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# Consistency Index and Its Correlation with EPB Excavation of Mixed Clay–Sand Soils

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**Abstract** The behavioural properties of excavated ground have significant influence on the excavation process performed by an Earth Pressure Balance Machine (EPBM), as they are among the main factors responsible for maintaining the pressure ahead of the face, which affects face stability. Therefore, understanding the characteristics of the excavated material along with its flow behaviour is essential for a successful EPB tunnel drive. In scenarios involving the excavation of fine-grained soils containing clay minerals, the consistency index has been widely used as a guideline to define the ideal state of the excavated material. However, there are certain restrictions for the use of this index, the first of which are the Atterberg limits. These limits become more restrictive when mixed soils are involved. This study presents a brief review of the application of the consistency index and Atterberg limits in order to predict the performance of an EPB excavation. This study presents the results of a laboratory testing campaign with artificially mixed clay–sand soils by using a flow table as a preliminary flow assessment of cohesive soils.

**Keywords** Earth Pressure Balance Machine · EPB soil conditioning · Mixed sand and clay soils · Atterberg limits · Consistency index · Flow table

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## 1 Introduction

The Earth Pressure Balance Machine (EPBM) was first commissioned by the Japanese in the early 1970s (Maidl et al. 2012) and is currently the most frequently applied tunnel boring machine in soft ground (Herrenknecht et al. 2011). An EPBM is a closed shield used for the excavation of soft ground where face support and groundwater pressure control is obtained by means of the material excavated by the cutting wheel, which serves as a support medium itself. To obtain an ideal paste of excavated material, it may be necessary to add water or other additives (foam, polymers, and slurries with fines) to bring the mixture, also called "muck", to a satisfactory workability and permeability. This muck should maintain the required face pressure, as well as other excavation requirements, such as its extraction and transport. This process is called soil conditioning, and it is considered to be one of the most important parts of

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the EPB excavation process (Maidl 1995; Langmaack 2000; EFNARC 2005; Thewes et al. 2012).

In cases involving soils with fine particles, including clay minerals,[1] muck workability is addressed in terms of consistency through a correlation between the plasticity index and the water content. Maidl (1995) stated that in an EPB excavation, whenever face support is required (unstable ground; groundwater occurrence), the consistency of the soil should range from pasty to soft (consistency should be between 0.4 and 0.75), and the soil should feature a low permeability (Herrenknecht et al. 2011; Galli and Thewes 2014; Galli 2016).

In terms of an EPB excavation, concerns related to the state of the muck go not only to the excavation process, but also to its removal and disposal. Liquid muck would produce complications regarding its transportation along conveyor belts or muck cars, which would cause a significant increase in the extracted volume and later disposal (Maidl et al. 2012; Herrenknecht et al. 2011). Consequently, the "ideal" characteristics of this excavated and conditioned material is a balance between all the parts that this element plays in the EPB operation: excavation, support medium, removal, and later disposal.

Several authors (Casagrande 1958; Sivapullaiah and Sridharan 1985; Nagaraj et al. 2012; Claveau-Mallet et al. 2012; Mishra et al. 2012; Haigh et al. 2013; Haigh 2016) mentioned the numerous limitations of the Atterberg limits. Considering that the consistency index is based on these limits, their limitations would certainly have an influence on the delimitation of the consistencies of the "ideal muck", or any other parameter that would consider the Atterberg limits. Those limitations are mentioned in this study, in addition to a description of how this standard method would not always provide a realistic consistency index.

This is accomplished by providing several examples of laboratory tests with artificially mixed clay–sand soils with three different clay minerals and different clay–sand proportions. The standard methodology of the Atterberg limits was modified including bigger grains, always comparing with the standardized results, proving that by changing the methodology more reasonable results could be obtained, which would represent closely the real characteristics of a mixed soil.

In addition, the flow of mixed clay–sand soils was analysed using a flow table originally designed for laboratory tests on concrete. This method of testing soil samples with soil conditioning agents was already proposed in EFNARC (2005), mainly to test the plasticising effect of certain conditions for non-cohesive soil. Here, the method has been modified to better suit cohesive soils. Several conclusions could be drawn concerning the flow behaviour of the tested soils. Mainly showing that cohesive soils holding the exact same consistency index, do not necessarily have similar flow behaviour.

## 2 Theoretical Considerations

In the first decade of the twentieth century, Atterberg determined the amount of water content at which soil would change from a liquid to a plastic condition (liquid limit—$W_L$), as well as from a plastic condition to a solid condition (plastic limit—$W_P$) (Atterberg 1911), by studying different soil states. Later, Terzaghi[a] (1926) realized that understanding this soil property was essential to understanding the overall behaviour of the soil, and that a more efficient method should be developed to allow for reproducibility by other operators. Subsequently, Casagrande (1932) designed a more reliable method to define the plastic and liquid limits.

Casagrande (1958), after realizing issues with the standardization of his method around the world, suggested a standard specification for the Casagrande cup and grooving tool to measure the liquid limit, which is still used today and standardised by the *ASTM D4318-17* (2017). Casagrande (1958) advised that his method for obtaining the liquid limit should be replaced by a more rational test that is based on the shear strength of the material. In some countries, this sort of test has already been implemented—it is known as the fall cone test, and it is used as an alternative method to attain the liquid limit (Houlsby 1982). However, Casagrande´s method is still the most commonly applied test to obtain the Atterberg limits

[1] When working with fine-grained soils, it is essential to correctly differentiate between clay as a clay grain size and clay as a clay mineral. For additional detail, see Bergaya and Lagaly (2013), Baille (2014) and several publications of the CMS—Clay Mineral Society (i.e. Grim 1952; Rosenqvist 1960; Brindley 1966; CSM-Clay Minerals Society 1991; Guggenheim and Martin 1995).

of soils, even after several attempts have been made to replace it (Sivapullaiah and Sridharan 1985; Mishra et al. 2012; Nagaraj et al. 2012; Haigh 2012, 2016).

Many authors have investigated the application of this method and pointed out its limitations for determining liquid and plastic limits; several examples of these limitations are listed as follows.

### 2.1 Liquid Limit

- The method has limited reproducibility for identical soil. There is a coefficient of variation (standard variation/mean) of up to 8% due to differences in the volume and mass of the clay placed in the cup, the grooving tool used, the hardness of the Casagrande base, and the fall height adjustment (Claveau-Mallet et al. 2012; Haigh 2012, 2016). Also, essential to mention, that even when presenting a good reproducibility, for same soils, variations in its coefficient may occur due to its natural variability.
- There are difficulties associated with utilizing this method within low-plasticity soils. The soil mass would rather slide rather than flow, which makes it difficult to cut a proper groove. (Mishra et al. 2012; Sivapullaiah and Sridharan 1985).
- Different results are obtained depending on the type of the standardised base used, either soft or hard, according to different specifications, which depends on the national design standard (British Standard, ASTM, French code) (Haigh 2016).
- The above result does not correlate with any change in the soil behaviour, and the results are more dependent on the device rather than on a physical characteristic of a certain soil state (Haigh 2016).
- The stiffness of the bench upon which the test is performed could induce different results (Casagrande 1958).

### 2.2 Plastic Limit

- The brittle failure that determines the plastic limit while rolling the thread is related to either air entry or cavitation in the clay when the water ceases to behave as a continuum and capillary suction starts to prevail; therefore, the point which this failure occurs is not related to a constant strength, as initially considered (Haigh et al. 2013);
- The test relies on the judgment of the operator, which would imply that different results could be obtained when different operators are involved (Sherwood 1970; Nagaraj et al. 2012; Haigh et al. 2013).

In addition to the aforementioned issues, it is important to note that these limits are traditionally defined only for fine-grained soils, the threshold of which is soil that can pass through a sieve number of 40. This means that only the portion of the soil containing grains below 0.425 mm (fine-sand) would be considered in the test. In the case of mixed soils, for a typical residual ground sample with a well-graded distribution, medium and coarse grains should be added to the evaluation. However, it is not well-defined how this should be accomplished.

There have been several attempts to define correlations concerning the influence of different proportions of clay fractions and clay minerals in the Atterberg limit values. Seed et al. (1964) verified that with clay content above 40%, there would be a linear relationship between the plasticity index and clay content. This was true for studies on Kaolinite, Illite, and Montmorillonite. However, this was not always the case for scenarios involving lower clay content soil, and the liquid limit would be mainly influenced by its clay content (as clay grain size—grains smaller than 0.002 mm) and by the clay mineral.

