ANALYSIS OF A DILATOMETER TEST IN OVER-CONSOLIDATED SEDIMENTS, BASIN OF THE DUERO RIVER, SPAIN

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Abstract

A dilatometer test is a useful method for in-situ geotechnical surveys. It can be compared with the results obtained using a mathematical model. The mathematical model of concentric rings shown in this article is governed by the constitutive equation of the "Hardening Soil Model". A large number of tests made on the Dueñas Geological Facies, with a consistency ranging from firm clays to soft rocks, are compared to the model results. In this way, the "Hardening Soil Model" parameters are adjusted to the Dueñas Facies materials.

1 INTRODUCTION

The pressuremeter test is very useful for determining the geotechnical feature parameters of original soils and for reducing the most common changes in the mechanical characteristics caused by sampling. It also allows us to test the original soil in its natural state of effective and pore-water pressure [1,2] Another advantage of this test is that a greater volume of material is tested in situ than would be tested in the laboratory, thus being closer to the real loading state encountered afterwards in engineering works.

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, any disturbance to the borehole wall must be minimized.

The equipment used is called a PBP (pre-bored pressuremeter), because it is best suited to the type of substrate studied (over-consolidated clays and soft rocks) and the standard applied is the ASTM D 4719-87 [3]. Table 1 shows the applicability of different types of pressuremeters [4].

In lightly over-consolidated clays, in a lot of consolidated ones as well as in a wide range of soft rocks, the volume

Table 1. Applicability.

SUBSTRATE	PROBE PBP	PROBE SBP	PROBE PIP
Soft Clays	A	A	A
Hard Clays	A	A	A
Sparse Sands	B[1]	A	A
Dense Sands	B[1]	B	C
Gravels	C	N	N
Soft rocks	A	B	N
Hard rocks	A	N	N

A = High; B = Medium; C = Low; N = None (1) Using fluted sleeve (Clarke 1996).

of soil examined by the pressuremeter test reaches states of tension that exceed the elastic levels.

The interpretation and study of the strain and deformation states, produced in these cases, requires the use of soil constitutive models that allow this simulation.

The current wide variety of computing programs includes soil advanced constitutive equations that simulate plastic states. Although these models should be implemented according to some initial parameters, these are usually far from those obtained in geotechnical tests. In the case of the over-consolidated clays and soft rocks, the constitutive model known as the "Hardening Soil Model" reliably reproduces the different stages of strain and deformation observed in field and laboratory tests. This effect on the constitutive model has been studied [1-2]. Thanks to the interpretation of the pressuremeter test using this constitutive model, a sound comprehension of the soil tensional processes, as well as a suitable adjustment of the tested model parameters, are achieved. During the geotechnical studies carried out for the high-speed train line between Valladolid and Burgos, several dilatometer tests were carried out in Dueñas Geological Facies. The geotechnical knowledge of Dueñas and the dilatometer test have been thoroughly studied with the support of the constitutive equation.

A specific programme has been developed to define the pressuremeter tests that are also appropriate for other geotechnical units and used in the mentioned unit. This article explains the most relevant features of the method and compares its results with those obtained in the tests' campaign. Finally, this study brings forward the geotechnical parameters of Dueñas Facies according to the "Hardening Soil Model" constitutive equation.

2. GEOGRAPHICAL AND GEOLOGICAL FRAME OF THE SURVEY AREA

The survey area is located in the Arlanzón river valley, between the Burgos and Palencia provinces, in the Iberian Peninsula NW quadrant. From the geological point of view, it is located inside the sedimentary basin of the Duero river, which spreads along approximately 50,000 km^2 (Figure 1).

