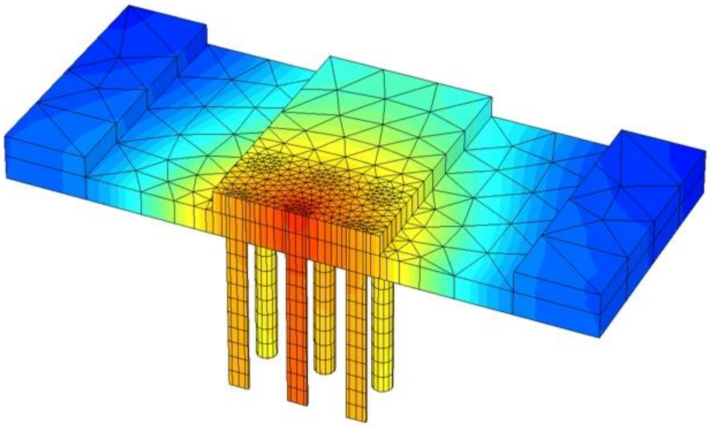
Modelling of clayey soil by using Finite Element Method (FEM) under Pressuremeter test

Final Report



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Preface

This research is a graduation thesis project of the BCs civil engineering program at HZ University of Applied Sciences mainly conducted to emphasize acquiring the required knowledge and competencies during the four years. As a result of the fast development in the field of civil engineering and construction, better technologies are needed in order to bear this fast development in the construction industry. Therefore, the purpose of this research is to design a new systemic approach that allows estimating the necessary input parameters for modelling the soil behaviour from Pressuremeter test. During the whole period of this research, I have learned a lot in the field of geotechnical engineering, particularly in the soil investigations field. It was quite challengeable to conduct my graduation research in such field; I faced many obstacles concerning the new terminologies, devices and software that I had to use. But with the support of my supervisor and experts in both UCA and Sergeyco whom without their support I could not have achieved the research goals, they directed me to deal with those obstacles in a professional manner. The research involves a wide range of reliable sources such books, scientific reports, digital sources and meeting with experts, which make the research process and its further results are beneficial to read.

Acknowledgement

This final thesis wouldn't meet this level of satisfaction without the kind support and help of groups and individuals. I would like to extend my sincere gratitude to all of them.

Foremost, I acknowledge with gratitude Prof. Paco Moreno and Prof. Javier Manzano, my in-Company supervisors, who would have to share a headache with me during the whole research process. They were always sincere and helpful from the first day of the research till the last day. Their kind support which had always made me understands the different modelling techniques and soil investigations which were quite inscrutable for me in the first days. I am thankful for their guidance, coordination and quality assurance they offered during the different research stages.

I would like also to express my attitude to Dr. Giuliana Scuderi, the HZ-Supervisor, for her support, quality assurance, valuable feedback and smooth communication showed from even before the research started. She was helpful in terms of planning the overall layout of the research as well as imparting her technical and professional knowledge to ensure that the research achieves the required professional level.

I would also exploit the opportunity to send my genuine thanks to all the individuals and experts from Sergeyco Company and UCA who have helped me to carry out the practical parts of the research whether the in-situ tests experts that exposed to the sun heat or those who were in the laboratories.

Finally, Special thanks and appreciation to HZ University of Applied Sciences for the support and effort being made to facilitate this research

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Abstract

There are a lot of problems associated with clayey soil. More geotechnical pathologies appeared as more developed project brighten up. The geotechnical engineers encounter problems with modelling the stiff clayey soil behaviour especially while using the classical methods for obtaining the input parameters of Finite Element Methods for modelling the soil behaviour. Therefore, This research paper intended to design a systematic approach that allows estimating necessary parameters from in-situ test specifically the Pressuremeter test to model the clayey soil behaviour using Finite Element Methods.

The method used to assure the feasibility of using the pressuremeter test to estimate the input parameters was by implementing both Pressuremeter test and classical laboratory test in the same location before simulating the Pressuremeter test using the input parameters obtained from the laboratory test to compare the soil response of the modelling and the real test. A three different modelling techniques namely MohrCoulomb, hardening Soil and Cam-Clay model were assessed based on relevant criteria to be chosen as a method of conducting the modelling and further comparing and interpreting the results in PLAXIS.

The main findings are that the agreement between the actual soil response and the modelling response curves are founded only with increasing the cohesion to relatively high values. Besides, inverse correlation founded between the Ep and E50 displays illogical behaviour and therefore more samples required to be studied to find a significant relation.

As it was agreed upon by the In-Company and In-school supervisors during the In-company meeting, this report doesn’t include any design or advance hand calculations in view of the fact that there was enough advanced modelling in PLAXIS, laboratory and site work together with the extra assignment represented in a pile foundation drawing to fulfill the professional competencies.

Abbreviations

UCA: University of Cadiz

FEM: Finite Element Method

PMT: Pressuremeter Test

MH: Mohr-Coulomb

HS: Hardening Soil

CC: Cam-Clay

CU: Consolidated-Undrained

CD: Consolidated Drained

UU: Unconsolidated Undrained

ASTM: American Society for Testing and Materials

MCA: Multi-Criteria Analysis

Ep: Pressuremeter modulus

E50: secant modulus at 50% stress difference

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1. Introduction

Geotechnical engineering without a doubt is fundamental in the construction industry, particularly in ensuring the functionality and long-term stability of various structures and earthworks. This research intended to study a geotechnical problem. Specifically, design approach to allow modelling clayey soils by using Finite Element Methods under Pressuremeter Test. This chapter will start by illustrating the background information related to the topic and the client. Subsequently, the problem definition and research objectives will be reviewed. And finally, the report outlines will be highlighted.

1.1 Background Information

1.1.1 Client:

This research is led and supported by both Universidad de Cádiz (UCA) Research Centre and Sergeyco Andalucía S.L. Company in a joint venture. Sergeyco is an engineering company dedicated to provide solution and consultancy for off-shore projects and studies in geotechnical studies, quality control laboratory tests and environmental inspections. The research took place in both locations namely Algeciras where Civil Engineering campus of the University of Cadiz located, and San Roque as the nearest office and laboratories of Sergeyco Andalucía S.L. Both organizations contributed by means of expertise, technical insight, local knowledge and quality control to supervise the author of this research.

1.1.2 Study area

The chosen study area is located in the place where the University of Cadiz and laboratories located, the city of Algeciras. This place has been chosen due to the availability of the soil type that will be examined. Moreover, both clients location can be reached within a reasonable distance from the examined area. Therefore, there weren't further difficulties and costs for the transportation and preparing the tests equipment in addition to the fact that the laboratories were within easy reach from the site which made the research process and execution goes faster and smoother. A detailed information of the

Figure 1 Study area (Google, 2018)

study area can be found in Chapter 2.1.

1.1.3 Assignment background

Geotechnical investigations play a big role in the foundation designs and consequently in all civil engineering projects. Due to the fast development of projects and variety in the field of civil engineering, more geotechnical pathologies have been appeared, especially in the clayey soils. The clayey soils are more exposed to geotechnical problems such as long-term settlement and slope stability (Universiti Teknologi MARA (UiTM), 2014). Thus, the soil behaviour needs to be defined for each project to have better knowledge of the applications and design performance of the soil tested (Roy, 2017).

Those behaviours normally defined by more complex models such Hardening Soil Small Strain Stiffness (HSSS) or Hardening Soil Model (HS) which demand a large list of parameters to be obtained from soil testing (Vermeer, Bonnier, & Schanz, 1999). Accordingly, more time and cost-effective testing have to be examined to obtain same values and parameters of ordinary tests. The company conducts both in-situ tests represented in Pressuremeter test and laboratory test such triaxial test to obtain the soil parameters before using the Finite Element method for modelling the soil behaviour. Pressuremeter test has been widely used recently as a result of the fact that it is an in-situ investigation which can determine the stress-strain behaviour of the examined soil. There are different ways to evaluate the results obtained from the Pressuremeter and for design the foundation, correlations with other soil parameters or direct design method needed since it can't depend upon the fundamental analysis yet. On contrary, taking samples and other coring techniques are facing difficulties in functioning in more deep waters especially with very stiff clays. Therefore, the importance of Pressuremeter tests appears when providing a direct access to the soil properties on sites without the need for sampling. But there is doubt in the accuracy of the results without combination with laboratory tests.

Different studies have been made to find a good correlation between the real soil response obtained from PMT and the soil behaviour obtained from the numerical modelling for sandy or soft clayey soil. On contrary, the current study intended to study the stiff clayey soil and attain an experience on its behaviour and its correlation between the real in-situ test and the constitutive modelling response and therefore to conclude possibility of estimating the soil parameters from PMT.

1.2 Problem Analysis

Sergeyco has conducted a lot of offshore and on-shore projects related to the soil investigation, the company experienced obstacles in obtaining the samples of stiff clayey soil. This returns to the fact that stiff clay is more susceptible to disintegration, degradation and lose its properties during the transportation or processing to the laboratory tests (Arbanas, Grošić, & Briški, 2007) due to the fact that working with stiff clayey soils is very difficult especially when it comes to taking samples. An accurate and efficient knowledge of the clayey soil behaviour is essential to avoid damages and failures in infrastructure as well as to improve the design since more conservative design applied because of the lack of reliable information. The stiff or rocky soil may not give reliable and accurate properties in the laboratory tests since it may lose its properties during the process of sampling tests. Hence, there are difficulties to obtain the necessary input data for modelling the soil using advanced modelling techniques when only lab testing is used. For instance, the FEM constitutive model requires 3-12 input parameters (Townsend, Anderson, & Rahelison, 2001). In essence, lab testing results in lack input data for modelling the soil. Additionally, in order to describe the mechanical properties of the soil from the laboratory tests, a very complex and arduous tasks needs to be done for the sampling process (Oliva, n.d.). However, the combination of laboratory and field tests has to carry out for such soils are directly affecting the design decisions and long-term geotechnical pathologies in addition to the fact that these currently used processes are negatively influencing the time and costs of the projects.

1.2.1 Problem statement

Sergeyco Company, as any other geotechnical firm, facing difficulties in describing the mechanical behaviour on stiff clayey soil by performing the classical laboratory tests. Moreover, the common method used doesn't provide enough data for constitutive modelling inputs in comparison with PMT.

1.3 Research Objective

In order to solve the stated problem, Sergeyco tends to improve the method of obtaining the clayey soil parameters without the need for laboratory tests to achieve more accurate and efficient investigations. Hence, this research aimed to design a systematic approach which allows estimating the soil parameters from the Pressuremeter test. In principle, obtain input parameters of soil in order to define soil behaviour through one of the modelling techniques by using Pressuremeter test curves and without a need for additional laboratory tests. This research practically examined a new methodology, and its feasibility has been investigated by developing an additional triaxial CU (Consolidated Undrained) tests to compare, interpret and calibrate both results by using Finite Element Method (FEM).

Main question:

Design of a systematic approach that allows the estimation of clayey soil parameters from in-situ Pressuremeter test?

Sub-question:

- Current situation analysis:

1- What are the geotechnical and hydrological conditions of the studied area?
2- How the stiff clayey soil parameters are being obtained nowadays?

- Program of requirements

3- What is the program of requirements to perform the tests followed by the soil modelling?
4- What are the special requirements defined by the client concerning the experiment?

- Assessing and choosing design variant

5- What are the available modelling techniques can be used to model and allow comparing both laboratory and in-situ test of the same location?
6- What are the pertinent criteria to be considered for the selection of the modelling technique?
7- What is the feasible modelling technique and the most appropriate to meet the research objectives?

- Detail engineering of the chosen variant.

8- What are the activities and the procedure to model the soil behaviour and allow comparing both the in-situ the laboratory results in an experimental and scientific manner?
9- To what extent are the PMT results comparable to the laboratory tests after modelling the soil behaviour?

- Discussion and Conclusion

10- What is the feasibility of the examined approach to measure and obtain stiff clayey soil parameters in-situ?

1.4 Reading Guide

The research report consists of six chapters namely Introduction, theoretical framework, Research Strategy, Results, Recommendations and Conclusion. Excluding the introduction, the content and a brief introduction to each chapter are presented below.

Chapter 2 (Theoretical Framework): In this chapter, a literature review of a similar project, detailed theoretical discussions, and key concepts will be held to form a base of the research process. Specifically, the chapter will analyze the current situation of the project area in term of the geotechnical and hydraulic conditions. Thereafter, an analysis of the functional and technical requirements for the relevant stakeholders of this research will be presented. The three selected variants will be further introduced together with the pertinent criteria that used for assessing the variants. Finally, an introduction to the codes and standards that were used to achieve the defined requirements will be briefly introduced.

Chapter 3 (Research Design and Strategy): This chapter describes the research strategy whereby the research guidelines, products, activities and planning which were undertaken to answer the main question are analyzed. The chapter analyzes each sub-question separately in term of activities, products and resources used to conduct those activities. Additionally, an overview of the communication strategy held by the author and the relevant stakeholders (the host organizations and the educational institute) is presented in this chapter.

Chapter 4 (Results): This chapter will epitomize the research results by means of showing the outcomes of the activities stated in Chapter 3 before discussing and interpreting those results in Chapter 5. In general, the chapter will present the evaluation of the different variants and the conclusion of the chosen one. Additionally, the outcomes of the in-situ and laboratory test will be shown and together with its further modelling outcomes.

Chapter 5 (Discussion): This chapter will discuss the results listed in Chapter 4 and the reasons behind the possible result variations will be described.

Chapter 6 (Conclusion and recommendations): The concluding chapter will highlight the main research aspects followed by answering the main question and presenting the final research outcomes. Besides, the recommendation chapter will conclude the improvements which can be made to improve the research outcomes in further studies and how the undesired results could be avoided.

2. Theoretical Framework:

The base of the research and designing a new testing approach highly depends on the theoretical framework to describe the theory behind the research problem and to serve as fundamentals in implementing the further research activities. In this chapter, a literature survey has been conducted to discuss the current testing method characteristics, the geotechnical conditions of the studied area, possible modelling techniques, the program of requirements and the standards that have been used for executing this research. The desk research has risen to conservation with experts and consultation when needed.

2.1 Current situation

2.1.1 Geological conditions

In this section, the information if the geological condition of the project will be highlighted since it is a practical interest of the project. The field of study is the area next to Guadarranque River, Los Barrios and Cadiz. The exact location of the experiments is emphasized in the red circle.

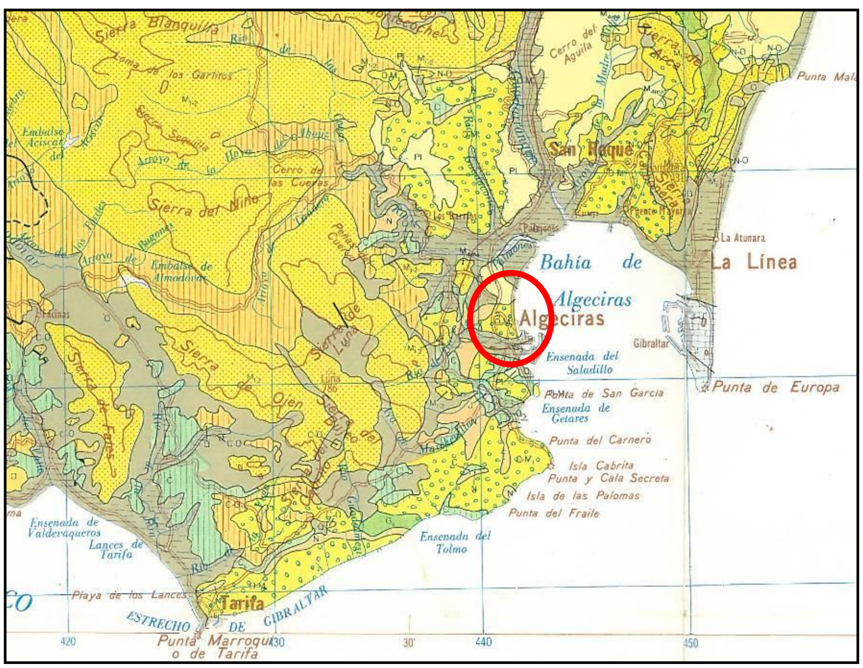


Figure 2 Field of Gibraltar Cartographic locations (Sergeyco Andalucía S.L., 2018)

Geologically, the study area belongs to combo de Gibraltar complexes. In general, these areas consist of Cretaceous-Tertiary sedimentary repetitions of the oceanic crust of the Betics flysch. Its deep marine facies are with polychrome clay lithologies and carbonated basal sandstones, mainly siliciclastic. Flysch facies are characterized by rhythmic layering with few fossils, which deposited mainly by turbidity currents or mesooceanic ocean environments and under conditions of high tectonics instability (Sergeyco Andalucía S.L., 2018).

Sergeyco Company has a database in GIS about the general geological condition of the area. The company provides the following typical geological section information representing the soil properties of the study area.

Depth (m)	Soil Description	Density (kN/m2)	Cohesion (kN/cm2)	Friction angle
0 - 1.5	Madefill (Madeground)	18.5	5	20
1.5 - 2	Weathered stiff clay	19.5	15	22
2 - 30	Stiff over consolidated marly clay	205	50	25

Table 1 Soil properties of underlying layers of the study area (Sergeyco Andalucía S.L., 2018)

The exact underlying layers of the study area were not known until the borehole was completed. But as explained earlier, the information provided by the client affirms that the area consists of a clayey soil. Moreover, and to comply with the research objectives, the exact research area were chosen after a preliminary investigation based on experience to make sure that the samples taken are including stiff clayey layers. The exact soil parameters of the tested samples will be presented later in Chapter 4.

2.1.2 Hydraulic conditions

As explained in the previous chapter, the study area mostly consists of impermeable layers of clay soil whereby water tables are not present. Figure 3 represents the aquifers distribution in the Combo De Gibraltar area, it the dotted, gray and yellow areas are the once which has aquifers. Alternatively, the dark yellow or brownie areas represent the areas where aquifers don’t show up. The city of Algeciras, as highlighted with a red circle, shows that there are no aquifers which mean that the hydraulic conditions and water tables are negligible and has no influence on the research outcomes.

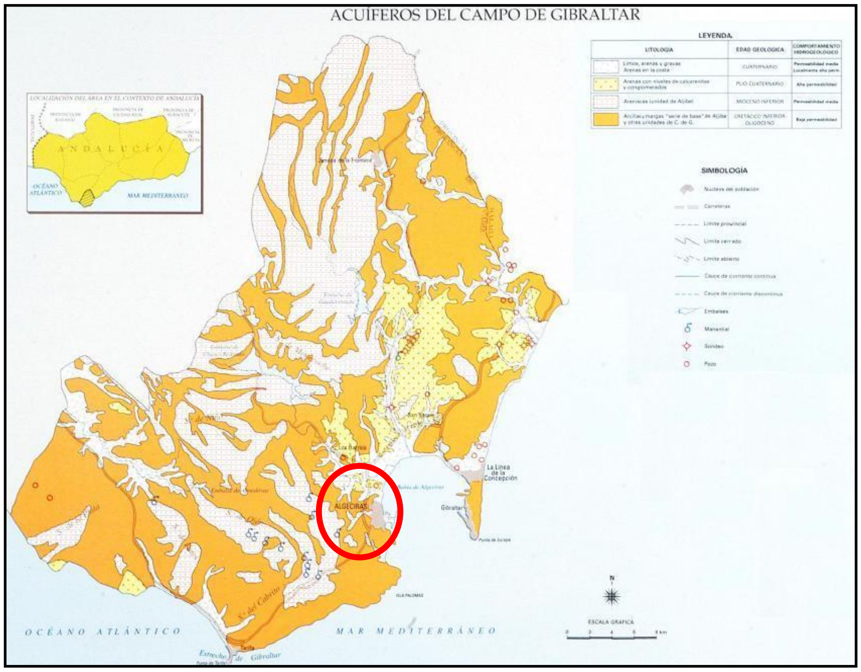


Figure 3 Aquifers of Combo de Gibraltar (Sergeyco Andalucía S.L., 2018)

2.1.3 Existing methodology

There are many projects which have been executed in a rocky or stiff clayey soil. Most of those projects have been experienced difficulties or unreliable values while trying to obtain the soil parameters from laboratory tests or using combinations of the laboratory and in-situ tests. In this chapter, a theory of similar projects and current methods used to obtain the soil parameters will be discussed.

One of the projects which involve a similar problem is the construction of the Adriatic motorway near the city of Rijeka in Croatia. The section involved complex geotechnical situations whereby limestone, flysch, limestone rocks and flysch rock mass covered by deposits of colluvial and residual soils. The project completed in 2006. Two years later, during the monitoring, a long-term deformation observed while the analysis conducted during the construction shows that the deformation is significant to the measured value. (Arbanas et al., 2007) Expressed the needs for an in-situ soil investigation or further studies since the conventional methods which obtained the parameters based on laboratory tests and Geological Strength Index (GSI) doesn't show accuracy especially for complex projects.

Another finding from the project, the same researcher affirmed that the siltstone, clay and rock mass are differed in term of the weathering grades from completely weathered (CW) to fresh rock mass (F). Since the rock mass is disintegrated in highly weathered (HW) and moderate weathered (MW) siltstone, it is not possible to get samples of it during the geotechnical investigation works. Moreover, after extracting the loads and presenting a contact with water and air, these types of soil are susceptible to degradation and disintegration in slightly weathered (SW) and fresh soil (F). Similarly, it is not possible to achieve undisturbed samples in both completely weathered (CW) to the moderate weathered soil (MW).

One of the current methods used to obtain the soil parameters in such situations is to apply Point Load Test (PLT) directly after the sampling (Ulusay, 2006). But this method results in a dispersal of the outcomes, especially in weak rock. (Arbanas et al., 2007)

As discussed in the previous chapters, the research includes both in-situ Pressuremeter test and laboratory test to compare the results and check the feasibility and accuracy of the examined method results with ordinary methods. Hereby the principle, work procedure, and applications of those tests will be described.

Triaxial test:

Triaxial test one of the most used laboratory tests in the field of geotechnical engineering to determine the shear strength and stiffness of specific soil or rock sample. The direct shear test and measurement of pore water pressure are the main features of the test. The triaxial test allows obtaining primary soil parameters such as the cohesion, the angle of shear resistance and undrained shear strength. It is also possible to measure the stiffness and permeability using the developed equipment (Gawen, 2018).

The Triaxial test works as a cylindrical specimen of the soil that bolted within rubber membrane placed into the cell that can be pressurized. Several initial preparations can be followed such making the specimen saturated, sheared, and consolidated to allow having soil response more comparative to the real in-situ conditions (Rees, 2013) During the sample, the in-situ stresses are simulated by applying stress conditions to the specimen. Figure 4 clearly illustrates the set-up procedure of the triaxial test.

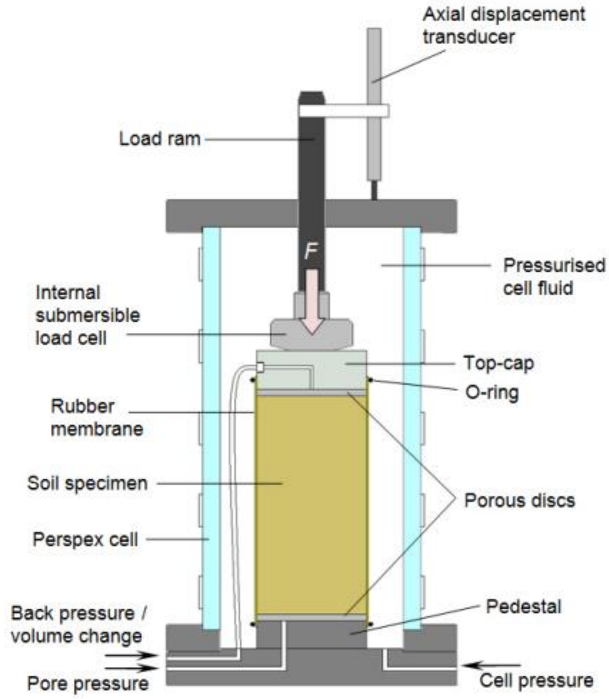


Figure 4 General set-up of a soil specimen inside a triaxial cell (Rees, 2013)

There are three different types if Triaxial test namely UU Triaxial (Unconsolidated Undrained), CU Triaxial (Consolidated Undrained Triaxial) and CD Triaxial (Consolidated Drained Triaxial). Briefly, the UU Triaxial is describing the total stress, while the CD and CU Triaxial tests are involving the effective stress (Gawen, 2018). The CU test has been used in this study.

In accordance with (Rees, 2013), the execution procedure of the triaxial test is divided into four stages as follow:

- Specimen and system preparation

The exact preparation of the specimen depends on the type of the specimen itself. For instance, for cohesive soil, trimming or cutting of the specimen from Shelby tube or block samples may take part. While for the granular soil, it involves direct preparation in the pedestal. In general, the disturbance of the specimen must be kept a minimum during the preparation.

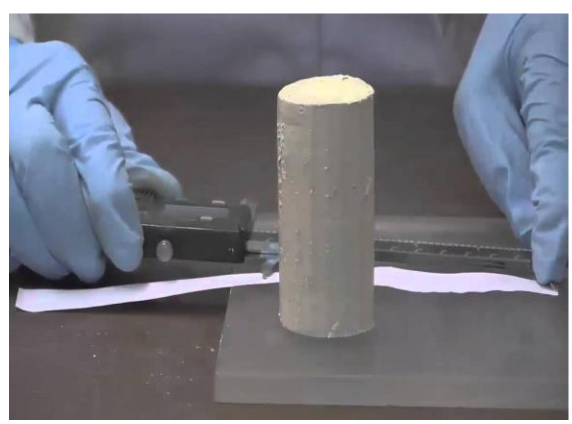


Figure 5, Specimen preparation (Nazhat, 2015)

- Saturation

The aim of this stage is to guarantee that all voids are filled with water. This can be achieved by applying vacuum to the specimen to remove the air followed by draw water into the transducer and drainage lines. The effective stress needs to be under the required shear value at all time. A short test can be done before moving to the next stage to ensure that the specimen is fully saturated. This test called B-check, and it demands the specimen drainage to be closed whereas the cell pressure is raised. The value of $B \geq 0.95$ mostly used to proof that specimen fully saturated. However, the dense soil such stiff clay may only result in a $B \approx 0.91$ even if full saturation occurred (Rees, 2013).

- Consolidation

The main purpose of this stage is to assure that the effective stress brought to the desired shearing as well as to assume the appropriate rate of strain for the cohesive specimen. This reached by increasing the cell pressure while maintaining constant back pressure. When about 95% of the pore pressure dissipated or volume change ΔV is not sufficient anymore, the process discontinued (Rees, 2013).

- Shearing

The final stage is to apply axial strain to the specimen at a constant rate by compressing and extending the load arm. The actual rate if the axial strain depends on the type of the triaxial test. Table 2 summarizes the test conditions during the shearing stage.

Test type	Rate of axial strain	Drainage
UU	Typically fastest, reaching failure criterion in 5 – 15 minutes	Closed, no excess pore pressure measurement
CU	Slow enough to allow adequate equalisation of excess pore pressures	Closed, record excess pore pressure
CD	Slow enough to result in negligible pore pressure variation	Open, record ΔV & maintain constant back pressure

Table 2: test conditions during the shearing stage. (Rees, 2013)

The specimen outcomes are monitored by placing the deviator stress (q) against the axial strain (εa). The stage continues until a failure in the specimen occurs and thus an identification of peak deviator stress, or identification of the peak deviator stress or peak effective principal stress ratio, observation of constant stress and excess pore pressure, or a specific value of axial strain being reached (Rees, 2013).

As every other testing method, the triaxial test has advantages and disadvantages. Below, the main pros and cons of the triaxial test according to (Thakkar, 2017):

- Advantages:

- Possible to have executed the test with complete control under all three drainage conditions.
- Direct measurement of the pore pressure and volumetric change
- Uniform stress distribution over the failure plane
- Adequate for accurate research work
- Allow failure of the specimen on the weakest plane (anywhere).
- Mohr circle can be drawn at any stage of the shear

- disadvantages:

- Possible to take a long time in case of drained test
- The triaxial apparatus is expensive.
- At large strains, hard to measure the cross-sectional area of the specimen accurately.

Pressuremeter test

The Pressuremeter test considered as one of the fast in-situ ways to measure the stress-strain relationship of the soil and in turn present other parameters such as the elastic modulus. Pressuremeter has been commonly used in both off-shore and onshore projects. (Geotechdata.info, 2014).

The main concept of the test is to apply pressure into the cylindrical probe that has a flexible cover that allows it to expand radially in the borehole. Due to the pressure into the hole wall, the hole volume increase which allocates the soil deformation to be measured through the pressure-volume relation. The Pressuremeter test has been used also for measuring the strength of the soil, the lateral stress in the

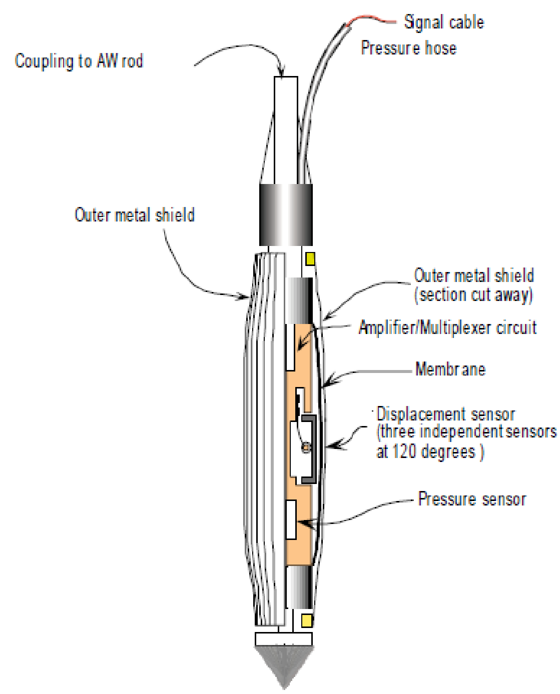


Figure 6 Schematic detail of the Pressuremeter. Source: (HOLT Engineering, 2009)

ground and the stress-strain behaviour (Cambridge Insitu Limited, 2018). There are two main components of the Pressuremeter test. The first component is the readout unit which locates above the ground, while the second component is the probe which inserted into the ground to examine the pressure. The pre-bored Pressuremeter is the one has been used in this research. Thus, a detailed procedure of this Pressuremeter type will be explained later in this section.

Pressuremeter test has advantages over the different in-situ or laboratory tests. Those advantages are summarized as following as stated by (Roger, 2017):

- The test applicable for both dense and soft soils. But it is the best to use it for dense sand, hard clay or weathered rock which can't be tested by normal bushing equipment.
- Can be used with drilling or direct pushing equipment
- Results in an extensive database which allows the geotechnical engineer to use accurately for various designs.

Moreover, (Schnaid, 2012) stated that the Pressuremeter has uniqueness in term of measuring the stress-strain in-situ and the applying cavity expansion theory. Conversely, (Cosentino, 2009) has reported some disadvantages as follows:

- The test hole must be prepared accurately in case of pre-pored Pressuremeter type.
- The Possibility of membrane failure results in a half-day delay.
- Undrained and fully drained are the only stress paths can be followed in practice
- Complicated procedures, qualified specialists are required to execute the test.

There are different types of Pressuremeter. The equipment, installation procedure and application may slightly differ for each one. Below a brief description of the different Pressuremeter types:

<u>Pre-bored Pressuremeter</u>

The pre-bored Pressuremeter is the one has been used in this research. The instrument is placed in the hole that pre-formed using traditional drilling tools. The main defect in this method is the complete unloading of the cavity that occurs between the stages of removing the drilling tool and applying the pressure. The method suits both rock and stiff clay. But it is important to mention that the self-bored method provides shows data related to the cavity that may have to expand before the insertion disturbance erased. The operation itself requires supporting from the drilling rig which makes it possible to conduct laboratory test for the same cored material and compare the results. Additionally, the routine depth of the pre-bored method is 200 meters, but in fact, there is an experience of executing the method to greater than 500 meters. If the method used for dense sand, a drilling mud needs to support the open borehole. However, the methods seemed not suitable for loose sand (Cambridge Insitu Limited, 2018).

The test procedures start with drilling to prepare the test hole to the desired test level which is the most important step of the test. In order to make the test cavity satisfactory, the diameter of the hole has to be within the specified tolerance and the drilling equipment has to lower the possible disturbance to the surrounded soil and hole's wall. When this step is complete, the test has to be performed immediately precedent by cleaning any debris or cuttings. The test starts with setting the V0 at 0 (volume of the measuring portion of the uninflated probe at 0 volume reading at the ground surface, cm3) by de-airing all circuits while the probe at atmospheric pressure. Then, lower the probe to the test depth as the depth of the midpoint of the probe and starts applying pressure in an equal incensement until the limit of the equipment reached. Readings of the volume V0 have to be taken after 30 seconds and 1 minute after the pressure increment applied. Once the maximum volume or pressure reached, the test stops by deflating the probe and take it out (ASTM International, 2016).

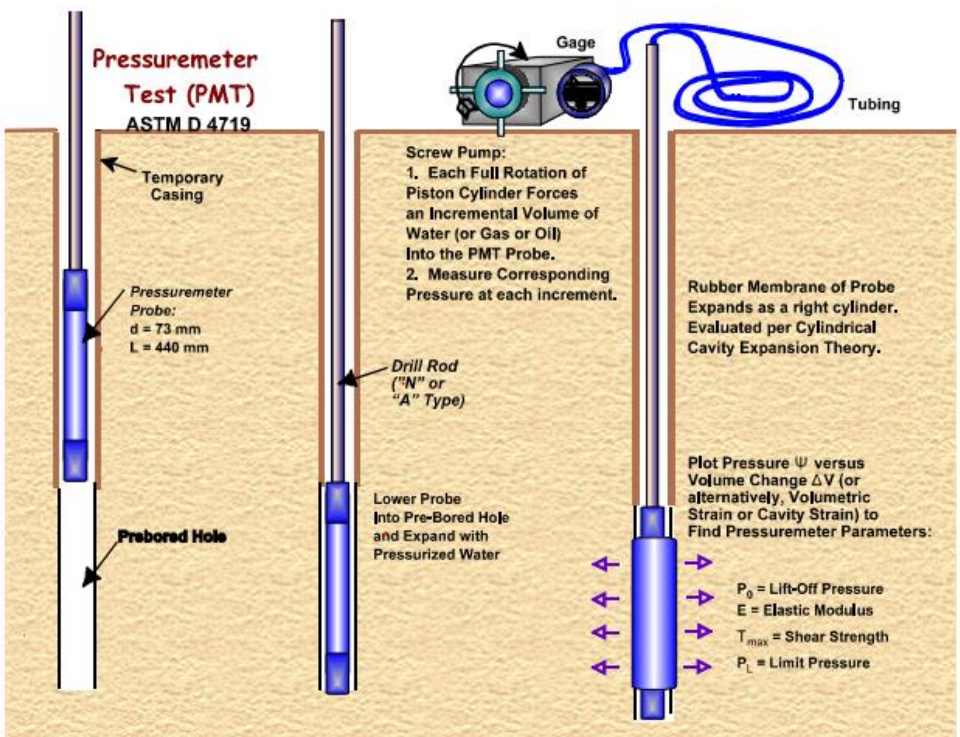


Figure 7 Procedures of Pre-bored Pressuremeter test (GeotechnicalDesign.Info, 2017)

Self-boring Pressuremeter:

Self-boring Pressuremeter has considerably the lowest disruptive where the disturbance is small enough and within the elastic range of the material (Campridge Insitu LTD, 2017). And it is mainly designed to minimize the disturbance to the surrounding soil (Schnaid, 2012).

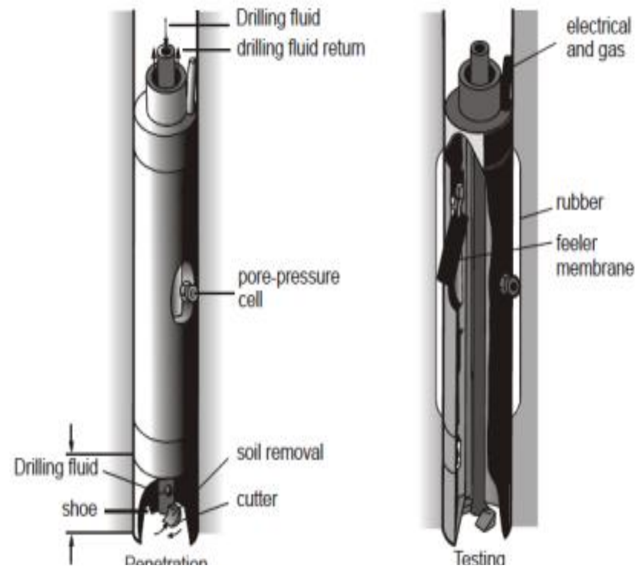


Figure 8 Self-boring Pressuremeter instruments

The installation starts by making a pocket in the whole using the diminutive tunneling machine in which the device exactly fit. A sharp-edged cutting shoe also attached to the foot of the device. When boring starts, the instrument jacked into the ground and a rotating cutting device tends to cut the material by the sharp-edged cutting shoe. In stiff soils, it is usual to use a flush with the cutting shoe as the distance between the leading edge of the shoe and the cutter head to allow the cutting device to take multiple forms.

The Self-boring method is suitable for soils ranges from loose sand and soft clay to weak rock. Other materials, such as gravel and hard rock are not measured with self-boring instrument since it may damage the cutting edge. The self-boring technique usually takes part in 60 meters depth or more for a vertical hole. It also required the modest amount of reaction and less supporting tools, especially for soft clay.

Pushing Pressuremeter

This method involves full disruption and raising the stress of the soil surrounds the probe during the penetration (Schnaid, 2012). During the loading stage, the limit pressure is the only obtained parameter because of the full disturbance of the soil. The strength parameters are obtained from the contraction curve while the stiffness parameters from the rebound cycle outcomes. The main advantages of this method are it enables direct measurement for stiffness and strength parameters, considerably faster than the other methods and it can be done in all type of soil that allows inserting the cone. In contrary, the stresses required to make persuasive test are higher compared to the other methods which may result in crushing of the soil particles.

The previously mentioned tests, triaxial and Pressuremeter, are the once that are relevant to this study. For the Pressuremeter test, the research will be restrained to only pre-bored Pressuremeter. This considers the fact of the type of soil examined since the pre-bored Pressuremeter is the more suitable for the stiff soils. Moreover, Sergeyco Company is well-experienced in this type of tests. And in keeping with (Townsend et al., 2001), both tests were sufficient to attain the defined scope and interpreting the results.

2.2 Program of requirements

In this part, the research specifications and demands which have been accomplished were listed as a program of requirements. Those requirements were divided into two parts namely functional requirements and technical requirements.

2.2.1 Functional requirement:

Those requirements were mainly derived from client's goals and the research objectives. The further requirements were defined according to the main research's objectives with dimensional tolerance declared by the client. Literally, there are no direct requirements concerning the cost or accuracy. But in fact, some of the below mentioned requirements are based on accuracy, time, and cost issues as clarifies below.

Location

The location of the in-situ test has to be in the area of Combo De Gibraltar. This was due to limit the travel expenses as well as due to the availability of the type of soil to be tested.

Soil type

The client's main goal was to develop or improve the current methodology used to obtain stiff clayey soil parameters. Therefore, stiff to the rocky soil is the allowed area to conduct the experiment and has to be considered. This also concerns the level of accuracy that the client wants to acquire.

Finite Element Method

The research conclusions should allow using the FEM based on the experiment conducted. Apart from the different numerical methods used to solve the mathematical problems, the Finite Element Method was required in this research to be used for the soil modelling. Therefore, the calculations were held using Finite Element analysis (FE). The use of Finite Element analysis modelling has been increased in the field of geotechnical engineering. This mainly due to the fact this kind of analysis tends to amend and control the engineering tasks (Obrzud, 2010). And it confers modelling different geometric and soil conditions in addition to the diverse interface with complicated and non-linear behaviours (Jalali et al., 2012). The Finite Element modelling provides more realistic indications for the ground movement and states the pre-failure behaviour of the soil and non-linear –stress-strain relation before it meets the ultimate state. Those behaviours are known by its big differentiation of the soil stiffness and the pre-failure stiffness plays a substantial role in term of modelling common geotechnical problems such as retaining walls and tunnels excavation in dense areas.

PLAXIS

It can be noted that there is no requirement considering the accuracy or the cost. But in fact, the PLAXIS software has been elected due to its availability within the facilities of both UCA and Sergeyco and to avoid any further expenses of buying other software and further stuff training needed. Besides, the PLAXIS software has been selected due to its accuracy. For instance, there are different software can be used for modelling the soil behaviour, the input of those programs may slightly differ. In this project, clients required the use of PLAXIS Software for modelling the soil behaviour after obtaining the soil parameters. This as a result of the usual practice of the client, the availability of the different modelling alternatives as well as the worldwide standards used by PLAXIS. (AECOM, 2009).

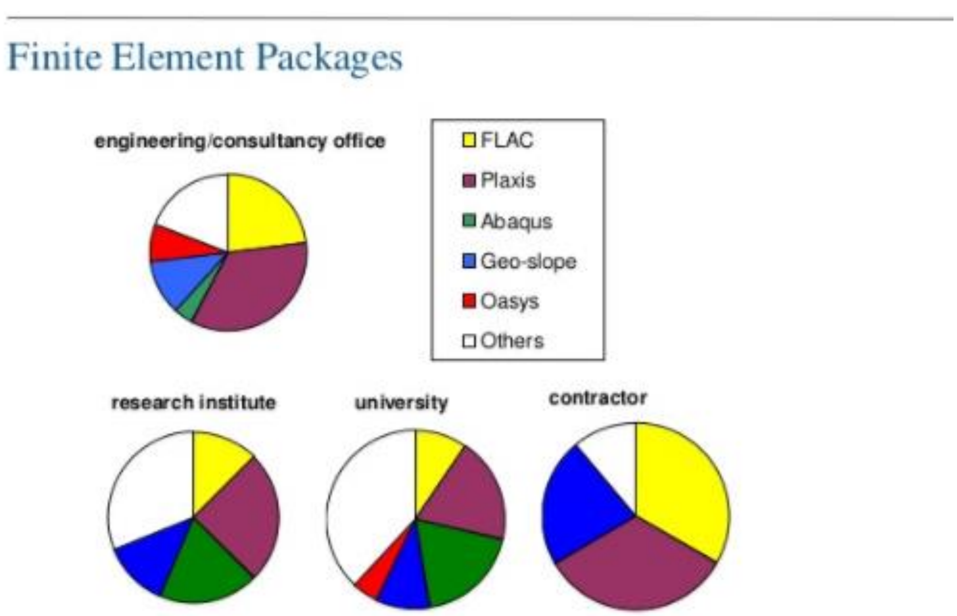


Figure 9: Popular Finite Element software used for modelling the soil behaviour

PLAXIS is software developed by TU Delft University which intended initially for analyzing soft soil before it has been developed for extensive geotechnical issues. PLAXIS become the most used software nowadays for modelling soil in consultancy offices, universities, research centres, and contractors. (AECOM, 2009)

Apart from its ability to provide analysis of deformation and stability, PLAXIS allows modelling of the interaction between the soil and the structure as well as the hydrostatic pore pressure of the soil. The PLAXIS boundary conditions are defined in Chapter 3.2.4.

