

École polytechnique de Louvain

Impact of sample size and clipping method on triaxial test results for coarse granular soil

Optimisation of the embankment slope angle

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Abstract

In civil engineering, embankments are often built using a class of materials known as "*coarse soils*". They are characterised by the large size of their constituent elements, which are over-dimensioned compared with standard-sized devices. Assessing their resistance to shear requires the use of large devices, the testing of which represents a cost that few can afford. The value of the internal friction angle for this type of soil is not measured and is therefore assumed to be at least 35° .

This master thesis investigates the evolution of the shear strength characteristics of a soil as a function of the device's size used and the sample clipping method, with the aim of increasing the slope angle of an embankment. After defining a coarse reference soil, the simple clipping method and the parallel gradation modelling method are presented and used to produce standard-sized samples. Tests under consolidated drained conditions are carried out using triaxial devices of different diameters $\phi = 38, 70$ et 100mm . The influence of the size of the device and the clipping method on the values of the internal friction angle and cohesion are studied jointly. It has been found that both clipping methods show an increase in the internal friction angle as the size of the device increases. For the simple clipping method, increasing the diameter of the device from 38mm to 100mm implies a gain of $2,6^\circ$. For the parallel method, an increase of 11° is observed between the 38mm and 70mm samples. Cohesion for the first method is 13,08, 6,5 and 6,38 kPa for samples 38C, 70C and 100C. For the second method, the values are 30,71 and 11,67 kPa for samples 38P and 70P. This difference in shear strength characteristics shows the involvement of soil properties, such as fines content and coarse particle size, defined by the clipping method used. These shear strength characteristics are then used to define an acceptable embankment slope. Lastly, an analysis of the gains following this change in slope is carried out. A saving of between 19,9 and 24,84% compared with the cost of a reference embankment is achieved with the use of 38mm devices. This saving rises to between 22,9 and 28,59% for a 70mm diameter and increases again for 100mm to between 29,41 and 36,72%.

Keywords : coarse soils, triaxial devices, shear strength, clipping methods, embankment slope

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Introduction

When building an embankment, the determination of the slope angle is an important economical factor. Indeed, changing the angle by a few degrees can significantly change the volume of soil to be transported and placed or removed. This is a key economical factor for earthwork companies.

For this type of work, soils considered as *coarse soils* are often used. These ones are made up of small and large particles. The presence of large elements is problematic when characterising and assessing the mechanical properties of the soil [1]. The main issue is that for this type of material, traditional triaxial tests are not appropriate. The friction angle is often not measured but is assumed to be at least 35-40°. The use of large-scale equipment would be necessary to confirm this angle's value, but the tests carried out with this equipment are expensive. It is interesting to know if the cost of testing a large device is justified. The global strategy is to define the expected savings from using coarse materials whose shear strength enable steeper embankment slopes, compared with the cost of using large-scale triaxial devices to characterise these materials.

Several researchers have previously studied the influence of modifying the parameters of a soil by changing its fine particle content, its particle size distribution, the size of coarse particles, etc. Holtz and Gibbs found that the value of the internal friction angle increase when the maximum particles size increase [2]. The same trend was observed by Afriani [3]. Pedro observed that shear strength increases as the volume fraction of fine particles decreases [4]. Seif El Dine reached the same conclusion [5]. Results of Afrazi and Yazdani showed that the internal friction angle is affected by the particles size distribution [6]. Others researchers have investigated the influence of device size on soil shear strength by testing the same soil with different device sizes. Reiffsteck *et al.* have established that the increase of device's size induces an increase of the internal friction angle and a decrease of the cohesion [7]. However, research linking these two issues is rare, if not non-existent.

The purpose of the study is to investigate the influence of the sample diameter

with the same ratio height/diameter in triaxial tests ($38mm$, $70mm$ and $100mm$) in conjunction with changes in soil parameters on shear strength of a reference soil and therefore on the slope design and its optimization. A schematic diagram of the triaxial device used is shown in Figure 1.

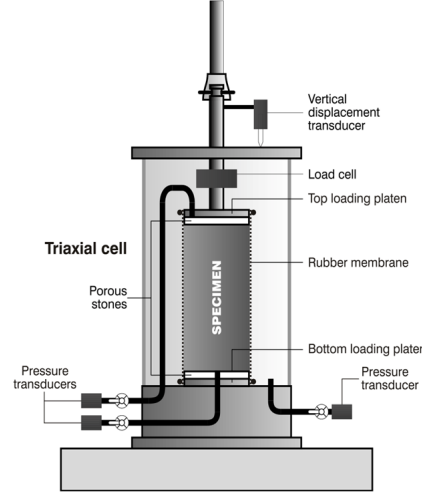


Figure 1: Schematic diagram of the triaxial device

The results will be compared and an attempt will be made to establish a link between the mechanical properties and the size of the device. A trend will then be suggested. In addition, a sample with larger particles tested with a bigger apparatus should make it possible to obtain higher soil resistance values. Another objective will be to evaluate the influence of this additional resistance on the acceptable slope angle for the formation of embankments. Finally, an economical study of an earthworks project using the reference soil will be carried out.

To achieve these goals, it is important to understand the influence of device's size and coarse elements in the mechanical behavior of soils and to analyze their effect.

This master thesis is divided into seven chapters :

- The first one puts the subject of this report into context. The characteristics of coarse soils are presented after establishing a definition of these soils based on several standards ;
- In the second chapter a literature review of the soil parameters influencing its mechanical behaviour is carried out. The influence of fines content, coarse

particle size and particle size distribution is studied. The reference coarse soil is then chosen. Two granulometric reconstitution methods will then be presented and applied to the selected soil ;

- The third focuses on the devices used. A literature review is made of the various existing devices and their impact on soil properties. The triaxial devices of different sizes, ranging from $38mm$ to $100mm$, required to perform the tests are then described ;
- The fourth chapter will cover methodology. This begins with the formation of samples and their installation. Then the various stages of a consolidated drained triaxial test are presented. The way in which the results will be processed is also detailed ;
- The results are presented in chapitre 5. The consolidation and shearing phases will be the subject of interest. The internal friction angle and cohesion will be studied for different device sizes ;
- The purpose of the sixth chapter is to apply the results obtained from an analysis of slope stability. An economical analysis is also carried out and the added value of using large devices is discussed ;
- Finally, the outlook and recommendations for future research can be found in the seventh chapter.

1 | Context

In this chapter, the different soil classifications and the parameters used will be explained. Then, the focus will be on coarse soils and a definition of these will be given. Finally, a distinction will be made between the fine and larger particles that make up coarse soils. This distinction will be necessary as it plays an important role in the properties that influence shear strength.

1.1 Soil classification

Soil classification sort different soils based on their geotechnical characteristics.

According to the *Internal Union of Soil Sciences* (IUSS), there are two main categories of soils: cohesive soils with cohesion and granular soils without it [8]. The first category refers, for example, to clays and silts. These are soils with a high clay content, which has cohesive strength. The diameter of their grains is less than $20\mu m$, with particles between $2\mu m$ and $20\mu m$ for silt and smaller than $2\mu m$ for clay [8]. Their behavior will be impacted by the water content [9]. Indeed, it will significantly influence brittle or plastic behavior, with the transition from one to the other for different types of clay plasticity [10]. The second refers to sand, gravel, etc. Their grains have a diameter higher than $20\mu m$. The interaction between the particles will directly influence their behaviour. On the other hand, compared to cohesive soils, water content has a lower impact on their behaviour [11]. According to the *Unified Soil Classification System* (USCS classification), sand consists of at least 50% particles between $0,08mm$ and $2mm$ [12].

In nature, soils are often composed of particles of different diameters from both categories, consisting of particles corresponding to a granular soil and others to a cohesive soil forming an intermediary state. In Figure 1.1, some examples of soils type from literature are presented, with their particle size curve.

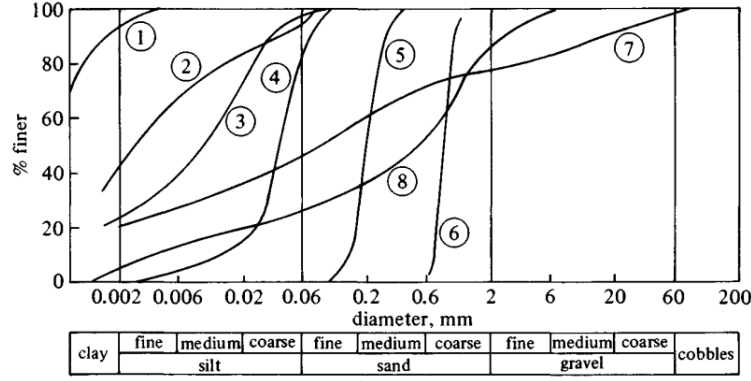


Figure 1.1: Grading curves for (1) kaolin, (2) London clay, (3) Vienna clay, (4) estuarine silt, (5) medium sand, (6) coarse sand, (7) glacial till (after Atkinson and Bransby, 1978), and (8) residual granitic soil (data from Yudhbir, 1982) [13]

According to the standard NF P11-300 [14], there are several parameters to classify soils [15]:

1. The granulometry
2. The plasticity index, I_p
3. The value of methylene blue VBS

Thanks to these parameters, soils can be classified in 4 distinct classes. The Table 1.1 illustrates this classification.

Table 1.1: Soils classification according to NF P11-300

Class	Definition	Characteristics	Sub-class
A	Fine soils	$d_{max} \leq 50mm$ and $80\mu m \geq 35\%$ sieving	A1 to A4 according to VBS or I_p
B	Sandy and gravelly soils with fines	$d_{max} \leq 50mm$ and $80\mu m \leq 35\%$ sieving	B1 to B6 according to VBS or I_p and sieve
C	Soils with fines and large elements	$d_{max} \geq 50mm$ and $80\mu m \geq 12\%$ sieving or $80\mu m \leq 12\%$ sieving + $VBS \geq 0,1$	30 sub-class according to VBS or I_p and $50mm$ sieving
D	Water-insensitive soils with fines	$VBS \leq 0,1$ and $80\mu m \leq 12\%$ sieving	D1 to D3

1.2 Definition of coarse soils

The term "*coarse soil*" can be related to one of the classes mentioned above. According to this standard, a soil is considered as coarse when the maximum diameter d_{max} of its grains is greater than $50mm$. However, coarse soils are not mentioned in the classification used in soil mechanics and in the earthmoving field. It is difficult to give an exact definition of this type of soil. Several authors in literature, that will be cited later, have tried to give their own definition of this type of soil according to the size of the particles but also to the tests carried out to characterize the soil. Their studies are detailed below. An important criterion will be the size of the device used to define the maximum diameter of the particles.

It is mainly in mountainous areas that coarse soils are found in their natural form. They can also be of alluvial origin or come from landslides. Coarse soils can also be reconstituted by mixing several types of soils of different granulometry. They are used as construction materials for civil engineering works.

A number of studies have been carried out on alluvium. These can be pebbles, gravels, cobbles, sands and silt grouped together, unconsolidated solid materials, which have accumulated following the passage of water.

Valle (1998, 2001) used a natural alluvial gravel with a grain size of $0/200mm$. The characteristics of this soil were defined by triaxial and direct shear box tests [16, 17].

Holtz and Willard (1961) used soil-gravel mixtures representing the most commonly used materials in the creation of embankments. The clay portion is characterized by a density between $2,66g/cm^3$ and $2,7g/cm^3$, to which are mixed gravels of maximum diameter $d_{max} = 76,2mm$. Figure 1.2 represents the different particle sizes chosen for their study. The triaxial device used has a diameter of $\phi = 228,6mm$ [18].

According to the USCS classification, coarse-grained soils have more than 50% of their elements larger than $2mm$. The maximum size of the elements is not relevant [12].

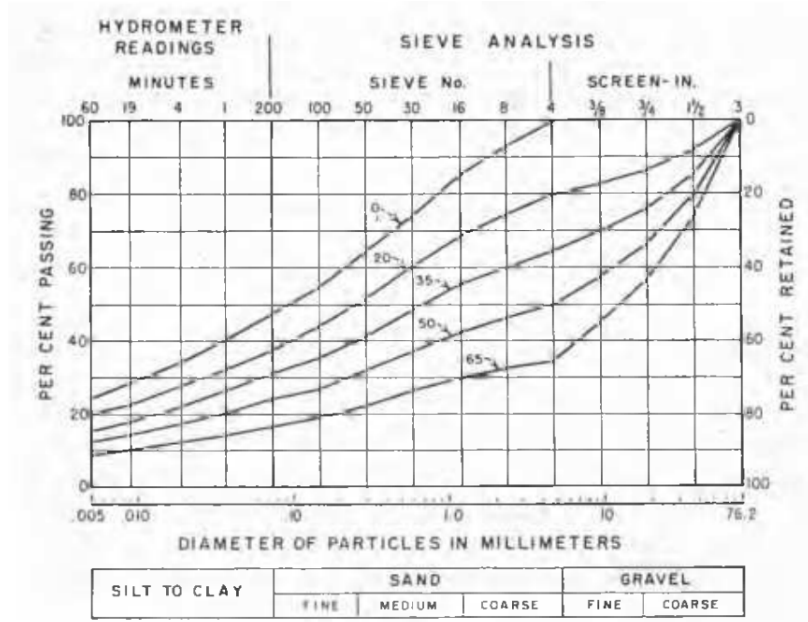


Figure 1.2: Gradation of clayey gravel soils (Holtz and Willard, 1961) [18]

Research was carried out by Nichiporovitch and Rasskazov (1967) in the laboratory on a shear box but also on triaxial device with soils having fractions up to $50mm$ [19].

Torrey and Donaghe (1985), in their study of the behavior of coarse soils, studied a mixture of gravel, sand, silt and clay of $d_{max} = 76mm$ with a triaxial device of diameter $\phi = 381mm$ [20].

Gotteland (2000) studied a soil from the torrent of Manival. It is composed of angular calcareous and marly-calcareous grains in a fine silty-clay matrix. The soil has a very large grain size with elements up to $1000mm$. He carried out its studies with a box of shearing of great dimension, taking a fraction corresponding to the $0/100mm$ of the material [21].

Kouakou (2020) is interested in a soil composed of $0 - 80mm$ aggregate, composed of 1,9% of fine, and natural gravel resulting from a villafranchian sedimentary deposit, whose granulometry is very spread out with 4,3% of fine and particles as large as $120mm$. It uses a large direct shear box of $300 \times 300 \times 180mm$ [22].

Pedro (2004), studies an alluvial gravel as a reference soil consisting of large inclusions in a sandy matrix, corresponding to the fine fraction, show in Figure 1.3. It is a particular type of soil [4]. Some clarification of the terms inclusions and matrices is provided in Section 1.3.

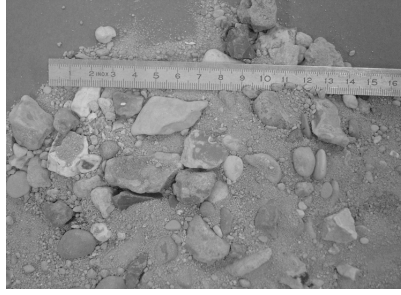


Figure 1.3: View of the alluvial gravel 0/20mm (Pedro, 2004) [4]

Reiffsteck (2007) has chosen for his study a natural 0/80mm gravel from a sandy-limestone alluvial deposit [7].

Afriani (2002) selected the same reference soil clipped to 50mm and 80mm [3].

All the soils mentioned above can be considered as coarse soils although they are all very different. Some characteristics are common to all of them :

- A spread out granulometry. Elements of different nature and size which can go from some microns to several tens of decimeters
- Elements of significant size in relation to the size of the experimental devices used for the studies

Therefore, it is important to note that these characteristics are complementary and must be taken into account simultaneously to define a coarse soil.

1.3 Distinction between matrix and inclusions of the coarse soil

Coarse soils are often considered to be a mixture of a fine portion called matrix and larger particles called inclusions. Several possibilities are presented for defining the boundary between these two categories of constituents.

Gotteland (2000) considered grains larger than the maximum size allowed by the apparatus used as inclusions [21]. This value may vary from one author to another for the same soil, depending on the equipment used by different authors. Medley (2002) defined the matrix as the particles whose size does not control the mechanical behavior of the soil, while the inclusions have a significant impact on soil behavior [23].

Pedro (2004) considered that the shear strength values of a soil are considered correct when the size of the inclusions is sufficiently large compared to the size

of the matrix constituents. He defined the coarse matrix soil, whose inclusions constituting the coarse fraction are chosen such that the ratio $d_{50,incl}/d_{50,mat}$ (with $d_{50,incl}$ is the average particle size of the matrix and $d_{50,mat}$ the average size of the inclusions) is greater than 10 and the ratio between the diameter of the specimen and the maximum size of inclusions is greater than 10 [4].

Jiang *et al.* (2016) considered a criterion related to the shape of the particle size curve. The limit between the inclusions and the matrix corresponds to a level, meaning a percentage of almost identical passing between two grain sizes. In this case, the coarse soil is really considered as a mixture of two distinct fractions (Figure 1.4) [24].

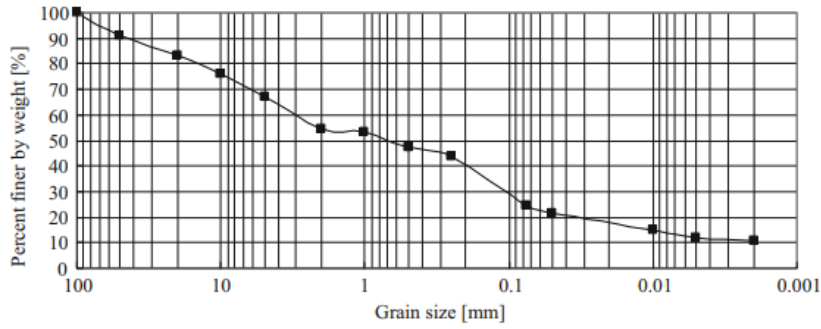


Figure 1.4: Particle size distribution with limit between matrix and inclusions (Jiang, 2016) [24]

These different criteria may be more or less relevant depending on the characteristics that the authors want to emphasize.

2 | Studied soils

In this chapter, the focus is on the soil studied during this master thesis. Firstly, a review of the research that has already been done by several authors on the parameters that influence the mechanical characteristics of soils was achieved. Then the reference soil chosen for this study is carried out. Finally, the focus will be on the adaptation of this coarse soil to standard-sized equipment and the choices made to achieve this. To do this, the possible methods for clipping this reference soil will be analysed and applied to obtain the physical properties of the samples tested in the study.

2.1 Parameters influencing the mechanical behavior of a soil

To study the mechanical characteristics of coarse soils, the use of clipping methods will often be preferred. These methods consist in a change in soil particle size distribution and the constituents of its fine fraction and inclusions. It is therefore important to study the effect of these fundamental parameters in order to improve the knowledge of the mechanical behavior of coarse soils, and especially the shear strength. In the following, the results obtained by different authors on the influence of the parameters related to the soil are presented.

2.1.1 Coarse particle sizes

Holtz and Gibbs (1956) carried out two series of tests by fixing in the first case the mass percentage of inclusions to 20 % then in the second case to 50 %. In both series, the maximum grain size will vary from 19,1mm to 76,2mm and a triaxial cell of 230mm diameter and 570mm height was used. Figure 2.1 presents the granulometric curves obtained. Looking at the results obtained in Table 2.1, there is a very slight size effect. Indeed, the internal friction angle of the soil increases by 1,1° and 1,3° between the smallest and largest value of the maximum diameter for

an inclusion mass fraction of 20% and 35% respectively. This increase is therefore minimal compared to the values of 37,2° and 40,0° obtained [2].

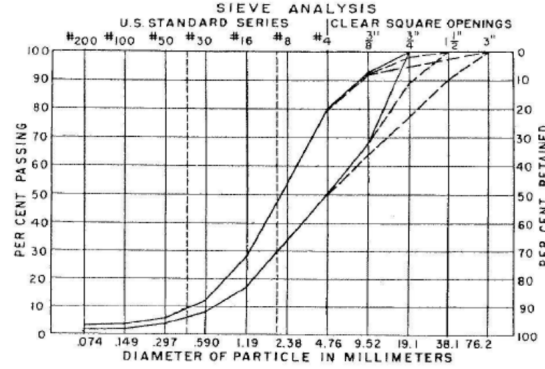


Figure 2.1: Particules size distribution of the materials tested (Holtz and Gibbs, 1956) [2]

Table 2.1: Effect of inclusion size on soil characteristics (Holtz and Gibbs, 1956)[2]

	Mass fraction of inclusion					
	20%			50%		
d_{max} (mm)	19,1	38,1	76,2	19,1	38,1	76,2
φ (°)	37,2	37,9	38,3	40,0	40,0	41,3

Afriani *et al.* (2003) carried out several series of tests on a large shear box of dimensions $500 \times 500mm$ to study the effect of the size of the grains of a natural alluvial material whose maximum diameter is nearly 200mm. After capping, the diameters retained for the study are 25, 50 and 80mm. The results obtained are presented in Table 2.2. Afriani finds that the angle of friction and the cohesion increase when the maximum diameter of the grains increases [3].

Table 2.2: Effect of the grain size of an alluvial gravel (Afriani, 2002)[3]

d_{max} (mm)	c (kPa)	φ (°)
25	46,7	35,8
50	56,4	38,5
80	59,8	40,2

Dupla *et al.* (2007) carried out tests on soils reconstituted with gravels of different classes and sizes (4/5mm, 8/10mm, 16/20mm), for volume fractions

($f_v = \frac{\text{Solid volume of coarse particles}}{\text{Sample's total volume}}$) of 20% and 35%. The results obtained for $f_v=20\%$ show that there is no significant influence on the shear strength or the volume deformation (Figure 2.2). This is confirmed for $f_v=35\%$. The conclusion of these observations is that the gravels see the matrix as a homogeneous medium and that the elementary volume tested is well representative of the material [25].

