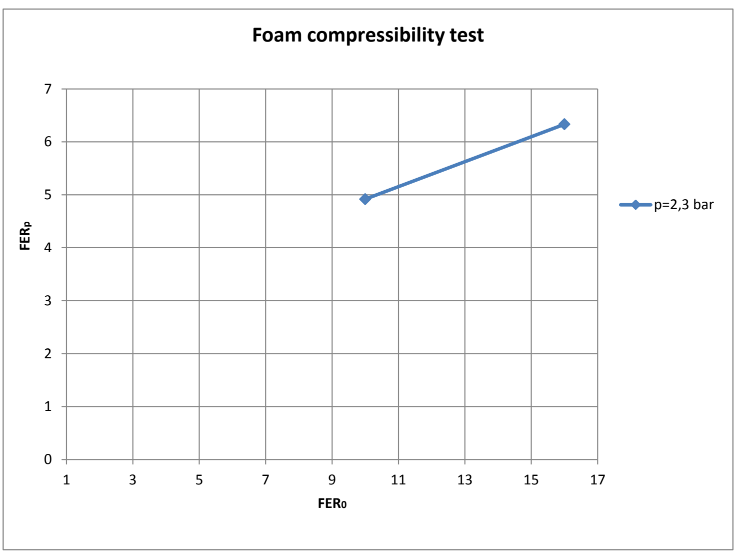


Figure 7.2: Foam compressibility test, ratio between FER_0 and FER_p for a certain pressure.

7.2 Undrained direct shear testing

7.2.1 Introduction

The tested soils have been described in Chapter 3 through granulometric analysis, conditioning dosage determination campaign and conventional direct shear tests. The collected data are elemental for the further analysis of the results obtained with the experimental set up employed.

The results are presented starting by undrained direct shear tests, followed by vane shear tests are presented separately by soil type. For this campaign four soil types have been considered (A, B, C and D), while Soil E has a too wide size distribution and the biggest grain is too large for fitting into the shear box.

Regarding direct shear tests, these have been executed on dry, saturated and conditioned soil samples. The exerted vertical pressure, ranging between 0–30 kPa, is applied by a hanger with dead loads in several steps as explained in Table 7.1. Shear force and horizontal displacement have been registered and the τ–horizontal displacement graphs are presented, as well as the relation between τ–σ from the registered peak shear stresses for dry and saturated conditions and from the τ_{med} for conditioned samples, in order to obtain easily the parameters of the Mohr-Coulomb criterion. In addition, some tests were carried out for the same sample as a function of the time from the moment in which the conditioning agent was added and the soil mixed (τ_{med}–time graphs).

Table 7.1: Normal loads and pressures used for the direct shear tests.

Normal load (N)	Resulting normal pressure (kPa)
9.1	3.21
19.1	6.75
29.1	10.29
39.1	13.82
47.3	16.72
57.3	20.26
67.3	23.80
77.3	27.33
87.3	30.87

Concerning Vane shear tests, soil specimens have been tested alike mentioned before for direct shear tests in the same three conditions (dry, saturated and conditioned). Once the soil was ready, the sample was prepared in a standard container and depending on the presumed shear strength, the most suitable vane blade was chosen. Tests on each soil were carried out several times with more than one blade size in order to avoid scale effects, getting a more certain measure and the data are presented in C_u–time. Furthermore, respectively to conditioned samples tests were also carried as a function of the time from the mixing moment and results are presented as a C_u–time representation.

7.2.2 Results

Sand A

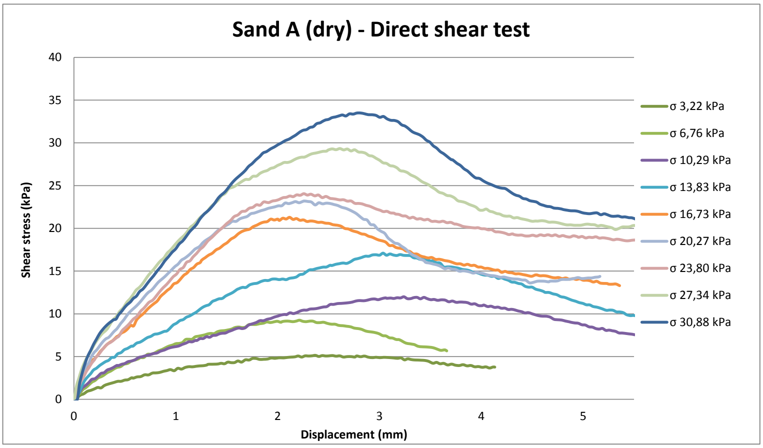


Figure 7.3: Sand A (dry) direct shear tests, shear stress–horizontal displacement.

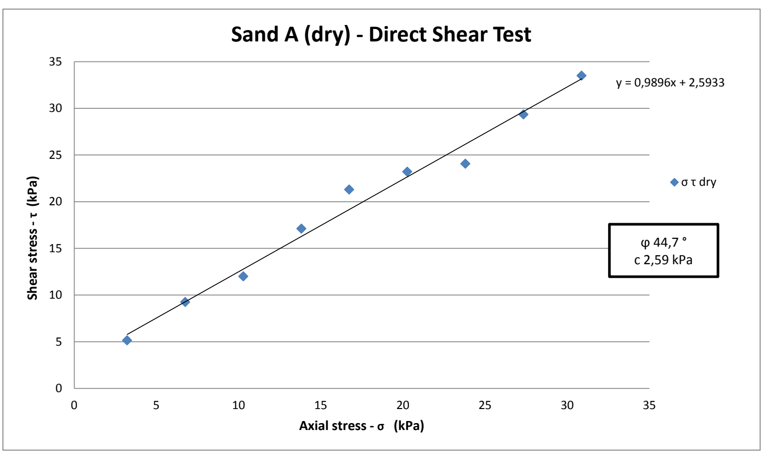


Figure 7.4: Sand A (dry) direct shear tests, shear stress–normal stress and M-C parameters.

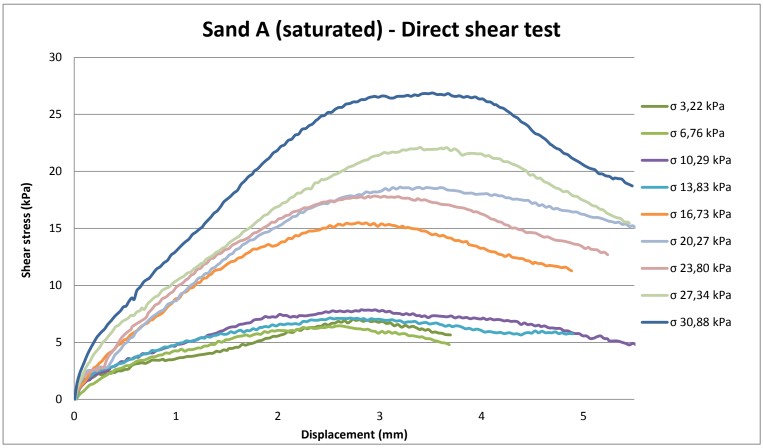


Figure 7.5: Sand A (saturated) direct shear tests, shear stress–horizontal displacement.

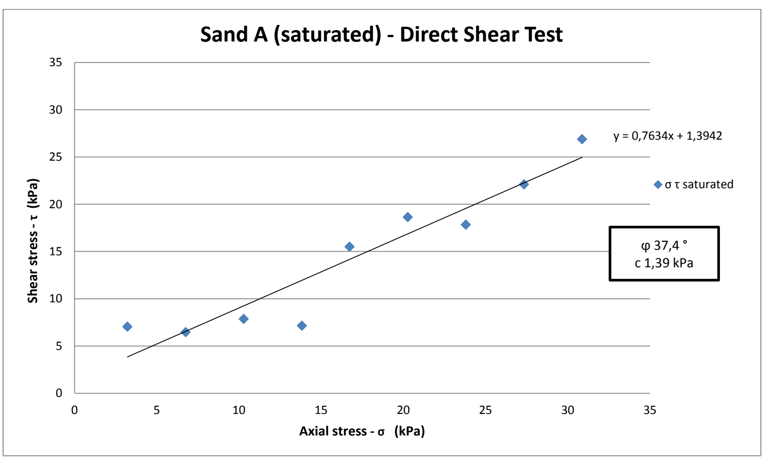


Figure 7.6: Sand A (saturated) direct shear tests, shear stress–normal stress and M-C parameters.

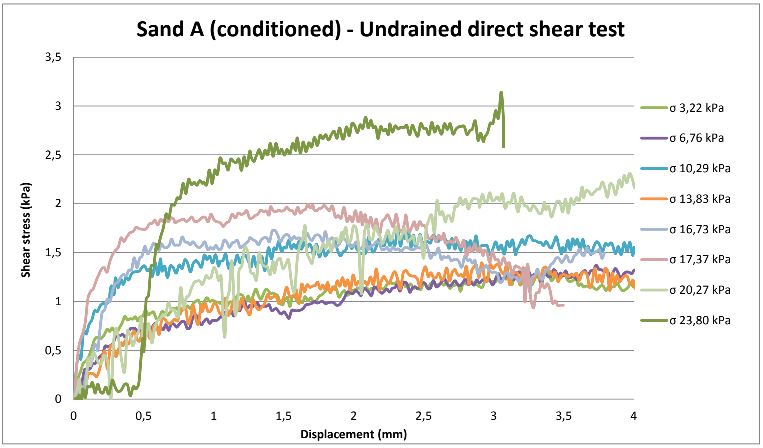


Figure 7.7: Sand A (conditioned) direct shear tests, shear stress–horizontal displacement.

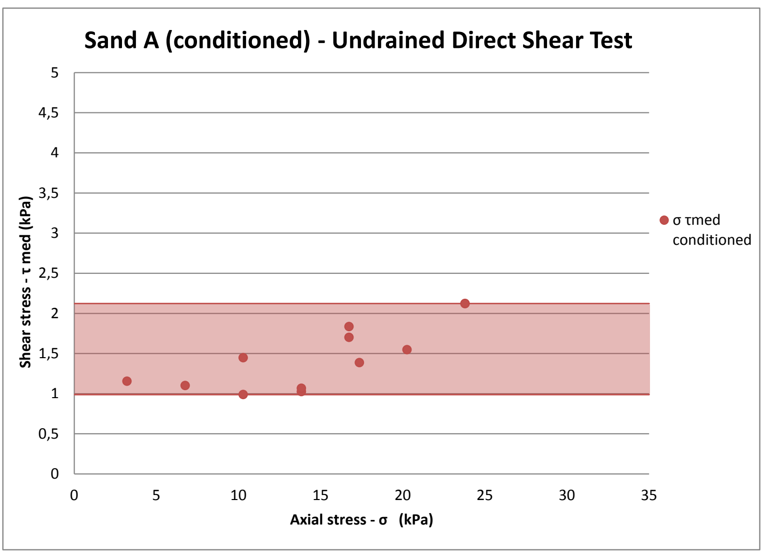


Figure 7.8: Sand A (conditioned) direct shear tests, shear stress–normal stress.

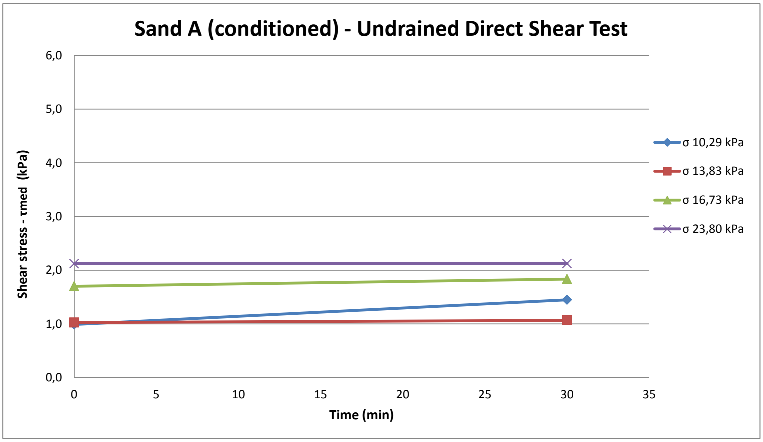


Figure 7.9: Sand A (conditioned) direct shear tests, shear stress–time.

As seen in Figures 7.3 and 7.5 the uniformity and fine gradation of this material exhibit a regular trend of the τ–horizontal displacement correspondence. The material appears to be, in dry and saturated conditions, almost ideally elasto-plastic at low confinements and it changes more clearly at higher pressures, with a softening behaviour. It is also important to remark the decrease of friction angle and apparent cohesion from dry to saturated condition (Figures 7.4 and 7.8), becoming almost negligible for conditioned samples (Figure 7.8). Finally as shown in Figure 7.9 this material behaves very stable in time regarding conditioned properties, this may be due to the small grain size of the soil that preserves foam bubbles through high surface tension phenomena. In this case the material is characterized with a slight hardening behaviour.

Sand B

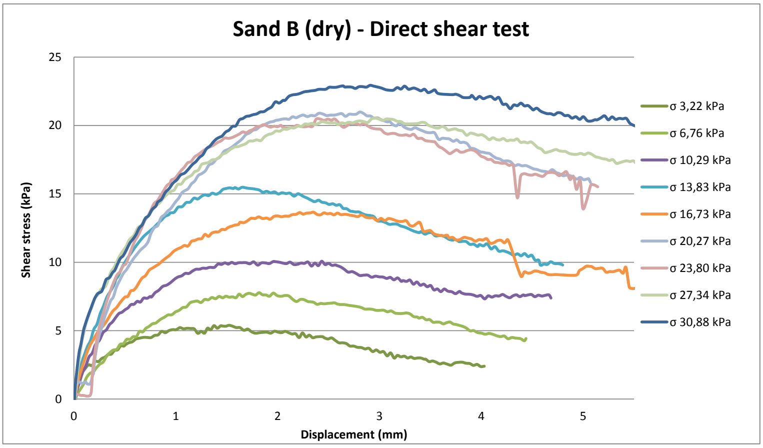


Figure 7.10: Sand B (dry) direct shear tests, shear stress–horizontal displacement.

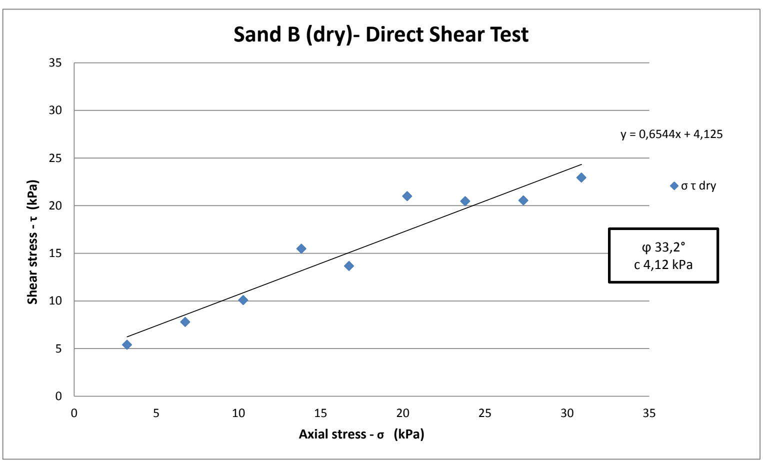


Figure 7.11: Sand B (dry) direct shear tests, shear stress–normal stress and M-C parameters.

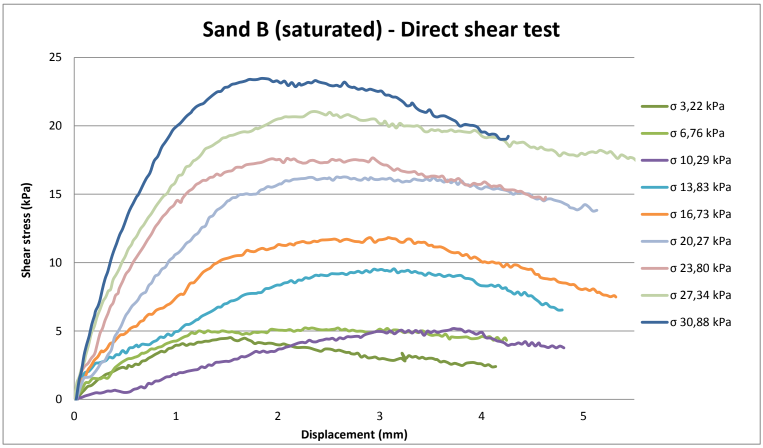


Figure 7.12: Sand B (saturated) direct shear tests, shear stress–horizontal displacement.

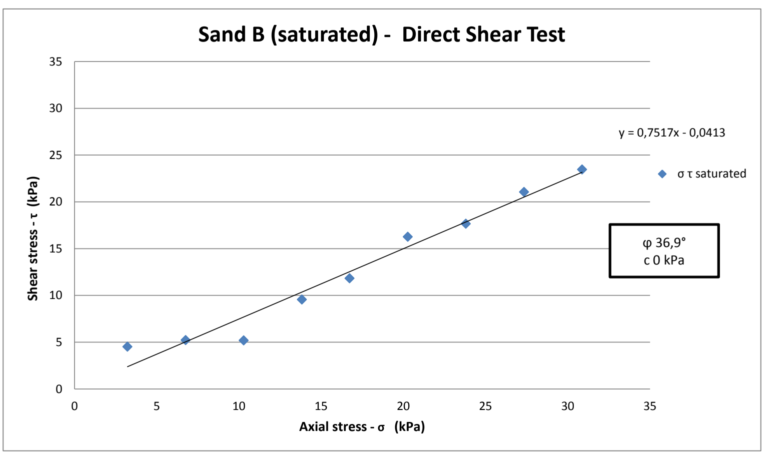


Figure 7.13: Sand B (saturated) direct shear tests, shear stress–normal stress and M-C parameters.

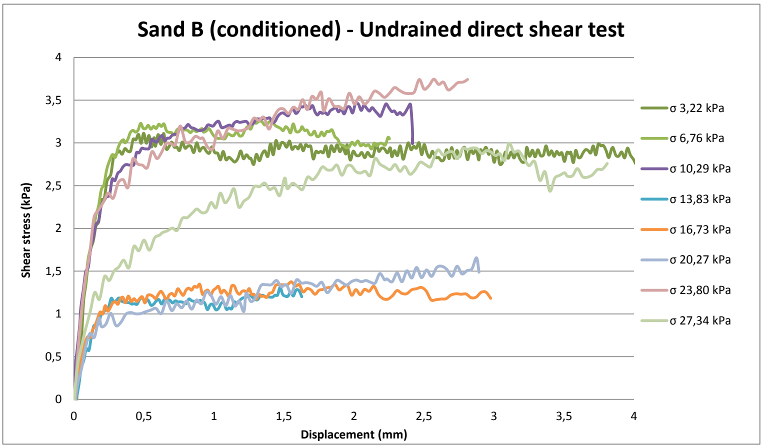


Figure 7.14: Sand B (conditioned) direct shear tests, shear stress–horizontal displacement.

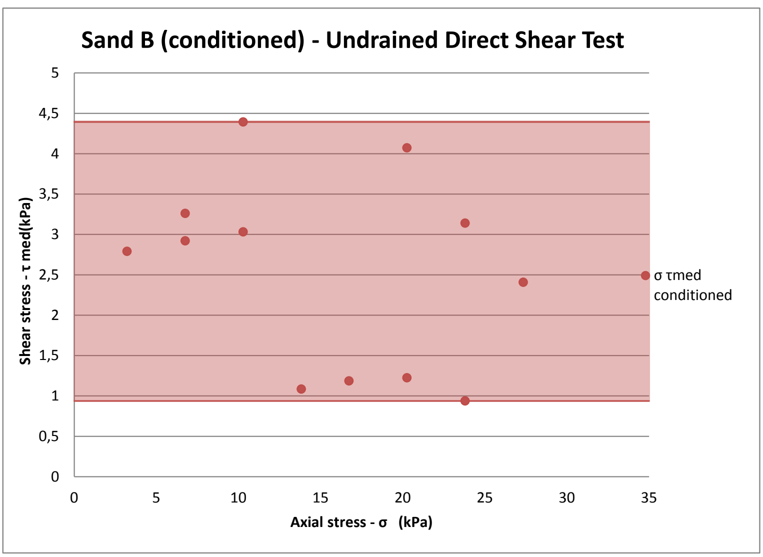


Figure 7.15: Sand B (conditioned) direct shear tests, shear stress–normal stress.