2.2.2 Technical requirements:

The technical requirements are the technical parts that the research has to fulfil. The research consists of field and laboratory works. That has been done according to standards used in Sergeyco Company. In this part, the general standards and codes will be mentioned.

- Performing the Pressuremeter test according to ASTM Standards ASTM D2850, D4767 and D2166 (ASTM Interational, 1994)
- Performing the Triaxial test according to ASTM Standards ASTM 4767
- Analyzing the triaxial test and PMT to extract necessary parameters based on the MCA results.
- Optimizing the soil parameters obtained from triaxial testing.
- Modelling soil behaviour using PLAXIS based according to Eurocode 7.

Detailed technical requirements and specifications of the soil testing are explained later in Chapter 3.2.2. Additionally, modelling procedure, limitations and boundary conditions are presented in Chapter 3.2.4.

2.3 Modelling techniques

A lot of constitutive modelling techniques have been developed especially in last four decades for modelling the stress-strain behaviours of the soil. Some techniques formulated based on theoretical principle and others based on experimental evidence (Lade, 2005). In this research, the constitutive modelling techniques were the design variants that have been evaluated based on the criteria chosen based on the research objectives. There are more than 10 modelling techniques are available in PLAXIS Software, but the client suggested to conduct the research within one of three techniques namely Mohr-coulomb, Hardening Soil and Cam-Clay modelling technique. This suggestion based on the experience in the field, availability, the type of soil, and the relevance of the research objectives. Therefore, this research confined to those techniques only.

The constitutive modelling techniques initially developed based on two laws namely the Hoke's law and Coulomb's law. The first is made upon the linear elasticity for describing the soil behaviour and the latter upon perfect plasticity for describing soil behaviour under collapse state. The Combination of those laws was made which known as Mohr-Coulomb model. And since the soil is not linearly elastic and has very complicated behaviour, different constitutive models have been studied and proposed to define the soil characteristics and behaviour in details and implement such Finite Element method for the geotechnical engineering perspective (Ti, 2009). This emphasizes the importance of the modelling techniques to compare the current and examined method for obtaining the soil parameters and its effect on the soil behaviour. According to (Lade, 2005), the advance soil modelling involved in solving different geotechnical problems such soil reinforcement and anchorage, dams, embankment, tunnels, and settlement due to fluid extraction in addition to cut slopes. Hereby an overview of different modelling techniques will be discussed with its purpose, limitation, and principles to allow allocating the most suitable technique for this project.

2.3.1 Mohr-Coulomb model:

Mohr-Coulomb model considered to handle what is called first-order which is to obtain a first approximation of soil or rock behaviour. This returns to the fact that this model deals with an assumption of plastic-perfectly and elastic-perfectly as clearly shows in Figure 10 which in turn offers an advantage of making the analysis runs fast. (Plaxis, 2011).

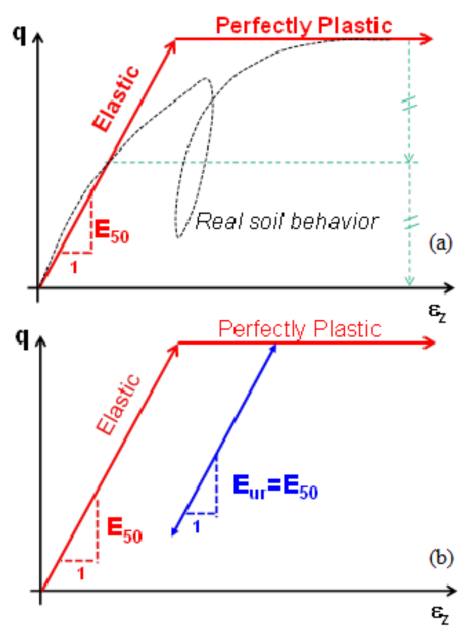


Figure 10 Elastic-perfectly and Plastic-perfectly assumption of MC model (Gouw, 2014)

The input parameters needed to for this model are Poisson's ratio (ν), cohesion c, friction angle phi (ϕ), dilatancy angle psi (ψ) and Young's modulus (E) (Ti, 2009). The Mohr-Coulomb is very applicable and simple model technique especially when it comes to three-dimensional stress space and due to the need for only two strength parameters. The areas of application of this model are shallow foundations, slopes, the stability of dams and embankments. In contrary, one of the main disadvantages of this model is the overestimate of the soil strength which may lead to unpractical design decisions and soil failures (Surarak et al., 2012).

For retaining wall case, the Mohr-Coulomb model uses a linearly elastic response for the soil behind the retaining wall, which in turn leads to underestimation of the horizontal displacement or wall deflection (Obrzud, 2010). Figure 11 illustrates the results of computing the retaining wall in Berlin sands using the Mohr-Coulomb modelling technique. The figure provides global information about the wall lifting, but there is no precise prediction for the settlement or the displacement.

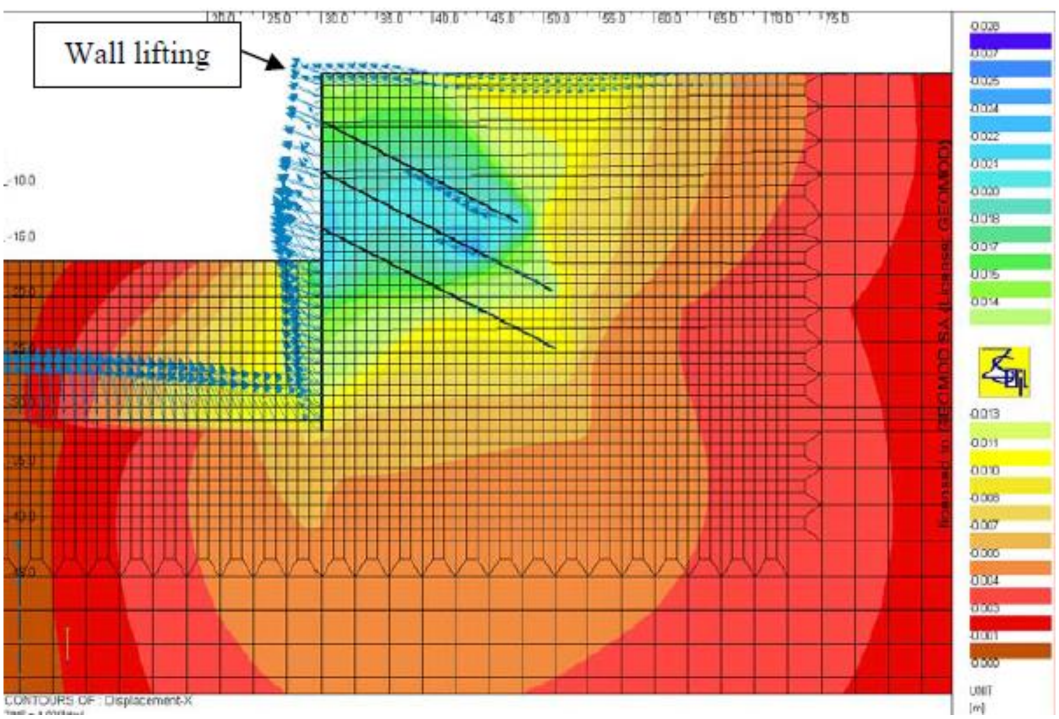


Figure 11 Results of computing retaining wall using Mohr-Coulomb model. Source: (Obrzud, 2010)

2.3.2 Hardening Soil model:

The need for Hardening Soil modelling arises as a result of the fact that the soil behaviours which stems from nature are more complex and provides elastic and plastic non-linearity that is hard to be calculated without such advance modelling techniques (Obrzud, 2010). A development has been made to Mohr-Coulomb model which arise the Hardening Soil (HS) model at first (Surarak et al., 2012). This modelling technique is accounting for two hardening mechanisms namely isotropic and deviatoric. The first one is a formulation of the cap mechanisms which developed to measure the threshold point below which significant plastic straining takes place. The isotropic modelling plays an important role when modelling consolidation problems related to the footing or groundwater lowering. Moreover, it allows degradation of soil stiffness while raising the strain. The second mechanism, deviatoric, formulated to take over the soil hardening produced by the plastic shear strains. This can be seen in such as settlement behind a retaining wall occurs, whereby a domination of plastic shear strains can be observed for soil elements (Obrzud, 2010). Hardening Soil model is capable to analyze the behaviour of both soft and hard soils as well as its adequate to model any type of application (Ti, 2009). To ensure an accurate modelling of the soil stiffness, and in contrary to other modelling techniques, three input stuffiness are needed: Triaxial loading stiffness, triaxial unloading stiffness and the oedometer loading stiffness rather than friction angle, the cohesion, and the dilatancy angel. Those stiffnesses are corresponding to the triaxial loading, triaxial unloading, and oedometer tests. It is acceptable to have data from Triaxial or Oedometer test but for better quality data a one type test correlations or in-situ test such Pressuremeter test is more preferred (PLAXIS, 2016). In total, 10 parameters are necessary to be obtained whether from laboratory or in-situ tests to conduct the Hardening Soil Modelling For stiff clayey soil; oedometer or Triaxial test can be used to obtain the parameters. Table 3 states the parameters and its descriptions.

Parameter	Description
Φ’	Internal friction angle
C’	cohesion
R_f	Failure ratio
Ψ	Dilatancy angel
E_{ref50}	Reference secant stiffness from a drained triaxial test
$E_{ref\ oed}$	Reference tangent stiffness for oedometer primary loading
$E_{ref\ ur}$	Reference unloading/reloading stiffness
M	Exponential power
v_{ur}	Unloading/reloading Poisson’s ratio
K_o	Coefficient of earth pressure at rest (NC state)

Table 3 Hardening soil model input parameters

The model contains two types of hardening, shear and compression hardening. Therefore, the model is accurately analyzing situation whereby a reduction in mean effective stress and mobilization of shear strength occur, such as retaining walls and tunnel construction projects. It is also known for its accuracy to forecast the displacement of different geotechnical situations and diverse applications.

In contrary, despite the fact that the Hardening Soil (HS) modelling considered an advanced modelling technique for predictions of complex soil behaviour, there are several limitations for specific soil which can't be approximated. One of those limitations is that the HS is not able to account for the behaviour related to soil dilatancy and soil restructuration which is mostly insensitive soils. Moreover, it is not possible to obtain hysteric behaviour during the cycling loading (Obrzud, 2010).

An example of the modelling outcomes, Figure 12 shows the results of the Hardening Soil (HS) modelling for the Berlin sand case. It can be clearly seen that with the HS produces realistic information of the settlement behind the wall as well as a prediction of the direction of the horizontal soil displacement.

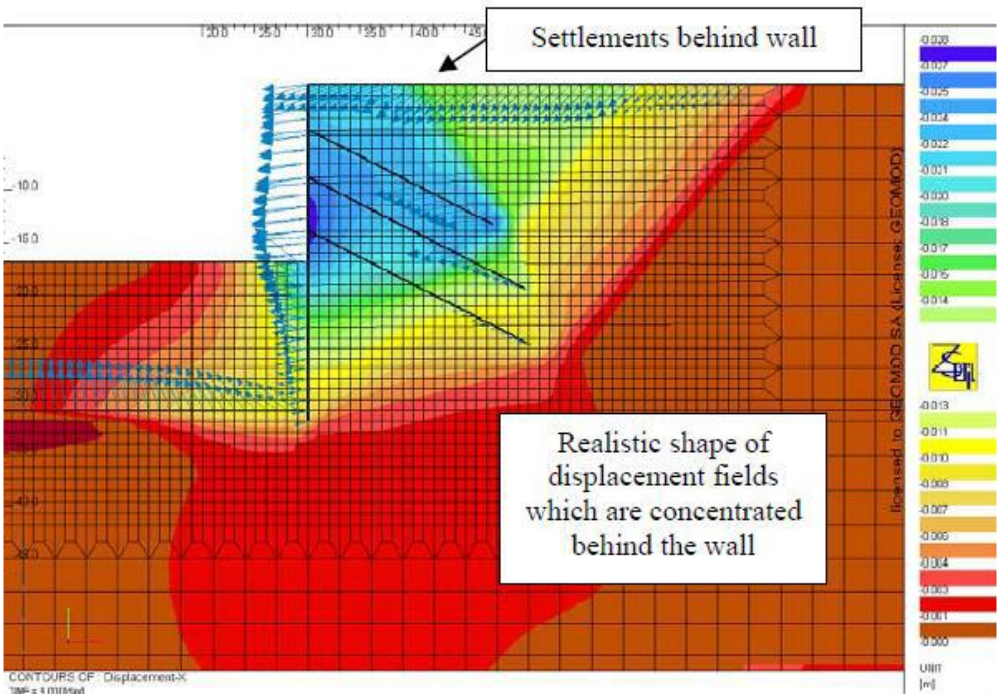


Figure 12 results of computing retaining wall using Hardening Soil model. (Obrzud, 2010)

Finally, it is worth to mention that an additional computational effort needed for this model. The user may conduct many irritations for each computational comparing to the simple model. It is also depending on the number of stages and phases that need to be modelled and the type of modelling as most techniques can be observed by 2D or 3D.

2.3.3 Cam-clay model (CC)

The yield point controlling the stress-strain response is capable to degrade in term of rebounding at high over-consolidation ratios. This may result in a bonding breakdown. The main principle of the Cam-Clay modelling is to allocate the degradation of the yield point and its influence on the soil behaviours (Kraft & Amerasinghe, 1983). In other words, due to the reloading which leads to residual strain, irreversible straining occurs before the maximum stress reached. The Cam-clay model is the use of the stress hardening theory of elasticity to contrive full stress-strain model of normally consolidated of lightly over-consolidated clay in the triaxial test. The model has been modified to an elastic plastic strain hardening model which allows modelling the non-linear behaviour through hardening plasticity (Ti, 2009). The Cam-clay (CC) and Modified Cam-Clay (MCC) are both represent three significant parts of the soil behaviours clarified as the strength, compression or dilatancy and the Critical state at which soil particles can experience unlimited deformation without any changes in stress or volume.

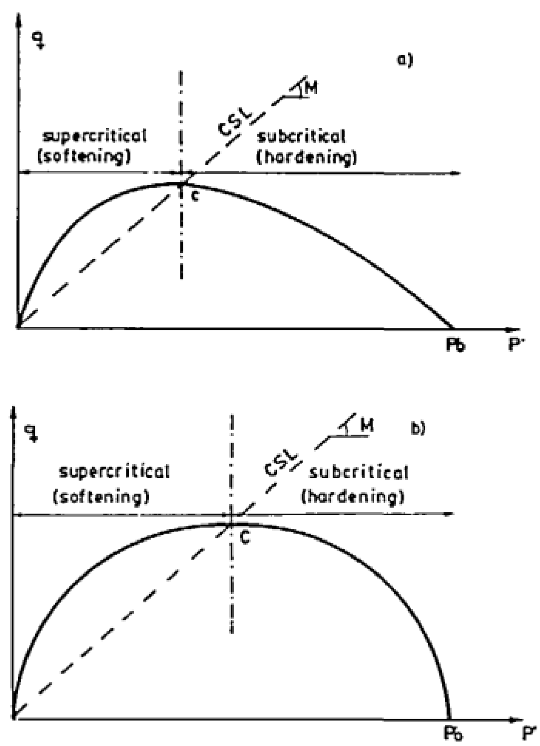


Figure 13 (a) Yield surface for cam clay model; (b) Yield surface for modified cam-clay model. (Potts & Gens, 1988)

As reported by (Ti, 2009), the model is based on the Critical State theory. This theory assumed the logarithmic relation between the mean effective stress (p) and the void ratio (e). Figure 14 shows the Virgin compression and recompression are linear in (e-nl) (p) space which is suitable and most realistic for near-normally consolidated clay but it is noted also by (Potts & Gens, 1988) that this theory is not suitable to model silt, saturated clay and stiff overconsolidated clays.

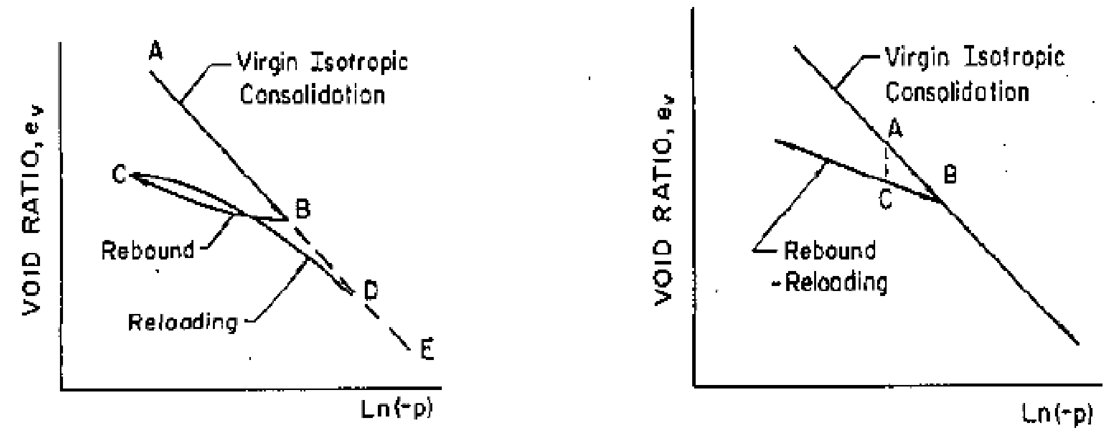


Figure 14 Response of real soil to hydrostatic stress and Response of idealized soil to hydrostatic stress. (Ti, 2009)

The Cam-Clay model is very practical for the modelling of the deformation rather than failure, particularly in the normally consolidated soft soil. Additionally, it is more applicable and performs best in modelling projects that involve loading situations such as embankment and foundations.

As stated by (Doherty, Alguire, & Wood, 2012), there are five input parameters needed for the model as listed in the table below:

Parameter	Description
Λ	Isotropic logarithmic compression index
K	swelling index
M	friction constant
E	initial void ratio
v_{ur}	Unloading/reloading Poisson's ratio

Table 4 Cam-Clay input parameters

Both k and λ can be specified either triaxial test or oedometer test in which both give same values (ZACE Services Ltd, 2011). It is also known that those parameters may need an optimization because those parameters may not be available directly from the test data. Thus, reduplicate or irrigation preaches may be required in the most cases for identifying the parameters needed for the Cam-Clay model. This approach is required further modelling and changing the modelling parameter itself to reach match with those parameters obtained from the laboratory test. This optimization process is time-consuming and will be further discussed if the Cam-Clay will be the feasible alternative for this project (Doherty et al., 2012).

2.4 Assessment criterion:

The criteria that have been used to select the variant have further explained in this section. Those criteria are chosen based on the previous experiments and with the supported by consultation with expertise and client. Some of those criteria can be considered as a project boundary conditions as to keep the specific criterion to the minimum such cost and time. While some others had to be within reasonable values such the precision.

2.4.1 Precision

As clarified earlier in chapter 2.3, the reality and precisions of predictions are varying among the different alternatives depends mainly on their assumption of the stress-strain relations and possibly other factors. And in line with (Ti, 2009), the main criteria for choosing the modelling technique is to assure its consistency with theoretical requirements in terms of continuity, stability and uniqueness. Hence, the theoretical requirement of any research is to achieve more adequate results which require precise execution techniques. Moreover, (Obrzud, 2010) reported that the choice of the modelling technique highly depend on the expected precision and prediction. Thus, the more accurate and precise is more favourable to attain the research purpose. As long as the precision is the most important aspect towards the research objective and the result accuracy, it had been given an importance of 35% out of the total criteria.

2.4.2 Applications:

The geoengineering computing can be divided into two main parts; those related to Ultimate Limit State (ULS) such wall stability and slopes assessment. And the once related to Serviceability Limit State (SLS) such deep excavation and tunnel excavations (Obrzud, 2010). Chapter 2.3 illustrates that each technique is more applicable in a specific situation than other. Even though there is no specific application has been planned for the tested area, this will be taken into consideration during the assessment of the alternatives as the virtual load that is a deep foundation in this case. This took in consideration if further studies to be optimized using such structure as it is the most common application in the area. Therefore, and since there is no real structure to be model in this research while this criterion considered only if a virtual loads needed to be studied -such the deep foundation in this case- , this criterion had been weighted 5% out of the overall scale.

2.4.3 Cost:

Apart from the mutual cost of the testing execution, such the mobilization, field inspection, site clearing… etc., the different modelling techniques need different input parameters. Hence, there are different tests required to be carried out which means that the prerequisite equipment, labor, and further soil analysis will differ. The cost is an important criterion for any project; the lower cost is the more preferred option. Therefore, the cost had been given a significance of 25% in this project.

2.4.4 Time:

The time of performing the modelling itself may vary for each modelling in the software due to the need for examining of different cases and loading situations and the time needed for modelling these cases may vary in each modelling technique. Even though the time of in-situ and laboratory testing are different for each technique, and this mainly depends on the requirements of the technique, the difference between the involved techniques is unremarkable. Therefore, this criterion had been given an importance of 15%.

2.4.5 Experience-based evaluation

This criterion aims to evaluate the model based on the previous practices of the experts and assess the suitability to fit the experimental objectives and available parameters. In accordance with (Ti, 2009), the second main criterion of assessing the constitutive modelling technique is evaluating the appropriateness to fit the variety available tests and the ease of determining the material parameters from the available data. More so, (Obrzud, 2010) reported the available knowledge of the material plays a key role in the choice of the constitutive model. As a result, and since the experience is more accurate than the theoretical provided information concerning the knowledge of the material and the suitability to fit the available tests, this criterion had been given a weight of 20%.

2.4 Safety and Design Standards

The standards must be addressed before any engineering practice, this to ensure that materials, products, processes and services are appropriate for their intent. Therefore, the following international and national standards were considered during the execution of this research based on the approach that the Sergeyco Company follows in similar practices.

2.4.3 ASTM:

ASTM is the American Society for Testing and Materials. It develops and publishes voluntary consensus technical standards for different materials, products, systems, and services (ASTM, 2018). The laboratory and in-situ practical tests were preceded based on ASTM standards. This as a result of the usual approach of the local branch of Sergeyco Company in Spain, the availability of the standards and the working procedures.

2.4.4 Eurocode 7: Geotechnical Design

Even though the Eurocode 7 is not very popular to use within Spain, the PLAXIS Software offers to model according to the Eurocode. Therefore, the modelling and calculations of the soil behaviours and allocating the partial factors were conducted according to the Eurocode 7.

Sergeyco

3 Research Design and Strategy

This chapter clarifies the research design and strategy that illustrates the research activities were executed to come up with the following stated results. Firstly, the research sets out with analytical examinations including literature review and interviewing experts. Next, an experiment was held involving modelling and explicating results before reaching the final conclusion and recommendations. The steps toward the research outcomes and the execution of mentioned elements will be described minutely in this chapter.

3.1 Communication:

The research involved three different organizations, the communication between the researcher and the organization was as follow:

3.1.1 Host Organizations:

The host institution is the organization where the research was held and supported by means of consultation, laboratories, and local conditions during the research period from February 2018 to June 2018. In this case, there were two host organizations and the research conducted in both locations depends on the research process and needs.

Organization name	Universidad de Cadiz
Visiting address	s/n, Av. Ramón Puyol, 11202 Algeciras, Cádiz
City	Algeciras
Country	Spain
Phone	+34 956 02 80 00
Host Institution supervisor	Prof. F.J.M. Aguado
E-mail	paco.moreno@uca.es

Communication

Communication with Prof. Moreno took place through day-to-day meetings for expert advice, local conditions, and guidance regarding the process of the research.

Organization name	Sergeyco Andalucía S.L
Visiting address	Carretera San Roque - La Línea km 1, 11360 San Roque, Cádiz
City	San Roque
Country	Spain
Phone	+34 956 78 00 76
In-company tutor	Francisco Javier Manzano Diosdado
E-mail	franciscojavier.manzano@uca.es

Communication

Communication with Eng. Javier Manzano was via e-mail and weekly meetings for expert advice, local conditions, and experiments requirements.

3.1.2 Educational Institute:

Organization name	HZ University of Applied Sciences
Visiting address	Edisonweg 4, 4382 NW Vlissingen, Netherlands
City	Vlissingen
Country	The Netherlands
Phone	+31 118 489 000
In-school supervisor	G.Scuderi
E-mail	scud0001@hz.nl

Communication

Communication with Dr. Scuderi through e-mail and Skype meetings to follow up the research progress and feedback related to the research's requirements and competencies.

3.2 Activities and Products:

To answer the main question, the sub-questions mentioned in Chapter 1 had to be answered. The activities were carried out to answer each sub-question will be described later in this chapter. Figure 15 Research strategy flowchart summarizes the main research products and provides an overview of the sequence of the research activities and strategy.

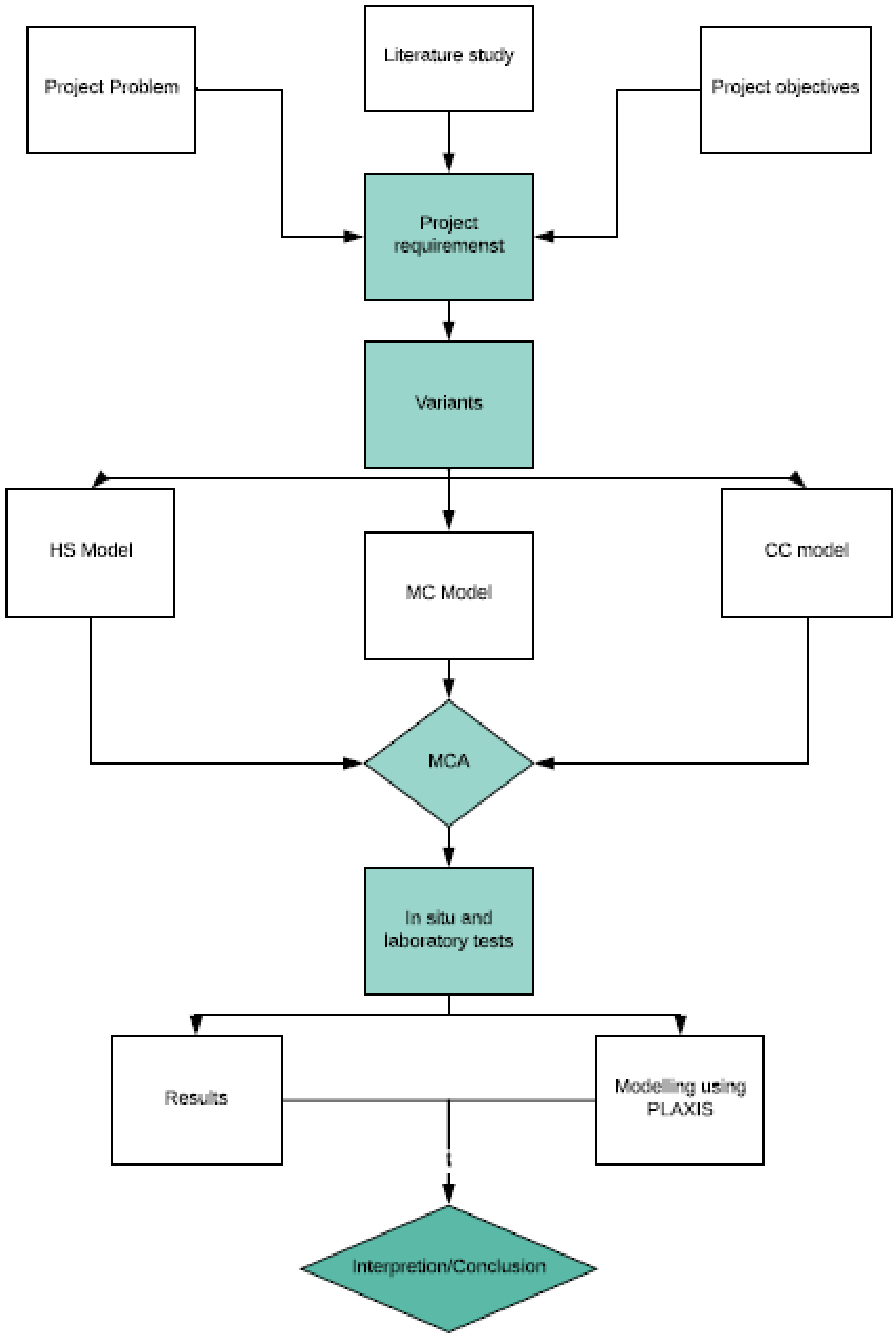


Figure 15 Research strategy flowchart

3.2.1 Current situation analysis (Answering sub-question 1 and 2):

The below mentioned activities are carried out and their pointed products can be found in Chapter 2. The table below shows the activities conducted and the reality of the source that leads up to the required products.

	Activities	Products	Sources
1	Meeting with the client	○ Define the geotechnical and hydrological information from the documents provided by the client ○ List of boundary conditions ○ List of starting points ○ Knowledge about existing situation and methodology	○ Document provided by the client. ○ Previous studies
2	Desk research and literature review about the study area and previous similar studies		

3.2.2 Program of requirements (Answering sub-questions 3 and 4)

The program of requirements is important to provide comprehensive specification and guidance to the research. To draw the schedule of requirements, the research goals and client wishes have been taken into account. A functional requirement has been listed earlier in chapter 2.2 to meet the client's expectations. Besides, preliminary technical requirements defined in 2.2 based on the codes and standards that were used in the research. For the tests, there was no specific protocol have been followed in UCA or Sergeyco. Hence, the national and international standards used for the tests procedures. Moreover, there are no certain requirements for FE modelling. However, boundary conditions have to be defined depending on the situation and the project. Therefore, a list of boundary conditions which were used in the project are listed in Chapter 3.2.4

Fieldwork: Pressuremeter testing and sampling

- ❖ Scheduled: Week 13: 26th March 2018
- ❖ Location: Algeciras
- ❖ Equipment:

- Data acquisition system

- Pressuremeter. Type: Elastmeter-2. Which consist of the following instruments: (See Appendix A: Pressuremeter Model and Specifications)
    - Probe: consists of three parts: two guard cells and one main cell (measuring cell)
    - Control unit
    - Tubing
    - Membrane
- Drilling rig. Type: DeltaBase 520/525. (See Appendix B: Drilling rig Model and Specifications) includes:
    - Shelby tubes
    - Sampling tools

❖ Technical and safety requirements:

- Performing the Pressuremeter test according to ASTM Standards ASTM D2850, D4767 and D2166 (ASTM Interational, 1994)

  - Safety requirements (California Department of Transportation, 2017)
    - Implement and approve site safety plan
    - Communicate to the Driller Worker about the operational needs for the drilling program
    - Ensure wearing the needed protective devices
    - Safety information in case of an accident
    - Discontinue the field work if any unsafe condition exists
  - Site preparation
    - Site clearance
    - Define borehole location
    - Equipment mobilization
    - Barrier
  - Test Execution according to ASTM standards D4719 including (See Appendix C: PMT ASTM standards):
    - Boring/Drilling

- Drilling desired depth
- Calibration of the membrane
- Applying pressure
- Load-deformation and volume-change diagrams deducted

❖ Aim and General Test Procedure

The test performed to observe the deformation and to obtain the stress-strain relation of the soil by applying a pressure to the borehole sidewalls. The test started by means of site preparation and defining the borehole location. After that, the borehole drilled and a Shelby sampling conducted for the laboratory tests by forcing the sampler into the soil using a constant pressure. Meanwhile, the probe positioned in the borehole at the same depths where sampling performed and an increment of the equivalent pressure applied. Next, the outcomes and readings of unload-reload, stress and strain at the start of any load-unload cycles, and the pressures in the transducers are recorded at a frequency of 30 seconds intervals were specified. These activities were repeated in depths 1.5m, 7m and 9m. Further test outputs and results can be found in Chapter 4.

Figure 16 In-situ test activities and equipment

Lab work: Triaxial test

- Scheduled: Week 16 - 17 (16th April to 27th of April)
- Location: Universidad De Cadiz (UCA) laboratories.
- Equipment:

- Triaxial apparatus (Compression machine, triaxial cell accessories, control panels and system accessories)
- Data acquisition computer

- Technical requirements:

- Performing the Triaxial test according to ASTM Standards ASTM 4767 (Appendix D: Triaxial Test ASTM standards)

    - Apparatus

        - Required apparatus to perform satisfactory test is correspond to what described in ASRM D4767 section 5

    - Specimen preparation

        - test Specimen preparation is done according to ASTM D4767 6.1,6.2 and 6.3

    - Mounting specimen

        - Before mounting the specimen into the triaxial chamber, preparations specified in ASTM D4767 7.1.1-7.1.4 has been followed
        - For a wet mounting method, procedures specified in ASTM D4767 7.2.1-7.2.5 have been followed
        - Placing the rubber membrane around the specimen with a positive seal at each end
        - Adding the drainage at the top and checking the alignment of both specimen and specimen cap.

    - Performing the test

        - Proceed with the test corresponding to the rules and conditions in ASTM D4767 section 8

- Removing the specimen
    - Removing the specimen after the shear is completed, referring to the instruction in D4767 section

❖ Aim and General Test Procedure

The test performed is a consolidated undrained (CU) test. This mainly aims to extract the strength parameters of the soil under effective pressure. The parameters were later used in modelling the soil behaviour and as input parameters for the modelling. The test carried out using the apparatus mentioned in the previous section (See Figure 17). Considering the different samples from different depths, the test executed for more than five specimens which took more than a week to be completed. The test starts with preparing the specimen which had a diameter of 3.8 cm and 7.7 cm height in average for the different samples. The preparation stage involved extracting the specimen from Shelby tubes, trimming and placing a rubber membrane around the specimen before placing it in the triaxial cell and being filled with fluid. Then, a vacuum and effective pore pressure applied to ensure no voids have remained in the specimen as this called the saturation stage. This was guaranteed by determining the Skempton's B-value whereby the cell pressure increased and drainage closed. Then, an increase in the cell pressure applied with performing a back pressure to bring the specimen to the effective pressure required. Finally, a slow axial load applied and the drainage kept closed while the excess pore pressure recorded until the failure occurred. All the results and records of the triaxial test can be seen in chapter 4. These activities were repeated for the different samples from different relevant depths. Method of the analysis and interpreting the results are later described in Chapter 3.2.4.

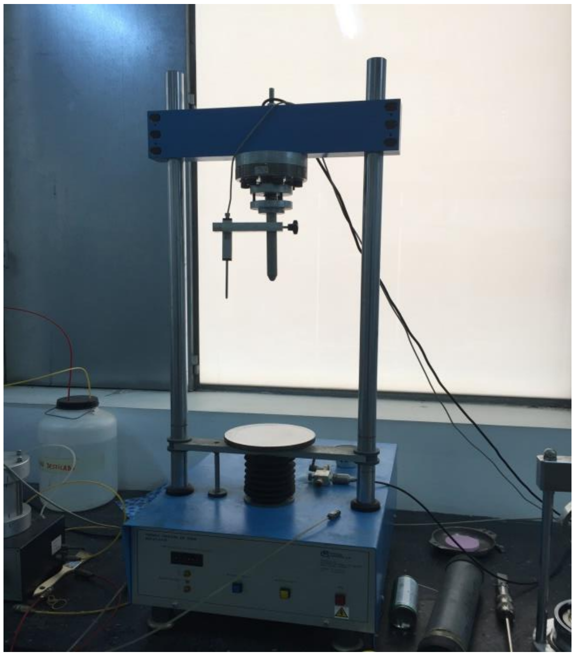


Figure 17 Triaxial Test equipment

Test analysis:

Since the CU triaxial test doesn't provide the necessary parameters directly, an analysis of the tests results was needed to extract the main parameters. Determining the advance sensitive parameters will be explained later in this chapter according to the MCA results and the chosen model. For consolidated clay, the following steps have been followed to determine the main shear strength parameters:

First, the following parameters were known

$\sigma3$: Total major principal stress at failure.
$(\Delta\sigma_d)_f$: Deviator stress.
$(\Delta u)_f$: Pore pressure.

Total and effective stress

For normally consolidated clay ($\sigma' = \sigma - \Delta u$)
For over consolidated clay ($\sigma' = \sigma - (-\Delta u)$)
To calculate the increase in pore pressure

Skempton's pore water pressure: $\bar{A} = \bar{A}f = (\Delta ud)f / (\Delta\sigma d)f$

Determination of Shear strength (c' (cohesion), ϕ(friction angle) and Ψ (dilatancy angel):

- Draw the total stress Mohr's circles based on the laboratory test report
- Draw a line that touches all the Mohr's circles. expressed by $\tau f = \sigma \tan \Phi$

In which:

σ: total stress
Φ: the angle that the total stress failure makes with the normal stress axis (Angle of shearing resistance)

Thus: $\phi = \sin^{-1}(\frac{\sigma 1-\sigma 3}{\sigma 1+\sigma 3})$

or:

$M= \frac{6 \sin \phi}{3- \sin \phi}$ where M is the slope of the Mohr-Coulomb line as stated in (PLAXIS, 2016)

Figure 18 also illustrates clearly another method of calculating the shear strength such the friction angle and soil cohesion parameters based in MC circles. With the help of Excel sheets, the following method used to analyze the results and obtain the main shear strength such as the friction angle, cohesion and the density of the specimens and further the other sufficient parameters have been extracted based on the chosen variant. However, Figure 19 sows the typical stress-strain curves of different clayey samples.

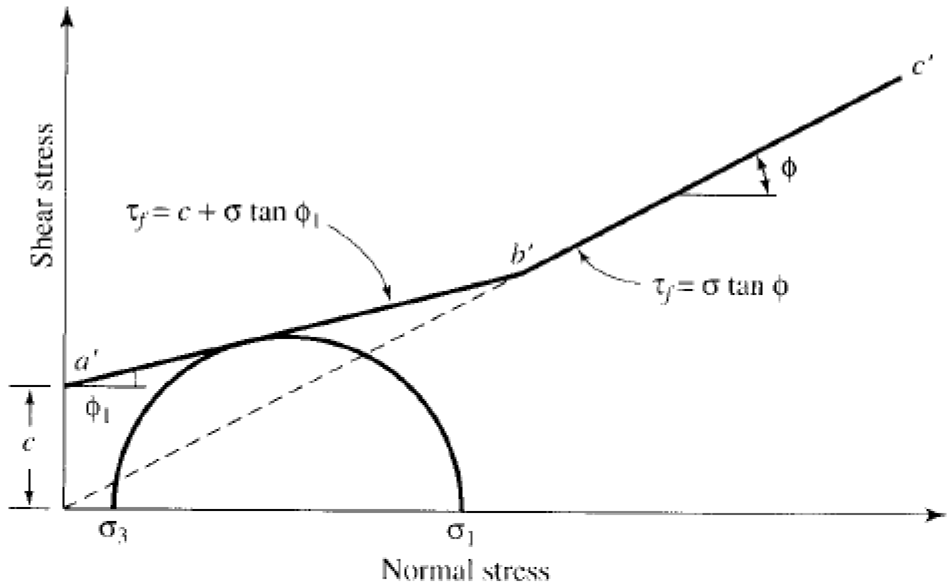


Figure 18 Typical Total stress failure from CU test in OC clay

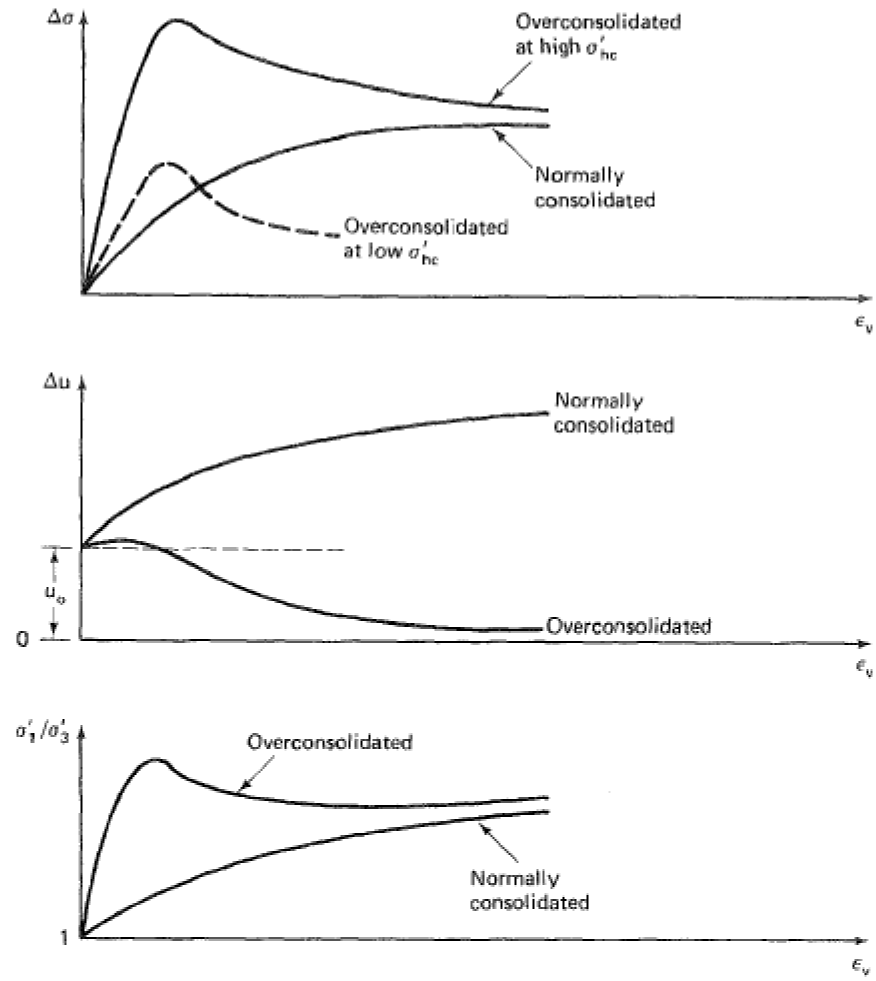
Centre of the circle = $(\sigma1 + \sigma3)/2$
Radius = $(\sigma1 - \sigma3)/2$
Soil Density (p) = $\frac{1000\,m}{A\,L}$ (mg/m3)

Where:

m: mass of the specimen

A: The initial cross-sectional area of the specimen

L: The initial length of the specimen



Note: For hydrostatic consolidation, $\sigma'_1/\sigma'_3 = 1$ at the start of the test; for non-hydrostatic consolidation, $\sigma'_1/\sigma'_3 > 1$.