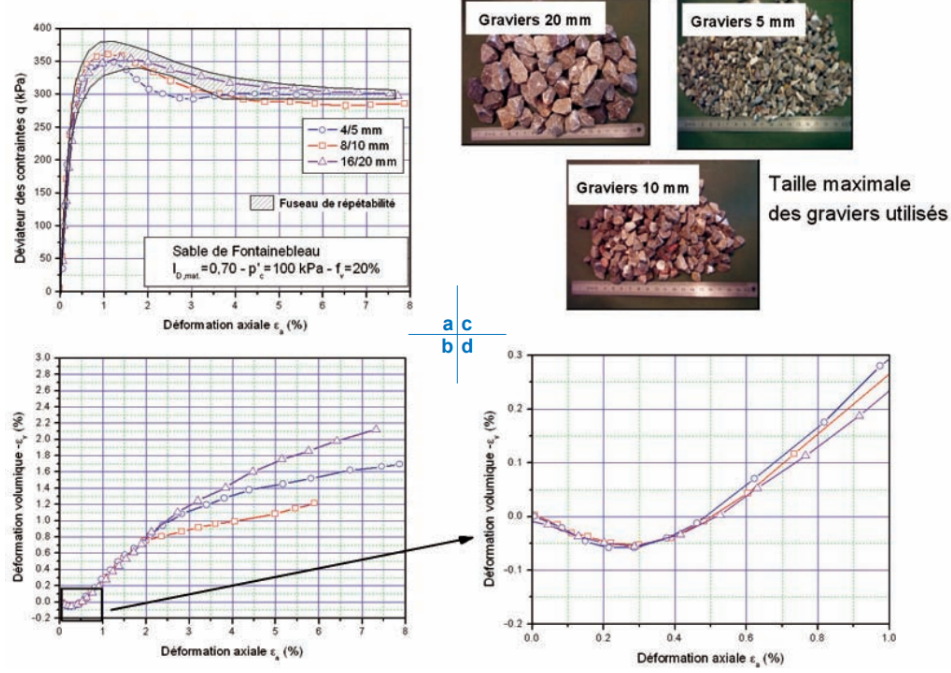


Figure 2.2: Influence of the size of the inclusions on the behavior of the material (Dupla *et al.*, 2007)[25]

To analyze the impact of the size of the coarse particles on the mechanical behavior of a soil, Pedro (2004) analysed two types of soil. On one hand a soil with a volume fraction of inclusion equal to 20 % and on the other hand a volume fraction equal to 35 %. These volume fractions contain inclusions of size 4/5mm, 8/10mm and 16/20mm [4]. The diameter of the specimen is 100mm and the height is 200mm. When the volume fraction is 20%, the stress deviation curves at failure are independent of the size of the inclusions. In the case with 35% inclusions, the size of the elements has no influence on the shear strength nor on the evolution of the volume variation. On Figure 2.3 and Figure 2.4, the absence of size effect is noticed, the curves of all the tests are in the 8/10 zone. The matrix is evaluated as a homogeneous medium by the gravels and that the diameter of the specimen is large enough for the soil to contain inclusions ranging from 4 to 20mm. Under these conditions the behavior of the soil is unchanged.

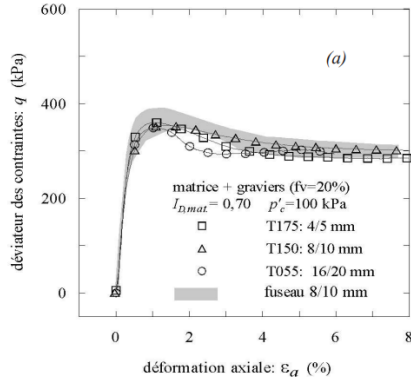


Figure 2.3: Influence of the size of the inclusions on the deviator stress, 20% of inclusions (Pedro, 2004)[4]

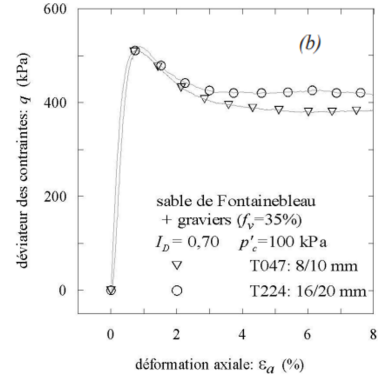


Figure 2.4: Influence of the size of the inclusions on the deviator stress, 35% of inclusions (Pedro, 2004)[4]

Seif El Dine (2007) performs a series of triaxial tests using two different particle size distributions 10/20mm and 30/60mm [5]. The device used was a triaxial with a diameter of 300mm and a height of 600mm. The Figure 2.5 shows that for a coarse particle volume fraction of 20%, the particle size has very limited influence on the mechanical behaviour of the soil, the curves corresponding to the two particle sizes are within the repeatability range, hatched on Figure 2.5.

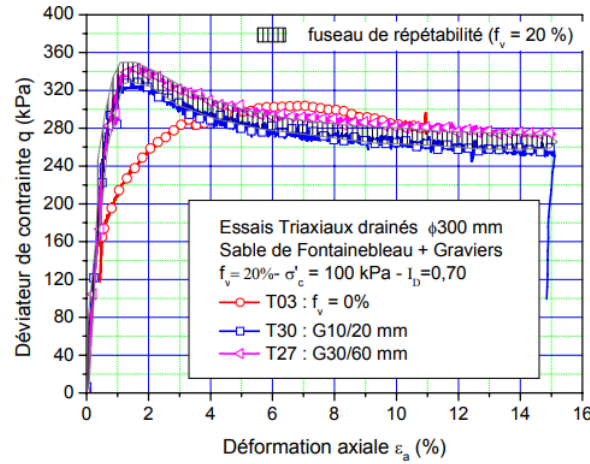


Figure 2.5: Influence of the size of inclusions (Seif El Dine, 2007)[5]

These studies show that coarse particle size does not have a strong influence on the shear strength of coarse soils.

2.1.2 Influence of the inclusion proportion and the fines content

In a sample, proportion of inclusion in the total mass will be directly related to the fines content. Indeed, when the proportion of inclusion increases, the fines content will decrease, thus keeping the total mass the same. It is an important parameter because to characterize coarse soils, it is necessary to clip it. This consists in studying a soil whose volume fraction of the largest particles will change, decrease or even disappear. This will be detailed in Section 2.3.

Many researchers have studied the effects of a variation of one criterion. Most of the studies show that the shear strength of coarse soils increases with the percentage of inclusions (Holtz and Gibbs (1956), Pedro (2004) , Dupla *et al.* (2007), Seif El Dine (2007), Wang *et al.* (2018)). On the other hand, an opposite tendency is observed in the work of Salgado *et al.* (2000) [26] and Andrianatrehina (2015) [27]. They note an increase in mechanical strength with an increase in the internal angle of friction at the peak when the fines content increases with a decrease in the coarse fraction.

Holtz and Gibbs (1956) carried out triaxial tests under drained conditions on specimens of 228,6mm in diameter and 571,5mm in height. These specimens are made of a mixture of gravels of size between 4,76mm and 19mm and a matrix of sandy elements of size lower than 4,76mm [2]. The Figure 2.6 shows the different particles size distribution curves tested.

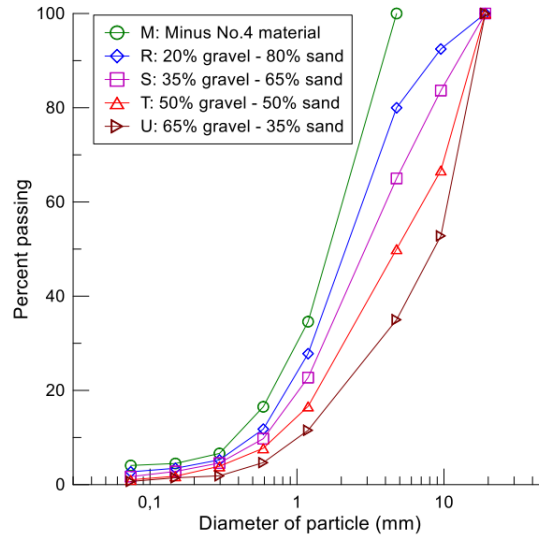


Figure 2.6: Particles size distribution of tested materials (Holtz and Gibbs, 1956)[2]

On the Figure 2.7, results show that the internal friction angle of the material increases when the proportion of inclusions increases from 0 to 50%, and then decrease between 50 and 65%. The values of the friction angle are shown in the Table 2.3

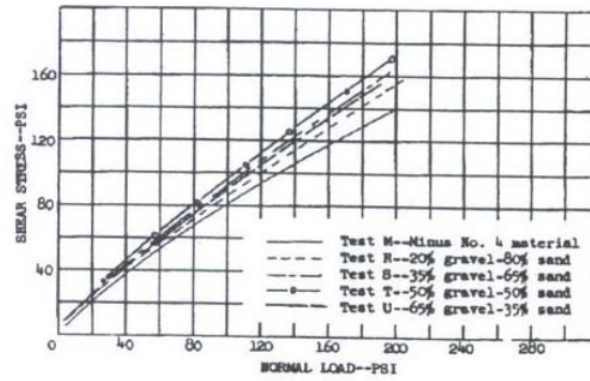


Figure 2.7: Effect of the inclusion proportion (Holtz and Gibbs, 1956)[2]

Table 2.3: Influence of the inclusion proportion on the friction angle (Holtz and Gibbs, 1956)[2]

Mass fraction of inclusions (%)	φ (°)
0	35,7
20	37,2
35	39
50	40
65	37,9

In his study, Pedro (2004) highlighted the effect of the inclusion proportion on the mechanical behavior of coarse soils for a cylindric sample of $\phi 100$ [4]. To do so, he changes the volume fraction of gravel f_v from 0 to 35%. He observed that the isotropic compressive modulus and Young's modulus increase with the volume fraction (Figure 2.8), as well as the shear strength at rupture (Figure 2.9).

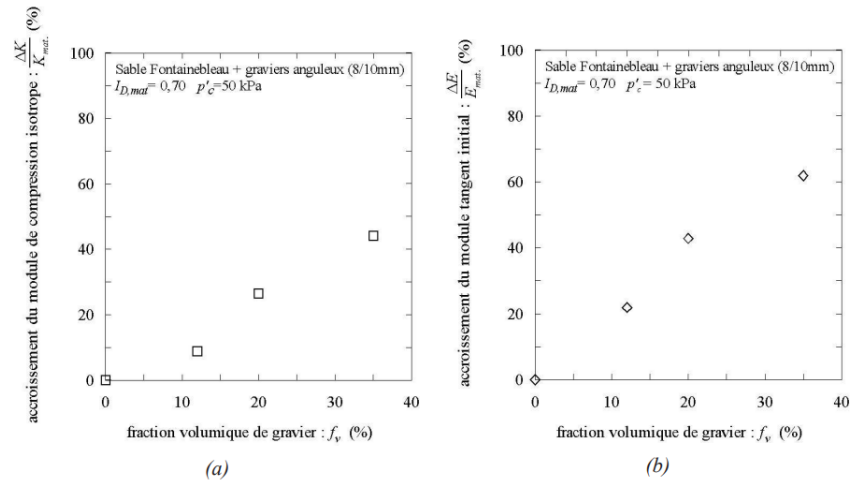


Figure 2.8: Influence of the inclusion proportion : (a) on the increase of the isotropic modulus ; (b) on the increase of the Young modulus (Pedro, 2004)[4]

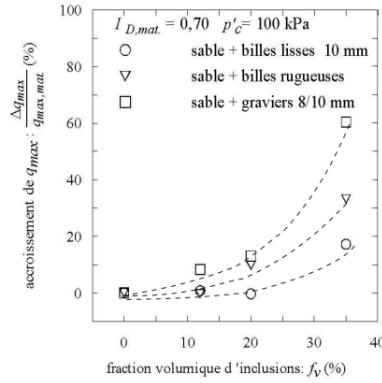


Figure 2.9: Influence of the inclusion proportion on the increase of the mechanical characteristics at failure (Pedro, 2004)[4]

Seif El Dine (2007) performed a series of tests on reconstituted soils composed of a matrix of Fontainebleau sand and a volume fraction varying from 0% to 35% composed of gravels of different sizes, considered as inclusions : 0/10mm, 10/20mm, 30/60mm, 10/60mm and 4/60mm [5]. Figure 2.10, show that the internal friction angle φ at peak increases significantly together with the increase of the inclusion volume fraction, for all size of inclusion.

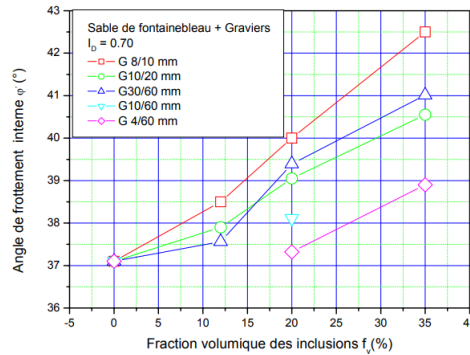


Figure 2.10: Evolution of the internal friction angle φ at maximum strength as a function of the volume fraction of the inclusions (Seif El Dine, 2010)[5]

Wang *et al.* (2018) studied a coarse soil made of ballast grains with a maximum diameter of 20mm. The device used allowed sample of 100mm diameter. Grains vary f_v in six different ways : 0, 5, 10, 20, 35 et 45% [28]. They consider the coarse part as the particles whose diameter is larger than 2mm and the fine part as the particles smaller than 2mm. For the same confining pressure of 60 kPa, the

deviatoric stress q_{max} increases with the increase of f_v . Regarding the volumetric deformations, a trend is observed regardless of f_v . The samples show a contracting phase followed by a dilating one. This trend becomes more pronounced for a larger proportion of coarse particles (Figure 2.11). On Figure 2.12 the internal friction angle at peak state increases when f_v varies from 0 to 45%. The rate of increment is lower for values of f_v close to zero. The cohesion presents an opposite tendency to that of internal friction angle. It decreases with higher values of f_v . Indeed, as the quantity of coarse particles increases, the quantity of fine particles is reduced, leading to a smaller cohesion.

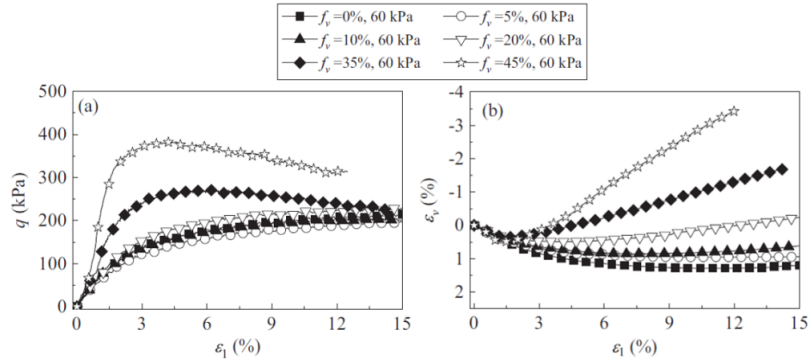


Figure 2.11: Stress-strain behaviour for all specimens under a confining pressure of 60 kPa (Wang *et al.*, 2018)[28]

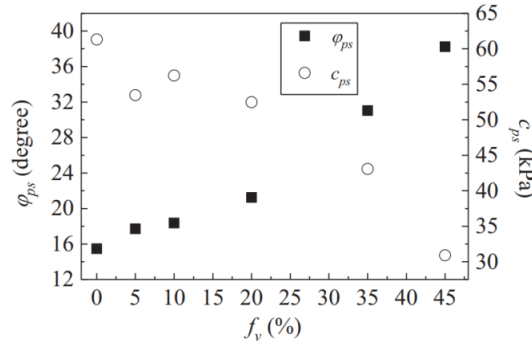


Figure 2.12: Variation of the internal friction angle and cohesion as a function of the volume fraction of coarse particles (Wang *et al.*, 2018)[28]

Following these studies, it can be said that an increase in the fraction of inclusions in a soil, and thus a decrease in the fines part, leads in most cases to

higher shear strength of the latter. The use of inclusion clipping will therefore imply an underestimation of the shear strength of coarse soils during their determination.

2.1.3 Particle size distribution

To study this parameter, Seif El Dine (2007) carried out tests by varying the particle size distribution of inclusions d_{min}/d_{max} using a 300mm diameter triaxial device. The inclusions tested are gravels : 8/10mm, 10/20mm, 30/60mm, 10/60mm and 4/60mm, the volume fraction of these soils being 20% ou 35%. In both cases, it can be seen that the value of the deviator stress at peak state decreases when the particle size distribution increases. Moreover, the distribution does not influence the volume deformations during the contracting phase, but the soil will tend to be less dilatant if the particle size distribution of the inclusions is spread out (Figure 2.13). The conclusion is that a soil with a tight grading and a higher volume fraction of inclusions will have a higher shear strength than a soil with opposite properties [5].

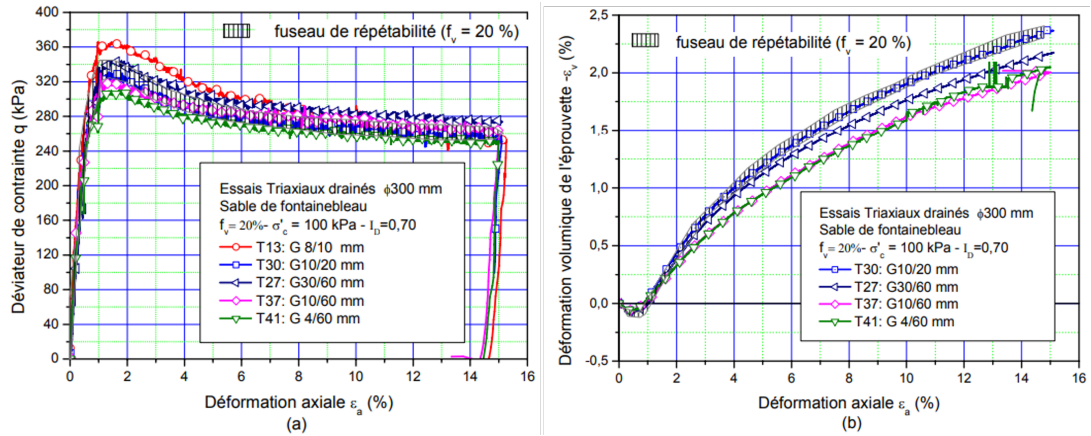


Figure 2.13: Influence of the particle size distribution of the inclusions: (a) shear curves; (b) volume deformation curves (Seif El Dine, 2007)[5]

Dupla *et al.* (2007), showed that there is no significant effect of particles size distribution observed on the shear strength of the soil [25].

The results of Afrazi and Yazdani (2021), showed that the friction angle of the soil is affected by the particle size distribution. If the average value of particle size increases, the angle of friction and dilation will increase [6].

Yagis (2001) had already shown that soil resistance increased with increasing average grain diameter [29].

Studies contradict each other with regard to the influence of the particle size distribution. no clear conclusion can be drawn from them.

2.2 Reference soil

A coarse soil will be formed of coarse particles of different nature and a fine fraction which can be sand, silt, clay or a mixture of these materials. The difference between inclusion and matrix, the two composant of the coarse soil, can be defined differently according to the authors (Section 1.3). A soil with a spread and discontinuous particle size is chosen. It will be composed of a mixture of fine and coarse particles, corresponding to a matrix with inclusions of any size.

The study is based on one type of soil from Australia. A basecourse material which is composed of coarse particles and marine sediments that have a particle size corresponding to sand according to the *USCS* with more than 50% of elements larger than $0,08mm$ and smaller than $2mm$ [12] are used. 15% of marine sediments are thinner particles.

On Figure 2.14, the particle size distribution curves (PSD) are represented. For the basecourse material, the final PSD is already available thanks to the tests performed in 2020 (Appendix A), but for the marine sediments a study was conducted to determine this curve (see Appendix B).

The basecourse material is composed of elements whose size varies between 2 and 50 mm while the size of the grains for the marine sediments varies between fines of diameter lower than $63\ \mu m$ and particles of 2 mm . The fines lower than $63\ \mu m$ are considered like one single sieve, which explains the steep slope at this point on the Figure 2.14. A more detailed study of this part of the marine sediments has been carried out and can be found in Appendix C. The maximum diameter of the coarse particle, $d_{max} = 50mm$, is large enough to avoid potential scaling effects that would be due to a ratio between the largest elements and the size of the specimen being too small.

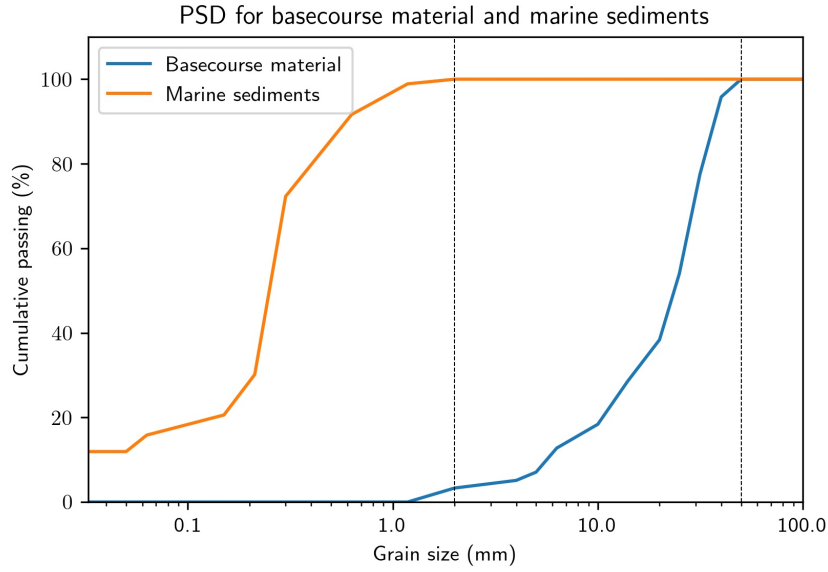


Figure 2.14: Particle size curve for the basecourse material and the marine sediments

For reference soil, proportions of 85 % of the basecourse material with 15 % of marine sediments are used. These proportions had been used in the past to perform tests on a ϕ 300mm triaxial device (see Appendix D). The objective of this previous study was to determine the minimum density and the minimum marine sediments content that will provide a stable sample of 300mm diameter that can be transferred to the triaxial cell for testing. This amount of sediment allowed the large sample to stand upright and hold a uniform shape when the sample preparation mold was removed.

For the reference soil, the PSD curve obtained is showed in Figure 2.15. This one is realized by taking the curves of the two materials composing the reference soil, in the proportions quoted above. The mixture corresponding to the reference soil is visible on Figure 2.16.

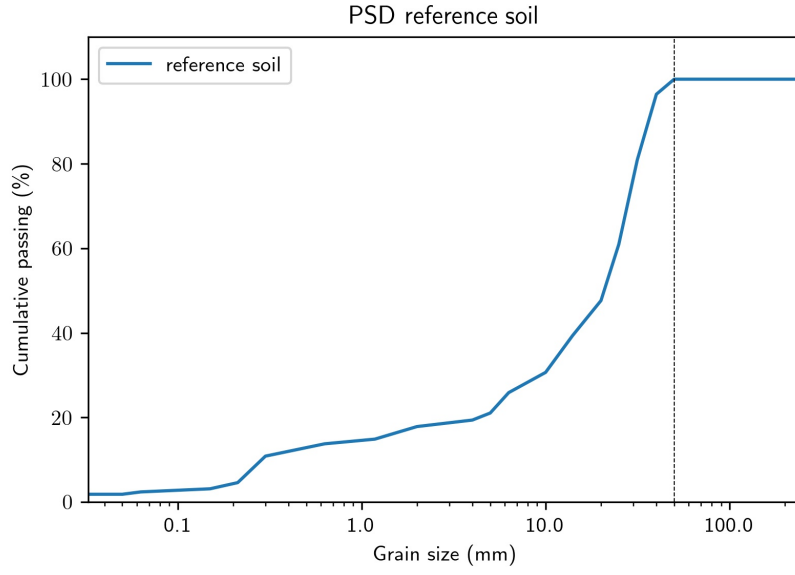


Figure 2.15: Particle size curve for the reference soil



Figure 2.16: View of the reference soil

Taking into account the percentage of basecourse and marine sediments, physical characteristics of the reference material can be determined. These are listed in Table 2.4 and taken for granted by the tests conducted in 2020 (see Appendix D).