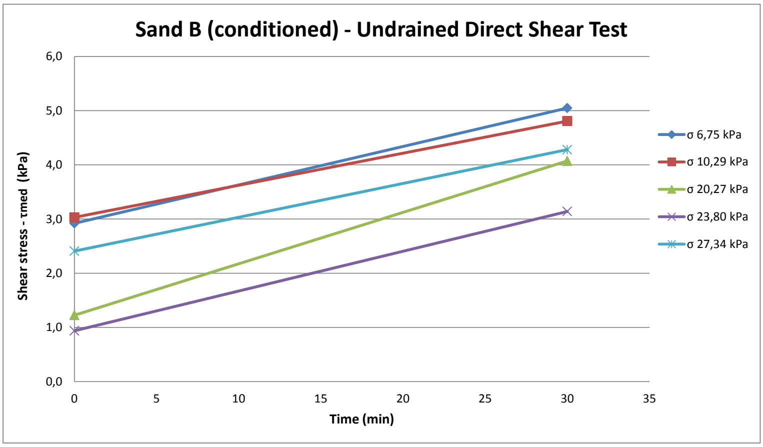


Figure 7.16: Sand B (conditioned) direct shear tests, shear stress–time.

In this case, from Figures 7.10 and 7.12 a smooth τ–horizontal displacement correspondence still valid for this soil. Also in this case the changing from almost ideally elasto-plastic to softnening is visible, even though in this case is less clear. Concerning Mohr-Coulomb parameters, the evolution from dry to saturated condition (Figures 7.11 and 7.15) showed a short increase of friction angle causing apparent cohesion tends to zero. As for conditioned sample shear resistance registered values still very low but with a higher scattering than Sand A (Figure 7.15). This phenomenon may be explained by the unstable behaviour in time of the conditioned properties, confirmed by Figure 7.16 and possibly due to a considerable lack of fine portion presented by this soil. Also in this case the material seems to have a slight hardening behaviour.

Sand C

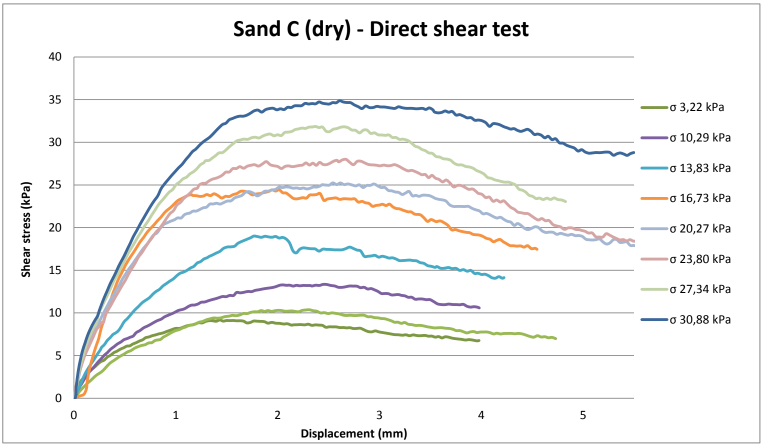


Figure 7.17: Sand C (dry) direct shear tests, shear stress–horizontal displacement.

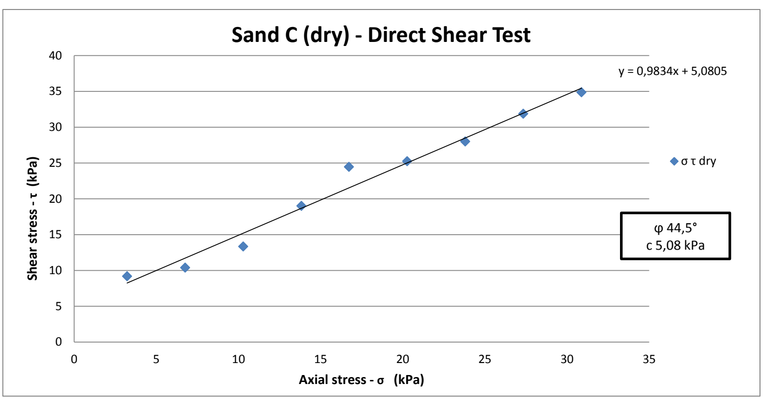


Figure 7.18: Sand C (dry) direct shear tests, shear stress–normal stress and M-C parameters.

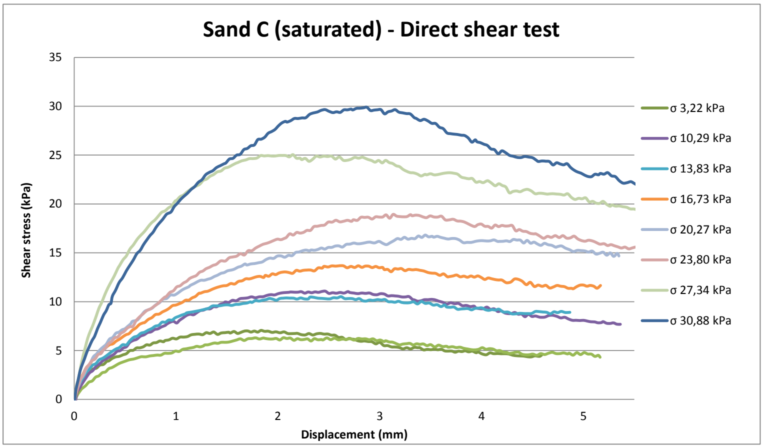


Figure 7.19: Sand C (saturated) direct shear tests, shear stress–horizontal displacement.

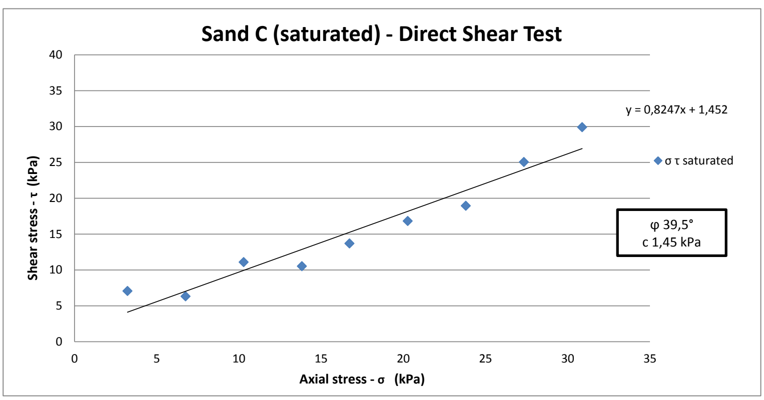


Figure 7.20: Sand C (saturated) direct shear tests, shear stress–normal stress and M-C parameters.

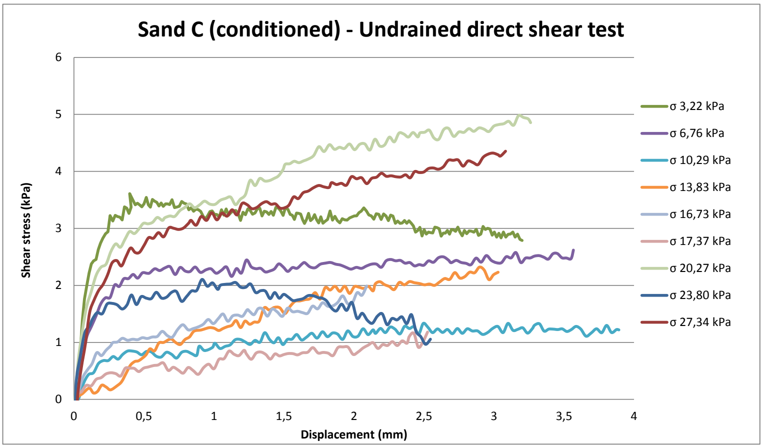


Figure 7.21: Sand C (conditioned) direct shear tests, shear stress–horizontal displacement.

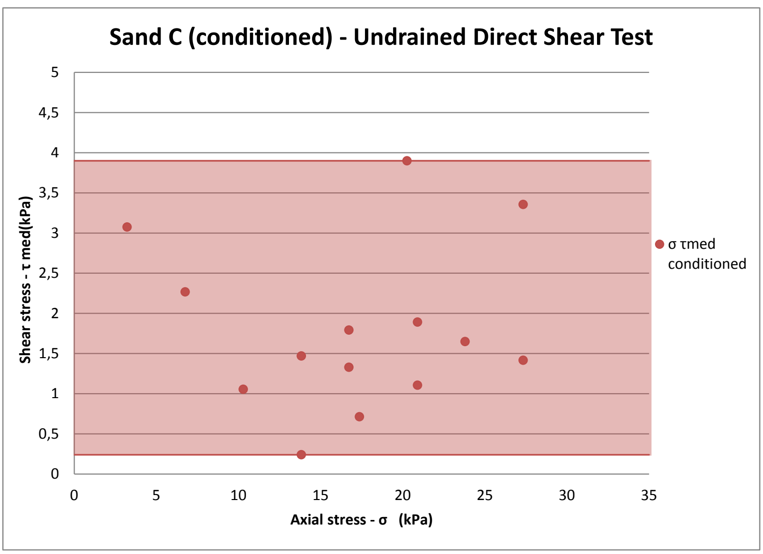


Figure 7.22: Sand C (conditioned) direct shear tests, shear stress–normal stress.

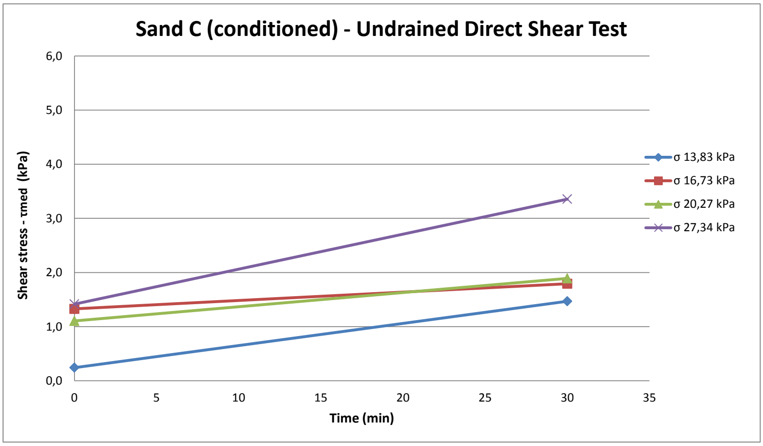


Figure 7.23: Sand C (conditioned) direct shear tests, shear stress–time.

In this case, from Figures 7.17 and 7.19, τ–horizontal displacement relation behaved some local peaks, possibly due to the greater size of soil grains. Also in this case the changing from almost ideally elasto-plastic to softening is visible, a bit clearer than in Sand B. Concerning Mohr-Coulomb parameters, the evolution from dry to saturated condition (Figures 7.18 and 7.20), resulted in a decrease of both friction angle and apparent cohesion. Regarding conditioned sample, alike sand B a high scattering of the registered shear resistance values have been found (Figure 7.22). This could be explained by the similarity between size gradation of soils B and C, behaving the before mentioned instability in time of the conditioned properties, depicted by Figure 7.23. Also in this case the material seems to have a slight hardening behaviour, especially at higher confinements.

Sand D

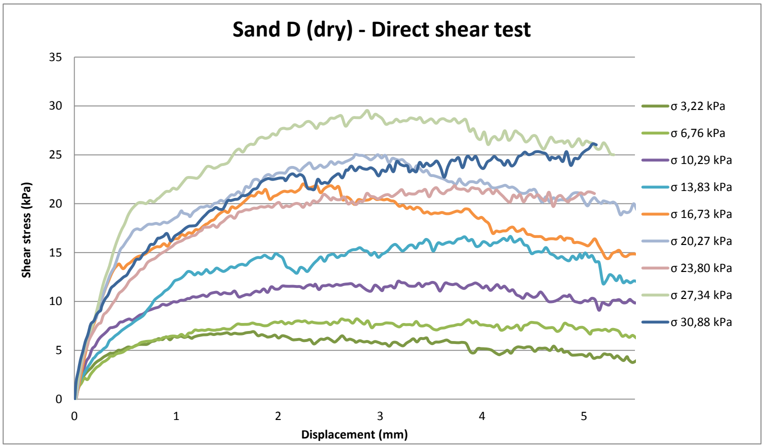


Figure 7.24: Sand D (dry) direct shear tests, shear stress–horizontal displacement.

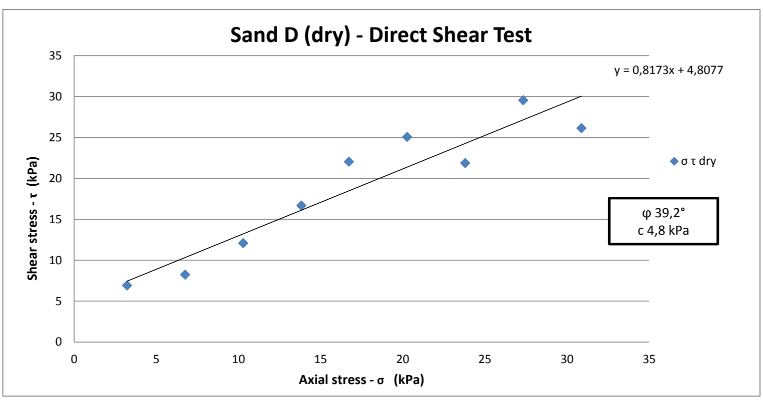


Figure 7.25: Sand D (dry) direct shear tests, shear stress–normal stress and M-C parameters.

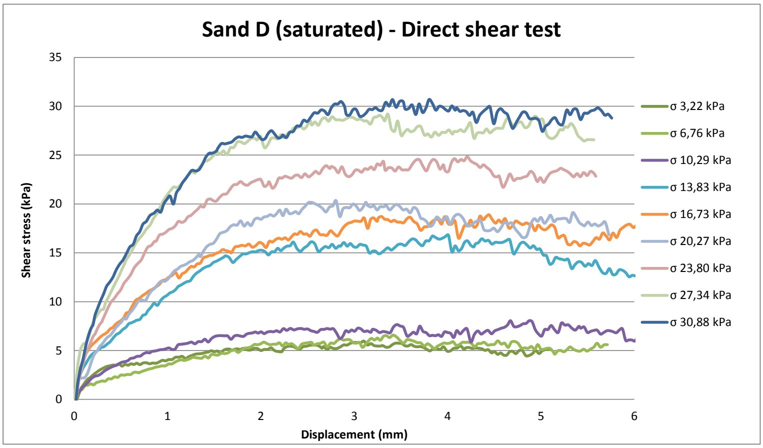


Figure 7.26: Sand D (saturated) direct shear tests, shear stress–horizontal displacement.

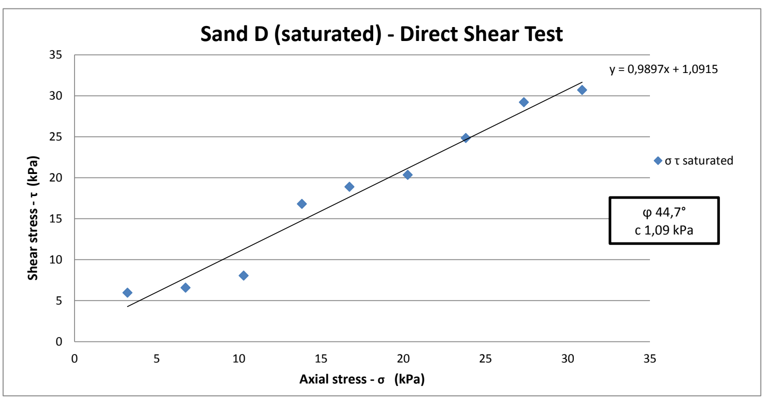


Figure 7.27: Sand D (saturated) direct shear tests, shear stress–normal stress and M-C parameters.

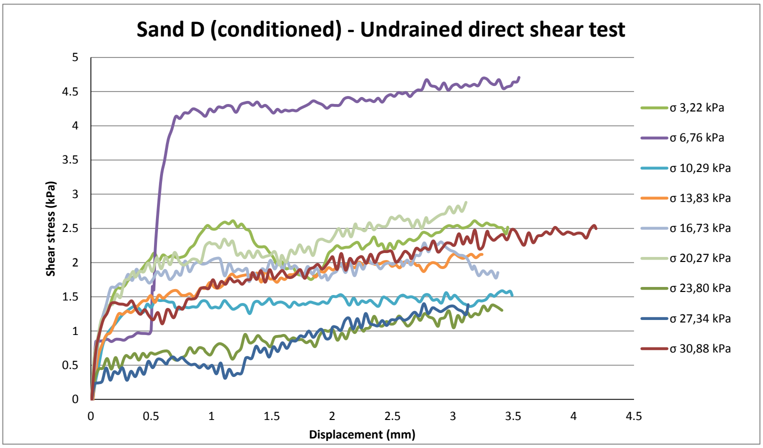


Figure 7.28: Sand D (conditioned) direct shear tests, shear stress–horizontal displacement.

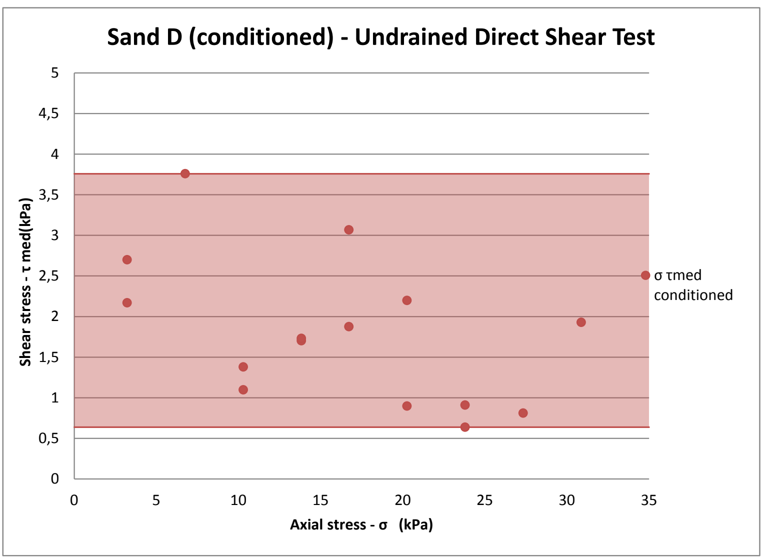


Figure 7.29: Sand D (conditioned) direct shear tests, shear stress–normal stress.

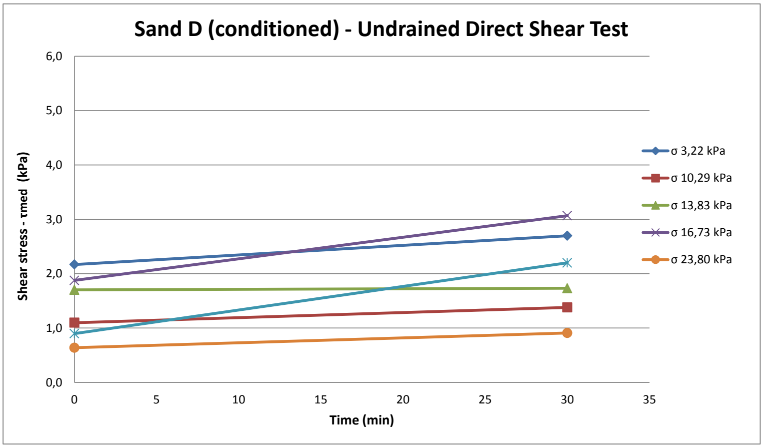


Figure 7.30: Sand D (conditioned) direct shear tests, shear stress–time.