Figure 19 Typical Stress-strain curves for CU triaxial test (Rodney & Eng, 2008)

3.2.3 Assessing and choosing of design variant (Answering sub questions 5, 6, and 7)

The possible modelling techniques have been explained earlier in Chapter 2.3, namely the Mohr-Coulomb (MC) model, Hardening Soil (HS) model, and Cam-Clay (CC) model. An overview of the principle, area of applications, and the parameters required for each modelling technique are described and elaborated earlier in Chapter 2.3. Those techniques have been analyzed based on criteria that fit the project objectives and client requirements whereby an advantage and disadvantages were weighted with respect to the determined criteria. Each criterion has been evaluated by setting a score varying from 1 to 5. The significance of the scoring can be seen in Figure 20.

Figure 20 significance of the evaluation scoring

Chapter 2.4.describes the weighting factor has been given for each criterion and its importance and relevance to the desired objectives.

	Activities	Products
1	Setting up alternative and variants	○ List of modelling techniques and its working principle ○ List of the criteria that are relevance to the chosen variance based on the functional and technical requirements ○ Defining the most suitable variant according to the analysis outcomes
2	Setting up a criteria	
3	Assessment of each alternative against the defined criteria	

Assessment method

Multi-Criteria Analysis is the method which was used for the assessment of the appropriate modelling technique that fit the requirements and the asserted criterions. The method allows evaluating different options when multiple criteria are present. The method is well-known and popularly used in the decision making in civil engineering projects and other different fields to assign the most preferred alternative. Table 5 clearly shows the assessment criterion, weight factors and the involved variants.

Table 5 Assessment method

		Modelling Variants		
Criteria	Weight factors	Mohr Coulomb	Hardening Soil	Cam-Clay
Precision	35%			
Applications	5%			
Cost	25%			
Time	15%			
Experience-based evaluation	20%			

An overview of the variant evaluation and the analysis results can be found in chapter 4.1.

3.2.4 Detail engineering of the chosen variant. (Answering questions 8 and 9

Once the Multi-Criteria Analysis is completed and the alternative is chosen according to the criteria, the details engineering starts with extracting the necessary parameters, modelling, calculations and drawing up the technical conclusion.

Modelling of soil behaviours

This part is the main part to verify if the Pressuremeter test allows estimating the soil parameters and thus to attain the main research objective. This was verified by PLAXIS using the following activities:

- Analyzing and extracting the necessary parameters from the triaxial test results

To start the PLAXIS modelling, input parameters were needed from both the CU triaxial and the PM tests conducted. Figure 21 illustrates the necessary input parameters needed for the HS model from CU triaxial test and the method of extracting those parameters. Note that some parameters were not available directly from the test and advance level of analyzing and calculation needed. Therefore, the parameter optimization function used to allow optimizing the estimated values towards real laboratory results. Some input parameters of the

Hardening Soil Model	Units	Explaination	Status
E_{50}^{ref}	(F/L^2)	Primary Loading Modulus under Reference Stress	lab, Multiple Correlations from Literature
E_{ur}^{ref}	(F/L^2)	Unload/Reload Modulus under Reference Stress	lab =4*E_{50}
E_{oed}^{ref}	(F/L^2)	1-D Compression Modulus	lab =E_{50}
ν_r	(-)	Unload/Reload Poisson's Ratio	Usually 0.2 to 0.4 for soil
p^{ref}	(F/L^2)	Reference Traxial Cell Confining Pressure	lab reference number
m	(-)	Stress dependency exponent	assumed
c	(F/L^2)	Cohesion	lab,Multiple Correlations from Literature
ϕ	(°)	Friction Angle	lab,Multiple Correlations from Literature
ψ	(°)	Dilatency Angle	typically ϕ - 30
K_0^{nc}	(-)	At rest lateral stress for NC	1 - sinϕ
$\sigma_{tension}$	(F/L^2)	Tensile Strength	0 or small value for stability
R_f	(-)	Failure Ratio	lab test (0.9 good estimate)
$c_{increment}$	$(F/L^2/L)$	Increment for increasing cohesion with depth	user defined
PCP or	(F/L^2)	Preconsolidation Pressure	lab test
OCR	(-)	Overconsolidation Ratio	lab test
ρ	(F/L^3)	unit weight	typical (90-120 pcf)

Figure 21 summary of HS model input parameters (Townsend, Anderson, & Rahelison, 2001)

optimization tool were estimated within the recommended scientific values.

Table below shows the estimated values of the parameters entered for the optimization step which were not available directly from the test as advised by the research supervisor. The optimization activity requires entering estimated values of the expected outputs before optimizing the real test parameters.

In which:

E_{ref50}	13000 KPa (estimated
$E_{ref\ oed}$	9561 KPa (estimated)
$E_{ref\ ur}$	112700 KPa (estimated)
M	0.5 (constant)
v_{ur}	0.2 (estimsted)
K_o	1

Table 6 The Input paarmeters for the Parameter Optimization tool

The above mentioned values were initially estimated based on the experience of similar projects before optimizing them and obtaining the real values. The input of the estimated values can be seen in the section below. The calculation and analysis results of the triaxial used as a reference for optimizing the soil parameters as clarified in the next section.

- Optimizing the parameters obtained from the triaxial test

This activity completed by using the Parameter Optimization function in SoilTest tool in PLAXIS. This function allows back-calculating the triaxial test and therefore finding the most desired values of the selected parameters of the measured data. It allows choosing the parameters that need to be optimized (See figure 15). By ending this activity, the best possible parameters obtained from the triaxial test were available.

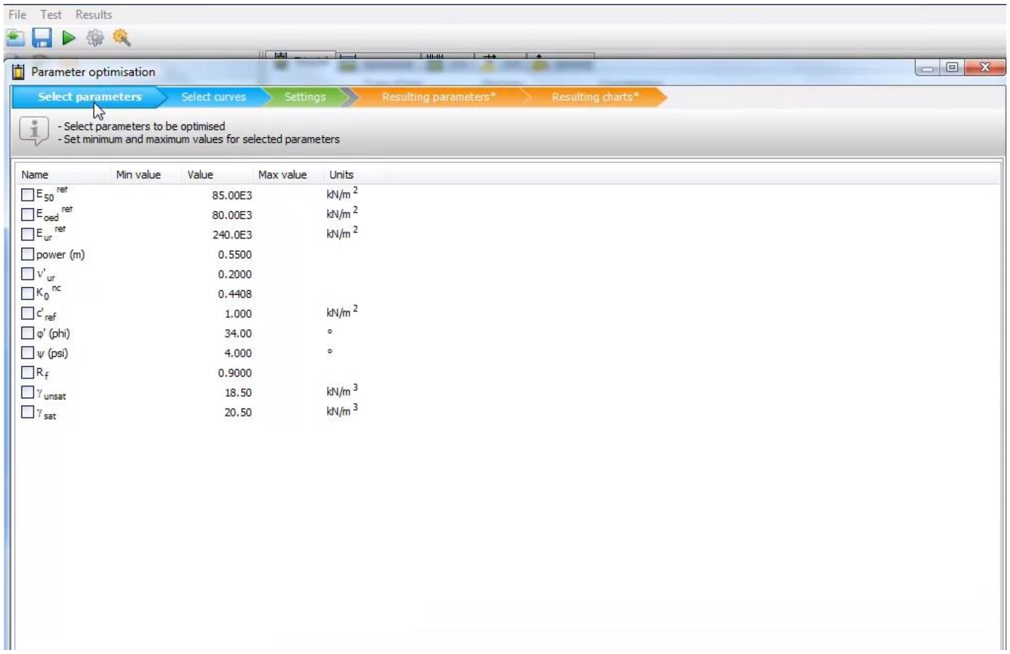


Figure 22 Parameter optimization function (PLAXIS, 2013)

- <u>Analyzing and extracting the necessary parameters from the PMT results</u>

For the PMT, some data such as applied pressure, corrected pressure and volume change are reported directly to the data acquisition system from the Pressuremeter cell. Other parameters such as the pressure limit (P_L) and Pressuremeter modulus (Ep) are calculated as follow:

- Calculation of the Pressure limit: (P_L)

Pl = ((r2 + ro2 - (2 r. ro)) / ro2

- Calculation of the Pressuremeter modulus (Ep):

Ep = (1+ϑ) . rm . (ΔP / Δr)

In which:
Ep = Pressure module or Menard module
θ = Poisson's coefficient
ΔP = Pressure increase considered in the elastic branch.
Δr = Variation of radius considered in the elastic branch, after the discharge-recharge cycle
rm = Average radius

- Modelling the PMT in PLAXIS

First, the geometry and PLAXIS boundary conditions are defined as follow:

- Hardening Soil (HS) model set as the material model based on the MCA results.
- The geometry: the geometry created corresponding to the soil profile of the study area. In both tests, the top boundary was drawn as y=0 while the bottom boundary was chosen to be 0.5m deeper than the cell pressure depth.
- Hydraulic conditions: none: this means there is no special hydraulic condition applied and the standard fixities were used for the calculations.
- Loading situation: a uniform horizontal line load was used to introduce the pressure of the real PMT in PLAXIS. This was sufficient to represents the PMT loading situation and it further lead up to the intended outputs.
- Due to the clayey soil profile, undrained condition chosen as recommended by (PLAXIS, 2016).
- Fine mesh selected as mesh coarse as a result of its known accuracy and quick simulation (Peaker, Cao, Jinyuan, Kanagaratnam, & Balachandran, 2016).
- The bottom of the model was vertical fixity and the vertical faces of the model were chosen to be horizontally fixed.
- Axisymmetric model was chosen for simulating the PMT as recommended by (Schanz, Vermeer, & Bonnier, 1999).

The following figures illustrate some of the inputs and boundary conditions used before modelling the tests.

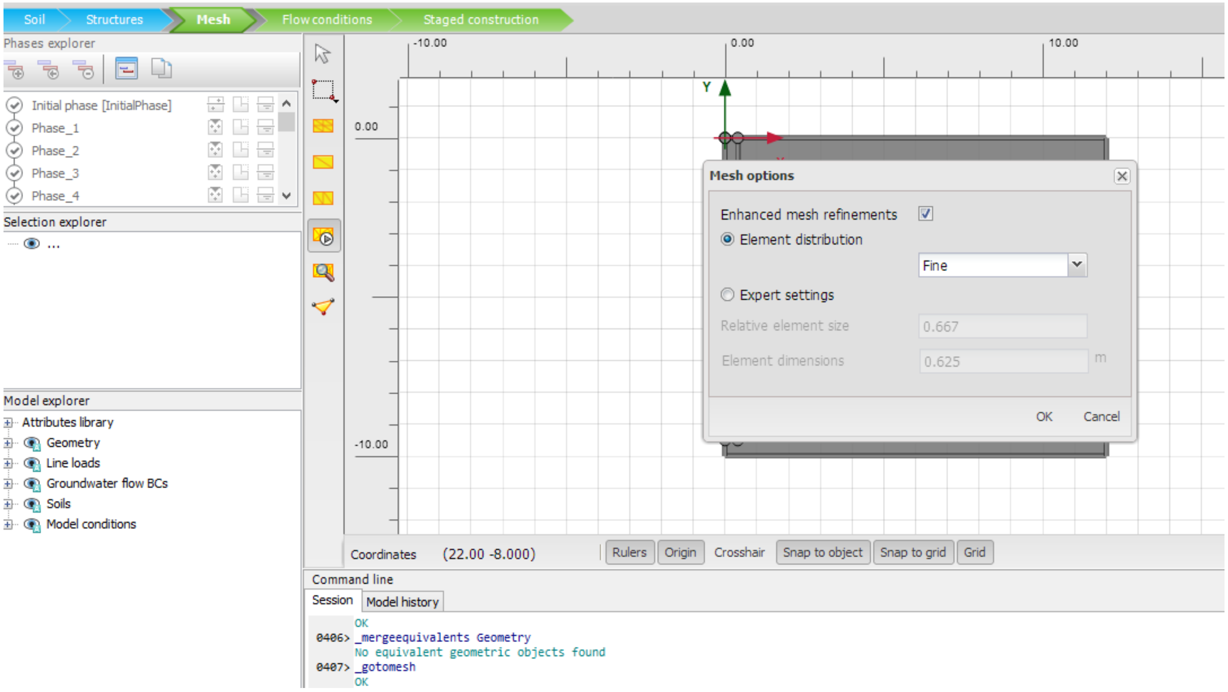


Figure 24 PLAXIS boundary conditions (Mesh type)

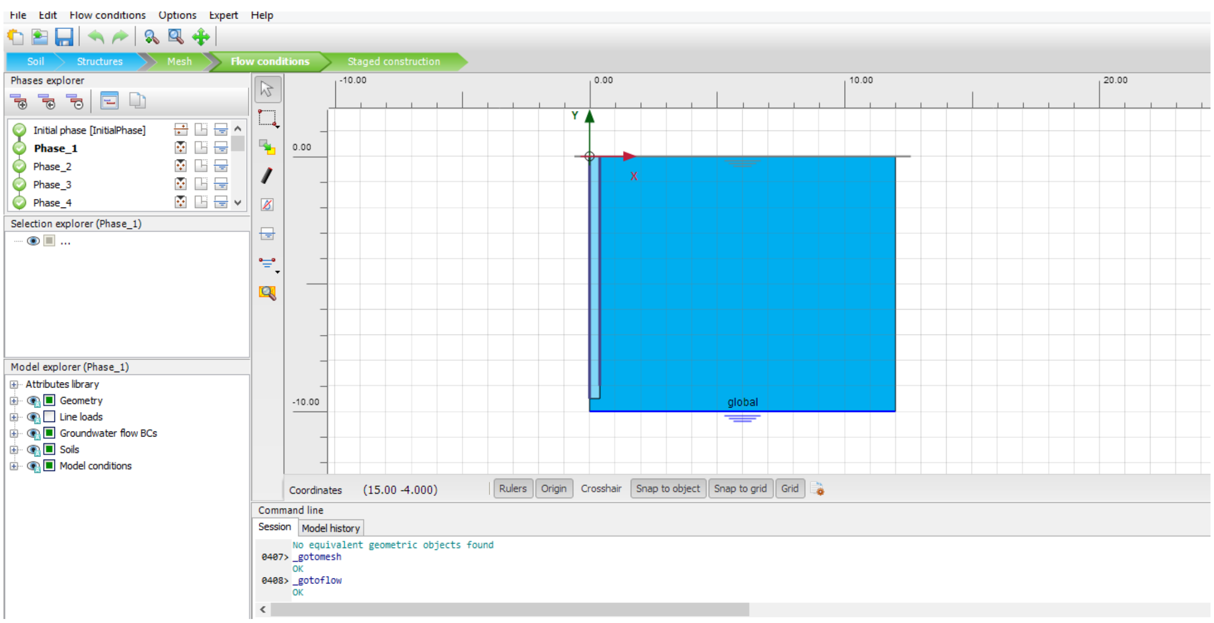


Figure 23 PLAXIS boundary conditions (Flow conditions)

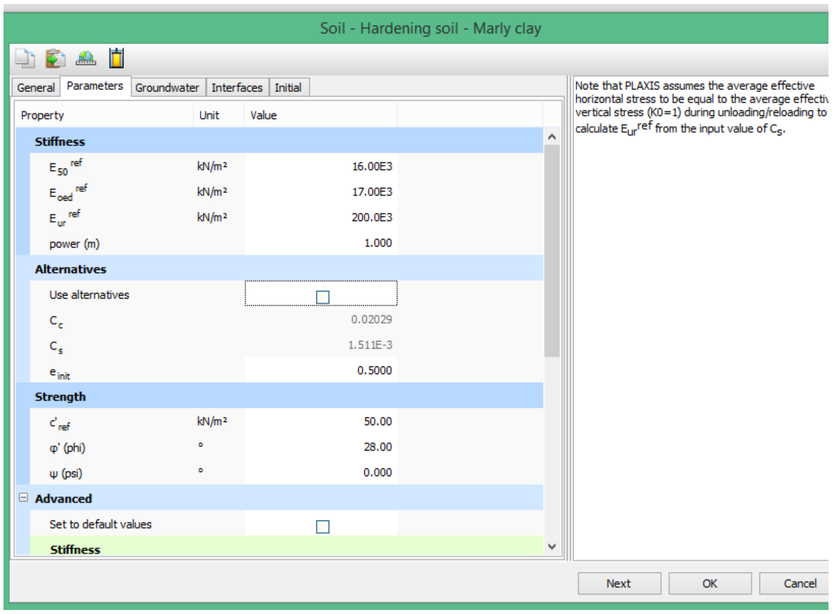


Figure 25 PLAXIS boundary conditions (soil parameters)

PMT simulation carried out for the two depths in the following two stages:

- Borehole drilling
- Applied pressure at the probe borehole interfaces progressively.

In this part, a simulation of the PMT made with the same loading situation of the real test. The experiment held with pre-bored Pressuremeter, the simulation done by applying the theory of the cylinder expansion in the soil mass. A progressive pressure applied identically to the one applied in the real test. The change in the horizontal displacement of the nodes connected to the central cell is measured for the purpose of acquiring the expansion volume of the borehole. In this research, the Pressuremeter was simulated as asymmetric model to allow carrying analysis using cylindrical coordinates such the radial direction (r) and the vertical direction (z) according to (Peaker et al., 2016) while the HS model chosen to model the material behaviour based on the MCA outcomes.

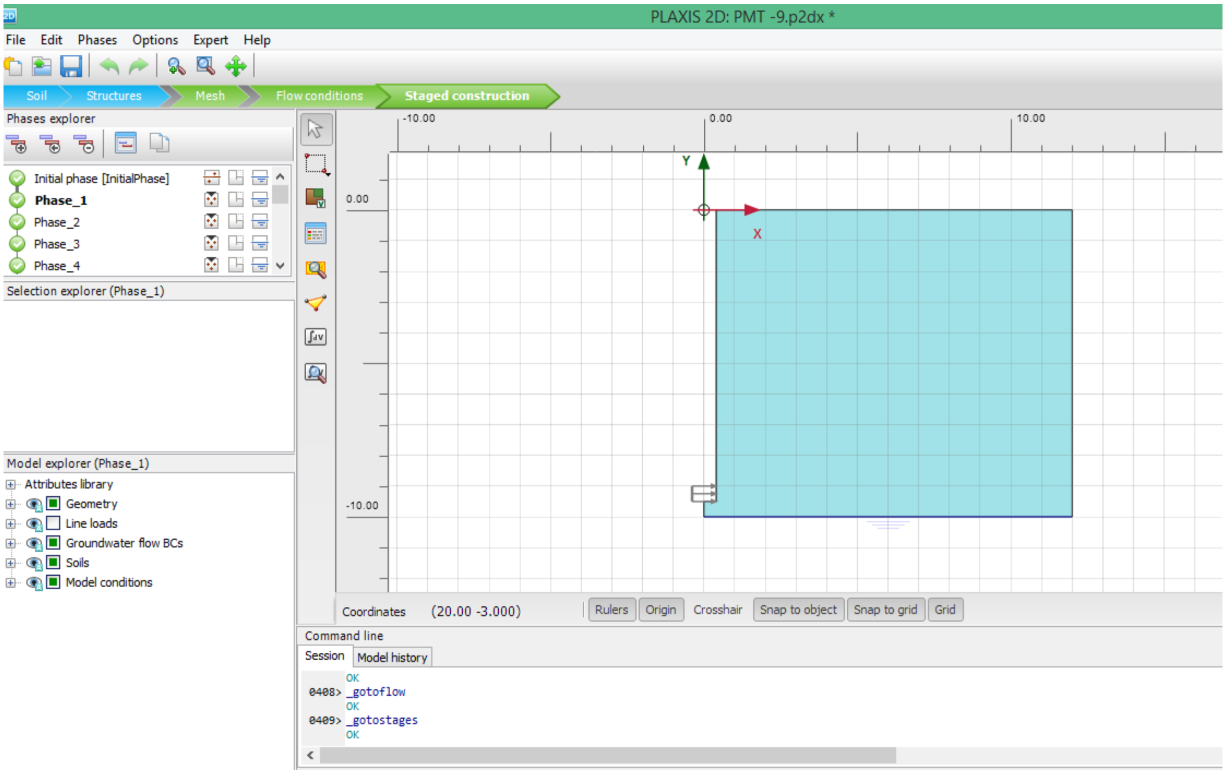


Figure 26 PLAXIS loading stage

Once a correlation was not found using the real parameters, the parameters have been changed followed by check if an acceptable correlation can be found between both tests.

- Comparison of Elastic modulus obtained from the triaxial test with Pressuremeter modulus (E_{50} and E_p).

Comparing the E50 and Ep is one of the main techniques to check the feasibility of estimating the soil parameters using the Pressuremeter outcomes especially once a persistent formula found between them. This has been done by plotting both values from different samples and establishing a formula which represents the relation between them.

3.2.5 Discussion and Conclusion (Answering question 10)

The results from the numerical analyses are plotted in Pressure-displacement (Ux) curves and further compared with the test curves in the field. Moreover, an analysis of the reliability of the values of Ep and E50 from the available samples is conducted. The discussion chapter was based upon the available knowledge and the involved experts' opinions.

3.3 Quality control

There are several actions and procedures have been followed to ensure that the research is upon the required standards and demanded competences. To start with, the communication between the researcher and the three involved parties is subjected to a communication plan as clarified in Chapter 3.1. A periodical follow up from the HZ supervisor was held and the consequent feedback has taken into consideration with respect to the thesis manual provided by HZ University. Furthermore, a guidance and advice from supervisors in UCA institution and Sergeyco Company to ensure that the product meets the professional standards since both organizations have sufficient experience in the field of geotechnical and civil engineering.

Moreover, a supporting document including activities, sources and planning with a logical structure were approved by HZ supervisor before starting the execution phase. In addition to the research proposal and research layout provided by the HZ Supervisor, a (Baarda, 2014) book followed for better research quality.

4 Results

4.1 Variant Analysis

Complying with the design variants and the set criteria described earlier in Chapter 2, and to provide clear arguments of the given score for each variant in order to clarify how the best variant was chosen, the characteristics of each variant will be deeply evaluated in this Chapter. The cost and time are judged based on information and activities needed for each modelling technique and the standard amount rate and time of those tests according to the manual used in Sergeyco Company. The final analysis results are provided at the end of this chapter. Please note that the cost estimation used in the evaluation was based on the standard prices of geotechnical services in Spain as provided in Appendix I: Standard Prices of Geotechnical Services in Spain

4.1.1 Variant Evaluation

Mohr Coulomb model:

Precision	This modelling technique is more recommended for the first analysis of the problem and first-order approximation of the soil behaviour. This due to the plastic-perfectly and elastic-perfectly assumptions which make it not accurate and therefore it is only recommended for simple projects (Plaxis, 2011). Moreover, the MC model usually overestimates or underestimates the soil strength which negatively affects the researches outcomes as stated by (Wang, 1993) and (Obrzud, 2010). Furthermore, (Teo & Wong, 2012) affirms that the main shortcoming of the MC model is the production of unrealistic soil behaviour.
Applications	The areas of application of the MC model are shallow foundations, slopes, the stability of dams and embankments (Surarak et al., 2012). Therefore, it is not applicable for the virtual load in this research which is a deep foundation. Hence, a score of 1 has been dedicated to this variant
Cost	Considering the input parameters and the tests needed to obtain them, the Mohr Coulomb model requires slightly less cost compared to the other variants as a consequence of the sort of the parameters needed. The parameters of MC model can be obtained from a triaxial test or direct shear box text. The direct shear box test costs 88 euro for each sample (See Appendix I: Standard Prices of Geotechnical Services in Spain). The amount stated is very appropriate for the specified criteria and therefore a score of 4 has been given to this criterion.

Time	Even though the modelling itself needs considerably less time in comparison with the other variants due to the perfectly elastic and plastic assumptions, the five input parameters have to be obtained from in-situ measurements and laboratory tests in which a time and effort needed which is sort of less than the other variants. The parameters of the MC model can be obtained by the direct shear box test in 2 days for each sample. 2 days are very suitable in term of the research requirements; this explains the score of 4 which was given in this criterion.
Experience-based evaluation	Apart from the fact that it requires less time, effort and cost. Mohr-Coulomb is not accurate especially in such sensitive research and therefore not advisable based on the level of the accuracy that clients headed for at the stated research objectives. And since the accuracy plays an important role towards the final research outcomes, a score of 1 has been given to this criterion.

Hardening Soil model:

Precision	The Hardening soil model allows estimating the complex soil behaviour. Besides, there are three input stiffnesses in this modelling technique to ensure an accurate modelling (Refer to Chapter 2.3.2). Moreover, there are non-linear assumptions of the elastic and plastic behaviours as well as the isotropic and deviatoric mechanisms which makes the Hardening Soil Model is the more precise in comparison with the other variants (Obrzud, 2010). The comparison between Figure 11 Figure 12 in Chapter 3 clearly illustrates the difference in the prediction of the soil behaviour between the MC model and HS model.
Applications	The Hardening Soil model as suitable type of any application including both hard and soft soil. Additionally, it is well-known in the prediction of different to geotechnical situations including the deep foundation as well as the situations whereby a reduction in the mean effective stress occurs (Ti, 2009).
Cost	Due to the large parameters required for this model, a combination of in-situ test and laboratory test is preferred for obtaining the necessary inputs. However, it is acceptable to acquire the data from the triaxial or oedometer test only. Therefore, this variant will have considerably higher cost in general compared to the other variants which require fewer input parameters. The triaxial test costs 332 euro for each 3.8cm sample according to the company manual (See Appendix I: Standard Prices of Geotechnical Services in Spain). Thus, a score of 2 has been given t this criterion.

Time	As explained in the above criterion, due to the large list of parameters needed for this model, it requires analyzing a combination of laboratory and in-situ tests to extract the necessary parameters. The standard time required to obtain the necessary parameters from the triaxial test of the HS model is 4 working days including consolidation, saturation and shear. But mostly it requires an optimization for the parameters before executing the modelling. This describes why a score of 3 was given to this criterion.
Experience-based evaluation	In line with (Schanz et al., 1999), the hardening soil model is the most accurate and popular comparing to the other variants. Moreover, (Oliva, n.d.) Highly recommends using the Hardening Soil since it is used in the characterization of material behaviour using Pressuremeter test. Likewise, the experts from UCA and Sergeyco who involved in this study were strongly advised to use the HS soil model before the intention of performing the MCA arise. This due to their knowledge of the suitability of the model to the available data and material.

Cam-Clay model:

Precision	Since the Cam-Clay allows modelling the non-linear behaviour through the hardening elasticity (Ti, 2009), it means that the precision level is acceptable and can meet the research accuracy objectives. Therefore, a score of 3 has been given in this criterion
Applications	Even though that this model is more realistic for near-normally consolidated clay, according to (Potts & Gens, 1988), The Cam-Clay model is not suitable to model silt, saturated clay and stiff clay, Therefore, this model will not be applicable for modelling the stiff clay soil as our case study. It is also more applicable in projects that situations were loading situations such embankment and foundations which means it is acceptable for the virtual load planned.
Cost	The overall cost of this model is relatively higher than MC model. This due to the type of parameters needed. The CC model requires a combination of oedometer and shear box test to obtain its parameters. The cost of the shear box test is 88 Euro and the oedometer test is 137 Euro. Which mean the total cost is 225 Euro for each sample which seems suitable to the project constraints. Therefore, a score of 3 has been dedicated to this criterion.

Time	Because of the selling index and Isotropic logarithmic compression index input parameters, a number of optimizations or reduplicate needed since those parameters are may not be available directly from the tests. This means that this model is time-consuming as well. More so, 10 working days needed to obtain the CC model parameters according to the standard manual provided by Sergeyco Company. Hence, a score of 1 has been given to this criterion since it is not favourable for the research objectives.
Experience-based evaluation	According to (Potts & Gens, 1988), the theory in which the Cam-Clay works upon is not suitable to model silt, saturated clay and stiff overconsolidated clays. That is the reason behind the low score of the model in this criterion.

4.1.2 Analysis of the variants

In this section, the final analysis will be shown. The selection of the criteria and its weighting factors have been already discussed earlier in Chapter 2.4. The assessment and the scoring values were based on the evaluation which carried out in the previous chapter. It can be clearly seen that the Hardening Soil (HS) model has the highest score with 3.7 out of 5.

		Modelling technique		
Criteria	Weight factor	Mohr-Coulomb	Hardening Soil	Cam-Clay
Precision	35%	2	5	3
Applications	5%	1	4	2
Cost	25%	4	2	3
Time	15%	4	3	1
Experience-based evaluation	20%	1	4	2
Weighted total	100%	2.55	3.7	2.45

Figure 27 Variant Analysis

4.1.3 Conclusion

As can be observed, the Hardening Soil (HS) model was the best variant with respect to the defined criteria. Therefore, The Hardening Soil model has been used in later stages for modelling the tested soil and interpreting the results towards achieving the final research goals. It is also worth to mention that due to the prominent disparity between the winning variant and the other options there was no need for sensitivity analysis especially in a case whereby uncertainties, project risks and margin of error are limited.

4.2 PMT:

The test executed in three different depths. The execution errors such the borehole drilling and the limitation of the Pressuremeter in penetrating the gravel and claystone causing undesired test results at the shallower depth and therefore are not provided and used in this study. Those errors will be widely discussed later in Chapter 5. Figure 28 and Figure 29 presents an overview of the PMT results conducted at 7-7.5 m and 9-9.5m depths. The detailed results can be found in the excel sheet in Appendix E: PMT results.

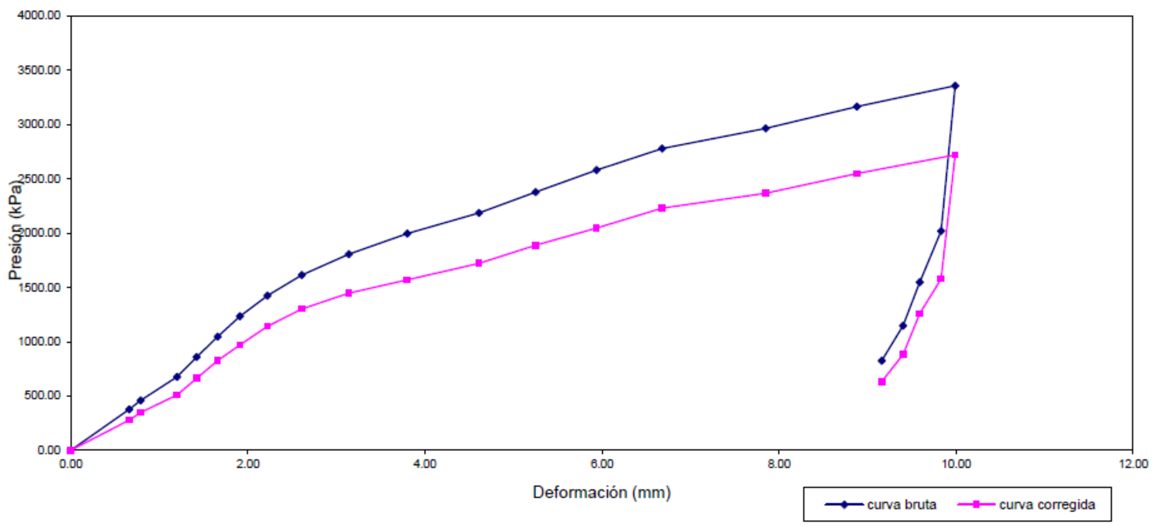


Figure 28 Actual and corrected curves of the pressure-volume relation at depth 7-7.5 m

The graph displays that the displacement or the volume change goes up while increasing the pressure at the borehole. The apparent drop at the end is due to decreasing the pressure again to the lowest point. The declined pressure and the reduction in the volume usually recorded for unloading behaviour of the soil which is not intended to be studied in this research. A 10 mm maximum displacement logically was at the maximum applied pressure which is 2719 kN.

As explained in the previous chapter, the test includes different loading stages, the readings have been taken for each loading stage at every 30 seconds according to the standards. The following table illustrates the readings of the pressure and the change in the deformation as a detail representation of Figure 28 as generated from the data acquisition system connected to the Pressuremeter probe.

Nº Lectura	Preal kPa	Pcorregida kPa	r 0" mm	r 30" mm	r 60" mm	Δr (r 60" - r 30") mm
1	0.00	0.00	0.00	0.00	0.00	0.00
2	377.30	278.32	0.45	0.60	0.66	0.06
3	459.62	346.92	0.65	0.76	0.79	0.03
4	677.18	510.58	1.00	1.15	1.20	0.05
5	857.50	661.50	1.20	1.38	1.42	0.04
6	1045.66	825.16	1.48	1.62	1.66	0.04
7	1233.82	969.22	1.73	1.86	1.91	0.05
8	1423.94	1139.74	2.00	2.16	2.22	0.06
9	1614.06	1300.46	2.34	2.53	2.61	0.08
10	1804.18	1446.48	2.91	3.03	3.14	0.11
11	1994.30	1568.00	3.50	3.66	3.80	0.14
12	2185.40	1719.90	4.00	4.43	4.61	0.18
13	2376.50	1886.50	4.82	5.00	5.25	0.25
14	2579.36	2045.26	5.29	5.57	5.94	0.37
15	2778.30	2229.50	6.00	6.25	6.68	0.43
16	2964.50	2366.70	6.82	7.35	7.85	0.50
17	3164.42	2547.02	8.00	8.27	8.88	0.61
18	3356.50	2719.50	9.00	9.25	9.99	0.74
19	2018.80	1577.80	9.87	9.85	9.83	-0.02
20	1548.40	1259.30	9.63	9.61	9.59	-0.02
21	1146.60	882.00	9.47	9.43	9.40	-0.03
22	823.20	632.10	9.32	9.17	9.16	-0.01

Table 7 Readings of the soil deformation at different loading stages at every 30 seconds at 7m depth

The above-shown table is directly produced from the data acquisition system. The corrected pressure readings are lower than the actual pressure because it considers the hydrostatic pressure of the water in the tubing (Oliva, n.d.). Accordingly, the corrected values are the once used in the calculations. From the table, the Pressure limit and Pressuremeter modulus were calculated as follow:

The pressure limit (Pl) at depth 7m and according to the formula ((r2 + ro2 - (2 r. ro)) / ro2, = **2720 KPa**

While Calculation of the Pressuremeter modulus: Ep = (1+ϑ) . rm . (ΔP / Δr)
Ep = **22470 KPa**

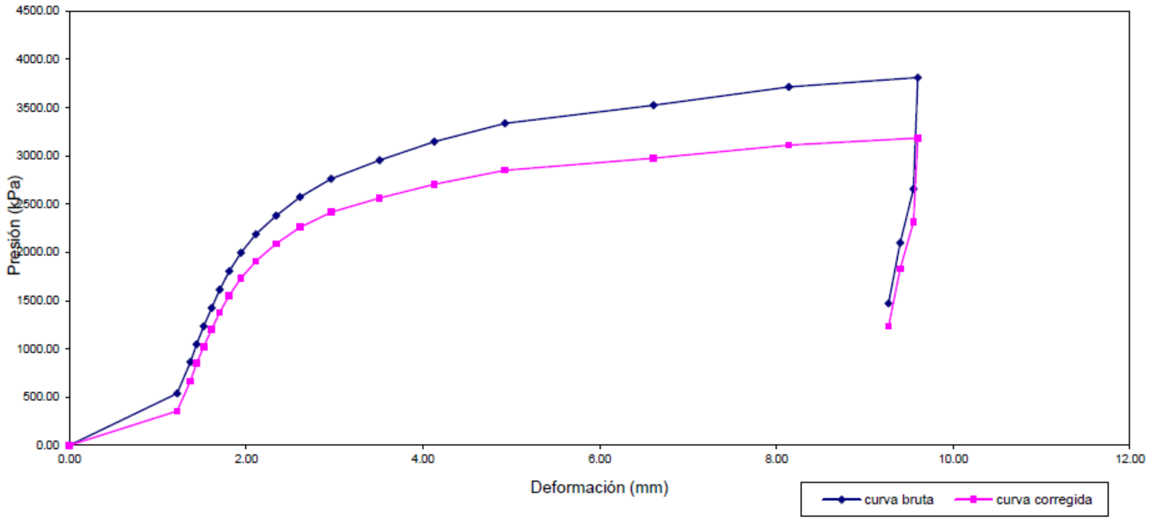


Figure 29 Actual and corrected curves of the pressure-volume relation at depth 9-9.5 m

At 9m depth, the maximum displacement was 9.6 mm at the maximum corrected pressure 3184 kN as can be shown in Table 8. Therefore, it can be noticed that the deeper layer is stiffer in which it has less displacement even with higher pressure applied.

Nº Lectura	Preal kPa	Pcorregida kPa	r 0" mm	r 30" mm	r 60" mm	Δr (r 60" - r 30") mm
1	0.00	0.00	0.00	0.00	0.00	0.00
2	537.04	355.74	1.00	1.20	1.22	0.02
3	862.40	666.40	1.24	1.35	1.37	0.02
4	1044.68	848.68	1.40	1.43	1.44	0.01
5	1232.84	1022.14	1.48	1.50	1.52	0.02
6	1422.96	1202.46	1.54	1.59	1.61	0.02
7	1613.08	1377.88	1.63	1.68	1.70	0.02
8	1804.18	1554.28	1.73	1.79	1.81	0.02
9	1996.26	1731.66	1.83	1.91	1.94	0.03
10	2187.36	1908.06	1.96	2.07	2.11	0.04
11	2379.44	2090.34	2.15	2.28	2.34	0.06
12	2571.52	2257.92	2.40	2.54	2.61	0.07
13	2762.62	2414.72	2.70	2.85	2.96	0.11
14	2954.70	2562.70	3.11	3.37	3.51	0.14
15	3145.80	2704.80	3.72	3.97	4.13	0.16
16	3335.92	2850.82	4.27	4.66	4.93	0.27
17	3524.08	2975.28	5.21	6.12	6.61	0.49
18	3714.20	3111.50	7.00	7.75	8.14	0.39
19	3811.22	3184.02	8.48	9.13	9.60	0.47
20	2657.76	2312.80	9.59	9.57	9.55	-0.02
21	2099.16	1829.66	9.50	9.42	9.40	-0.02
22	1470.00	1229.90	9.35	9.29	9.27	-0.02

Table 8 Readings of the soil deformation at different loading stages at every 30 seconds at 9m depth

From the above table, the Pressure limit and Pressuremeter modulus at 9m depth were caclculated as follow:

The pressure limit (Pl) at depth 9m and according to the formula ((r2 + ro2 - (2 r. ro)) / ro2, = **3200 KPa**

While Calculation of the Pressuremeter modulus: Ep = (1+ϑ) . rm . (ΔP / Δr)
Ep = **69235 KPa**

4.3 CU Triaxial Test:

The Consolidated Undrained triaxial test conducted for different specimens of the same depths where PMT performed under different confining pressure (650 KPa and 900 KPa). Next subchapters display a summary of the results from sample 7.3 -7.6m and 9 – 9.3m respectively. While the detail calculations and lab report including pressure applied, Mohr Circles, parameters analysis and the specimen response can be found in a separate Excel sheet in Appendix F: Triaxial Test lab report

4.3.1 Sample at 7m depth

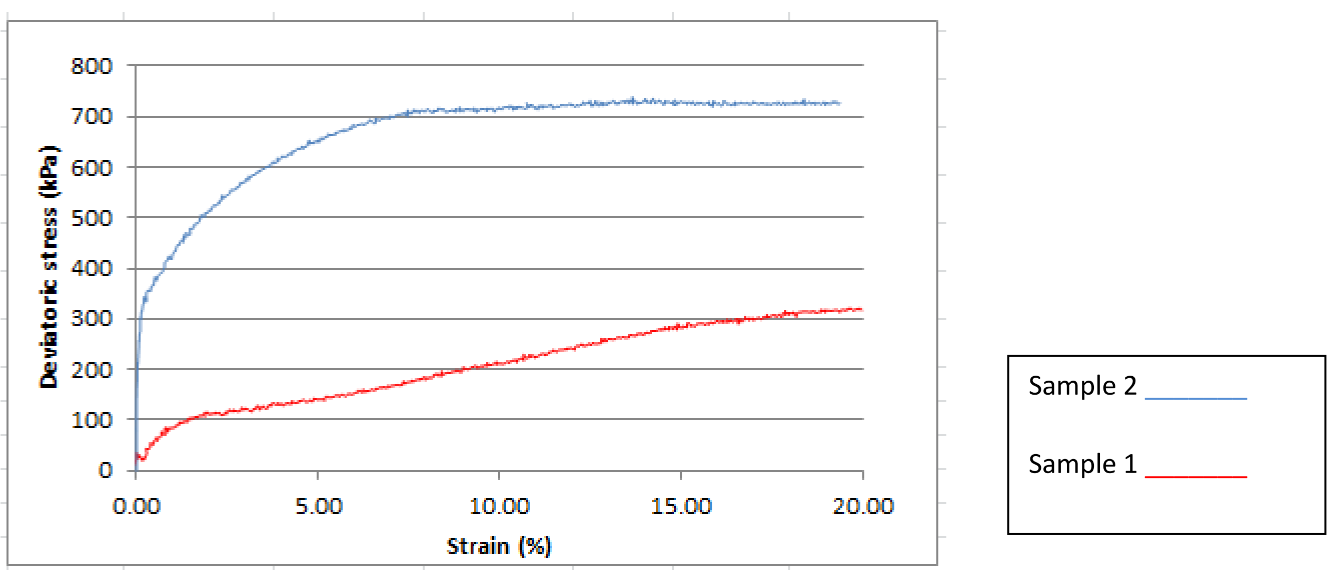


Figure 30 Stress-strain relation obtained from triaxial test at 7m depth

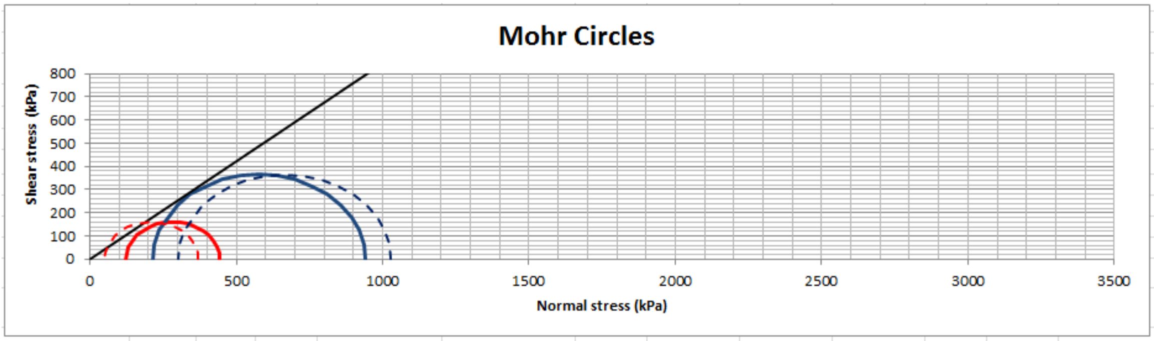


Figure 31 Mohr-Coulomb envelope for CU triaxial test at 7m depth

Based on the Mohr circles and MC failure envelope, the dry density, friction angle and the soil cohesion were calculated by excel sheet using the method described earlier in Chapter 3. The results for the two samples were 1.75, 176 (g/m3), 40 (degree) and 0 (KPa) respectively. It was quickly noted that the soil cohesion (c') =0 doesn't appear to be realistic value. But after using the parameter optimization function, a pragmatic value presented as will be shown in Chapter 4.4.1.

