

Stress–strain behaviour of the sediments in the tertiary basins associated with the Alentejo–Plasencia fault in the province of Cáceres (Spain)

F. Escolano Sánchez · M. Bueno Aguado

Abstract An analysis of the geotechnical information obtained from a large number of field tests (pressuremeter) and laboratory tests (identification, state, and mechanics) on the clay deposits in the small tertiary basins associated with the Alentejo–Plasencia fault in the province of Cáceres (Spain) has made it possible to classify them and predict their response to different levels of stress and strain. This geotechnical classification process must consider an appropriate model of constitution. The present article is based on the use of the Hardening Soil Model to facilitate predictions of the stress–strain behaviour of these tertiary clay deposits.

Keywords Hardening Soil Model · Pressuremeter testing · Triaxial testing · Alentejo–Plasencia fault · Spain

Introduction

Geotechnical surveys for new linear infrastructure projects provide researchers with valuable information in the form of field and laboratory tests on the geological formations

where these projects are developed. Beyond the specific purpose for which they are conducted, these surveys are an opportunity to deepen our technical and scientific knowledge with regard to these formations.

This article presents the results of a stress–strain behaviour study on certain deposits, mostly clay, which were silted into small tertiary basins associated with the Alentejo–Plasencia fault in the Extremadura area of the Iberian Massif (Spain).

The focus of this analysis is the behaviour of these deposits in both a drained and non-drained state, as well as the effect of the initial stress state.

The joint interpretation of field (pressuremeter) tests and laboratory (pedometer, direct shear, and triaxial) tests requires the use of constitutive equations that represent the behaviour of the material throughout the process, both in the preliminary break phase as well as once this has occurred. To this end, the constitutive model known as the “Hardening Soil Model (HS Model)” is a useful tool for the interpretation of these tests in this type of clay material.

Study phases

The stress–strain behaviour study of the clay deposits silted into small tertiary basins along the Alentejo–Plasencia fault was structured in four phases.

Phase 1

The specific geographic and geological setting of the study area was determined in order to record the presence of small discontinuous sedimentary basins distributed all along the Alentejo–Plasencia fault route in the province of Cáceres.

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Phase 2

Logging was completed of all available boreholes and new laboratory tests, which, along with those already existing, were used to describe this type of tertiary deposit from a geotechnical perspective.

Phase 3

Interpretation of the field work was completed – specifically, the pressuremeter testing conducted inside the perforated boreholes in the clay of these small tertiary basins associated with the fault (Aenor 1999).

Phase 4

Finally, the compressive strength and strain of the soil was studied and a set of parameters was proposed to define and adjust the HS Model constitutive equation.

Geological setting

The Alentejo–Plasencia fault, situated in the southwestern quadrant of the Iberian Peninsula, affects the Hercynian material of the southwestern Iberian Massif domain of a predominantly granite and granitic magma composition, with the exception of the central part, which affects low- to very low-grade metamorphic schist rocks pertaining to the Schist–Greywacke Complex (Istituto Geologico y Minero de España 1983).

This fault belongs to a system of normal and fractures in tearing mode that is widespread throughout the Iberian Massif, with a NE–SW direction and a route more than 400 km in length from the Central Iberian area to the Algarve area in the south of Portugal (Fig. 1).

A set of small, isolated sedimentary basins of tertiary age have been identified as associated with this fault, the genesis of which features marked structural control. These small basins have been grouped together in two areas in order to situate their geographic location (Fig. 2).

This figure represents the situation of the defined areas, located throughout the central part of the route of this fault, from the city of Plasencia to the town of Cañaveral in the province of Cáceres in the southwest of Spain.

The contour of these areas is prolonged, with dimensions of 2–7 km, maximum width of up to 3.5 km, and silted deposit thickness of up to 120 m.

The silting process took place in a continental environment with alluvial fan systems that denude the reliefs comprising the source area of a metamorphic composition

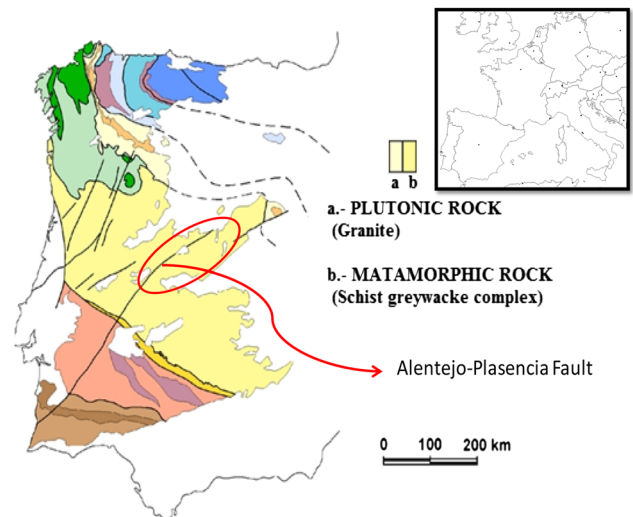


Fig. 1 Situation of the Alentejo–Plasencia fault in the Iberian Massif (coloured area)

recognised in geological literature as the Schist–Greywacke Complex, giving rise to reddish-to-orange clay and sandy clay (Fig. 3).

Geotechnical description

The information obtained from the laboratory and field tests conducted in the clay deposits of the small tertiary basins associated with the Alentejo–Plasencia fault have enabled the identification and quantification of the parameters of these deposits.

Laboratory tests

From a geotechnical perspective, the clay and sandy clay deposits are characterised by a percentage of the material weight that passes through a No. 200 ASTM sieve of greater than 80 %.