Clearly from Figures 7.24 and 7.26, τ–horizontal displacement correspondence is very unsteady due to the greater size of soil grains and its angularity. In this case the material does not have any positive or negative hardening, it keeps an ideal elasto-plastic behaviour. Regarding Mohr-Coulomb parameters, from dry to saturated condition was analogous to the one showed by Sand B (Figures 7.25 and 7.27), where a short increase of friction angle caused apparent cohesion tend to zero. Regarding conditioned sample, reduction of the shear resistance is still very clear in Figure 7.29. In this case the addition of bentonite solution as a conditioning agent showed a higher stability of conditioning parameters in time (Figure 7.30). Also in this case the material seems to have a slight hardening behaviour, especially at higher confinements. The results in this conditioned material appear more sparse.

7.2.3 Vane shear tests

The results of these tests are presented by soil (A, B, C and D) for each testing condition (dry, saturated and conditioned) in the present section. The obtained shear resistance values are lower than those obtained by Undrained direct shear apparatus but this simplified test is still a good method to measure the reduction rate of resistance parameters between non-conditioned and conditioned soil mass. Figure 7.35 confirmed the behaviour in time of shear resistance for each conditioned material (Figures 7.9, 7.16, 7.23 and 7.30), where material A and D are the most stable due to the higher fine content (bentonite addition in the case of Soil D).

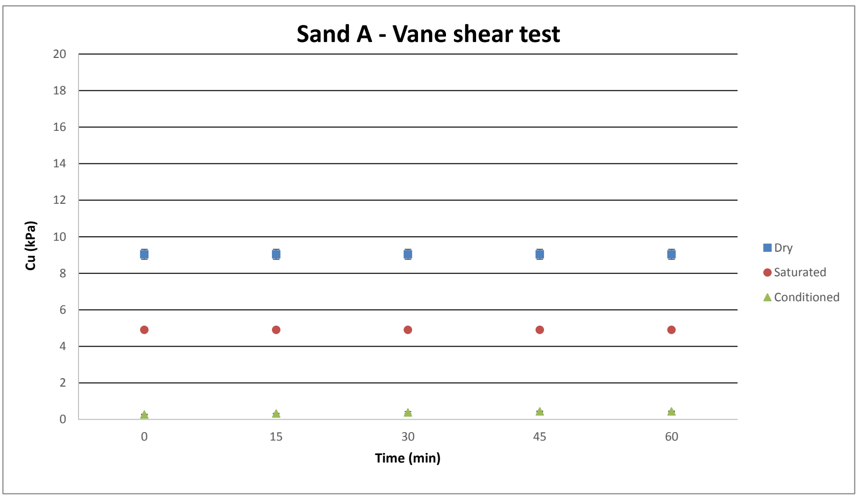


Figure 7.31: Sand A vane shear tests, comparison between dry, saturated and conditioned samples Cu–time path.

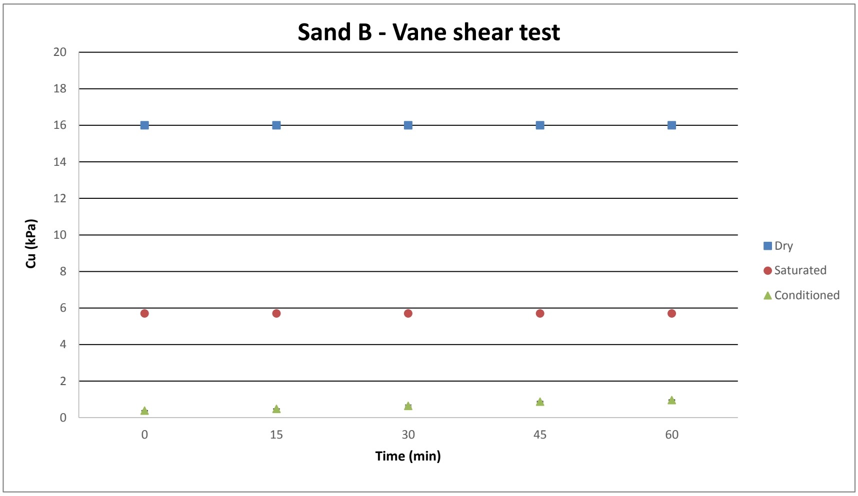


Figure 7.32: Sand B vane shear tests, comparison between dry, saturated and conditioned samples Cu–time path.

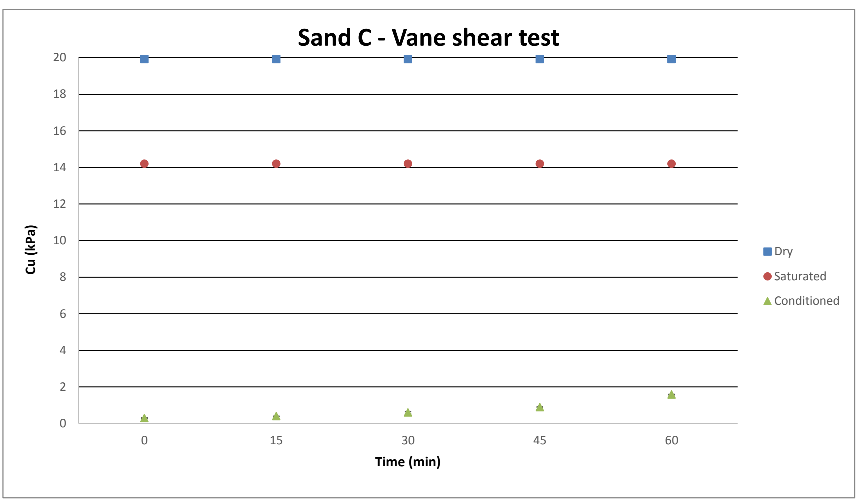


Figure 7.33: Sand C vane shear tests, comparison between dry, saturated and conditioned samples Cu–time path.

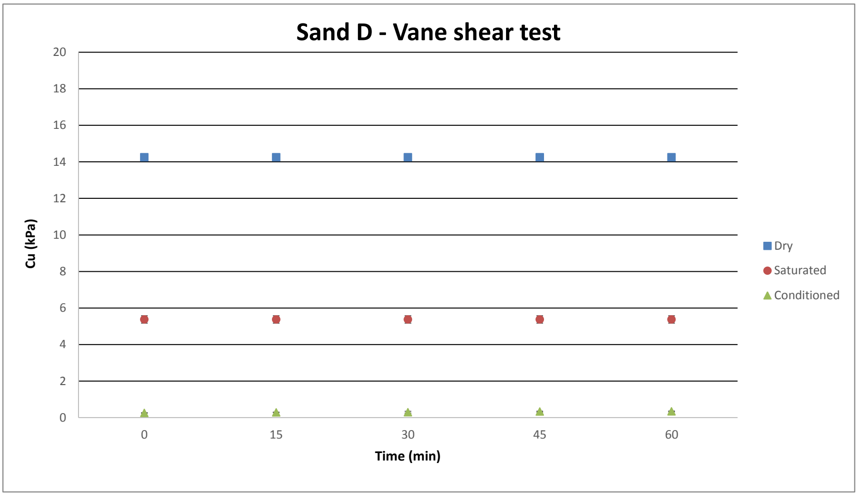


Figure 7.34: Sand D vane shear tests, comparison between dry, saturated and conditioned samples Cu–time path.

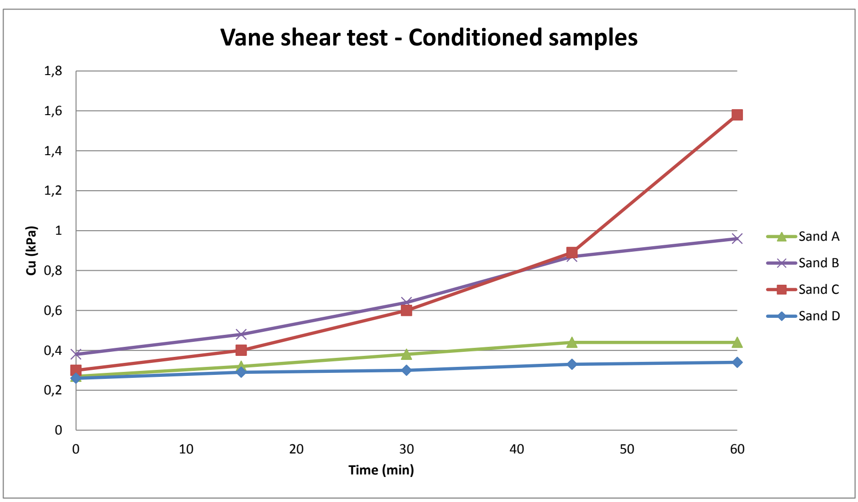


Figure 7.35: Vane shear tests on conditioned samples, comparison between behaviour of the four tested soils Cu–time path.

7.2.4 Analysis of the results

In this section a comparative analysis between the results obtained under different conditions of the sample is presented in such a way to depict the effect of soil conditioning, mainly reducing shear resistance of the material. As mentioned before in Chapter 1, reduction of the shear strength of the excavated soil brings about reduction of the required torque to rotate the cutting wheel. Therefore, this parameter was one of the fundamental mechanical properties of conditioned soil of importance in tunnelling, and without an undrained approach these results would have been unreachable.

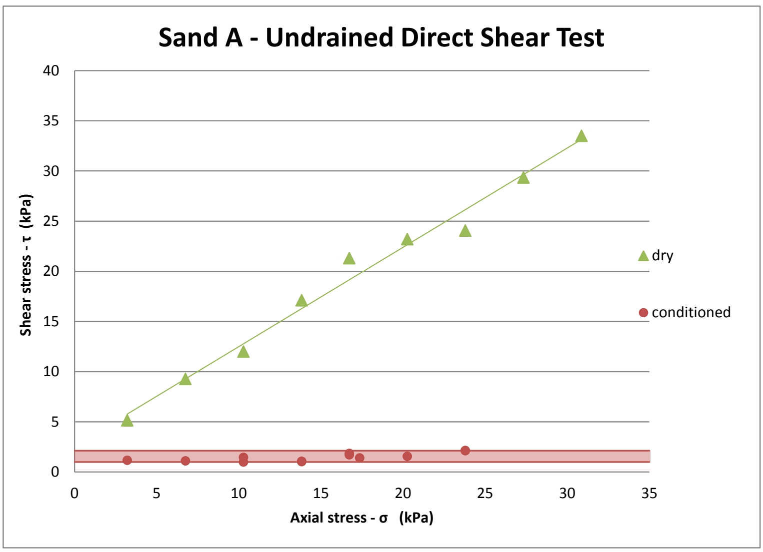


Figure 7.36: Sand A, comparison between dry and conditioned samples shear stress–normal stress path.

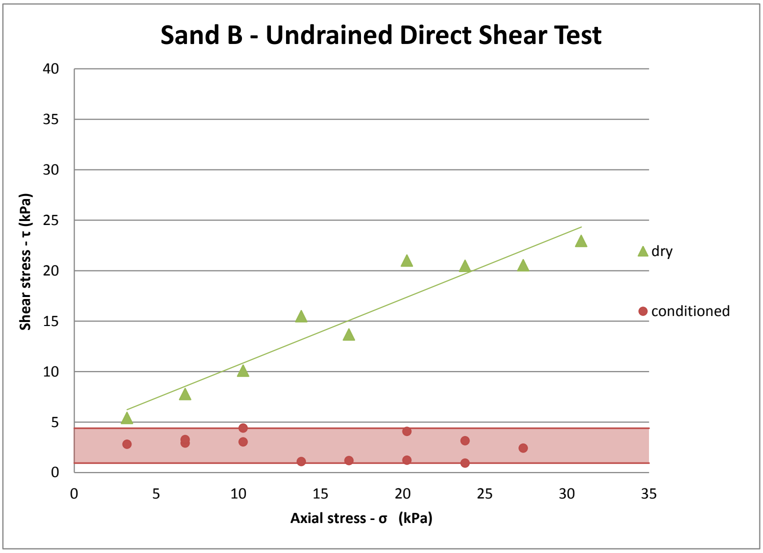


Figure 7.37: Sand B, comparison between dry and conditioned samples shear stress–normal stress path.

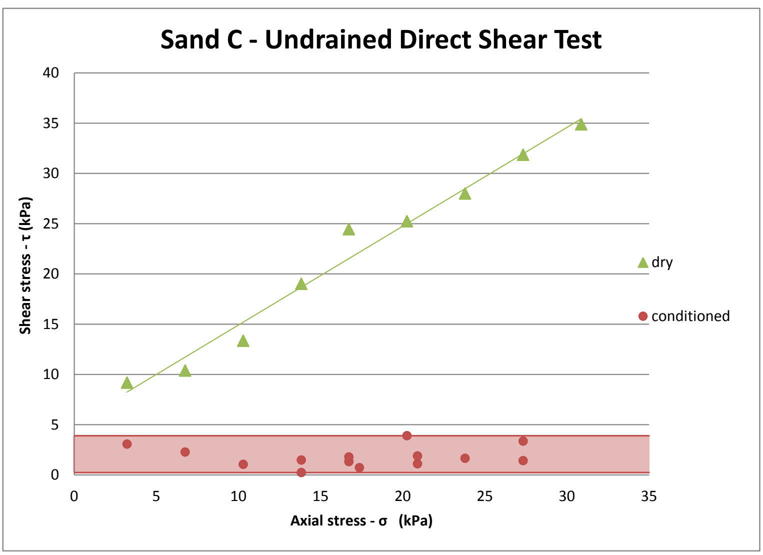


Figure 7.38: Sand C, comparison between dry and conditioned samples shear stress–normal stress path.

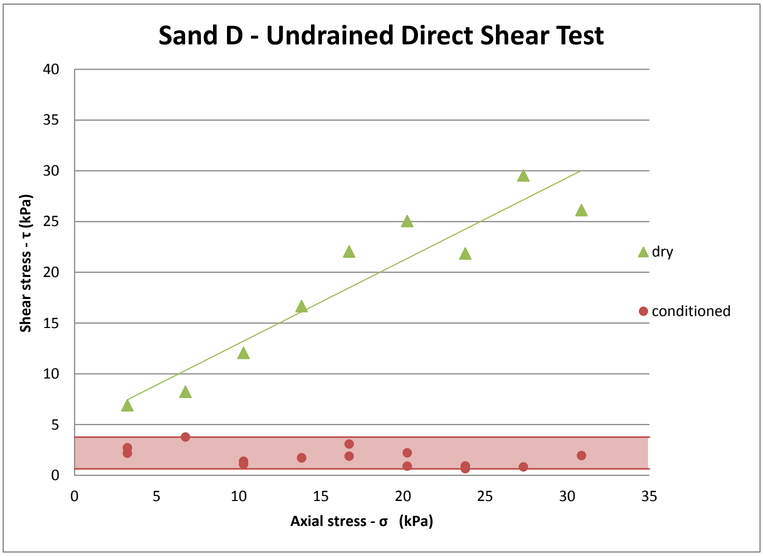


Figure 7.39: Sand D, comparison between dry and conditioned samples shear stress–normal stress path.

7.3 Modified large diameter triaxial test

7.3.1 Introduction

The apparatus has been applied for testing two of the soils used for this study, Soil A and Soil E. These two soils are the most heterogenous and representative of a actual possible soil which can be found in a real site.

The first campaign of tests has been carried out on Soil A, by using the compression loading stress path. The dry soil did not present any particular problem during the testing, while when testing the conditioned samples it was not possible to apply the depression, as stated in Section 6.3.3 and shown in Figure 6.19. For this reason the testing method has been changed by using a different stress path, that is the extension unloading. In this way the problem of applying effectively the depression can be overtaken.

The campaign carried out on Sand E has been performed just by using the extension unloading stress path. First of all the dry samples have been studied, in order to get the shear strength which was not possible to assess with the shear test. Then a campaign of tests has been carried out on the conditioned samples.

The conditioned samples have been prepared according to the usual way (Peila, 2014) and directly inserted in the mold, while the dry samples, as they are loose material without any cohesion, they have been directly inserted in the mold. Every 20 cm the layer of material has been regularized up to the top and the final stratum is well-groomed by mean of a aluminum bar.

The measured parameters, needed for this research, are the vertical load, the confinement pressure and the displacement of the piston. The output of the sensors is in Volt, which is then converted depending on the type and the calibration. In Figure 7.40, an example of the output obtained by the test before the post processing of the data is shown, where in the x axis is indicated the time (s) and in the y axis the output signal (V).

As shown in the example, the value of radial pressure σ_r is slightly fluctuating on the time due to the fact that the system for providing the compressed air is manually controlled. this is due to the size of the cell, it would be impossible to balance perfectly the pressure with the losses distributed in the cell (especially at the bottom plate and inside the loading ram. By the way these fluctuations represents a very small diversion of pressure from the needed value, usually few kPa compared to the hundreds of kPa needed for the confinement (so less than the 5%). This difference, for the purpose of assessing and especially comparing the shear strengths of the material in different conditions, is not influencing the final results obtained.

The use of two stress paths on the testing, brought of course to two different failure of the samples, by compression or by extension. A picture of the broken samples in the two cases is presented in Figure 7.41.

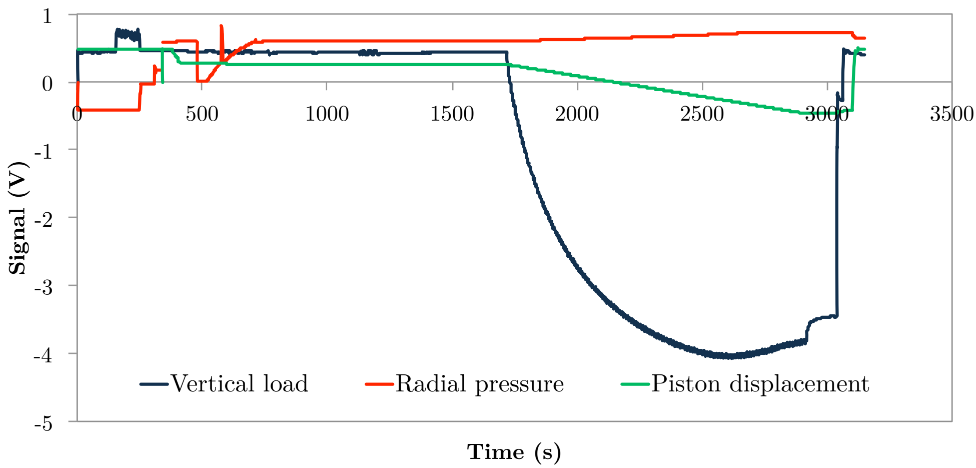


Figure 7.40: Example of the output obtained by the test before the post processing of the data.

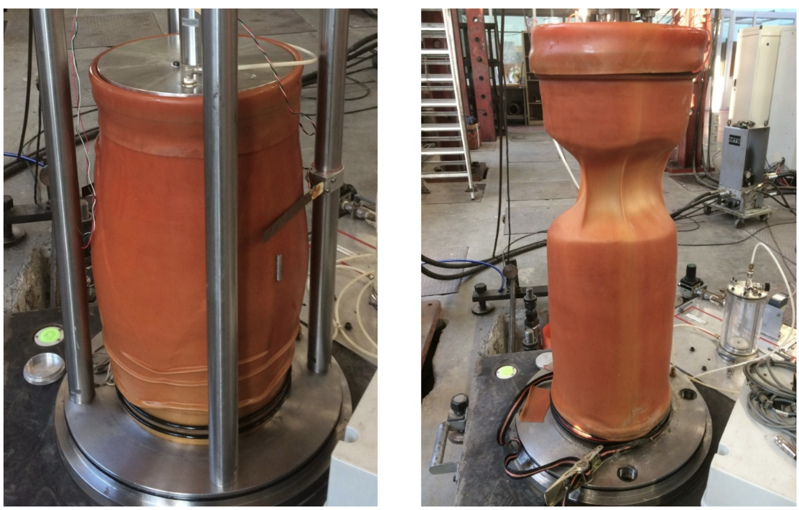


Figure 7.41: Samples after the failure at the end of a compression loading (a) and extension unloading (b) tests.