4.3.2 Sample at 9m depth

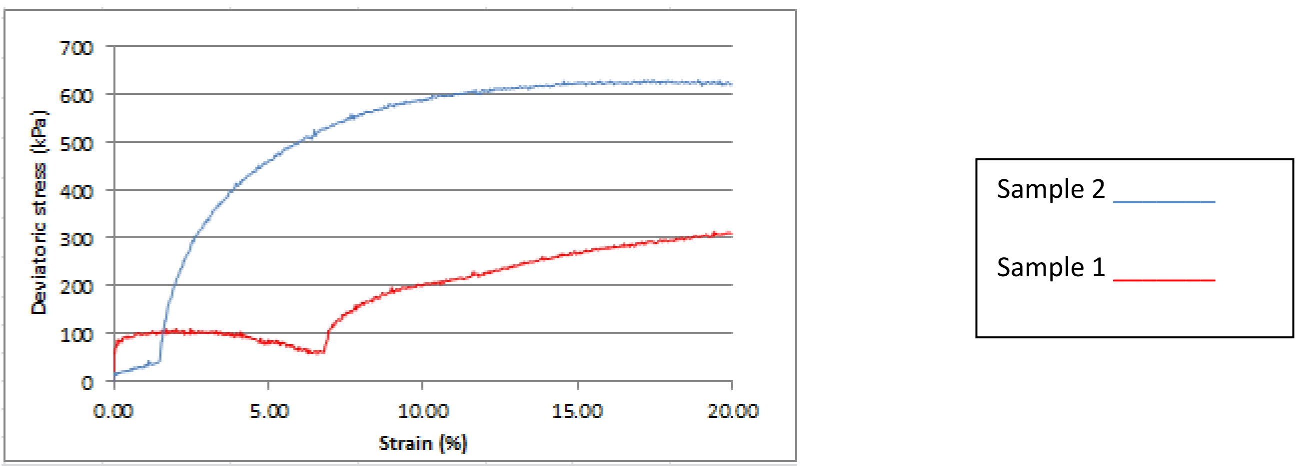


Figure 32 Stress-strain relation obtained from triaxial test at 9m depth

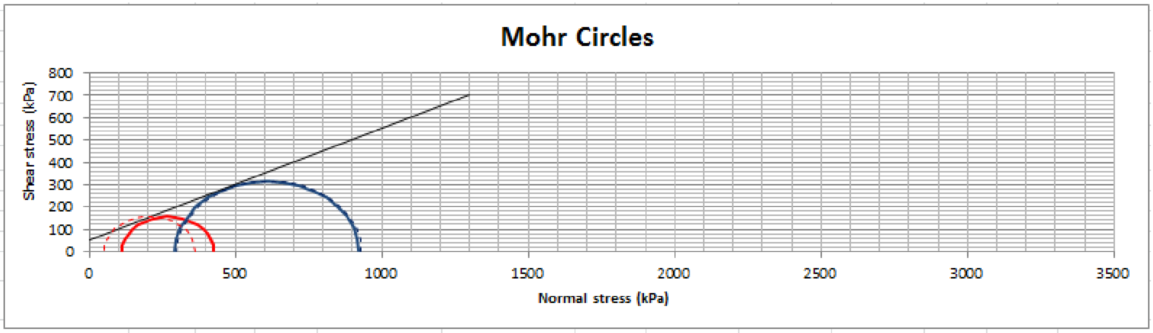


Figure 33 Mohr-Coulomb envelope for CU triaxial test at 9m depth

At 9m depth sample, the results for the three samples were 1.75, 1.67 (g/m3), 52 (KPa), 26.49 (degree) and 52 KPa respectively.

On the whole, both outcomes give the same value for the density while a remarkable variance in respect of cohesion and friction angle. It is sensible that deeper layer demonstrates a higher value of cohesion than the shallower layer, but a cohesion value of 0 KPa is not feasible in this type of soil. Therefore, the further parameter optimization step exhibits a rational value.

4.4 Triaxial Test Simulation

In this chapter, the results of simulating the triaxial test are presented. The simulation was done according to the method clarified earlier in Chapter 3.2.4.

4.4.1 Parameters optimization

The results the optimized parameters are presented in this chapter. The main goal of conducting this activity was to ensure obtaining the best possible parameters from the laboratory test to be used in the modelling of PMT. The detailed calculations can be found in Appendix G: Parameters Optimization The test simulation was carried out using the aforementioned winning variant in the MCA, the Hardening Soil model (HS).

Parameters optimization of the first sample (7m depth):

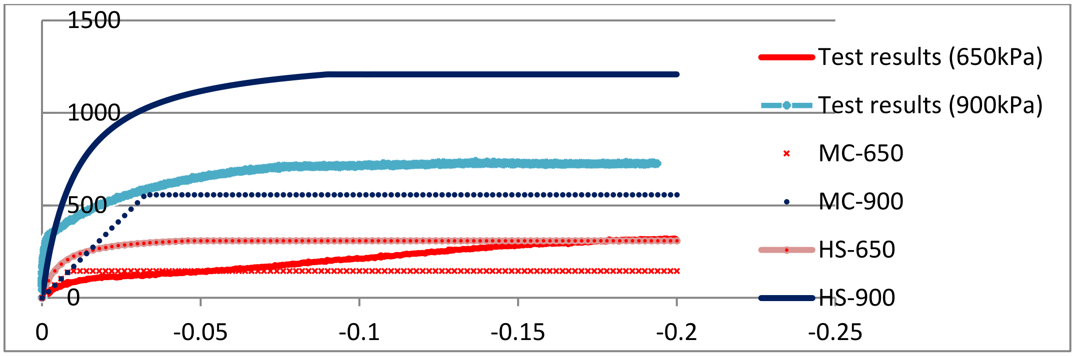


Figure 34 Stress-strain resulting charts of parameter optimization function of 7m depth samples

As can be observed, the MC model has been considered in this activity even though the HS were chosen to be used in all the modelling activities of this study based on the MCA outcomes. This took place because this PLAXIS function can consider both models and further produce their results without any noteworthy time or effort needed. Thus, the activity preceded using both modelling techniques to illustrate the eligibility of the chosen variant and its accuracy in comparison to the MC model as it can be clearly shown in Figure 34. The HS model shows an agreement with the test results especially at the lower pressure applied. Moreover, it represents a non-linear behaviour corresponding to the real soil behaviour as can be seen in the real test results. On contrary, even though the MC model shows some agreement in some specific events in the test, but it represents perfectly-linear behaviour which is not reasonable at the real tests.

The Parameter Optimization function produces tables of both 650 and 900 KPa chamber pressure in accordance with the real triaxial test pressures. The produced tables were further copied to an excel sheet in order to create resulting charts to be compared with the real test data. However, the optimized parameters are directly outputted from the aforementioned function as can be seen in the next table.

Table 9 Optimized parameters for 7m depth samples

Optimized parameters		
E50	4.50E+04	kN/m2
Eoed	4.70E+04	kN/m2
Eur	1.30E+05	kN/m2
v	2.20E-01	
c´	3.20E+01	kN/m2
phi´	4.00E+01	º

Parameters optimization of the second sample (9m depth):

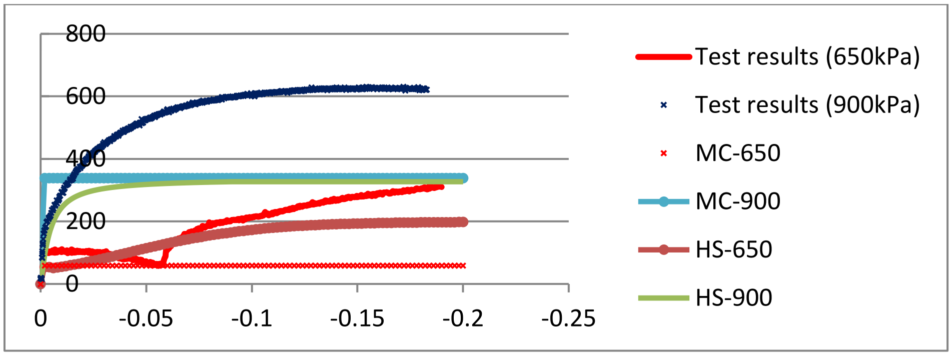


Figure 35 Stress-strain resulting charts of parameter optimization function of 7m depth samples

Just as the same steps performed for the sample at 7m depth, the triaxial test simulated using same pressure applied in the real test. The resulted stress tables were further compiled as charts representing stress-strain response of the samples using different chamber pressure and with different modelling techniques of the material behaviours as presented in Figure 35.

Table 10 Optimized parameters for 9m depth samples

Optimized parameters		
E50	1.60E+04	kN/m2
Eoed	1.70E+04	kN/m2
Eur	2.00E+05	kN/m2
v	2.20E-01	
c´	5.00E+01	kN/m2
phi´	2.80E+01	º

It can be noticed that the soil cohesion (c´) and the friction angle (phi´) at this depth are greater than the once at 7m depth. While the E50 and Eoed are lower.

4.5 Modelling of PMT

This activity was carried out in accordance with the method mentioned in Chapter 3.2.4. The modelling of PMT shows the soil displacement for each loading stage. These displacements were listed in order with the applied pressure comparable with the loading situation of the real test before it has been interpreted to a chart. The modelling results seem to over-predict the soil displacement when the exact parameters used. The acceptable correlation was not found with same parameters obtained and optimized from the triaxial test. Therefore, the modelling conducted using different cohesion until a reasonable correlation found as demonstrated in Figure 36. The detailed results of the modelling can be found in Appendix H: PMT simulation results. The vertical axis in the following chart represents the applied pressure in kPa while the horizontal axis states the soil displacement in mm.

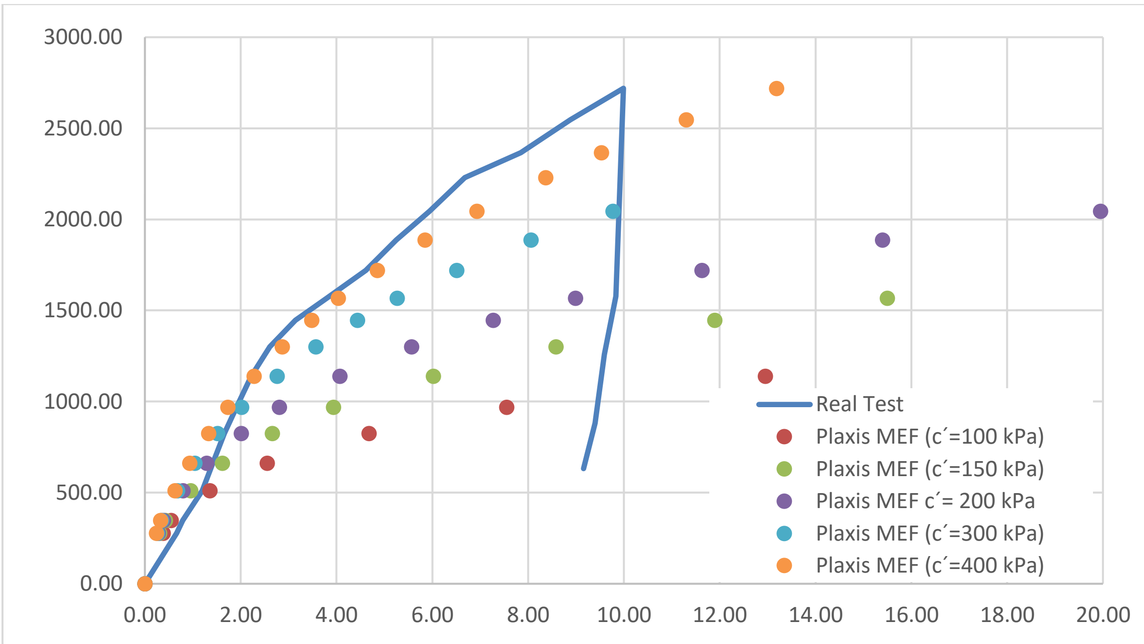


Figure 36 Comparison of the experimental and the numerical stress-displacement results

The actual test results are represented in the solid blue line, while the dotted lines illustrate the PLAXIS outcomes when simulating the real test but using different cohesion value as an input parameter to achieve an acceptable correlation. It can be clearly seen that the simulation gives an acceptable correlation at lower cell pressure, while it overestimates the displacement once the pressure noticeably increased. Additionally, it's apparent that the reasonable correlation found once the cohesion has been greatly increased to impracticable value (c'=400). These circumstances will be discussed later in Chapter 5.

4.6 Ep vs E50 correlation

The following figure shows the correlation and the formula between E50 and Ep derived from the available values of the two samples.

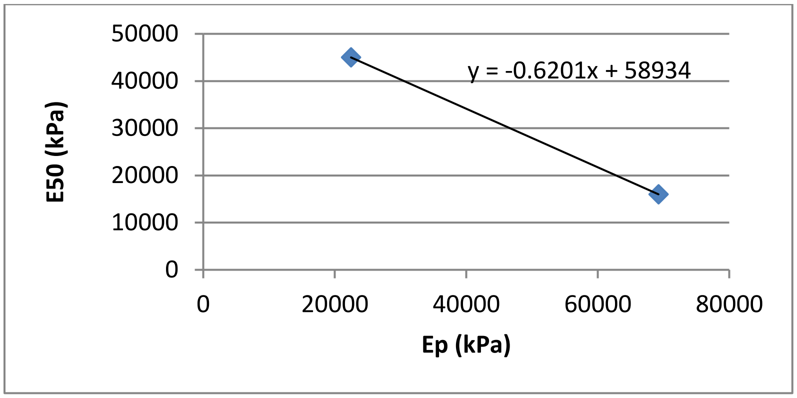


Figure 37 Ep versus E50 correlation

The correlation presents an inverse relation between the Secant modulus (E50) and the Pressuremeter modulus (Ep). This outcome was not expected and doesn't seem to be realistic as both parameters representing the shear strength of the same tested soil while showing an inverse relation. Moreover, the formula derived from the relation shows uncertain values mainly as a result of the fact that two samples are not enough to establish such formula. The further causes of these unexpected results will be discussed in the next chapter.

5 Discussion

In the current chapter, a discussion of the MCA, PMT, laboratory test and the modelling results will be discussed.

5.1 Multi-Criteria Analysis

The evaluation of the MCA was done based on a theory and facts of the three different models retrieved from desk research and literature review especially for the precision and the application of each variant. The cost and time criteria were judged based on the standards used in Spain as provided by the client (See Appendix I: Standard Prices of Geotechnical Services in Spain while the experience-based evaluation assessed by the experts who were involved in this study. As it has been concluded in the previous chapter, the HS model had the highest score among the other variants. The main characteristics of the HS model were the precision and the experience-based evaluation. Both criteria have a total weight of 55% per cent which outbalanced this variant amongst CC model and MC model. Other related points to consider are that the MC model is slightly higher than the HS model in terms of the cost and time but the inconsiderable difference and the lower weight of those criteria couldn't make it sufficient to be the most appropriate variant. On the other hand, CC model isn't influential in any of the specified criteria whereby ended up with the lowest score. It is noteworthy that these results were highly expected from the experts in Sergeyco even before the MCA starts due to the basis of theory such as the better performance of the HS model in most geotechnical applications while the MCA conducted to allocate the best variant based on more scientific and evidential method. More so, the sensitivity analysis was not favourable to be done in a case were the potentials of risks and uncertainties are limited.

5.2 Laboratory and in-situ work results

To start with, the PMT was conducted at three different depths as explained earlier in Chapter 3.2.2. There was a problem with the shallower test conducted due to the limitations of the Pressuremeter probe. The probe has a diameter of 76 mm and maximum probe radial expansion is only 10mm which requires an accurate drilling of the borehole. Therefore, the shallower PMT results show a high disturbance during the drilling which makes it unusual to be used in our case. Likewise, the reason of the irregularity in test results could be the requirement of the support from conventional drilling techniques as mentioned by (Cambridge Insitu Limited, 2018) At the remaining depths, the results illustrate corrected and real values of the pressure applied and it can be clearly seen that the corrected curves in both tests had a lower pressure than the real pressure applied. This due to the hydrostatic pressure of the water in the tubing and membrane resistance and the volume difference is referred to the water compression in the Pressuremeter circuits (Oliva, n.d.) Both results show nearly the same maximum volume change at the maximum pressure applied; this explains that the shear strength of the soil at the examined location is higher at deeper layers.

For the triaxial test results, the sample extracted at shallower depth was not considered in the research mainly because the PMT results at same depth was not sufficient and hence a comparison and a correlation and can't be found. Additionally, the sample was extremely disturbed as noticed during the specimen preparations as a consequence of the transportation procedure assumed. At the other two depths, the samples provide practical results and show very stiff parameters which made them sufficient to be used further in the study except for some values such the cohesion in the second sample (-7 m depth) which gave a zero value. This value doesn't look realistic especially in this type of soil under CU testing conditions. This may be happened because of the wrong indication of the Mohr Circle in which there were higher values should be drawn to increase the tangent. In the other hand, using the SoilTest Optimization tool results in more realistic values and were consistent with the primary values arise from the real test. This displays that the laboratory test went well and the optimized parameters can be used further in the study.

5.3 Modelling of PMT

The graph presented in chapter 4.5 shows that the input parameter obtained from the triaxial doesn't give agreement with the real test when it used in the numerical analysis of the PMT. the agreement was not found when using the actual input parameters. This may refer to the frequent existence of the rocky elements as well as the heterogeneousness kind of soil (Oliva, n.d.). The numerical curves are in sensible agreement only when the cohesion considerably increased while the other parameters remain unchanged. In fact, such input parameters used when a much stiffer soil is present than the tested samples. One of the main reasons behind the variance of the curves once the actual parameters used, is the process of the transporting, cutting and preparing the specimens. This produces a weakening of the soil and therefore it loses its authentic properties particularly the soil cohesion. Moreover, the the different at early stages refer to the soil disturbance caused by drilling the borehole. Accordingly, the PMT is more accurate in terms of estimating the soil parameters and results in more reliable stress-strain relation of the soil. But a doubt still in the relation between the Ep and E50 as will be clarified in next chapter.

5.4 E50 versus Ep correlation

The results provided in chapter 4.6 shows there is an inverse relationship between the 50% secant modulus (E50) and the Pressuremeter modulus (Ep) which is not logical and practical behaviour. Additionally, the sample at 9m depth shows a value of 16000 KPa for the E50 and 69236 KPa for Ep. In practice, this difference a relatively is high compared to the sample obtained at 7m depth and can’t be sufficient for obtaining a credible correlation. This becomes clear in the correlation formula inferred from the values stated in Figure 37. Moreover, these results contradict with the findings provided by (Sedran, Failmezger, & Dravininkas, 2013) as they stated that there should be a constant relation between the Ep and E50 even though if the Ep doesn’t directly represent the E50. The same writers indicate that the degradation of the elastic modulus caused by tension resulted from drilling the borehole as well as the soil disturbance may be the reasons behind the difference between the two values. Hence, two samples present no statistical significance and hence not adequate to derive an accurate correlation for the relation between the E50 and Ep, especially in the clayey soil.

6 Conclusion and recommendations

6.1 Conclusion

In order to solve the problem of inaccurate parameters results from the laboratory testing and hence in an erroneous indication of the soil behaviour, a new approach represented in PMT is examined and compared with the classical laboratory tests using FEM. The modelling techniques were defined earlier by the client based on the available resources

The analysis of three different variants was done using a Multi-Criteria Analysis (MCA). After a detailed evaluation in term of cost, time, precision, application and experts point of view, the Hardening Soil (HS) model chosen to for modelling the soil behaviour which had the highest score by 3.7 out of 5 comparing to 2.45 and 2.55 for Cam-Clay (CC) model and Mohr-Coulomb (MC) model respectively

After simulating the PMT using PLAXIS, plotted curves are created to compare the numerical outputs with real test results. The agreement wasn't found with the existing optimized triaxial test parameters. The simulation re-performed with increasing the cohesion until an acceptable agreement was founded. This leads to inferring that the stiff clayey soil loses its parameters after the sampling and therefore results in weak soil comparing to the one in-situ.

Moreover, from the first glance, the correlation between the elastic modulus (E50) and the Pressuremeter modulus (Ep) doesn't provide a logical behaviour and clearly shows that only two samples are statically insignificant to consider that the founded formula $y = -0.6201x + 58934$ is credible.

On the whole, the used method, boundary conditions and the project requirements were not sufficient to design the systematic approach which allows estimating the parameters of stiff clayey soil from PMT. On the other hand, this research can be considered as a key point and baseline towards attaining the defined goal once the research outcomes taken into account in addition to the recommendations provided in the next chapter.

6.2 Recommendations

The conducted experiments and the analysis to check the feasibility of estimating the stiff clay parameters from Pressuremeter test, together with the conclusion summed up in the previous, have all drove to several recommendations concerning the optimization of the further related studies. The research scope was bounded with some functional requirements as well as factors such the available resources and time. Thus, many uncertainties presented at the study conclusion which can be minimized and thus more precise outcome acquired once the following recommendation implemented.

First of all, to establish a significant correlation and adequate formula of the comparison between the Ep and E50, much more samples need to be studied and further modelled for which imposes more time and resources to obtain dependable relation.

Furthermore, the disturbance of the samples obtained from the in-situ tests must be kept at very limited levels. This can be achieved by high-quality execution and well-trained experts as well as choosing proper drilling method.

Lastly, since the modelling techniques as a design variants were selected by the client as a functional requirement due to the limitation of time and resources, further studies can take into account more accurate or suitable modelling techniques such Hardening Soil Small Strain (HSSS) model and relook to the method of executing the model as more stages could be included. Accordingly, (Townsend et al.,2001) recommends using unload-reload cycle for calculating the stiffnesses while it is stated that those values may not be available usually.

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Appendices

Appendix A: Pressuremeter Model and Specifications

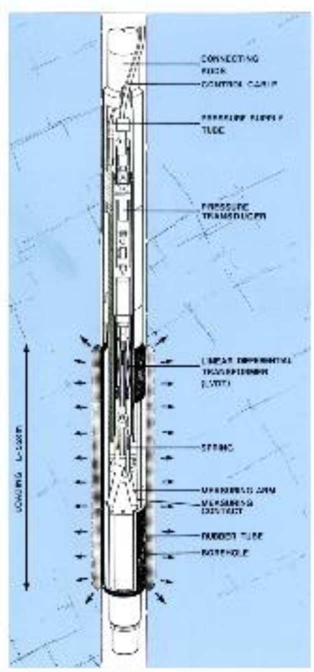
Model-4180

GEOLOGICAL, GEOPHYSICAL, GEOTECHNICAL SERVICES AND INSTRUMENTS

OYO

Pressuremeter

Elastmeter-2



<Abstract>

It is indispensable for designing the foundation for the construction of large scale structures such as buildings, dams or bridges to grasp the characteristics of deformation under a certain stress. The Elastmeter-2 is new version of lateral load tester capable of testing wide range ground from soft rock to hard rock. Highly accurate transducers and related electrical circuitry are built in the probe to improve the measurement accuracy, reliability and operability. With the two arms stretched inside the rubber packer, the system converts the displacement with the inner radius of the rubber packer into that representing the deformation with the borehole diameter.

<Features>

- Both pressure and displacement are measured directly in the probe using electrical transducers.
- Mechanical arm is used for the measurement of displacement making maintenance easier.
- Applied pressure is measured by precise semi-conductor transducer in the probe.

RP-1959

< Specifications >

Elastmeter HQ Sonde (Model-4180)

Deformation detection method	: Caliper arm method (to measure the inside diameter of rubber packer)
Max. Pressurization	: 20MPa
Outside diameter	: 62mm
Measurement distance length	: 520mm

Digital indicator for Elastmeter2 (Model-4018)

Display	: Pressure, displacement to be displayed in digital values (3 1/2 digits)
Displacement display (△Rn)	: ±19.99mm
Pressure display (P)	: ±19.99MPa
Power supply	: External power supply DC12V(±10%) Built-in battery 12V 4.5 A/h Voltage to charge the Built-in battery AC100V～220V
Operating temperature range	: 0 ～50℃
Outer dimensions	: 220(W) x 80(H) x 306(D) mm
Weight	: 5.5kg

High pressure hand pump (Model-4185)

Pressure	: Max. 20MPa
Unit exhaust quantity	: 5.0cc
Tank capacity	: 18 liters
Pressure gage	: 20 MPa
Outer dimensions	: 290(w) x 610(L) x 375(H) mm
Weight (Dry weight)	: 8.0kg

High-pressure tubing (Model-4153)

Outer sheath	: Neoprene rubber
Inner pipe	: Nylon tube (braid reinforced)
Working pressure	: 20 MPa max
Outside diameter	: 8mm
Length	: 100m (standard)

Control cable (Model 04181-2001)

Outer sheath	: Polyurethane (black colored)
Outside diameter	: 5mm (4-cored)
Length	: 100m (standard)
Tensile strength	: 15 N

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Please note specifications are subject to change without notice for the improvement.

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■ Your representative

Appendix B: Drilling rig Model and Specifications

www.boartlongyear.com/db520-525

- Wireline Coring
- DTH drilling
- Flush Rotary
- Make and Break hydraulic rod clamps

DeltaBase 520/525

Multipurpose drill rigs

The DeltaBase 520 and 525 represent a family of compact and lightweight drill rigs capable of multiple drilling methods. The DB520's efficient drilling system with speeds up to 740 rpm and pullback of 38 kN (8550 lbf) provides a versatile rig in a very small package. The DB525 includes all of the features of the DB520 but with an upgraded mast allowing for deeper drilling depths. The DB525 also offers a larger drill head providing increased torque and speed. This family of rigs is designed to meet the wide range of challenges on today's jobsite.

Advantages

- Quick and simple switching between drill-methods. Wireline coring, DTH, flush rotary and auger drill posibilities integrated into one rig.
- Hydraulic mast raising and stabilization jacks plus an onboard water pump allow for fast mobilization of the rig on the drill hole
- Standard safety features including an interlocked safety cage, as well low speed, low torque rotation for safe rod management
- Swing out drilling control panel to provide clear view of the drill hole
- Optional automatic SPT and pressure meter connections for quick change to testing and sampling

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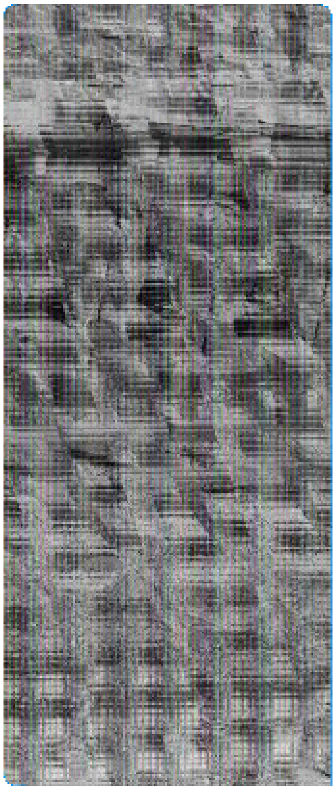

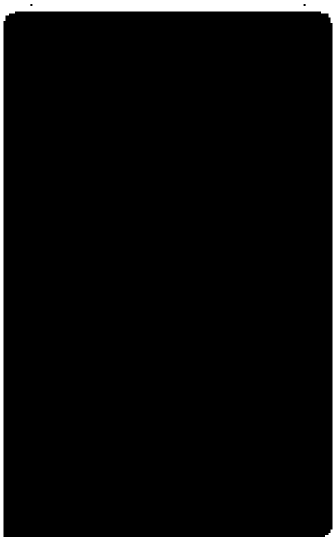

DeltaBase 520/525

Multiple Drilling Methods

- Wireline and Convential coring
- Down The Hole hammer drilling
- Flush rotary drilling
- Auger drilling

One Rig - Many Uses

- Site investigation
- Core sampling
- Undisturbed sampling with automatic SPT equipment
- Pressure meter testing
- Micropiling
- Tube a' Manchette grouting
- Jet grouting
- Water well drilling
- Minerals exploration

Fast Rig Mobilization

- Mounted on hydraulically powered steel tracks to cover difficult terrain and driven by electric or radio remote control the rig will get on site quickly
- Hydraulic mast raising and leveling jacks simplify rig set up
- On-board accessories including water pump, automatic SPT, and rod rack means that everything is on-site and ready to drill

Features to simplify drilling

- Hydraulic mast dump of 500 mm (1.6 ft) to get the mast close to the drill hole
- Standard hydraulic side shift for drill head to give clear access to the drill hole and make rod pulling easier
- Standard 220 mm (8.6") diameter double clamp for making and breaking rod joints quickly and safely
- Swing out drilling control panel to provide optimum view of the drill hole

Safety Built In

- Standard safety cage with electrical interlocks on every rig
- Low speed, low torque hydraulic rotation circuit for making up rod joints
- Hydraulic break out clamps for safely making and breaking rod joints
- Meets all CE safety standards for hydraulic drilling rigs

Options for any Job

- DTH air oiler, shock absorber and air connections
- High altitude engine package
- Skid or towable trailer mounting
- Night lights

Additional DeltaBase 525 Advantages

- Higher torque head options (up to 6000 Nm - 4440 lb-ft)
- Deeper core, rotary and DTH drilling with higher pullback (Pull back force up to 55 kN - 12364 lbf)
- Longer mast dump (1000 mm - 3.2 ft)

Refer to Technical Data sheets for additional technical information

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Appendix C: PMT ASTM standards

Designation: D 4719 – 00

Standard Test Method for Prebored Pressuremeter Testing in Soils[1]

This standard is issued under the fixed designation D 4719; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope *

1.1 This test method covers pressuremeter testing of soils. A pressuremeter test is an in situ stress-strain test performed on the wall of a borehole using a cylindrical probe that is expanded radially. To obtain viable test results, disturbance to the borehole wall must be minimized.

1.2 This test method includes the procedure for drilling the borehole, inserting the probe, and conducting pressuremeter tests in both granular and cohesive soils, but does not include high pressure testing in rock. Knowledge of the type of soil in which each pressuremeter test is to be made is necessary for assessment of (*1*) the method of boring or probe placement, or both, (*2*) the interpretation of the test data, and (*3*) the reasonableness of the test results.

1.3 This test method does not cover the self-boring pressuremeter, for which the hole is drilled by a mechanical or jetting tool inside the hollow core of the probe. This test method is limited to the pressuremeter which is inserted into predrilled boreholes or, under certain circumstances, is inserted by driving.

1.4 Two alternate testing procedures are provided as follows:

1.4.1 *Procedure A*—The Equal Pressure Increment Method.

1.4.2 *Procedure B*—The Equal Volume Increment Method.

NOTE 1—A standard for the self-boring pressuremeter is scheduled to be developed separately. Pressuremeter testing in rock may be standardized as an adjunct to this test method.

NOTE 2—Strain-controlled tests also can be performed, whereby the probe volume is increased at a constant rate and corresponding pressures are measured. This method shall be applied only if special requirements must be met and is not covered by this test method. Strain-controlled tests may yield different results than the procedure described in this test method.

1.5 The values stated in SI units are to be regarded as the standard.

1.6 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.* See Note 6.

2. Referenced Documents

2.1 *ASTM Standards:*

D 1587 Practice for Thin-Walled Tube Sampling of Soils[2]

D 2113 Practice for Diamond Core Drilling for Site Investigation[2]

3. Terminology

3.1 *Definitions*—For definitions of terms in this test method, refer to Terminology D 653.

3.1.1 *limit pressure, P_l $[FL^{-2}]$, n*—the pressure at which the probe volume reaches twice the original soil cavity volume.

3.1.2 *pressuremeter modulus, E_p $[FL^{-2}]$, n*—the modulus calculated from the slope of the pseudo-elastic portion of the corrected pressure-volume curve experiencing little to no creep.

3.1.3 *unload-reload modulus, E_R $[FL^{-2}]$, n*—the modulus calculated from an unload-reload loop.

3.1.3.1 *Discussion*—The unload-reload modulus varies with stress, or strain level, or both, and thus, the modulus values should be reported with the pressure and volume at the start of the unloading, at the bottom of the loop and at the crossover point.

3.2 *Abbreviations:*

3.2.1 *PBP*—prebored pressuremeter test

4. Summary of Test Method

4.1 A pressuremeter cavity is prepared either by drilling a borehole, or by advancing some type of sampler. Under certain circumstances, the pressuremeter probe is driven into place, usually within a casing. The various tools and methods available to prepare the cavity produce different degrees of disturbance. The recommended methods to be used at a site depend on the soil and the conditions met. The proper choice of tools and methods is covered by this test method.

NOTE 3—It is recommended that several drilling techniques be available on the site to determine which method will provide the most suitable test hole.

4.2 The pressuremeter test basically consists of placing an inflatable cylindrical probe in a predrilled hole and expanding this probe while measuring the changes in volume and pressure in the probe. The probe is inflated under equal pressure increments (Procedure A) or equal volume increments (Procedure B) and the test is terminated when yielding in the soil

[1] This test method is under the jurisdiction of ASTM Committee D-18 on Soil and Rock and is the direct responsibility of Subcommittee D18.02 on Sampling and Related Field Testing for Soil Investigations.

Current edition approved Feb. 10, 2000. Published May 2000. Originally published as D 4719 – 87. Last previous edition D 4719 – 87(1994)$^{\epsilon 1}$.

[2] *Annual Book of ASTM Standards*, Vol 04.08.

***A Summary of Changes section appears at the end of this standard.**

becomes disproportionately large. A conventional limit pressure is estimated from the last few readings of the test and a pressuremeter modulus is calculated from pressure-volume changes read during the test. It is of basic importance that the probe be inserted in a borehole with a diameter close to that of the probe to ensure adequate volume change capability. If this requirement is not met, the test could terminate without reaching sufficient probe expansion in the soil to permit evaluation of the limit pressure. The instrument may be either of the type where the change in volume of the probe is directly measured by an incompressible liquid or the type where feelers are used to determine the change in diameter in the probe. The volume measuring system must be well protected and calibrated against any volume losses throughout the system while the feeler operated probe must be sensitive enough to measure relatively small displacements.

NOTE 4—This test method is based on the type of apparatus where volume changes are recorded during the test. For the system measuring probe diameters, alternate evaluation methods are given in the notes.

5. Significance and Use

5.1 This test method provides a stress-strain response of the soil in situ. A pressuremeter modulus and a limit pressure is obtained for use in geotechnical analysis and foundation design.

5.2 The results of this test method are dependent on the degree of disturbance during drilling of the borehole and insertion of the pressuremeter probe. Since disturbance cannot be completely eliminated, the interpretation of the test results should include consideration of conditions during drilling. This disturbance is particularly significant in very soft clays and very loose sands. Disturbance may not be eliminated completely but should be minimized for the prebored pressuremeter design rules to be applicable.

6. Apparatus

6.1 *Hydraulic or Electric Probe*—The apparatus shall consist of a probe to be lowered in the borehole and a measuring or readout device to be located on the ground adjacent to the boring. The probe may be either the hydraulic type or the electric type. The hydraulic probe may be of a single cell or triple cell design. In the latter case, the role of which is to provide effective end restraint and ensure radial expansion of the central cell (Fig. 1a[3]). The combined height of the measuring and guard cells, if any, shall be at least six diameters. The design of the probe shall be such that the drilling liquid may flow freely past the probe without disturbing the sides of the borehole during insertion or removal. For both systems, the nominal hole diameter shall not be more than 1.2 times the nominal probe diameter. Typical probe dimensions and corresponding borehole diameters are indicated in Table 1.

6.1.1 *Probe Walls*—The flexible walls of the probe may consist of a single rubber membrane (single cell design) or of an inner rubber membrane fitted with an outer flexible sheath or cover (triple cell design) which will take up the shape of the borehole as pressure is applied. In a coarse-grained material like gravel, a steel sheath made of thin overlapping metal strips is often used. The accuracy of the test will be impaired when the probe cannot take up the shape of the borehole accurately.

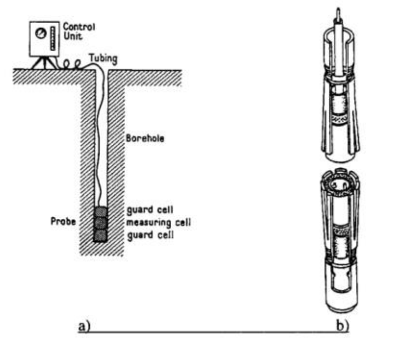


FIG. 1 a) Basic Principles of the Triple Cell Design Pressuremeter (Baguelin, Jézéquel and Shields, 1978,[3] b) Slotted Tube with Probe

TABLE 1 Typical Probe and Borehole Dimensions

Hole Diameter Designation	Probe Diameter, mm	Borehole Diameter	
		Nominal, mm	Max., mm
Ax	44	45	53
Bx	58	60	70
Nx	74	76	89

NOTE 5—Various membrane and sheath, or cover, materials may be used to better accommodate soil types; identify the membrane and sheath, or cover, used in the report.

6.1.2 *Measuring Devices*—Changes in volume of the measuring portion of the probe are measured in the hydraulic apparatus, and alternatively, the probe diameter can be measured by the use of feelers in the electric apparatus. Provisions to measure the diameter in directions at a 120° angle shall be provided with the electric apparatus. The measuring cell shall be prevented from expanding in the vertical direction by guard cells or other effective restraints in the hydraulic apparatus. The accuracy of the readout device shall be such that a change of 0.1 % in the probe diameter is measurable.

6.2 *Lines*—Lines connecting the probe with the readout device consist of plastic tubing in the hydraulic apparatus. To reduce measuring errors, a coaxial tubing is used, whereby the inner tubing is prevented from expanding by a gas pressure at its perimeter. By applying the correct gas pressure, expansion of the inner tubing is reduced to a minimum. Single tubing can also be used. In both cases, requirement for volume losses given in 7.3 should apply. Electric lines need special protection against groundwater.

6.3 *Readout Device*—The readout device includes a mechanism to apply pressure (Procedure A) or volume (Procedure B) in equal increments to the probe and readout of volume change

[3] Baguelin, F., Jézéquel, J.F., and Shields, D.H., "The Pressuremeter and Foundation Engineering," Trans Tech Publications, Series on Rock and Soil Mechanics, Vol 2, No. 4, 1978, p 617.

(Procedure A) or pressure change (Procedure B). The equipment using the hydraulic system and guard cells shall also include a regulator whereby the pressure in the gas circuit is kept below the fluid pressure in the measuring cell. The magnitude of pressure difference between gas and fluid must be adjustable to compensate for hydrostatic pressures developing in the probe. In the electrical system the volume readings are substituted by an electrical readout on the diameter of the probe.

6.4 *Slotted Tube*—A steel tube, (Fig. 1b) that has a series of longitudinal slots (usually six) cut through it to allow for lateral expansion, sometimes is used as a protective housing when the probe is driven, vibrodriven, or pushed into deposits that cannot be prevented from caving by drilling mud alone. The PBP test is performed within the slotted tube.

7. Calibration

7.1 The instrument shall be calibrated before each use to compensate for pressure losses (P_c) and volume losses (V_c).

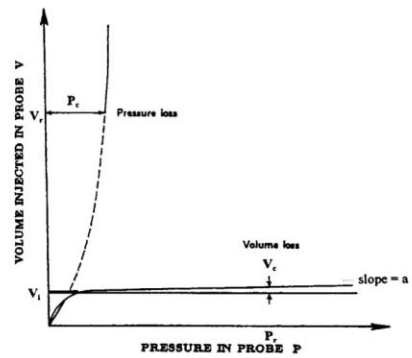
7.2 *Pressure Losses*—Pressure losses (P_c) occur due to the rigidity of the probe walls. The pressure readings obtained during the test on the readout device include the pressure required to expand the probe walls; this membrane resistance must be deducted to obtain the actual pressure applied to the soil. Calibrations for membrane resistance shall be performed by inflating the probe, completely exposed to the atmosphere, with the probe placed at the level of the pressure gage.

NOTE 6—**Warning:** The performance of the pressuremeter test, and particularly the calibration procedures, may present a safety hazard to the operator and persons assisting in the test. The blowout of the probe if on the ground or at shallow depth in the hole may cause injuries from flying debris. Wearing protective devices over the eyes and face or other measures such as putting the probe in a protective cylinder during calibration are recommended.

7.2.1 Apply pressures in 10-kPa increments for Procedure A and hold for 1 min. Make volume readings after 1-min elapsed time. When Procedure B is used, increase the volume of the probe in increments equal to 5% of the nominal volume of the measuring portion of the uninflated probe (V_0). Apply the volume increase in about 10 s and hold constant for 1 min. Continue steps in both procedures until the maximum probe volume is reached. Plot results using a pressure versus volume plot. The obtained curve is the pressure calibration curve. The pressure correction (P_c)is the pressure loss obtained from the calibration for the volume reading (V_r) (Fig. 2).

7.2.2 The pressure correction (P_c) must be deducted from the pressure readings obtained during the test. The maximum value of P_c should be less than 50 % of the limit pressure as defined in 10.6.