Table 2.4: Physical properties of the reference soil

d_{max} (mm)	d_{min} (mm)	$e_{initial}$ (-)	ρ_s (g/cm^3)	ρ_d (g/cm^3)
50	0,063	0,57	2,788	1,774

2.3 Choices for less coarse soils

Due to the presence of large particles, it is challenging to analyze and describe coarse soils using traditional laboratory equipment. Therefore, researchers often relied on some methods to adapt the PSD curve. Two specific ones are subject of interest: the simple clipping method and the parallel gradation modeling method. However, the use of these methods will impact the results when characterizing coarse soils whose maximum particle size is greater than that allowed in the tests. In what follows, the effects of these methods will be analysed, using the parameters studied in Section 2.1. Then, the observations from the literature will guide the choice of clipped soil.

2.3.1 Simple clipping method

This method consists in removing from the soil the particles whose maximum diameter is higher than the admissible diameter. This modification will lead to a change in the particle size curve with an increase in the fine fraction and a decrease in the coarse fraction (Figure 2.17).

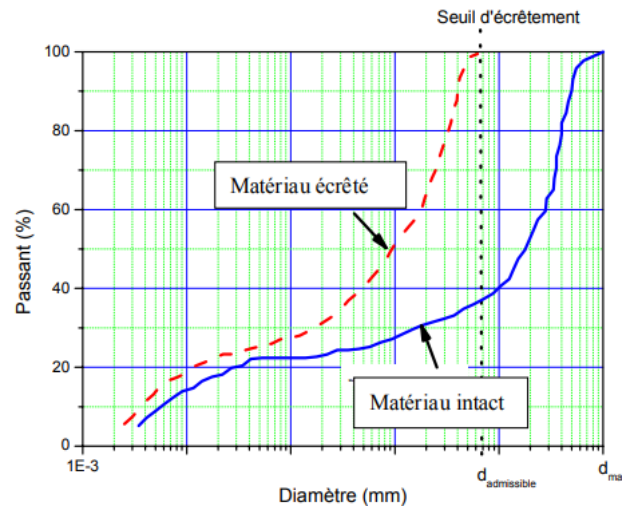


Figure 2.17: Illustration of the simple clipping method (Seif El Dine, 2007) [5]

A number of authors have studied the effect of this simple clipping method on the mechanical behaviour of coarse soils.

Pedro (2004) studied the mechanical behavior of a coarse soil composed of Fontainebleau sand and a mass fraction of gravel varying from one test to another [4]. The results show that capping, characterized by a decrease in the coarse

fraction, causes a clear decrease in the failure characteristics of coarse soils, i.e. internal friction angle and cohesion. This decrease of characteristics is all the greater as the proportion of clipped elements is greater. Its results are in line with the conclusions drawn from the work of Afriani (2002) and Valle (2001) which indicated a decrease in cohesion and friction angle at peak during tests with capped materials [3, 17]. This method therefore leads to a reduction in the shear strength of the coarse soil.

Gotteland *et al.* (2000) confirmed that capping induces a decrease of the characteristics at peak, with especially a strong decrease of the cohesion [21]. The important part of the coarse fractions brings a gain of resistance, while substituting them for the soil reduces shear strength.

Dano (2001) performed tests on an alluvial soil whose particle size distribution is shown in Figure 2.18. The author noted an increase in the angle of friction from $1,5$ to $2,1^\circ$ at peak state following the clipping of 10mm grains into particles of maximum size worth $6,3$ and $3,15\text{mm}$ [30].

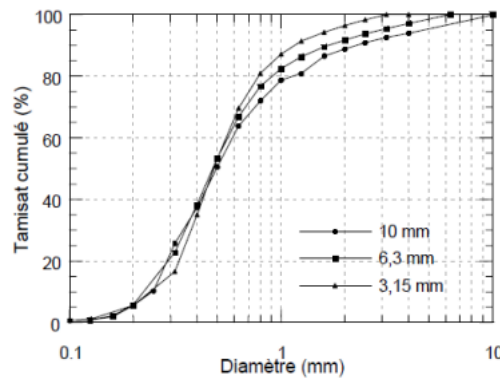


Figure 2.18: Particle size curves of the tested samples (Dano, 2001)[30]

Following these results, contradictory conclusions are established, showing the difficulty to characterize coarse soils with this method. This can be explained by characteristic elements of the soil studied, such as the type of soil, the clipping threshold used, the particle size distribution, etc. It is therefore complicated to conclude on the positive or negative impact of this method, as these studies show that simple clipping will modify the shear behavior of the soil. However, some authors agree that this method will decrease the mechanical characteristics of a coarse soil.

2.3.2 Parallel gradation modeling method

In this method, the soil elements tested will be equal to or smaller than the allowable particle size ($d_{max} \leq d_{adm}$) and the particle size distribution of the model soil will be similar to that of the real reference soil. According to Anagnosti (1982) [31], several conditions must be met to apply this method :

1. Respect of a similarity of the granulometric composition. The nominal grain diameter of the coarse material must be reduced by a constant ratio A :

$$A = \frac{d_{max}^r}{d_{max}^e} = \frac{d_{75}^r}{d_{75}^e} = \frac{d_{50}^r}{d_{50}^e} = \dots = \frac{d_i^r}{d_i^e} \quad (2.1)$$

d^r : grain size of the real coarse material

d^e : grain size of the model material

d_i : grain diameter corresponding to the relative percentage i of the total mass

2. Respect of a similarity on the porosity. This condition depends on the mode of distribution of the grains, to respect it the index of the voids of the coarse soil and the index of the voids of the model material must be identical.
3. Respect of a similtude of the representative volume. For this purpose, sufficiently large specimens must be chosen so that the behavior of the soil is not influenced by size effects. The representative volume is very complicated to determine. For each type of test or model, however, it is possible to determine minimum sample dimensions such that an increase in size does not change the strength parameters.
4. Respect of a similarity of the grain quality. This condition is based on the following parameters: deformability, resistance to fragmentation, resistance to relative displacement grain on grain by rotation and translation. This condition is complicated to respect. Indeed, it is difficult to obtain a similar quality between grains of small sizes and others of large sizes.

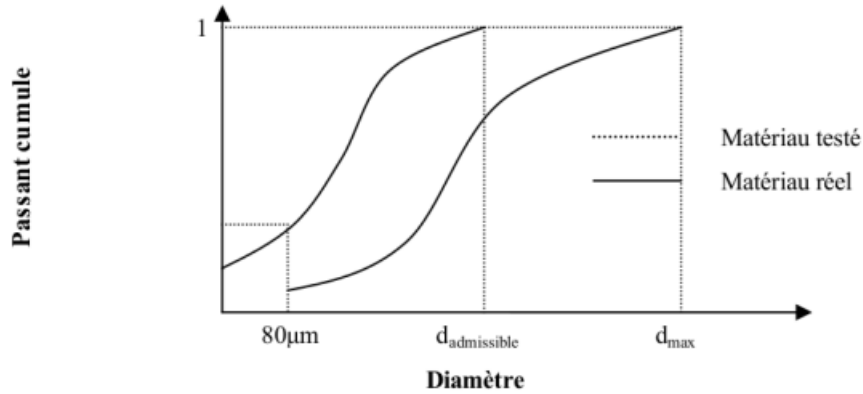


Figure 2.19: Illustration of the parallel gradation modeling technique (Seif El Dine, 2007) [5]

Using this modeling techniques, the following results have been obtained :
 Varadarajan *et al.* (2003) have shown that the shear strength increases with the size of the particles. The friction angle for alluvial materials increases with particle size, while the trend is reversed for blasting materials [32].
 Kouakou *et al.* (2020) wants to study the impact of fines content on the shear strength of a sample using the parallel grading method for peak state. For low fines contents, the shear strength of the initial coarse soil is well estimated. However, for higher and higher fines contents, although the friction angle is not impacted, this method leads to an overestimation of the cohesion [33].
 This good estimation of the friction angle can be explained by the fact that it depends on the inter-particle contacts. A condition to apply parallel grading is to respect similar arrangements between the grains.
 Other studies have noted that a change in soil structure occurs at a percentage of fines greater than 20 to 30%.

To ensure the variability of the results, this parallel grading method can be used up to a percentage of fines of 30%. Up to this value, the friction angle will be correctly estimated. As far as cohesion is concerned, a parallel grading could lead to an overestimation, especially because of the difference between the amount of fines between the initial and the reconstructed soil. Not all research results are consistent and divergent. Special attention should be paid to the results when using this method.

2.3.3 Application for tests carried out

To analyze the impact of coarse particles as well as the effect of size on the mechanical behavior of the reference coarse soil, two series of tests are performed. A first series using the simple clipping method and a second one using the parallel grading.

It will be seen in a future chapter (Section 3.1.2) that a ratio $\phi/d_{max}=6$ is often used to limit the size effects when forming samples based on the reference soil. It is therefore this ratio that will be used to cap reference soil. The particle size distribution curves are shown in Figures 2.20 and 2.21. Table 2.5 includes the physical characteristics of the samples.

Table 2.5: Physical characteristics of the samples

Specimen	Fine content (%)	d_{max} (mm)
38C	7,07	6,3
70C	5,97	10
100C	4,65	14
38P	17,84	6,3
70P	13,76	10
300	1,83	50

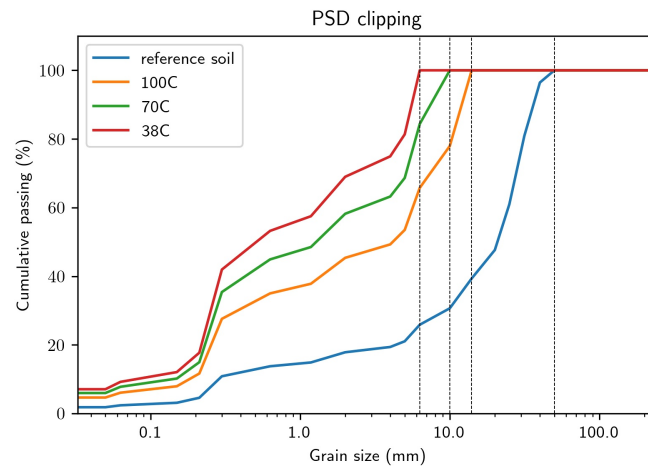


Figure 2.20: Particles size distribution for simple clipping method

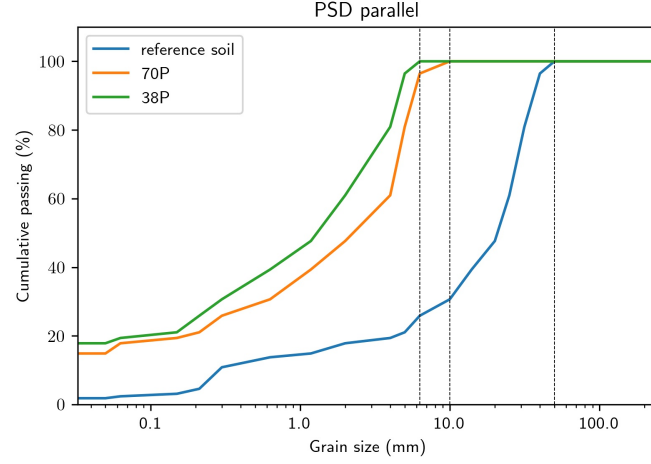


Figure 2.21: Particles size distribution for parallel gradation method

In the simple clipping method, the volume fraction of the inclusions will be decreased as well as the particle size distribution, which corresponds to a decrease in d_{min}/d_{max} . It is expected, as the authors point out in the section 2.3.1, a decrease in the global resistance of the soil. This method therefore tends to underestimate the shear strength of the soil. When parallel grading method is used, the percentage of fines increases, resulting in a variation in characteristics. The characteristics obtained from this method are overestimated.

It should also be noted that for the conduct of tests, several sieves were performed to separate the marine sediments to best reconstruct the samples formed by parallel grading method. Figure 2.14 shows that the percentage of fines in the marine sediments ($d < 63\mu m$) is very low. A replacement material for this portion of soil was used for samples 38P and 70P, because of the time consuming process to obtain enough fines by sieving. The material chosen is clay. Particular attention should be paid to the cohesion obtained during these tests. Indeed, this replacement will induce an additional cohesion compared to the reference soil.

For all sample, dry density ρ_d and the initial void ratio e_i are presented in Table 2.6. The name of the samples, composed of a number and a letter, corresponds to the diameter of the sample and to the method chosen for the clipping (C for the simple clipping and P for the parallel grading method). Because of timing constraints, the 100P sample could not be produced. The quantity of information with the tests carried out being sufficient to be able to draw conclusions thereafter.

Table 2.6: Dry densities and initial void ration for all sample

Sample	ρ_d (g/cm^3)	e_i
38Ca	1,762	0,504
38Cb	1,775	0,493
38Cc	1,772	0,495
38Pa	1,687	0,570
38Pb	1,696	0,563
38Pc	1,688	0,570
70Ca	1,768	0,499
70Cb	1,767	0,500
70Cc	1,768	0,499
70Pa	1,774	0,494
70Pb	1,775	0,493
70Pc	1,775	0,493
100Ca	1,770	0,497
100Cb	1,770	0,497
100Cc	1,770	0,497

3 | Experimental devices

In this chapter, the different devices used in the characterization of coarse soils to test samples according to their size will be presented. The first part focuses on the devices that allow to test large specimens and their impact on the shear strength of the tested soil. The second part will focus on the standard size devices used in the research carried out.

3.1 Coarse soils characterization

3.1.1 Large scale devices

It is essential to know the mechanical characteristics of coarse soils, such as their internal friction angle or their cohesion, when designing civil engineering structures. To study them two approaches are possible: the tests *in situ* and the laboratory tests. Different authors have studied these two approaches. Characterization through to tests *in situ*, requires the use of specific devices of very large dimensions. Tests execution and interpretation generate a relatively high cost. More information about this *in situ* characterization approach can be found in the research done by Pedro (2004) [4] or by Kouakou (2020) [22]. For these reasons, it is the tests carried out in laboratories that will be the subject of the research. Two main categories of devices are considered : the direct shear box and the triaxial apparatus.

The triaxial test will generally be preferred to evaluate the shear strength characteristics. Indeed, it allows to control the pore pressure during the whole test and to measure the axial deformation of the sample according to normal stress. This test is therefore very complete. The shearing box test, although easier to implement and less expensive, has the disadvantage of imposing a pre-determined horizontal shear plane and does not allow to control the pore pressures during the test. It is therefore necessary to be more critical and careful when analyzing the results [34].

Direct shear box test

This type of test consists in shearing a soil specimen according to a shear plane. The specimen is placed between two half boxes on which a normal force and a tangential force are applied. The following authors have used the direct shear box for their studies : Jain and Gupta (1974), Bourdeau (1997), Valle (2001), Afriani (2002), Reiffsteck (2007) and Kouakou (2020) [35, 36, 17, 3, 7, 22].

The size of the shear boxes can vary. For example Valle (2001) used two square boxes whose dimensions are $250 \times 250 \times 200mm$ and $500 \times 500 \times 300mm$; Afriani (2002) used these same devices for its studies ; Reiffsteck (2007) carried out his tests with a square box of $500 \times 500 \times 300mm$ (Figure 3.1) and another round one of $600mm$ of diameter and $600mm$ in height.

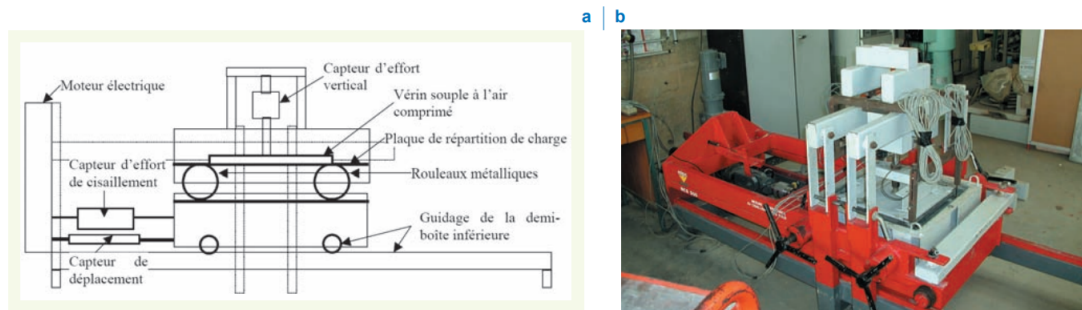


Figure 3.1: Direct shear box of square section of the CER of Rouen: - a : working principle operation; - b : general view (Reiffsteck, 2007)[7]

When using this type of device several difficulties are encountered. The first one concerns the roughness of the shear box. However, this inevitably leads to a reduction in the shear strength of the box. Bourdeau *et al* (1989) suggested to put a liner between the sample and the box. Another problem concerns the friction developed between the two half-boxes when they move horizontally. The weight of the two half-boxes causes significant forces. To limit these efforts some authors proposed to place in the middle of the boxes an element or a product in order to favour the sliding of the boxes [37].

Triaxial test

The principle of this test is to apply a uniform stress field to a cylindrical specimen. A schematic representation of the device is shown in the Figure 3.2.

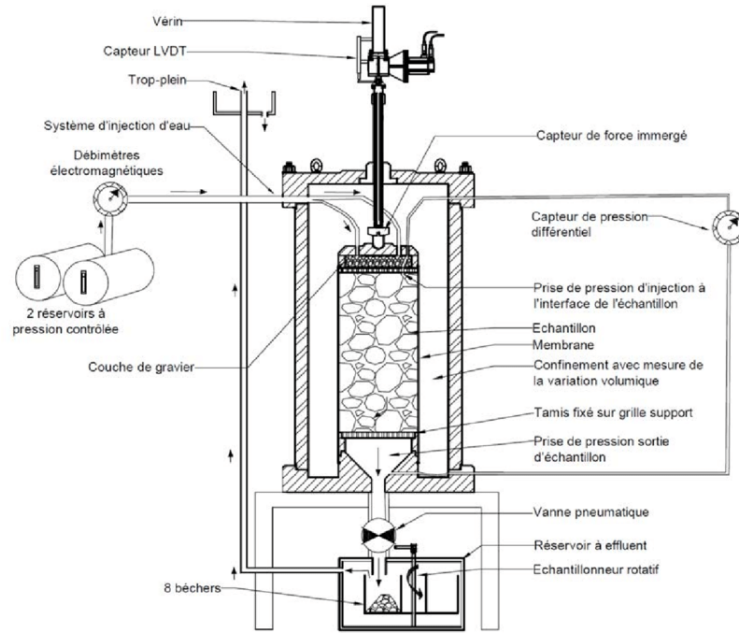


Figure 3.2: Schematic representation of the large triaxial device (Marot *et al.*, 2018) [38]

The main objective is to determine the relationship between the stresses applied to the test specimen and the deformations undergone. It will then be possible to determine the shear strength of the soil [39]. The use of triaxial devices with large induced dimensions is less limiting than the use of shear boxes. The big difference between small and large devices is the financial cost of such tests. This increase in cost is related to the device itself. Indeed these devices are prototypes and there are not yet many that are in working order. Also, a test of larger dimensions will require a longer preparation time and harder test procedures to perform. Large scale triaxial testing was implemented very early and continues to be improved today [5].

Several authors have performed triaxial tests on large devices. These include: Varadarajan (2003) [32], Pedro (2004) [4], Hu *et al.* (2011) [40] and Xiao *et al.* (2014) [41].

But also in particular Holtz and Willard (1961) who used a triaxial device of a diameter $\phi = 230mm$ and $570mm$ in height [18].

Valle (2001) carried out its experiments on a $152mm$ diameter device for a height of $304mm$ [17].

Seif El Dine (2007) studied the properties of coarse soils through tests on a triaxial of $300mm$ diameter and $600mm$ height [5].

Ovalle *et al.* (2020) gathered in an article a compilation of data obtained by triaxial tests on coarse materials. They analyzed these data according to the rupture of the particles, the shear strength of the materials, for different applied confinement pressures [42].

Device choice for tests performed

From informations found in literature, it can be seen that triaxial tests are less limiting than direct shear boxes. However, the "cost" parameter during the study is not negotiable. Moreover, with the use of large devices, it is not easy to guarantee the repeatability of the results obtained by these tests. An alternative solution is therefore to use standard size devices, using coarse soil clipping processes explained in Section 2.3 [37]. Standard-sized triaxial devices will be chosen for this study. They will be detailed later in the Section 3.2

3.1.2 Impact of the size of the triaxial device used

As mentioned earlier, there are different sizes of test devices and the soils studied will have narrow or spread out particle sizes depending on the study performed. The choice of the volume of soil tested will be limited by the dimensions of these devices. It is then necessary to consider a specimen whose volume is not too small compared to the size of the particles which compose it, that to avoid the "effect of scale". The size ratio between the diameter of the specimen used and the maximum diameter of the grains will be expressed as : ϕ/d_{max} .

Holtz and Gibbs (1956) tested specimens of $35mm$, $82,5mm$, $152mm$ and $229mm$. The specimens are composed of a mixture of gneiss gravels; granite and schist with $d_{max} = 19mm$ and a sand with $d_{max} = 4,76mm$, whose particle size curves are presented in Figure 3.3. It shows that a specimen whose diameter is 4 times larger than the maximum grain size is sufficient to avoid size effects and correctly determine the mechanical characteristics of coarse soils [2].

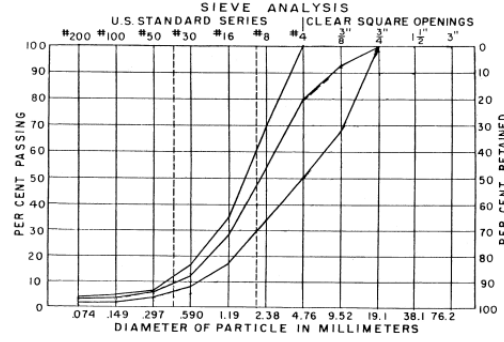


Figure 3.3: Particles size distribution of the studied materials (Holtz and Gibbs, 1956)[2]

However, Fukuoka (1957) showed that a small specimen ($\phi/d_{max}=4$) has lower rupture characteristics than a larger one ($\phi/d_{max}=6$) [43].