7.3.2 Sand A

Dry

The campaign has been carried out by using 3 different confinement pressures σ_r, as shown in Table 7.2.

Table 7.2: Confinement pressure σ_r used for the triaxial compression loading testing campaign on dry samples of Sand A.

Test	σ_r (kPa)
1	150
2	200
3	300

The graphs showing the results of the 3 tests are shown in Figures 7.42, 7.43 and 7.44.

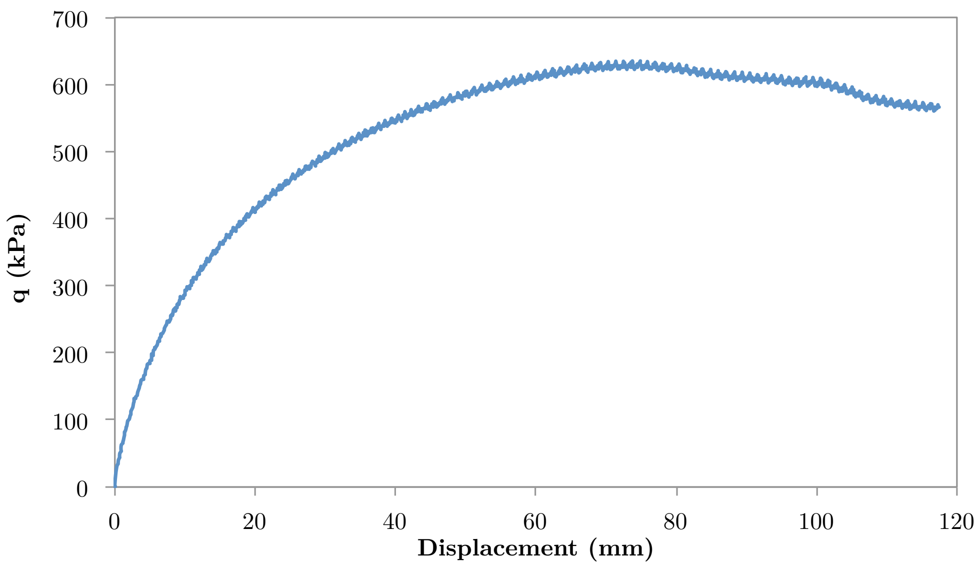


Figure 7.42: Triaxial compression loading test of dry Soil A with $\sigma_r = 150$ kPa.

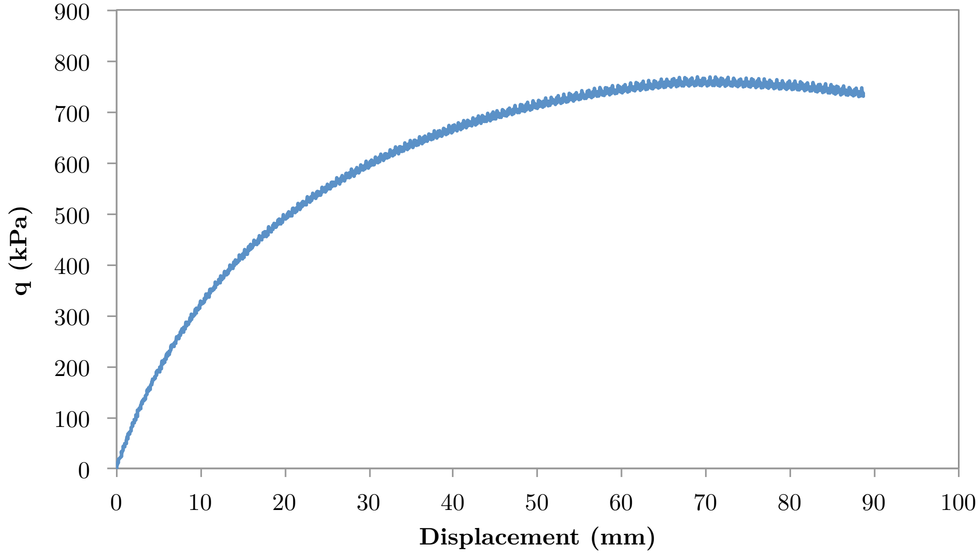


Figure 7.43: Triaxial compression loading test of dry Soil A with $\sigma_r = 200$ kPa.

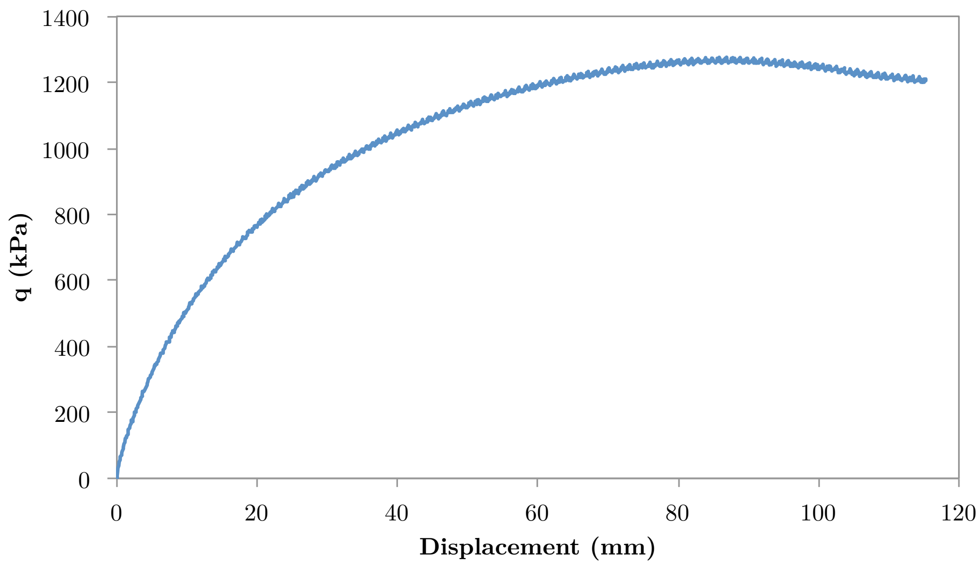


Figure 7.44: Triaxial compression loading test of dry Soil A with $\sigma_r = 300$ kPa.

As stated in Chapter 6, when testing the conditioned soil with the compression loading test the depression was not guaranteed and the tests have been carried out by using extension unloading configuration. In order to better compare the results, also on dry Soil A a campaign of extension unloading triaxial tests have been performed. Table 7.3 shows the used confinements, while Figures 7.45, 7.46 and 7.47 shows the results of the campaign.

Table 7.3: Confinement pressure σ_r used for the triaxial extension unloading testing campaign on dry samples of Sand A.

Test	σ_r (kPa)
1	150
2	200
3	300

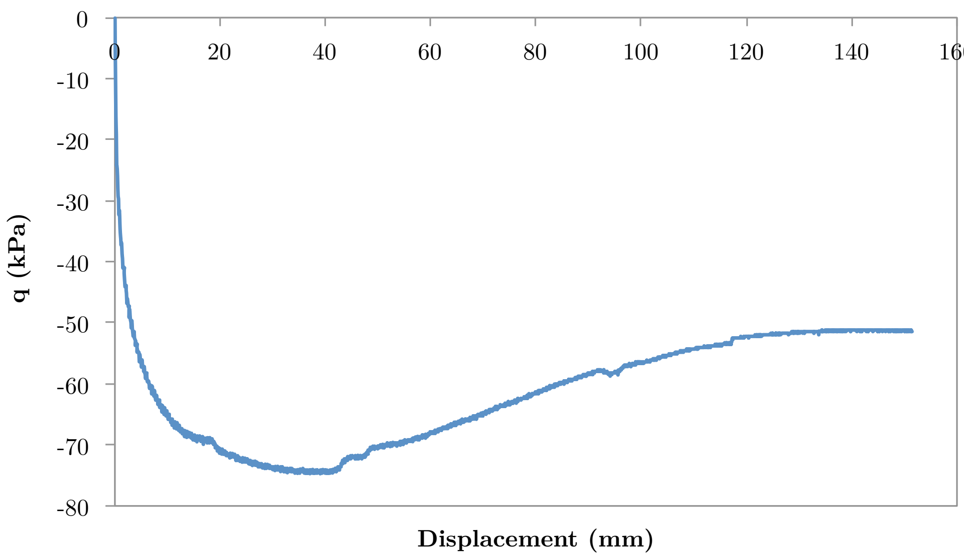


Figure 7.45: Triaxial extension unloading test of dry Soil A with $\sigma_r = 100$ kPa.

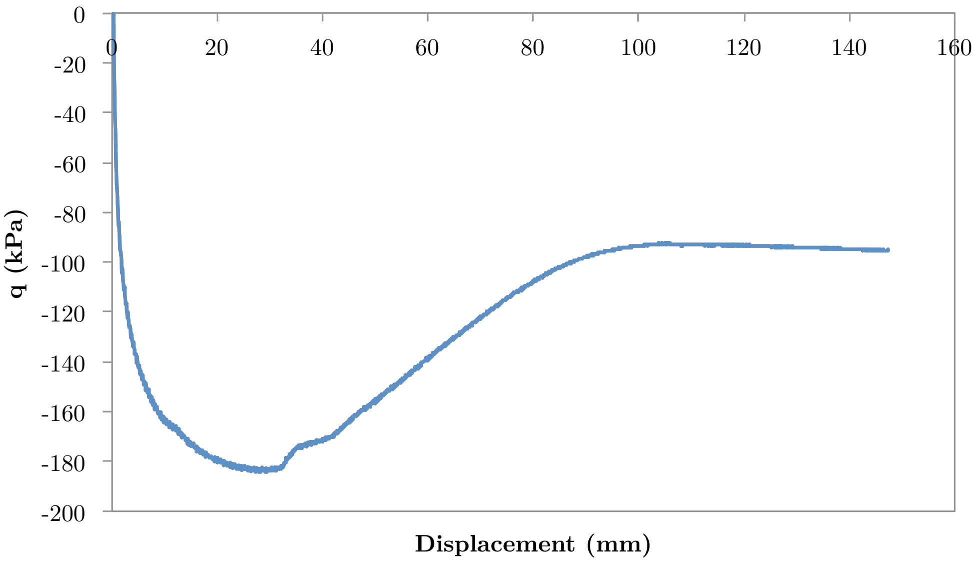


Figure 7.46: Triaxial extension unloading test of dry Soil A with $\sigma_r = 250$ kPa.

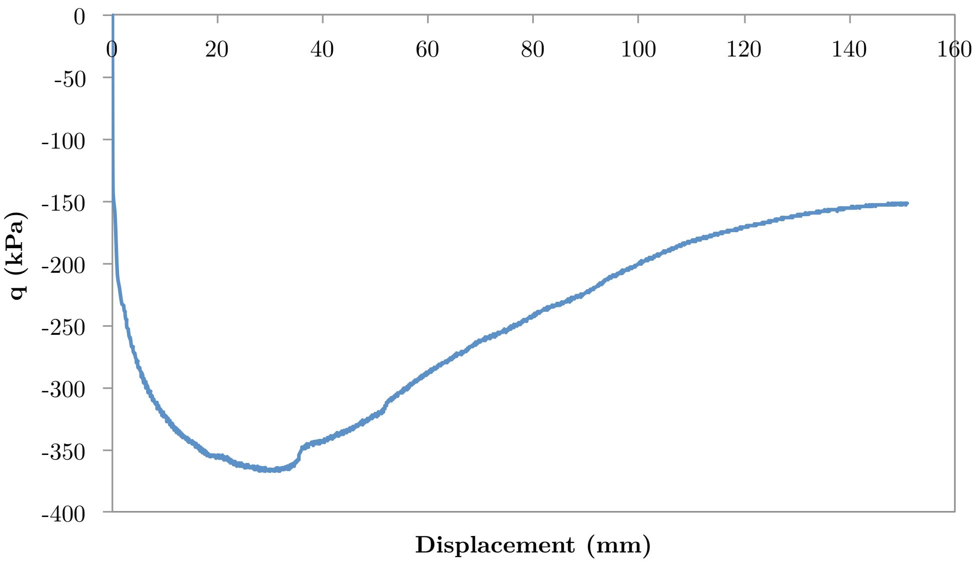


Figure 7.47: Triaxial extension unloading test of dry Soil A with $\sigma_r = 500$ kPa.

Combining the results obtained from the 6 tests (in Figure 7.48 the 6 tests are plotted together), it is possible to get the Mohr-Coulomb failure envelope through the Mohr's circle in Figure 7.49.

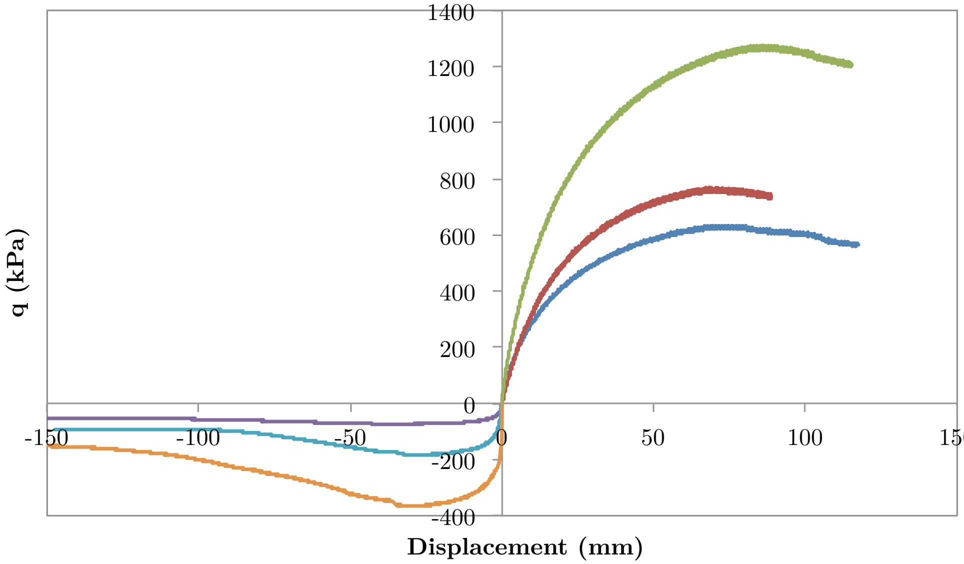


Figure 7.48: Outcome of the triaxial campaign on dry Soil A.

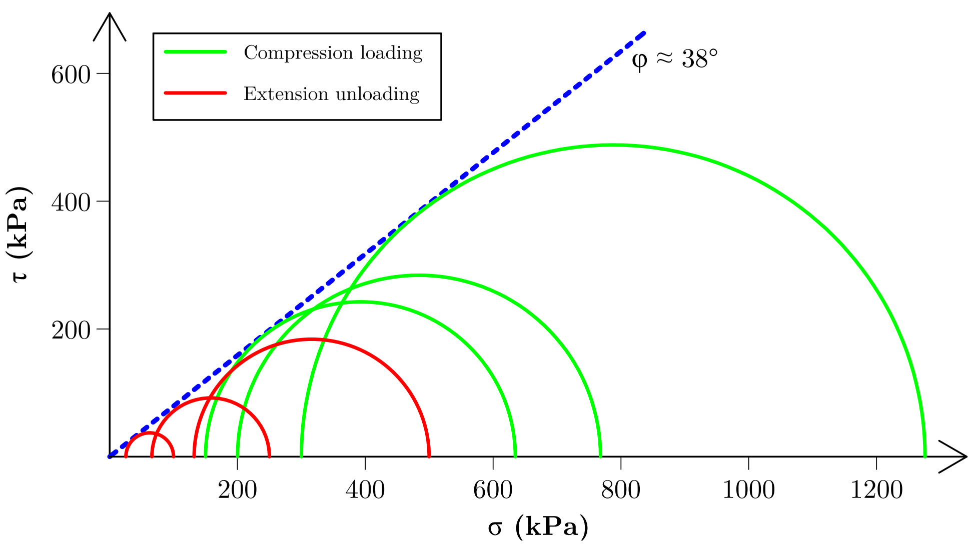


Figure 7.49: Failure envelope and Mohr's circles from the triaxial tests performed on the dry Soil A.

Conditioned

The campaign has been carried out by using 4 different confinement pressures σ_r, as shown in Table 7.4. The choice of using an additional test compared to the usual procedure is due to the results given from the test with $\sigma_r = 100$ kPa which returned a unusual graph (Figure 7.50). This might be explained by the fact that the material is acting completely as a fluid at this confinement, and does not reach a pressure able to compress the bubbles enough to guarantee the contact between the grains. In this state, at this pressure, the material can transmit effectively the pressure in a EPB shield excavating chamber. The graphs of the other three tests are shown in Figures 7.51, 7.52 and 7.53.

Table 7.4: Confinement pressure σ_r used for the triaxial extension unloading testing campaign on conditioned samples of Sand A.

Test	σ_r (kPa)
1	100
2	250
3	325
4	400

Another test performed on the conditioned sand, carried out in order to assess the behaviour of the mass during the application of the pressure in the cell, is the lateral confinement increase test. The maximum reached pressure has been set to 500 kPa. The test has been performed in steps of 50 kPa after the first pressure set to 150 kPa and each step has been kept some minutes (around 5-6 minutes) to stabilize the pressures. The outcome of this test is shown in Figure 7.54.

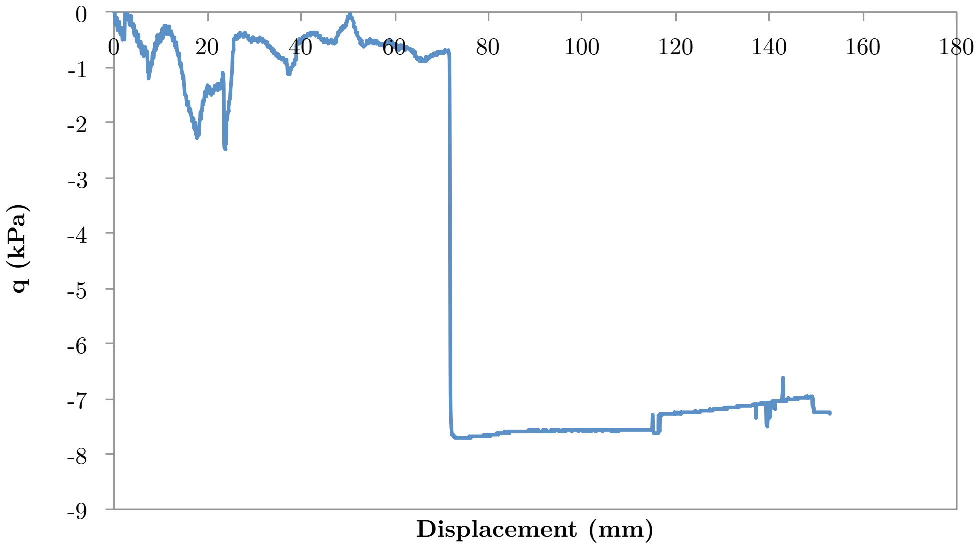


Figure 7.50: Triaxial extension unloading test of conditioned Soil A with $\sigma_r = 100$ kPa. The graph does not show a clear peak, which might be explained by the fact that the material is acting completely as a fluid at this confinement.