7.3 *Volume Losses*—Volume losses (V_c) occur due to expansion of tubing and compressibility of any part of the testing equipment, including the probe and the liquid. Calibration is made by pressurizing the equipment with the probe in heavy duty steel casing or pipe. A suggested procedure is to increase the pressure in steps of 100 kPa or 500 kPa depending if the probe is designed for a maximum expansion pressure of 2.5 MPa or 5.0 MPa, respectively. Each pressure increment should be reached within 20 s and once in contact with the steel tube, held constant for 1 minute. The resulting graph of injected volume (V_r) at the end of each pressure increment (P_r) is the volume calibration curve. The zero volume calibration is obtained by first fitting a straight line extension of the curve to zero pressure, as shown in Fig. 2. The resulting intercept V_i can be used to estimate the deflated volume of the probe measuring cell (V_o) as follows:



NOTE 1—The schematic graphs are not to scale; each calibration requires different volumes and pressures.

FIG. 2 Calibration for Volume and Pressure Losses

$$V_o = (\pi/4)\, L D_i^2 - V_i \qquad (1)$$

where:

D_i = inside diameter of the heavy duty steel casing or pipe, and

L = length of the measuring cell.

The volume loss (V_c) of the instrument for a particular pressure is obtained by using the factor a corresponding to the slope of the volume versus pressure calibration plot (Fig. 2) as follows:

$$V_c = V_r - aP_r \qquad (2)$$

This volume loss correction (V_c) must be deducted from the measured volumes during the test. This correction is relatively small in soils and can be neglected if the correction is less than 0.1 % of the nominal volume of the measuring portion of the uninflated probe (V_0) per 100 kPa (1 tsf) of pressure. In very hard soils or rock, the correction is significant and must be applied. In no case should this correction exceed 0.5 % of the nominal volume of the measuring portion of the deflated probe (V_0) per 100 kPa (1 tsf) of pressure.

7.4 Corrections for temperature changes and head losses due to circulating liquid are usually small and may be disregarded in routine tests for soils. For tests at depths greater than 50 m (150 ft), special procedures are required to account for head losses.

7.5 The amount of hydrostatic pressure (P_δ) exerted on the probe by the column of liquid in the testing equipment must be determined as follows:

$$P_\delta = H \times \delta_t \qquad (3)$$

where:

H = depth of probe below the control unit, m, and
δ_l = unit weight of test liquid in instrument, KN/m^3.

The test depth (H) is the distance from the center of the pressure gage to the center of the probe (Fig. 3). The obtained pressure is exerted on the probe but is not registered by the pressure gages. This pressure must accordingly be added to the pressure readings obtained on the readout device.

7.6 For triple cell pressuremeters, the pressure of the guard cells (P_G) must be set below the actual pressure generated in the probe to provide effective end restraint. This is obtained by subtracting this pressure from the test pressures as follows:

$$P_G = P_R + P_\delta - P_d \quad (4)$$

where:
P_G = guard cells pressure, kPa,
P_R = pressure reading on control unit, kPa,
P_δ = hydrostatic pressure between control unit and probe, kPa (see 7.5), and
P_d = pressure difference between guard cells and measuring cell, kPa (usually twice the limit pressure of the membrane).

7.6.1 A tabulation of gas and liquid pressures for a pressure difference of P_d = 100 kPa for various test depths is shown by Table 2.

8. Drilling

8.1 Whenever possible, place the pressuremeter probe by lowering it into a prebored hole. Two conditions are necessary to obtain a satisfactory test cavity: the diameter of the hole should meet the specified tolerances, and the equipment and method used to prepare the test cavity should cause the least possible disturbance to the soil and the wall of the hole. When testing soils, the pressuremeter tests must be performed immediately after the hole is formed.

8.2 The preparation of a satisfactory borehole is the most important step in obtaining an acceptable pressuremeter test. An indication of the quality of the test hole is given by the magnitude of scatter of the test points and by the shape of the pressuremeter curve obtained. Fig. 4 shows the typical shape of a pressuremeter curve obtained from a prebored test cavity. Fig. 5 shows a pressuremeter curve obtained when the borehole is too small or when the test is performed in a swelling soil. Fig. 6 shows a curve obtained when the borehole is too large.

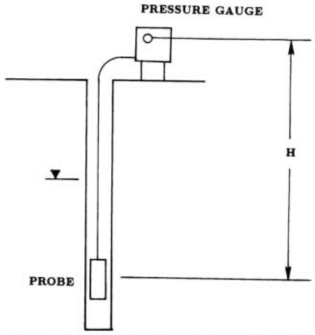


FIG. 3 Depth *H* for Determination of Hydrostatic Pressure in Probe

TABLE 2 Pressure Compensation for Guard Cells Based on Test Depth

Test Depth (H) m	Test Depth (H) ft	Liquid Pressure from Head of Test Liquid on Probe P, kPa	Gas Pressure Reduction on Readout Gages[A] P_d, 100 (kPa)
0	0	0	−100
5	17	50	−50
10	33	100	0
15	50	150	+ 50
20	67	200	+ 100

[A] To maintain guard cell pressure 100 kPa below the measuring cell pressure, deduct (−) or add (+), these pressures to the guard cell circuit.

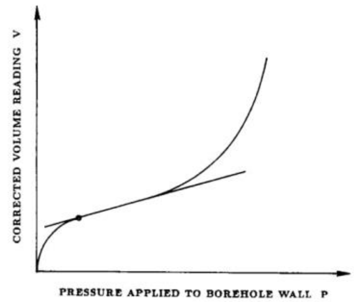


FIG. 4 Ideal Shape of the Pressuremeter Corrected Curve

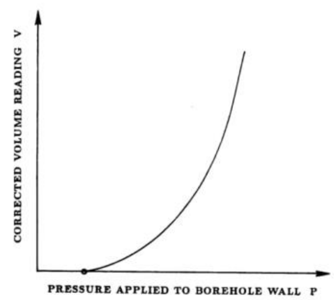


FIG. 5 Pressuremeter Corrected Curve When the Borehole is too Small

NOTE 7—The shape of the pressuremeter test curve is not sufficient to ensure that the test is reliable. The hole diameter requirements developed in 8.3.1 should also be met.

8.3 *Requirements of Test Cavity with Respect to Probe Diameter*:

8.3.1 *Hole Diameter*—Dimensions used in this test method are as follows:

8.3.1.1 *Diameter of the Pressuremeter Probe, D*—The typical diameter *D* of the pressuremeter probe varies from approximately 32 to 74 mm (1.25 to 3 in.).

8.3.1.2 *Diameter of Test Cavity, D_H*—The diameter of the

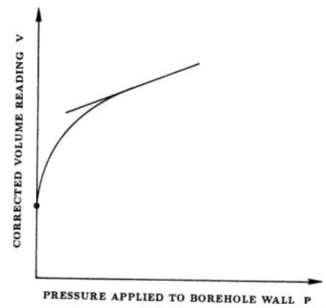


FIG. 6 Pressuremeter Corrected Curve When the Borehole is too Large

test cavity D_H should satisfy the following condition derived from experience:

$$1.03D < D_H < 1.2D \tag{5}$$

8.3.2 *Cutting Tool Diameter*:

8.3.2.1 When determining the diameter of the necessary cutting tool for a bored hole, three factors must be considered: (*a*) the required diameter of the cavity, (*b*) the overcutting of the cavity resulting from the wobble of the cutting tool or the wall erosion by the mud circulation in medium to large-grained soils, or both, and (*c*) the inward yielding that occurs between the removal of the cutting tool and the probe placement. Inward yielding can be reduced by the use of drilling mud.

8.3.2.2 When selecting equipment for the site, several bits of various sizes should be available so as to adjust the size of the bit depending on whether overcutting or inward yielding prevails.

8.3.2.3 When selecting the tool consider also that the wall of the test cavity should be as smooth as possible and the diameter D_H should be as constant as possible over the length of the hole.

NOTE 8—If D_H varies significantly over the length of the probe, because of ravelling for example, or if the borehole is noncylindrical, the quality of the test will be impaired.

8.4 *Methods and Tools Used to Prepare the Test Cavity*:

8.4.1 Any method and tool that can satisfy the general requirements of 8.1 through 8.3 may be used.

8.4.2 The following methods are used to prepare the test cavity for the pressuremeter probe:

8.4.2.1 *Rotary Drilling*—The drill bits used are usually drag bits in clays and roller bits in sands and gravels. Advance the rotating drill bit into the soil while satisfying the following conditions: low vertical pressure on the drilling tool (200 kPa (30 psi)), slow rotation (less than 60 rotations per minute) and a regulated low drilling fluid flow (to less than 15 L/min (4 gal/min)). Inject the drilling fluid by axial bottom discharge to cause the least damage to the borehole wall. The fluid must have a viscosity high enough to remove the cuttings at low pumping rates.

8.4.2.2 *Tube Sampling*—Thin wall samplers similar to those described in Practice D 1587 are used. The sampling tube must be long enough to ensure that the length of cavity to be tested is obtained with a single push. If the tube plugs or if full recovery is not obtained, then another method of preparing the test cavity should be considered. Withdraw the tube slowly to limit inward yielding of the cavity wall due to suction. If thick wall samplers are used, an inward bevel cutting edge must be provided to minimize pre-testing stressing of the borehole wall.

8.4.2.3 *Continuous Flight Augering*—Use a single 1.52-m (5-ft) length of auger at the bottom of a drill string to advance the borehole to the testing level. The cutting head must be slightly greater in diameter than the auger flight to prevent smearing the borehole wall. Rotate the auger during withdrawal. The same rotation and penetration pressure parameters as in 8.4.2.1 apply to continuous flight augering.

8.4.2.4 *Hand Augering*—Use an Iwan-Type auger with or without a hand pump for bottom discharge injection of mud.

NOTE 9—The use of hand auger is difficult below a depth of 6 m (20 ft), and should accordingly be considered only for testing at shallow depths.

8.4.2.5 *Driving or Vibrodriving a Sampler*—Drive a split barrel sampler into the soil. Driving or vibrodriving a flush sampling tube may also be used. The requirements of 8.4.2.2 apply.

8.4.2.6 *Core Drilling*—This method is described in Practice D 2113.

8.4.2.7 *Rotary Percussion*—Use a pneumatic or hydraulic drifter working with a bottom discharge bit. The removal of cuttings can be done by compressed air in dry formations, or by mud in wet soils.

8.4.2.8 *Pilot Hole Drilling and Subsequent Tube Sampling*—Drill a pilot hole smaller in diameter than the pressuremeter probe. Trim the hole to the proper diameter by a pushed or driven sampler. The requirements of 8.4.2.2 apply.

8.4.2.9 *Pilot Hole Drilling and Simultaneous Shaving*—Drill a pilot hole smaller in diameter than the pressuremeter probe. Immediately behind the drill bit, (Fig. 7) on the string of the drilling rods is a thin hollow cylinder that trims the cavity. Advance the drill bit and cylinder with high viscosity drilling fluid.

8.4.2.10 *Driving, Vibrodriving, or Pushing a Slotted Tube*—A slotted tube (see 6.4 and Fig. 1b) generally is used as a protective housing for the probe in formations that cannot be prevented from caving by drilling mud alone or when testing is

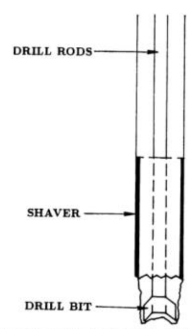


FIG. 7 Preparing the Test Cavity by the Pilot Hole Drilling and Simultaneous Shaving Technique

done in larger particle size soils. Place the probe in the slotted tube and drive, vibrodrive, or push the whole assembly into the soil to the testing depth. The test is performed within the slotted tube. This method is a full displacement method and should only be used when non-displacement methods cannot be employed. Calibrate the probe within the slotted tube prior to testing.

8.5 *Selecting Methods for Hole Preparation*:

8.5.1 Make the proper choice from the previously mentioned or other acceptable methods. This choice depends on the type of soil to be tested. The major influencing factors are:

8.5.1.1 Particle size distribution.

8.5.1.2 Plasticity.

8.5.1.3 Strength.

8.5.1.4 Degree of saturation.

8.5.2 Table 3 gives guidelines for selecting methods for borehole preparation in typical soils classified according to the factors mentioned in 8.5.1.1-8.5.1.4. Table 3 does not cover all possible methods of borehole preparation or probe placement, or both, and is included as a guide for selecting drilling methods.

9. Procedure

9.1 Perform the drilling of the borehole in accordance with Section 8.

9.2 Advance the hole to the test level and clean any debris or cuttings.

9.3 Before the probe is positioned in the hole for testing, make an accurate determination of the 0 volume reading (V_0). The volume V_0 is the volume of the measuring portion of the uninflated probe at atmospheric pressure. Accomplish this by deairing all circuits and adjusting all gages of the instrument to 0 while the probe is at atmospheric pressure. Close the volume circuit, preventing any further change in the volume of the measuring circuit. Lower the probe to test depth in this condition. Determine the test depth as the depth of the midpoint of the probe.

9.4 When using Procedure A, place the probe in test position and apply the pressure on the control unit in about equal increments, until the expansion of the probe during one load increment exceeds about ¼ of V_0 as defined in 9.3 (typically 200 cm^3 for a 800-cm^3 probe). Generally, 25, 50, 100, or 200-kPa pressures are selected for testing soils. Too small steps will result in an excessively long test, too large steps may yield results with inadequate accuracy. The pressure steps should be determined in such a way that about 7 to 10 load increments are obtained.

9.5 When using Procedure B, increase the volume of the probe in volume increments of 0.05 to 0.1 times the volume V_0 (as defined in 9.3) until the limit of the equipment is reached.

9.6 For both procedures, take readings after 30 s and 1 min after the pressure or volume increments have been applied. Volume readings are recorded to an accuracy of 0.2% of V_0 (as defined in 9.3) and pressure readings to an accuracy of 5% of the limit pressure.

9.7 Once the test has reached the maximum test step as determined in 9.4 and 9.5, terminate the test by deflating the probe to its original volume and removing the probe from the hole.

9.8 One or several load-unload cycles may also be performed in this test within the elastic expansion range (see Fig. 8). These cycles, if a probe with guard cells is used, requires the accurate control of gas pressure in the guard cells to obtain a representative reading on decreased volumes. The performance of unload-reload cycle(s) is encouraged but not required. Prebored pressuremeter design rules were established historically based on testing without unload-reload loops.

9.9 *Spacing and Testing Sequence*:

9.9.1 Minimum spacing between consecutive tests (center to center of probe) should not be less than 1½ times the length of the inflatable part of the probe. Common spacings vary from 1 to 3 m (3 to 10 ft).

9.9.2 In soft, loose, and sensitive soils, the hole should be predrilled ahead of the testing depth only far enough so that the cuttings settling at the bottom of the hole will not interfere with the test.

9.9.3 In stiff soils and weathered rocks where degradation

TABLE 3 Guidelines for Selection of Borehole Preparation Methods and Tools[A]

Soil	Type	Rotary Drilling With Bottom Discharge of Prepared Mud	Pushed Thin Wall Sampler	Pilot Hole Drilling and Subsequent Sampler Pushing	Pilot Hole Drilling and Simultaneous Shaving	Continuous Flight Auger	Hand Auger in the Dry	Hand Auger With Bottom Discharge of Prepared Mud	Driven or Vibro-driven Sampler	Core Barrel Drilling	Rotary Percussion	Driven Vibro-driven or Pushed Slotted Tube
Clayey soils	Soft	2[B]	2[B]	2	2	NR	NR	1	NR	NR	NR	NR
	Firm to stiff	1[B]	1	2	2	1[B]	1	1	NR	NR	NR	NR
	Stiff to hard	1	2	1	1	1[B]	NA	NA	NA	1[B]	2[B]	NR
Silty soils	Above GWL[C]	1[B]	2[B]	2	2[B]	1	1	2	2	NR	NR	NR
	Under GWL[C]	1[B]	NR	NR	2[B]	NR	NR	1	NR	NR	NR	NR
Sandy soils	Loose and above GWL[C]	1[B]	NR	NR	2	2	2	1	2	NA	NR	NR
	Loose and below GWL[C]	1[B]	NR	NR	2	NR	NR	1	NR	NA	NR	NR
	Medium to dense	1[B]	NR	NR	2	1	1	1	2	NR	2[B]	NR
Sandy gravel or gravely sands below GWL	Loose	2	NA	NA	NA	NA	NA	NA	NR	NA	2	2
	Dense	NR	NA	NA	NA	NR	NA	NA	NR	NA	2	1[D]
Weathered rock	...	1	NA	2[B]	NA	1	NA	NA	1	2	2	NR

[A] 1 is first choice; 2 is second choice; NR is not recommended; and NA is nonapplicable.
[B] Method applicable only under certain conditions (see text for details).
[C] GWL is ground water level.
[D] Pilot hole drilling required beforehand.

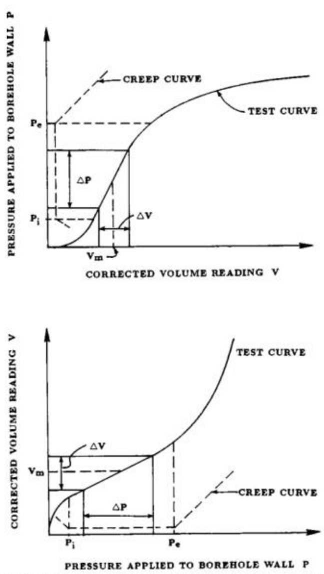


FIG. 8 Pressuremeter Test Curves for Procedure A

due to exposure is not significant, the hole can be predrilled to several test depths.

9.9.4 When the probe is driven into the soil, testing can take place continuously, while observing the minimum spacing requirements indicated in 9.9.1. No withdrawal is required between tests.

10. Calculations

10.1 The pressure transmitted to the soil by the probe from the pressure readings is calculated as follows:

$$P = P_R + P_\delta - P_c \tag{6}$$

where:

P = pressure exerted by the probe on the soil, kPa,
P_R = pressure reading on control unit, kPa,
P_δ = hydrostatic pressure between control unit and probe, kPa (see 7.5), and
P_c = pressure correction due to stiffness of instrument at corresponding volume, kPa, determined in accordance with 7.2.

10.2 Calculate the corrected volume reading of the probe from the volume readings as follows:

$$V = V_R - V_c \tag{7}$$

where:

V = corrected increase in volume of the measuring portion of the probe, cm^3,
V_R = volume reading on readout device, cm^3, and
V_c = volume correction determined in accordance with 7.3 and made at the test pressure readings corresponding to $P = P_R + P_\delta$, cm^3.

10.3 Plot the pressure-volume increase curve by entering the corrected volume and the corrected pressure on a coordinate system. Connect the points by a smooth curve. This curve is the corrected pressuremeter test curve and is used in the determination of the results (Fig. 8(a) and Fig. 8(b)). Other plots, such as pressure versus relative increase in radius, may also be used (Fig. 9).

NOTE 10—Historically, pressures were plotted on the horizontal axis and volume on the vertical axis. Considering the stress-strain nature of this test, it has become increasingly customary to reverse the coordinates. According to this test method, both presentations are acceptable.

10.4 For Procedure A, plot the volume increase readings (V_{60}) between the 30 s and 60 s reading on a separate graph. Generally, a part of the same graph is used, see Fig. 8. For Procedure B, plot the pressure decrease reading between the 30 s and 60 s reading on a separate graph. The test curve shows an almost straight line section within the range of either low volume increase readings (V_{60}) for Procedure A or low pressure decrease for Procedure B. In this range, a constant soil deformation modulus can be measured. Past the so-called creep pressure, plastic deformations become prevalent.

10.5 The pressuremeter modulus is determined as follows:

$$E_p = 2(1 + \gamma)(V_0 + V_m)\frac{\Delta P}{\Delta V} \tag{8}$$

where:

E_p = pressuremeter modulus, kPa, an arbitrary modulus of deformation as related to the pressure- meter based on data reduction included herein,
γ = Poisson ratio,

NOTE 11—For compatibility with tests performed with this instrument earlier, a value of 0.33 is recommended by this test method. Other values

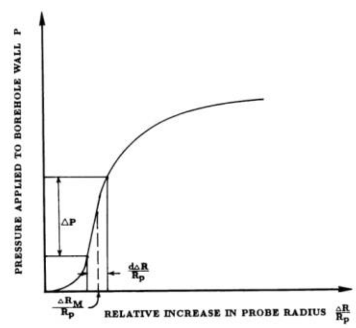


FIG. 9 Pressure Versus Relative Increase in Radius

may be used, but the value must be reported.

V_0 = volume of the measuring portion of the uninflated probe at 0 volume reading at ground surface, cm^3,
V = corrected volume reading of the measuring portion of the probe,
ΔP = corrected pressure increase in the center part of the straight line portion of the pressure-volume curve (see Fig. 8).
ΔV = corrected volume increase in the center part of the straight line portion of the pressure-volume curve, corresponding to ΔP pressure increase (see Fig. 8), and
V_m = corrected volume reading in the center portion of the ΔV volume increase.
$V_0 + V$ = current volume of inflated probe.

NOTE 12—If a break in the straight line portion of the pressuremeter curve is observed, calculations shall include a pressuremeter modulus for each straight line section of the pressuremeter test curve.

NOTE 13—A pressuremeter modulus can also be calculated from an unload-reload cycle. This modulus should be identified as the unload-reload pressuremeter modulus (Fig. 10).

NOTE 14—For tests where the probe diameter (radius) is measured, the pressuremeter modulus can be determined by converting the measurements into volume changes of the probe, in which case the formula given in this test method will apply (10.5). The pressuremeter modulus may also be calculated from diameter measurements directly as follows:

$$E_p = (1 + \gamma)(R_p + \Delta R_m)\Delta P/d\Delta R \quad (9)$$

where:
R_p = radius of probe in uninflated condition, mm,
ΔR_m = increase in radius of probe up to the point corresponding to the pressure where E_p is measured, mm,
$d\Delta R$ = increase of probe radius corresponding to ΔP pressure increase, mm,
ΔR = increase in probe radius, mm, and
$R_p + \Delta R$ = current radius of inflated probe, mm.

10.6 The conventional limit pressure is determined as follows: the limit pressure (P_l) is defined as the pressure where the probe volume reaches twice the original soil cavity volume, defined as the volume $V_0 + V_i$, (Fig. 8) where V_i is the corrected volume reading at the pressure where the probe made contact with the borehole. The volume reading at twice the original soil cavity volume is ($V_0 + 2V_i$). The limit pressure is usually not obtained by direct measurements during the test due to limitation in the probe expansion or excessively high pressure. If the test was conducted to read sufficient plastic deformation, the limit pressure can be determined by a $1/V$ to P plot, as shown by Fig. 11.

10.6.1 Points from the plastic range of the test generally fall in an approximate straight line. The extension of this line to twice the original probe volume will give the limit pressure (P_l) on the plot.

NOTE 15—The theoretical limit pressure is defined as the pressure where infinite expansion of the probe occurs. For practical purposes the definition outlined in 10.6 is recommended. Several methods are used to estimate the limit pressure from points measured during the test. These methods may also be used but should be properly reported.

NOTE 16—When the requirement of 8.3.1 about hole diameter tolerances is not met, only part of the test curve may be suitable for interpretation. The limit pressure is less sensitive to borehole size.

11. Report

11.1 For each pressuremeter test the following observations shall be recorded:

11.1.1 Date.

11.1.2 Boring number.

11.1.3 Type of test (Procedure A or B).

11.1.4 Type of probe (single or triple cells, measuring system for pressure, and volume or displacement, etc.).

11.1.5 Outside diameter of expandable section of probe.

11.1.6 Length of expandable probe section.

11.1.7 Description of membrane and sheath on probe.

11.1.8 Depth to center point of expanding portion of probe.

11.1.9 Time elapsed between end of borehole preparation and start of test.

11.1.10 Pressure or volume steps.

11.1.11 Volume readings at 30 and 60-s elapsed time for each load increment for Procedure A, pressure readings at 30 and 60-s elapsed time for each volume increment for Procedure B.

11.1.12 Notes on any deviation from standard test procedure.

11.1.13 Volume versus pressure graph, pressuremeter modulus, limit pressure.

11.1.14 Description of calibrations and calibration curves.

11.2 In addition, the following observations shall be recorded for the boring:

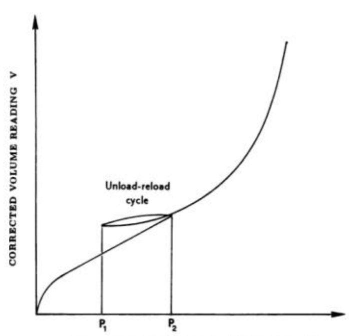


FIG. 10 Cyclic Pressuremeter Test Curve

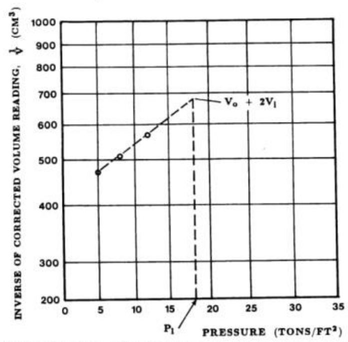


FIG. 11 Determination of Limit Pressure from Inverse of Volume Versus Pressure

11.2.1 Boring number.
11.2.2 Log of soil conditions.
11.2.3 Reference elevation.
11.2.4 Depth of water in the hole at the time of test.
11.2.5 Method of making the hole and method of preparing the cavity.
11.2.6 Type of testing equipment used.
11.2.7 Notes on driving resistance in the boring (SPT test *N* value).
11.2.8 Weather and temperature.
11.2.9 Name of drilling foreman.

12. Precision and Bias

12.1 The single most important factor in the successful completion of a preboring pressuremeter test is the preparation of a good hole. A good hole is very difficult to prepare in very soft clays and very loose sands. The pressuremeter limit pressure is less sensitive to the quality of the borehole; however, the pressuremeter modulus is much more sensitive to the quality of the borehole.

12.2 The subcommittee is seeking pertinent data from users of this test method to develop a precision statement.

13. Keywords

13.1 in situ test; modulus; limit pressure; stress-strain

SUMMARY OF CHANGES

In accordance with Committee D 18 policy, this section identifies the location of the changes to this standard since the last edition (D 4719–87(1994)$^{\epsilon 1}$) that may impact the use of this test method.

(*1*) Changed the title to prebored pressuremeter testing in soils to reflect the method of probe installation.
(*2*) Added Section 3 on Terminology and renumbered all subsequent sections.
(*3*) Added a sentence in 5.2 (formerly 4.2) to clarify that disturbance during installation is not to be completely eliminated but simply minimized for the design rules to be directly applicable.
(*4*) Added a schematic of the pressuremeter probe on Fig. 1.
(*5*) Modified Fig. 2 and associated 7.2.1 and 7.3 (formerly 6.2.1 and 6.3) to clarify calibration procedures and corrections.
(*6*) Modified 7.5 (formerly 6.5) and deleted former 9.1.1.
(*7*) Added a statement in 9.8 (formerly 8.8) encouraging the performance of unload-reload cycles.
(*8*) Renamed the reload modulus to unload-reload modulus.
(*9*) Renumbered Note 9 to Note 2.
(*10*) Corrected the expression for limit pressure in 10.6 (formerly 9.6).
(*11*) Fig. 11 was replaced to show the extrapolation to the limit pressure using an arithmetic scale rather than an inverse (l/V) scale.
(*12*) Modified and renumbered Section 11.

Appendix D: Triaxial Test ASTM standards

Designation: D4767 – 04

Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils[1]

This standard is issued under the fixed designation D4767; the number immediately following the designation indicates the year of original adoption or, in the case of revision, the year of last revision. A number in parentheses indicates the year of last reapproval. A superscript epsilon (ε) indicates an editorial change since the last revision or reapproval.

1. Scope*

1.1 This test method covers the determination of strength and stress-strain relationships of a cylindrical specimen of either an undisturbed or remolded saturated cohesive soil. Specimens are isotropically consolidated and sheared in compression without drainage at a constant rate of axial deformation (strain controlled).

1.2 This test method provides for the calculation of total and effective stresses, and axial compression by measurement of axial load, axial deformation, and pore-water pressure.

1.3 This test method provides data useful in determining strength and deformation properties of cohesive soils such as Mohr strength envelopes and Young's modulus. Generally, three specimens are tested at different effective consolidation stresses to define a strength envelope.

1.4 The determination of strength envelopes and the development of relationships to aid in interpreting and evaluating test results are beyond the scope of this test method and must be performed by a qualified, experienced professional.

1.5 All observed and calculated values shall conform to the guidelines for significant digits and rounding established in Practice D6026.

1.5.1 The method used to specify how data are collected, calculated, or recorded in this standard is not directly related to the accuracy to which the data can be applied in design or other uses, or both. How one applies the results obtained using this standard is beyond its scope.

1.6 The values stated in SI units shall be regarded as the standard. The values stated in inch-pound units are approximate.

1.7 *This standard does not purport to address all of the safety concerns, if any, associated with its use. It is the responsibility of the user of this standard to establish appropriate safety and health practices and determine the applicability of regulatory limitations prior to use.*

2. Referenced Documents

2.1 *ASTM Standards:*[2]

D422 Test Method for Particle-Size Analysis of Soils
D653 Terminology Relating to Soil, Rock, and Contained Fluids
D854 Test Methods for Specific Gravity of Soil Solids by Water Pycnometer
D1587 Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes
D2166 Test Method for Unconfined Compressive Strength of Cohesive Soil
D2216 Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
D2435 Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading
D2850 Test Method for Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils
D3740 Practice for Minimum Requirements for Agencies Engaged in Testing and/or Inspection of Soil and Rock as Used in Engineering Design and Construction
D4220 Practices for Preserving and Transporting Soil Samples
D4318 Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils
D4753 Guide for Evaluating, Selecting, and Specifying Balances and Standard Masses for Use in Soil, Rock, and Construction Materials Testing
D6026 Practice for Using Significant Digits in Geotechnical Data

3. Terminology

3.1 *Definitions*—The definitions of terms used in this test method shall be in accordance with Terminology D653.

3.2 *Definitions of Terms Specific to This Standard:*

3.2.1 *back pressure*—a pressure applied to the specimen pore-water to cause air in the pore space to compress and to

[1] This test method is under the jurisdiction of ASTM Committee D18 on Soil and Rock and is the direct responsibility of Subcommittee D18.05 on Strength and Compressibility of Soils.

Current edition approved Nov. 1, 2004. Published December 2004. Originally approved in 1988. Last previou edition approved in 2002 as D4767 – 02. DOI: 10.1520/D4767-04.

[2] For referenced ASTM standards, visit the ASTM website, www.astm.org, or contact ASTM Customer Service at service@astm.org. For *Annual Book of ASTM Standards* volume information, refer to the standard's Document Summary page on the ASTM website.

***A Summary of Changes section appears at the end of this standard.**

pass into solution in the pore-water thereby increasing the percent saturation of the specimen.

3.2.2 *effective consolidation stress*—the difference between the cell pressure and the pore-water pressure prior to shearing the specimen.

3.2.3 *failure*—the stress condition at failure for a test specimen. Failure is often taken to correspond to the maximum principal stress difference (maximum deviator stress) attained or the principal stress difference (deviator stress) at 15 % axial strain, whichever is obtained first during the performance of a test. Depending on soil behavior and field application, other suitable failure criteria may be defined, such as maximum effective stress obliquity, $\sigma'1/\sigma'3$, or the principal stress difference (deviator stress) at a selected axial strain other than 15 %.

4. Significance and Use

4.1 The shear strength of a saturated soil in triaxial compression depends on the stresses applied, time of consolidation, strain rate, and the stress history experienced by the soil.

4.2 In this test method, the shear characteristics are measured under undrained conditions and is applicable to field conditions where soils that have been fully consolidated under one set of stresses are subjected to a change in stress without time for further consolidation to take place (undrained condition), and the field stress conditions are similar to those in the test method.

NOTE 1—If the strength is required for the case where the soil is not consolidated during testing prior to shear, refer to Test Method D2850 or Test Method D2166.

4.3 Using the pore-water pressure measured during the test, the shear strength determined from this test method can be expressed in terms of effective stress. This shear strength may be applied to field conditions where full drainage can occur (drained conditions) or where pore pressures induced by loading can be estimated, and the field stress conditions are similar to those in the test method.

4.4 The shear strength determined from the test expressed in terms of total stresses (undrained conditions) or effective stresses (drained conditions) is commonly used in embankment stability analyses, earth pressure calculations, and foundation design.

NOTE 2—Notwithstanding the statements on precision and bias contained in this test method. The precision of this test method is dependent on the competence of the personnel performing it and the suitability of the equipment and facilities used. Agencies which meet the criteria of Practice D3740 are generally considered capable of competent testing. Users of this test method are cautioned that compliance with Practice D3740 does not ensure reliable testing. Reliable testing depends on several factors; Practice D3740 provides a means of evaluating some of those factors.

5. Apparatus

5.1 The requirements for equipment needed to perform satisfactory tests are given in the following sections. See Fig. 1 and Fig. 2

5.2 *Axial Loading Device*—The axial loading device shall be a screw jack driven by an electric motor through a geared transmission, a hydraulic loading device, or any other compression device with sufficient capacity and control to provide the rate of axial strain (loading) prescribed in 8.4.2. The rate of advance of the loading device shall not deviate by more than ±1 % from the selected value. Vibration due to the operation of the loading device shall be sufficiently small to not cause dimensional changes in the specimen or to produce changes in pore-water pressure when the drainage valves are closed.

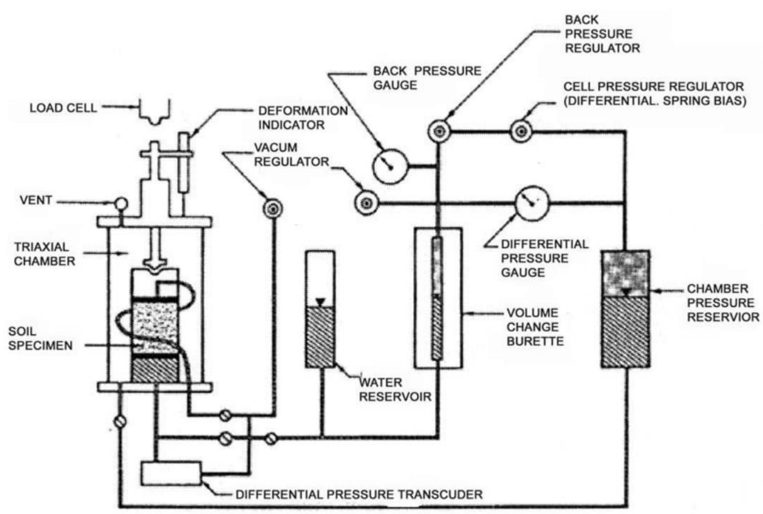


FIG. 1 Schematic Diagram of a Typical Consolidated Undrained Triaxial Apparatus

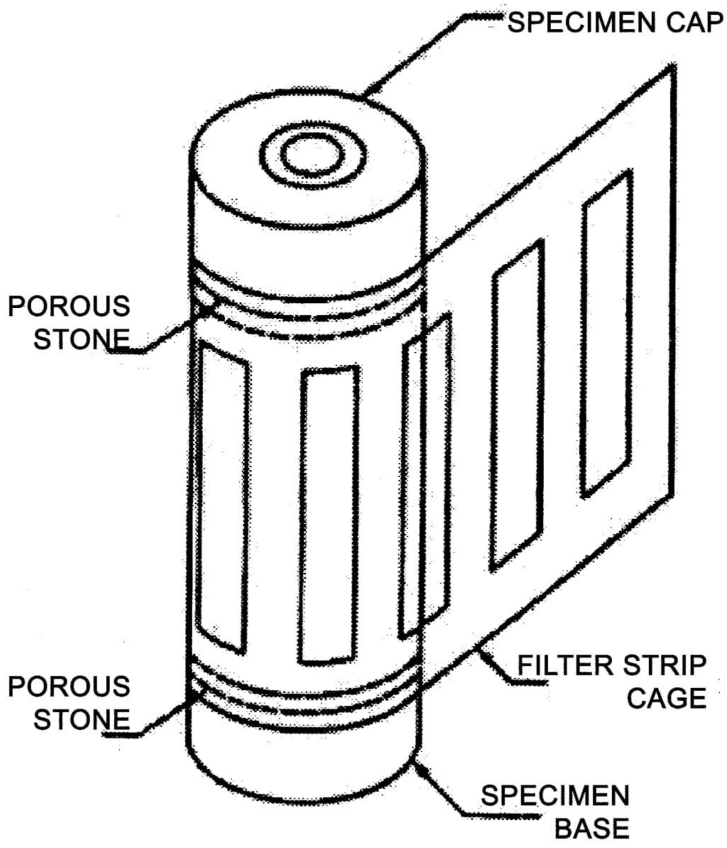


FIG. 2 Filter Strip Cage

NOTE 3—A loading device may be judged to produce sufficiently small vibrations if there are no visible ripples in a glass of water placed on the loading platform when the device is operating at the speed at which the test is performed.

5.3 *Axial Load-Measuring Device*—The axial load-measuring device shall be a load ring, electronic load cell, hydraulic load cell, or any other load-measuring device capable of the accuracy prescribed in this paragraph and may be a part of the axial loading device. The axial load-measuring device shall be capable of measuring the axial load to an accuracy of within 1 % of the axial load at failure. If the load-measuring device is located inside the triaxial compression chamber, it shall be insensitive to horizontal forces and to the magnitude of the chamber pressure.

5.4 *Triaxial Compression Chamber*—The triaxial chamber shall have a working chamber pressure equal to the sum of the effective consolidation stress and the back pressure. It shall consist of a top plate and a base plate separated by a cylinder. The cylinder may be constructed of any material capable of withstanding the applied pressures. It is desirable to use a transparent material or have a cylinder provided with viewing ports so the behavior of the specimen may be observed. The top plate shall have a vent valve such that air can be forced out of the chamber as it is filled. The baseplate shall have an inlet through which the pressure liquid is supplied to the chamber, and inlets leading to the specimen base to the cap to allow saturation and drainage of the specimen when required. The chamber shall provide a connection to the cap.

5.5 *Axial Load Piston*—The piston passing through the top of the chamber and its seal must be designed so the variation in axial load due to friction does not exceed 0.1 % of the axial load at failure and so there is negligible lateral bending of the piston during loading.

NOTE 4—The use of two linear ball bushings to guide the piston is recommended to minimize friction and maintain alignment.

NOTE 5—A minimum piston diameter of ⅙ the specimen diameter has been used successfully in many laboratories to minimize lateral bending.

5.6 *Pressure and Vacuum-Control Devices*—The chamber pressure and back pressure control devices shall be capable of applying and controlling pressures to within ±2 kPa (0.25 lb/in.^{2}) for effective consolidation pressures less than 200 kPa (28 lb/in.^{2}) and to within ±1 % for effective consolidation pressures greater than 200 kPa. The vacuum-control device shall be capable of applying and controlling partial vacuums to within ±2 kPa. The devices shall consist of pressure/volume controllers, self-compensating mercury pots, pneumatic pressure regulators, combination pneumatic pressure and vacuum regulators, or any other device capable of applying and controlling pressures or partial vacuums to the required tolerances. These tests can require a test duration of several day. Therefore, an air/water interface is not recommended for either the chamber pressure or back pressure systems, unless isolated from the specimen and chamber (e.g. by long tubing).

5.7 *Pressure- and Vacuum-Measurement Devices*—The chamber pressure-, back pressure-, and vacuum-measuring devices shall be capable of measuring pressures or partial vacuums to the tolerances given in 5.6. They may consist of Bourdon gages, pressure manometers, electronic pressure transducers, or any other device capable of measuring pressures, or partial vacuums to the stated tolerances. If separate devices are used to measure the chamber pressure and back pressure, the devices must be calibrated simultaneously and against the same pressure source. Since the chamber and back pressure are the pressures taken at the mid-height of the specimen, it may be necessary to adjust the calibration of the devices to reflect the hydraulic head of fluids in the chamber and back pressure control systems.

5.8 *Pore-Water Pressure-Measurement Device*—The specimen pore-water pressure shall also be measured to the tolerances given in 5.6. During undrained shear, the pore-water pressure shall be measured in such a manner that as little water as possible is allowed to go into or out of the specimen. To achieve this requirement, a very stiff electronic pressure transducer or null-indicating device must be used. With an electronic pressure transducer the pore-water pressure is read directly. With a null-indicating device a pressure control is continuously adjusted to maintain a constant level of the water/mercury interface in the capillary bore of the device. The pressure required to prevent movement of the water is equal to the pore-water pressure. Both measuring devices shall have a compliance of all the assembled parts of the pore-water pressure-measurement system relative to the total volume of the specimen, satisfying the following requirement:

$$(\Delta V/V)/\Delta u < 3.2 \times 10^{-6}\ \mathrm{m}^2/\mathrm{kN}\ (2.2 \times 10^{-5}\ \mathrm{in.}^2/\mathrm{lb}) \quad (1)$$

where:

ΔV = change in volume of the pore-water measurement system due to a pore pressure change, mm^3(in.^{3}),

V = total volume of the specimen, mm^3(in.^{3}), and

Δu = change in pore pressure, kPa (lb/in.^{2}).

NOTE 6—To meet the compliance requirement, tubing between the specimen and the measuring device should be short and thick-walled with small bores. Thermoplastic, copper, and stainless steel tubing have been used successfully.

5.9 *Volume Change Measurement Device*— The volume of water entering or leaving the specimen shall be measured with an accuracy of within ±0.05 % of the total volume of the specimen. The volume measuring device is usually a burette connected to the back pressure but may be any other device meeting the accuracy requirement. The device must be able to withstand the maximum back pressure.

5.10 *Deformation Indicator*—The vertical deformation of the specimen is usually determined from the travel of the piston acting on the top of the specimen. The piston travel shall be measured with an accuracy of at least 0.25 % of the initial specimen height. The deformation indicator shall have a range of at least 15 % of the initial height of the specimen and may be a dial indicator, linear variable differential transformer (LVDT), extensiometer, or other measuring device meeting the requirements for accuracy and range.

5.11 *Specimen Cap and Base*—The specimen cap and base shall be designed to provide drainage from both ends of the specimen. They shall be constructed of a rigid, noncorrosive, impermeable material, and each shall, except for the drainage provision, have a circular plane surface of contact with the porous disks and a circular cross section. It is desirable for the mass of the specimen cap and top porous disk to be as minimal as possible. However, the mass may be as much as 10 % of the axial load at failure. If the mass is greater than 0.5 % of the applied axial load at failure and greater than 50 g (0.1 lb), the axial load must be corrected for the mass of the specimen cap and top porous disk. The diameter of the cap and base shall be equal to the initial diameter of the specimen. The specimen base shall be connected to the triaxial compression chamber to prevent lateral motion or tilting, and the specimen cap shall be designed such that eccentricity of the piston-to-cap contact relative to the vertical axis of the specimen does not exceed 1.3 mm (0.05 in.). The end of the piston and specimen cap contact area shall be designed so that tilting of the specimen cap during the test is minimal. The cylindrical surface of the specimen base and cap that contacts the membrane to form a seal shall be smooth and free of scratches.