Fumagalli (1969) conducted tests with a 100mm specimen diameter using a limestone soil of different grain sizes. He noted a reduction in the angle of friction as the ratio ϕ/d_{max} increases (Table 3.1) [44].

Table 3.1: Influence of the specimen size (Fumagalli, 1969)[44]

ϕ (mm)	d_{max} (mm)	ϕ/d_{max}	φ (°)
100	20	5	46
100	12	8,3	44,3
100	10	10	44
100	4	25	42

The soil tested by Pedro (2004), is composed of Fontainebleau sand and 20% of 8/10mm gravel. To determine the potential size effect, he performed two triaxial tests. The first test on a medium size device ($\phi = 100mm$) and the second on a large size device ($\phi = 300mm$). He noted that the behaviour of the test specimens in terms of shear strength and volume variation remains the same whatever the size of the apparatus [4].

Reiffsteck *et al.* (2007) studied the effect of the size of the devices on a soil composed of natural gravel 0/80mm and skimmed to 25mm. Devices of size $\phi = 150mm$ and $\phi = 300mm$ were used. A decrease in cohesion of 10 kPa and a decrease of 4,6 ° for internal friction angle are observed when going from a ϕ 150mm to a ϕ 300mm. The ratio ϕ/d_{max} must be at least equal to 6 and the higher the material has a uniform gradation in size [7].

Hu *et al.* (2011) concentrated their research on Loire sand and will vary the size of the samples between 100, 500 and 1000mm. The ratio of height to diameter

was 1,5 for the largest sample and 2 for the other two, so these two samples will be compared. The peak behavior does not change with the size of the sample, the friction angles are identical. There is no size effect that has been observed. They then studied samples of limestone riprap whose H/ϕ ratio was constant and equal to 1,5. In this case, the shear strength did not decrease with the increase of the particle size, a behavior that goes against the previous observations of Marsal (1967) and Marachi (1969) [40].

To investigate the effect of specimen size on the mechanical behavior of granular soils, Omar and Sadrekarimi (2015) performed undrained monotonic compression tests on Ottawa sand. The specimens tested are 38, 50 and 70mm in diameter. The results showed that the behavior is affected by size : the larger specimens show less compressible isotropic compression behavior than the smaller specimens. Furthermore, the influence of size on the friction angles and shear strength are non-negligible, with an increase in the latter as the specimen size becomes smaller (Figure 3.4). The authors recommended the use of triaxial devices allowing specimens of dimensions $318 \times d_{50}$ ou $117 \times d_{max}$ for granular soils, d_{max} corresponding to the maximal particles's size [45].

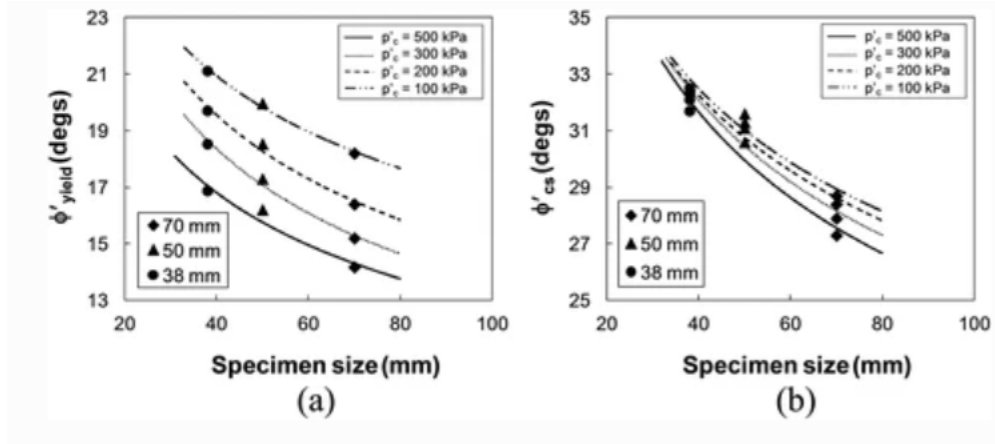


Figure 3.4: Effect of specimen size on (a) s_u (yield); (b) s_u (cs) (Omar and Sadrekarimi, 2015)[46]

Finally, the standard (Afnor NF EN ISO 17892-9, 2018) recommends, for triaxial tests, that the diameter of the specimen be greater than 6 times the maximum grain diameter. This is the size ratio used for testing.

3.2 Standard size devices

To study the behavior of coarse soils, triaxial devices of standard dimensions as announced in Section 3.1.1 are used. Granulometries studied are detailed in Section 2.3.3. Depending on these, different devices will be used. Standard size triaxial devices can be separate into two categories: the medium size devices ($\phi = 100mm$, $H = 200mm$ and $\phi = 70mm$, $H = 140mm$) and those of small sizes ($\phi = 38mm$, $H = 76mm$). In this section, these devices will detailed.

3.2.1 Medium scale devices ($\phi 100mm$ and $\phi 70mm$)

The test campaign on $\phi 70$ and $\phi 100mm$ devices allows to test specimens of respectively 140 and 200mm height for diameters of 70 and 100mm. For those size of test, an *Advanced Dynamic Triaxial Testing System* is used [47]. The device is showed in Figure 3.5. The only differences between the configuration of the $\phi 70mm$ and the $\phi 100mm$ are the dimensions of the topcap and pedestal which will be 70mm and 100mm respectively.



Figure 3.5: Triaxial device $\phi 100mm$ and $\phi 70mm$

The triaxial cell is equipped with a 25 kN force cell, allowing a vertical loading on the sample, with an accuracy of 0,1%. Moreover, the pore pressure and the cell

pressure are controlled by two separate controllers. These allow to measure the volume variations of the system, with a capacity of $200mL$. They are connected respectively to the topcap and to the base of the sample for the first one, and to the cell for the second one, allowing to apply a drainage by the top of the sample and a counter pressure to this one, and a confinement pressure in the cell. The displacement is measured during the entire test in order to measure the length variation of the sample, allowing to evaluate the axial deformation of the sample ε_a . The displacement range is $100mm$ with a resolution of $0,2\mu m$. All measurements taken during the test are displayed through a GDSLab software that allows a follow-up of the test in progress.

3.2.2 Small scale device ($\phi 38mm$)

For the smaller test, a device who accept sample of $38mm$ diameter and $76mm$ height is used. For this size of test, a typic *Dynamic Triaxial Testing System* (*TriSCAN Pro* [48]) is selected. The device is presented in Figure 3.6



Figure 3.6: Triaxial device $\phi 38mm$

The triaxial cell is equipped with a $5kN$ force cell, allowing a vertical loading on the sample. Moreover, the pore pressure and the cell pressure are controlled by two separate controllers. These devices are VJ Tech. These allow to measure the volume variations of the system, with a capacity of $200mL$ and the pressure is applied with an accuracy of $0,1\%$ [49]. VJ TEch are connected respectively to the topcap and to the base of the sample for the first one, and to the cell for the second one, allowing to apply a drainage by the top of the sample and a counter

pressure to this one, and a confinement pressure in the cell. The displacement is measured during the entire test in order to measure the length variation of the sample, allowing to evaluate the axial deformation of the sample ε_a . The displacement range is $100mm$ with a resolution of $0,2\mu m$.

4 | Test methods

To be able to characterize a coarse soil thanks to standard size laboratory equipment, the use of a clipping procedure was necessary. For the two possibilities considered, the same method was applied and the followed procedure for the fabrication of the sample is identical. Firstly, the formation of the sample using Ladd's undercompaction method and its implementation up to the launch of the test will be explained. Then, the stages of this test will be detailed and the choices concerning the different confinement pressures applied will be explained. Finally, details about how the data recovered after each test was processed.

4.1 Ladd's undercompaction method

The Ladd's undercompaction method (Ladd, 1978) describes the preparation method of recompacted sand and silt specimens [50]. The concept of undercompaction acknowledges that as subsequent layers of soil are compacted, the layers beneath will also experience further compaction. To achieve a uniform final density for each layer, the initial layers are compacted to lower densities compared to the following layers. This method allows to have a more uniform density throughout the specimen, allowing a good repeatability of the results [51].

4.1.1 Test method

Saturation ratio

The initial degree of saturation of the compacted material must be between 20 % and 70 %. To achieve this, the water content of the air-dried material must be adjusted. As show in Table 4.1, materials that have a lower amount of fines necessitate a lower level of saturation.

Table 4.1: Recommended water content based on particule size

Particle size majority	Recommended water content
Fine silt	16%
Medium silt	17%
Coarse silt	12%
Fine sand	10%
Medium sand	8%
Coarse sand	6%

To perform the tests, the water content determined during the tests carried out previously is used (see Appendix D). This value is considered as the one obtained at the proctor optimum. It only concerns the part of the sample composed of particles smaller than $2mm$ and is equal to 12% of this mass. The mass of the grains smaller than $63\mu m$ being not negligible, this means that the soil is composed in part of silt, the value of 12% is therefore a value that is expected from the Table 4.1 The initial water content of the sample and the saturation ratio are show in Tables 4.2 and 4.3.

Table 4.2: Degree of saturation as function of sample size

	ϕ 38mm	ϕ 70mm	ϕ 100mm
Simple clipping method	37,35 %	31,29 %	24,46 %
Parallel gradation modeling method	30,68 %	21,47 %	-

Table 4.3: Water content as function of sample size

	ϕ 38mm	ϕ 70mm	ϕ 100mm
Simple clipping method	6,96%	5,83%	4,56%
Parallel gradation modeling method	5,74%	3,7%	-

Table 4.2 shows that the saturation's degree is between 20 % and 70 % for all samples. Table 4.1, shows the water content for all sample sizes, taking into account the mass of water added in proportion to the mass of particles smaller than $2mm$.

In the Ladd's undecomposition method, only the particles of diameter lower than $2mm$ are used. Here the particles used correspond to $d_{max} = \frac{\phi}{6}$ (Section 3.1.2). Considering also the available sieves, the value in Table 4.4 for the maximal diameter of the particles, corresponding to each diameter of the sample is obtained.

The mixture with all the particles and water is placed in a sealed plastic bag during one night to be sure to obtain a constant water content throughout the sample.

Table 4.4: Particle size according to the diameter of the samples

Sample size [mm]	$d_{max,real}$ [mm]
38	6,3
70	10
100	14

Initial undercompaction value

In the undercompaction method, the compaction values of each layer are influenced by the layers above. Starting from the subcompaction value of the first layer, the subcompaction values of the layers will be calculated as a function of the density of the first layer. The compaction parameter is indicated by U_{ni} in each layer i and is given in the equation (4.1). In this equation, U_{n1} and U_{nt} represent the compaction parameter for the first layer at the bottom and the last layer at the top respectively [50].

$$U_{ni} = U_{n1} - \frac{U_{n1} - U_{nt}}{n_t - 1} \cdot (n_i - 1) \quad (4.1)$$

The value of 5% is used for the first layer undercompaction. This value is in the range between zero and 15%, for dense and very loose samples respectively.

Compaction

To begin this step of the method, there are have two possibilities :

- Shake the bag very well then place the required masses for each layer in separate sealed plastic bags;
- Keep the entire sample in one bag and collect the required mass as you go

The whole sample is kept in one bag to avoid the risk of losing some part of the sample by separating it in several bags. For the preparation of the specimen, an external split mould is used to compact it. This kind of mould allows a better control of the sample preparation. The number of theoretical layers to be made will depend on the height of the sample. To calculate it, the height of the sample is divided in millimeter by 15mm. The theoretical value for the umber of layers is showed in Table 4.5. A lower number of layers is applied because if the recommended one is

made, some of the larger particles would have tended to be poorly distributed in the specimen. The number of layers for each specimen size is referenced in the Table 4.5.

Table 4.5: Number of layers depending of the sample size

Diameter [mm]	Height [mm]	Theoretical number of layers	Effective number of layers
38	76	5	3
70	140	9	6
100	200	13	10

The height of each sub-layer h_i can be defined as in equation (4.2). The total height of the sample is defined by h and the total number of effective layers by n . This sub-height varies according to the compaction parameter defined in the equation (4.1).

$$h_i = \frac{h}{n} \cdot \left((n_i - 1) + \left(1 + \frac{U_{ni}}{100} \right) \right) \quad (4.2)$$

4.2 Sample set-up

Samples are prepared following a number of predefined steps. Figure 4.1 illustrates these steps. All of them are applicable for the ϕ 100mm and ϕ 70mm samples. For the ϕ 38mm, all the steps are followed in this order with the exception of the demolding of the sample (step 4) which is done directly after the compaction (step 1).

1. Compaction of the sample in a split mold using the Ladd's undercompaction method describes in Section 4.1.
2. Installation of a porous stone and a paper filter (preventing the fines from circulating through the porous stone) on the bottom side of the sample ;
3. Placing a second porous stone and filter paper on the upper side of the sample ;
4. Delicate demolding of the sample onto the pedestal of the triaxial cell ;
5. Fixing a latex membrane around the sample by sliding it from above. This will be fixed with O-rings on the side of the lower base, placed thanks to a metal split mold, of diameter larger than the sample itself. The membrane will be folded over the joints ;

6. Restarting the last step as the grains are relatively large, two membranes are placed for safety ;
7. Folding the top part of the membrane to place the topcap, which will allow the drainage of the top of the sample, and placing two O-rings on each membrane ;
8. Placing the triaxial cell on the lower base, tightening the bolts evenly and making the contact of the loading piston with the topcap ;
9. Filling the cell with water (be careful to open the purge to allow the air in the cell to escape) ;
10. Connecting the sources of cell pressure, back pressure and measurement sensors, and placing the valves in the right positions ;
11. Application of a low cell pressure of 50 kPa to hold the sample.

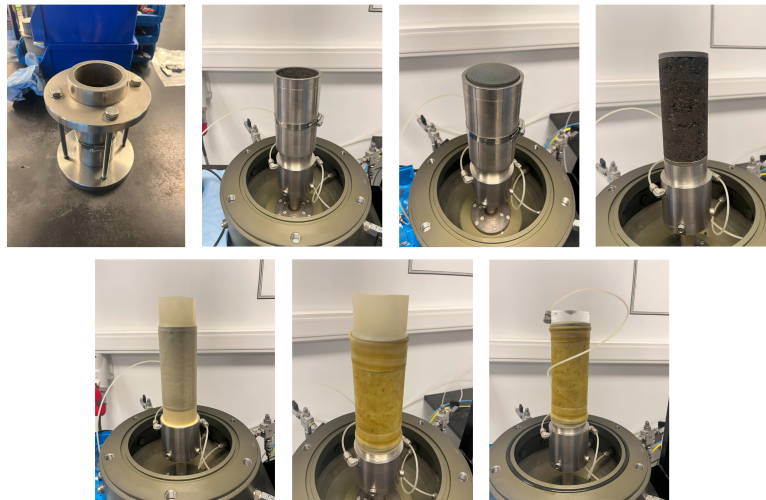


Figure 4.1: Formation of sample

4.3 Stages of a triaxial test

Drained isotropic consolidated tests were performed on the materials defined earlier in Section 2.3.3. In this section, the different stages of the triaxial test are introduced, along with the way in which they are applied to soil characterisation [39, 52, 53, 54, 55].

4.3.1 Saturation

The saturation refers to the process of completely filling the test specimen with the saturating fluid, which is usually demineralized and deaerated water. During this stage, water is introduced into the sample from the bottom of this one. This is an important step because it ensures that the measured mechanical properties are representative of the in-situ state of the sample ; the unsaturated behaviour can be significantly different from that of saturated soils.

This stage can be separated into four sub-steps :

1. **CO_2 flush** : This step is used to expel the air present in the sample by circulating at low pressure carbon dioxide CO_2 . This will accelerate the saturation process. Indeed, CO_2 is much more soluble than air in water, so it will prevent contamination and eliminate residual air that could affect the test results. This step takes 20 minutes to make sure that the CO_2 has replaced all the air in the sample,
2. **H_2O flush** : The sample is saturated from bottom to top with deaerated water with a pressure of 40 kPa. The objective is to expulse the CO_2 , but also to ensure a uniform test environment. This is a rapid stage that ends when no more water appears. By performing this step every time, remaining CO_2 and impurities are removed and test conditions are kept constant, the results obtained are more accurate and reliable,
3. **Application of a back-pressure** : This step allows to improve the saturation of the specimen. Applying back pressure is equivalent to compensating for high pore pressure by applying high cell pressure. This is not an instantaneous phenomenon, the different pressures are increased by increments, with open drainage. The final value of the pore pressure must be at least 300 kPa. The cell pressure must be increased so that the effective stress applied prevents swelling but does not exceed the value required at the end of consolidation.
4. **Skempton's coefficient** : The Skempton's coefficient is defined as the ratio between the induced pore pressure to the change of stress loading under undrained condition (Skempton, 1954 ; Wang, 2000) [56, 57]. After cutting the drainage of the saturation stage, a cell pressure variation $\delta\sigma_c$ of 50 kPa is applied, then the variation of the pore pressure Δu as function of time is measured. Equation (4.3) is obtained:

$$B = \frac{\Delta u}{\Delta \sigma_c} \quad (4.3)$$

If the B-value is lower than 0,95, the back pressure must be increased until a satisfactory criterion will be obtain. This increase is done in steps between 50 and 100 kPa. In general the criterion is respected or not for a time variation of 10 minutes at each pressure increase.

To validate the saturation stage, 2 criterions have to be checked:

- The value of the B-coefficient is bigger than 0,95 as said before.
- The pore pressure is steady and its graph shows a plateau. Whereas the non-saturation does not allow it and a leak causes a fall of this pressure.

4.3.2 Consolidation

Consolidation is an important phase in a triaxial test. It is the dissipation of pore water pressure and thus caused particle rearrangement. This one affects the stability of the soil and its deformations. This can cause problems in construction projects, leading to instability or damage to structures. With the development of civil engineering projects, more and more structures are built on low quality soils. It is therefore necessary to understand the consolidation process of soils and how their behavior will be affected during civil engineering projects [58].

Two kind of consolidation are possible : anisotropic and isotropic. The first one is more realistic because it takes into account the non-uniformity of the pressure in the soil in real situation, especially in the case of anisotropic soils. The second one is easier to implement because of the uniformity of the pressures to be applied to the sample [54, 59]. As part of this study, the focus will be on the isotropic consolidation.

Consolidation consists of applying in a progressive and controlled way an isotropic cell pressure during a given time, keeping the back pressure defined in the saturation step constant. That refers to the process by which the voids between particles are eliminated and therefore the particles are tightened. This fixed back pressure allowing the water to escape through the sample by porous stones, causing the soil to become denser and stronger.

Determination of the effectives pressures

The choice of consolidation stresses will depend on the stresses acting on the soil element. They must therefore take into account the weight of the overlying soil layers or the stresses generated by the structures that are being considered and

covered a certain interval allowing to define cohesion c and the internal friction angle φ with a good accuracy. Whatever the size of the sample, the set of three effective constraints applied to it will be the same. The ones chosen are as follows in the Table 4.6. The samples will be named according to the constraint a, b or c.

Table 4.6: Effective constraints

Sample	σ' [kPa]
a	50
b	100
c	200

To determine their value, the height of the embankment is assumed to be between 5 and 10m ; 10m corresponds to the limit of large embankments according to *Guide technique "Réalisation des remblais et des couches de forme"* [60].

A cell pressure must be applied such that the difference between it and the back pressure of the specimen is equal to the specified effective stress σ' .

Volume change

During the consolidation, the volume change is measured thanks to the measures of specimen's expelled water volume, this process can take a significant amount of time to complete, depending on the characteristics of the soil and the applied pressure.

Consolidation is considered complete when the change in volume of the sample is less than 0,1% of its volume per hour or $0,1\text{cm}^3/h$. The additional deformations are minimal and will therefore be negligible. The choice of the criterion is made according to the highest value and therefore the strictest. When consolidation is over, strength and density of the soil are higher and the void ratio is lower. The corresponding acceptable volume variations for the tests are as follows in Table 4.7, according to the standard applicable in Fugro laboratories. These are the values considered for the realization of the tests.

Table 4.7: Acceptable volume variation

Sample size [mm]	δv [cm^3/h]
38	0,02
70	0,11
100	0,31

4.3.3 Shear

Shearing is the final step of a consolidated drained triaxial test. This one allows to measure the mechanical properties of the sample. The cell pressure and the back pressure have to be constant. A desired constant shearing velocity v_{max} is applied (Equation (4.4)), which will lead to a variation in axial stress.

Shearing rate is calculate by :

$$v_{max} = \frac{(h_0 - \delta h) \cdot a}{60} [mm/min] \quad (4.4)$$

With :

- h_0 : initial height of the specimen
- δh : height variation of the specimen
- a : coefficient depending of the type of soil, 1, 3% for a sandy soil

The mechanic response of the specimen to this shearing rate will be recorded in terms of stress and strain. A 22% limit is imposed on axial strain. The test ends when this limit is reached. During this phase, it is important to select a good data recording frequency. The data obtained during the shearing can be used to determine the shear strength of the test sample. The focus is on cohesion and internal friction angle.

If a set of three consolidation stresses are applied with the same failure criterion and homogeneous specimens, it is possible to obtain the cohesion and the internal friction angle of the sample by the Mohr's circle. The equation of the mohr coulomb line is given by Equation (4.5). An illustration of the Mohr's Circle and Mohr-Coulomb line are showned in Figure 4.2 :

$$\tau = c' + \sigma' \cdot tg\varphi' \quad (4.5)$$

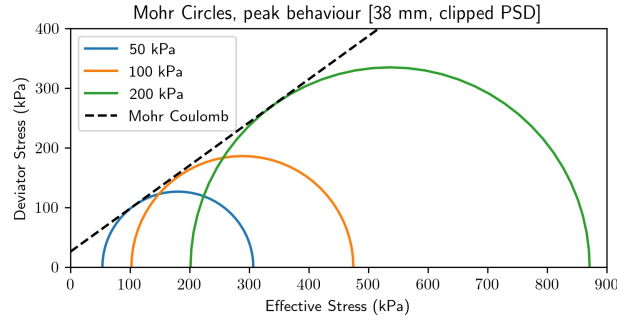


Figure 4.2: Exemple of Mohr's circles and Mohr Coulomb line

4.4 Data preprocessing

This section will detail the processing of the data obtained during different tests. Two stage of the test have been studied : the consolidation phase and the shearing one. Data collected during the two stages are gathered in Table 4.8. The initial height, diameter, mass of the sample and the particle density are know quantities.

Table 4.8: Recorded data

Variable's name	Variable	Units
Time of the stage	t	s
Cell pressure	p_c	kPa
Back pressure	p_b	kPa
Pore water pressure	p_w	kPa
Back volume	v	mm^3
Axial displacement	d_a	mm
Load cell	F	N

Data processing is based on the reference book "*Soil Behaviour and Critical State Soil Mechanics*" [13]. Table 4.9 contains all the data to be graphed and used in the analysis of results.