The liquid limits observed fluctuate between 28 and 52, with extreme erratic values of 63, meaning they may be classified as mean-plasticity clay (CL). Figure 4 shows the values obtained on the plasticity chart.

With respect to their state properties, Fig. 5 shows the dry specific weight values against natural moisture. The values are compared with regard to degree of saturation.

The natural moisture of the material is between 10 % and 15 %. The dry specific weight varies between 18 and 20 kN/m³. The degree of saturation is around 90 %. It can thus be confirmed that these are materials that may be classified as saturated or very near saturation.

Fig. 2 Situation of the two “sedimentary basin” areas studied in the Alentejo–Plasencia fault route

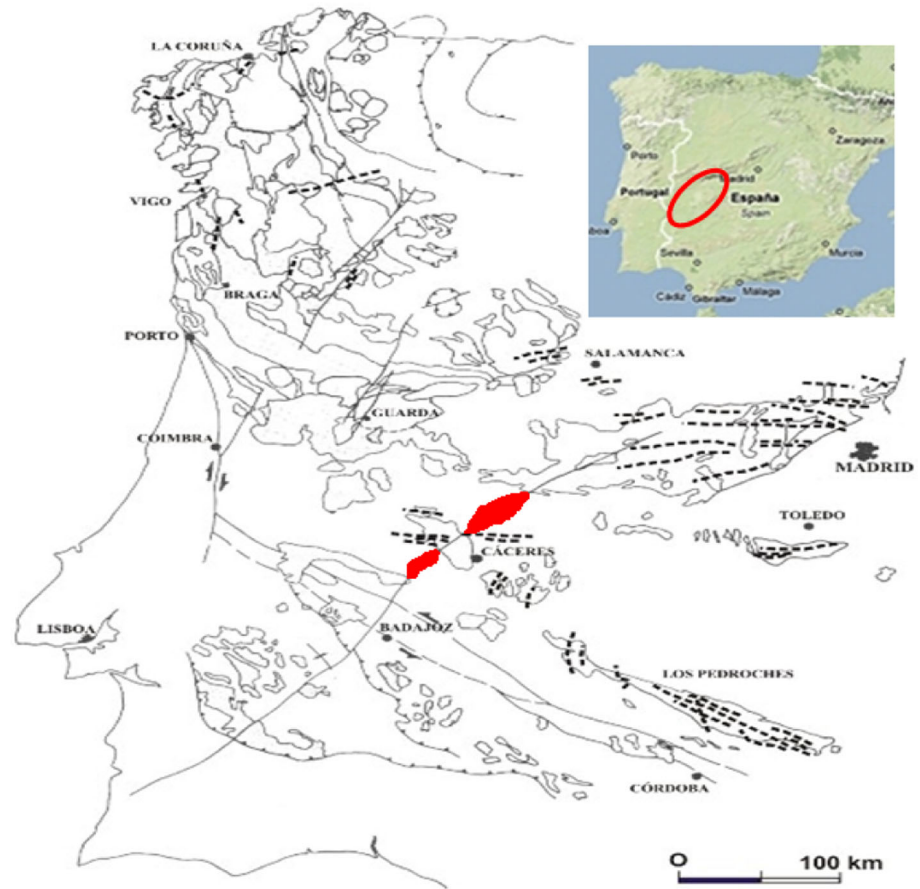


Fig. 3 The clay and sandy clay described in the tertiary basins associated with the Alentejo–Plasencia fault

Field tests: pressuremeter testing

One of the best opportunities for determining the mechanical behaviour of soil, while avoiding the alteration that occurs when taking samples, consists of “in situ” testing while mechanically drilling boreholes for ground reconnaissance. Pressuremeter testing is one such method

that makes it possible to characterise the stress–strain behaviour of the soil.

Pressuremeter testing affects the volume of the ground around the pressuremeter cell (Eurocode 7) in the section tested. As a result, the behaviour of the soil around the pressuremeter cell, which changes in volume, depends on a large number of factors, not all of which are directly related to the geotechnical parameters.

In order to more accurately understand the behaviour of the soil around a pressuremeter cell, a conceptual model has been developed that better adjusts the “stress–strain” path of the test. Based on a series of concentric rings, the thickness of which increases with the size of the radius, this model develops the behaviour of the land against pressure increases (Escolano and Bueno 2013). In other words, each increase in pressure leads to an increase in the radius, which implies a strain on the ground with each increase. Each of these rings fits in the “Hardening Soil Model (HS Model)” constitutive equation developed by Schanz and Vermeer (2000).

The HS Model was formulated as part of the elastic–plastic theoretical framework that outlines the behaviour of the soil simulated by pseudo-elastic models, including the