5.12 *Porous Discs*—Two rigid porous disks shall be used to provide drainage at the ends of the specimen. The coefficient of permeability of the disks shall be approximately equal to that of fine sand (1×10^{-4} cm/s (4×10^{-5} in./s)). The disks shall be regularly cleaned by ultrasonic or boiling and brushing and checked to determine whether they have become clogged.

5.13 *Filter-Paper Strips and Disks*— Filter-paper strips are used by many laboratories to decrease the time required for testing. Filter-paper disks of a diameter equal to that of the specimen may be placed between the porous disks and specimen to avoid clogging of the porous disks. If filter strips or disks are used, they shall be of a type that does not dissolve in water. The coefficient of permeability of the filter paper shall not be less than 1×10^{-5} cm/s (4×10^{-6} cm/s) for a normal pressure of 550 kPa (80 lb/in.^{2}). To avoid hoop tension, filter strips should cover no more than 50 % of the specimen periphery. Filter-strip cages have been successfully used by many laboratories. An equation for correcting the principal stress difference (deviator stress) for the effect of the strength of vertical filter strips is given in 10.4.3.1.

NOTE 7—Whatman's No. 54 Filter Paper has been found to meet the permeability and durability requirements.

5.14 *Rubber Membrane*—The rubber membrane used to encase the specimen shall provide reliable protection against leakage. Membranes shall be carefully inspected prior to use and if any flaws or pinholes are evident, the membrane shall be discarded. To offer minimum restraint to the specimen, the unstretched membrane diameter shall be between 90 and 95 % of that of the specimen. The membrane thickness shall not exceed 1 % of the diameter of the specimen. The membrane shall be sealed to the specimen cap and base with rubber O-rings for which the unstressed inside diameter is between 75 and 85 % of the diameter of the cap and base, or by other means that will provide a positive seal. An equation for correcting the principal stress difference (deviator stress) for the effect of the stiffness of the membrane is given in 10.4.3.2.

5.15 *Valves*—Changes in volume due to opening and closing valves may result in inaccurate volume change and pore-water pressure measurements. For this reason, valves in the specimen drainage system shall be of the type that produce minimum volume changes due to their operation. A valve may be assumed to produce minimum volume change if opening or closing the valve in a closed, saturated pore-water pressure system does not induce a pressure change of greater than 0.7 kPa (±0.1 lb/in.^{2}). All valves must be capable of withstanding applied pressures without leakage.

NOTE 8—Ball valves have been found to provide minimum volume-change characteristics; however, any other type of valve having suitable volume-change characteristics may be used.

5.16 *Specimen-Size Measurement Devices*— Devices used to determine the height and diameter of the specimen shall measure the respective dimensions to within ±0.1 % of the total dimension and shall be constructed such that their use will not disturb the specimen.

NOTE 9—Circumferential measuring tapes are recommended over calipers for measuring the diameter.

5.17 *Recorders*—Specimen behavior may be recorded manually or by electronic digital or analog recorders. If electronic recorders are used, it shall be necessary to calibrate the measuring devices through the recorder using known input standards.

5.18 *Sample Extruder*—The sample extruder shall be capable of extruding the soil core from the sampling tube at a uniform rate in the same direction of travel as the sample entered the tube and with minimum disturbance of the sample. If the soil core is not extruded vertically, care should be taken to avoid bending stresses on the core due to gravity. Conditions at the time of sample removal may dictate the direction of removal, but the principal concern is to minimize the degree of disturbance.

5.19 *Timer*—A timing device indicating the elapsed testing time to the nearest 1 s shall be used to obtain consolidation data (8.3.3).

5.20 *Balance*—A balance or scale conforming to the requirements of Specification D4753 readable (with no estimate) to 0.1 % of the test mass or better.

5.21 *Water Deaeration Device*—The amount of dissolved gas (air) in the water used to saturate the specimen shall be decreased by boiling, by heating and spraying into a vacuum, or by any other method that will satisfy the requirement for saturating the specimen within the limits imposed by the available maximum back pressure and time to perform the test.

5.22 *Testing Environment*—The consolidation and shear portion of the test shall be performed in an environment where temperature fluctuations are less than ±4°C (±7.2°F) and there is no direct contact with sunlight.

5.23 *Miscellaneous Apparatus*—Specimen trimming and carving tools including a wire saw, steel straightedge, miter box, vertical trimming lathe, apparatus for preparing compacted specimens, membrane and O-ring expander, water content cans, and data sheets shall be provided as required.

6. Test Specimen Preparation

6.1 *Specimen Size*—Specimens shall be cylindrical and have a minimum diameter of 33 mm (1.3 in.). The average height-to-average diameter ratio shall be between 2 and 2.5. An individual measurement of height or diameter shall not vary from average by more than 5 %. The largest particle size shall be smaller than ⅙ the specimen diameter. If, after completion of a test, it is found based on visual observation that oversize particles are present, indicate this information in the report of test data (11.2.23).

NOTE 10—If oversize particles are found in the specimen after testing, a particle-size analysis may be performed on the tested specimen in accordance with Test Method D422 to confirm the visual observation and the results provided with the test report (11.2.4).

6.2 *Undisturbed Specimens*—Prepare undisturbed specimens from large undisturbed samples or from samples secured in accordance with Practice D1587 or other acceptable undisturbed tube sampling procedures. Samples shall be preserved and transported in accordance with the practices for Group C samples in Practices D4220. Specimens obtained by tube sampling may be tested without trimming except for cutting the end surfaces plane and perpendicular to the longitudinal axis of the specimen, provided soil characteristics are such that no significant disturbance results from sampling. Handle specimens carefully to minimize disturbance, changes in cross section, or change in water content. If compression or any type of noticeable disturbance would be caused by the extrusion device, split the sample tube lengthwise or cut the tube in suitable sections to facilitate removal of the specimen with minimum disturbance. Prepare trimmed specimens, in an environment such as a controlled high-humidity room where soil water content change is minimized. Where removal of pebbles or crumbling resulting from trimming causes voids on the surface of the specimen, carefully fill the voids with remolded soil obtained from the trimmings. If the sample can be trimmed with minimal disturbance, a vertical trimming lathe may be used to reduce the specimen to the required diameter. After obtaining the required diameter, place the specimen in a miter box, and cut the specimen to the final height with a wire saw or other suitable device. Trim the surfaces with the steel straightedge. Perform one or more water content determinations on material trimmed from the specimen in accordance with Test Method D2216. Determine the mass and dimensions of the specimen using the devices described in 5.16 and 5.20. A minimum of three height measurements (120° apart) and at least three diameter measurements at the quarter points of the height shall be made to determine the average height and diameter of the specimen.

6.3 *Compacted Specimens*—Soil required for compacted specimens shall be thoroughly mixed with sufficient water to produce the desired water content. If water is added to the soil, store the material in a covered container for at least 16 h prior to compaction. Compacted specimens may be prepared by compacting material in at least six layers using a split mold of circular cross section having dimensions meeting the requirements enumerated in 6.1. Specimens may be compacted to the desired density by either: (*1*) kneading or tamping each layer until the accumulative mass of the soil placed in the mold is compacted to a known volume; or (*2*) by adjusting the number of layers, the number of tamps per layer, and the force per tamp. The top of each layer shall be scarified prior to the addition of material for the next layer. The tamper used to compact the material shall have a diameter equal to or less than ½ the diameter of the mold. After a specimen is formed, with the ends perpendicular to the longitudinal axis, remove the mold and determine the mass and dimensions of the specimen using the devices described in 5.16 and 5.20. Perform one or more water content determinations on excess material used to prepare the specimen in accordance with Test Method D2216.

NOTE 11—It is common for the unit weight of the specimen after removal from the mold to be less than the value based on the volume of the mold. This occurs as a result of the specimen swelling after removal of the lateral confinement due to the mold.

7. Mounting Specimen

7.1 *Preparations*—Before mounting the specimen in the triaxial chamber, make the following preparations:

7.1.1 Inspect the rubber membrane for flaws, pinholes, and leaks.

7.1.2 Place the membrane on the membrane expander or, if it is to be rolled onto the specimen, roll the membrane on the cap or base.

7.1.3 Check that the porous disks and specimen drainage tubes are not obstructed by passing air or water through the appropriate lines.

7.1.4 Attach the pressure-control and volume-measurement system and a pore-pressure measurement device to the chamber base.

7.2 Depending on whether the saturation portion of the test will be initiated with either a wet or dry drainage system, mount the specimen using the appropriate method, as follows in either 7.2.1 or 7.2.2. The dry mounting method is strongly recommended for specimens with initial saturation less than 90 %. The dry mounting method removes air prior to adding backpressure and lowers the backpressure needed to attain an adequate percent saturation.

NOTE 12—It is recommended that the dry mounting method be used for specimens of soils that swell appreciably when in contact with water. If the wet mounting method is used for such soils, it will be necessary to obtain the specimen dimensions after the specimen has been mounted. In such cases, it will be necessary to determine the double thickness of the membrane, the double thickness of the wet filter paper strips (if used), and the combined height of the cap, base, and porous disks (including the thickness of filter disks if they are used) so that the appropriate values may be subtracted from the measurements.

7.2.1 *Wet Mounting Method*:

7.2.1.1 Fill the specimen drainage lines and the pore-water pressure measurement device with deaired water.

7.2.1.2 Saturate the porous disks by boiling them in water for at least 10 min and allow to cool to room temperature.

7.2.1.3 If filter-paper disks are to be placed between the porous disks and specimen, saturate the paper with water prior to placement.

7.2.1.4 Place a saturated porous disk on the specimen base and wipe away all free water on the disk. If filter-paper disks are used, placed on the porous disk. Place the specimen on the disk. Next, place another filter-paper disk (if used), porous disk and the specimen cap on top of the specimen. Check that the specimen cap, specimen, filter-paper disks (if used) and porous disks are centered on the specimen base.

7.2.1.5 If filter-paper strips or a filter-paper cage are to be used, saturate the paper with water prior to placing it on the specimen. To avoid hoop tension, do not cover more than 50 % of the specimen periphery with vertical strips of filter paper.

7.2.1.6 Proceed with 7.3.

7.2.2 *Dry Mounting Method*:

7.2.2.1 Dry the specimen drainage system. This may be accomplished by allowing dry air to flow through the system prior to mounting the specimen.

7.2.2.2 Dry the porous disks in an oven and then place the disks in a desiccator to cool to room temperature prior to mounting the specimen.

7.2.2.3 Place a dry porous disk on the specimen base and place the specimen on the disk. Next, place a dry porous disk and the specimen cap on the specimen. Check that the specimen cap, porous disks, and specimen are centered on the specimen base.

NOTE 13—If desired, dry filter-paper disks may be placed between the porous disks and specimen.

7.2.2.4 If filter-paper strips or a filter-paper cage are to be used, the cage or strips may be held in place by small pieces of tape at the top and bottom.

7.3 Place the rubber membrane around the specimen and seal it at the cap and base with two rubber O-rings or other positive seal at each end. A thin coating of silicon grease on the vertical surfaces of the cap and base will aid in sealing the membrane. If filter-paper strips or a filter-paper cage are used, do not apply grease to surfaces in contact with the filter-paper.

7.4 Attach the top drainage line and check the alignment of the specimen and the specimen cap. If the dry mounting method has been used, apply a partial vacuum of approximately 35 kPa (5 lb/in.^{2}) (not to exceed the consolidation stress) to the specimen through the top drainage line prior to checking the alignment. If there is any eccentricity, release the partial vacuum, realign the specimen and cap, and then reapply the partial vacuum. If the wet mounting method has been used, the alignment of the specimen and the specimen cap may be checked and adjusted without the use of a partial vacuum.

8. Procedure

8.1 *Prior to Saturation*—After assembling the triaxial chamber, perform the following operations:

8.1.1 Bring the axial load piston into contact with the specimen cap several times to permit proper seating and alignment of the piston with the cap. During this procedure, take care not to apply an axial load to the specimen exceeding 0.5 % of the estimated axial load at failure. When the piston is brought into contact, record the reading of the deformation indicator to three significant digits.

8.1.2 Fill the chamber with the chamber liquid, being careful to avoid trapping air or leaving an air space in the chamber.

8.2 *Saturation*—The objective of the saturation phase of the test is to fill all voids in the specimen with water without undesirable prestressing of the specimen or allowing the specimen to swell. Saturation is usually accomplished by applying back pressure to the specimen pore water to drive air into solution after saturating the system by either: (*1*) applying vacuum to the specimen and dry drainage system (lines, porous disks, pore-pressure device, filter-strips or cage, and disks) and then allowing deaired water to flow through the system and specimen while maintaining the vacuum; or (*2*) saturating the drainage system by boiling the porous disks in water and allowing water to flow through the system prior to mounting the specimen. It should be noted that placing the air into solution is a function of both time and pressure. Accordingly, removing as much air as possible prior to applying back pressure will decrease the amount of air that will have to be placed into solution and will also decrease the back pressure required for saturation. In addition, air remaining in the specimen and drainage system just prior to applying back pressure will go into solution much more readily if deaired water is used for saturation. The use of deaired water will also decrease the time and back pressure required for saturation. Many procedures have been developed to accomplish saturation. The following are suggested procedures:

8.2.1 *Starting with Initially Dry Drainage System*—Increase the partial vacuum acting on top of the specimen to the maximum available vacuum. If the effective consolidation stress under which the strength is to be determined is less than the maximum partial vacuum, apply a lower partial vacuum to the chamber. The difference between the partial vacuum applied to the specimen and the chamber should never exceed the effective consolidation stress for the test and should not be less than 35 kPa (5 lb/in.^{2}) to allow for flow through the sample. After approximately 10 min, allow deaired water to percolate from the bottom to the top of the specimen under a differential vacuum of less than 20 kPa (3 lb/in.^{2}) (Note 14).

8.2.1.1 There should always be a positive effective stress of at least 13 kPa (2 lb/in.^{2}) at the bottom of the specimen during this part of the procedure. When water appears in the burette connected to the top of the specimen, close the valve to the bottom of the specimen and fill the burette with deaired water. Next, reduce the vacuum acting on top of the specimen through the burette to atmospheric pressure while simultaneously increasing the chamber pressure by an equal amount. This process should be performed slowly such that the difference between the pore pressure measured at the bottom of the specimen and the pressure at the top of the specimen should be allowed to equalize. When the pore pressure at the bottom of the specimen stabilizes, proceed with back pressuring of the specimen pore-water as described in . To check for equalization, close the drainage valves to the specimen and measure the pore pressure change until stable. If the change is less than 5 % of the chamber pressure, the pore pressure may be assumed to be stabilized.

NOTE 14—For saturated clays, percolation may not be necessary and water can be added simultaneously at both top and bottom.

8.2.2 *Starting with Initially Saturated Drainage System*—After filling the burette connected to the top of the specimen with deaired water, apply a chamber pressure of 35 kPa (5 lb/in.^{2}) or less and open the specimen drainage valves. When the pore pressure at the bottom of the specimen stabilizes, according to the method described in 8.2.1, or when the burette reading stabilizes, back pressuring of the specimen pore-water may be initiated.

8.2.3 *Back-Presuure Saturation*—To saturate the specimen, back pressuring is usually necessary. Fig. 3 [3] provides guidance on back pressure required to attain saturation. Additional guidance on the back-pressure process is given by Black[4] and Lee.[5]

8.2.3.1 *Applying Back Pressure*—Simultaneously increase the chamber and back pressure in steps with specimen drainage valves opened so that deaired water from the burette connected to the top and bottom of the specimen may flow into the specimen. To avoid undesirable prestressing of the specimen while applying back pressure, the pressures must be applied incrementally with adequate time between increments to permit equalization of pore-water pressure throughout the specimen. The size of each increment may range from 35 kPa (5 lb/in.^{2}) up to 140 kPa (20 lb/in. ^{2}), depending on the magnitude of the desired effective consolidation stress, and the percent saturation of the specimen just prior to the addition of the increment. The difference between the chamber pressure and the back pressure during back pressuring should not exceed 35 kPa unless it is deemed necessary to control swelling of the specimen during the procedure. The difference between the chamber and back pressure must also remain within ± 5 % when the pressures are raised and within $\pm$ 2 % when the

[3] Lowe, J., and Johnson, T. C., "Use of Back Pressure to Increase Degree of Saturation of Triaxial Test Specimens," *Proceedings, ASCE Research Conference on Shear Strength of Cohesive Soils* , Boulder, CO, 1960

[4] Black, A. W. and Lee, K. L. (1973), "Saturating Laboratory Samples by Back Pressure," *Journal of the Soil Mechanics and Foundation Division,* ASCE, Vol. 99, No. SM1, Proc. Paper 9484, Jan., pp. 75–93.

[5] Head, K. H., (1986), *Manual of Soil Laboratory Testing, Volume 3: Effective Stress Tests*, Pentech Press Limited, Graham Lodge, London, United Kingdom, pp. 787–796.

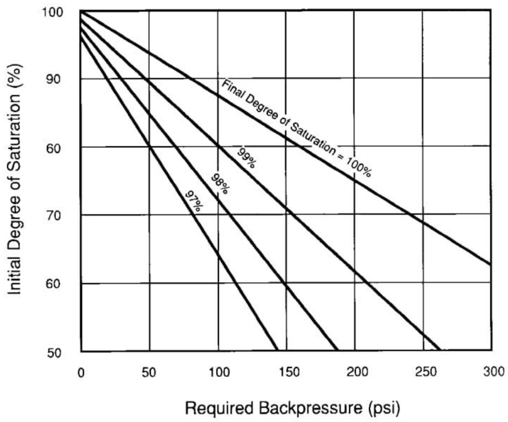


FIG. 3 Pressure to Attain Various Degrees of Saturation

pressures are constant. To check for equalization after application of a back pressure increment or after the full value of back pressure has been applied, close the specimen drainage valves and measure the change in pore-pressure over a 1-min interval. If the change in pore pressure is less than 5 % of the difference between the chamber pressure and the back pressure, another back pressure increment may be added or a measurement may be taken of the pore pressure Parameter B (see 8.2.4) to determine if saturation is completed. Specimens shall be considered to be saturated if the value of *B* is equal to or greater than 0.95, or if *B* remains unchanged with addition of back pressure increments.

NOTE 15—The relationships presented in Fig. 4 are based on the assumption that the water used for back pressuring is deaired and that the only source for air to dissolve into the water is air from the test specimen. If air pressure is used to control the back pressure, pressurized air will dissolve into the water, thus reducing the capacity of the water used for back pressure to dissolve air located in the pores of the test specimen. The problem is minimized by using a long (>5 m) tube that is impermeable to air between the air-water interface and test specimen, by separating the back-pressure water from the air by a material or fluid that is relatively impermeable to air, by periodically replacing the back-pressure water with deaired water, or by other means.

NOTE 16—Although the pore pressure Parameter *B* is used to determine adequate saturation, the *B*-value is also a function of soil stiffness. If the saturation of the sample is 100 %, the *B*-value measurement will increase with decreasing soil stiffness. Therefore, when testing soft soil samples, a *B*-value of 95 % may indicate a saturation less than 100 %.

NOTE 17—The back pressure required to saturate a compacted specimen may be higher for the wet mounting method than for the dry mounting method and may be as high as 1400 kPa (200 lb/in.^{2}).

NOTE 18—Many laboratories use differential pressure regulators and transducers to achieve the requirements for small differences between chamber and back pressure.

8.2.4 *Measurement of the Pore Pressure Parameter B*—Determine the value of the pore pressure Parameter *B* in accordance with 8.2.4.1 through 8.2.4.4. The pore pressure Parameter *B* is defined by the following equation:

$$B = \Delta u / \Delta \sigma_3 \qquad (2)$$

where:

Δu = change in the specimen pore pressure that occurs as a result of a change in the chamber pressure when the specimen drainage valves are closed, and

$\Delta \sigma_3$ = change in the chamber pressure.

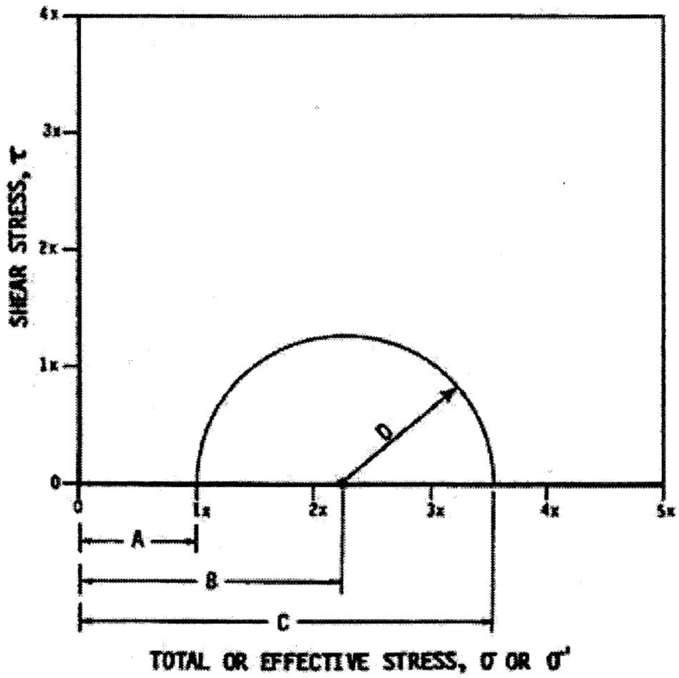
8.2.4.1 Close the specimen drainage valves, record the pore pressure, to the nearest 0.7 kPa (0.1 psi), and increase the chamber pressure by 70 kPa (10 lb/in.^{2}).

8.2.4.2 After approximately 2 min, determine and record the maximum value of the induced pore pressure to the nearest 0.7 kPa (0.1 psi),. For many specimens, the pore pressure may decrease after the immediate response and then increase slightly with time. If this occurs, values of Δu should be plotted with time and the asymptotic pore pressure used as the change in pore pressure. A large increase in Δu with time or values of Δu greater than $\Delta \sigma_3$ indicate a leak of chamber fluid into the specimen. Decreasing values of Δu with time may indicate a leak in that part of the pore pressure measurement system located outside of the chamber.

8.2.4.3 Calculate the *B*-value using Eq 2.

8.2.4.4 Reapply the same effective consolidation stress as existed prior to the *B*-value by reducing the chamber pressure by 70 kPa (10 lb/in.^{2}) or by alternatively, increasing the back pressure by 70 kPa. If *B* is continuing to increase with increasing back pressure, continue with back pressure saturation. If *B* is equal to or greater than 0.95 or if a plot of *B* versus back pressure indicates no further increase in *B* with increasing back pressure, initiate consolidation.

8.3 *Consolidation*—The objective of the consolidation phase of the test is to allow the specimen to reach equilibrium in a drained state at the effective consolidation stress for which a strength determination is required. During consolidation, data is obtained for use in determining when consolidation is complete and for computing a rate of strain to be used for the shear portion of the test. The consolidation procedure is as follows:



A = TOTAL OR EFFECTIVE MINOR PRINCIPAL STRESS (σ_3 or σ_3')
B = AVERAGE OF TOTAL OR EFFECTIVE PRINCIPAL STRESSES
C = TOTAL OR EFFECTIVE MAJOR PRINCIPAL STRESS (σ_1 or σ_1')
D = RADIUS OF THE MOHR'S CIRCLE; HALF THE PRINCIPAL STRESS DIFFERENCE

FIG. 4 Construction of Mohr Stress Circle

8.3.1 When the saturation phase of the test is completed, bring the axial load piston into contact with the specimen cap, and record the reading on the deformation indicator to three significant digits. During this procedure, take care not to apply an axial load to the specimen exceeding 0.5 % of the estimated axial load at failure. After recording the reading, raise the piston a small distance above the specimen cap, and lock the piston in place.

8.3.2 With the specimen drainage valves closed, hold the maximum back pressure constant and increase the chamber pressure until the difference between the chamber pressure and the back pressure equals the desired effective consolidation pressure. Consolidation in stages is required when filter strips for radial drainage are used, and the load increment ratio shall not exceed two.

8.3.3 Obtain an initial burette reading, and, then, open appropriate drainage valves so that the specimen may drain from both ends into the burette. At increasing intervals of elapsed time (0.1, 0.2, 0.5, 1, 2, 4, 8, 15, and 30 min and at 1, 2, 4, and 8 h, and so forth) observe and record the burette readings, and, after the 15-min reading, record the accompanying deformation indicator readings obtained by carefully bringing the piston in contact with the specimen cap. If burette and deformation indicator readings are to be plotted against the square root of time, the time intervals at which readings are taken may be adjusted to those that have easily obtained square roots, for example, 0.09, 0.25, 0.49, 1, 4, and 9 min, and so forth. Depending on soil type, time intervals may be changed to convenient time intervals which allow for adequate definition of volume change versus time.

NOTE 19—In cases where significant amounts of fines may be washed from the specimen because of high initial hydraulic gradients, it is permissible to gradually increase the chamber pressure to the total desired pressure over a period with the drainage valves open. If this is done, recording of data should begin immediately after the total pressure is reached.

8.3.4 Plot the burette and deformation indicator readings versus either the logarithm or square root of elapsed time. Allow consolidation to continue for at least one log cycle of time or one overnight period after 100 % primary consolidation has been achieved as determined in accordance with one of the procedures outlined in Test Method D2435. A marked deviation between the slopes of the burette and deformation indicator curves toward the end of consolidation based on deformation indicator readings indicates leakage of fluid from the chamber into the specimen, and the test shall be terminated.

8.3.5 Determine the time for 50 % primary consolidation, t_{50}, in accordance with one of the procedures outlined in Test Method D2435.

8.4 *Shear*—During shear, the chamber pressure shall be kept constant while advancing the axial load piston downward against the specimen cap using controlled axial strain as the loading criterion. Specimen drainage is not permitted during shear.

8.4.1 *Prior to Axial Loading*—Before initiating shear, perform the following:

8.4.1.1 By opening or closing the appropriate valves, isolate the specimen so that during shear the specimen pore-water pressure will be measured by the pore-pressure measurement device and no drainage will occur.

8.4.1.2 Place the chamber in position in the axial loading device. Be careful to align the axial loading device, the axial load-measuring device, and the triaxial chamber to prevent the application of a lateral force to the piston during shear.

8.4.1.3 Bring the axial load piston into contact with the specimen cap to permit proper seating and realignment of the piston with the cap. During this procedure, care should be taken not to apply an axial load to the specimen exceeding 0.5 % of the estimated axial load at failure. If the axial load-measuring device is located outside of the triaxial chamber, the chamber pressure will produce an upward force on the piston that will react against the axial loading device. In this case, start shear with the piston slightly above the specimen cap, and before the piston comes into contact with the specimen cap, either (*1*) measure and record the initial piston friction and upward thrust of the piston produced by the chamber pressure and later correct the measured axial load, or (*2*) adjust the axial load-measuring device to compensate for the friction and thrust. The variation in the axial load-measuring device reading should not exceed 0.1 % of the estimated failure load when the piston is moving downward prior to contacting the specimen cap. If the axial load-measuring device is located inside the chamber, it will not be necessary to correct or compensate for the uplift force acting on the axial loading device or for piston friction. However, if an internal load-measuring device of significant flexibility is used in combination with an external deformation indicator, correction of the deformation readings may be necessary. In both cases, record the initial reading on the pore-water pressure measurement device to the nearest 0.7 kPa (0.1 psi) immediately prior to when the piston contacts the specimen cap and the reading on the deformation indicator to three significant digits when the piston contacts the specimen cap.

8.4.1.4 Check for pore pressure stabilization. Record the pore pressure to the nearest 0.7 kPa (0.1 psi). Close the drainage valves to the specimen, and measure the pore pressure change until stable. If the change is less than 5 % of the chamber pressure, the pore pressure may be assumed to be stabilized.

8.4.2 *Axial Loading*—Apply axial load to the specimen using a rate of axial strain that will produce approximate equalization of pore pressures throughout the specimen at failure. Assuming failure will occur after 4 %, a suitable rate of strain, $'\varepsilon$, may be determined from the following equation:

$$'\varepsilon = 4\,\%/(10\,t_{50}) \quad (3)$$

where:

t_{50} = time value obtained in 8.3.5.

If, however, it is estimated that failure will occur at a strain value lower than 4 %, a suitable strain rate may be determined using Eq 3 by replacing 4 % with the estimated failure strain. This rate of strain will provide for determination of accurate effective stress paths in the range necessary to define effective strength envelopes.

8.4.2.1 At a minimum, record load and deformation to three significant digits, and pore-water pressure values to the nearest 0.7 kPa (0.1 psi), at increments of 0.1 to 1 % strain and, thereafter, at every 1 %. Take sufficient readings to define the stress-strain curve; hence, more frequent readings may be required in the early stages of the test and as failure is approached. Continue the loading to 15 % strain, except loading may be stopped when the principal stress difference (deviator stress) has dropped 20 % or when 5 % additional axial strain occurs after a peak in principal stress difference (deviator stress).

NOTE 20—The use of a manually adjusted null-indicating device will require nearly continuous attention to ensure the criterion for undrained shear.

9. Removing Specimen

9.1 When shear is completed, perform the following:

9.1.1 Remove the axial load and reduce the chamber and back pressures to zero.

9.1.2 With the specimen drainage valves remaining closed, quickly remove the specimen from the apparatus so that the specimen will not have time to absorb water from the porous disks.

9.1.3 Remove the rubber membrane (and the filter-paper strips or cage from the specimen if they were used), and determine the water content of the total specimen in accordance with the procedure in Test Method D2216. (Free water remaining on the specimen after removal of the membrane should be blotted away before obtaining the water content.) In cases where there is insufficient material from trimmings for index property tests, that is, where specimens have the same diameter as the sampling tube, the specimen should be weighed prior to removing material for index property tests and a representative portion of the specimen used to determine its final water content. Prior to placing the specimen (or portion thereof) in the oven to dry, sketch or photograph the specimen showing the mode of failure (shear plane, bulging, and so forth).

10. Calculation

10.1 Measurements and calculations shall contain three significant digits.

10.2 *Initial Specimen Properties*—Using the dry mass of the total specimen, calculate and record the initial water content, volume of solids, initial void ratio, initial percent saturation, and initial dry unit weight. Calculate the specimen volume from values measured in 6.2 or 6.3. Calculate the volume of solids by dividing the dry mass of the specimen by the specific gravity of the solids (Note 20) and dividing by the density of water. Calculate the void ratio by dividing the volume of voids by the volume of solids where the volume of voids is assumed to be the difference between the specimen volume and the volume of the solids. Calculate dry density by dividing the dry mass of the specimen by the specimen volume.

NOTE 21—The specific gravity of solids can be determined in accordance with Test Method D854 or it may be assumed based on previous test results.

10.3 *Specimen Properties After Consolidation*—Calculate the specimen height and area after consolidation as follows:

10.3.1 Height of specimen after consolidation, H_c, is determined from the following equation:

$$H_c = H_o - \Delta H_o \tag{4}$$

where:
H_o = initial height of specimen, and
ΔH_o = change in height of specimen at end of consolidation.

See Fig. 4.

10.3.2 The cross-sectional area of the specimen after consolidation, A_c, shall be computed using one of the following methods. The choice of the method to be used depends on whether shear data are to be computed as the test is performed (in which case Method A would be used) or on which of the two methods, in the opinion of a qualified person, yield specimen conditions considered to be most representative of those after consolidation. Alternatively, the average of the two calculated areas may be appropriate.

10.3.2.1 *Method A*:

$$A_c = (V_o - \Delta V_{sat} - \Delta V_c) / H_c \tag{5}$$

where:
V_o = initial volume of specimen,
ΔV_c = change in volume of specimen during consolidation as indicated by burette readings, and
ΔV_{sat} = change in volume of specimen during saturation as follows:
ΔV_{sat} = $3V_o[\Delta H_s/H_o]$

where:
ΔH_s = change in height of the specimen during saturation.

10.3.2.2 *Method B*:

$$Ac = (V_{wf} + V_s) / H_c \tag{6}$$

where:
V_{wf} = final volume of water (based on final water content), and
V_s = volume of solids as follows:
V_s = $w_s/(G_s p_w)$

where:
w_s = specimen dry mass,
G_s = specific gravity of solids, and
p_w = density of water.

10.3.3 Using the calculated dimensions of the specimen after consolidation, and assuming that the water content after consolidation is the same as the final water content, calculate the consolidated void ratio and percent saturation.

NOTE 22—The specimen will absorb water from the porous disks and drainage lines during the time it is being removed from the apparatus. When this effect is significant, Method A will yield more reasonable values.

NOTE 23— In this test method, the equations are written such that compression and consolidation are considered positive.

10.4 *Shear Data*:

10.4.1 Calculate the axial strain, ε_1, for a given applied axial load as follows:

$$\varepsilon_1 = \Delta H / H_c \tag{7}$$

where:
ΔH = change in height of specimen during loading as determined from deformation indicator readings, and
H_c = height of specimen after consolidation.

10.4.2 Calculate the cross-sectional area, A, for a given applied axial load as follows:

$$A = A_c / (1 - \varepsilon_1) \tag{8}$$

where:
A_c = average cross-sectional area of the specimen after consolidation, and
ε_1 = axial strain for the given axial load.

NOTE 24—The cross-sectional area computed in this manner is based on the assumption that the specimen deforms as a right circular cylinder during shear. In cases where there is localized bulging, it may be possible to determine more accurate values for the area based on specimen dimension measurements obtained after shear.

10.4.3 Calculate the principal stress difference (deviator stress), $\sigma_1 - \sigma_3$, for a given applied axial load as follows:

$$\sigma_1 - \sigma_3 = P / A \tag{9}$$

where:
P = given applied axial load (corrected for uplift and piston friction if required as obtained in 8.4.1.3), and
A = corresponding cross-sectional area.

10.4.3.1 *Correction for Filter-Paper Strips*— For vertical filter-paper strips which extend over the total length of the specimen, apply a filter-paper strip correction to the computed values of the principal stress difference (deviator stress), if the error in principal stress difference (deviator stress) due to the strength of the filter-paper strips exceeds 5 %.

(*1*) For values of axial strain above 2 %, use the following equation to compute the correction:

$$\Delta(\sigma_1 - \sigma_3) = K_{fp} P_{fp} / A_c \quad (10)$$

where:

$\Delta(\sigma_1 - \sigma_3)$ = correction to be subtracted from the measured principal stress difference (deviator stress),
K_{fp} = load carried by filter-paper strips per unit length of perimeter covered by filter-paper,
P_{fp} = perimeter covered by filter-paper, and
A_c = cross-sectional area of specimen after consolidation.

(*2*) For values of axial strain of 2 % or less, use the following equation to compute the correction:

$$\Delta(\sigma_1 - \sigma_3) = 50\varepsilon_1 K_{fp} P_{fp} / A_c \quad (11)$$

where:
ε_1 = axial strain (decimal form) and other terms are the same as those defined in Subparagraph (*1*) of 10.4.3.1.

NOTE 25—For filter-paper generally used in triaxial testing, K_{fp} is approximately 0.19 kN/m (1.1 lb/in.).

10.4.3.2 *Correction for Rubber Membrane*— Use the following equation to correct the principal stress difference (deviator stress) for the effect of the rubber membrane if the error in principal stress difference (deviator stress) due to the strength of the membrane exceeds 5 %:

$$\Delta(\sigma_1 - \sigma_3) = (4E_m t_m \varepsilon) / D_c \quad (12)$$

where:

$\Delta(\sigma_1 - \sigma_3)$ = correction to be subtracted from the measured principal stress difference (deviator stress),
D_c = $\sqrt{4A_c/\pi}$ = diameter of specimen after consolidation,
E_m = Young's modulus for the membrane material,
t_m = thickness of the membrane, and
ε_1 = axial strain (decimal form).

(*1*) The Young's modulus of the membrane material may be determined by hanging a 15-mm (0.5-in.) circumferential strip of membrane using a thin rod, placing another rod through the bottom of the hanging membrane, and measuring the force per unit strain obtained by stretching the membrane. The modulus value may be computed using the following equation:

$$E_m = (F / A_m) / (\Delta L / L) \quad (13)$$

where:

E_m = Young's modulus of the membrane material,
F = force applied to stretch the membrane,
L = unstretched length of the membrane,
ΔL = change in length of the membrane due to the force, F, and
A_m = area of the membrane = 2 t_m W_s

where:

t_m = thickness of the membrane, and
W_s = width of circumferential strip, 0.5 in. (15 mm).

NOTE 26—A typical value of E_m for latex membranes is 1400 kPa (200 lb/in.).

NOTE 27—The corrections for filter-paper strips and membranes are based on simplified assumptions concerning their behavior during shear. Their actual behavior is complex, and there is not a consensus on more exact corrections.

10.4.4 Calculate the effective minor principal stress, σ'_3 for a given applied axial load as follows:

$$\sigma'_3 = \sigma_3 - \Delta u \quad (14)$$

where:

σ_3 = effective consolidation stress, and
Δu = induced pore-water pressure at the given axial load (total pore-water pressure minus the total back pressure).

10.5 *Principal Stress Difference (Deviator Stress) and Induced Pore-Water Pressure versus Strain Curves*—Prepare graphs showing relationships between principal stress difference (deviator stress) and induced pore-water pressure with axial strain, plotting deviator stress and induced pore-water pressure as ordinates and axial strain as abscissa. Select the principal stress difference (deviator stress) and axial strain at failure in accordance with 3.2.3.

10.6 *p′ – q Diagram*— Prepare a graph showing the relationship between p', $(\sigma'_1+\sigma'_3)/2$ and q, $(\sigma_1 - \sigma_3)/2$, plotting q as ordinate and p' as abscissa using the same scale. The value of p' for a given axial load may be computed as follows:

$$p' = ((\sigma_1 - \sigma_3) + 2\sigma'_3) / 2 \quad (15)$$

where:

$\sigma_1 - \sigma_3$ = principal stress difference (deviator stress), and
σ'_3 = effective minor principal stress.

10.7 Determine the major and minor principal stresses at failure based on total stresses, σ_{1f} and σ_{3f} respectively, and on effective stresses, σ'_{1f} and σ'_{3f} respectively, as follows:

$$\sigma_{3f} = \text{effective consolidation stress,} \quad (16)$$

$$\sigma_{1f} = (\sigma_1 - \sigma_3) \text{ at failure } + \sigma_{3f}, \quad (17)$$

$$\sigma'_3 f = \sigma_{3f} - \Delta u_f \text{, and} \quad (18)$$

$$\sigma'_{1f} = (\sigma_1 - \sigma_3) \text{ at failure } + \sigma'_{3f} \quad (19)$$

where Δu_f is the induced pore-water pressure at failure.

10.8 *Mohr Stress Circles*—If desired, construct Mohr stress circles at failure based on total and effective stresses on an arithmetic plot with shear stress as ordinate and normal stress as abscissa using the same scales. The circle based on total stresses is drawn with a radius of one half the principal stress difference (deviator stress) at failure with its center at a value equal to one half the sum of the major and minor total principal stresses. The Mohr stress circle based on effective stresses is drawn in a similar manner except that its center is at a value equal to one half the sum of the major and minor effective principal stresses.

11. Report: Test Data Sheet(s)/Form(s)

11.1 The methodology used to specify how data are recorded on the data sheet(s)/form(s), as given below, is covered in 7.2.1.3.

11.2 Record as a minimum the following general information (data):

11.2.1 Identification data and visual description of specimen, including soil classification and whether the specimen is undisturbed, compacted, or otherwise prepared, 11.2.2 Values of plastic limit and liquid limit, if determined in accordance with Test Method D4318,

11.2.3 Value of specific gravity of solids and notation if the value was determined in accordance with Test Method D854 or assumed,

11.2.4 Particle-size analysis, if determined in accordance with Test Method D422,

11.2.5 Initial specimen dry unit weight, void ratio, water content, and percent saturation, (specify if the water content specimen was obtained from cuttings or the entire specimen),

NOTE 28—The specific gravity determined in accordance with Test Method D854 is required for calculation of the saturation. An assumed specific gravity may be used provided it is noted in the test report that an assumed value was used.

11.2.6 Initial height and diameter of specimen,

11.2.7 Method followed for specimen saturation (that is, dry or wet method),

11.2.8 Total back pressure,

11.2.9 The pore pressure Parameter B at the end of saturation,

11.2.10 Effective consolidation stress,

11.2.11 Time to 50 % primary consolidation,

11.2.12 Specimen dry unit weight, void ratio, water content, and percent saturation after consolidation,

11.2.13 Specimen cross-sectional area after consolidation and method used for determination,

11.2.14 Failure criterion used,

11.2.15 The value of the principal stress difference (deviator stress) at failure and the values of the effective minor and major principal stresses at failure, (indicate when values have been corrected for effects due to membrane or filter strips, or both),

11.2.16 Axial strain at failure, percent,

11.2.17 Rate of strain, percent per minute,

11.2.18 Principal stress difference (deviator stress) and induced pore-water pressure versus axial strain curves as described in 10.5,

11.2.19 The $p' - q$ diagram as described in 10.6,

11.2.20 Mohr stress circles based on total and effective stresses, (optional),

11.2.21 Slope of angle of the failure surface (optional),

11.2.22 Failure sketch or photograph of the specimen, and

11.2.23 Remarks and notations regarding any unusual conditions such as slickensides, stratification, shells, pebbles, roots, and so forth, or other information necessary to properly interpret the results obtained, including any departures from the procedure outlined.

12. Precision and Bias

12.1 *Precision*—Test data on precision is not presented due to the nature of the soil materials tested by this procedure. It is either not feasible or too costly at this time to have ten or more laboratories participate in a round-robin testing program. Subcommittee D18.05 is seeking any data from users of this test method that might be used to make a limited statement on precision.

12.2 *Bias*—There is no accepted reference value for this test method, therefore, bias cannot be determined.

13. Keywords

13.1 back pressure saturation; cohesive soil; consolidated undrained strength; strain-controlled loading; stress-strain relationships; total and effective stresses

SUMMARY OF CHANGES

In accordance with Committee D18 policy, this section identifies the location of changes made to this standard since the last edition (2002) that may impact the use of this standard.