Table 4.9: Processing data

Variable's name	Variable	Units
Initial specimen height	h_0	mm
Initial specimen diameter	d_0	mm
Final specimen height	h_f	mm
Final specimen diameter	d_f	mm
Dry density	ρ_d	g/mm^3
Particles density	ρ_p	g/mm^3
Initial specimen volume	v_0	mm^2
Final specimen volume	v_f	mm^2
Volume change	Δv	mm^3
Deviator stress	q	kPa
Pore water pressure	u	kPa
Total stress	σ	kPa
Radial effective stress	σ'_r	kPa
Axial effective stress	σ'_a	kPa
Mean effective stress	p'	kPa
Axial strain	ε_a	%
Volumetric strain	ε_v	%
Radial strain	ε_r	%
Shear strain	ε_γ	%
Internal friction angle	ϕ	°
Void ratio	e	-
Stress ratio	η	-
Specific volume	ν	-
Dilatance	D	-

Some relations below are applied to stress or strain increments. This increment is represented by δ .

In relation (4.6), the negative sign is required because compressive strains are taken as positive in convention.

$$\delta\varepsilon_v = \frac{-\delta v}{v} \quad (4.6)$$

The area in relation (4.7) is not the corrected one. Any such correction is ignored for the tests realised.

$$\delta q = \frac{\delta F}{A} \quad (4.7)$$

In compression test, the vertical axial stress σ_a is the major principal stress σ_1 and the horizontal radial stress σ_r is the minor and intermediate principal stress such

that $\sigma_2 = \sigma_3$. Thanks to the relation of the total stress 4.8, this gives the following relations (4.9), (4.10) and (4.11).

$$\sigma' = \sigma - u \quad (4.8)$$

$$\sigma'_r = p_c - p_b \quad (4.9)$$

$$\sigma'_a = \sigma'_r + q \quad (4.10)$$

$$p' = \frac{\sigma'_a + 2\sigma'_r}{3} \quad (4.11)$$

$$\eta = \frac{q}{p'} \quad (4.12)$$

$$\delta\varepsilon_a = \frac{-\delta d_a}{h_0} \quad (4.13)$$

$$\varepsilon_r = \frac{\varepsilon_v - \varepsilon_a}{2} \quad (4.14)$$

$$\varepsilon_\gamma = \frac{3 \cdot \varepsilon_a - \varepsilon_v}{2} \quad (4.15)$$

The dilatancy, see relation (4.16), is the ratio between the two work conjugate strain increment invariants. It is the change in volumetric strain over the change in shear strain [13].

$$D = \frac{\dot{\varepsilon}_v}{\dot{\varepsilon}_\gamma} \quad (4.16)$$

$$e = \frac{\rho_p}{\rho_d} - 1 \quad (4.17)$$

$$\nu = 1 + e \quad (4.18)$$

At the end of each phase, the final volume variation, area, mass, height and diameter, as well as density and void ratio, can be calculated. When shearing begins, the final data from the consolidation phase will be considered as the initial data.

5 | Results and discussions

As mentioned in Section 2.3, the study focuses on two different clipping methods. The reconstituted soils will therefore exhibit different mechanical behaviors from the reference soil. In this chapter, the focus is on the impact of the clipping method and sample size on the results obtained from the experimental study. Analyses will be carried out first for the consolidation phase and then for the shearing of the material.

All the data about volume and void index for the consolidation and shearing phases are presented in Table 5.1.

Table 5.1: Tests values for all samples

	v_i (cm^3)	ρ_d (g/cm^3)	e_i (-)	Δv_{conso} (g/cm^3)	v_{conso} (cm^3)	e_{conso} (-)	Δv_{shear} (cm^3)	v_f (cm^3)	e_f (-)
38Ca	86,19	1,76	0,50	0,62	85,57	0,49	-0,36	85,93	0,50
38Cb	86,19	1,77	0,49	1,22	84,97	0,47	1,10	83,87	0,45
38Cc	86,19	1,77	0,50	1,90	84,29	0,46	2,25	82,04	0,42
38Pa	89,03	1,69	0,57	1,47	87,55	0,54	0,58	86,98	0,53
38Pb	89,60	1,70	0,56	4,40	85,20	0,49	1,92	83,28	0,45
38Pc	89,03	1,69	0,57	4,68	84,35	0,49	2,69	81,66	0,44
70Ca	539,71	1,77	0,50	6,12	533,59	0,48	10,01	523,58	0,45
70Cb	539,71	1,77	0,50	8,01	531,70	0,48	14,13	517,58	0,44
70Cc	539,71	1,77	0,50	14,79	524,92	0,46	20,35	504,57	0,40
70Pa	539,71	1,77	0,49	16,39	523,32	0,45	12,53	510,78	0,41
70Pb	539,71	1,78	0,49	25,32	514,39	0,42	13,57	500,81	0,39
70Pc	539,71	1,77	0,49	22,41	517,30	0,43	18,97	498,33	0,38
100Ca	1567,73	1,77	0,50	17,19	1550,55	0,48	39,77	1510,78	0,44
100Cb	1567,73	1,77	0,50	41,75	1525,99	0,46	71,79	1454,20	0,39
100Cc	1567,73	1,77	0,50	58,88	1508,86	0,44	87,62	1421,23	0,36

5.1 Volume change during consolidation

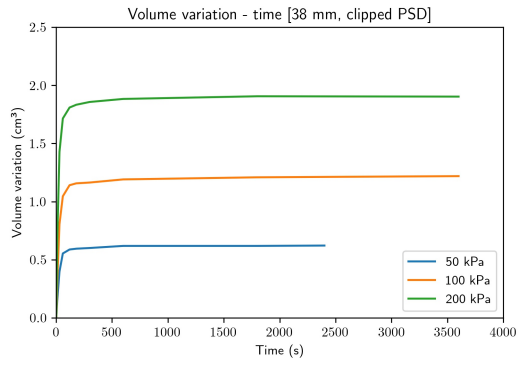
In the consolidation phase, volume change is analysed. This will influence the final void ratio of this phase, which will be taken as the initial value of the shear phase.

As previously stated (Section 4.3.2), the stopping condition for the consolidation phase is such that the change in sample volume must be less than 0,1% of its volume per hour. Volume stabilization is expected for each specimen. Figure 5.1 below show these volume variations. On these graphs, the stabilization is clearly visible for the majority of samples. For 70Cc (200 *kPa*) and 70Pb (100 *kPa*), this is less distinctive. Data analysis reveals volume variations of 0,066 and 0,051 cm^3/h respectively for these samples, which is within the acceptable range of variation. This variations correspond to $1,2 \times 10^{-4}$ and $9,4 \times 10^{-4}$ initial volume percentage per hour.

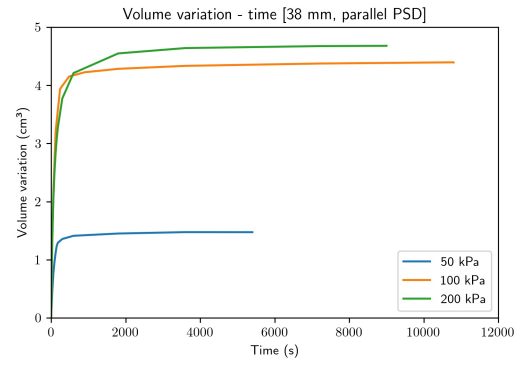
For each series produced using the simple clipping method, the volume change curves show the same trend. The volume change is greater and greater as the confining pressure and sample size increase. In addition, the curves do not cross. However, for the parallel gradation modeling method, the volume change curves for stresses of 100 kPa and 200 kPa intersect for both sample sizes. The volume change for the 70Pc sample is smaller than for the 70Pb.

By comparing the change in volume for samples of the same dimensions according to the type of clipping, it can be seen that volume change is bigger when using the parallel clipping method than when using simple clipping.

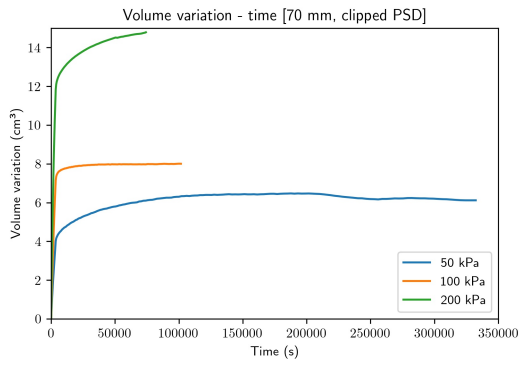
Columns e_i and e_{conso} in Table 5.1 showing the initial voids index and that one after the consolidation phase show a decrease in this index.



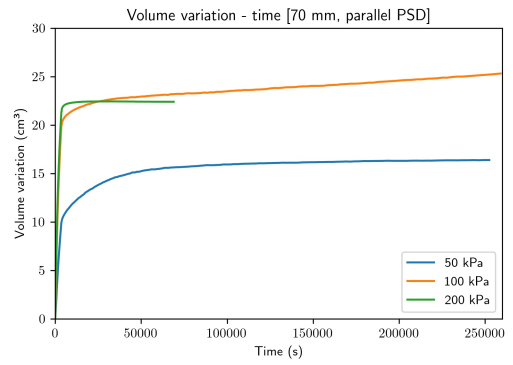
(a)



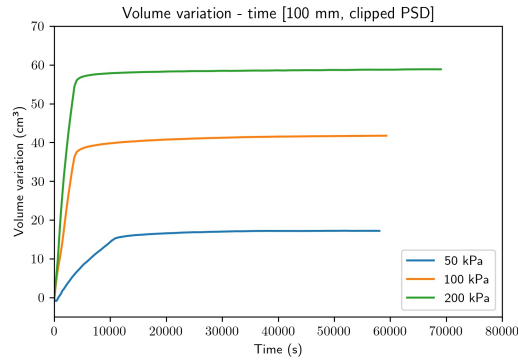
(b)



(c)



(d)



(e)

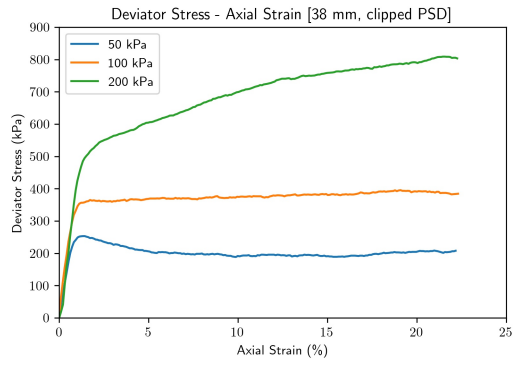
Figure 5.1: Volume change during consolidation for samples : (a) 38C ; (b) 38P ;
(c) 70C ; (d) 70P ; (e) 100C

5.2 Deviator stress - axial strain

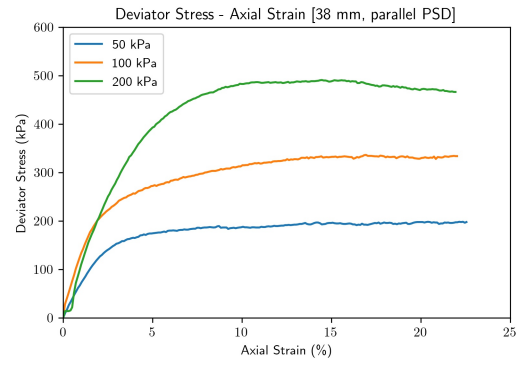
Drained consolidated triaxial tests were carried out for 3 different confining pressures of 50, 100 and 200 kPa to obtain peak and end shear strengths. The initial moisture content depend on the mass of particles less than 2mm (see Table 4.3 in Section 4.1.1).

Figure 5.2 show the plots of the axial strain and deviatoric stress depending on the confining pressure, the sample size and the clipping method. Compression is assumed to be positive. On the graph concerning 70P, it shows that the test under a confinement stress of 100 kPa (test 70Pb) stopped before the 22% axial deformation criterion was reached. This early stop may be due to an error in encoding the test stop conditions or a technical problem in the laboratory. The end-of-test value will therefore not be taken from the same moment of the test as for all the others. Particular attention will therefore be paid to the results.

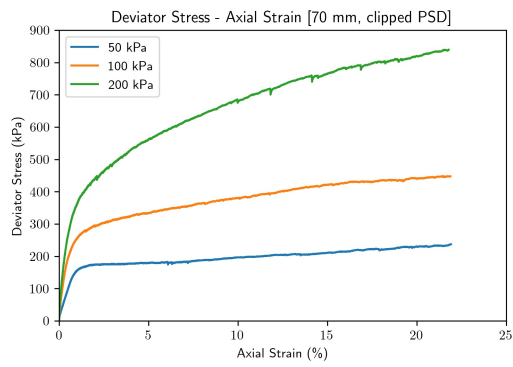
When a significant increase or decrease in stress is observed simultaneously with an increase in strain, the phenomenon is called strain hardening or softening behaviour [61]. To get an idea of the density of the soil and its state of failure, the stress-strain curve can be observed. Dense sandy materials show a peak in strength followed by a horizontal asymptote. This is reflected in a void ratio that decreases slightly, then increases towards a critical void ratio value. Sample 38Ca shows this behaviour. Void ratio after the shearing is smaller than just after the consolidation (Table 5.1). Tests on initially loose soil reflect contracting behavior, with the void ratio decreasing towards a critical value. The stress in a loose soil will increase until it reaches an asymptote [62]. All the sample except 38Ca are loose soil. This is confirmed by the value of the void ratio, which are smaller at the end of the test.



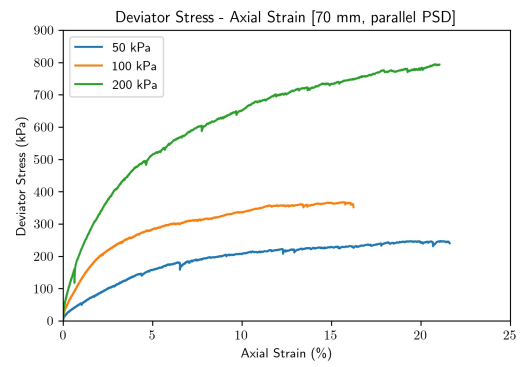
(a)



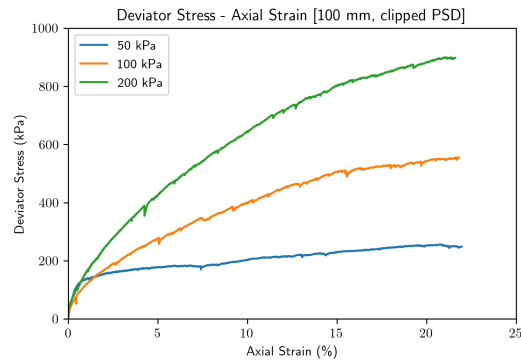
(b)



(c)



(d)



(e)

Figure 5.2: Deviator stress depending on the axial strain for samples : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C

5.2.1 Analysis of strength parameters according to the graph deviator stress - axial strain

The cohesion (c) and the internal friction angle (φ) were determined according to the Mohr-Coulomb theory [63]. All graphs showing Mohr's circles can be found in the Appendix E and the data obtained about the internal friction angle and cohesion are presented in Table 5.2.

Table 5.2: Cohesion and internal friction angle for all specimens

Specimen	At peak		At the end	
	c [kPa]	φ [°]	c [kPa]	φ [°]
38C	13,08	41,15	0	41,48
38P	30,71	29,31	30,16	28,49
70C	6,50	42,01	0	43,17
70P	11,67	40,34	0	41,58
100C	6,38	43,75	0	43,75

At peak

Peak values are taken where deviatoric stress is greatest. The data obtained are showed in Figure 5.3.

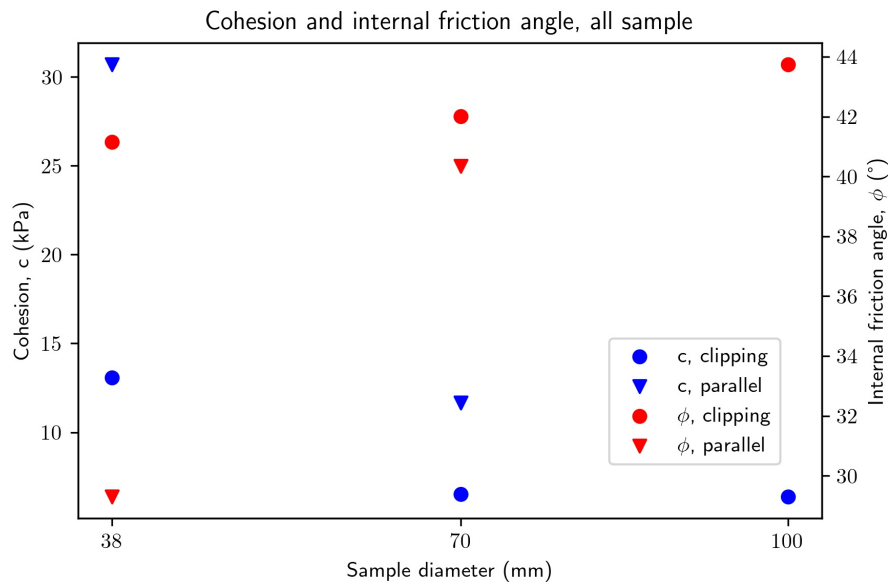


Figure 5.3: Cohesion and internal friction angle at peak for all specimens

In conventional soil mechanics, a sandy soil is considered non-cohesive and has no cohesion. The cohesion present in the tests can be explained in several ways. On the one hand, the reference soil tested is made up of particles finer than sand, which can bring cohesion to the soil. Secondly, when there is water in the sand, the pores shrink under the negative pressure of capillary water, creating cohesion. This cohesion is called apparent cohesion, as it disappears with increasing water content in the sample [64].

Cohesion is greater when the parallel gradation modeling method is used. This method will tend to overestimate cohesion because the percentage of fines is higher in the reconstituted soil than in the reference soil. For sample 38P, this value is very high at 30,71 kPa. The fines content is almost 18%, compared with less than 2% for the reference soil. Lower cohesion will be observed as the percentage of fines decreases, with increasing sample size. With the simple clipping method, cohesion decreases by 50% of the maximum value when the diameter increases from 38 to 70 mm. When the parallel grading method is used, the decrease is 62%. Cohesion between 70C and 100C samples hardly varies at all, with a variation of less than 1 kPa, which corresponds to a 1,8% reduction in the value achieved for sample 70C, and can be expected to stabilize for larger and larger samples.

As far as the internal friction angle is concerned, it is much lower for 38P. This is unusual behaviour as the parallel gradation modeling method maintains intergranular contacts, on which the angle of friction depends (see Section 2.3.2). For all the other samples, the value of the internal friction angle is between 40,34° and 43,75°. The difference between the minimum and maximum values is only 3,41°. Considering the 38P sample, this difference between the extreme values rises to 14,44°. For samples 38C, 70C and 100C, the internal friction angle increases with their diameters, with an increase of 2,6° between the smallest and the largest. This increase, visible in Figure 5.3, has a linear trend. Between samples 38C and 70C and between samples 38P and 70P, there is an increase in the angle of internal friction of 0,86 and 11,03 ° respectively. Which corresponds to an increase of just 2,04% on the maximum value for the simple clipping method, while the second method accounts for an increase of 27,34%. It can be seen that the parallel grading method has a greater impact than the simple clipping method as the diameter of the sample increases.

The impact of increasing the diameter of the sample is greater on cohesion than on friction angle. In addition, the variations for simply clipping samples are less significant than when the parallel grading method is used.

End state

The residual behaviour of the soil at the end of the shear period, when the deformation reaches 22%, will be analysed. To calculate the angles of friction and cohesion, the last value of the deviatoric stress is used. Mohr's circles are shown in Appendix E (Figure 8.3). Mohr-Coulomb line will be drawn as tangent as possible to as many circles as possible. This line is a good approximation because it is tangent to most of the 3 circles except in the case of sample 38P where one of the circles is not taken into account at all. The residual angle of internal friction and the cohesion can then be determined.

For all samples except 38P, the cohesion in the end state is zero. The angle of friction for both clipping methods with increasing sample size is observed.

If the two clipping methods are compared, the difference between the values for the angle of friction of samples 38C and 70C is much smaller than that between samples 38P and 70P. A difference between the minimum value corresponding to the 38 mm diameter and the maximum value for the 70 mm diameter of 31,48% of the maximum value for the parallel grading method and a difference of 3,94% for the simple clipping method for the same two samples has been found.

Comparison between the two states

With the exception of samples 38P and 100C, the values for the angle of friction at the peak are smaller than those in the end state. However, a maximum variation of $1,24^\circ$ is observed for specimens 38C, 38P, 70C and 70P. For sample 100C, the value of the angle of internal friction does not vary. This can be explained by the fact that the maximum value of the deviatoric stress is close than the value of the stress when the axial strain is 22%, so the Mohr circles will be almost similar.

5.2.2 Volumetric strain - shear strain curves

The variation in volume during the shear phase is linked to the phenomenon of dilatation and contraction. Dilatancy represents the variation in volume deformation since the initial state. Generally, if the soil is initially dense, its volume will increase during the shear phase. However, the soil may be compacted at the start of this phase. If, on the other hand, the soil was initially loose, it will tend to contract.

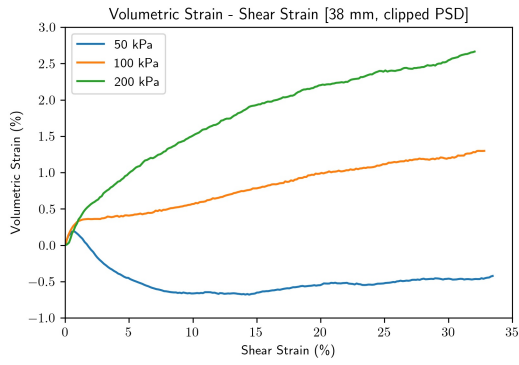
The evolution of the volume can be linked to the volumetric strain. This strain as a function of shear strain is shown in Figure 5.4. In this study, compression is

considered as positive. Several trends are observed depending on the size of the samples, the confinement constraints and the type of clipping used to make the sample.

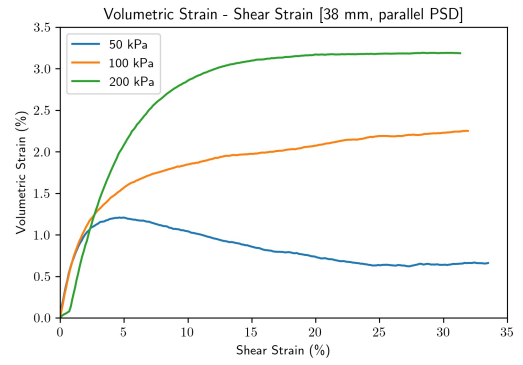
For the tests carried out on the smallest diameters, 38Ca and 38Pa (50 kPa) show a dilatation during shearing. The sample 38Ca present dilative trend at large strain. For sample 38Ca, the final volumetric strain will be greater than the initial one point, with expansion. On the graph 38C in Figure 5.2, the curve corresponding to this sample shows a stress peak. However, although sample 38Pa shows a dilatant phase, this is weaker than the contracting phase present at the start of the test. Total volumetric deformation indicates contracting behaviour. As the confining stress increases, the soil undergoes a transition from expansion to contraction. The axial strain - deviatoric stress curves do not show a clear peak and have the appearance of strain hardening. Deviatoric stress continued to increase during the test at the same time as axial strain. This may explain why the friction angle at the end of the test is greater than that at the peak state [64]. The time required to reach the axial deformation of 22% is twice as long for the 100Cc sample than for all the others tests.