Fig. 4 Casagrande plasticity graph for the clay deposits

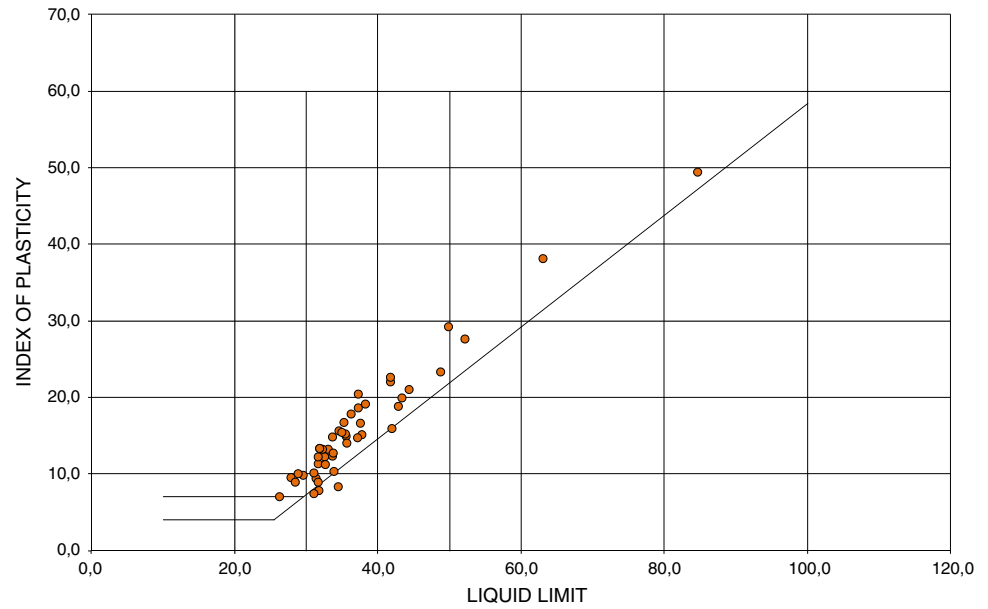
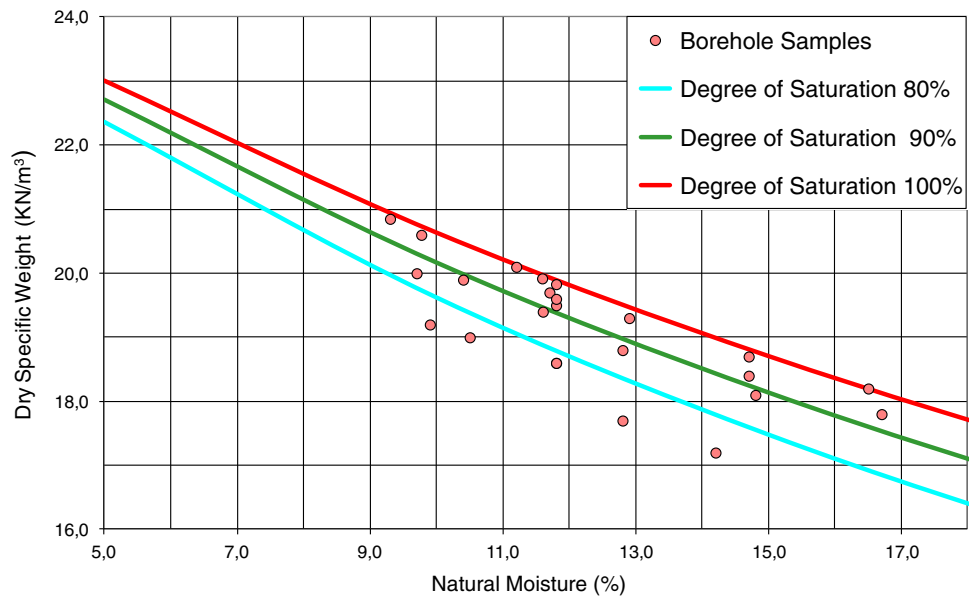


Fig. 5 The moisture, density, and degree of saturation of the clay deposits



most well-known, which is the hyperbolic model (Duncan and Chang 1970).

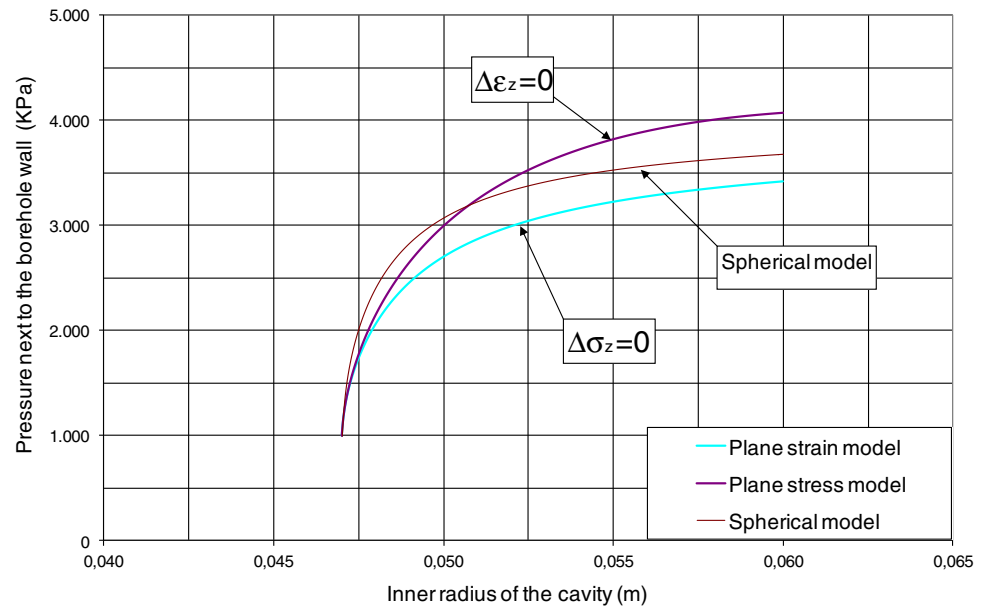
An initial variable on which the radial response of the soil depends as the pressuremeter cell expands is the vertical stress behaviour. A test conducted near the surface of the ground showed that the variation of the vertical stress would be null, and therefore that the soil would fit a plane-strain model. At great depth, the vertical strain would be null, thereby fitting with a three-dimensional strain model. In an intermediate case, both the vertical stress and circumferential stress could be equal at all times, which would then imply a concentric spherical strain model, or “spherical model”.