After detailed investigations on Bentonite and Kaolinite mixed with different sizes and shapes of sand, Sivapullaiah & Sridharan (1985) concluded that the shape of the sand would not influence the liquid limit; however, this was not true for its grain size. They also suggested a procedure to obtain the liquid limit for low-plasticity soils by mixing them with soils of higher plasticity and assuming a linear relationship.

Polidori (2007) found a correlation between different clay fractions that applied mainly to platey clays and excluded non-platey clays (such as Halloysite, Allophane, and Attapulgite mixed with sand) and its resulting limits, which, except for the materials with low clay content, would follow a linear relationship. A relationship between the plastic limit ($W_P$), liquid limit ($W_L$), and the clay fraction (CF) was defined by an empirical equation (Eq. 1):

$$W_{P(\%)} = 0.04W_{L(\%)} + \left(0.26CF_{(\%)} + 10\right) \quad (1)$$

After an extended literature review, Nagaraj et al. (2012) concluded that even though the limits defined by Atterberg (1911) are essential for understanding the behaviour of fine-grained soils, it is not feasible to expect similar correlations for the undrained shear strength of different soil in a same state, as initially presumed by Casagrande (1932, 1958).

The consistency index ($I_C$) is derived from the Atterberg limits and indicates the firmness of soil and the changes in water content that allow it to vary from the following states: liquid, very soft, soft, stiff, very stiff, and hard (Terzaghi[b]1926). At a consistency index of zero (0), soil is equivalent to its liquid limit, and at a consistency index of one (1), soil is equivalent to the plastic limit. The consistency index can be calculated with the following equation (Eq. 2):

$$Ic = W_{L(\%)} - W_{(\%)}/W_{L(\%)} - W_{P(\%)} \quad (2)$$

where $W_L$ refers to the liquid limit, W refers to the water content, and $W_P$ refers to the plastic limit.

In the case of EPB excavation and soil conditioning, the consistency index of both the soil and the conditioned material is an important parameter. This consistency parameter has been used to define the ideal state of a cohesive material to function as a suitable support medium. The consistency index is also applied as one of the main parameters for the evaluation of clogging, which occurs when clayey soils stick to the metallic parts of the machine (Thewes 1999, 2004; Hollmann and Thewes 2012, 2013).

Considering the consistency index of a soil along with its water content, Hollman and Thewes (2012) defined an empirical evaluation for the occurrence of clogging, which led to the creation of a clogging chart, which is shown in Fig. 1.

The green double arrow indicates the ideal consistency range of the support medium of the material inside the excavation chamber of an EPB, as defined by Maidl (1995), and the brown double arrow indicates the partial area that would be less vulnerable to clogging. Therefore, both the Atterberg limits and the consistency index are substantial parameters for the EPB excavation and soil conditioning. If there are limitations to the Atterberg limits, they are going to be extended to the consistency index.

## 3 Methodology

There are two methodologies here described, one is a modification of the original Atterberg limits methodology, and another, the flow table applied to cohesive soils. As mentioned in the introduction, the modified Atterberg was conducted to confirm initial assumptions that for a mixed soil, leaving out the grains bigger than 0.425 mm, could be providing an unrealistic characterization of the material. Surely, a size limitation must be defined, as it would not be feasible to conduct the tests from a certain grain size, for example, with gravels and cobbles, as it will be demonstrated.

The tests with the flow table were conducted to investigate the flow of different soils, with different clay fractions, for same consistencies, in order to understand the variance in the flow, which could be, in the future, related to the flow of soils in the EPB excavation, especially along the transportation of the soil in the screw conveyor and conveyor belt.

All tested soils were assembled in the laboratory by mixing quartz sand and three different clay mixtures. A total of 71 soil mixtures were reconstituted for this study, and they contain different grain sizes and proportions of sand with Bentonite and Kaolinite clays. A clay mixture called Friedland, which contains Montmorillonite and Illite, along with other minerals as specified in "Appendix", are included in these 71 soil mixtures. For all of the mixtures, the Atterberg limits were obtained and 153 tests on the flow table were performed. Details for all of the mixtures are included in "Appendix", including their Atterberg limits and the measured and calculated plastic limits. The calculated limits were obtained by using an equation from Polidori (2007) for comparison purposes.

The samples were assembled using six different grades of quartz sand, as shown in Fig. 2. All of the samples containing grains that were coarser than that recommended by the Atterberg limits standard, *ASTM D4318-17 (2017)*, even when retained in the 40-mesh sieve (medium and coarse sand, grains bigger than 0.425 mm), therefore, the whole material, were included in the tests. Aside from this modification, all of the remaining Atterberg limit testing followed the recommendations of *ASTM D4318-17 (2017).* The method used to determine the liquid limit was multipoint method A, where 3–4 different points were obtained with the Casagrande cup.

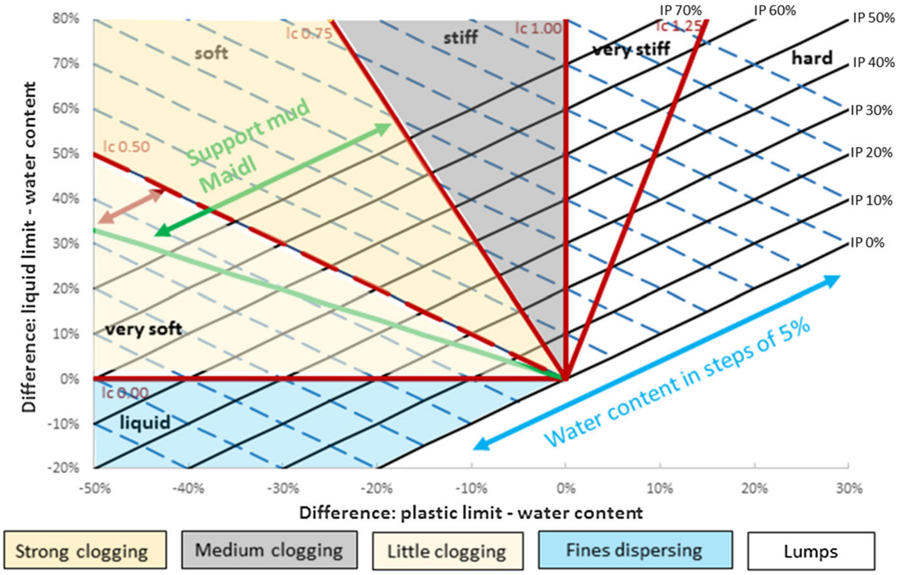
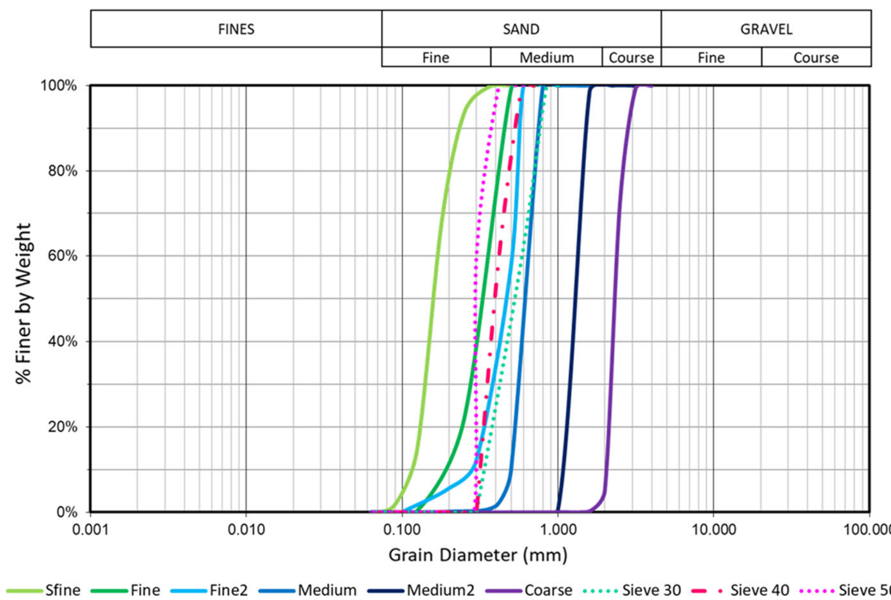



**Fig. 1** Evaluation diagram for clogging potential, taking into consideration the consistency index of the soil and changes in the water content. Reproduced with permission from Hollmann and Thewes (2012)

**Fig. 2** Grain size distribution of all six different sand sizes used in most of the tests




The flow table, also known as the Hagerman flow table, was originally introduced in Germany in the 1930s, and is a test that the concrete industry uses to analyse the flow of concrete. It involves measuring the spread of material after it is subjected to jolting (Tattersall 1991). It is standardised by *ASTM C230/C230 M-14* (2014) and by *ASTM C1437-15* (2015).