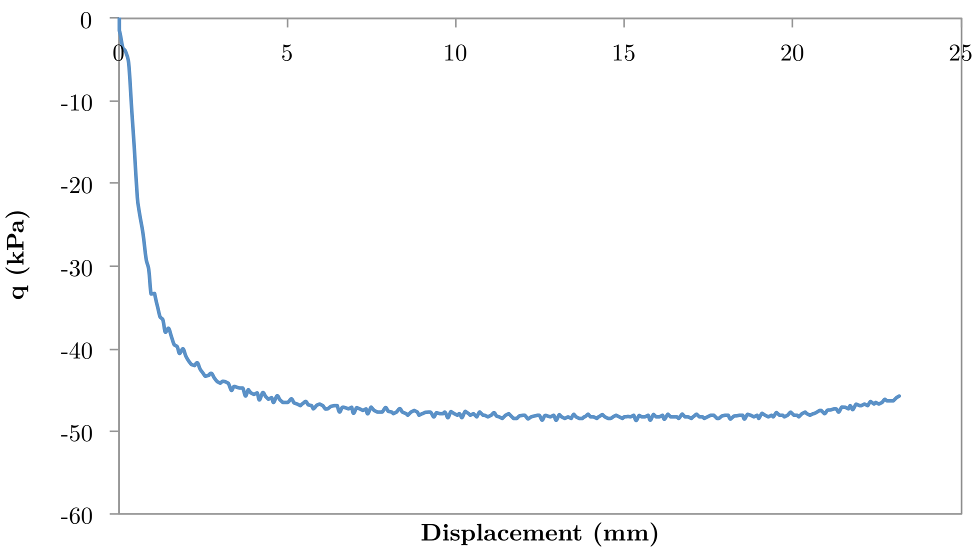


Figure 7.51: Triaxial extension unloading test of conditioned Soil A with $\sigma_r = 250$ kPa.

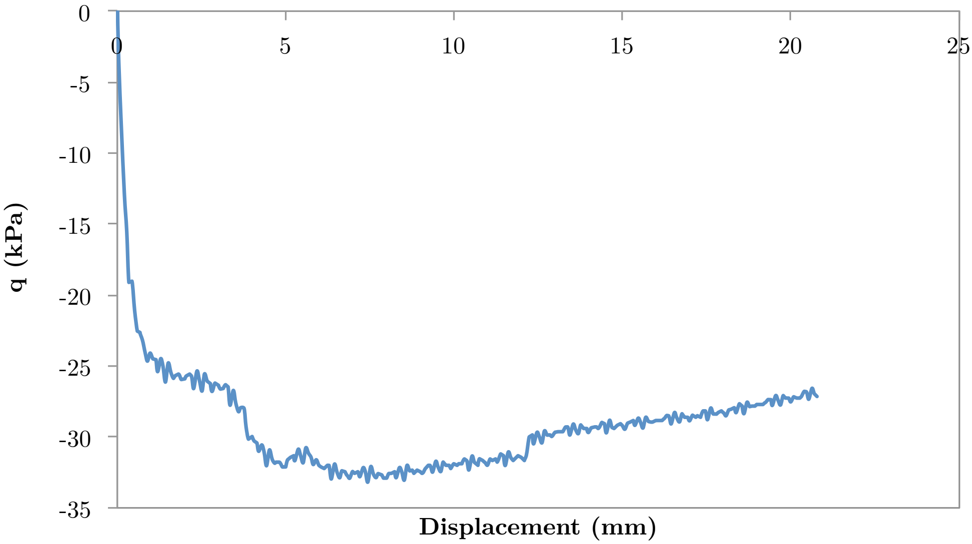


Figure 7.52: Triaxial extension unloading test of conditioned Soil A with $\sigma_r = 325$ *kPa.*

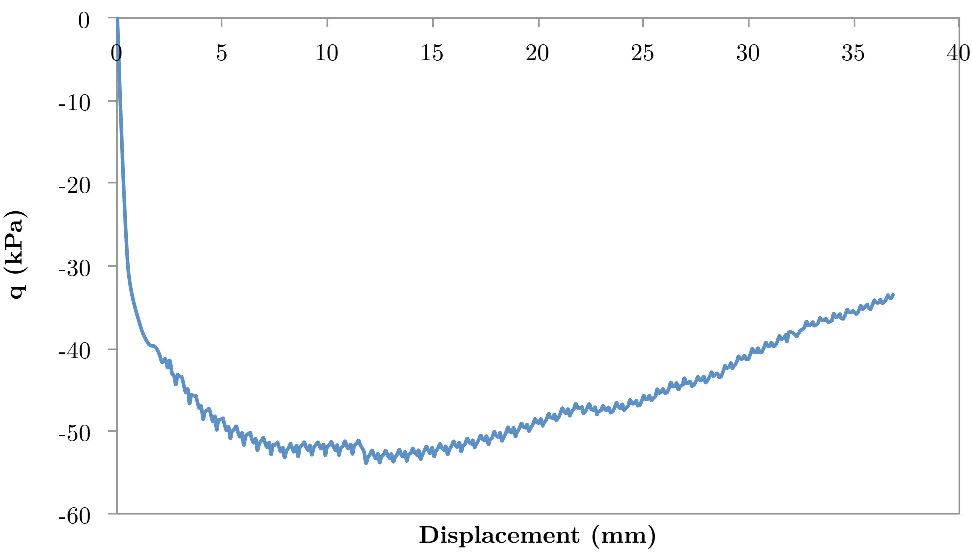


Figure 7.53: Triaxial extension unloading test of conditioned Soil A with $\sigma_r = 400$ *kPa.*

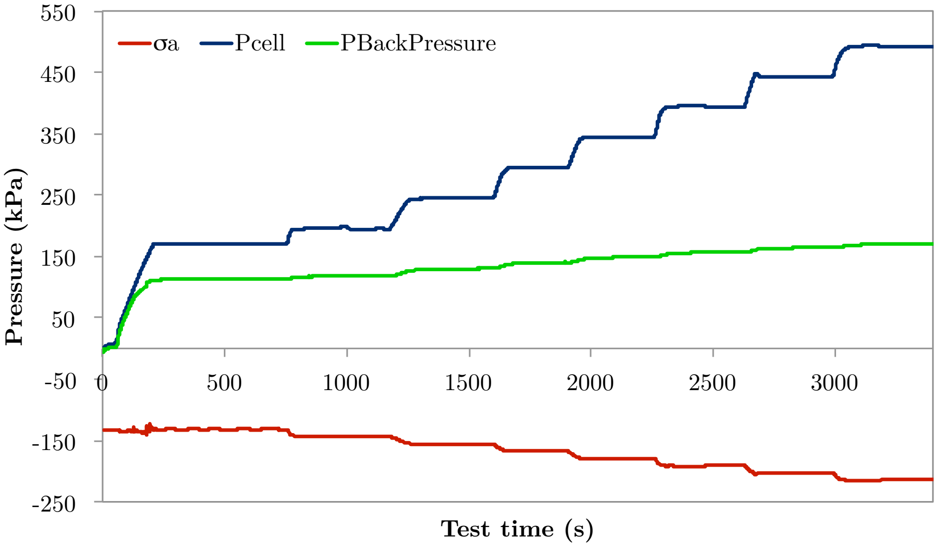


Figure 7.54: Triaxial lateral confinement increase test of conditioned Soil A up to $\sigma_r = 500$ kPa.

The results obtained from this test campaign on conditioned Soil A allowed to better understand the mechanical behaviour of the conditioned mass under different pressure conditions. An interesting outcome is presented in Figure 7.55, where the 5 tests of the campaign are plotted in a graph Δ pressure, which is representing difference between the pressure applied in the cell (σ_r) and the interstitial pressure (σ_{int}), and the pressure in the cell itself (σ_r). This graph is interesting to understand the moment in which the material is starting to become more rigid due to the contact between the grains described already in Chapter 5.

The graph is showing that for confinement pressures σ_r lower than 150 kPa the Δ pressure is small, confirming the fact that at 100 kPa of confinement the material is still behaving as a fluid (Figure 7.50).

Another important result is given by an unexpected failure of the membrane which occurred during the test with $\sigma_r = 400$ kPa much after the peak. In Figure 7.56 this event is showed through a pressure vs. time graph which shows the axial pressure peak at around 500 s from the test starting and the failure of the membrane, indicated by the sudden rise of the interstitial pressure to the cell pressure. After the removal of the cell the failure was visible from the membrane, with the foam flowing out from the sample (Figure 7.57). This failure has proved to be positive, as in this way it was possible to check the behaviour of the material in drained conditions.

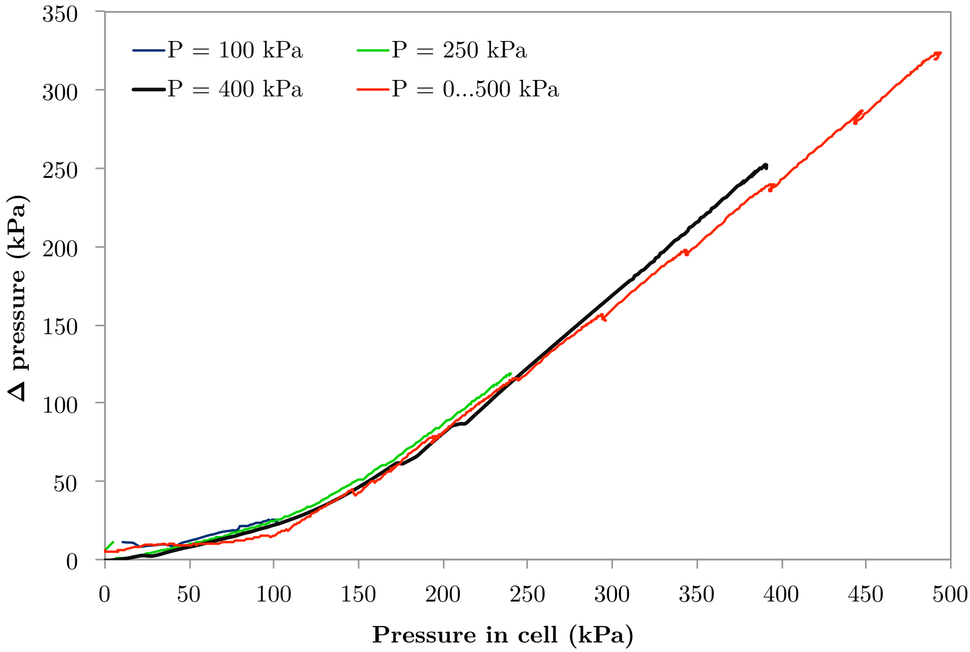


Figure 7.55: Δ pressure vs. pressure in the cell (σ_r) for the conditioned Soil A.

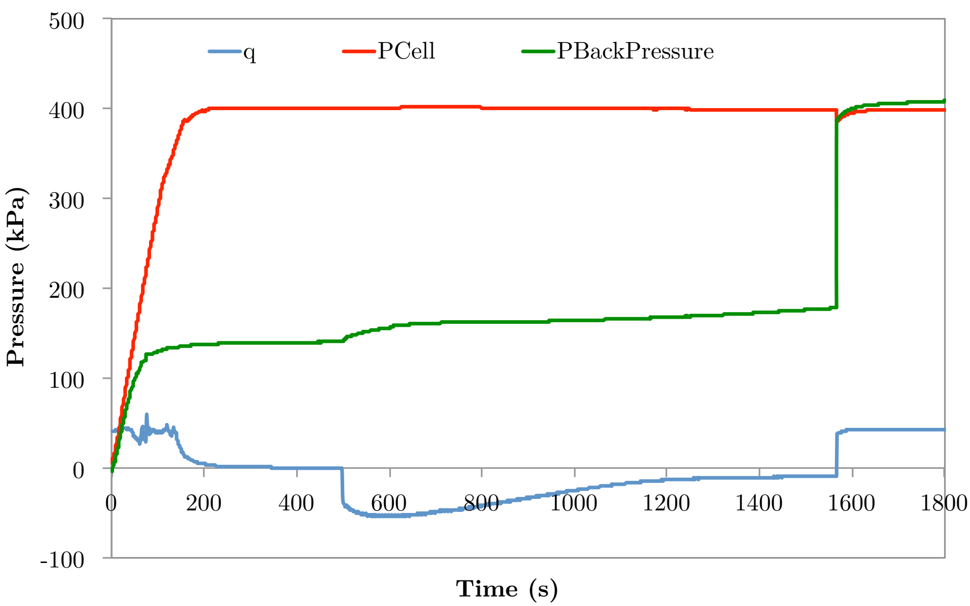
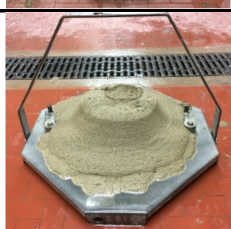
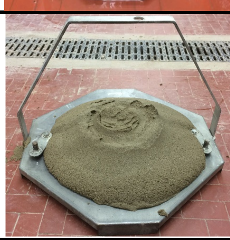


Figure 7.56: Graph pressure vs. time of the triaxial test on conditioned Soil A with $\sigma_r = 400$ kPa with the evidence of the membrane failure.

Figure 7.57: Broken membrane with the foam flowing out from the sample (drained conditions).

As stated in the description of the new apparatus (Section 6.3, to verify the material after each test the slump test has been used. In this way, it is possible to compare the material by using an almost standardized procedure at different pressures. Table 7.5 shows the results of all the tests, carried out on the material at room pressure which was stored in a sealed tank during the triaxial test, on the material inside the membrane in the top and bottom part. In this way also the stability of the mass can be assessed: if the top and bottom samples are similar, it means that the foam is not flowing down because of the gravity. Also from this test it has been possible to notice the difference of behaviour of the material in drained and undrained conditions: at the row corresponding to $\sigma_r = 400$ kPa, the top slump shows a dry material due to the failure of the membrane, on the contrary the bottom slump shows that the material kept its properties due to the good stability of the conditioned mass which prevented too big loss of foam through the breach on the membrane.

Table 7.5: Slump values and pictures from the samples taken after each triaxial test. For each pressure there is a slump of the sample at room pressure (P_0), from the top and from the bottom of the cell. The slump of the material which lost the undrained conditions is highlighted in red.

σ_r (kPa)	**Time (min)**	**Slump (cm)**		
		P_0	**Top**	**Bottom**
100	160	22 	19 	17 
250	135	22 	18 	18 
325	135	21 	17 	19 
400	150	22 	3 	14 
0...500	180	23 	19 	16 

7.3.3 Sand E

Dry

The campaign has been carried out by using 3 different confinement pressures σ_r, as shown in Table 7.6.

Table 7.6: Confinement pressure σ_r used for the triaxial compression loading testing campaign on dry samples of Sand E.

Test	σ_r (kPa)
1	100
2	250
3	500

The graphs showing the results of the 3 tests are shown in Figures 7.58, 7.59 and 7.60.

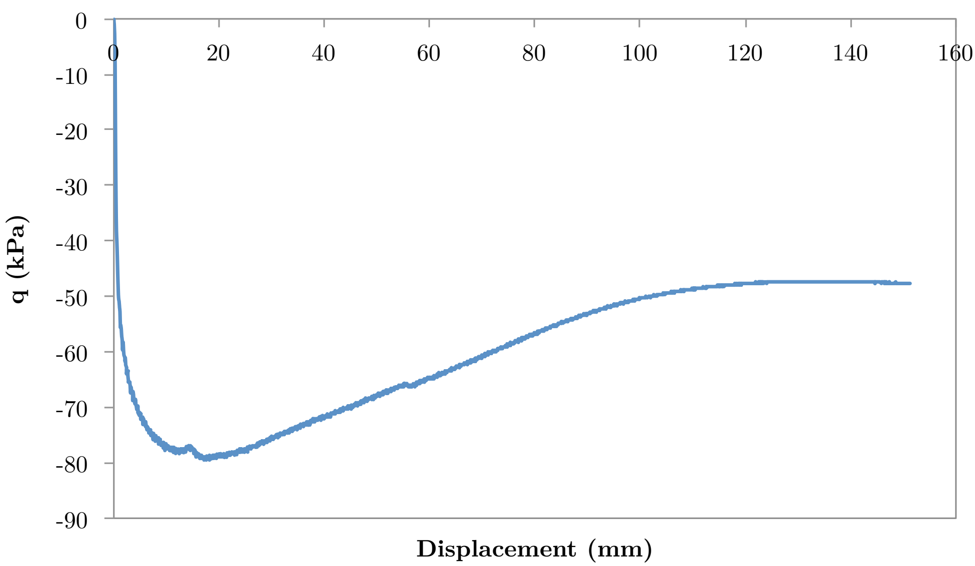


Figure 7.58: Triaxial extension unloading test of dry Soil E with $\sigma_r = 100$ kPa.

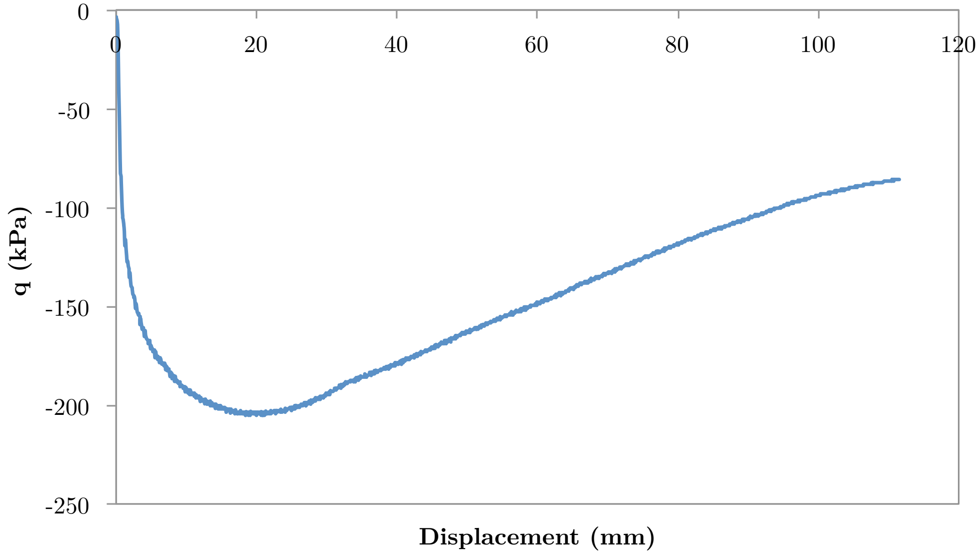


Figure 7.59: Triaxial extension unloading test of dry Soil E with $\sigma_r = 250$ *kPa.*

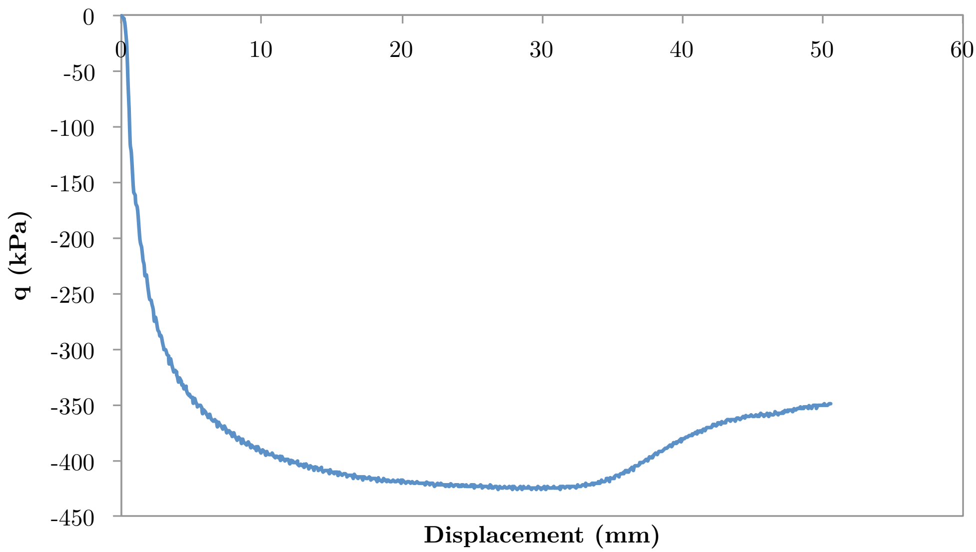


Figure 7.60: Triaxial extension unloading test of dry Soil E with $\sigma_r = 500$ *kPa.*

Combining the results obtained from the 3 tests (in Figure 7.61 the 3 tests are plotted together), it is possible to get the Mohr-Coulomb failure envelope through the Mohr's circle in Figure 7.62.