The graph in Fig. 6 represents these three soil behaviour models applied to a material with the same geotechnical parameters and identical initial conditions.

This figure shows how the plane-stress model (increase in the vertical strain equal to zero) is capable of bearing most of the radial pressure for the same level of strain. The plane-strain model (null vertical strain) is the one that bears the least radial pressure. The concentric spherical strain model, or “spherical model”, is found between the two.

The development of this conceptual model considered the two initial ground conditions: drained and non-drained conditions.

Fig. 6 Strain models determined by the variation in vertical stress



Under drained conditions—in other words, without any generation of interstitial pressure—the expansion of a pressuremeter cell in saturated material occurs over a relatively small period of time so that drainage can occur, and therefore the generation of interstitial pressure is expected. The three behaviour models were applied prior to a drained situation, as shown in the graph in Fig. 7.

Under non-drained conditions, the models continue the same behaviour defined for the drained case, with the exception that the radial stress increases for the same level of strain in comparison to the other models (Fig. 8).

The generation of interstitial pressure is proportional to the change in volume that takes place in each ring. Since it is a non-drained calculation, the volumetric strain should be null; however, the model accepts that the order of magnitude is lower than the radial strain.

For this model, it is accepted that the interstitial water is compressible with a volumetric module and an order of magnitude that is higher than that of the soil. This approximation to the non-drained behaviour of the soil is habitually used in other mathematical models (Haberfield and Johnston 1990; Clough 1990).

Finally, the state of initial stress, which is defined by the depth and level of over-consolidation, influences the results of the tests. Figure 8 shows this behaviour for the non-drained case and the plane-stress model.

In conclusion, the results of the pressuremeter tests are influenced at least by the constitutive equation the material fits with, the behaviour of the vertical pressure, the state of saturation, and the state of initial stress.

These models do not represent, under any circumstance, the limit pressure or the fluency obtained in the pressuremeter testing.

The graph figures above were prepared by conserving the same set of geotechnical parameters for the constitutive equation. The only thing that was modified was the type of model, the drainage conditions, and the state of initial stress.

Analysis of the geotechnical parameters

The analysis of the laboratory, triaxial, and direct shear tests, along with the results of the field pressuremeter tests, will make it possible to propose geotechnical parameters that are representative of the clay deposits in the tertiary basins associated with the Alentejo–Plasencia fault under drained and non-drained conditions.

Analysis under drained conditions

This analysis will be carried out for the geotechnical parameters that are needed for the constitutive HS Model equation. It is an equation under effective pressure, and therefore it works with parameters that are representative of the drained behaviour (Aubeny et al. 2000; Benoit 1995).

The representative breakage parameters, cohesion and angle of friction, are obtained from the triaxial and direct shear tests. Figure 9 shows the point of breakage of the samples tested. The geometric location of the breakage

Fig. 7 Variation in the initial testing stresses. Plane-strain model under drained conditions

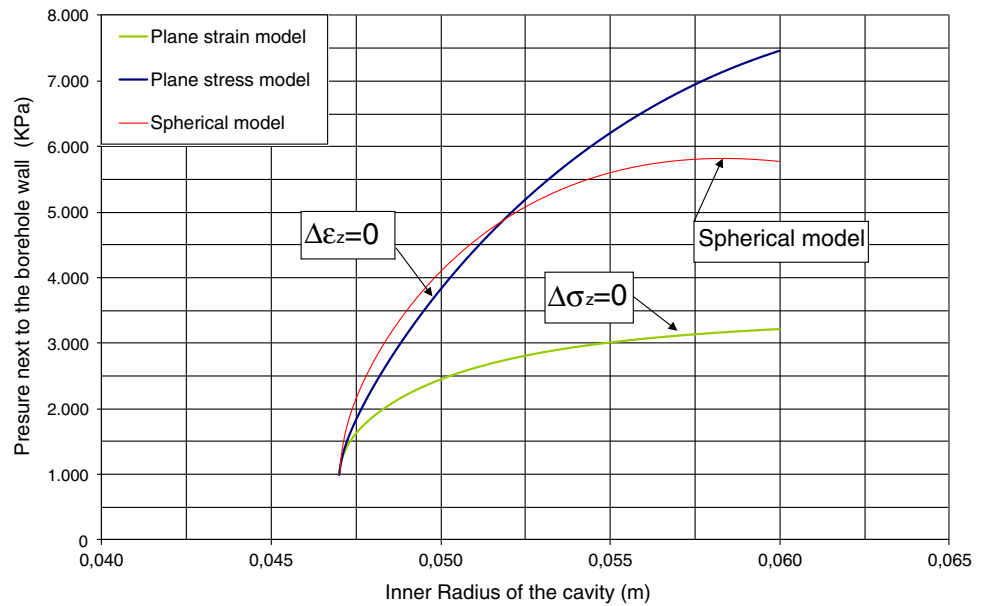
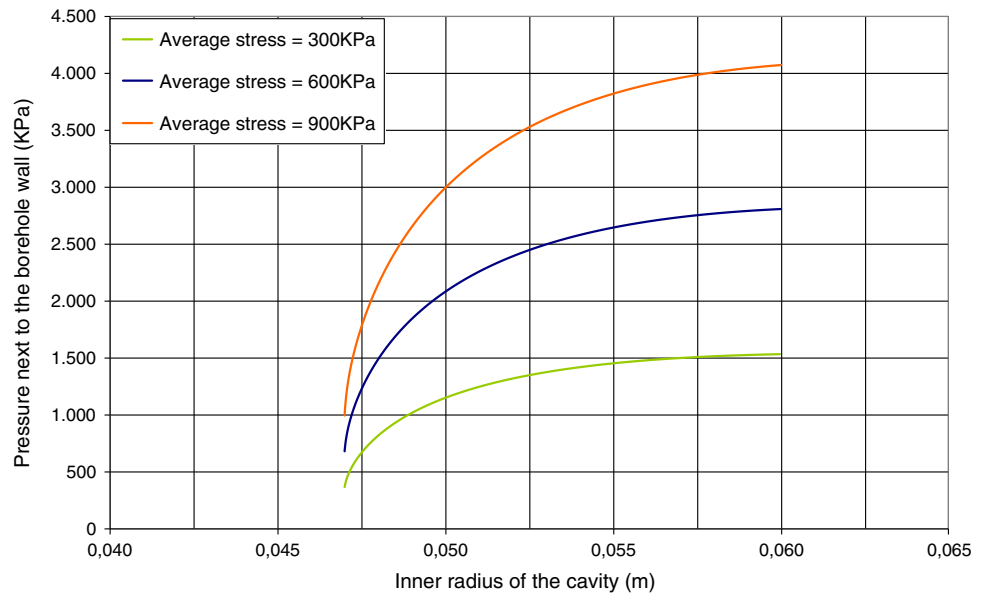