In all of these graphs, the volume deformation curves are such that as the stress increases, the sample contracts as the shear deformation increases. For the 70Pa, the volume strain for the sample decreases. As mentioned previously, the 70Pb test was stopped before it was due to finish. This explains why the shear strain is not as large as the others. If the change in volume is examined before the shear strain equals 20%, the volume contraction of 70Pa is still larger than 70Pb. Equation (4.15) shows the dependency on axial deformation.

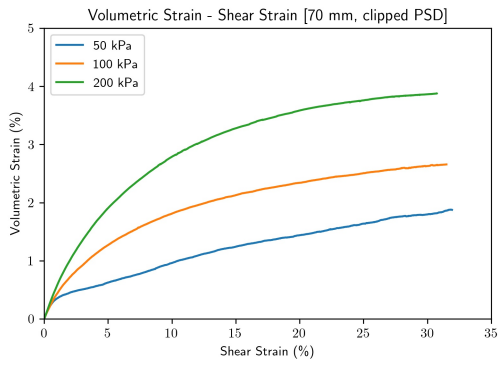
Graphics showing dilatancy as a function of stress ratio can be found in Appendix F and ones showing volume variation as a function of time in Appendix G. These show when the volume of the samples increases or decreases.



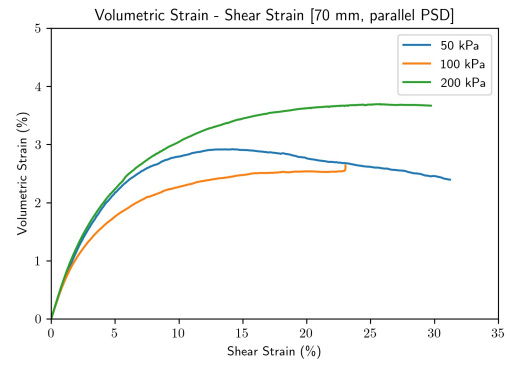
(a)



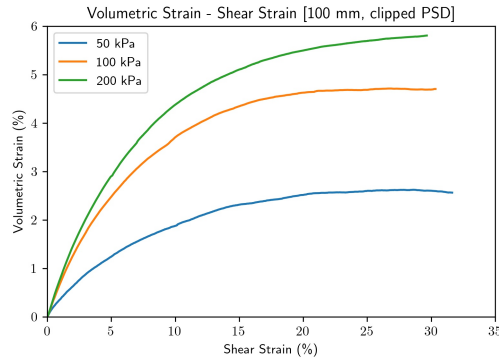
(b)



(c)



(d)



(e)

Figure 5.4: Volumetric strain depending on the shear strain for samples : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C

5.3 Shear band

In this section, the visual deformation of the sample will be studied. Two types of deformation at rupture are possible: a swelling of the central part of the sample or a rupture along an inclined plane. This analysis is purely visual and is based on the images shown in Figures 5.5, 5.6, 5.7, 5.8 and 5.9. The denser a sample is, the more likely it is to break along an inclined plane. This is due to grain dislocation. Conversely, the looser the sample, the more likely it is to have a swelling zone as a result of the grains settling.

Samples with a diameter of 38 *mm*, produced using both the simple clipping method and the parallel gradation modeling technique visible in Figures 5.5 and 5.6, have a greater tendency to break along an inclined fracture plane. For sample 38Ca, the graph in Figure 5.2 shows dense behaviour. This is less obvious for the other samples of the same diameter under greater confinement stress, although the shear plane is clearly visible in the photos. Looking at the graphs corresponding to samples 38P in Figure 5.2, this is not particularly the expected behaviour because these graphs do not show a stress peak.

For samples 70P, 70C and 100C, swelling was observed in the central part of the specimens. Some were more pronounced than others. Samples subjected to a confinement stress of 50 kPa showed greater swelling than those subjected to stresses of 100 kPa and 200 kPa.

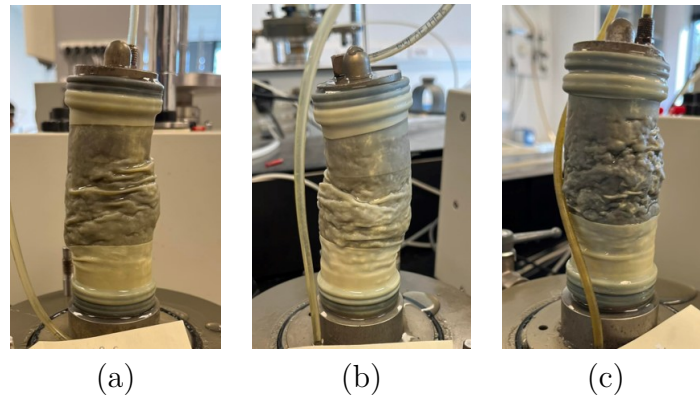


Figure 5.5: Samples 38C after shearing : (a) 50 kPa ; (b) 100 kPa ; (c) 200 kPa

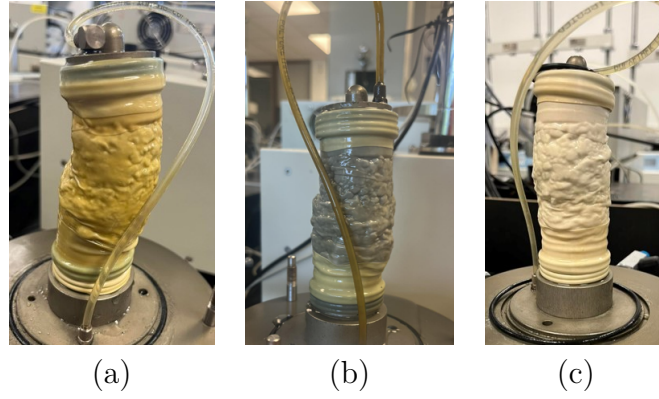


Figure 5.6: Samples 38P after shearing : (a) 50 kPa ; (b) 100 kPa ; (c) 200 kPa

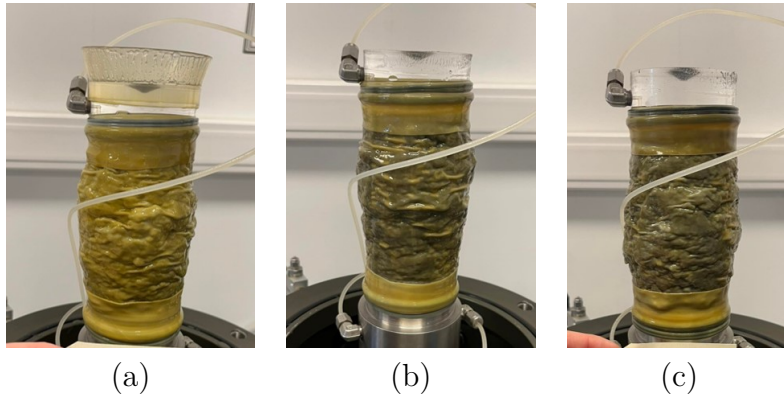


Figure 5.7: Samples 70C after shearing : (a) 50 kPa ; (b) 100 kPa ; (c) 200 kPa

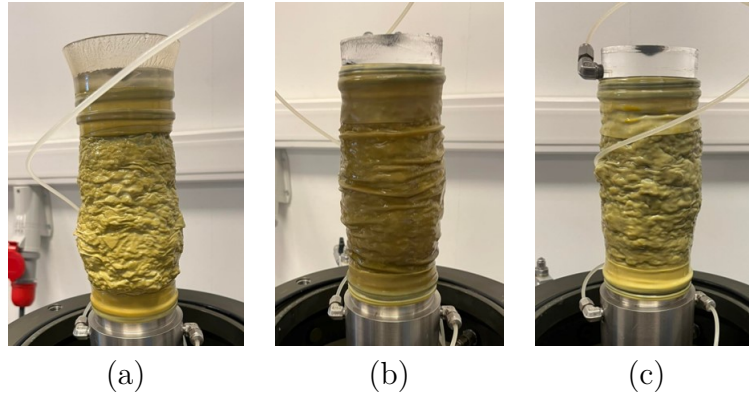


Figure 5.8: Samples 70P after shearing : (a) 50 kPa ; (b) 100 kPa ; (c) 200 kPa

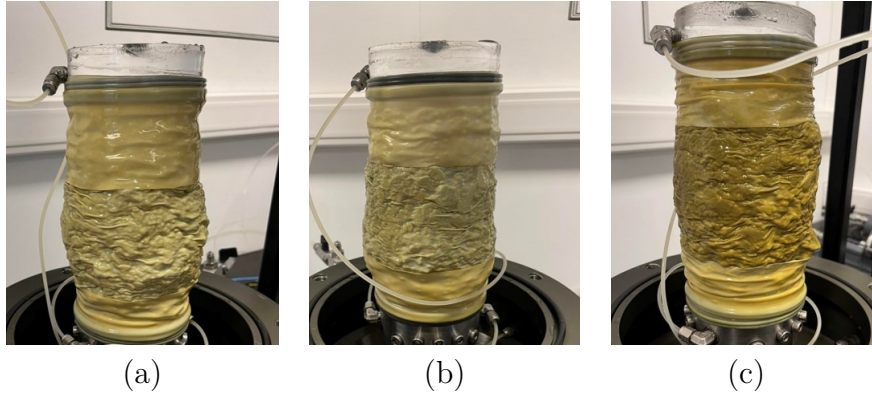


Figure 5.9: Samples 100C after shearing : (a) 50 kPa ; (b) 100 kPa ; (c) 200 kPa

5.4 Void ratio

Figure 5.10 shows the void ratio after shearing for all the tests carried out. Each point represents a sample [65, 66]. Figure 5.11 shows the specific volume as a function of stress. It is generally in this form that the volume of voids is interpreted.

For all the tests carried out, the voids ratio at the end of shearing decreases as the mean effective stress increases. In addition, the small-diameter samples are found in higher positions in the compression plane. This is represented on the graph by straight lines between points with negative slopes. All these lines show the same trend, but they are all different from each other.

Although the mean effective stresses are not the same for all the tests, three sets of points stand out, corresponding to the three confinement stresses applied. However, for sample 38P, the point corresponding to the highest stress is slightly outside the group. This test has a lower average effective stress than the others.

By comparing samples of the same diameter, the void ratio at the end of the test is greater for samples made using the simple clipping method than for those made using the parallel clipping method.

Figure 5.11 shows the Critical state line for all sample. These are represented by linear regression in such a way as to give the best possible result. The 3 points corresponding to the specific volume at the end of the test for each sample are used to carry out this regression. Figures of the specific volume as a function of the mean effective stress for each set of samples can be found in Appendix H. These regressions are evaluated using the coefficient of determination, the value of

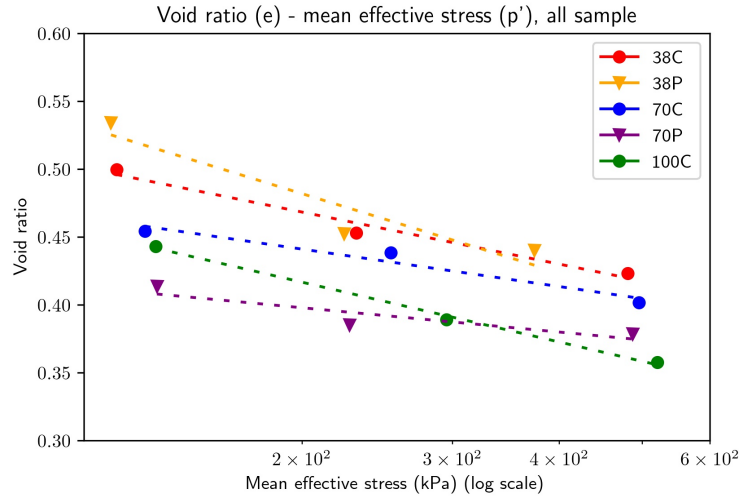


Figure 5.10: Void ratio depending on the mean effective stress for all sample size, clipping method and constraint

which is shown in the graphs in the appendix. For the 38C, 70C and 100C tests, coefficients are 0,9740, 0,9532 and 0,9965. For the 38P and 70P tests it is 0,8854 and 0,8031 respectively. This indicates good regression quality for all the tests. The value of the coefficient for the 100C test is almost equivalent to 1, which is the optimum value

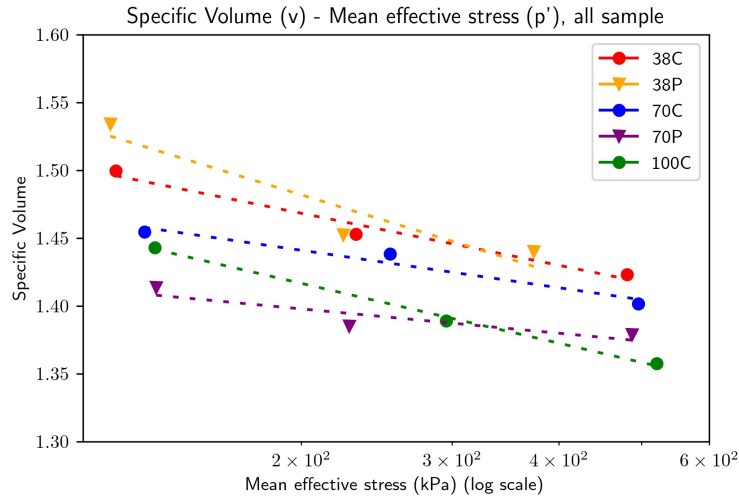


Figure 5.11: Volume specific depending on the mean effective stress for all sample size, clipping method and constraint

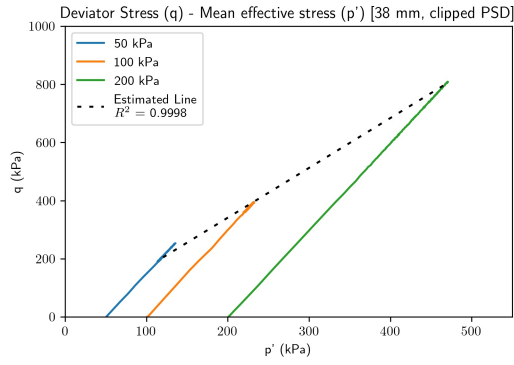
5.5 Stress path

Figure 5.1 show the stress paths of samples of different diameters and clipping methods under different confinement stresses. There is a linear relationship between the deviatoric stress q and the mean average stress p' . The curves for the various stresses are parallel to each other for all the samples. This can be explained by the equations (4.11) and (4.10). From these equations, it is possible to find the stress path equation (5.1) :

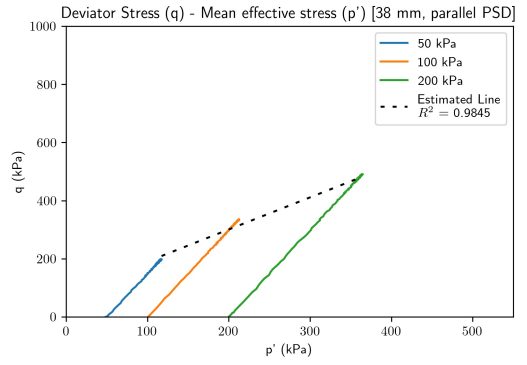
$$q = 3 \cdot (p' - \sigma'_3) \quad (5.1)$$

The deviatoric stress q increases linearly with the increasing average effective stress p' [65].

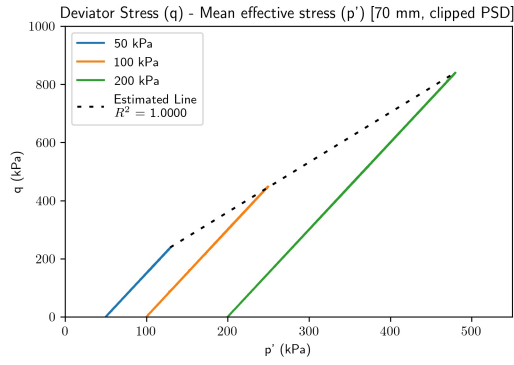
The estimated line visible as a dotted line on the graphs represents the linear regression. R^2 is the coefficient of determination. This is an indication of the quality of the linear regression. For all the sample sets, the value of R^2 is very close or equal to 1. This indicates that the linear regression from the last values of the deviatoric stress and the mean effective stress is very good.



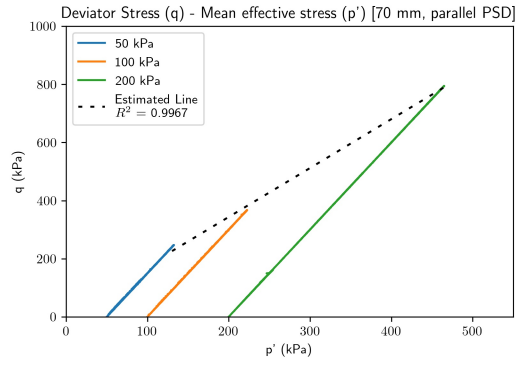
(a)



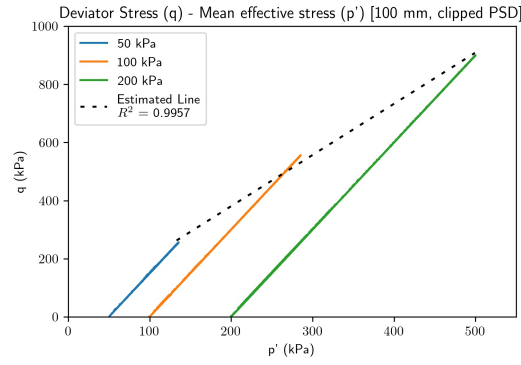
(b)



(c)



(d)



(e)

Figure 5.12: Stress path for samples : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C

6 | Slope stability analysis

Having determined the mechanical characteristics of the soil studied in Chapter 5, it is now possible to look at the maximum slope angle possible with this type of soil and the potential gain when carrying out earthworks.

6.1 Slope angle determination

Triaxial tests are performed under drained conditions. For the following analysis, slope stability study will focus on these conditions. There are several ways of calculating breakage. There are two main categories: statically defined cases or indefinite cases and blocs or slice method. In this thesis, a study on an infinite slope will be carried out with a translational slip plane. It is assumed that failure occurs along a continuous surface. This method provides a good first approximation while giving results on the safety side [67, 68].

In the study of slope stability, the calculation of the safety factor is the first task to be carried out. To ensure failure stability, the safety factor must be greater than or equal to 1. In civil engineering a value of 1,3 is often used [67, 69]. Admissible slope angles will be calculated for this safety factor of 1,3.

To carry out the breakage calculations, the results obtained in Chapter 5 must be correctly selected. As mentioned in Section 2.3.2, the parallel method tends to overestimate cohesion. With a value of more than 30 kPa, the 38P sample set is not retained compared to the other tests which present a cohesion between 6,38 and 13,08 kPa. Also, to remain consistent with the data used, the 70P sample set will not be used in this chapter. Only results obtained using the simple clipping method are taken into account. According to [69], for sands or gravels with less than 10% fine particles, cohesion should not be taken into account in long or short-term design situations. Indeed, the cohesion obtained during testing for this type of soil is in most cases apparent cohesion, which is not a reliable source of shear strength. For samples 38C, 70C and 100C carried out using the simple clipping method, the

percentage of fines is 7,07, 5,97 and 4,65 respectively. The cohesion observed in Table 5.2 is therefore considered to be zero for further analysis.

6.1.1 Translational failure

Safety factor FS is defined as *"the ratio of the available shear strength of the soil tot hat required to maintain equilibrium"* [70].

$$FS = \frac{\text{shear strength}}{\text{shear stress}} = \frac{\tau_f}{\tau} = \frac{c' + \gamma H \cos^2 \beta \tan \varphi}{\gamma H \cos \beta \sin \beta} \quad (6.1)$$

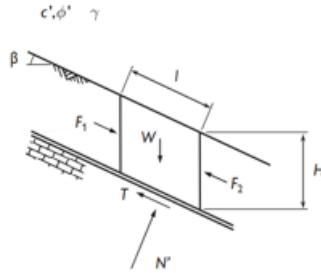


Figure 6.1: Equilibrium on an infinite slope [71]

The height of the slope H and the dry density γ of the soil are assumed to be known. With the safety factor equal to 1,3, the maximum angle of slope β can be expressed as a function of the cohesion c and the internal friction angle φ . Considering $c = 0$ and $\varphi \neq 0$, for the peak state, equation (6.1) become (6.2). In this case, the acceptable slope for the embankment will only depend on the internal friction angle.

$$FS = \frac{\text{shear strength}}{\text{shear stress}} = \frac{\tau_f}{\tau} = \frac{\cos \beta \tan \varphi}{\sin \beta} \Leftrightarrow \tan \varphi = FS \tan \beta \quad (6.2)$$

For φ values between 30° and 50° , Table 6.1 shows the acceptable values for slope angle. The values of internal friction angle corresponding to those of the reference soil for the different sizes of devices and simple clipping method (see Section 5.2.1) are in bold.

Table 6.1: Embankment slope angle (β) depending on the internal friction angle (φ) for non cohesive soil, translational failure

$\varphi(^{\circ})$	$\beta(^{\circ})$	$\varphi(^{\circ})$	$\beta(^{\circ})$
30	23,95	41	33,77
30,5	24,38	41,15	33,92
31	24,81	41,5	34,24
31,5	25,24	42	34,71
32	25,67	42,5	35,18
32,5	26,11	43	35,65
33	26,54	43,5	36,13
33,5	26,98	43,75	36,37
34	27,42	44	36,61
34,5	27,86	44,5	37,09
35	28,31	45	37,57
35,5	28,75	45,5	38,05
36	29,20	46	38,54
36,5	29,65	46,5	39,03
37	30,10	47	39,52
37,5	30,55	47,5	40,01
38	31,01	48	40,51
38,5	31,46	48,5	41,01
39	31,92	49	41,51
39,5	32,38	49,5	42,01
40	32,84	50	42,51
40,5	33,30		

6.2 Economical benefit on an earthwork project

To understand the potential gain when the embankment slope angle increases, the variation in the quantity of soil used for its construction will be calculated in relation to a reference value for the slope angle.