EFNARC (2005) recommended the use of a flow table to test the plasticising effect of foam agents (with or without polymers) on sandy samples. According to the specifications, it is suggested that the test was to be first performed with water followed by soil conditioning additives. The table should be jolted 15 times and the flow of the sample will be measured by comparing the initial diameter with the final diameter, after the dropping stage is completed.

The original procedure was modified for the tests with clay–sand soils. The suggestion of dropping only

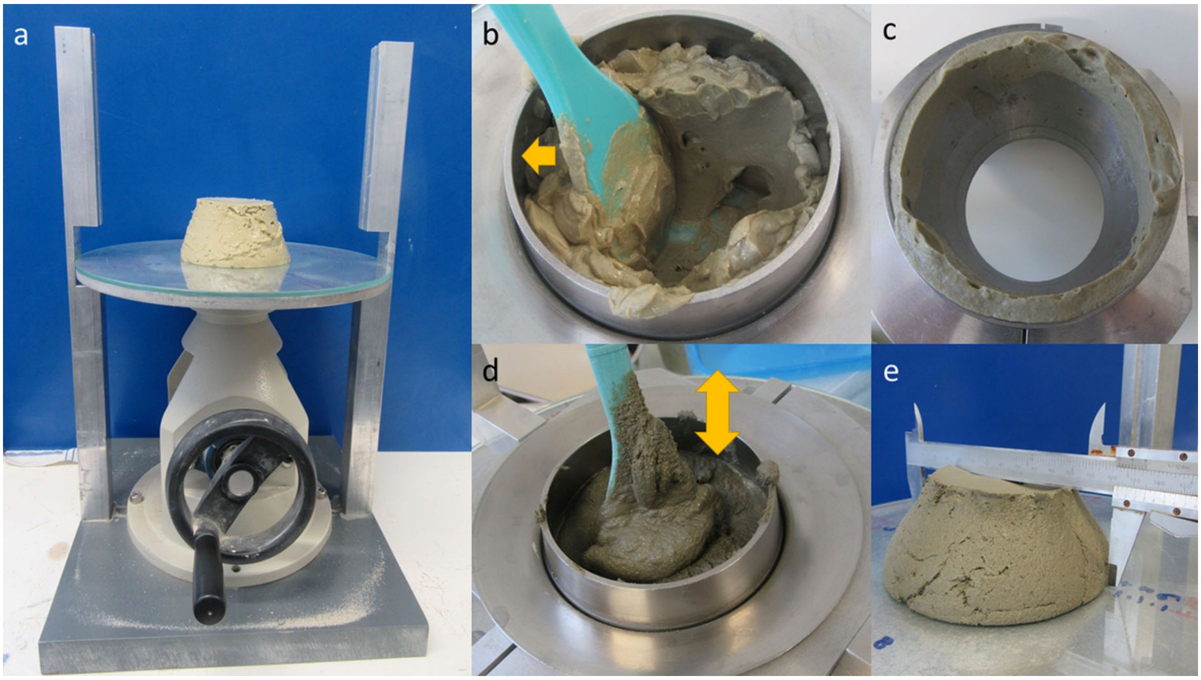



**Fig. 3** Flow table setup for tests with mixed clay–sand soils: **a** general setting with one sample; **b** initially filling the metal cone, pressing the material against the corners, and avoiding bubbles of air inside the specimen; **c** without coating the cone with oil, some soil can remain in the frame if it is sticky; **d** shaking the soil with up-and-down movements to ensure full distribution inside the cone; **e** measuring the specimen with a caliper after jolting

15 times (EFNARC 2005) was not enough to establish a comparison between the samples containing clay minerals and the tested consistency indexes. Besides the number of the drops, the EFNARC (2005) flow table testing methodology was conducted mainly with conditioned sand. The complete modified methodology is described below in detail.

Figure 3 illustrates the setting and details for the testing of clay–sand mixed soils on the flow table. All of the tests were conducted on a glass surface that was fitted to a metal plate, which helped with the cleanup after each test (Fig. 3a). Each soil sample was prepared at least 24 h before the intended test, and a high-speed mixer was used to ensure there was uniform mixing of the sample, especially in terms of moisture homogeneity. The metal cone was initially greased with a fine coat of vegetable oil (olive oil) to prevent the sample from sticking in the cone. This test could also be conducted without the oil coating to have an initial evaluation of the clogging potential by observing the amount of soil that became stuck in the cone after lifting it (Fig. 3c).

It was essential to make sure that there were no air bubbles in the soil while placing the material inside the cone. To accomplish that, the soil was first placed in the corners of the cone by applying pressure (Fig. 3b). After half of the cone was filled, up-and-down movements were repeatedly performed using with a spoon (around 20 times) to spread all the soil inside the cone. The remainder of the casing was covered by repeating the up-and-down movements with a spoon to ensure uniform filling, and excess material was removed with a spatula. Then, the cone was lifted and the initial diameter was measured with a caliper. The value obtained is $m_0$.[2]

The table was jolted 40 times and the diameter was measured three times to obtain an average (Fig. 3e), which was used to obtain the $m_{40}$ parameter. Initially, the test was continued and measured again after 20 more drops were added, making 60 drops in total. This allowed us to obtain a third parameter, $m_{60}$. The

[2] If the soil is completely filling the cone on its base, then it is possible to consider the inner diameter of the cone as $m_0$, which in this case was 100 mm.

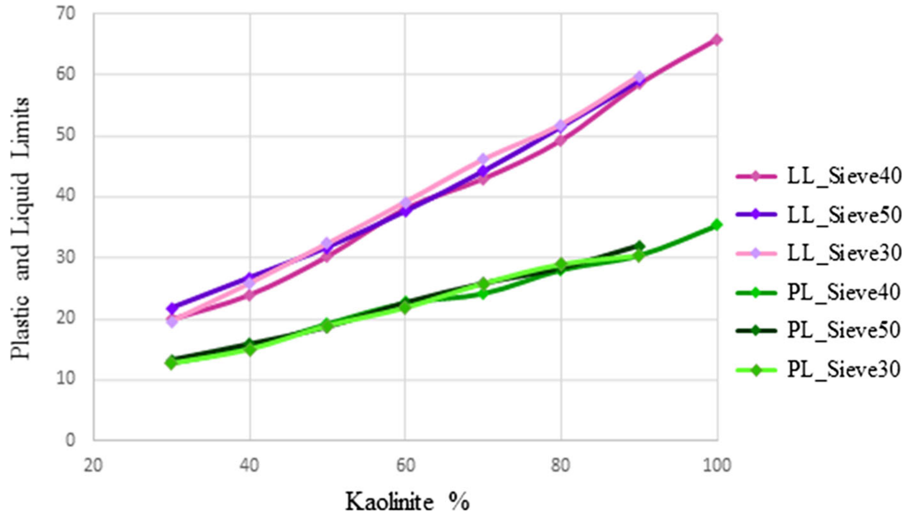



**Fig. 4** Liquid and plastic limits for three different grain sizes of sand mixed with Kaolinite

prospect of dropping the table 60 times seemed to create a test that would be too long, and 40 jolts were already considered sufficient to allow for a comparison to be made comparison between different samples.

The following equation was used obtain the results for the flow (Eq. 3):

$$Flow_{40} = ((m_{40} - m_0)/m_0) * 100 \quad (3)$$

where $m_0$ refers to the average of three measurements taken for the initial specimen diameter, and $m_{40}$ refers to the average of three measurements of the specimen diameter taken after 40 jolts.

The same equation can be used for $flow_{60}$, where $m_{40}$ will be replaced by $m_{60}$.

All the samples were weighted while wet and after being dried in an oven for at least 48 h at 100 °C. The bulk and dry densities were calculated after measuring the sample volume using the metal cone, as well as the void ratio and saturation degree. The water content of each test was checked using the standard of moisture content determination from *ASTM D2216-10* (2010), and the consistency index of each sample was calculated. At least two tests for each consistency index of each sample mixture were obtained.