Fig. 8 Variation in the initial testing stresses. Plane-strain model under non-drained conditions



points forms a straight line that directly obtains the geotechnical parameters sought.

The remainder of the geotechnical parameters for the model are obtained by adjusting the pressuremeter testing curves to the “conceptual ring model” curves. The pressuremeter tests conducted at different depths are grouped into curves with considerably homogeneous shapes.

Figures 10 and 11 show the results of the curves obtained for two ranges of depth:

- Less than 10 m
- More than 10 m

The two test groups fit with the “conceptual ring model”. The one that best fits with the field results

obtained proved to be the one corresponding to the plane-stress model under non-drained conditions.

Figure 12 shows the adjustment made for the first group of tests at depths lower than 10 m. Figure 13 shows the adjustment made for the tests at depths greater than 10 m.

The two adjustments are achieved with the same set of geotechnical parameters, changing only the initial testing conditions. The set of parameters used is itemised in Table 1.

The initial state of the pressuremeter at depths lower than 10 m uses a vertical pressure of 150 kPa, which is equivalent to the weight of the ground and an initial horizontal stress of 300 kPa.

Fig. 9 Breakage results of the triaxial and direct shear tests

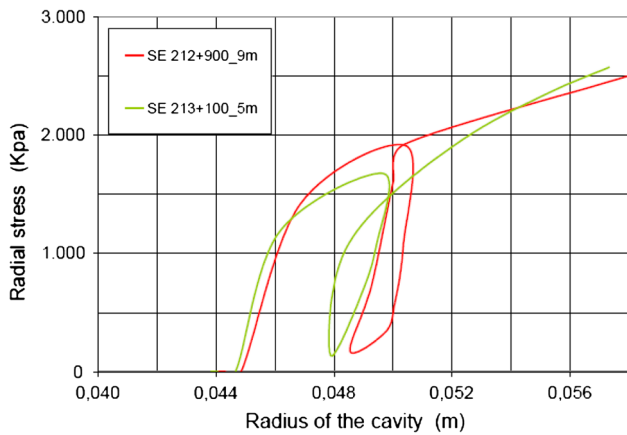
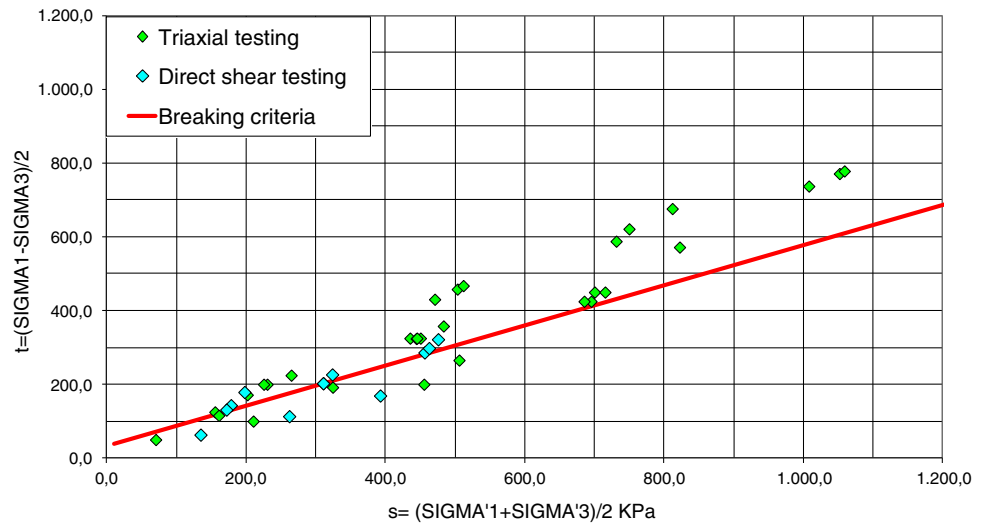


Fig. 10 Results of the pressuremeter tests at depths of less than 10 m

The initial state of stress of the pressuremeter at depths greater than 10 m considers a vertical pressure of 450 kPa, which is equivalent to the weight of the ground and an initial horizontal stress of 900 kPa.