Price about excavation or embankment budgets is expressed in $\text{€}/m^3$. Depending on the nature of the soil, the accessibility of the site and the height difference, the price will vary [72]. The study focuses on embankments with a height of between 5 and 10m, composed of coarse-grained soil [73]. At first glance, the price can be set at $30 \text{ €}/m^3$. To this must be added the price for this type of soil, which varies between 50 and 60 $\text{€}/m^3$. An embankment operation will cost an average of 85 $\text{€}/m^3$ [72, 74].

An internal friction angle of 30° is considered. In addition, the gain in € is based on the volume of soil saved. This soil's volume saved is calculated with relation (6.4), for a height H , a length L (depending on the angle, relation (6.3)) and a width l of 1 meter. For the embankment slope angle's value (β) found in Section 6.1.1, the gain in m^3 (G_{m^3}) can be given for slope heights of 5, 7,5 and 10m. The values found are available in Table 6.2. To obtain a gain in €, the gain in m^3 is multiplied by the cost of one m^3 (relation (6.5)). These gains for embankments of heights 5, 7,5 and 10m are shown in Table 6.3.

The volumetric and economic gains for all internal friction angle between 35° and 50° compared with a internal friction angle of 35° can be found in Appendix I.

$$L = H \cdot tg(\beta) \quad (6.3)$$

$$G_{m^3} = \frac{H L_i - H L_f}{2} [m^2] \cdot 1[m] = \frac{H^2}{2} (tg(\beta_i) - tg(\beta_f)) [m^3] \quad (6.4)$$

$$G_{\text{€}} = G_{m^3} \cdot 85 [\text{€/}m^3] \quad (6.5)$$

Table 6.2: Gain in m^3 for different embankment height and slope angle compared to a reference internal friction angle of 35°

ϕ (mm)	φ ($^\circ$)	β ($^\circ$)	Gain (m^3) for :		
			H=5m	H=7,5m	H=10m
-	35	28,31	0,00	0,00	0,00
38	41,15	33,92	1,673	3,763	6,690
70	42	34,71	1,925	4,332	7,701
100	43,75	36,37	2,472	5,563	9,889

Table 6.3: Economical gain for different embankment height and slope angle compared to a reference internal friction angle of 35°

ϕ (mm)	φ ($^\circ$)	β ($^\circ$)	Gain (€) for :		
			H=5m	H=7,5m	H=10m
-	35	28,31	0,00	0,00	0,00
38	41,15	33,92	142,17	319,89	568,69
70	42	34,71	163,65	368,21	654,59
100	43,75	36,37	210,15	472,83	840,59

In addition to the gain achieved, it is possible to compare this gain with the cost of an embankment with a slope angle corresponding to the value of the reference

internal friction angle (i.e. a slope of $28,31^\circ$ for $\varphi = 35^\circ$). These costs are shown in Table 6.4. It is then possible to express the gain in euros as a percentage of the cost. This is shown in Table 6.5.

Table 6.4: Cost in € for an embankment with an internal friction angle of 35°

H (m)	Cost (€)
5	714,51
7,5	1287,76
10	2289,35

Table 6.5: Gain in % compared to the cost of the embankment for different size of device and height of embankment

ϕ (mm)	H=5m	H=7,5m	H=10m
38	19,90 %	24,84 %	24,84 %
70	22,90 %	28,59 %	28,59 %
100	29,41 %	36,72 %	36,72 %

The values obtained show that the percentage gain increases as the size of the device increases. All the savings represent at least 20% of the initial cost, which is not negligible for earthmoving projects.

7 | Perspectives and recommendations

Based on the results obtained in the previous chapters, a number of recommendations and improvements for future research can be formulated. These concern both the material used and the sample formation methods, as well as the corrections to be made to the results. The use of a large-scale device is also discussed.

7.1 Material used

The question arises as to whether the choice of soil from Australia is a wise one. Several points can be made in this regard.

Let's start with the mechanical properties of the soil. With the exception of the set made with 38P samples, it was observed that the soil has an internal angle of friction greater than 40° . This corresponds to the range expected when using a coarse reference soil. In this respect, the choice of soil from Australia was a wise one.

Another aspect concerns the quantity of reference soil available. Indeed, when all the soil available in the laboratory is used up, it will be complicated to continue the experiments. The samples tested as part of this thesis were kept in bags for future research. However, the consequences of reusing a soil that has already been subjected to shear stresses should be studied. A new sieve test should be carried out to define the new PSD after testing.

An alternative would have been to choose a different type of soil. A soil with known basic properties to measure more precisely the impact of the size of the device on the mechanical properties. But also to be able to carry out an unlimited number of tests because of the quantity available.

7.2 Effect of the clipping method

The parallel gradation modeling method requires a large quantity of small diameter particles. This was an obstacle for the 100mm diameter tests. In addition, this method tends to overestimate the fines content and it is difficult to assess the accuracy of the mechanical properties obtained. An alternative would have been to use the fine particles already available for all the tests, or to limit their use by adapting the particle size curves for large samples.

7.3 Selection of test's constant

For the different sets of samples, the aim was to keep the dry density of the soil constant. However, this assumption was implemented too strictly and the value of this parameter was modified. In addition, most of the existing studies (Chapter 2) are based on tests carried out with a controlled initial or relative void index or a constant relative density. This would allow better repeatability of the tests and greater consistency between samples of different sizes.

7.4 Area and membrane corrections

During shear tests, the soil deforms, supported by a membrane that also deforms. In some places on the outer surface of the sample, cavitations can be seen at the end of the test. Figure 5.9.(c) clearly shows that the outer surface is not smooth. Holes in the sample are visible in several places as well as some grains that are "outside" the volume. The membrane then applies a stress in its voids. Several techniques are possible to take account of this more pronounced change in the diameter of the sample at certain points, but also to take account of the deformations of the sample as a whole, as presented in Section 5.3. These are explained in [75].

7.5 Large scale device (ϕ 300 mm)

To confirm the gain in terms of the angle of internal friction obtained by increasing the size of the device, tests on a 300mm triaxial were initially planned. Unfortunately, it was not possible to carry out these tests, but it would be beneficial to do so in the future to confirm the results obtained. Details of how the specimen was made can be found in appendix J. In the article [76], the setting up of a sample of this size and the difficulties encountered are presented.

8 | Conclusion

The aim of this master thesis is to study the mechanical characteristics of coarse soils using devices of varying size in order to assess the potential gain during earthworks.

The first stage involved choosing a reference soil to be tested. The components of the soil had to meet size and particle size distribution criteria in order to achieve a sufficiently large angle of friction. An Australian soil was therefore used. In addition to this choice of reference soil, two clipping methods were used to adapt the material's PSD curve for testing on smaller devices. These were characterised using triaxial apparatus. Several drained consolidated shear tests were carried out with the same set of confining stresses for each set of samples. The test and sample formation methods used are detailed.

These tests were then used to study the mechanical properties and deformations of the soil:

- Regarding the internal friction angle, an increase is observed whatever the method of sample formation with an increase in the size of the sample. It can be concluded that this property depends on the diameter of the particles present in the soil, and that the resistance to shearing increases as the soil contains larger and larger particles. A further increase can be expected if the sample size increases further. Furthermore, this angle is always smaller in the case of the clipping method using parallel gradation modelling when comparing the results obtained for samples of the same dimensions. With the simple clipping method, the increase has a linear slope. This point confirms the value of using large devices. The reference soil with a maximum particle size of $50mm$ would require the use of a $300mm$ diameter device to assess its properties more accurately.
- Concerning cohesion, it decreases as the size of the device increases. The cohesion obtained on 38P test is much higher than on the others, due to the high fines content of this sample set. For the simple clipping method, a slight

decrease is still expected if the sample size continues to increase, stabilising thereafter.

- Deformations were analysed visually and the mode of fracture established. The general trend was towards swelling in the central zone of the sample for those with diameters of $70mm$ and $100mm$, while the $38mm$ diameter samples showed an inclined fracture plane.
- Voids ratio at the end of shearing decreases with increasing confinement stress and sample size.

Finally, an economic analysis was carried out. The permissible slope angle was calculated for the angles of internal friction characteristic of the soil studied. The savings achieved by using devices with an assumed angle of internal friction of 35° are significant. The percentage saved compared with the initial cost increases with the size of the device.

The results of the study live up to expectations. An increase in the internal friction angle is noted when the size of the devices used to carry out the tests increases. This implies an increase in the permissible slope for an embankment and the gain in terms of percentage also increases. The benefits of using larger and larger devices have been demonstrated.

Appendices

A Particles size distribution of the basecourse material from previous report on $\phi 300mm$ carried out in 2020



GRAIN SIZE DISTRIBUTION (2)

Sieving method

Project Informations	Sample Informations
Project n° : 163798 Project : SABR - Australia Client : AURECON	Sampling type : Quarry Sampling date : - Conservation mode : Bag Identification number : - Reception Date : 04-05-20 Description : Sample 4
	Borehole n° : Bag n°2 Depth : - Date : 12-11-20 Dmax (mm) : 50

Grain size distribution - BS1377 - 2

Dmax (mm)	50
Sample dry mass (kg)	63,838

Aperture size (mm)	Cumulative mass retained (g)	cumulative % retained	cumulative % passing	coal quantity
250	0	0,00	100,00	N/A
125	0	0,00	100,00	N/A
100	0	0,00	100,00	N/A
80	0	0,00	100,00	N/A
63	0	0,00	100,00	N/A
50	2380	3,73	96,27	N/A
40	11117	17,41	82,59	N/A
31,5	29175	45,70	54,30	N/A
25	39212	61,42	38,58	N/A
20	45317	70,99	29,01	N/A
14	51660	80,92	19,08	N/A
10	55699	87,25	12,75	N/A
6,3	58884	92,24	7,76	N/A
5	60737	95,14	4,86	N/A
4	61696	96,64	3,36	N/A
2	63421	99,35	0,65	N/A
1	DISCARDED			
0,5				
0,425				
0,25				
0,125				
0,063				

Drying Method : ☒ at 105 °C ☐ at 50 °C

Comments

- Second particle size distribution (PSD) is performed after washing the sample
- Some particles were lost when removing the sample from the pedestal (estimated to 417 gr)
- Initial PSD is taken from the sample prepared for the tests

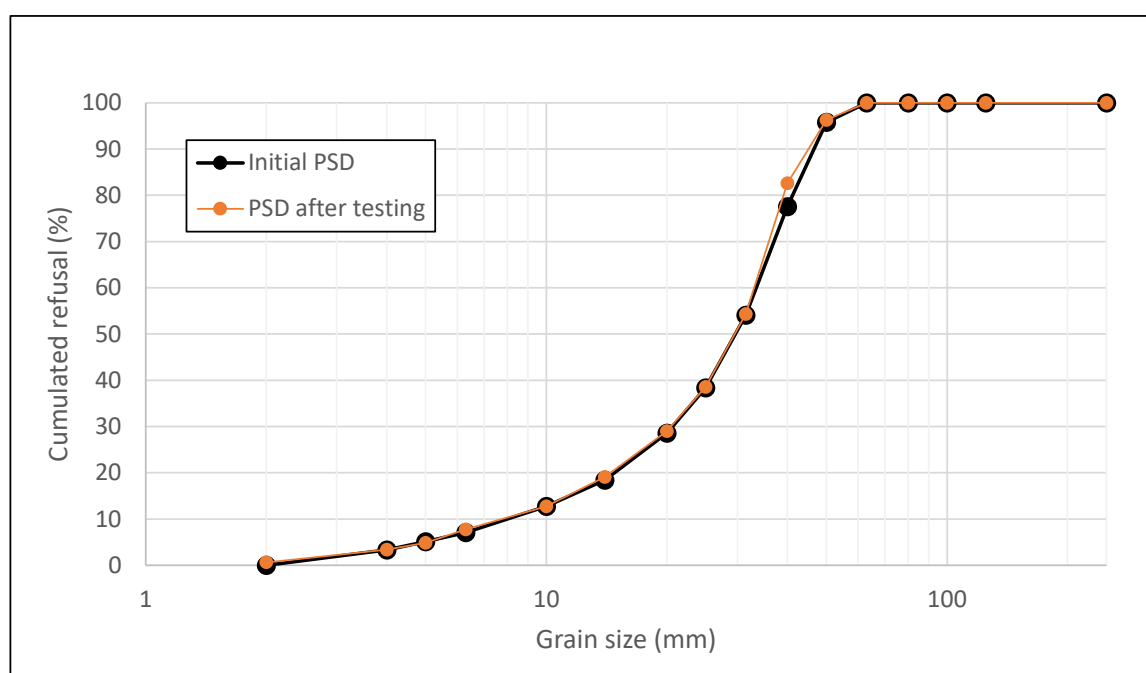
Laboratory manager
CHRISTIAENS Damien

Technician
Itulamya Arsene Mango

Sieving method

Project Informations	Sample Informations	
Project n° : 163798 Project : SABR - Australia Client : AURECON	Sampling type : Quarry Sampling date : - Conservation mode : Bag Identification number : - Reception Date : 04-05-20 Description : Sample 4	Borehole n° : Bag n°2 Depth : - Date : 12-11-20 Dmax (mm) : 50

Grain size distribution - BS1377 - 2



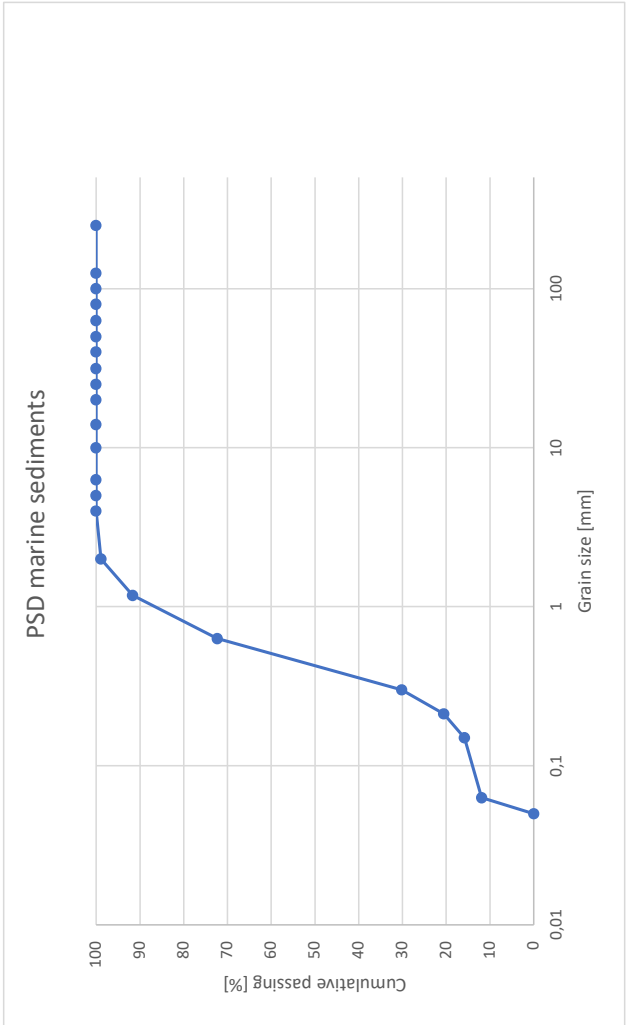
B Realisation of the particles size distribution of marines's sediments

Test 1						
dmax [mm]	Masse de refus [g]	Masse de refus cumulées [g]	Pourcentage de refus [%]	Pourcentage de refus cumulé [%]	Pourcentage de passant cumulé [%]	
100	0	0	0	0	100	100
80	0	0	0	0	100	100
63	0	0	0	0	100	100
50	0	0	0	0	100	100
40	0	0	0	0	100	100
31,5	0	0	0	0	100	100
25	0	0	0	0	100	100
20	0	0	0	0	100	100
14	0	0	0	0	100	100
10	0	0	0	0	100	100
6,3	0	0	0	0	100	100
5	0	0	0	0	100	100
4	0	0	0	0	100	100
2	2,04	2,04	1,123843103	1,123843103	98,8761569	98,8761569
1,18	13,33	15,37	7,343543411	8,467386514	91,53261349	91,53261349
0,63	35,2	50,57	19,39180256	27,85918907	72,14081093	72,14081093
0,3	75,65	126,22	41,67584839	69,53503746	30,46496254	30,46496254
0,212	17,62	143,84	9,706919348	79,24195681	20,75804319	20,75804319
0,15	8,9	152,74	4,903040987	84,1449978	15,8550022	15,8550022
0,063	6,69	159,43	3,685544293	87,83054209	12,16945791	12,16945791
0,05	22,09	181,52	12,16945791	100	0	0
>2mm	0					
<=2mm	181,52					
total	182					
masse sèche totale	180,91					
masse sèche >63µm	159					
masse de fines	21,91					
% de fines	12%					

80

Valeurs moyennes pour PSD

100	
100	
100	
100	
100	
100	
100	
100	
100	
100	
100	
100	
100	
100	
100	
98,888575	1,11142495
91,664608	8,33539203
72,2934698	27,7065302
30,1544362	69,8455638
20,5786145	79,4213855
15,818501	84,181499
11,9258382	88,0741618
0	100
	11,9258382
	100



C Particles size distribution of marine sediments's finer part

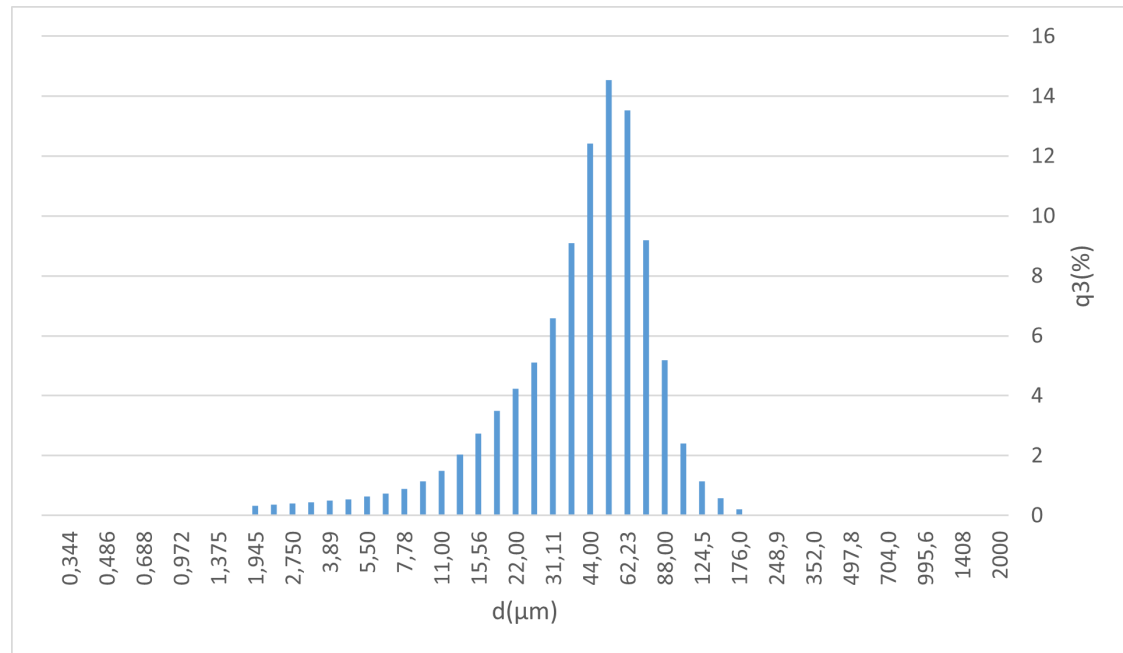


Figure 8.1: PSD of marine sediments's smaller part

D Report from tests carried out in 2020

SUMMARY REPORT



Project name	SABR
Project Ref	163798 01
Client	AURECON
Preparation date	20/10/2020

SAMPLE 4: PREPARATION SUMMARY

Sample 4: 85% Basecourse + 15% marine sediments, as specified by the client in Revision 2, Trial Placement Mix Density, 14-10-2020

The sample for the triaxial test is composed of 6 basecourse layers, following the density given by the client. Each layer is prepared separately using the mass of the corresponding particle size distribution (Basecourse and marine sediments). Each is mixed before introduction into the sample, 12% in mass water was added for the marine sediments (corresponding to 228 g of water for each layer). The exact amount of materials is given on the document in Appendix.

The sample where the material is introduced is divided into 6 height limits (Photo 1), giving an idea of the compaction level to reach. The membrane from sample 2 was reused for the preparation.

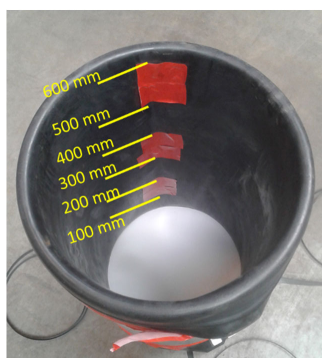


Photo 1 : Sample height division

The material is gently placed inside the mould. A smooth compaction was needed to reach the given density, after the height was uniformized. The compaction energy was not measured. Compaction was carried out using a wood rod.

Photos 2 and 3 show different layers during sample preparation.



Photo 2 : Sample mix and after compaction – layer 2



Photo 3 : Sample mix and after compaction – layer 6

The sample was then placed on the load cell and the mould was removed. The sample remained standing and presented a uniform shape. It was then chosen to prepare the cell. No difficulties were encountered during the closing of the cell.