## 4 Test Results and Discussions

The results of all the tests are presented and discussed in this section.

### 4.1 Mixed Clay–Sand Soils: Atterberg Limits

Figure 4 shows the results for the Atterberg limit tests for samples mixed with kaolinite and sands with sieve values of 30, 40, and 50.

It was not possible to observe significant differences in the results between these particular grain size ranges (with minor variations); however, the results did follow a clear linear trend, even for clay contents as low as 30%. It was possible to successfully proceed with the liquid and plastic limit methodologies when including grain sizes slightly above the threshold defined by the Atterberg limit standardization (material passing through a 20-mesh sieve and retained at a 30- or 40-mesh sieve).

However, the limitations of those tests with bigger grains than the standard threshold became clear when the tests were performed with other samples. Figure 5 shows the Atterberg limits obtained for Bentonite–sand mixtures, and Fig. 6 shows that for the Friedland clay–sand samples. The Casagrande cup and the thread rolling methods became more difficult to perform as the grain size increased and the clay percentage decreased. The main issues with determining the liquid limit were to properly cut the grove and, with bigger grains, it was clear that the soil was sliding rather than flowing.

The main limitation for the plastic limit was to roll the thread with coarser grains, even with high clay content. Many times, it was noted that the thread was crumbing due to the size of a sand grain, and not

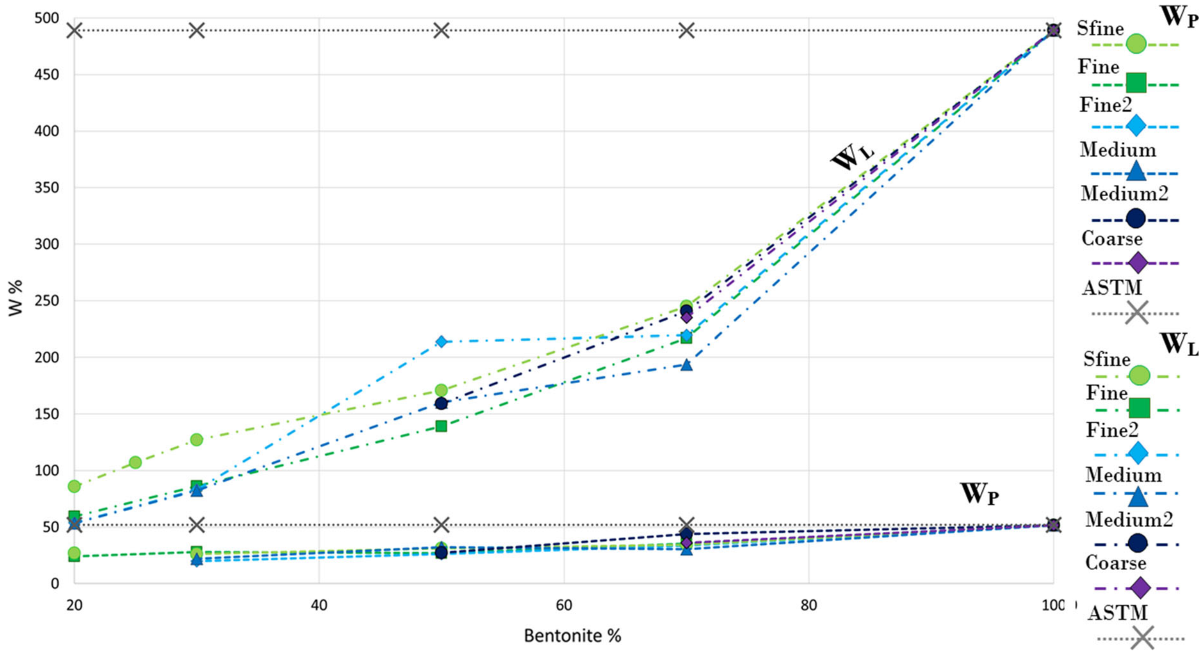



**Fig. 5** Atterberg limit values for mixed Bentonite clay–sand soils ($W_L$—liquid limit; $W_P$—plastic limit). The grey dashed lines represent the plastic limits based on the ASTM standard specifications, and only take into consideration the portion of soil with grains that passed through the 40-mesh sieve; therefore, a pure Bentonite sample

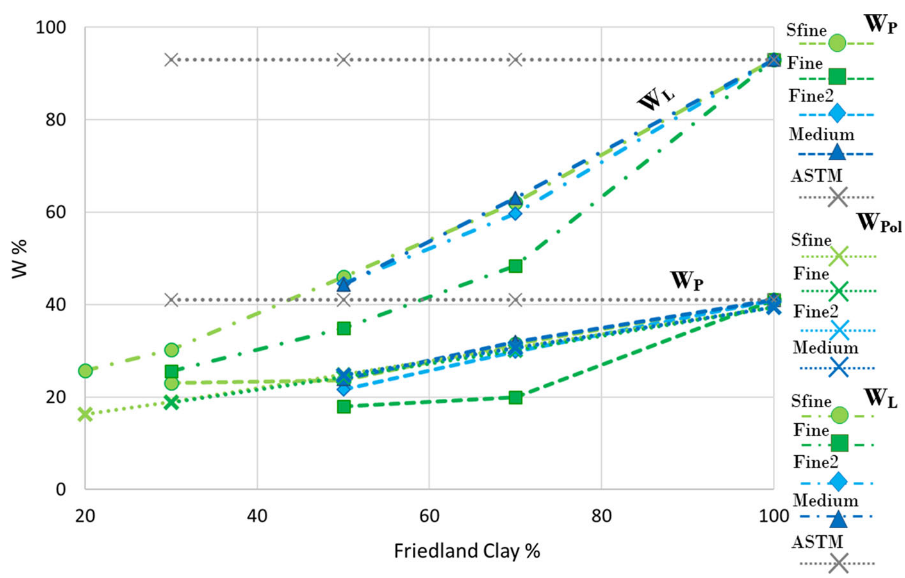



**Fig. 6** Atterberg limit values for mixed Friedland clay–sand soils ($W_L$—liquid limit; $W_P$—plastic limit). The dashed grey lines represent the limits considering the ASTM specifications (therefore, only considering the Bentonite portion), and the dashed coloured lines represent the plastic limit values ($W_{POL}$) calculated with the equation provided by Polidori (2007)

necessarily because it achieved the plastic limit point, providing an unrealistic value.

The limitation of the plastic limit started with 2 mm grains (limit between medium and coarse), approximately, because the samples with coarse sand (even with a high clay content of 70%) could not provide reliable results. It would be safe to assume that the plastic limit could be easily performed for grain sizes up to 1 mm in size. The liquid limit could be extended to grains up to around 2 mm; Fig. 7 illustrates those above-mentioned limitations.


**Fig. 7** Physical limitations of performing the plastic (**a**) and the liquid (**b**) limits on samples with coarser grains than the assumed threshold of the method (grains smaller than 0.425 mm). The bigger grains in the thread would induce its cracking and these grains would make it hard to cut a grove with the proper tool

In Figs. 5 and 6, looking only for the “sfine” and “fine” lines for the liquid limit, it is possible to affirm that the water content needed to achieve a liquid limit is lower for coarser grains, but that does not seem to be the case for other samples. Nevertheless, even with all the limitations, it was still possible to attain a linear relationship, as already mentioned by other authors. By modifying the rule of the 40-mesh sieve of the standard *ASTM D4318-17* (2017), it was possible to obtain more realistic values for the Atterberg limits.

In the same figures, the grey dashed line represents the limits for the pure clay samples. In the case of this clay mixed with a medium-grain sand; for instance, if only the fine particles are considered (clay component), the $W_P$ and $W_L$ values would be much higher than the modified ones for any clay content below 100%. Consequently, this would directly affect the values of the consistency index, as shown in Fig. 8, where the left sample (a) was mixed considering the Atterberg limits as defined by the standards. This lead to an extremely liquid mixture for an $I_C$ of zero (liquid limit). The sample on the right (b) was used with the modified version of the standard, including coarser grains. Visually, it is evident that the modified consistency values are a better fit than the values that were obtained by following the standard methodology.