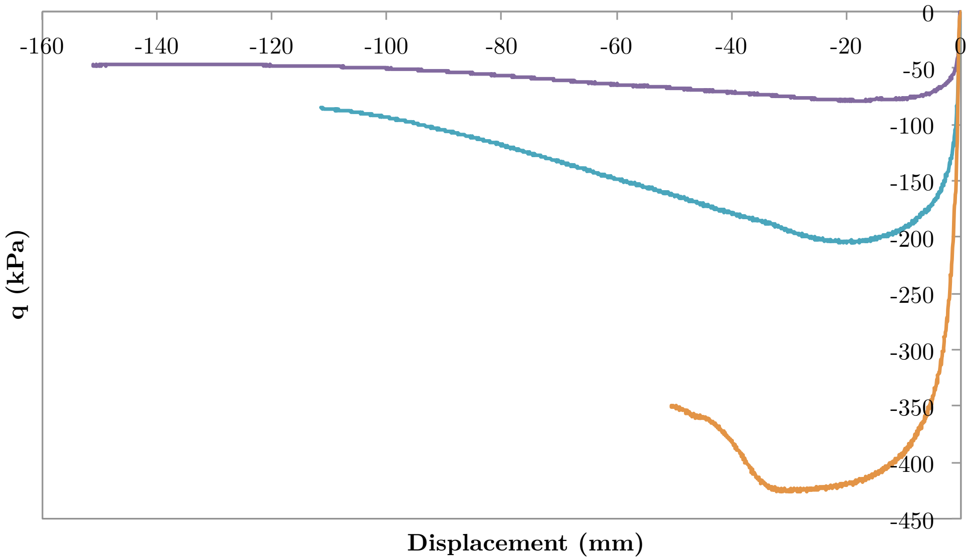


Figure 7.61: Outcome of the triaxial campaign on dry Soil E.

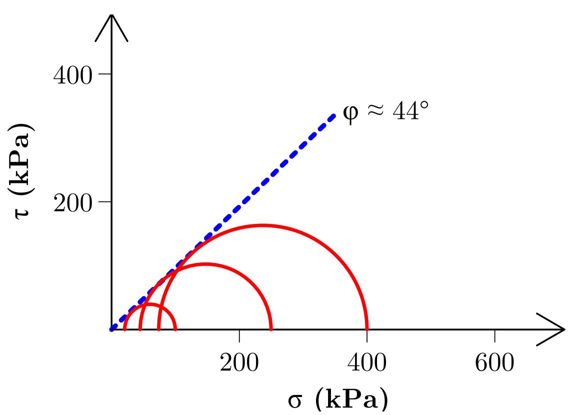


Figure 7.62: Failure envelope and Mohr's circles from the triaxial tests performed on the dry Soil E.

Conditioned

The testing of this soil by using the triaxial test has been more difficult compared to the previous material. Two tests have been performed with confinement pressures equal to 150 and 250 kPa; the test with this last σ_r has been repeated twice because the first test did not return valid results. Although the testing campaign did not return the expected results, but anyhow the overall behaviour of the material during the testing phases was sufficient to give useful indications. Figures 7.63 and 7.64 show the graphs of the positive test, while Figure 7.64 shows the failed test after few millimeters of piston displacement.

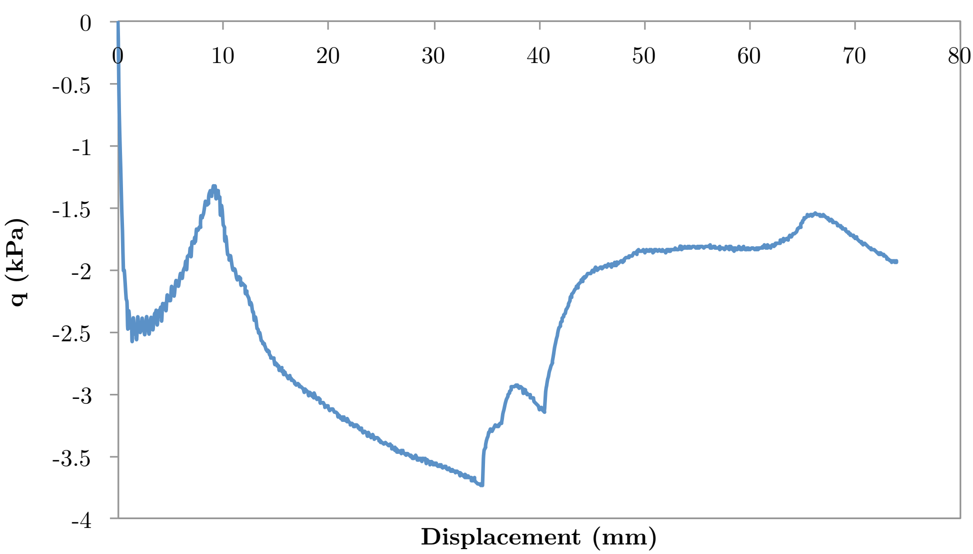


Figure 7.63: Triaxial extension unloading test of conditioned Soil E with $\sigma_r = 150$ kPa.

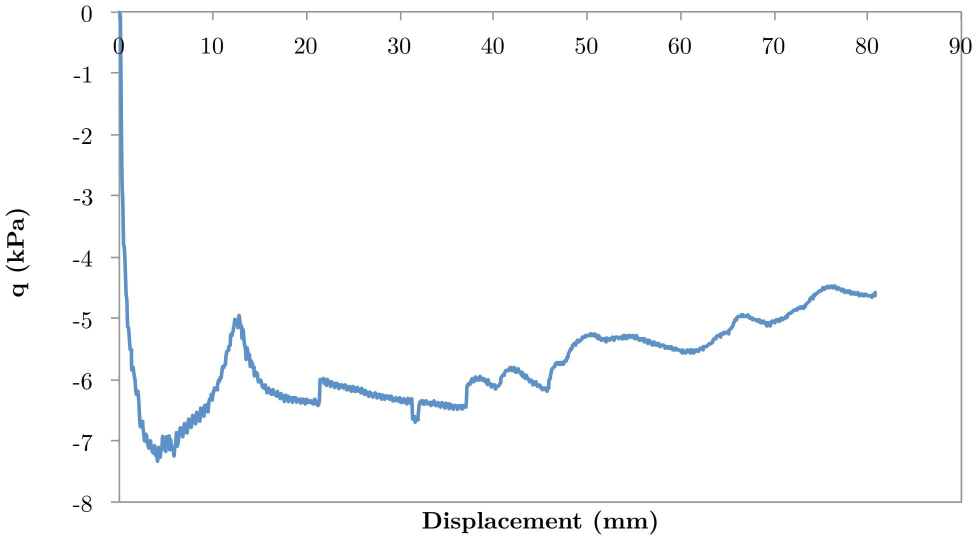


Figure 7.64: Triaxial extension unloading test of conditioned Soil E with $\sigma_r = 250$ kPa.

As visible from the graphs, the results do not appear so satisfactory as for the Soil A, probably due to the high heterogeneity of the conditioned mass which was affected by the gravel part. Nevertheless it is interesting to analyze the trend of the cell and interstitial pressure during the positive tests and after increasing the pressure up to 500 kPa (Figures 7.66 and 7.67), as the mass is behaving effectively when the pressure is increased, probably due to the fluid behavior of the fine part of the soil (clay and silt). In fact the clayey conditioned soils are usually transmitting the pressure much more effectively, thus also in this case the finer part is helping the increase of the interstitial pressure.

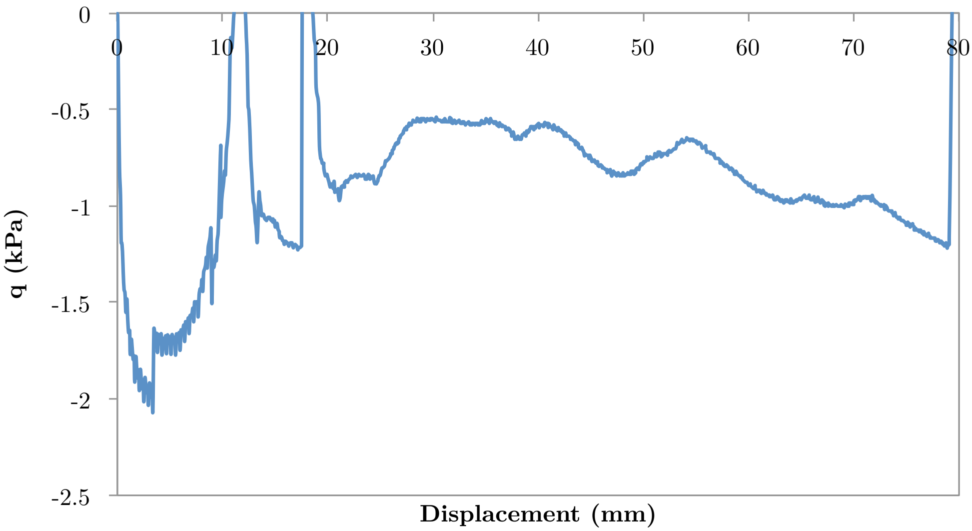


Figure 7.65: Failed triaxial extension unloading test of conditioned Soil E with $\sigma_r = 250$ kPa.

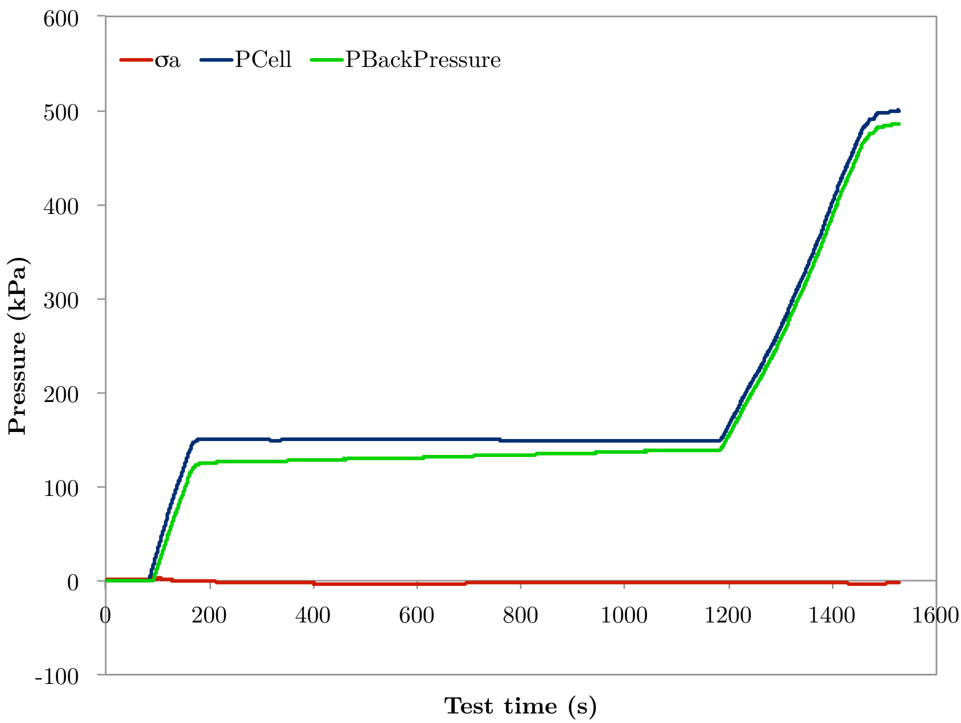


Figure 7.66: Triaxial extension unloading test of conditioned Soil E with $\sigma_r = 250$ kPa.

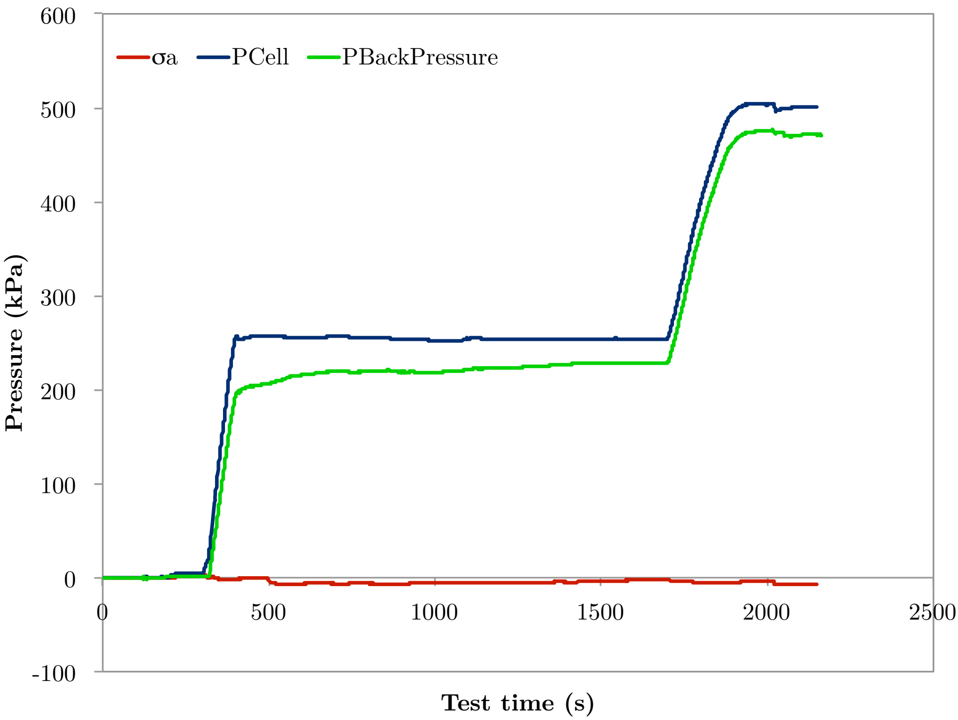


Figure 7.67: Triaxial extension unloading test of conditioned Soil E with $\sigma_r = 250$ *kPa.*

7.3.4 Analysis of the results

The campaign carried out on Soil A showed really interesting results regarding the study of the mechanical behaviour of the conditioned masses. The method allows to easily compare the behaviour of the material in different states, by studying similar parameters proper of the geotechnical engineering and especially by assessing parameters useful to the mechanized tunnelling engineering. These lasts include the verification of the attitude of the material of transmitting the pressure effectively during the pressure increase phase and the verification after the test of the condition of the mass through for example the slump test. The campaign carried out on Soil E unfortunately was not as positive as with the other material, but this was mainly due to the impossibility of testing additional samples due to lack of raw material. This did not imply that the results obtained have been unusable at all, on the contrary the material showed a good attitude as well on providing effective counterpressure due to the finer part of the mass which created a perfect pulpy and plastic mass, where the bigger grains are trapped inside and the material is not segregating.

The use of extension unloading stress path allows to avoid to applied the negative pressure to a sample which is not saturated and needs to keep the air trapped inside the conditioned mass in form of foam bubbles.

7.4 Pressurized rotating mixer

7.4.1 Introduction

Also for this test, for the same reasons given for the modified large diameter triaxial test, the campaign has been conducted for Soil A and Soil E, first on the dry sample and then on the conditioned one.

7.4.2 Soil A

Dry

The first test, according to the method described in Section 6.4.3, has been carried out on dry Soil A. This test has been performed with the pressure step configuration, in order to study any time influence on the mass. The result of the test is shown in Figure 7.68. The result shows that the torque is constant during the 300 s of each step. A second test has been carried out on this soil, by using the constant pressure increase configuration, and the results are shown in Figure 7.69 (pressure increase) and Figure 7.70 (pressure decrease).

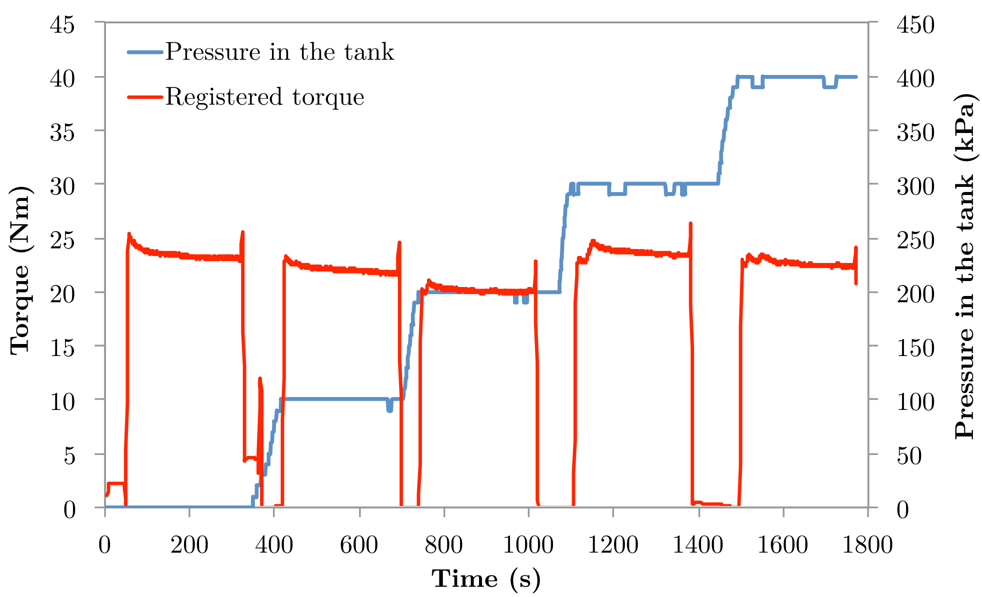


Figure 7.68: Pressurized rotating mixer test with pressure steps on dry Soil A.

Conditioned

Also in this case the test has been carried out with the pressure step configuration, in order to have a direct comparison with the dry test. The outline of this test is shown

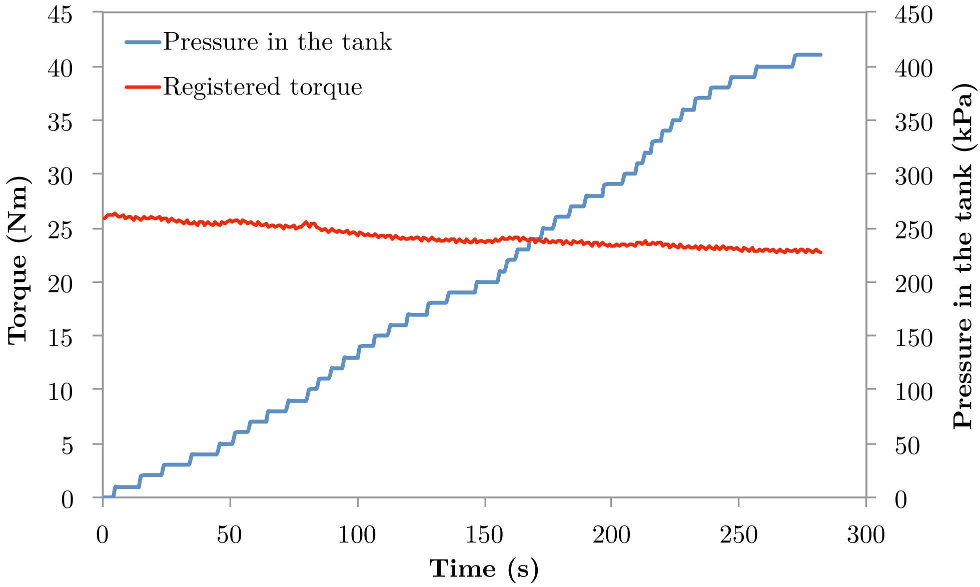


Figure 7.69: Pressurized rotating mixer test with constant pressure increase on dry Soil A.

in Figure 7.71.