In both cases, the initial stress conditions are represented by a push soil coefficient K_0 of around 2.

Analysis under non-drained conditions

When considering the parameters under drained conditions in the section above, there was an increase in the mechanical characteristics with the confining pressure. The very constitutive equation provides for this improvement.

The definition of the parameters under non-drained conditions or in total pressure does not provide for such a

Fig. 11 Results of the pressuremeter tests at depths of more than 10 m

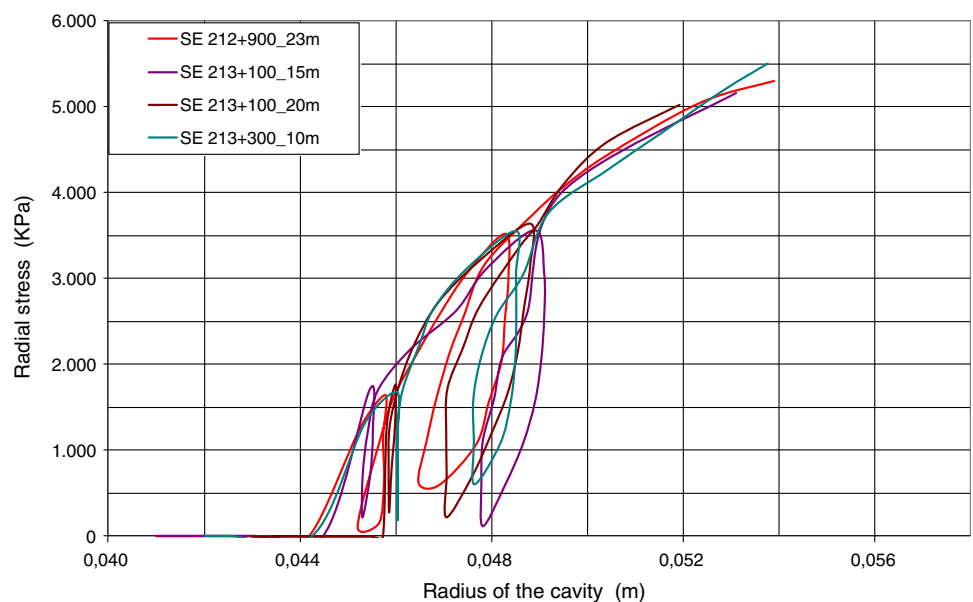


Fig. 12 Adjustment of the pressuremeter tests at depths of less than 10 m

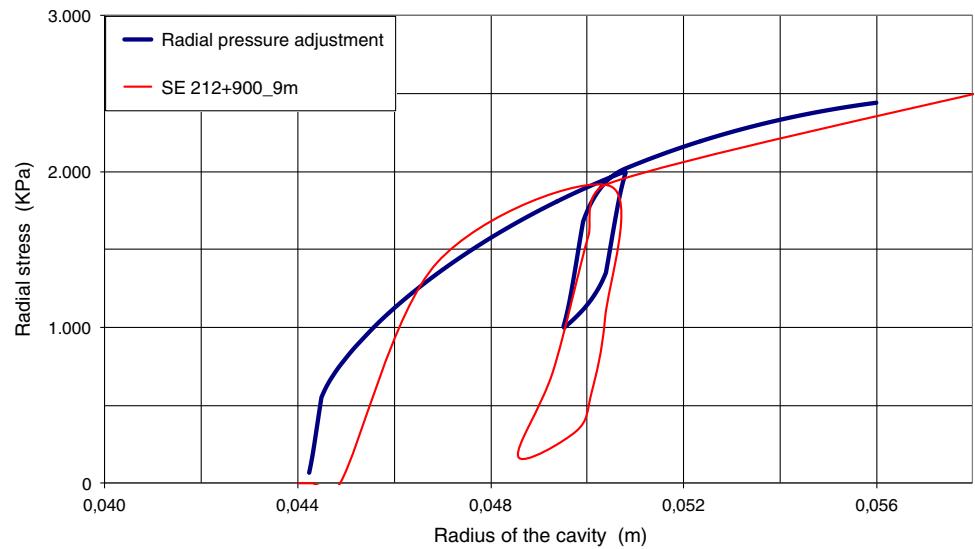


Fig. 13 Adjustment of the pressuremeter tests for depths greater than 10 m

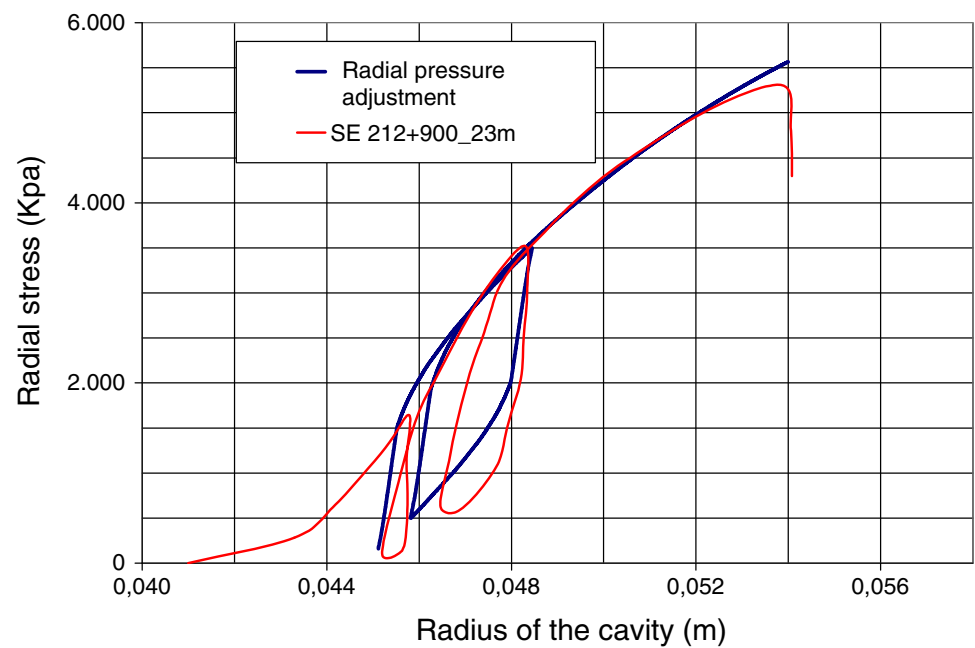


Table 1 HS model parameters for the clay deposits in the tertiary basins associated with the Alentejo–Plasencia fault

Geotechnical parameters	Value
Effective cohesion	40 kPa
Inner angle of friction	33°
Angle of dilation	3°