The measured results were also compared to the calculated values by considering the equation defined by Polidori (2007) for the plastic limit. This allowed us to obtain results that were consistent. Figure 9 shows these comparative results for the Bentonite clay–sand samples. These can be compared with Fig. 6 for the Friedland clay–sand results.

For the Friedland clay, two samples, “sfine” and “medium”, show very good correlations, even for low clay content, but for the “fine” sample, the correlations are not as close for the measured and calculated plastic limits. For the Bentonite clay, the correlations seem better at above 40% clay content, with exception of the “medium” and “medium2” samples, whereas at 70% clay content, the measured values seem to be quite far from the calculated limits. This may be emblematic of the difficulties associated with rolling the thread with bigger grains.

### 4.2 Flow Table

Several tests (minimum of three repeated tests) with different samples, each at three different consistency index values, were conducted using the flow table. Initially, this was conducted with the intention of searching for a replacement for the liquid limit test, assuming that a sample with similar consistency would flow in the same way. However, this did not occur. Nevertheless, it was possible to achieve certain insights concerning the flow of those mixtures, which could be most likely correlated with the flow inside the

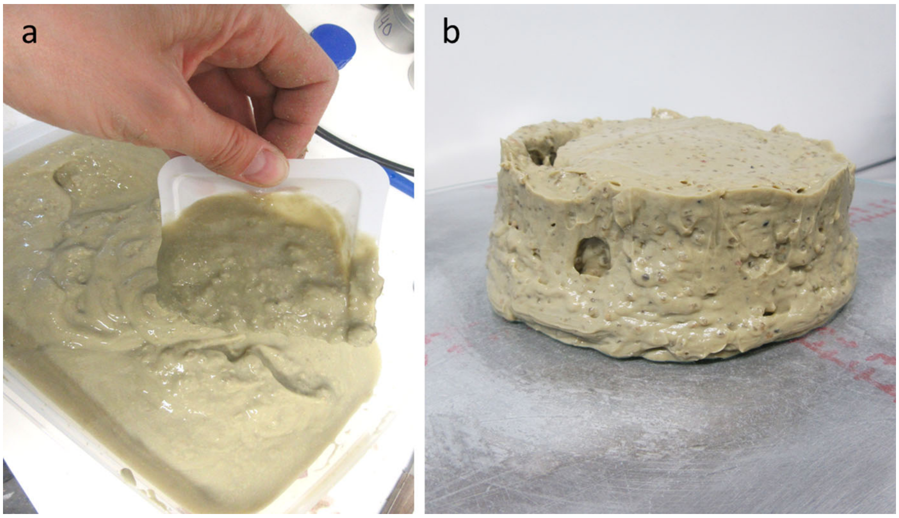
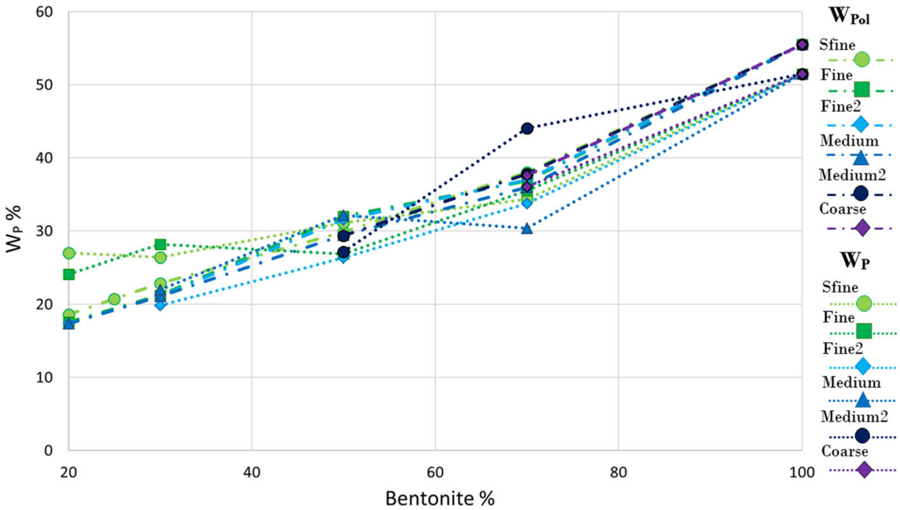



**Fig. 8** Two mixed clay–sand samples expected to have the following value: $I_C = 0$: **a** following the standard and not including the grain sizes bigger than 0.425 mm; **b** including all the grain sizes

**Fig. 9** Detail of the plastic limit values measured (dotted lines) and calculated (dashed lines) for the Bentonite clay–sand samples




excavation chamber and screw conveyor of an EPB machine.

Figure 10 exemplifies a test with three different clay mineral/mixtures (Bentonite, Kaolinite and Friedland), for a consistency index of zero, therefore, at their liquid limit state, and for different proportions of very fine sand ("sfine", see Fig. 2 for grain size distribution). For this $I_{C,}$ jolting was conducted 40 times ($Flow_{40}$) and 60 times ($Flow_{60}$), and it can be observed that they resulted in very similar flow. The average value is represented by the circle (Kaolinite), diamond (Friedland), and triangle (Bentonite), including the standard deviation values.

The different behaviour observed between the different clay mineral mixtures is evident in Fig. 11. For the Friedland samples, the flow increases almost linearly as the clay content increases. There is a slight downward curve from 30 to 50% of clay content for that mixture, but for the Bentonite and Kaolinite mixtures, this downward curve is quite pronounced. Initially, at low clay content, the flow decreases with the increase in clay content. The flow is at its maximum and turning point at around 65% for the Bentonite, and at 87% for the Kaolinite. After these turning points, the flow increases with the increase in clay content, which is more perceptible for the

**Fig. 10** $Flow_{40}$ and $Flow_{60}$ for different clay proportions, for three different clays and $I_C$ = 0.0

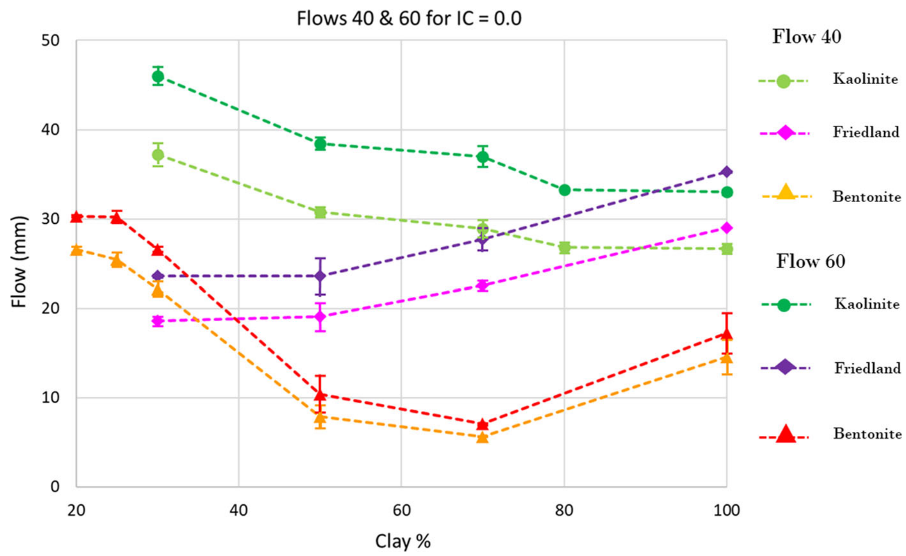
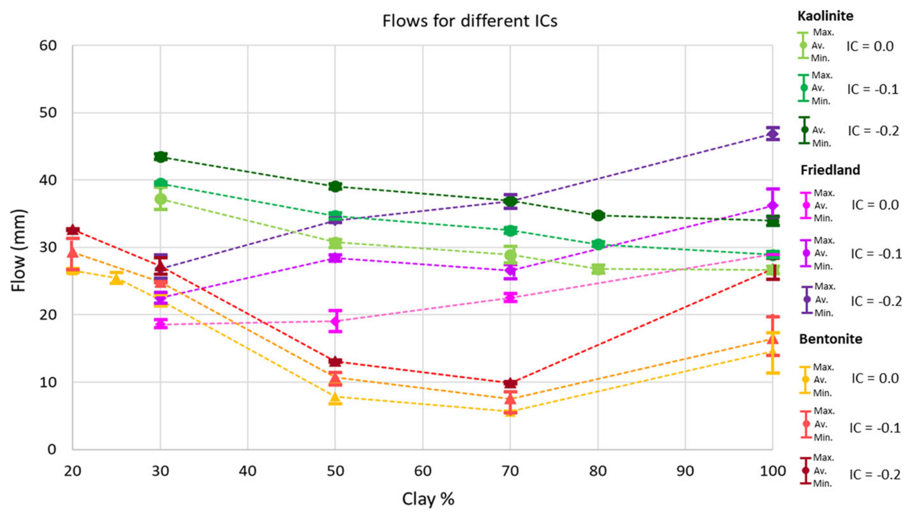



**Fig. 11** $Flow_{40}$ for different clay–sand mixtures for three different $I_C$: 0.0, − 0.1, and − 0.2




Bentonite clay. It should be taken into consideration that the Friedland mixture is a not a pure clay mineral, which could be affecting the results.