In this case the test shows a much different behaviour of the conditioned mass compared to the dry one. First of all the torque is not constant but it changes over time, indicating that the bubbles are not static but they change the state and the interaction with the soil grains. Second point, which is probably more important than the previous, is that the material changes suddenly and strongly the its stiffness (the torque increases almost 3 times) when the first step of pressure (100 kPa) is applied. This evidence confirms the theory which was underlined in Section 5.3 and especially in Figure 5.1, and it is aligned with the results obtained during the triaxial testing campaign (Section 7.3.2).

This important result, brought to repeat the test on the conditioned sample of Soil A by using the constant pressure increase configuration, in order to better understand this change in stiffness during the application of the pressure. The test has been carried out in cycles: first the pressure is constantly applied to 500 kPa, and then it is constantly dropped to room pressure. This procedure has been repeated 3 times. The outline of this test is shown in Figures 7.72,7.73, 7.74, 7.75, 7.76 and 7.77.

The results coming from this set of tests are really useful to understand the mechanical behaviour of the material when stressed in certain pressure conditions. From the increase of pressure of the first cycle (Figure 7.72), which represents the part of the test carried out on the “fresh”, it is clear that at around 150 kPa of pressure in the tank the conditioned mass becomes much stiffer, as the torque increases decisively. Then there are other sudden increase of torque as the pressure is reaching certain values, and the overall trend is always indicating a direct correlation between the pressure and the

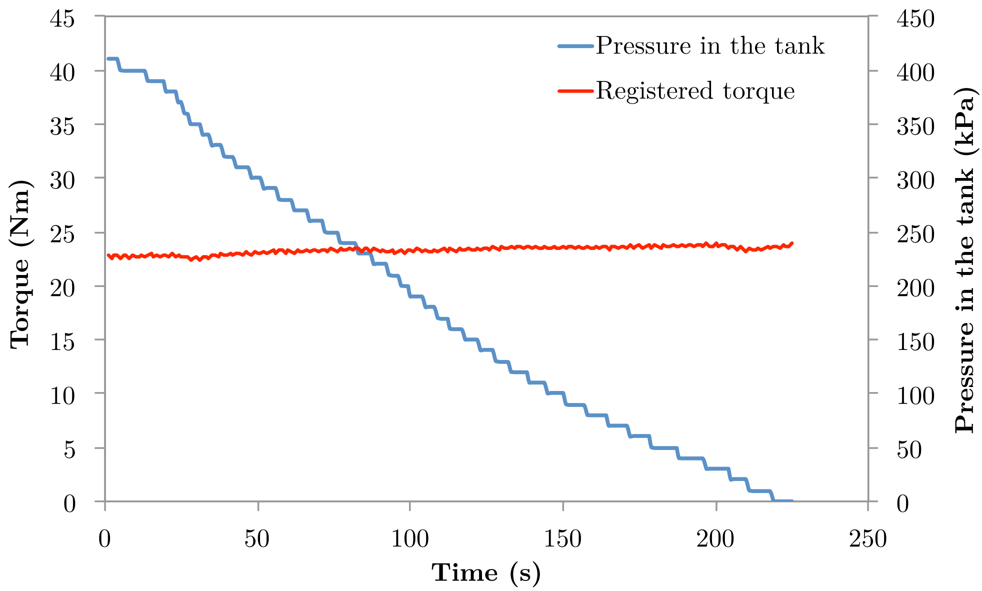


Figure 7.70: Pressurized rotating mixer test with constant pressure decrease on dry Soil A.

torque. Similar trend is found when the pressure is decreased (Figure 7.73), but the torque is decreasing more constantly with just one step at around 300–350 kPa. This difference with the previous test is probably due to the expansion of the foam bubbles at a bigger size. This probably means that the material in the tank is not at the same status from the beginning, thesis that is supported also by the visual analysis of the tank which shows a volume of material inside larger than at the beginning (Figure 7.78). This is supported by the fact that the extracted material has a unit weight volume equal to 0.7 kN/m^3, which is around 30% lower than the unit weight of the initial conditioned mass.

After the test, a slump test has been carried on the extracted material, in order to verify the consistency and suitability of the mass. The fall has been equal to 23 cm (Figure 7.79) and the material appears very fluid.

From the campaign carried out on Soil A, a final graph showing the overall behaviour of the dry and conditioned masses has been plotted (Figure 7.80).

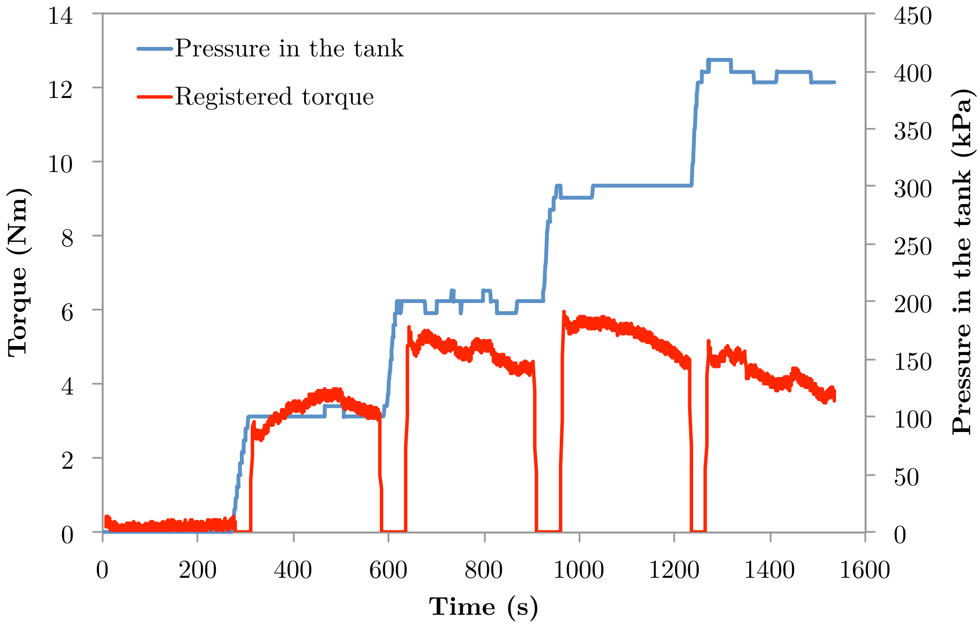


Figure 7.71: Pressurized rotating mixer test with pressure steps on conditioned Soil A.

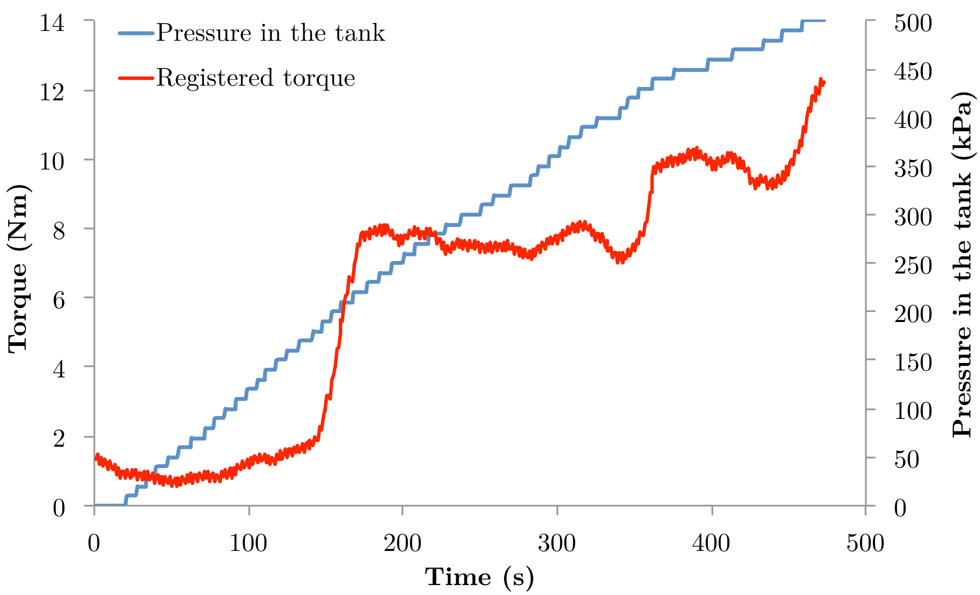


Figure 7.72: Pressurized rotating mixer test with constant pressure increase on conditioned Soil A (1st cycle).

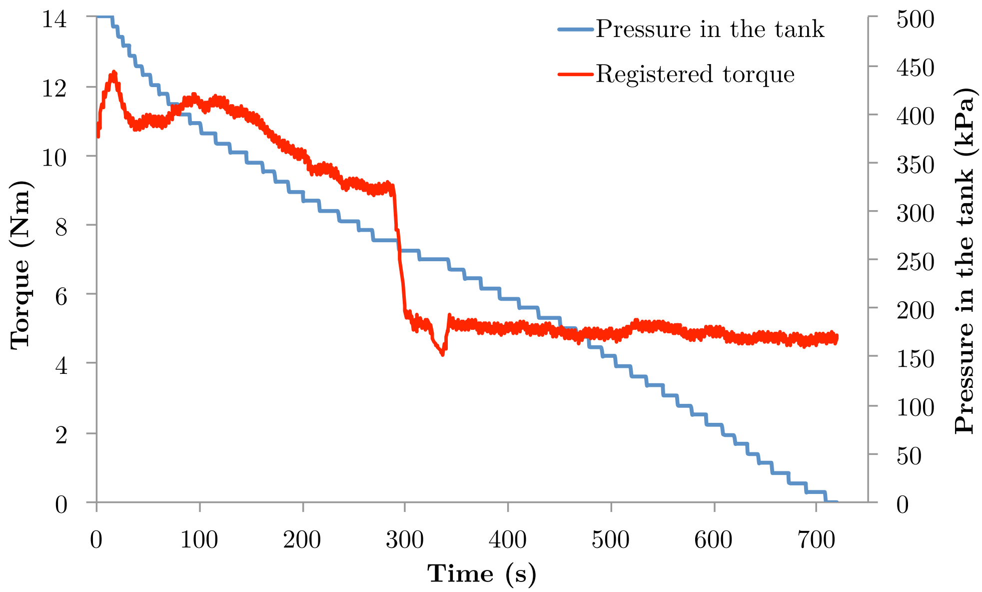


Figure 7.73: Pressurized rotating mixer test with constant pressure decrease on conditioned Soil A (1st cycle).

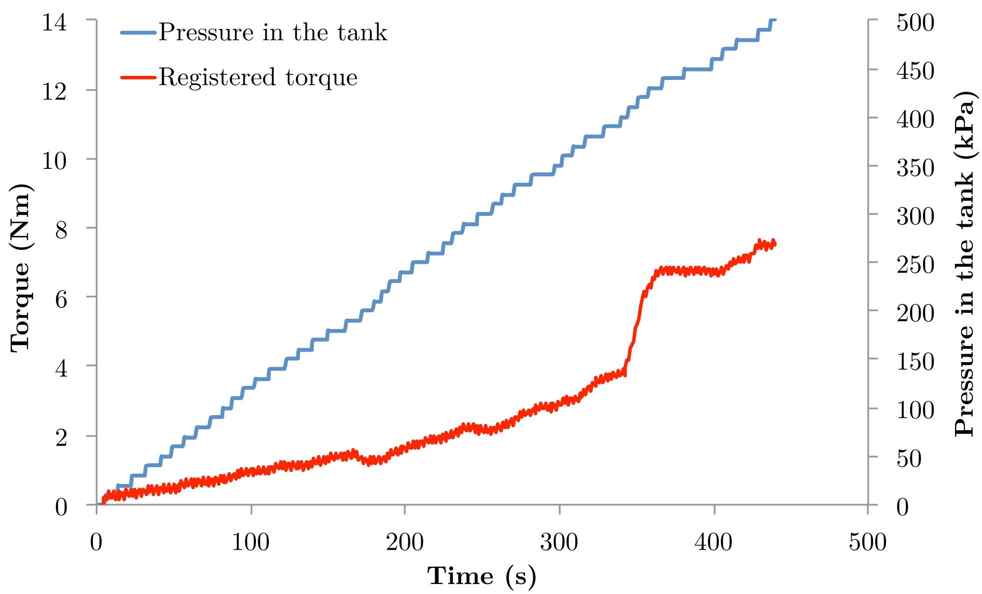


Figure 7.74: Pressurized rotating mixer test with constant pressure increase on conditioned Soil A (2nd cycle).

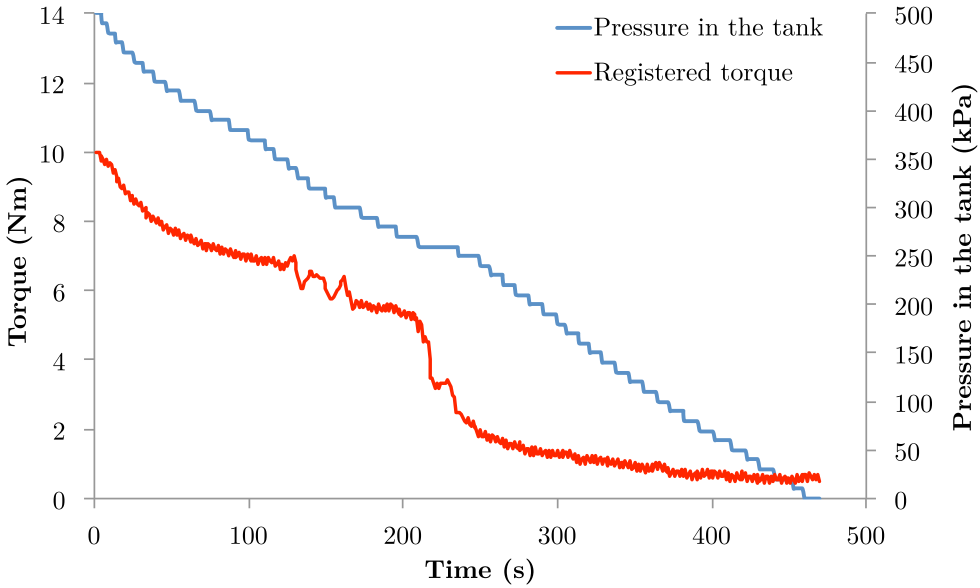


Figure 7.75: Pressurized rotating mixer test with constant pressure decrease on conditioned Soil A (2^{nd} cycle).

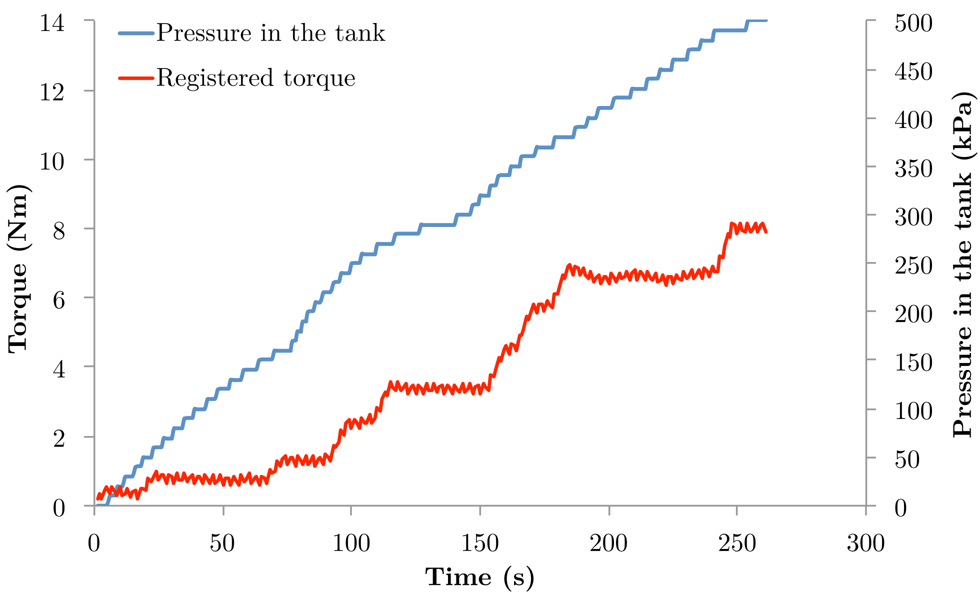


Figure 7.76: Pressurized rotating mixer test with constant pressure increase on conditioned Soil A (3^{rd} cycle).

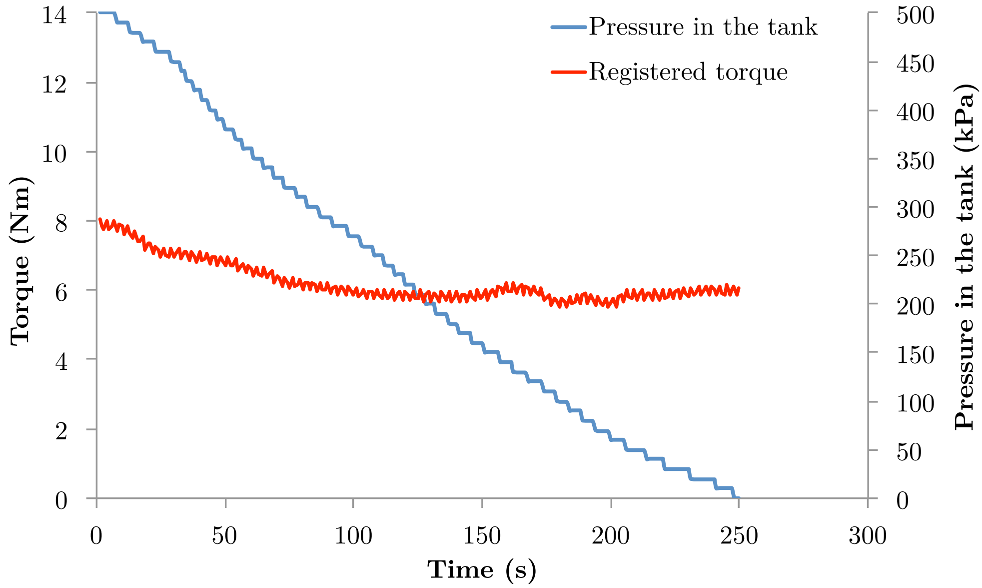


Figure 7.77: Pressurized rotating mixer test with constant pressure decrease on conditioned Soil A (3rd cycle).

(a) (b)

Figure 7.78: Conditioned material at the beginning of the test (a) and after the cycle increase/decrease of pressure at 500 kPa (b).

Figure 7.79: Slump test carried out on conditioned Soil A extracted from the test.