Photo 5 – 6 – 7: Sample placement on the load cell and removal of the mould

SABR - AURECON (Australia)

Sample 3 preparation

Sample 4 Basecourse material @ 1505 kg/m³ + 15% of MS

	Requested values		
	pd (kg/m ³)	Volume (m ³)	mass /triax (kg)
Basecourse material	1505	0,0424	63,829
Marine Sediments	269	0,0424	11,409

6 layers for sample preparation

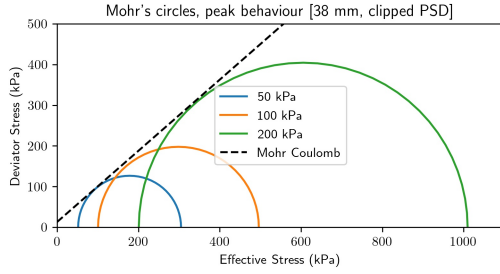
BC mass /layer 10,638 kg
Sediment mass/layer 1,901 kg

Aperture size (mm)	Mass (g)	Layer #					
		1	2	3	4	5	6
Basecourse							
50	446	433	451	443	449	444	453
40	1944	1953	1937	1945	1943	1949	1937
31,5	2498	2496	2502	2504	2502	2494	2496
25	1668	1668	1670	1675	1664	1668	1664
20	1040	1041	1041	1039	1043	1038	1040
14	1087	1087	1085	1088	1086	1088	1087
10	600	599	600	600	600	600	600
6,3	604	604	604	604	604	604	604
5	208	208	208	208	208	208	208
4	193	193	193	193	193	193	193
2	351	351	351	351	351	351	351
TOTAL (g)	10638	10633	10642	10650	10643	10637	10633
TOTAL (%)		99,95	100,04	100,11	100,04	99,99	99,95
Sediments							
<2,36 mm	0	1899,83	1899,84	1899,53	1899,84	1900,18	1899,63
added water	0	228,63	228,64	228,37	228,33	228,21	227,62
Total weight							
dry mass (g)		12532,83	12541,84	12549,53	12542,84	12537,18	12532,63
wet mass (g)		12761,46	12770,48	12777,9	12771,17	12765,39	12760,25

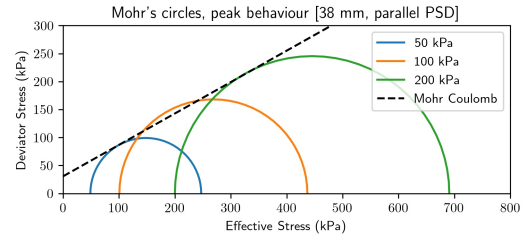
TOTAL MASS FOR SAMPLE 2	76606,65	g
REFUSAL ON LAYER 6 (Wet)	0	g

ρ 1806 kg/m³ Diameter = 300 mm Height = 600 mm
pd 1774 kg/m³ 100,00 % from the initial dry density request

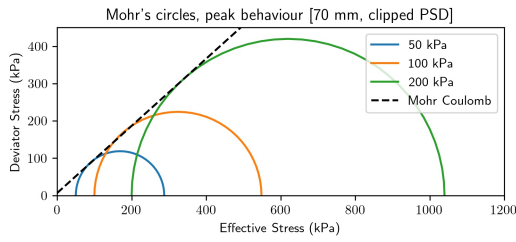
E Mohr's circles



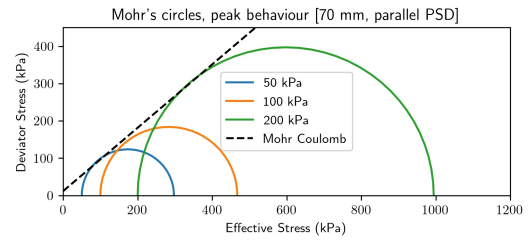
(a)



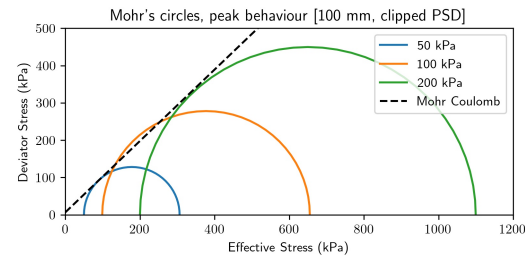
(b)



(c)

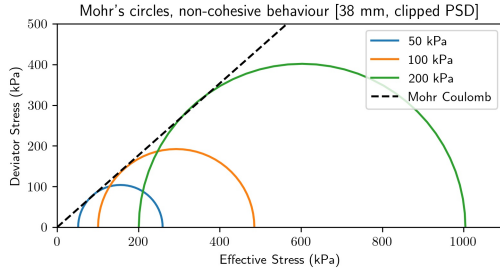


(d)

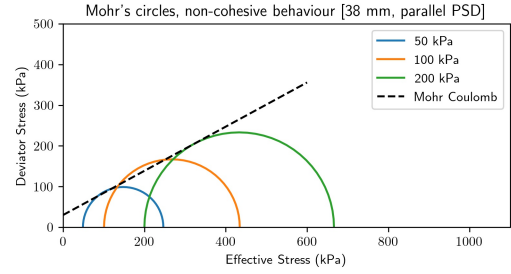


(e)

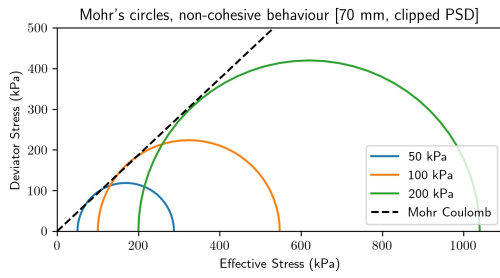
Figure 8.2: Mohr's circles at peak : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C



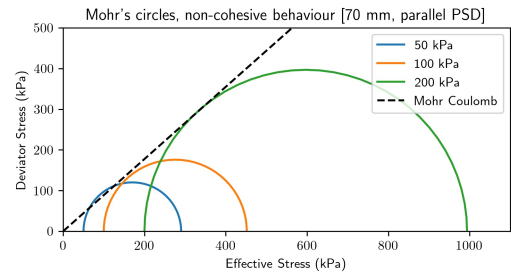
(a)



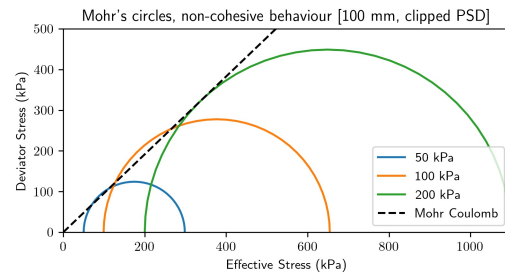
(b)



(c)



(d)



(e)

Figure 8.3: Mohr's circles at end state : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C

F Stress ratio - dilatancy graphs

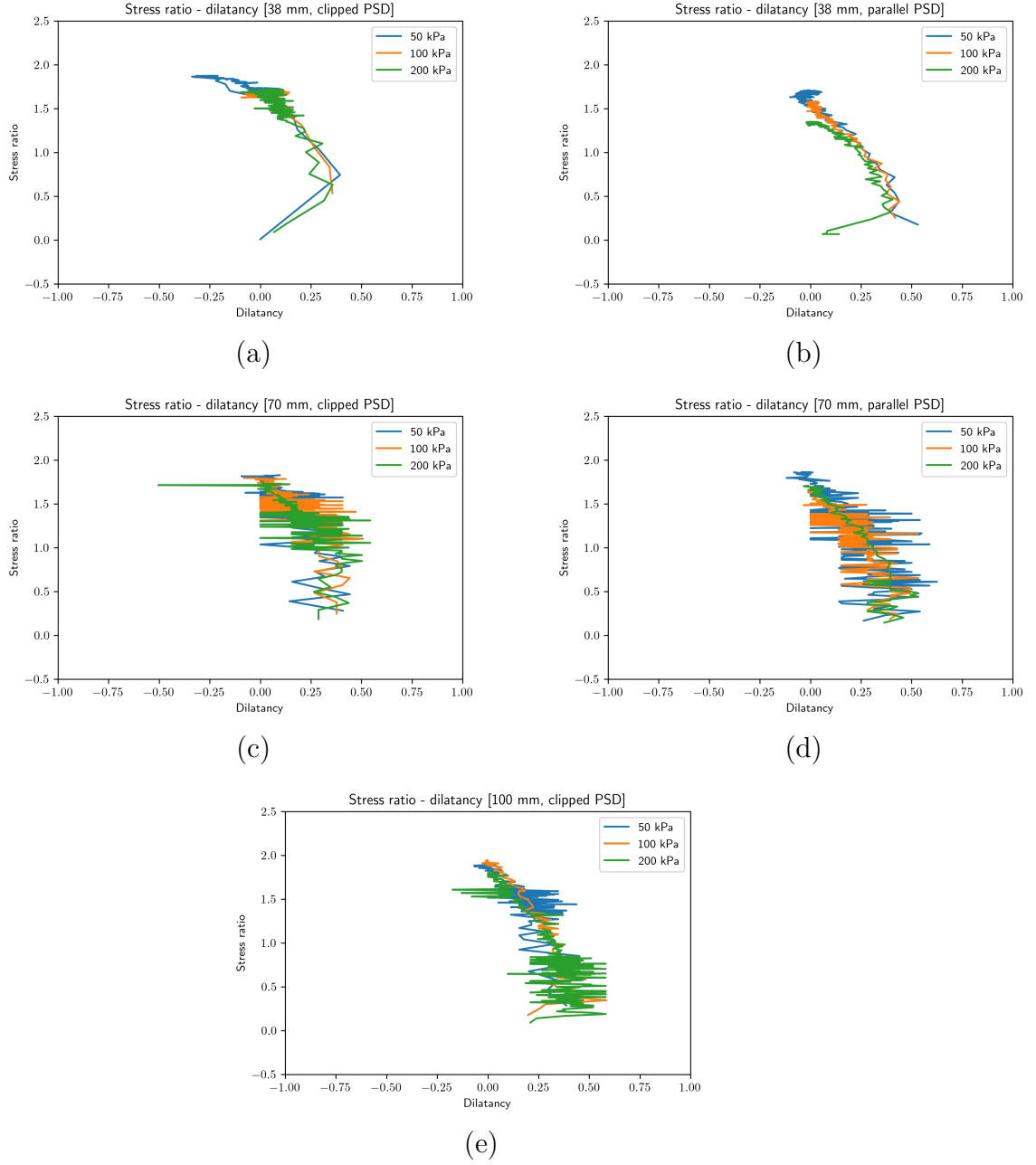


Figure 8.4: Stress ratio depending on dilatancy : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C

G Volume variation - time

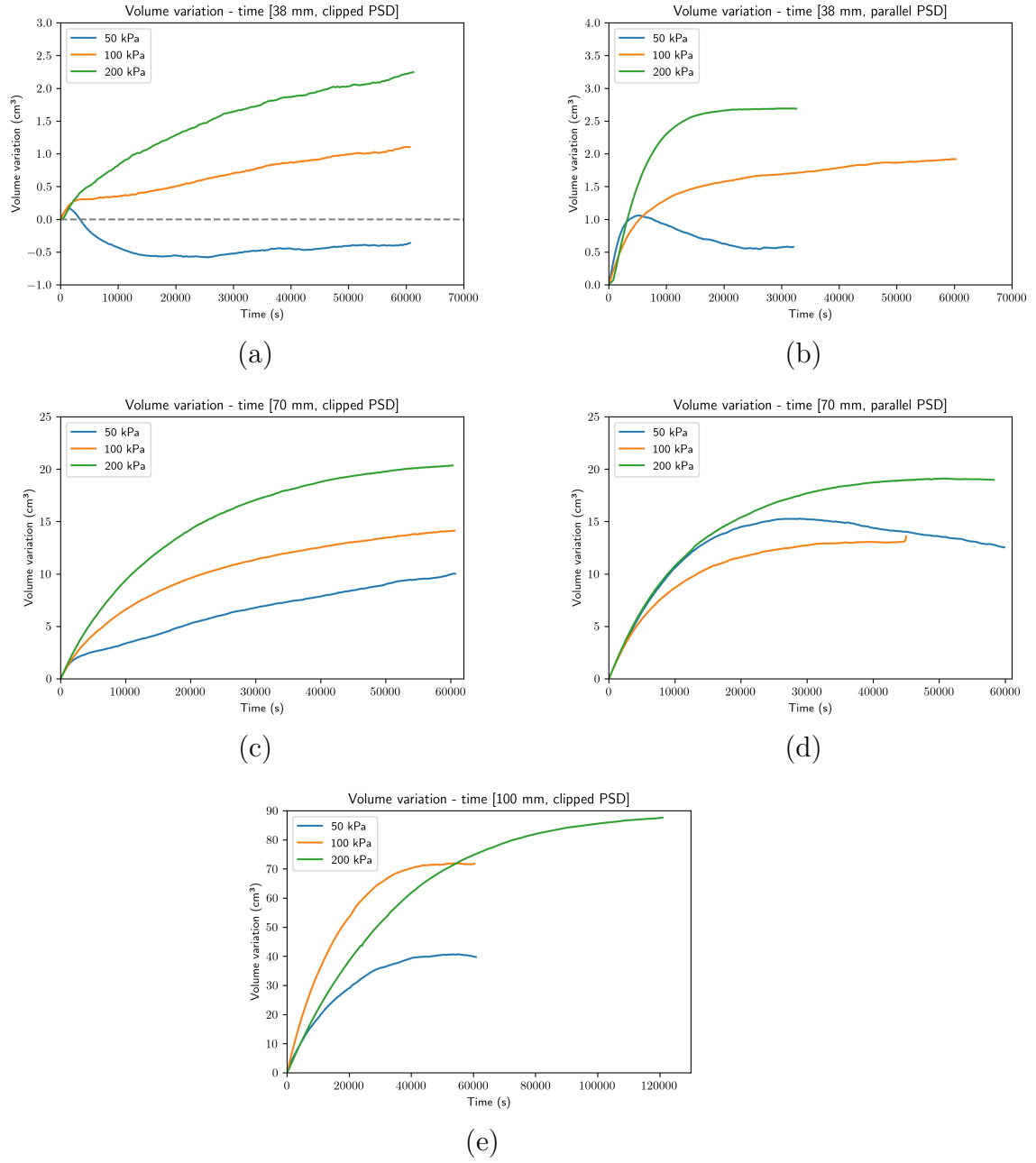


Figure 8.5: Volume variation during shearing for samples : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C

H Specific volume - mean effective stress graphs

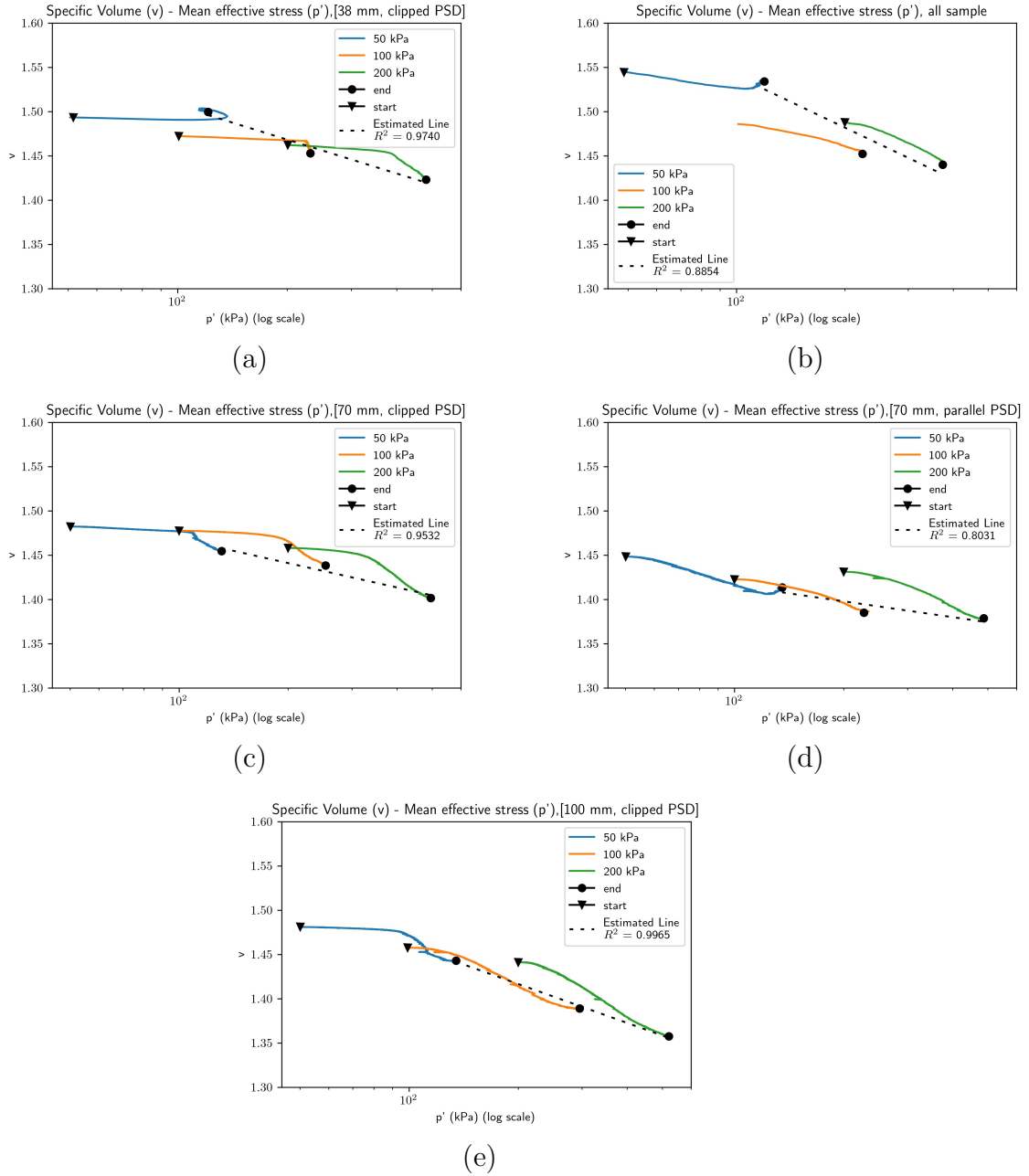


Figure 8.6: Specific volume depending on mean effective stress : (a) 38C ; (b) 38P ; (c) 70C ; (d) 70P ; (e) 100C

I Volumetric and economical gain

Table 8.1: Gain in m^3 depending on slope angle compared to a internal friction angle of 35°

φ ($^\circ$)	β ($^\circ$)	Gain (m^3) for :			φ ($^\circ$)	β ($^\circ$)	Gain (m^3) for :		
		H=5m	H=7,5m	H=10m			H=5m	H=7,5m	H=10m
35	28,31	0,000	0,000	0,000	43	35,65	2,234	5,026	8,935
35,5	28,75	0,126	0,283	0,503	43,5	36,13	2,392	5,382	9,568
36	29,20	0,253	0,570	1,013	44	36,61	2,553	5,744	10,211
36,5	29,65	0,382	0,860	1,529	44,5	37,09	2,716	6,112	10,865
37	30,10	0,513	1,154	2,052	45	37,57	2,883	6,486	11,530
37,5	30,55	0,645	1,452	2,582	45,5	38,05	3,052	6,867	12,208
38	31,01	0,780	1,754	3,118	46	38,54	3,224	7,255	12,897
38,5	31,46	0,916	2,060	3,663	46,5	39,03	3,400	7,649	13,599
39	31,92	1,054	2,371	4,214	47	39,52	3,578	8,052	14,314
39,5	32,38	1,194	2,685	4,774	47,5	40,01	3,761	8,461	15,042
40	32,84	1,336	3,005	5,342	48	40,51	3,946	8,879	15,785
40,5	33,30	1,480	3,329	5,918	48,5	41,01	4,135	9,305	16,542
41	33,77	1,626	3,658	6,503	49	41,51	4,328	9,739	17,314
41,5	34,24	1,774	3,992	7,097	49,5	42,01	4,525	10,182	18,102
42	34,71	1,925	4,331	7,700	50	42,51	4,726	10,634	18,906
42,5	35,18	2,078	4,676	8,312					

Table 8.2: Economical gain depending on slope angle compared to a internal friction angle of 35°

φ ($^\circ$)	β ($^\circ$)	Gain (€) for :			φ ($^\circ$)	β ($^\circ$)	Gain (€) for :		
		H=5m	H=7,5m	H=10m			H=5m	H=7,5m	H=10m
35	28,31	0,00	0,00	0,00	43	35,65	189,87	427,20	759,47
35,5	28,75	10,69	24,06	42,78	43,5	36,13	203,31	457,45	813,24
36	29,20	21,52	48,43	86,10	44	36,61	216,98	488,20	867,92
36,5	29,65	32,49	73,10	129,96	44,5	37,09	230,88	519,48	923,52
37	30,10	43,60	98,10	174,40	45	37,57	245,02	551,30	980,09
37,5	30,55	54,86	123,43	219,43	45,5	38,05	259,41	583,68	1037,65
38	31,01	66,27	149,10	265,06	46	38,54	274,06	616,64	1096,25
38,5	31,46	77,83	175,12	311,32	46,5	39,03	288,98	650,20	1155,91
39	31,92	89,56	201,50	358,23	47	39,52	304,17	684,38	1216,68
39,5	32,38	101,45	228,27	405,81	47,5	40,01	319,65	719,21	1278,60
40	32,84	113,52	255,41	454,07	48	40,51	335,43	754,71	1341,71
40,5	33,30	125,76	282,96	503,05	48,5	41,01	351,51	790,90	1406,05
41	33,77	138,19	310,93	552,76	49	41,51	367,92	827,82	1471,68
41,5	34,24	150,81	339,32	603,23	49,5	42,01	384,66	865,48	1538,64
42	34,71	163,62	368,15	654,49	50	42,51	401,74	903,92	1606,98
42,5	35,18	176,64	397,44	706,56					

J ϕ 300mm sample preparation

The samples produced for the $\phi 300$ triaxial device will have a diameter of 300 mm and a height of 600 mm. The procedure for producing them is as follows:

1. Preparing the particle size distribution:

- Soil drying at 105°
- Separation of aggregates according to maximum grain diameter
 - 50, 40, 31,5, 25, 20, 14, 10, 6,3, 5, 4, 2 mm
 - Marine sediments considered as a class of particles, its PSD can be found in the appendix B
- Storage in bags

2. Mould preparation

- Measure the height and internal diameter of the mould at 4 different points to determine its average dimensions and volume

3. Layers preparation

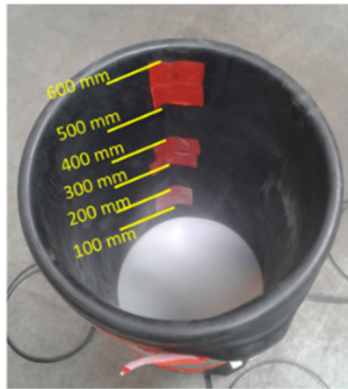
- Definition of the number of layers: For ease of future installation, sample in 6 layers 100mm high
- Each layer is prepared separately using the mass corresponding to the particle size distribution for the total volume.
 - The masses for each particle diameter are weighed and recorded in a preparation document.
 - Each sub-layer is stored in a bag
- 12% of the mass corresponding to the particles $\leq 2mm$ is added in water to each bag
 - The bags are sealed and shaken regularly
 - A waiting period is necessary for the water content to reach equilibrium throughout the sample and for the sample to be homogenous.

4. Membrane preparation, Figure 8.7

- Check that the membrane is not punctured and does not contain any singularity which could falsify the test
- Measure the thickness of the membrane
- Place the membrane in the mould provided for the sample, creating a vacuum
- Marking every 100mm for the different layers
- Weigh the mould with the membrane



(a)



(b)

Figure 8.7: (a) Membrane placement ; (b) Membrane mark making

5. **Installation of the mold with the membrane on a porous stone**
6. **Compaction**
 - Mix the bag used for each sub-layers
 - Place the contents in the mould
 - Gently compact so that the layers reach the level marked previously in step 4
7. **Sample placement on the load cell and removal of the mould**
8. **Placing a second porous stone on the top of the sample and fixing the membrane with steel's O-ring at the bottom and at the top**



Figure 8.8: Sample on the load cell

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20means%20clay%20(fine,exhibits%20significant%20cohesion%
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