(1) The cap connection was changed to be a requirement of the chamber equipment in 5.4, rather than a requirement specific to the baseplate.
(2) Pressure/volume controller were added as acceptable vacuum control devices in 5.6.
(3) A requirement was added for isolating air/water interfaces (if used) from the pressure systems in 5.6.
(4) Note 15 was made 7.2.1.3, making wetting of filter paper disks mandatory when using the wet mounting method.
(5) In section 8.2.3, references concerning back pressure saturation were provided. An associated figure was added as Figure 3. subsequent sections, notes, and figures were renumbered.
(6) Footnotes 3, 4, and 5 were added.

Appendix E: PMT results

ENSAYO PRESIOMÉTRICO

ASTM D-4719

OBRA: 21 vivienda en urb Albacerrado, Tarifa

PETICIONARIO: Lima Solar S.L.

SONDEO: SR 02

ENSAYO: P-2

PROFUNDIDAD: 7.00 - 7.50 m

SITUACIÓN: 7.00 - 7.50 m

LITOLOGÍA: Arcilla margosa

EQUIPO: PRESIOMETRO OYO ELASMETER-200

FECHA: 26/03/2018

Nº Lectura	Preal kPa	Pcorregida kPa	r 0" mm	r 30" mm	r 60" mm	Δr (r 60" - r 30") mm
1	0.00	0.00	0.00	0.00	0.00	0.00
2	377.30	278.32	0.45	0.60	0.66	0.06
3	459.62	346.92	0.65	0.76	0.79	0.03
4	677.18	510.58	1.00	1.15	1.20	0.05
5	857.50	661.50	1.20	1.38	1.42	0.04
6	1045.66	825.16	1.48	1.62	1.66	0.04
7	1233.82	969.22	1.73	1.86	1.91	0.05
8	1423.94	1139.74	2.00	2.16	2.22	0.06
9	1614.06	1300.46	2.34	2.53	2.61	0.08
10	1804.18	1446.48	2.91	3.03	3.14	0.11
11	1994.30	1568.00	3.50	3.66	3.80	0.14
12	2185.40	1719.90	4.00	4.43	4.61	0.18
13	2376.50	1886.50	4.82	5.00	5.25	0.25
14	2579.36	2045.26	5.29	5.57	5.94	0.37
15	2778.30	2229.50	6.00	6.25	6.68	0.43
16	2964.50	2366.70	6.82	7.35	7.85	0.50
17	3164.42	2547.02	8.00	8.27	8.88	0.61
18	3356.50	2719.50	9.00	9.25	9.99	0.74
19	2018.80	1577.80	9.87	9.85	9.83	-0.02
20	1548.40	1259.30	9.63	9.61	9.59	-0.02
21	1146.60	882.00	9.47	9.43	9.40	-0.03
22	823.20	632.10	9.32	9.17	9.16	-0.01

CÁLCULO DE LA PRESIÓN LÍMITE

Pl = ((r2 + ro2 - (2 r. ro)) / ro2

Pl = 2720 Kpa

PRESIÓN DE FLUENCIA

1300 (KPa)

CÁLCULO DE MÓDULO PRESIOMÉTRICO

Ep = (1+θ) . rm . (ΔP / Δr)

Ep = 22470 Kpa

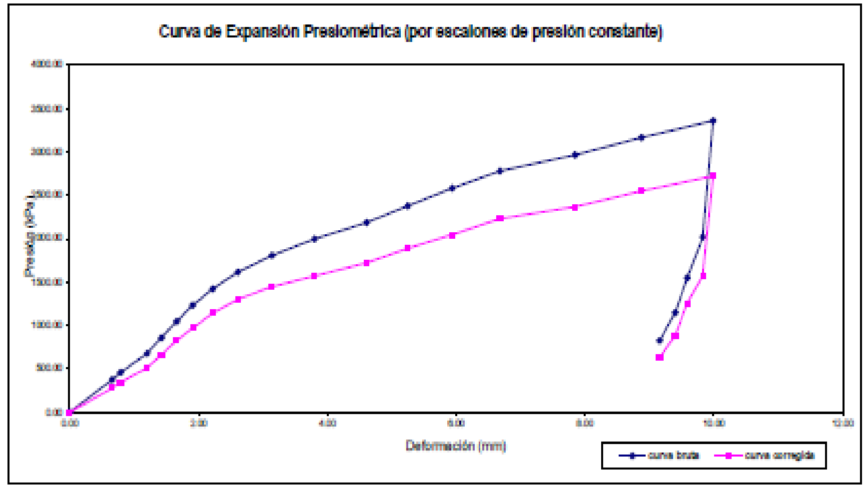
Ep = Módulo presiométrico ó Módulo de Menard

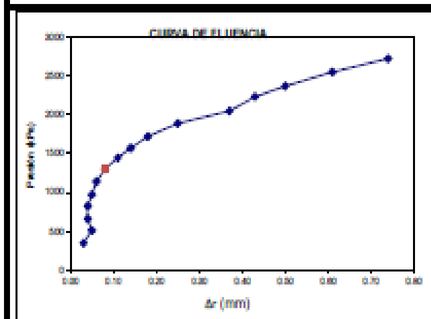
θ = Coeficiente de Poisson

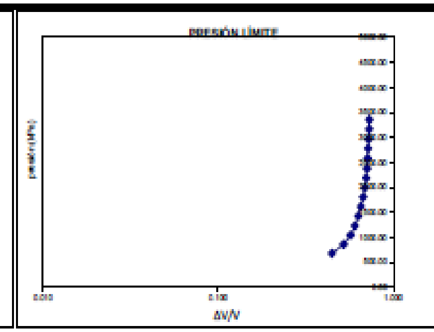
ΔP = Incremento de presion considerado en la rama elástica.

Δr = Variación de radio considerado en la rama elástica, tras el ciclo de descarga-recarga

rm = Radio medio







ENSAYO PRESIOMÉTRICO

ASTM D-4719

OBRA: 21 vivienda en urb Albacerrado, Tarifa

PETICIONARIO: Lima Solar S.L.

SONDEO: SR 02
ENSAYO: P-3
PROFUNDIDAD: 9.00 - 9.50 m

SITUACIÓN: 9.00 - 9.50 m
LITOLOGÍA: Marga arcillosa
EQUIPO: PRESIOMETRO OYO ELASMETER-200

FECHA: 26/03/2018

Nº Lectura	Preal kPa	Pcorregida kPa	r 0" mm	r 30" mm	r 60" mm	Δr (r 60" - r 30") mm
1	0.00	0.00	0.00	0.00	0.00	0.00
2	537.04	355.74	1.00	1.20	1.22	0.02
3	862.40	666.40	1.24	1.35	1.37	0.02
4	1044.68	848.68	1.40	1.43	1.44	0.01
5	1232.84	1022.14	1.48	1.50	1.52	0.02
6	1422.96	1202.46	1.54	1.59	1.61	0.02
7	1613.08	1377.88	1.63	1.68	1.70	0.02
8	1804.18	1554.28	1.73	1.79	1.81	0.02
9	1996.26	1731.66	1.83	1.91	1.94	0.03
10	2187.36	1908.06	1.96	2.07	2.11	0.04
11	2379.44	2090.34	2.15	2.28	2.34	0.06
12	2571.52	2257.92	2.40	2.54	2.61	0.07
13	2762.62	2414.72	2.70	2.85	2.96	0.11
14	2954.70	2562.70	3.11	3.37	3.51	0.14
15	3145.80	2704.80	3.72	3.97	4.13	0.16
16	3335.92	2850.82	4.27	4.66	4.93	0.27
17	3524.08	2975.28	5.21	6.12	6.61	0.49
18	3714.20	3111.50	7.00	7.75	8.14	0.39
19	3811.22	3184.02	8.48	9.13	9.60	0.47
20	2657.76	2312.80	9.59	9.57	9.55	-0.02
21	2099.16	1829.66	9.50	9.42	9.40	-0.02
22	1470.00	1229.90	9.35	9.29	9.27	-0.02

CÁLCULO DE LA PRESIÓN LÍMITE
Pl = ((r2 + ro2 - (2 r. ro)) / ro2

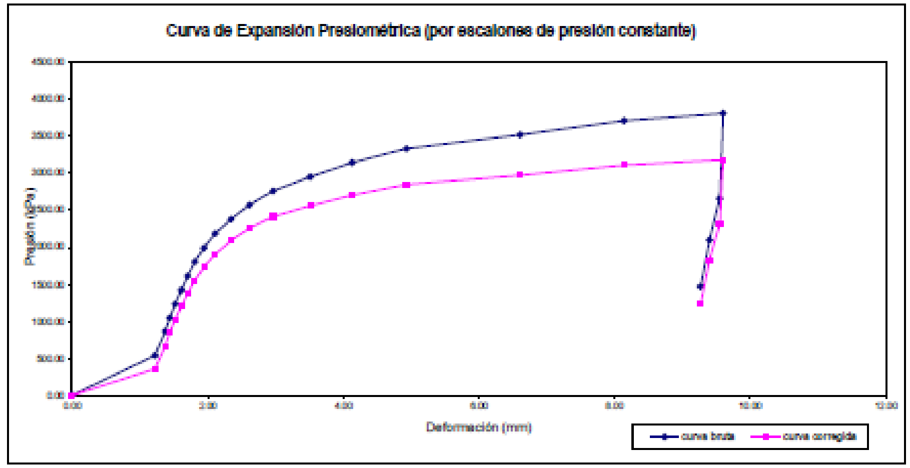
Pl = 3200 Kpa

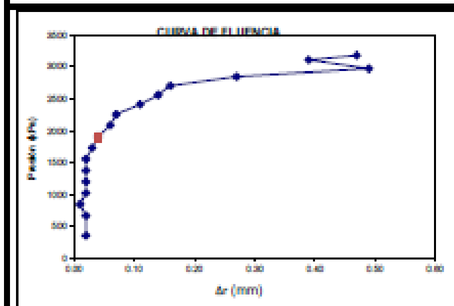
PRESIÓN DE FLUENCIA
1900 (KPa)

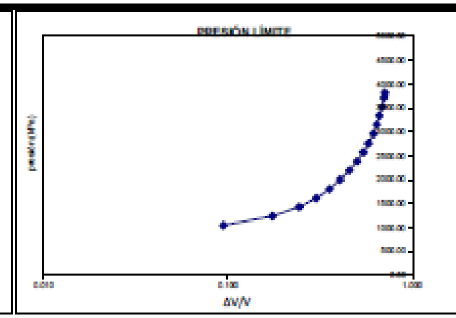
CÁLCULO DE MÓDULO PRESIOMÉTRICO
Ep = (1+θ) . rm . (ΔP / Δr)

Ep = 69236 Kpa

Ep = Módulo presiométrico ó Módulo de Menard
θ = Coeficiente de Poisson
ΔP = Incremento de presion considerado en la rama elástica.
Δr = Variación de radio considerado en la rama elástica, tras el ciclo de descarga-recarga
rm = Radio medio







Appendix F: Triaxial Test lab report

Sergeyco

TRIAXIAL COMPRESSION TEST

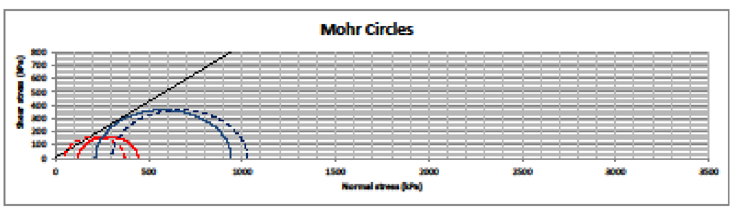
WORK:	0
APPLICANT:	0

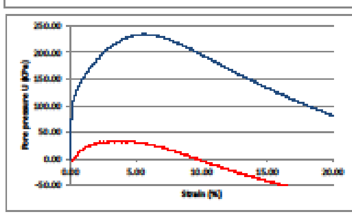
SAMPLE REFERENCE:	**SA-04-26-18**	**Sheet No:**		5.6
Sampling date:	26/03/2018	**Testing date:**	23/04/2018	
Sample location:	BH-2, between -7.35 & -7.600 m.	**Sample Type:**	Undisturbed	**Page:** 2/2
Test type:	CU			

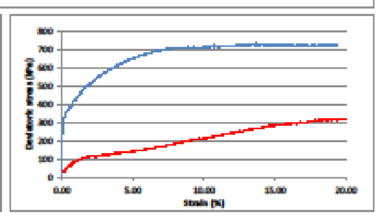
Tests:	**TRIAXIAL COMPRESSION TEST**

SAMPLE DATA

GENERAL DATA	I	II	III
Chamber pressure (KPa)	650		900
Inner pressure (KPa)	600		600
Diameter (cm)	3.82		3.82
Heigth (cm)	7.72		7.88
Initial moisture (%)	13.10		12.47
Final moisture (%)	16.19		15.87
Dry Density (g/cm^3)	1.76		1.76
Cohesion (KPa)	0		
Friction angle (°)	40.24		







Gibraltar, 17/05/2018

<u>TEST RESPONSIBLE:</u>
F. Javier Manzano Diosdado
GEOLOGIST

<u>LABORATORY HEADMASTER</u>
Daniel Mª. Sotillo Sáez.
GEOLOGIST

Sergeyco

Sergeyco

TRIAXIAL COMPRESSION TEST

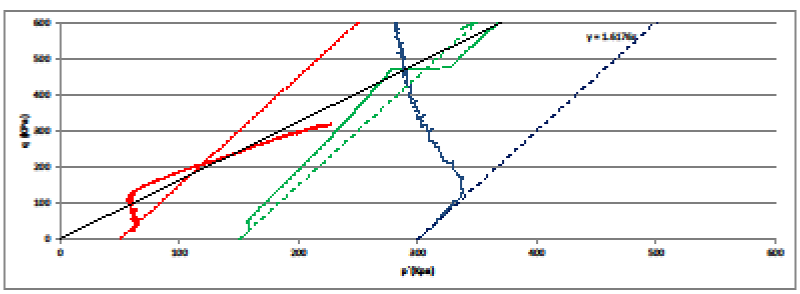
WORK:	0
APPLICANT:	0

SAMPLE REFERENCE:	**SA-04-26-18**	**Sheet No:**		5.6
Sampling date:	26/03/2018	**Testing date:**	23/04/2018	
Sample location:	BH-2, between -7.35 & -7.600 m.	**Sample Type:**	Undisturbed	**Page:** 2/2
Test type:	CU			

Tests:	**TRIAXIAL COMPRESSION TEST**

SAMPLE DATA

GENERAL DATA	I	II	III
Chamber pressure (KPa)	650	750	900
Inner pressure (KPa)	600	600	600
Diameter (cm)	3.82	3.8	3.82
Heigth (cm)	7.72	7.82	7.68
Initial moisture (%)	13.10	0.00	12.47
Final moisture (%)	16.19	0.00	15.87
Dry Density (g/cm^3)	1.76	0	1.76



Gibraltar, 06/06/2018

TEST RESPONSIBLE:
F. Javier Manzano Diosdado
GEOLOGIST

LABORATORY HEADMASTER
Daniel Mª. Sotillo Sáez.
GEOLOGIST

TRIAXIAL COMPRESSION TEST

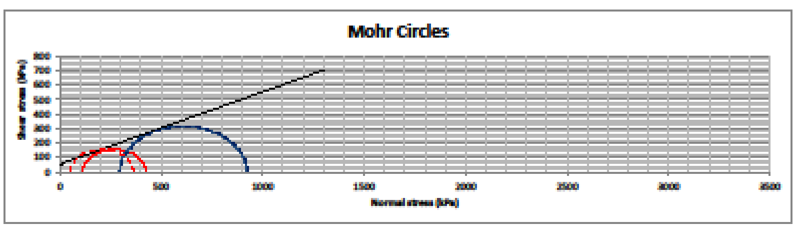
WORK:	0
APPLICANT:	0

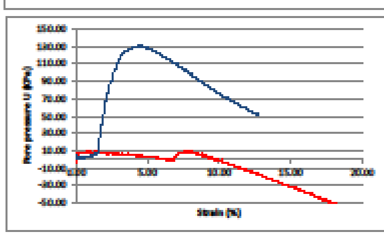
SAMPLE REFERENCE:	**SA-04-27-18**	**Sheet No:**		8.6
Sampling date:	26/03/2018	**Testing date:**	26/04/2018	
Sample location:	BH-2, between -9,00 & -9.30 m.	**Sample Type:**	Undisturbed	**Page:** 2/2
Test type:	CU			

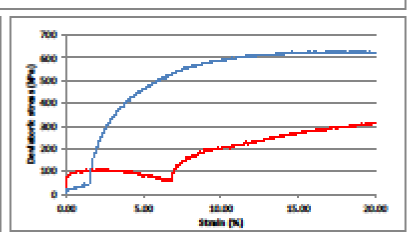
Tests:	**TRIAXIAL COMPRESSION TEST**

SAMPLE DATA

GENERAL DATA	I	II	III
Chamber pressure (KPa)	650		900
Inner pressure (KPa)	600		600
Diameter (cm)	3.81		3.82
Heigth (cm)	7.72		7.71
Initial moisture (%)	13.31		12.15
Final moisture (%)	16.58		15.11
Dry Density (g/cm³)	1.75		1.76
Cohession (KPa)	52		
Friction angle (°)	26.49		







Gibraltar, 17/05/2018

TEST RESPONSIBLE:
F. Javier Manzano Diosdado
GEOLOGIST

LABORATORY HEADMASTER
Daniel Mª. Sotillo Sáez.
GEOLOGIST

Sergeyco

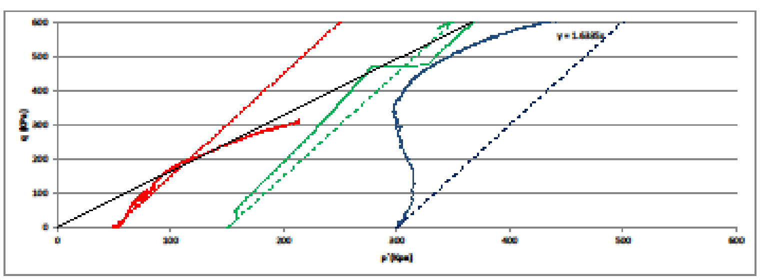
TRIAXIAL COMPRESSION TEST

WORK:	0
APPLICANT:	0

SAMPLE REFERENCE:	SA-04-27-18	Sheet No:		5.6
Sampling date:	26/03/2018	Testing date:	27/04/2018	
Sample location:	BH-2, between -9,00 & -9.30 m.	Sample Type:	Undisturbed	Page: 2/2
Test type:	CU			

Tests:	TRIAXIAL COMPRESSION TEST

SAMPLE DATA			
GENERAL DATA	I	II	III
Chamber pressure (KPa)	650	750	900
Inner pressure (KPa)	600	600	600
Diameter (cm)	3.81	3.8	3.82
Heigth (cm)	7.72	7.62	7.71
Initial moisture (%)	13.31	0.00	12.15
Final moisture (%)	16.58	0.00	15.11
Dry Density (g/cm^3)	1.75	0	1.78



Gibraltar, 17/05/2018

TEST RESPONSIBLE:
F. Javier Manzano Diosdado
GEOLOGIST

LABORATORY HEADMASTER
Daniel Mª. Sotillo Sáez.
GEOLOGIST

Appendix G: Parameters Optimization results

- -9m depth sample:

TEST				PLAXIS							
650		900		Morh - Coulomb (650)		Morh - Coulomb (900)		Hardening Soil (650)		Hardening Soil (900)	
Strain	**q**	**Strain**	**q**	**Strain**	**q**	**Strain**	**q**	**Strain**	**q**	**Strain**	**q**
0.01	0	0	0	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
0.00997	5.488	0	17.248	-2.00E-03	5.86E+01	-2.00E-03	3.39E+02	-2.00E-03	5.43E+01	-2.00E-03	9.62E+01
0.00998	2.352	0	19.796	-4.00E-03	5.86E+01	-4.00E-03	3.39E+02	-4.00E-03	5.41E+01	-4.00E-03	1.59E+02
0.01001	7.84	-0.00002	17.346	-6.00E-03	5.86E+01	-6.00E-03	3.39E+02	-6.00E-03	5.17E+01	-6.00E-03	1.99E+02
0.01	1.666	-0.00008	15.19	-8.00E-03	5.86E+01	-8.00E-03	3.39E+02	-8.00E-03	5.31E+01	-8.00E-03	2.26E+02
0.01001	5.88	-0.00017	13.328	-1.00E-02	5.86E+01	-1.00E-02	3.39E+02	-1.00E-02	5.53E+01	-1.00E-02	2.45E+02
0.00997	0.882	-0.00022	17.15	-1.20E-02	5.86E+01	-1.20E-02	3.39E+02	-1.20E-02	5.76E+01	-1.20E-02	2.58E+02
0.01002	7.644	-0.00031	15.092	-1.40E-02	5.86E+01	-1.40E-02	3.39E+02	-1.40E-02	6.00E+01	-1.40E-02	2.69E+02
0.00999	5.782	-0.00036	16.954	-1.60E-02	5.86E+01	-1.60E-02	3.39E+02	-1.60E-02	6.25E+01	-1.60E-02	2.77E+02
0.01	6.86	-0.00047	19.012	-1.80E-02	5.86E+01	-1.80E-02	3.39E+02	-1.80E-02	6.51E+01	-1.80E-02	2.84E+02
0.00999	4.606	-0.00053	14.896	-2.00E-02	5.86E+01	-2.00E-02	3.39E+02	-2.00E-02	6.77E+01	-2.00E-02	2.89E+02
0.01	11.074	-0.00066	85.848	-2.20E-02	5.86E+01	-2.20E-02	3.39E+02	-2.20E-02	7.05E+01	-2.20E-02	2.93E+02

0.01	15.974	-0.00073	83.496	-2.40E-02	5.86E+01	-2.40E-02	3.39E+02	-2.40E-02	7.33E+01	-2.40E-02	2.97E+02
0.00999	15.974	-0.00079	97.804	-2.60E-02	5.86E+01	-2.60E-02	3.39E+02	-2.60E-02	7.62E+01	-2.60E-02	3.00E+02
0.00998	19.894	-0.00082	105.35	-2.80E-02	5.86E+01	-2.80E-02	3.39E+02	-2.80E-02	7.92E+01	-2.80E-02	3.03E+02
0.01	27.832	-0.00093	128.87	-3.00E-02	5.86E+01	-3.00E-02	3.39E+02	-3.00E-02	8.22E+01	-3.00E-02	3.05E+02
0.00999	29.89	-0.00102	124.852	-3.20E-02	5.86E+01	-3.20E-02	3.39E+02	-3.20E-02	8.53E+01	-3.20E-02	3.07E+02
0.01	33.124	-0.00116	130.242	-3.40E-02	5.86E+01	-3.40E-02	3.39E+02	-3.40E-02	8.85E+01	-3.40E-02	3.09E+02
0.01002	38.808	-0.00127	143.472	-3.60E-02	5.86E+01	-3.60E-02	3.39E+02	-3.60E-02	9.18E+01	-3.60E-02	3.11E+02
0.01	33.418	-0.00133	161.21	-3.80E-02	5.86E+01	-3.80E-02	3.39E+02	-3.80E-02	9.50E+01	-3.80E-02	3.12E+02
0.01001	44.002	-0.00151	162.68	-4.00E-02	5.86E+01	-4.00E-02	3.39E+02	-4.00E-02	9.84E+01	-4.00E-02	3.13E+02
0.01002	46.844	-0.00177	170.52	-4.20E-02	5.86E+01	-4.20E-02	3.39E+02	-4.20E-02	1.02E+02	-4.20E-02	3.15E+02
0.01	44.1	-0.00184	172.97	-4.40E-02	5.86E+01	-4.40E-02	3.39E+02	-4.40E-02	1.05E+02	-4.40E-02	3.16E+02
0.01	48.706	-0.00219	181.202	-4.60E-02	5.86E+01	-4.60E-02	3.39E+02	-4.60E-02	1.08E+02	-4.60E-02	3.17E+02
0.00998	46.746	-0.00257	184.436	-4.80E-02	5.86E+01	-4.80E-02	3.39E+02	-4.80E-02	1.12E+02	-4.80E-02	3.17E+02
0.01	53.508	-0.0028	198.352	-5.00E-02	5.86E+01	-5.00E-02	3.39E+02	-5.00E-02	1.15E+02	-5.00E-02	3.18E+02
0.00999	57.232	-0.00317	201.096	-5.20E-02	5.86E+01	-5.20E-02	3.39E+02	-5.20E-02	1.18E+02	-5.20E-02	3.19E+02

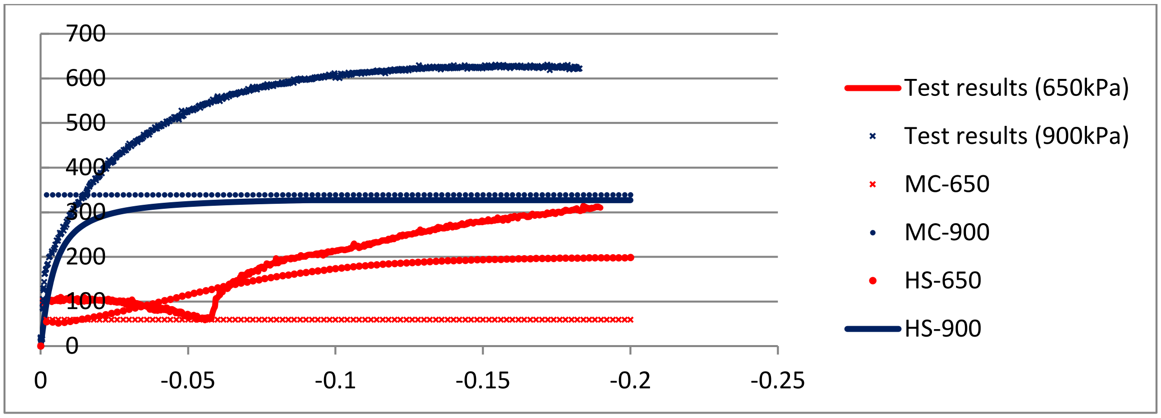
0.0099	60.956	-0.00378	208.054	-5.40E-02	5.86E+01	-5.40E-02	3.39E+02	-5.40E-02	1.21E+02	-5.40E-02	3.20E+02
0.00983	59.094	-0.00386	212.464	-5.60E-02	5.86E+01	-5.60E-02	3.39E+02	-5.60E-02	1.24E+02	-5.60E-02	3.20E+02
0.00974	61.446	-0.00446	215.894	-5.80E-02	5.86E+01	-5.80E-02	3.39E+02	-5.80E-02	1.27E+02	-5.80E-02	3.21E+02
0.00969	68.992	-0.0048	219.814	-6.00E-02	5.86E+01	-6.00E-02	3.39E+02	-6.00E-02	1.30E+02	-6.00E-02	3.22E+02
0.00963	70.756	-0.00498	227.36	-6.20E-02	5.86E+01	-6.20E-02	3.39E+02	-6.20E-02	1.33E+02	-6.20E-02	3.22E+02
0.00953	70.854	-0.00541	236.768	-6.40E-02	5.86E+01	-6.40E-02	3.39E+02	-6.40E-02	1.36E+02	-6.40E-02	3.23E+02
0.00943	70.266	-0.00583	235.886	-6.60E-02	5.86E+01	-6.60E-02	3.39E+02	-6.60E-02	1.39E+02	-6.60E-02	3.23E+02
0.00932	72.52	-0.00604	242.354	-6.80E-02	5.86E+01	-6.80E-02	3.39E+02	-6.80E-02	1.41E+02	-6.80E-02	3.23E+02
0.00923	76.244	-0.00656	253.82	-7.00E-02	5.86E+01	-7.00E-02	3.39E+02	-7.00E-02	1.44E+02	-7.00E-02	3.24E+02
0.00924	76.244	-0.00678	250.39	-7.20E-02	5.86E+01	-7.20E-02	3.39E+02	-7.20E-02	1.46E+02	-7.20E-02	3.24E+02
0.00915	78.204	-0.00703	253.232	-7.40E-02	5.86E+01	-7.40E-02	3.39E+02	-7.40E-02	1.49E+02	-7.40E-02	3.25E+02
0.00904	80.164	-0.00779	258.916	-7.60E-02	5.86E+01	-7.60E-02	3.39E+02	-7.60E-02	1.51E+02	-7.60E-02	3.25E+02
0.00895	81.438	-0.00765	267.442	-7.80E-02	5.86E+01	-7.80E-02	3.39E+02	-7.80E-02	1.53E+02	-7.80E-02	3.25E+02
0.00892	86.73	-0.00837	266.56	-8.00E-02	5.86E+01	-8.00E-02	3.39E+02	-8.00E-02	1.55E+02	-8.00E-02	3.26E+02
0.00881	77.224	-0.0088	271.852	-8.20E-02	5.86E+01	-8.20E-02	3.39E+02	-8.20E-02	1.57E+02	-8.20E-02	3.26E+02

0.0087	81.928	-0.00884	279.104	-8.40E-02	5.86E+01	-8.40E-02	3.39E+02	-8.40E-02	1.60E+02	-8.40E-02	3.26E+02
0.00859	83.594	-0.00926	282.828	-8.60E-02	5.86E+01	-8.60E-02	3.39E+02	-8.60E-02	1.61E+02	-8.60E-02	3.26E+02
0.00854	88.102	-0.00984	293.51	-8.80E-02	5.86E+01	-8.80E-02	3.39E+02	-8.80E-02	1.63E+02	-8.80E-02	3.27E+02
0.00818	83.202	-0.0101	295.666	-9.00E-02	5.86E+01	-9.00E-02	3.39E+02	-9.00E-02	1.65E+02	-9.00E-02	3.27E+02
0.00804	86.73	-0.01078	291.844	-9.20E-02	5.86E+01	-9.20E-02	3.39E+02	-9.20E-02	1.67E+02	-9.20E-02	3.27E+02
0.00747	82.81	-0.01051	291.06	-9.40E-02	5.86E+01	-9.40E-02	3.39E+02	-9.40E-02	1.69E+02	-9.40E-02	3.27E+02
0.00723	88.494	-0.0109	302.722	-9.60E-02	5.86E+01	-9.60E-02	3.39E+02	-9.60E-02	1.70E+02	-9.60E-02	3.27E+02
0.00728	91.042	-0.01122	308.406	-9.80E-02	5.86E+01	-9.80E-02	3.39E+02	-9.80E-02	1.72E+02	-9.80E-02	3.27E+02
0.00671	93.884	-0.01147	307.034	-1.00E-01	5.86E+01	-1.00E-01	3.39E+02	-1.00E-01	1.73E+02	-1.00E-01	3.27E+02
0.00618	90.16	-0.01187	316.246	-1.02E-01	5.86E+01	-1.02E-01	3.39E+02	-1.02E-01	1.75E+02	-1.02E-01	3.27E+02
0.006	94.962	-0.0122	316.736	-1.04E-01	5.86E+01	-1.04E-01	3.39E+02	-1.04E-01	1.76E+02	-1.04E-01	3.27E+02
0.00534	95.06	-0.01279	318.598	-1.06E-01	5.86E+01	-1.06E-01	3.39E+02	-1.06E-01	1.78E+02	-1.06E-01	3.27E+02
0.00521	91.042	-0.01271	322.518	-1.08E-01	5.86E+01	-1.08E-01	3.39E+02	-1.08E-01	1.79E+02	-1.08E-01	3.27E+02
0.00474	93.688	-0.01318	323.106	-1.10E-01	5.86E+01	-1.10E-01	3.39E+02	-1.10E-01	1.80E+02	-1.10E-01	3.27E+02
0.00421	92.022	-0.01386	330.652	-1.12E-01	5.86E+01	-1.12E-01	3.39E+02	-1.12E-01	1.81E+02	-1.12E-01	3.27E+02

0.00426	91.728	-0.01397	335.65	-1.14E-01	5.86E+01	-1.14E-01	3.39E+02	-1.14E-01	1.82E+02	-1.14E-01	3.27E+02
0.00383	94.57	-0.01444	333.2	-1.16E-01	5.86E+01	-1.16E-01	3.39E+02	-1.16E-01	1.83E+02	-1.16E-01	3.27E+02
0.00315	96.236	-0.0148	333.494	-1.18E-01	5.86E+01	-1.18E-01	3.39E+02	-1.18E-01	1.84E+02	-1.18E-01	3.27E+02
0.00325	100.45	-0.01504	340.354	-1.20E-01	5.86E+01	-1.20E-01	3.39E+02	-1.20E-01	1.85E+02	-1.20E-01	3.27E+02
0.00267	100.156	-0.01541	337.022	-1.22E-01	5.86E+01	-1.22E-01	3.39E+02	-1.22E-01	1.86E+02	-1.22E-01	3.27E+02
0.00216	100.254	-0.01555	344.666	-1.24E-01	5.86E+01	-1.24E-01	3.39E+02	-1.24E-01	1.87E+02	-1.24E-01	3.27E+02
0.00202	102.312	-0.01578	348.194	-1.26E-01	5.86E+01	-1.26E-01	3.39E+02	-1.26E-01	1.87E+02	-1.26E-01	3.27E+02
0.0014	100.058	-0.01601	351.624	-1.28E-01	5.86E+01	-1.28E-01	3.39E+02	-1.28E-01	1.88E+02	-1.28E-01	3.27E+02
0.00122	96.628	-0.01655	356.916	-1.30E-01	5.86E+01	-1.30E-01	3.39E+02	-1.30E-01	1.89E+02	-1.30E-01	3.27E+02
0.00101	99.666	-0.01648	358.19	-1.32E-01	5.86E+01	-1.32E-01	3.39E+02	-1.32E-01	1.89E+02	-1.32E-01	3.27E+02
0.00033	99.176	-0.01683	361.718	-1.34E-01	5.86E+01	-1.34E-01	3.39E+02	-1.34E-01	1.90E+02	-1.34E-01	3.27E+02
0.00018	101.528	-0.01709	364.658	-1.36E-01	5.86E+01	-1.36E-01	3.39E+02	-1.36E-01	1.91E+02	-1.36E-01	3.27E+02
-0.00056	100.352	-0.01758	367.402	-1.38E-01	5.86E+01	-1.38E-01	3.39E+02	-1.38E-01	1.91E+02	-1.38E-01	3.27E+02
-0.00078	96.824	-0.01777	366.814	-1.40E-01	5.86E+01	-1.40E-01	3.39E+02	-1.40E-01	1.92E+02	-1.40E-01	3.27E+02
-0.00073	99.274	-0.01798	367.696	-1.42E-01	5.86E+01	-1.42E-01	3.39E+02	-1.42E-01	1.92E+02	-1.42E-01	3.27E+02

-0.00128	101.332	-0.01846	375.634	-1.44E-01	5.86E+01	-1.44E-01	3.39E+02	-1.44E-01	1.92E+02	-1.44E-01	3.27E+02
-0.0018	100.646	-0.01878	371.42	-1.46E-01	5.86E+01	-1.46E-01	3.39E+02	-1.46E-01	1.93E+02	-1.46E-01	3.27E+02
-0.00192	101.234	-0.01879	375.634	-1.48E-01	5.86E+01	-1.48E-01	3.39E+02	-1.48E-01	1.93E+02	-1.48E-01	3.27E+02
-0.00246	106.232	-0.01941	377.692	-1.50E-01	5.86E+01	-1.50E-01	3.39E+02	-1.50E-01	1.94E+02	-1.50E-01	3.27E+02
-0.00279	102.606	-0.01977	379.75	-1.52E-01	5.86E+01	-1.52E-01	3.39E+02	-1.52E-01	1.94E+02	-1.52E-01	3.27E+02
-0.0031	101.43	-0.02	382.2	-1.54E-01	5.86E+01	-1.54E-01	3.39E+02	-1.54E-01	1.94E+02	-1.54E-01	3.27E+02
-0.00361	103.978	-0.02015	383.572	-1.56E-01	5.86E+01	-1.56E-01	3.39E+02	-1.56E-01	1.95E+02	-1.56E-01	3.27E+02
-0.00378	98.588	-0.02035	388.276	-1.58E-01	5.86E+01	-1.58E-01	3.39E+02	-1.58E-01	1.95E+02	-1.58E-01	3.27E+02
-0.00414	103.39	-0.02077	387.1	-1.60E-01	5.86E+01	-1.60E-01	3.39E+02	-1.60E-01	1.95E+02	-1.60E-01	3.27E+02
-0.00482	100.94	-0.02114	396.018	-1.62E-01	5.86E+01	-1.62E-01	3.39E+02	-1.62E-01	1.95E+02	-1.62E-01	3.27E+02
-0.00457	101.92	-0.02153	398.37	-1.64E-01	5.86E+01	-1.64E-01	3.39E+02	-1.64E-01	1.96E+02	-1.64E-01	3.27E+02
-0.0052	104.86	-0.02179	400.33	-1.66E-01	5.86E+01	-1.66E-01	3.39E+02	-1.66E-01	1.96E+02	-1.66E-01	3.27E+02
-0.00579	100.94	-0.02227	397.39	-1.68E-01	5.86E+01	-1.68E-01	3.39E+02	-1.68E-01	1.96E+02	-1.68E-01	3.27E+02
-0.00578	107.604	-0.0226	405.132	-1.70E-01	5.86E+01	-1.70E-01	3.39E+02	-1.70E-01	1.96E+02	-1.70E-01	3.27E+02
-0.00619	103.978	-0.02268	404.446	-1.72E-01	5.86E+01	-1.72E-01	3.39E+02	-1.72E-01	1.97E+02	-1.72E-01	3.27E+02

-0.00677	100.156	-0.02279	408.954	-1.74E-01	5.86E+01	-1.74E-01	3.39E+02	-1.74E-01	1.97E+02	-1.74E-01	3.27E+02
-0.00677	110.152	-0.02312	416.206	-1.76E-01	5.86E+01	-1.76E-01	3.39E+02	-1.76E-01	1.97E+02	-1.76E-01	3.27E+02
-0.00744	105.644	-0.0238	410.718	-1.78E-01	5.86E+01	-1.78E-01	3.39E+02	-1.78E-01	1.97E+02	-1.78E-01	3.27E+02
-0.00779	100.744	-0.02389	410.718	-1.80E-01	5.86E+01	-1.80E-01	3.39E+02	-1.80E-01	1.97E+02	-1.80E-01	3.27E+02
-0.00798	105.546	-0.02435	411.992	-1.82E-01	5.86E+01	-1.82E-01	3.39E+02	-1.82E-01	1.97E+02	-1.82E-01	3.27E+02
-0.00862	104.272	-0.0248	419.048	-1.84E-01	5.86E+01	-1.84E-01	3.39E+02	-1.84E-01	1.97E+02	-1.84E-01	3.27E+02
-0.0088	103.978	-0.0248	414.442	-1.86E-01	5.86E+01	-1.86E-01	3.39E+02	-1.86E-01	1.98E+02	-1.86E-01	3.27E+02
-0.00891	107.506	-0.02537	422.772	-1.88E-01	5.86E+01	-1.88E-01	3.39E+02	-1.88E-01	1.98E+02	-1.88E-01	3.27E+02
-0.00983	101.92	-0.02548	421.89	-1.90E-01	5.86E+01	-1.90E-01	3.39E+02	-1.90E-01	1.98E+02	-1.90E-01	3.27E+02
-0.00985	104.958	-0.02578	423.556	-1.92E-01	5.86E+01	-1.92E-01	3.39E+02	-1.92E-01	1.98E+02	-1.92E-01	3.27E+02
-0.01007	111.034	-0.02577	426.79	-1.94E-01	5.86E+01	-1.94E-01	3.39E+02	-1.94E-01	1.98E+02	-1.94E-01	3.27E+02
-0.0108	103.292	-0.02626	428.456	-1.96E-01	5.86E+01	-1.96E-01	3.39E+02	-1.96E-01	1.98E+02	-1.96E-01	3.27E+02
-0.01093	103.978	-0.0268	430.22	-1.98E-01	5.86E+01	-1.98E-01	3.39E+02	-1.98E-01	1.98E+02	-1.98E-01	3.27E+02
-0.01133	104.958	-0.02689	432.18	-2.00E-01	5.86E+01	-2.00E-01	3.39E+02	-2.00E-01	1.98E+02	-2.00E-01	3.27E+02



Optimized parameters		
E50	1.60E+04	kN/m2
Eoed	1.70E+04	kN/m2
Eur	2.00E+05	kN/m2
v	2.20E-01	
c´	5.00E+01	kN/m2
phi´	2.80E+01	º

❖ -7m depth sample:

TEST				**PLAXIS**							
650		**900**		**Morh - Coulomb (650)**		**Morh - Coulomb (900)**		**Hardening Soil (650)**		**Hardening Soil (900)**	
Strain	**q**	**Strain**	**q**	**Strain**	**q**	**Strain**	**q**	**Strain**	**q**	**Strain**	**q**
0.00119	0	0	0	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00	0.00E+00
-0.00002	24.01	0	43.512	-2.00E-03	3.43E+01	-2.00E-03	3.43E+01	-2.00E-03	9.30E+01	-2.00E-03	2.12E+02
-0.00003	22.638	0.00001	58.016	-4.00E-03	6.86E+01	-4.00E-03	6.86E+01	-4.00E-03	1.47E+02	-4.00E-03	3.68E+02
0.00001	21.364	-0.00003	65.954	-6.00E-03	1.03E+02	-6.00E-03	1.03E+02	-6.00E-03	1.83E+02	-6.00E-03	4.87E+02
-	28.8	-	65.3	-	1.37E	-	1.37E	-	2.08E	-	5.82E