Figure 11 also shows other consistency indexes (− 0.1 and − 0.2), and it is possible to affirm that lower consistencies would imply higher flows, as the mixture is more liquid-like and flows easily; this is logically consistent. Figures 10 and 11 can be compared with Fig. 12, which shows the physical indexes of those samples at a consistency index of 0.0—their liquid limit value. In terms of densities (ρ—bulk; ρd—dry), both values decrease as the clay content increases. However, the void ratio (e) and the saturation degree (S) values increase as the clay fraction increases.

Finally, Fig. 13 proposes a schematic model for the clay–sand interaction and its flow, on a grain-size level, before and after jolting. The top of the figure shows the loose state before any compaction or impact, and the bottom of the figure shows the compacted state after jolting procedure.

For low clay content, the sand grains touch each other, and the clay flakes only fill the voids between the sand grains. The clay flakes would have no influence in the compaction with the jolting stage, happening only the readjustment of the sand grains into a more compacted packing of grains. With the

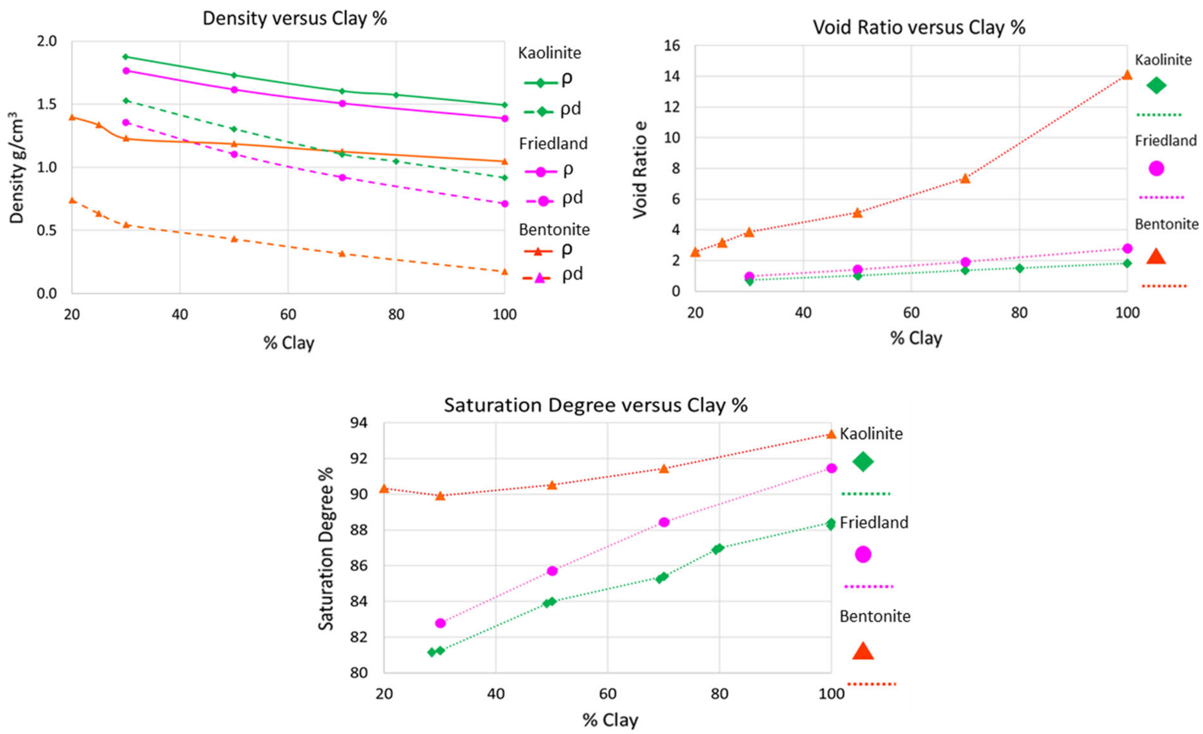
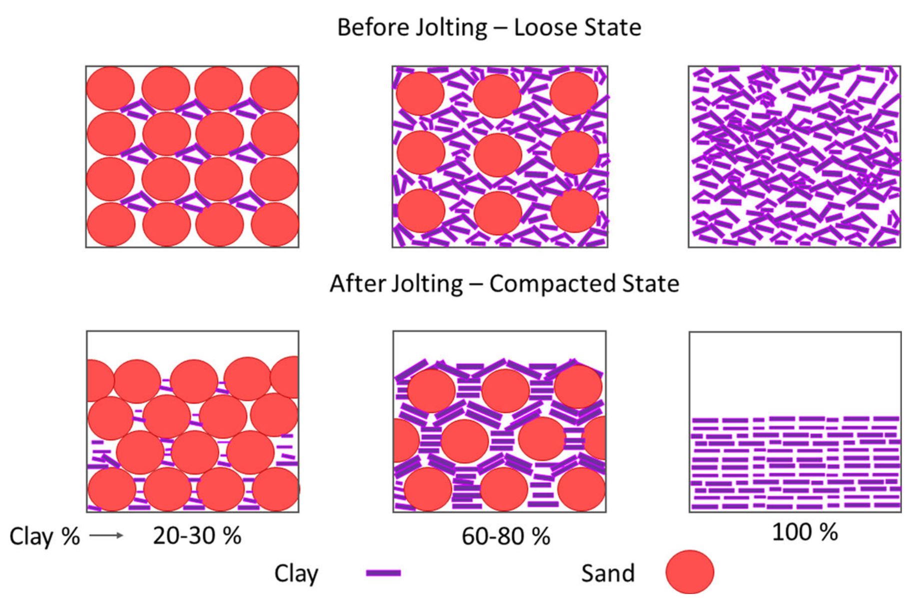



**Fig. 12** Physical indexes of the samples with IC = 0.0, with bulk (ρ) and dry (ρd) densities, void ratio (e), and saturation degree (S)

**Fig. 13** Proposed simplified arrangement for the microstructure of soils with different percentages of clay and sand content, before and after jolting




increase in clay content, the sand grains are "floating" in the clay matrix, and after the jolting impact the clay flakes will not allow as much rearrangement of the sand grains, when compared to the lower clay content scenario. This would result into a lower flow of the entire mixture, lower even than the case of pure clay samples. Therefore, there is a threshold of clay–sand content for the lowest flow possible, and this value will differ depending on the clay mineral.

## 5 Final Considerations

### 5.1 Tunnelling into Mixed Face: A Simple Case of Sand and Clay

Most of the research on EPB soil conditioning was conducted either for pure clay material (Mair et al. 2003; Merritt 2004; Spagnoli et al. 2011a, b; Zumsteg et al. 2012; Picchio and Boscaro 2013; Zumsteg 2014; Peila et al. 2016) or pure sand material (Vinai 2006; Borio 2010; Budach 2012; Galli 2016). Some questions concerning the soil conditioning and the behaviour of a mixed soil when excavated by an EPB machine lead into the following query: Is the current application of the Atterberg limit and the resulting consistency index sufficient to characterise those mixed soils? This question has resulted in testing with modified Atterberg limits, including tests with coarser grains.

This inquiry can be illustrated with a simple example (Fig. 14), which shows two different scenarios with distinct combinations between two materials: a medium-grain sand (in beige), with particle sizes above 425 μm, and pure clay (in green). In terms of the Atterberg limits, only the clay fraction would be considered for obtaining the plastic and liquid limit values and for providing same consistency index values.