Modulus at a load reference pressure $E_{50ref(*)}$	50 MPA
Modulus at a discharge reference pressure $E_{urref(*)}$	90 MPA
M	0.8
R_f	0.7
Poisson's ratio	0.3

(*) Reference pressure 100 kPa

developed constitutive equation, meaning the parameters must be defined based on the confining pressure.

Figure 14 shows the value of the shear strength under non-drained conditions (CU) for the test tubes tested, using the triaxial apparatus with the interstitial pressure measured based on the effective confining pressure.

There is great dispersion in the shear strength under non-drained conditions for a single level of stress. This dispersion is largely corrected when working with drained parameters, as was verified in the section above. This fact confirms the classical idea that the strength of a material is governed by the effective pressure and not by the total pressure.

Fig. 14 Shear strength under non-drained conditions (CU) obtained with triaxial testing

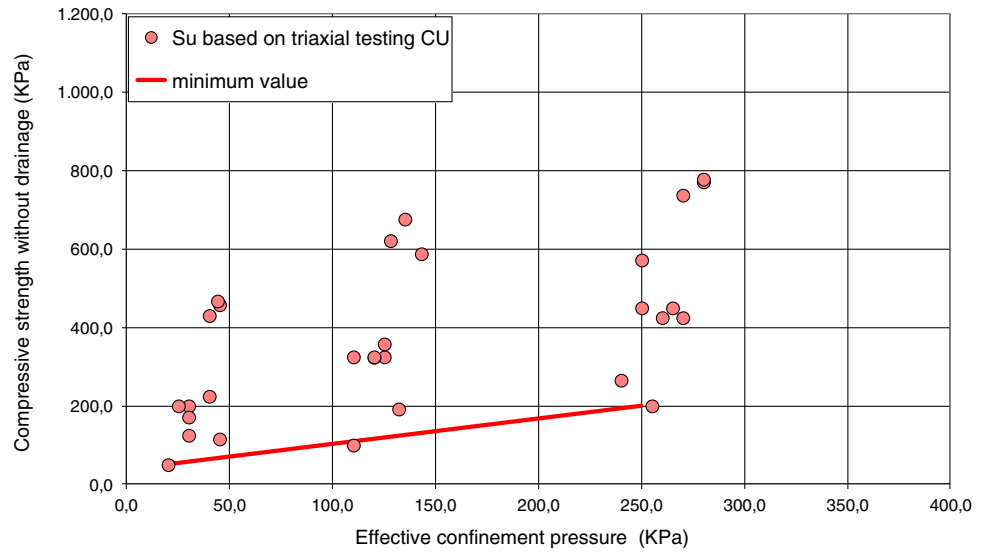
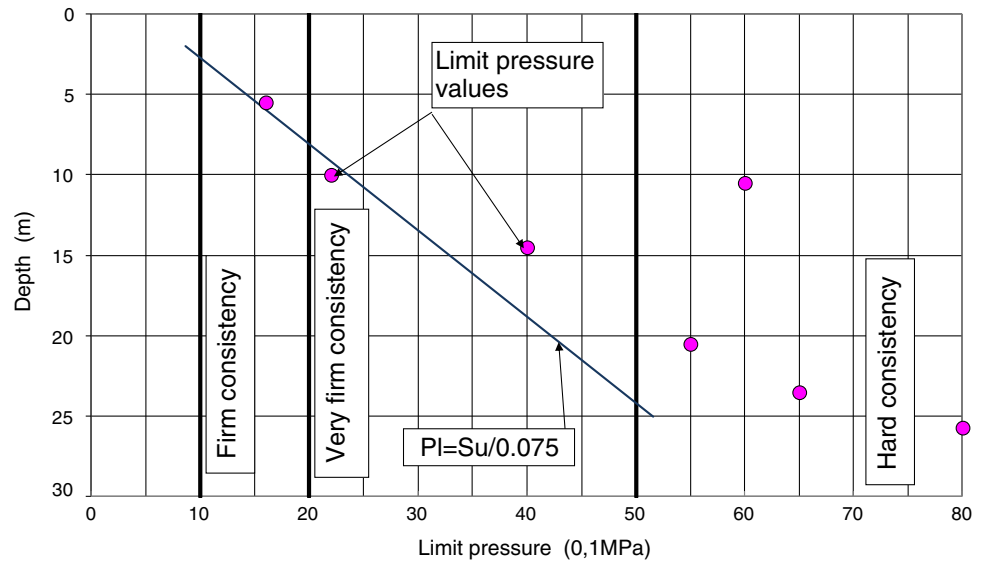