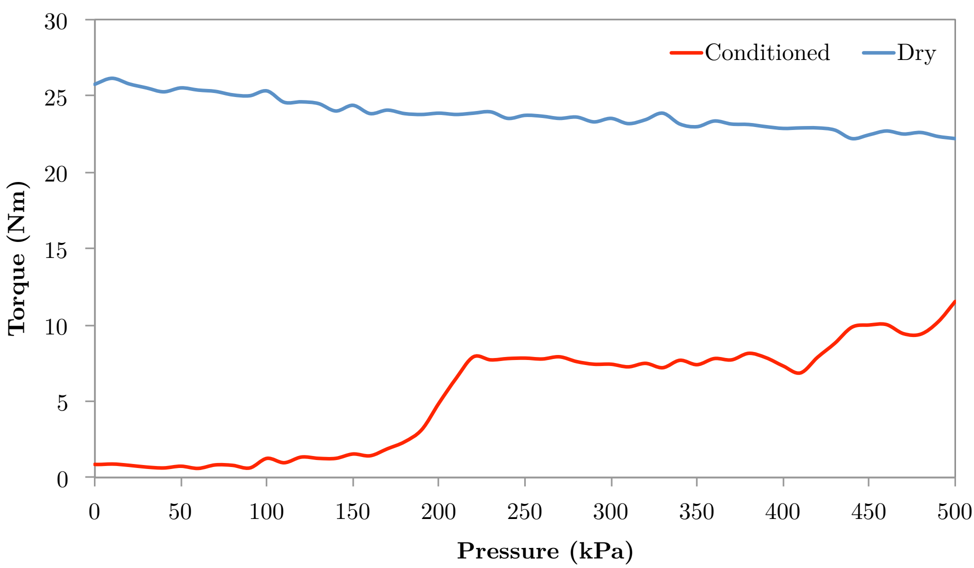


Figure 7.80: Overall correlation between torque and pressure in the tank for Soil A.

7.4.3 Soil E

Dry

The study of this soil has been carried out both with the pressure step configuration and with the constant pressure increase configuration. The outline of these tests is shown in Figure 7.81 (pressure step configuration) and in Figure 7.82 (constant pressure increase).

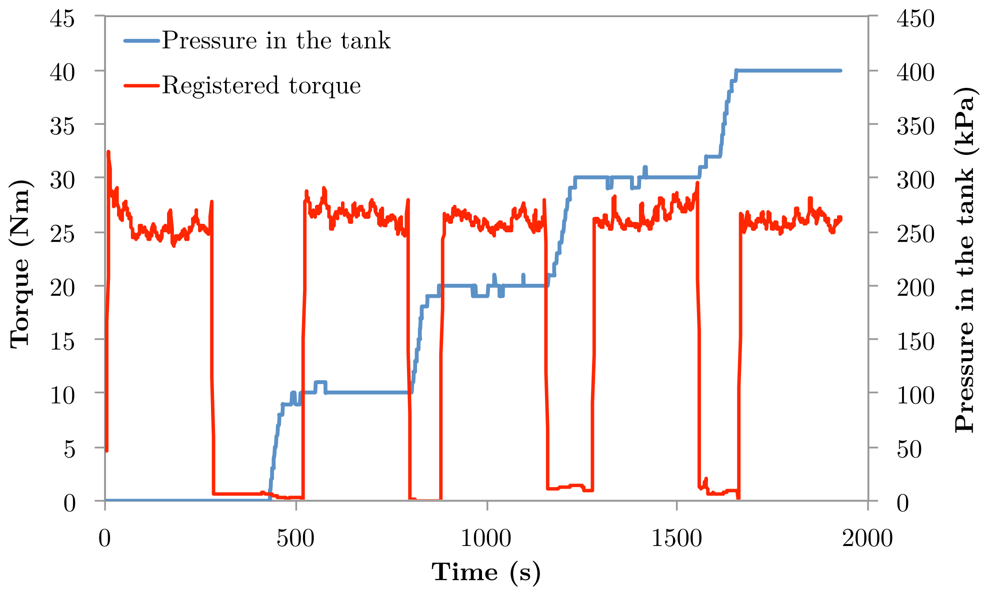


Figure 7.81: Pressurized rotating mixer test with pressure steps on dry Soil E.

In both the tests nothing relevant has been noticed, the torque remains almost constant during the all phases of pressure application, showing the good initial aggregation of Soil E.

Conditioned

Same configuration of tests also for the conditioned mass. The results of these tests are shown in Figure 7.83 (pressure step configuration) and in Figure 7.84 (constant pressure increase).

In this case the conditioned mass keeps better its stiffness during the pressure application, even if in general the value of torque is higher than in Soil A. It is interesting to notice the peak of torque reached in Figure 7.83, this is probably due to the fact that the mass is not mixing during the pressure step increase, so probably the mass can strain a bit more when is static.

After the test, a slump test has been carried on the extracted material, in order to verify the consistency and suitability of the mass. The fall has been equal to 16 cm

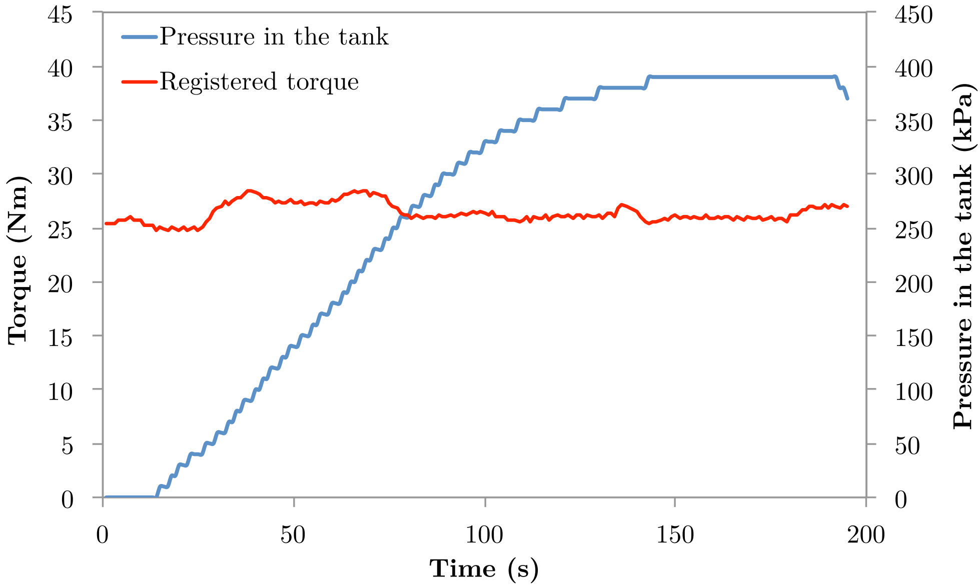


Figure 7.82: Pressurized rotating mixer test with constant pressure increase on dry Soil E.

(Figure 7.85) and the material appears still well conditioned.

Also from the campaign carried out on Soil E, a final graph showing the overall behaviour of the dry and conditioned masses has been plotted (Figure 7.86).

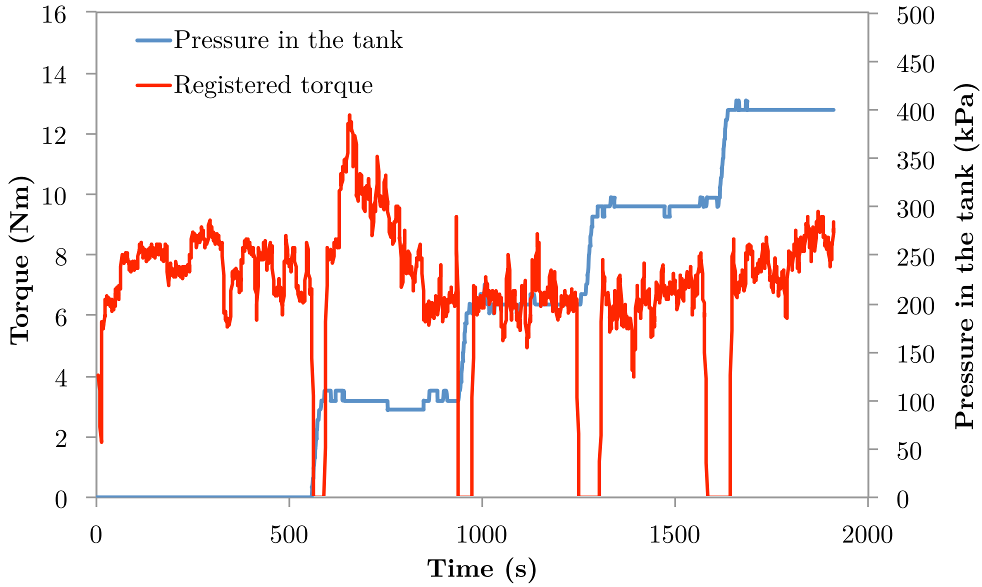


Figure 7.83: Pressurized rotating mixer test with pressure steps on conditioned Soil E.

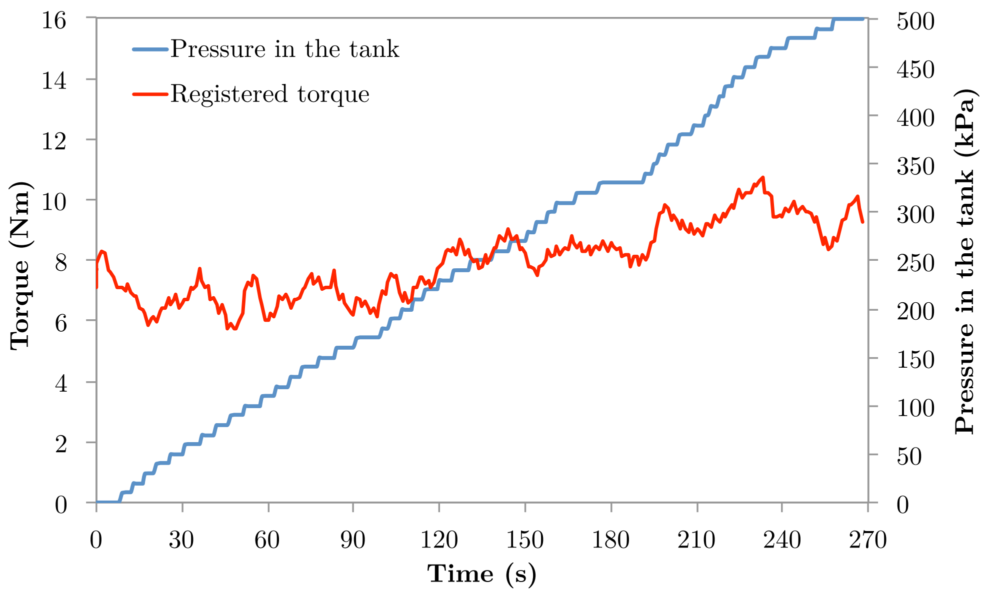


Figure 7.84: Pressurized rotating mixer test with constant pressure increase on conditioned Soil E.

Figure 7.85: Slump test carried out on conditioned Soil E extracted from the test.

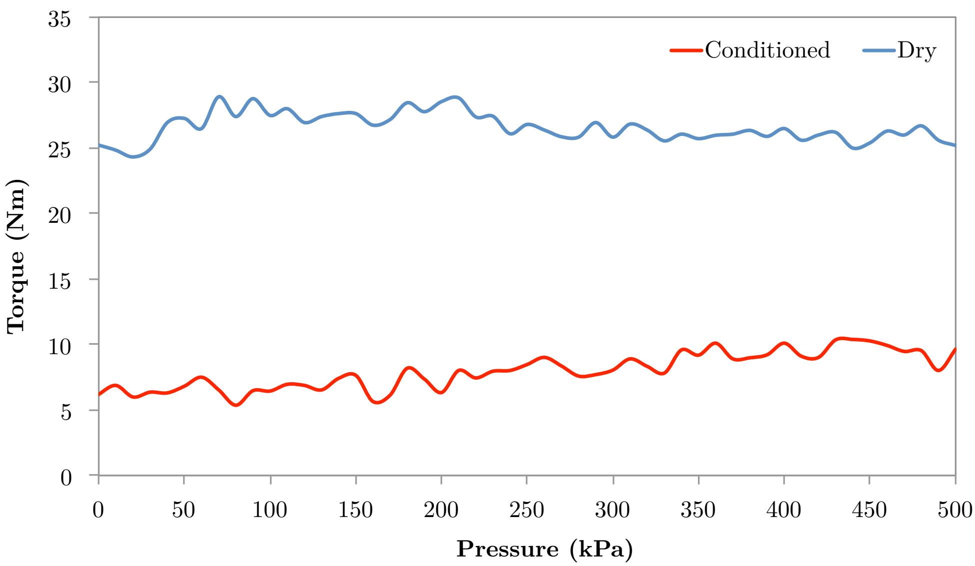


Figure 7.86: Overall correlation between torque and pressure in the tank for Soil E.

7.4.4 Analysis of the results

This new testing device, which allows to study the workability of the mass at different pressure levels and so different consistency of the material, provided excellent indications regarding both Soil A and Soil E.

A final comparison of the mechanical behaviour in terms of workability of the masses is given in Figure 7.87, which clearly shows first of all how the conditioning process dramatically reduces the torque of the mixer. Moreover, and this is the key of this research, the graph shows that when increasing the pressure, and so the confinement of the conditioned mass, the torque is increasing, which indicates the increase of stiffness and shear strength, which reduces the effectiveness of the counterpressure application process. This phenomenon is more evident in Soil A, which is the most granular and permeable soil, as the amount of fine is lower than in Soil E. This is due to the fact that the bubbles are filling more the voids in case of more permeable masses, and thus the stiffness can change much more than in case of finer masses.

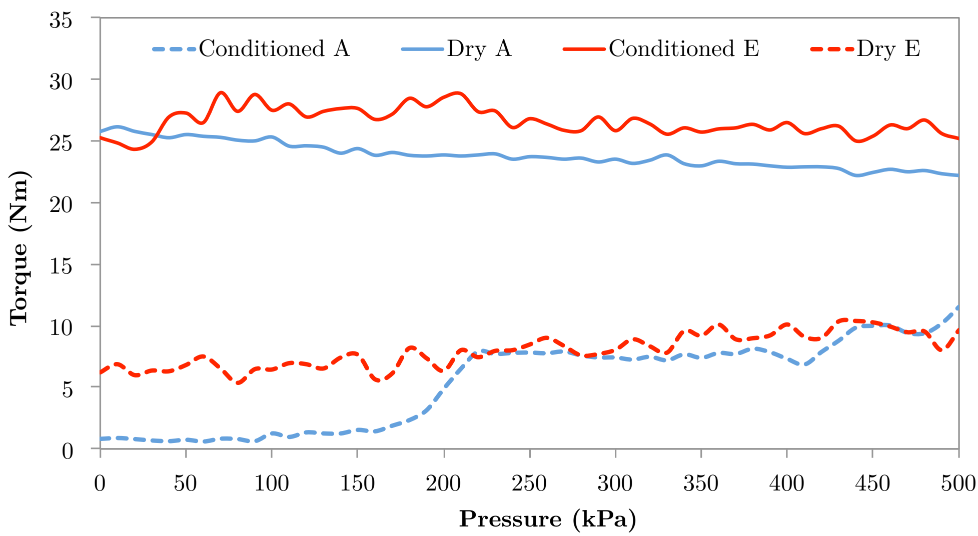


Figure 7.87: Mechanical behaviour comparison between dry and conditioned Soil A and Soil E in the pressurized rotating mixer.

Chapter 8

Conclusions

The goal of this work was supposed to be the assessment of the mechanical behaviour of different conditioned soils, in order to be used in a suitable and effective way for the excavation with an EPBS machine.

The research underlined the necessity of studying the conditioned mass by considering its pressure condition, which influences and in some cases radically changes the behaviour of the mass itself during an EPBS excavation. The use of this new philosophy of testing, through methods which are considering the pressure conditions and which prevent the leak of the conditioning agents (i.e. foam, water), allowed to study the conditioned soils in a different way.

The tests carried out enable us to assess some important considerations related to the effectiveness of the conditioning for EPB shield tunnelling. In particular the results of the research brought to define some important aspects:

- common testing methods usually used for assessing the conditioned soils, especially the slump test, is still necessary to assess the conditioning sets in a preliminary way. The slump test is fast and allows to check the overall behaviour of the conditioned mass, and it can be used also to verify the muck coming out from the excavating chamber through the screw conveyor;

- the use of experimental apparatuses as the screw conveyor can permit to the contractor to verify the conditioning sets assessed by using the slump test, by giving important indications like the homogeneity of the conditioned soil during the all test (stability of the mass). To some extent this test can also give indications about the fluidity of the tested soil, by verifying the difference of pressure at the top and the bottom of the tank. Moreover this device is the first important one which allows pressurized and partial undrained testing;

- in a conditioned mass a low shear strength is requested in order to allow an effective and suitable transmission of the pressure in a pseudo-hydrostatic way. The resulting material should be a pseudo-fluid with a certain consistency which eases the removal of the material itself when performing the mucking through the screw conveyor first and the belt conveyor after;

- the study of the conditioned mass must be carried out in particular undrained conditions, which usually do not coincide with the geotechnical one. This is mainly due to the fact that the conditioned sample is not saturated with water, it is usually in a condition close to the saturation but most of the pores are filled with foam bubbles. In this scenario it is thus clear that compared to a sample saturated with water, the conditioned sample is compressible, therefore the constant volume condition is not fulfilled. The saturation of the conditioned sample with water and foam is crucial in order to transmit effectively the pressure. If this is not happening, the material once compressed does not immediately transfer the pressure in all the directions, as the fluids are first absorbed by the drier mass. Thus the condition we are considering for testing the conditioned samples is just partially equal to the undrained condition used in geotechnics; in the samples studied in this research the medium is compressible and the pore pressures develop from a mixture of water, foaming agent and mostly air;

- pressure is giving a crucial contribution to the study of the conditioned soils, as in EPB tunnelling the material is usually conditioned under a certain stress condition, which is not zero, therefore studies on this material should be performed under particular pressure conditions, like in the excavating chamber in operation. For considering this particular and relevant aspect, new test procedures are requested in order to study the material at different pressure confinements: in conditioned soils at room pressure the grains are usually not in contact because the voids are filled with the bubbles which are reducing the shear strength of the mass; on the contrary with higher pressure the high compressibility of the bubbles compared to the grains of soil allows the contact of the grains themselves which causes an increase of the shear strength of the mass;

- the use of a Casagrande shear box modified in order to prevent the flow and leakage of liquid generator from the conditioned mass is a good tool to estimate the overall behaviour of the mass at low confinement, even though a clear shear surface is not present. This tool can be used to assess the reduction of mechanical strength from the original dry or wet sample to the conditioned one. The results of this test showed that, despite the applicable normal pressure low compared to the EPBS applications, the samples once conditioned have a reduced shear strength and thus the soils seem to be suitable for EPBS applications. The graphs shows that in undrained conditions, without saturating the samples with water but just using the conditioned mass, the friction angle is close to zero and the apparent cohesion really low;

- the use of a modified large triaxial is giving really good results for estimating several characteristics of the conditioned masses at different pressures and in undrained conditions. The use of a large cell allows to verify the suitability of the material for EPBS applications also because after the tests and after the application of different confinement pressures, it is possible to collect enough material to carry out slump tests. In this way it is possible to compare the suitability of the mass after being strained at a certain stage and at room pressure, which is the usual condition studied in standard testing. The results of this test underlined and confirmed that, below a certain limit of pressure confinement, the conditioned mass acts as a fluid, therefore would transmit effectively the pressure, while above this limit the material starts to loose the fluid effectiveness;

- the design of a new pressurized rotating mixer allowed to verify the mechanical behaviour of the soil at different states (dry and conditioned) and at different pressures. The results of this test pointed out that, despite the triaxial testing showed that at a certain limit the conditioned material is not behaving anymore as a fluid, the conditioned mass maintains a low shear strength even though is not totally fluid. This result is crucial for the research, as it means that a good conditioning keeps the material suitable also at higher pressures.

As a final consideration the results of this research confirms that this new trend of studying conditioned soils is extremely valid and needs to be developed more and more, as also confirmed by the studies carried out at Colorado School of Mines (Mori et al., 2015, Mooney et al., 2016) for the pressure related issues. An important development on the detailed characterization of conditioned soil is the study of the rheology of the mass, as confirmed by the Ruhr University Bochum research (Galli and Thewes, 2016). This topic, combined with the mechanical characterization, would allow to answer many issues which are still unsolved.

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