Even though it is mentioned in the standard methodology (item 1.7; *ASTM D4318-17* 2017) that the coarser fraction of the soil should be considered to evaluate the properties of this material, there are no clear instructions on how that must be considered, or in which occasions this really must be considered. And, as it was shown so far, this lack of caution could lead into unrealistic property characterisation of the excavated material, which could influence the optimal consistency of the muck and the soil conditioning process. It is essential that this is regarded for each project, adjusting the standardized methodology, and even modifying it, to better fit the needs of the project design.

In terms of finding the ideal EPB consistency and classifying the soil from liquid to hard, it might be necessary to include larger grain sizes, or consider other alternatives for obtaining a consistency index that would be closer to the material reality. For mixed soils, we suggested that they be excavated with an EPB machine and that the Atterberg limits be obtained. However, these limits should be obtained in a way that deviates from the standard to include coarser grains, at least for the liquid limit, and that the equation defined by Polidori (2007) be used to calculate the plastic limit for platey clays.

A second alternative, especially for soil fractions bigger than 2 mm (coarse), would be to replace the coarser grains of the original soil with laboratory fine sand and calculate the Atterberg limits for it. The real value should be closer to this new modified value than to the one considering only the fine particles of the natural soil, especially if the errors already assumed for the Atterberg limits are taken into consideration. This procedure is similar to the one suggested by Sivapullaiah and Sridharan (1985) for low-plasticity soils but utilize laboratory fine sand to replace the coarser grains.

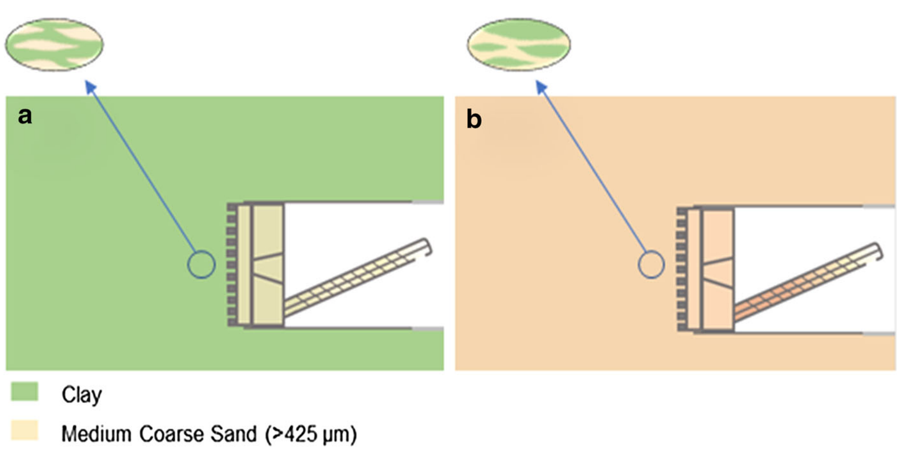



**Fig. 14** Comparison between two scenarios of interspersed lenses with same consistency index

These two alternatives are based on the fact that there is not much variation in the liquid and plastic limit values for different grain sizes, as seen in Figs. 4, 5 and 6, especially when there are so many limitations within these methods to obtain the liquid and plastic limits. They might not be completely precise, but they still provide a valuable evaluation of soil properties.

Mixed clay–sand soils are found in several geological settings, such as in sedimentary alluvial-fluvial deposits, or lagoon-beach environments, where there is a high frequency of clay–sand interspersed layers. Mixed clay–sand soils are also found in tropical residual grounds, where the soil is usually well-graded. There are areas containing high clay content, or other portions containing gravel, and all areas constantly change in size in the vertical and horizontal directions. Therefore, this subject is not a single scenario; it needs to be addressed and solutions should be proposed.

### 5.2 Flow Table as a Future Replacement for Slump Tests for Cohesive Soils

The flow table has been mainly used to analyse the flow of concrete mixtures, or, as suggested by EFNARC (2005), for sandy samples. The slump test, which was originally applied in the concrete industry, is already part of the current routine for soil-conditioning evaluation for EPB laboratory research and industry to evaluate the workability of the soil-conditioned soil (Vinai et al. 2008; Budach and Thewes 2015).

The main application for both tests so far has been low or non-cohesive soils. For instance, conducting slump tests with a certain percentage of clay might present a certain challenge in terms of the applying the mixing procedure needed to assure total homogeneity of the sample. In addition, its application to cohesive soils has not yet been investigated.

A slump test requires a high quantity of material, around 8–12 kgs, depending on the material composition. The first issue with that is the handling of the method itself, which requires bigger mixers to assure the homogenization of the sample, and a bigger foam generator (in the case of evaluating conditioned soils), then the higher volume of material to be disposed afterwards. The other problem is to provide such a quantity of material before the machine starts to excavate; for instance, to initially evaluate the efficiency of the soil conditioning (polymers, water, foam and so on). Usually, before the excavation, there is only borehole material available from the initial site investigation phase, which is not enough to run tests with the slump device.

The flow table; however, requires between 400 and 600 g of material only, which would be feasible to acquire from the borehole site investigation. Besides that, there are other advantages with handling procedure and disposal for a lower material quantity to be tested. This device also provides more dynamic flow information, which is not provided by a slump test, which is a more static evaluation (Galli 2016). During the jolting stage, it could be possible to obtain more refined mixture flow behaviour information, of the material conditioned or not. It could also, in future, be correlated with the flow inside the excavation chamber or screw conveyor of an EPB machine. This device could even be installed in tunnel boring machines to properly evaluate the muck while the excavation is taking place.

Undoubtedly, more tests should be conducted with this device. These additional tests could compare the results from slump tests with rheometer results to analyse the flow behaviour of soils and conditioned soils. In addition, the flow table device could offer an easier and more efficient solution to analyse the behaviour of conditioned materials for EPB excavation than the slump test, especially for cases of mixed soils with contribution of clay particles.

## 6 Conclusions

Tests with several artificially mixed clay–sand soils were conducted to provide conclusions regarding their Atterberg limits. These limits directly affect the consistency index values of those samples, which is necessary to characterise the material to be excavated by an EPBM. A new test routine was also proposed by using the flow table for mixed clay–sand soils, which allowed us to obtain the flow of the tested samples. Based on our experiments, we can draw the following conclusions:

- Atterberg limit methodology can produce errors and may not be very precise. Therefore, Atterberg methodology should be used with caution, especially in the case of mixed soils and, even more so for low-plasticity mixed soils.

- To obtain these Atterberg limits with a modified version of the standard for the Atterberg limits can be more reasonable in terms of characterizing the excavated material (or to be excavated) for an EPB operation, especially, to achieve a more realistic consistency index.
- This modification of the Atterberg limits can be performed by including the bigger grains, up to 2 mm; or, in case of coarser grains in the soil (above 2 mm, and depending on the clay content), these coarser particles can be replaced by artificial quartz fine sand. Then, the liquid limit value would be approximated from the measured value of this new natural/artificial sample, which is a more realistic approximation than only considering the fine particles.
- For coarser grains and low clay content, the equations from Polidori (2007) would provide a better fit for the plastic limit than trying to obtain those values directly from the plastic limit rolling-thread procedure.
- The flow table could deliver valuable data regarding the flow behaviour of soils, conditioned or not, and perhaps be correlated with the flow inside the EPB excavation chamber and screw conveyor. This would optimize the EPB excavation process, especially for cases involving mixed soils.
- This flow table could be a better replacement for the slump tests. It requires less material, features an easier handling procedure, and may offer better information about the flow behaviour of conditioned soil mixtures.
- More tests need to be conducted with the flow table. These tests may include more clay minerals and comparisons between slump test results and rheometer results in order to provide a better idea of how this flow table could replace/complement the slump test.

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## Appendix: Reconstituted Soil Samples

SeeTables 1, 2, 3 and 4.