Fig. 15 Comparison of the limit pressure and the law of shear strength variation under non-drained conditions with the depth



In any case, a law can be proposed based on the results above that guarantee a minimum value of shear strength with the confining pressure under non-drained conditions. This law would be as follows:

$$S_u = 37 + 0.7\sigma'_3 \quad (1)$$

This law can be transformed into a strength law based on the depth using the expression:

$$\sigma_3 = K_0(\gamma - \gamma_w)h \quad (2)$$

With $K_0 = 2$. The result is:

$$S_u = 37 + 14h \quad (3)$$

where h in metres, S_u in kPa.

A limit pressure value was obtained with the pressure-meter testing. It is defined as the pressure reached when the initial volume of the expanding cavity doubles.

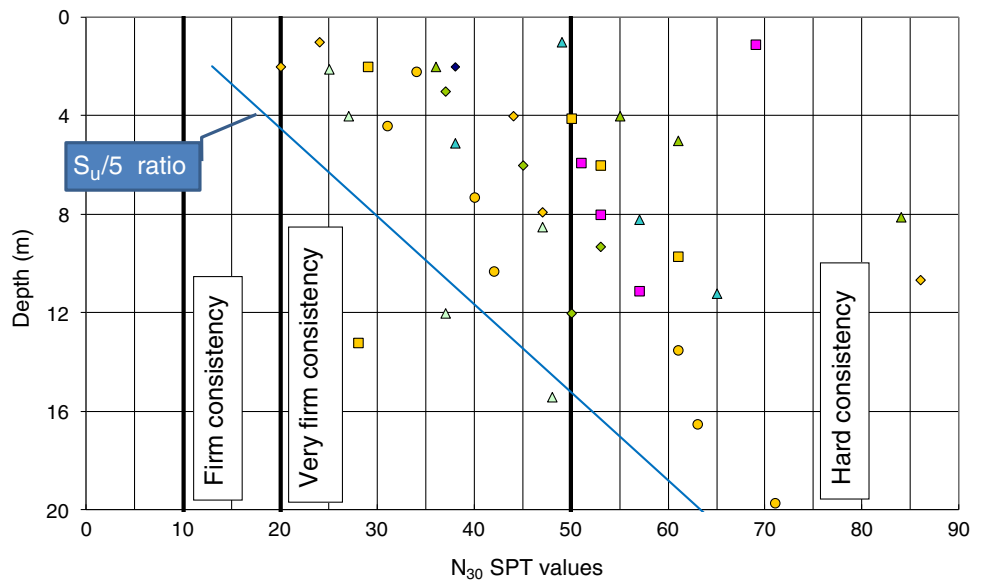
Figure 15 shows the value of the limit pressure with the depth. This value is compared with the law of variation in S_u .

The graph shows a possible correlation between them, which would be defined as:

$$S_u = 0.075 P_1 \quad (4)$$

Finally, the improved mechanical characteristics of the material with the depth are reflected in the SPT dynamic penetration tests (Yagiz et al. 2008). Figure 16 shows the values obtained in a large number of boreholes distributed

Fig. 16 Comparison of the SPT tests and the law of shear strength variation under non-drained conditions with the depth



throughout the basins and compared with the law deduced for the shear strength under non-drained conditions.

Again, there is correlation between both, which could tentatively be shown as:

$$S_u = 5N_{30}(\text{SPT}) \quad (5)$$

Discussion

Adequate geotechnical characterisation of a unit must feature a wide variety of tests, both in number and type, that make it possible to analyse the behaviour in different situations.

The actual characterisation work consists of predicting the behaviour of the material for different stress paths. To do so, advanced constitutive equations such as the HS Model must be used, as well as mathematical models that reproduce the stress–strain path of the tests, such as the ring model for pressuremeter testing.

Discerning the type of pressure being provided by the tests under drained and non-drained conditions along with the initial state of stress is essential to properly interpreting the results (Briaud 1990).

The strength–strain response of the ground, both in the short and long term, is governed by the effective pressure. However, the measuring devices most often used measure total pressure—in other words, the sum of the effective pressure and the interstitial pressure generated. Given that there is certain randomness in the generation of the latter, the parameters defined in total pressure will always offer a higher degree of dispersion.

On the other hand, the standard duration of field test requires measuring total pressure and, therefore, the unavoidable use of the results. Nonetheless, there are

analytical and complementary laboratory testing procedures, as shown in this article, that make it possible to obtain the right parameters for use with effective pressure.

Conclusions

The geomechanical behaviour of the somewhat over-consolidated sandy clay of the Miocene basins associated with the Alentejo–Plasencia fault is adequately defined with the HS Model constitutive equation. This article proposes a set of parameters for the equation that make it possible to adjust it to the behaviour measured in the field tests and laboratory tests conducted in order to characterise the unit.

The article highlights the importance of considering the initial state of pressure and the drained and non-drained behaviour.

The parameters proposed in this article are applicable in computer programs that develop the type of constitutive equations for which they are defined. Thus, an advanced ground behaviour model is available for designing foundations and structures planned in the Miocene basins associated with the Alentejo–Plasencia fault.

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