0.00002	12	0.00001	66	8.00E-03	+02	8.00E-03	+02	8.00E-03	+02	8.00E-03	+02
-0.00003	27.146	0	71.54	-1.00E-02	1.45E+02	-1.00E-02	1.72E+02	-1.00E-02	2.26E+02	-1.00E-02	6.58E+02
-0.00003	23.814	-0.00001	71.246	-1.20E-02	1.45E+02	-1.20E-02	2.06E+02	-1.20E-02	2.40E+02	-1.20E-02	7.20E+02
-0.00002	24.01	-0.00001	71.834	-1.40E-02	1.45E+02	-1.40E-02	2.40E+02	-1.40E-02	2.51E+02	-1.40E-02	7.73E+02
-0.00001	27.93	-0.00002	75.852	-1.60E-02	1.45E+02	-1.60E-02	2.74E+02	-1.60E-02	2.60E+02	-1.60E-02	8.17E+02
0.00001	22.736	-0.00002	86.436	-1.80E-02	1.45E+02	-1.80E-02	3.09E+02	-1.80E-02	2.67E+02	-1.80E-02	8.55E+02
-0.00002	24.99	0.00001	76.244	-2.00E-02	1.45E+02	-2.00E-02	3.43E+02	-2.00E-02	2.73E+02	-2.00E-02	8.88E+02
-0.00003	34.888	-0.00001	81.242	-2.20E-02	1.45E+02	-2.20E-02	3.77E+02	-2.20E-02	2.78E+02	-2.20E-02	9.17E+02
-0.00002	28.028	-0.00002	79.478	-2.40E-02	1.45E+02	-2.40E-02	4.12E+02	-2.40E-02	2.83E+02	-2.40E-02	9.42E+02
0.00002	29.4	-0.00008	79.184	-2.60E-02	1.45E+02	-2.60E-02	4.46E+02	-2.60E-02	2.87E+02	-2.60E-02	9.65E+02
-0.00001	26.656	0.00001	80.458	-2.80E-02	1.45E+02	-2.80E-02	4.80E+02	-2.80E-02	2.90E+02	-2.80E-02	9.85E+02
0.00002	35.574	0.00002	87.024	-3.00E-02	1.45E+02	-3.00E-02	5.15E+02	-3.00E-02	2.93E+02	-3.00E-02	1.00E+03
0.00001	28.616	0	89.572	-3.20E-02	1.45E+02	-3.20E-02	5.49E+02	-3.20E-02	2.96E+02	-3.20E-02	1.02E+03
0.00002	32.536	0.00001	82.712	-3.40E-02	1.45E+02	-3.40E-02	5.58E+02	-3.40E-02	2.98E+02	-3.40E-02	1.03E+03
-0.00001	28.224	-0.00001	92.316	-3.60E-02	1.45E+02	-3.60E-02	5.58E+02	-3.60E-02	3.01E+02	-3.60E-02	1.05E+03
0.000	33.6	-	101.	-	1.45E	-	5.58E	-	3.03E	-	1.06E

03	14	0.00003	332	3.80E-02	+02	3.80E-02	+02	3.80E-02	+02	3.80E-02	+03
-0.00003	34.104	0.00004	96.138	-4.00E-02	1.45E+02	-4.00E-02	5.58E+02	-4.00E-02	3.04E+02	-4.00E-02	1.07E+03
0.00002	32.83	-0.00001	102.018	-4.20E-02	1.45E+02	-4.20E-02	5.58E+02	-4.20E-02	3.06E+02	-4.20E-02	1.08E+03
0.00002	26.166	-0.00002	100.352	-4.40E-02	1.45E+02	-4.40E-02	5.58E+02	-4.40E-02	3.08E+02	-4.40E-02	1.09E+03
-0.00001	35.966	0.00001	103.782	-4.60E-02	1.45E+02	-4.60E-02	5.58E+02	-4.60E-02	3.09E+02	-4.60E-02	1.10E+03
-0.00005	31.458	-0.00001	98.882	-4.80E-02	1.45E+02	-4.80E-02	5.58E+02	-4.80E-02	3.09E+02	-4.80E-02	1.11E+03
-0.00003	33.026	-0.00004	107.604	-5.00E-02	1.45E+02	-5.00E-02	5.58E+02	-5.00E-02	3.09E+02	-5.00E-02	1.12E+03
-0.00003	27.146	-0.00001	130.046	-5.20E-02	1.45E+02	-5.20E-02	5.58E+02	-5.20E-02	3.09E+02	-5.20E-02	1.12E+03
-0.00001	28.812	-0.00004	138.768	-5.40E-02	1.45E+02	-5.40E-02	5.58E+02	-5.40E-02	3.09E+02	-5.40E-02	1.13E+03
-0.00009	25.97	-0.00011	164.64	-5.60E-02	1.45E+02	-5.60E-02	5.58E+02	-5.60E-02	3.09E+02	-5.60E-02	1.14E+03
-0.00019	23.226	-0.00015	171.696	-5.80E-02	1.45E+02	-5.80E-02	5.58E+02	-5.80E-02	3.09E+02	-5.80E-02	1.14E+03
-0.00024	29.106	-0.00022	177.478	-6.00E-02	1.45E+02	-6.00E-02	5.58E+02	-6.00E-02	3.09E+02	-6.00E-02	1.15E+03
-0.00031	33.32	-0.00025	194.432	-6.20E-02	1.45E+02	-6.20E-02	5.58E+02	-6.20E-02	3.09E+02	-6.20E-02	1.16E+03
-0.00038	31.164	-0.0003	214.718	-6.40E-02	1.45E+02	-6.40E-02	5.58E+02	-6.40E-02	3.09E+02	-6.40E-02	1.16E+03
-0.00046	25.872	-0.00035	208.936	-6.60E-02	1.45E+02	-6.60E-02	5.58E+02	-6.60E-02	3.09E+02	-6.60E-02	1.17E+03
-	26.2	-	218.	-	1.45E	-	5.58E	-	3.09E	-	1.17E

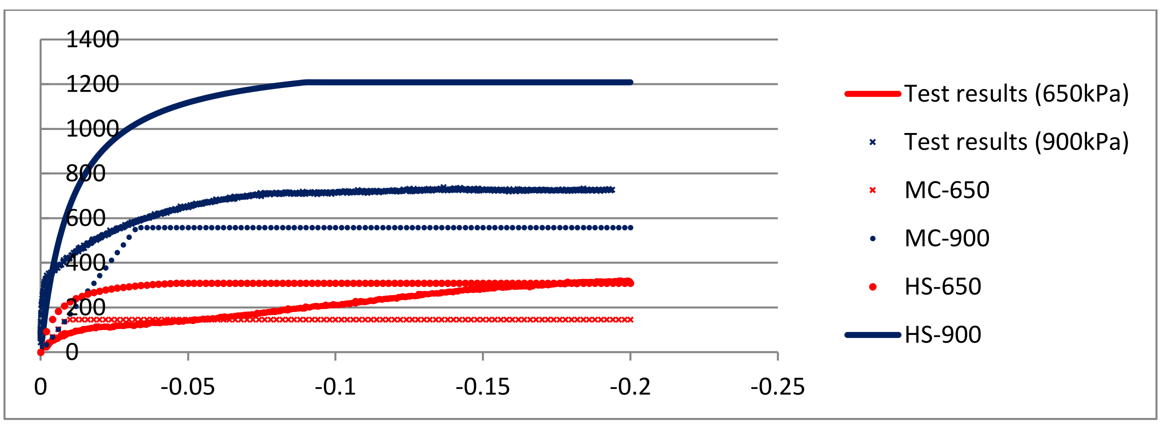
0.00059	64	0.00044	148	6.80E-02	+02	6.80E-02	+02	6.80E-02	+02	6.80E-02	+03
-0.00069	27.048	-0.00048	227.36	-7.00E-02	1.45E+02	-7.00E-02	5.58E+02	-7.00E-02	3.09E+02	-7.00E-02	1.17E+03
-0.0008	30.772	-0.0006	240.492	-7.20E-02	1.45E+02	-7.20E-02	5.58E+02	-7.20E-02	3.09E+02	-7.20E-02	1.18E+03
-0.00073	27.146	-0.00068	257.642	-7.40E-02	1.45E+02	-7.40E-02	5.58E+02	-7.40E-02	3.09E+02	-7.40E-02	1.18E+03
-0.00087	26.95	-0.00069	254.114	-7.60E-02	1.45E+02	-7.60E-02	5.58E+02	-7.60E-02	3.09E+02	-7.60E-02	1.19E+03
-0.00097	28.714	-0.00077	256.956	-7.80E-02	1.45E+02	-7.80E-02	5.58E+02	-7.80E-02	3.09E+02	-7.80E-02	1.19E+03
-0.00103	28.714	-0.00077	266.07	-8.00E-02	1.45E+02	-8.00E-02	5.58E+02	-8.00E-02	3.09E+02	-8.00E-02	1.19E+03
-0.00119	23.814	-0.00087	271.166	-8.20E-02	1.45E+02	-8.20E-02	5.58E+02	-8.20E-02	3.09E+02	-8.20E-02	1.20E+03
-0.00126	24.402	-0.00103	275.282	-8.40E-02	1.45E+02	-8.40E-02	5.58E+02	-8.40E-02	3.09E+02	-8.40E-02	1.20E+03
-0.00147	20.384	-0.00108	291.354	-8.60E-02	1.45E+02	-8.60E-02	5.58E+02	-8.60E-02	3.09E+02	-8.60E-02	1.20E+03
-0.00157	23.52	-0.00111	304.094	-8.80E-02	1.45E+02	-8.80E-02	5.58E+02	-8.80E-02	3.09E+02	-8.80E-02	1.21E+03
-0.0018	23.128	-0.00124	299.488	-9.00E-02	1.45E+02	-9.00E-02	5.58E+02	-9.00E-02	3.09E+02	-9.00E-02	1.21E+03
-0.00197	25.97	-0.00133	314.482	-9.20E-02	1.45E+02	-9.20E-02	5.58E+02	-9.20E-02	3.09E+02	-9.20E-02	1.21E+03
-0.00209	21.07	-0.00145	312.914	-9.40E-02	1.45E+02	-9.40E-02	5.58E+02	-9.40E-02	3.09E+02	-9.40E-02	1.21E+03
-0.00281	33.614	-0.00167	322.91	-9.60E-02	1.45E+02	-9.60E-02	5.58E+02	-9.60E-02	3.09E+02	-9.60E-02	1.21E+03
-	37.3	-	328.	-	1.45E	-	5.58E	-	3.09E	-	1.21E

0.00278	38	0.00181	692	9.80E-02	+02	9.80E-02	+02	9.80E-02	+02	9.80E-02	+03
-0.00262	43.806	-0.00201	331.24	-1.00E-01	1.45E+02	-1.00E-01	5.58E+02	-1.00E-01	3.09E+02	-1.00E-01	1.21E+03
-0.00326	42.532	-0.00205	342.804	-1.02E-01	1.45E+02	-1.02E-01	5.58E+02	-1.02E-01	3.09E+02	-1.02E-01	1.21E+03
-0.00379	46.942	-0.00242	339.668	-1.04E-01	1.45E+02	-1.04E-01	5.58E+02	-1.04E-01	3.09E+02	-1.04E-01	1.21E+03
-0.00385	55.174	-0.00277	333.69	-1.06E-01	1.45E+02	-1.06E-01	5.58E+02	-1.06E-01	3.09E+02	-1.06E-01	1.21E+03
-0.00459	51.352	-0.00271	351.918	-1.08E-01	1.45E+02	-1.08E-01	5.58E+02	-1.08E-01	3.09E+02	-1.08E-01	1.21E+03
-0.00482	58.016	-0.00337	356.426	-1.10E-01	1.45E+02	-1.10E-01	5.58E+02	-1.10E-01	3.09E+02	-1.10E-01	1.21E+03
-0.0053	62.426	-0.00385	354.662	-1.12E-01	1.45E+02	-1.12E-01	5.58E+02	-1.12E-01	3.09E+02	-1.12E-01	1.21E+03
-0.00583	58.702	-0.0041	364.462	-1.14E-01	1.45E+02	-1.14E-01	5.58E+02	-1.14E-01	3.09E+02	-1.14E-01	1.21E+03
-0.00544	65.562	-0.0048	366.716	-1.16E-01	1.45E+02	-1.16E-01	5.58E+02	-1.16E-01	3.09E+02	-1.16E-01	1.21E+03
-0.00602	64.484	-0.00475	369.852	-1.18E-01	1.45E+02	-1.18E-01	5.58E+02	-1.18E-01	3.09E+02	-1.18E-01	1.21E+03
-0.00683	68.894	-0.0051	383.572	-1.20E-01	1.45E+02	-1.20E-01	5.58E+02	-1.20E-01	3.09E+02	-1.20E-01	1.21E+03
-0.00684	77.126	-0.00508	377.006	-1.22E-01	1.45E+02	-1.22E-01	5.58E+02	-1.22E-01	3.09E+02	-1.22E-01	1.21E+03
-0.00739	72.618	-0.00582	377.398	-1.24E-01	1.45E+02	-1.24E-01	5.58E+02	-1.24E-01	3.09E+02	-1.24E-01	1.21E+03
-0.00788	70.56	-0.00561	379.456	-1.26E-01	1.45E+02	-1.26E-01	5.58E+02	-1.26E-01	3.09E+02	-1.26E-01	1.21E+03
-	81.3	-	387.	-	1.45E	-	5.58E	-	3.09E	-	1.21E

0.00776	4	0.00627	884	1.28E-01	+02	1.28E-01	+02	1.28E-01	+02	1.28E-01	+03
-0.00814	85.652	-0.0068	390.824	-1.30E-01	1.45E+02	-1.30E-01	5.58E+02	-1.30E-01	3.09E+02	-1.30E-01	1.21E+03
-0.00857	76.146	-0.0071	392.392	-1.32E-01	1.45E+02	-1.32E-01	5.58E+02	-1.32E-01	3.09E+02	-1.32E-01	1.21E+03
-0.00879	82.81	-0.00771	402.682	-1.34E-01	1.45E+02	-1.34E-01	5.58E+02	-1.34E-01	3.09E+02	-1.34E-01	1.21E+03
-0.00887	84.574	-0.00777	406.308	-1.36E-01	1.45E+02	-1.36E-01	5.58E+02	-1.36E-01	3.09E+02	-1.36E-01	1.21E+03
-0.00954	83.202	-0.0078	411.796	-1.38E-01	1.45E+02	-1.38E-01	5.58E+02	-1.38E-01	3.09E+02	-1.38E-01	1.21E+03
-0.00987	85.848	-0.00878	417.774	-1.40E-01	1.45E+02	-1.40E-01	5.58E+02	-1.40E-01	3.09E+02	-1.40E-01	1.21E+03
-0.01033	84.28	-0.00858	417.48	-1.42E-01	1.45E+02	-1.42E-01	5.58E+02	-1.42E-01	3.09E+02	-1.42E-01	1.21E+03
-0.01079	88.004	-0.00891	424.144	-1.44E-01	1.45E+02	-1.44E-01	5.58E+02	-1.44E-01	3.09E+02	-1.44E-01	1.21E+03
-0.0106	88.69	-0.00926	418.656	-1.46E-01	1.45E+02	-1.46E-01	5.58E+02	-1.46E-01	3.09E+02	-1.46E-01	1.21E+03
-0.01123	89.082	-0.0098	418.95	-1.48E-01	1.45E+02	-1.48E-01	5.58E+02	-1.48E-01	3.09E+02	-1.48E-01	1.21E+03
-0.01181	96.628	-0.00979	424.144	-1.50E-01	1.45E+02	-1.50E-01	5.58E+02	-1.50E-01	3.09E+02	-1.50E-01	1.21E+03
-0.01183	91.14	-0.01044	433.16	-1.52E-01	1.45E+02	-1.52E-01	5.58E+02	-1.52E-01	3.09E+02	-1.52E-01	1.21E+03
-0.01232	95.648	-0.01082	433.65	-1.54E-01	1.45E+02	-1.54E-01	5.58E+02	-1.54E-01	3.09E+02	-1.54E-01	1.21E+03
-0.01283	93.688	-0.01076	435.316	-1.56E-01	1.45E+02	-1.56E-01	5.58E+02	-1.56E-01	3.09E+02	-1.56E-01	1.21E+03
-	96.2	-	445.	-	1.45E	-	5.58E	-	3.09E	-	1.21E

0.01274	36	0.01122	312	1.58E-01	+02	1.58E-01	+02	1.58E-01	+02	1.58E-01	+03
-0.01355	97.51	-0.01178	446.096	-1.60E-01	1.45E+02	-1.60E-01	5.58E+02	-1.60E-01	3.09E+02	-1.60E-01	1.21E+03
-0.01383	95.06	-0.01203	452.466	-1.62E-01	1.45E+02	-1.62E-01	5.58E+02	-1.62E-01	3.09E+02	-1.62E-01	1.21E+03
-0.01403	100.744	-0.01277	456.876	-1.64E-01	1.45E+02	-1.64E-01	5.58E+02	-1.64E-01	3.09E+02	-1.64E-01	1.21E+03
-0.01452	103.782	-0.01286	450.898	-1.66E-01	1.45E+02	-1.66E-01	5.58E+02	-1.66E-01	3.09E+02	-1.66E-01	1.21E+03
-0.01478	99.764	-0.01297	465.402	-1.68E-01	1.45E+02	-1.68E-01	5.58E+02	-1.68E-01	3.09E+02	-1.68E-01	1.21E+03
-0.01485	104.272	-0.01304	459.032	-1.70E-01	1.45E+02	-1.70E-01	5.58E+02	-1.70E-01	3.09E+02	-1.70E-01	1.21E+03
-0.01579	102.9	-0.0138	462.658	-1.72E-01	1.45E+02	-1.72E-01	5.58E+02	-1.72E-01	3.09E+02	-1.72E-01	1.21E+03
-0.01572	104.86	-0.01344	469.028	-1.74E-01	1.45E+02	-1.74E-01	5.58E+02	-1.74E-01	3.09E+02	-1.74E-01	1.21E+03
-0.01602	105.84	-0.01433	466.872	-1.76E-01	1.45E+02	-1.76E-01	5.58E+02	-1.76E-01	3.09E+02	-1.76E-01	1.21E+03
-0.01683	106.722	-0.0148	466.97	-1.78E-01	1.45E+02	-1.78E-01	5.58E+02	-1.78E-01	3.09E+02	-1.78E-01	1.21E+03
-0.01673	104.076	-0.01469	478.436	-1.80E-01	1.45E+02	-1.80E-01	5.58E+02	-1.80E-01	3.09E+02	-1.80E-01	1.21E+03
-0.01718	109.074	-0.01542	477.946	-1.82E-01	1.45E+02	-1.82E-01	5.58E+02	-1.82E-01	3.09E+02	-1.82E-01	1.21E+03
-0.01781	107.506	-0.01586	480.592	-1.84E-01	1.45E+02	-1.84E-01	5.58E+02	-1.84E-01	3.09E+02	-1.84E-01	1.21E+03
-0.01765	107.31	-0.01585	486.178	-1.86E-01	1.45E+02	-1.86E-01	5.58E+02	-1.86E-01	3.09E+02	-1.86E-01	1.21E+03
-	112.	-	487.	-	1.45E	-	5.58E	-	3.09E	-	1.21E

0.01853	014	0.01628	746	1.88E-01	+02	1.88E-01	+02	1.88E-01	+02	1.88E-01	+03
-0.01879	106.232	-0.0168	488.53	-1.90E-01	1.45E+02	-1.90E-01	5.58E+02	-1.90E-01	3.09E+02	-1.90E-01	1.21E+03
-0.01873	115.248	-0.01725	492.45	-1.92E-01	1.45E+02	-1.92E-01	5.58E+02	-1.92E-01	3.09E+02	-1.92E-01	1.21E+03
-0.01982	112.112	-0.01747	499.016	-1.94E-01	1.45E+02	-1.94E-01	5.58E+02	-1.94E-01	3.09E+02	-1.94E-01	1.21E+03
-0.01965	115.934	-0.01784	499.996	-1.96E-01	1.45E+02	-1.96E-01	5.58E+02	-1.96E-01	3.09E+02	-1.96E-01	1.21E+03
-0.02013	111.328	-0.0177	504.7	-1.98E-01	1.45E+02	-1.98E-01	5.58E+02	-1.98E-01	3.09E+02	-1.98E-01	1.21E+03
-0.0208	112.504	-0.0181	503.818	-2.00E-01	1.45E+02	-2.00E-01	5.58E+02	-2.00E-01	3.09E+02	-2.00E-01	1.21E+03

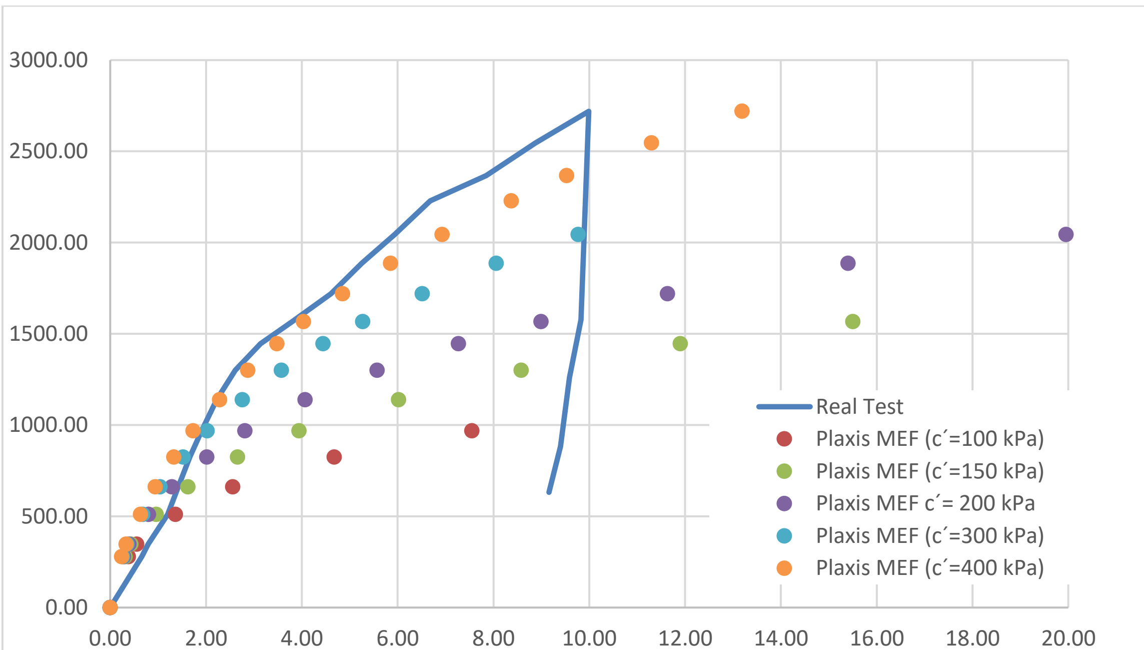


Optimized parameters		
E50	4.50E+04	kN/m2
Eoed	4.70E+04	kN/m2
Eur	1.30E+05	kN/m2
ν	2.20E-01	
c´	3.20E+01	kN/m2
phi´	4.00E+01	º

Appendix H: PMT simulation results

Real Test		Plaxis MEF		c´= 100 kPa	c´=150 kPa	c´=200 kPa	c´=300 kPa	c´=400 kPa
Pressure (kPa)	Deformation (mm)	Pressure (kPa)	Deformation (mm)	Deformation (mm)	Deformation (mm)	Deformation (mm)	Deformation (mm)	Deformation (mm)
0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
278.32	0.66	278.32	0.38	0.38	0.31	0.28	0.26	0.24
346.92	0.79	346.92	0.55	0.55	0.43	0.39	0.35	0.33
510.58	1.20	510.58	1.36	1.36	0.96	0.80	0.68	0.63
661.50	1.42	661.50	2.55	2.55	1.62	1.29	1.04	0.94
825.16	1.66	825.16	4.68	4.68	2.66	2.01	1.52	1.33
969.22	1.91	969.22	7.55	7.55	3.94	2.81	2.02	1.73
1139.74	2.22	1139.74	12.95	12.95	6.02	4.07	2.76	2.28
1300.46	2.61	1300.46	20.85		8.58	5.57	3.57	2.87
1446.48	3.14	1446.48	32.34		11.90	7.27	4.44	3.48
1568.00	3.80	1568.00			15.50	8.99	5.27	4.04
1719.90	4.61	1719.90			21.26	11.63	6.51	4.85
1886.50	5.25	1886.50			30.00	15.40	8.06	5.85
2045.26	5.94	2045.26			42.35	19.95	9.77	6.93
2229.50	6.68	2229.50						8.37
2366.70	7.85	2366.70						9.53
2547.02	8.88	2547.02						11.30
2719.50	9.99	2719.50						13.19
1577.80	9.83	1577.80						

1259.30	9.59	1259.30							
882.00	9.40	882.00							
632.10	9.16	632.10							



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Appendix I: Standard Prices of Geotechnical Services in Spain

Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIOS GEOTECNIA 2017

ACTUACIÓN:
Nº DE ENCARGO:
FECHA:
CLAVES:

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
	CAPÍTULO I : RECONOCIMIENTOS GEOTÉCNICOS							
	RECONOCIMIENTOS GEOTÉCNICOS HABITUALES							
1	Ud. Abono fijo por transporte de cada equipo de sondeo o penetrómetro estático al área de trabajo, incluyendo el primer emplazamiento	637.82		0.00	0.00		0.00	0.00
2	Ud. Traslado de sonda o penetrómetro estático entre puntos a reconocer, en obras lineales (uno menos que el nº total de puntos), incluyendo el emplazamiento	48.02		0.00	0.00		0.00	0.00
3	Ml. Perforación a rotación vertical o subvertical (< 30°) en suelos o roca de resistencia baja o muy baja, hasta 20 m de profundidad (según resistencia compresión, tabla IV.9, J. Salas, 1.975, Manual Taludes), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	60.73		0.00	0.00		0.00	0.00
4	Ml. Perforación a rotación vertical o subvertical (< 30°) en gravas o bolos, hasta 20 m de profundidad (con pasa tamiz 20 UNE inferior al 30 % y con pasa tamiz 0,080 UNE inferior al 10 %), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	115.08		0.00	0.00		0.00	0.00
5	Ml. Perforación a rotación vertical o subvertical (< 30°) en roca de resistencia media, hasta 20 m de profundidad (según resistencia compresión, tabla IV.9, J. Salas, 1.975, Manual Taludes), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	75.08		0.00	0.00		0.00	0.00

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Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
6	Ml. Perforación a rotación vertical o subvertical (< 30°) en rocas de resistencia alta o muy alta, hasta 20 m de profundidad (según resistencia compresión, tabla IV.9, J. Salas, 1.975, Manual Taludes), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	98.92		0.00	0.00		0.00	0.00
7	Ml. Perforación a rotación vertical o subvertical (< 30°) en suelos o roca de resistencia baja o muy baja, a partir de 20 m de profundidad y no superior a 50 m (según resistencia compresión, tabla IV.9, J. Salas, 1.975, Manual Taludes), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	69.33		0.00	0.00		0.00	0.00
8	Ml. Perforación a rotación vertical o subvertical (< 30°) en gravas o bolos, a partir de 20 m de profundidad y no superior a 50 m (con pasa tamiz 20 UNE inferior al 30 % y con pasa tamiz 0,080 UNE inferior al 10 %), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	131.06		0.00	0.00		0.00	0.00
9	Ml. Perforación a rotación vertical o subvertical (< 30°) en roca de resistencia media, a partir de 20 m de profundidad y no superior a 50 m (según resistencia compresión, tabla IV.9, J. Salas, 1.975, Manual Taludes), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	82.66		0.00	0.00		0.00	0.00
10	Ml. Perforación a rotación vertical o subvertical (< 30°) en rocas de resistencia alta o muy alta, a partir de 20 m de profundidad y no superior a 50 m (según resistencia compresión, tabla IV.9, J. Salas, 1.975, Manual Taludes), incluyendo testificación "in situ" a cargo de técnico experto y suministro de agua, ASTM D6286-98	107.65		0.00	0.00		0.00	0.00
11	Ml. Perforación a destroza en cualquier tipo de terreno y a cualquier profundidad, con diámetro de hasta 200 mm, incluido suministro de agua	41.85		0.00	0.00		0.00	0.00
12	Ud. Toma de muestra inalterada con tomamuestras de tipo abierto, ASTM D6169-98	29.87		0.00	0.00		0.00	0.00
13	Ud. Toma de muestra inalterada con tomamuestras de tipo pistón o Shelby, incluida camisa, D1587-00, XP P94-202	63.24		0.00	0.00		0.00	0.00
14	Ud. Testigo parafinado, ASTM D6640-01	16.18		0.00	0.00		0.00	0.00
15	Ud. Ensayo de penetración estandar (SPT), UNE-EN ISO 22476-3	29.20		0.00	0.00		0.00	0.00

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Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
16	Ml. Tubo ranurado de PVC, de diámetro útil no inferior a 60 mm, colocado en el interior de sondeo	8.66		0.00	0.00		0.00	0.00
17	Ud. Embocadura metálica de cierre de sondeo, de 1 m de longitud, con tapa roscada y resalte para llave inglesa, tomada con mortero e identificada con la denominación del punto, totalmente terminada	48.03		0.00	0.00		0.00	0.00
18	Ud. Lectura específicamente solicitada, de nivel freático en sondeo terminado, incluido achique y control de recuperación del mismo (mínimo de abono 5 sondeos/día. El abono se producirá, siempre que se hayan concluido todos los trabajos de campo de la actuación) UNE-	43.32		0.00	0.00		0.00	0.00
19	Ud. Lectura suplementaria de nivel freático en sondeo terminado, que exceda de 5 en el mismo día, incluido achique y control de recuperación del mismo, UNE-EN ISO 22475-1	21.10		0.00	0.00		0.00	0.00
20	Ud. Toma de muestras de las aguas, en sondeo, destinadas al análisis químico, UNE 41122	15.19		0.00	0.00		0.00	0.00
21	Ud. Caja portatestigos de PVC, incluido transporte a almacén designado y fotografía en color; ISO 2772-1, ISO 2772-2	16.23		0.00	0.00		0.00	0.00
22	Ud. Caja portatestigos de cartón parafinado, incluido transporte a almacén designado y fotografía en color, ISO 2772-1, ISO 2772-2	8.55		0.00	0.00		0.00	0.00
23	Ud. Caja portatestigos de madera, incluso transporte a almacén designado y fotografía en color, ISO 2772-1, ISO 2772-2	24.82		0.00	0.00		0.00	0.00
24	Ud. Abono fijo por transporte de penetrómetro dinámico superpesado al área de trabajo (incluyendo el primer emplazamiento)	326.71		0.00	0.00		0.00	0.00
25	Ud. Traslado de penetrómetro dinámico superpesado entre puntos a reconocer (uno menos que el nº total de puntos)	18.50		0.00	0.00		0.00	0.00
26	Ml. Penetración dinámica a cualquier profundidad (mínimo de abono 5 m. por ensayo, considerándose incluido en el precio el primer intento con rechazo a profundidad inferior a 2 m.), UNE-EN ISO 22476-2	23.58		0.00	0.00		0.00	0.00
27	Ud. Calicata manual o mecánica, incluidas toma de muestras, fotografías en color y reposición, incluyendo testificación "in situ" a cargo de técnico experto (mínimo de abono 3 ud.)	134.05		0.00	0.00		0.00	0.00

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Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
28	Ud. Toma de muestras superficiales o en calicata, de suelo de tipo inalterado, UNE 7371 bajo dirección "in situ" de técnico experto	60.00		0.00	0.00		0.00	0.00
29	Ud. Toma de muestras de las aguas, superficial o en calicata, destinadas al análisis químico, UNE 41122	6.98		0.00	0.00		0.00	0.00
30	Ud. Comprobación de la humedad natural "in situ", NLT-103	8.50		0.00	0.00		0.00	0.00
31	Ud. Determinación "in situ" de la densidad de un suelo por el método de la arena, UNE 103503: 1995, NLT-109	51.97		0.00	0.00		0.00	0.00
32	Ud. Determinación de la densidad "in situ" de un suelo por isótopos radioactivos (mínimo 6 determinaciones de abono), ASTM D3017-05, ASTM D2922-05	27.79		0.00	0.00		0.00	0.00
33	Ud. Toma de muestra de suelo, grava o piedra, de 80 Kg. de peso (2 sacos), mediante pala manual, en acopios o en superficie, bajo dirección "in situ" de técnico experto	26.26		0.00	0.00		0.00	0.00
	RECONOCIMIENTOS GEOTÉCNICOS ESPECIALES							
34	Ud. Ensayo de permeabilidad Lefranc, hasta 50 m. de profundidad, bajo dirección "in situ" de técnico experto, ASTM D4631-95(2000)	119.22		0.00	0.00		0.00	0.00
35	Ud. Ensayo de permeabilidad Lugeon, hasta 50 m. de profundidad, bajo dirección "in situ" de técnico experto, ASTM D4630-96	173.17		0.00	0.00		0.00	0.00
36	Ud.Toma de testigo en roca "in situ", con máquina sacatestigos o tallado de bloque	157.98		0.00	0.00		0.00	
37	Ud. Abono fijo por transporte de cada equipo de ensayos CPTU, incluso equipo técnico auxiliar	688.46		0.00	0.00		0.00	0.00
38	Ud. Traslado de penetrómetro estático (CPTU) entre puntos a reconocer (uno menos que el nº total de puntos)	18.50		0.00	0.00		0.00	0.00
39	Ml. Penetración estática tipo CPTU (piezocono), con medida y registro continuo de resistencia en punta, fuste y presión intersticial, i/p.p. de ensayos de disipación a cualquier profundidad, incluso p.p. de interpretación de resultados y presentación de informe. UNE EN ISO	129.50		0.00	0.00		0.00	0.00
40	Ud. Abono fijo por transporte de presiómetro, incluso equipo técnico auxiliar	265.25		0.00	0.00		0.00	0.00

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Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
41	Ud. Ensayo presiométrico, con ciclo intermedio de carga-descarga, bajo dirección "in situ" de técnico experto, i/p.p. de informe de interpretación.	312.97		0.00	0.00		0.00	0.00
42	Ud. Abono fijo por transporte de dilatómetro, incluso equipo técnico auxiliar	265.25		0.00	0.00		0.00	0.00
43	Ud. Ensayo dilatométrico, bajo dirección "in situ" de técnico experto, i/p.p. de informe de interpretación	255.04		0.00	0.00		0.00	0.00
44	Ud. Transporte de equipo de placa de carga al área de trabajo	111.54		0.00	0.00		0.00	0.00
45	Ud. Día de ensayos de carga de terrenos con placa cuadrada o circular, de superficie mínima superior a 700 cm^2, con o sin alargaderas, incluyendo medios de reacción e informe, UNE 103808, bajo dirección	465.39		0.00	0.00		0.00	0.00
	SUMA CAPÍTULO I :			0.00			0.00	

CAPÍTULO II : ENSAYOS DE LABORATORIO (I)

ENSAYOS DE SUELOS

PRECIO Nº	CONCEPTO	PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
46	Ud. Apertura y descripción	4.99		0.00	0.00		0.00	0.00
47	Ud. Determinación de la humedad de un suelo mediante secado en estufa, UNE 103300	11.11		0.00	0.00		0.00	0.00
48	Ud. Determinación de la densidad de un suelo. Método de la balanza hidrostática, UNE 103301	12.12		0.00	0.00		0.00	0.00
49	Ud. Determinación de la densidad relativa de las partículas de un suelo, UNE 103302	20.94		0.00	0.00		0.00	0.00
50	Ud. Determinación de la densidad mínima de una arena, UNE 103105	5.16		0.00	0.00		0.00	0.00
51	Ud. Determinación de la densidad máxima de una arena por el método de apisonado, UNE 103106	9.80		0.00	0.00		0.00	0.00
52	Ud. Determinación de la porosidad de un terreno, UNE 7045	27.32		0.00	0.00		0.00	0.00
53	Ud. Ensayo para determinar el índice "Equivalente de Arena" de un suelo, UNE-EN 933-8	15.14		0.00	0.00		0.00	0.00

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Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
54	Ud. Análisis granulométrico de suelos por tamizado, UNE 103101	31.36		0.00	0.00		0.00	0.00
55	Ud. Análisis granulométrico por sedimentación, UNE 103102	44.31		0.00	0.00		0.00	0.00
56	Ud. Determinación de los límites líquido y plástico de un suelo (límites de Atterberg), UNE 103103 y UNE 103104	25.82		0.00	0.00		0.00	0.00
57	Ud. Comprobación de la no plasticidad, UNE 103104	9.69		0.00	0.00		0.00	0.00
58	Ud. Determinación de las características de retracción de un suelo, UNE 103108	26.65		0.00	0.00		0.00	0.00
59	Ud. Ensayo de compactación Proctor normal, UNE 103500	47.84		0.00	0.00		0.00	0.00
60	Ud. Ensayo de compactación Proctor modificado, UNE 103501	66.70		0.00	0.00		0.00	0.00
61	Ud. Ensayo para determinar en laboratorio el índice C.B.R. de un suelo, UNE 103502	110.34		0.00	0.00		0.00	0.00
62	Ud. Ensayo de rotura a compresión simple en probetas de suelo, UNE 103400 (incluirá en el precio la preparación de la probeta, la humedad y la densidad)	26.06		0.00	0.00		0.00	0.00
63	Ud. Ensayo de consolidación unidimensional de un suelo en edómetro (hasta 1,20 MPa con al menos 7 escalones de carga y 3 de descarga i/curvas de consolidación), UNE 103405 (incluirá en el precio la preparación de la probeta a ensayar, el peso específico y el dibujo de las curvas de consolidación)	137.97		0.00	0.00		0.00	0.00
64	Ud. Incremento por cada escalón más	10.34		0.00	0.00		0.00	0.00
65	Ud. Ensayo de colapso en suelos, UNE 103406	59.92		0.00	0.00		0.00	0.00
66	Ud. Ensayo del hinchamiento libre de un suelo en edómetro, UNE 103601 (i/curvas).	67.65		0.00	0.00		0.00	0.00
67	Ud. Ensayo para calcular la presión de hinchamiento de un suelo en edómetro, con curva de descarga, UNE 103602 (i/curvas).	64.71		0.00	0.00		0.00	0.00
68	Ud. Ensayo de corte directo en suelos, sin consolidar y sin drenar, UNE 103401 (incluirá la preparación de las probetas a ensayar)	88.32		0.00	0.00		0.00	0.00
69	Ud. Ensayo de corte directo en suelos, consolidado y sin drenar, UNE 103401 (incluirá la preparación de las probetas a ensayar)	105.79		0.00	0.00		0.00	0.00

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Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
70	Ud. Ensayo de corte directo en suelos, consolidados y drenados, UNE 103401 (incluirá la preparación de las probetas a ensayar)	162.40		0.00	0.00		0.00	0.00
71	Ud. Triaxial sin consolidar y sin drenar en probetas de 1,5 pulgadas, (SC-SD-MI-1,5) UNE 103402	162.71		0.00	0.00		0.00	0.00
72	Ud. Triaxial saturado sin consolidar y sin drenar en probetas de 1,5 pulgadas, (CS-SC-SD-MI-1,5) UNE 103402	208.98		0.00	0.00		0.00	0.00
73	Ud. Triaxial consolidado sin drenar en probetas de 1,5 pulgadas, (CC-	246.07		0.00	0.00		0.00	0.00
74	Ud. Triaxial consolidado sin drenar y con medida de presión intersticial en probetas de 1,5 pulgadas, (CC-SD-PI-MI-1,5) UNE 103402	332.35		0.00	0.00		0.00	0.00
75	Ud. Triaxial consolidado y drenado en probetas de 1,5 pulgadas, (CC-CD-MI-1,5) UNE 103402	348.96		0.00	0.00		0.00	0.00
76	Ud. Determinación de la permeabilidad de una muestra de suelo. Método de carga constante, UNE 103403	61.39		0.00	0.00		0.00	0.00
77	Ud. Ensayo de permeabilidad con presión en cola	105.51		0.00	0.00		0.00	0.00
78	Ud. Determinación del contenido de carbonatos en los suelos, UNE 103200	30.98		0.00	0.00		0.00	0.00
79	Ud. Determinación del contenido de materia orgánica oxidable de un suelo por el método del permanganato potásico, UNE 103204	20.31		0.00	0.00		0.00	0.00
80	Ud. Determinación cuantitativa del contenido en sulfatos solubles de un suelo, UNE 103201	26.75		0.00	0.00		0.00	0.00
81	Ud. Determinación del contenido de sales solubles en los suelos, NLT-114, UNE 103205	24.81		0.00	0.00		0.00	0.00
82	Ud. Determinación del contenido de yesos en los suelos, UNE 103206	34.68		0.00	0.00		0.00	0.00
83	Ud. Determinación del pH de un suelo, UNE 77305	15.25		0.00	0.00		0.00	0.00
84	Ud. Análisis químico completo de suelo para determinar su agresividad frente al hormigón, según la Instrucción EHE: Grado de Acidez Baumann-Gully y determinación del ión sulfato, según procedimientos establecidos en la citada Instrucción	49.50		0.00	0.00		0.00	0.00

Agencia de Obra Pública de la Junta de Andalucía
CONSEJERÍA DE FOMENTO Y VIVIENDA

PRECIO Nº	CONCEPTO	GEOTECNIA PROPUESTA				GEOTECNIA REALIZADA		
		PRECIO UNITARIO	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)	Nº DE UNIDADES	IMPORTE (EUROS)	IMPORTE ACUMULADO (EUROS)
85	Ud. Análisis de aguas para agresividad al hormigón. Se determinan: pH según UNE 83952, dióxido de carbono agresivo según UNE-EN 13.577, determinación del ión amonio según UNE 83954, contenido en ión magnesio según UNE 83955:2008, determinación del ión sulfato según UNE 83956, determinación del residuo seco según UNE 83957	107.84		0.00	0.00		0.00	0.00
	ENSAYOS DE ROCAS							
86	Ud. Compresión simple de probetas de roca (incluyendo tallado sobre testigo), UNE EN 13383-2	46.29		0.00	0.00		0.00	0.00
87	Ud. Determinación de la resistencia a compresión simple de las rocas, con determinación del Modulo de Elasticidad (Young) y del Coeficiente de Poisson (medida de deformaciones longitudinales y transversales con bandas extensométricas u otro método preciso), incluido tallado sobre testigo, UNE 22950-3	98.08		0.00	0.00		0.00	0.00
88	Ud. Resistencia a la compresión triaxial de las rocas, i/ preparación de probetas UNE 22950-4	267.75		0.00	0.00		0.00	0.00
89	Ud. Determinación indirecta de la resistencia a tracción de las rocas (ensayo brasileño), i/ preparación de probeta, UNE 22950-2	33.24		0.00	0.00		0.00	0.00
90	Ud. Determinación de la resistencia a carga puntual de las rocas, i/ preparación de probeta, UNE 22950-5	13.28		0.00	0.00		0.00	0.00
91	Ud. Determinación de la estabilidad de los áridos y fragmentos de roca frente a la acción de desmoronamiento en agua, UNE 146510	61.84		0.00	0.00		0.00	0.00
92	Ud. Determinación de la estabilidad de los áridos y fragmentos de roca frente a la acción de los ciclos de humedad-sequedad, UNE 146511	134.45		0.00	0.00		0.00	0.00
93	Ud. Determinación del índice de Schimazek (incluida preparación de lámina y brasileño)	139.39		0.00	0.00		0.00	0.00
94	Ud. Estudio petrográfico con recuento mineralógico, i/ preparación de lámina delgada	155.46		0.00	0.00		0.00	0.00