**Table 1** Main characteristics of each component of the soils

| Product specification | Supplier | Spec. Grav. | Chemical Comp. | Specific Surface ($m^2/g$) | Cation exc. capacity (meq./100 gr) | pH | Absorption (%) | Moisture (%) |
|---|---|---|---|---|---|---|---|---|
| Silica sand | Euroquarz | 2.65 | $SiO_2$ | – | – | 6.5–7 | – | – |
| Friedland clay | MRG Blautonwerk Friedland | 2.71 | Mixture of several minerals | 165.5 | 35-45 | 8.7 | – | 6–8 |
| Kaolinite-EPK Kaolin | Edgar Minerals | 2.65 | $Al_20_3$, $2SiO_2$, $2H_20$ | 28.52 | 4.5 | 5.8 | 11.3 | Max. 1 |
| Kaolinite GHL KS 80 | Georg H. Luh GMBH | 2.62 | $Al_20_3$, $SiO_2$, $Fe_2O_3$ | 18.2 | – | 4.5–9.5 | – | 0.6–2 |
| Active Bentonite IBECO B1 | Imerys Civil Engineering | 2.65 | $Al_20_3$, $SiO_2$, $Fe_2O_3$, MgO, CaO, $K_2O$, $Na_2O$ | 600–800 | 70 ± 10 | 10 | 450 | 11 ± 3 |

Montmorillonite (17%); Illite–Montmorillonite (12%); Illite–Mica (28%); Fireclay (13%); Chlorite (2%); Quartz (25%); Feldspar (2%); Pyrite (1%)

**Table 2** Samples with Bentonite clay and sand and respective Atterberg limit values

| Sample ID | Sand (%) | Bentonite (%) | $W_L$ (%) | $W_P$ (%) | PI (%) | $W_{Pol.}$ (%) |
|---|---|---|---|---|---|---|
| Bsfine20 | 80 | 20 | 86 | 27 | 59 | 19 |
| Bsfine 25 | 75 | 25 | 107 | 27 | 80 | 21 |
| Bsfine30 | 70 | 30 | 127 | 26 | 101 | 23 |
| Bsfine50 | 50 | 50 | 171 | 31 | 140 | 30 |
| Bsfine70 | 30 | 70 | 245 | 34 | 211 | 38 |
| Bfine20 | 80 | 20 | 59 | 24 | 35 | 18 |
| Bfine30 | 70 | 30 | 86 | 28 | 58 | 21 |
| Bfine50 | 50 | 50 | 139 | 27 | 112 | 29 |
| Bfine70 | 30 | 70 | 217 | 36 | 181 | 37 |
| Bfine2_20 | 80 | 20 | 54 | NP | NP | 17 |
| Bfine2_30 | 70 | 30 | 83 | 20 | 63 | 21 |
| Bfine2_50 | 50 | 50 | 157 | 26 | 131 | 29 |
| Bfine2_70 | 30 | 70 | 219 | 34 | 185 | 37 |
| Bmedium20 | 80 | 20 | 53.5 | NP | NP | 17 |
| Bmedium30 | 70 | 30 | 82 | 22 | 60 | 21 |
| Bmedium50 | 50 | 50 | 160 | 32 | 128 | 29 |
| Bmedium70 | 30 | 70 | 194 | 30 | 163 | 36 |
| Bmedium2_20 | 80 | 20 | NP | NP | NP | – |
| Bmedium2_30 | 70 | 30 | NP | NP | NP | – |
| Bmedium2_50 | 50 | 50 | 159 | 27 | 132 | 29 |
| Bmedium2_70 | 30 | 70 | 241 | 44 | 197 | 38 |
| Bcoarse20 | 80 | 20 | NP | NP | NP | – |
| Bcoarse30 | 70 | 30 | NP | NP | NP | – |
| Bcoarse50 | 50 | 50 | NP | NP | NP | – |
| Bcoarse70 | 30 | 70 | 235 | 36 | 199 | 38 |
| B100 | 0 | 100 | 486 | 52 | 435 | 55 |

See Fig. 2 for grain size distribution curve for sfine, fine, fine2, medium, medium 2 and coarse grade sands

**Table 3** Samples with Friedland and Kaolinite clays plus sand, along with respective Atterberg limit values

| Sample ID | Sand (%) | Clay (%) | $W_L$ (%) | $W_P$ (%) | PI (%) | $W_{Pol.}$ (%) |
|---|---|---|---|---|---|---|
| Fsfine20 | 80 | 20 | 26 | NP | NP | 16 |
| Fsfine30 | 70 | 30 | 30 | 23 | 7 | 19 |
| Fsfine50 | 50 | 50 | 46 | 24 | 22 | 25 |
| Fsfine70 | 30 | 70 | 62 | 31 | 31 | 31 |
| Ffine20 | 80 | 20 | NP | NP | NP | – |
| Ffine30 | 70 | 30 | 26 | NP | NP | 19 |
| Ffine50 | 50 | 50 | 35 | 18 | 17 | 24 |
| Ffine70 | 30 | 70 | 48 | 20 | 28 | 30 |
| Ffine2_20 | 80 | 20 | NP | NP | NP | – |
| Ffine2_30 | 70 | 30 | NP | NP | NP | – |
| Ffine2_50 | 50 | 50 | 45 | 22 | 23 | 25 |
| Ffine2_70 | 30 | 70 | 60 | 30 | 30 | 31 |
| Fmedium20 | 80 | 20 | NP | NP | NP | – |
| Fmedium30 | 70 | 30 | NP | NP | NP | – |
| Fmedium50 | 50 | 50 | 44 | 24 | 20 | 25 |

**Table 3** continued

| Sample ID | Sand (%) | Clay (%) | $W_L$ (%) | $W_P$ (%) | PI (%) | $W_{Pol.}$ (%) |
|---|---|---|---|---|---|---|
| Fmedium70 | 30 | 70 | 63 | 32 | 31 | 31 |
| F100 | 0 | 100 | 94 | 41 | 53 | 40 |
| Ksfine30 | 70 | 30 | 22 | 16 | 6 | 19 |
| Ksfine50 | 50 | 50 | 32 | 21 | 11 | 24 |
| Ksfine70 | 30 | 70 | 44 | 29 | 16 | 30 |
| K100 | 0 | 100 | 62 | 40 | 22 | 38 |

See Fig. 2 for grain size distribution curve for sfine, fine, fine2, medium, medium 2 and coarse grade sands

**Table 4** Combination for samples with sieved sand and with respective Atterberg limit values

| Sample ID | Sand sieve | | | Kaolinite (%) | $W_L$ (%) | $W_P$ (%) | PI (%) | $W_{Pol}$ (%) |
|---|---|---|---|---|---|---|---|---|
| | 30 (0.59–0.85 mm) | 40 (0.42–0.59 mm) | 50 (0.29–0.419 mm) | | | | | |
| 11K_2 | – | 80 | – | 20 | 11.4 | NP | NP | 16 |
| 11K_3 | – | 70 | – | 30 | 20 | 13 | 7 | 19 |
| 11K_4 | – | 60 | – | 40 | 24 | 15 | 9 | 21 |
| 11K_5 | – | 50 | – | 50 | 30 | 19 | 11 | 24 |
| 11K_6 | – | 40 | – | 60 | 38 | 23 | 15 | 27 |
| 11K_7 | – | 30 | – | 70 | 43 | 24 | 19 | 30 |
| 11K_8 | – | 20 | – | 80 | 49 | 28 | 21 | 33 |
| 11K_9 | – | 10 | – | 90 | 59 | 30 | 28 | 36 |
| 12K_2 | – | – | 80 | 20 | 14 | NP | NP | 16 |
| 12K_3 | – | – | 70 | 30 | 22 | 13 | 9 | 19 |
| 12K_4 | – | – | 60 | 40 | 27 | 16 | 11 | 21 |
| 12K_5 | – | – | 50 | 50 | 32 | 19 | 13 | 24 |
| 12K_6 | – | – | 40 | 60 | 38 | 23 | 15 | 27 |
| 12K_7 | – | – | 30 | 70 | 44 | 26 | 18 | 30 |
| 12K_8 | – | – | 20 | 80 | 52 | 28 | 23 | 33 |
| 12K_9 | – | – | 10 | 90 | 59 | 32 | 27 | 36 |
| 13K_2 | 80 | – | – | 20 | 12.6 | NP | NP | 16 |
| 13K_3 | 70 | – | – | 30 | 20 | 13 | 7 | 19 |
| 13K_4 | 60 | – | – | 40 | 26 | 15 | 11 | 21 |
| 13K_5 | 50 | – | – | 50 | 32 | 19 | 13 | 24 |
| 13K_6 | 40 | – | – | 60 | 39 | 22 | 17 | 27 |
| 13K_7 | 30 | – | – | 70 | 46 | 26 | 20 | 30 |
| 13K_8 | 20 | – | – | 80 | 52 | 29 | 23 | 33 |
| 13K_9 | 10 | – | – | 90 | 60 | 30 | 29 | 36 |

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