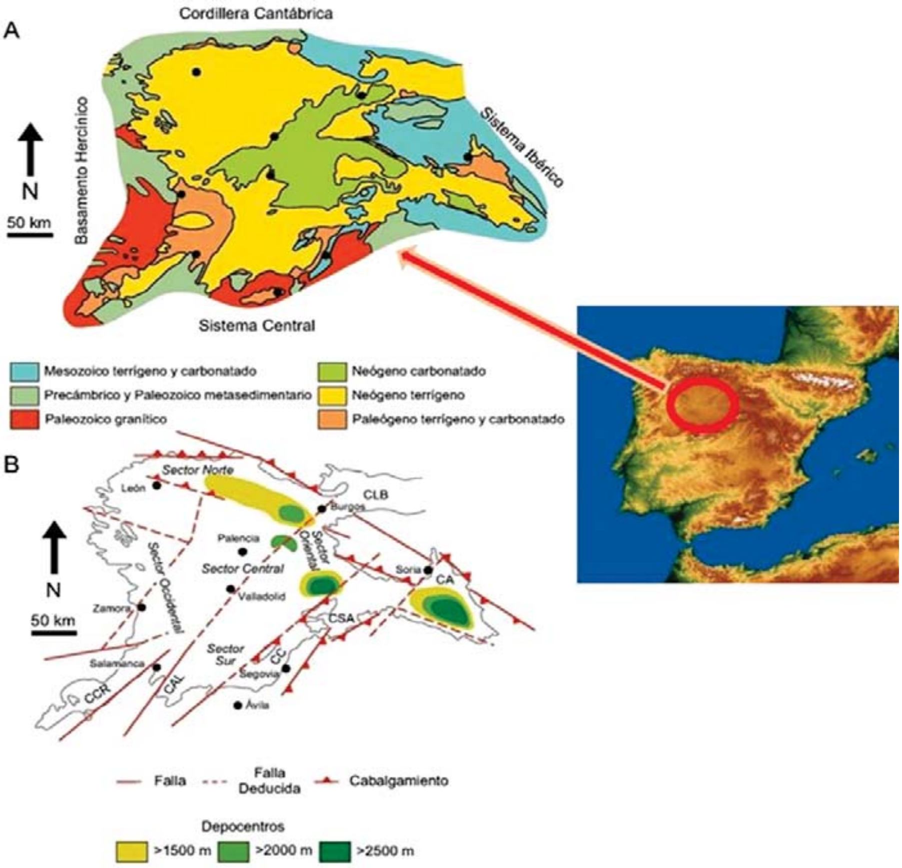

Figure 1. Geological and geographical location of the study area and the thickness of the sediments.

The sedimentary basin of the Duero river is an intra-plate depression that began to form at the end of the Cretaceous Period due to the Alpine movement of old basement fractures, produced during the Hercynian orogeny [5].

The sediments that filled this depression are organized according to a centripetal model in such a way that the terrigenous materials are disposed along the external edge of the basin and the chemical facies (carbonate) can only be found in the centre.

It should be pointed out that there is a noticeable asymmetry of the basin so that the chemical central facies are displaced towards the oriental edge. The thickness of the sediments that filled the depression is uneven, reaching a thickness of over 2500 m in some areas [5].

The thickest areas are located on a WSW-ESE oriented surface that impacts the cities of León, Palencia, Aranda de Duero and Soria (Figure 1). This pattern shows the behavior of the bedrock fractures during the basin sedimentation following the hors-graben general model.

The colmatation of this basin was not constantly and continuously carried out, it was achieved by "impulses" of maximum subsidence combined with periods of calm and even others of no sedimentation.

It was a continental sedimentation corresponding with a perimetral system of coalescent alluvial fans, which drain into a central saline endorheic lacustral basin. The climatic conditions were typical of an arid or semi-arid climate with a variable seasonal rainfall.

These general climatic conditions were almost constant during the whole Neogene Period. In the Quaternary geological period the basin fill is eroded, becoming in that way an exorheric basin and the fluvial net fits in. This process arranges along the time and is showed in the sedimentation of many terrace levels.

2.1. Stratigraphy of the survey area

All the exposed materials of the area are sedimentary and they can be gathered into four major soil groups:

- "Dueñas" formation. This is the most representative. It comprises an alternation of thin layers of clays, marlstones, marly limestones and gypsumy marlstones. This lithologic group has a colour that goes from light-green to whitish, its aspect is massive and its thickness is uniform.
- "Tierra de Campos" formation. This is a lithologic group composed by clays, whose colour goes from pink to reddish, mixed with sandstones, gravels and conglomerates, whose colour goes from yellowish to reddish.
- "Cuestas" formation. This comprises an alternation of limestones, marly clays and marlstones. Its general coloration is whitish or cream and it forms a uniform level of soil, whose thickness goes from 45 to 50 m, approximately.
- "Páramo" limestone formation. This is composed of a group of thick limestone layers whose grain is fine, its structure massive and its colour from whitish to cream. Its widest thickness is from 5 to 10 m approximately.

As was mentioned above, the importance and representativeness of the "Dueñas" formation, from a geological point of view, is the subject of this article.

3 DUEÑAS FACIES GEOTECHNICAL FEATURES

Dueñas Facies is part of the tertiary sediments in the Duero depression. The basin sediments have been eroded by the fluvial net and have formed wide valleys. In the hillsides and fields of these valleys, there are horizons of clays, marlstones and limestones corresponding to the units called "Cuestas Facies", "Tierra de Campos and Dueñas Facies".

The Dueñas Facies, chronologically older than the previous ones, comprises an alternation of 0.20–0.60 m, layers of clays, marlstones, marly limestones and gypsumy marlstones. Its colour goes from light-green to whitish and its general aspect is massive and uniform. The diffraction analyses with oriented aggregates carried out on samples from both layers give the mineralogical composition, as stated below in Table 2.

Table 2. Mineralogical composition of Dueñas Facies.

	MINERALS					
Unity	Quartz	Dolomite	Micas + kaolinites	Calcite	Gypsum	Motmorillonite
TD/y	<7%	30 s 50%	10 a 30%	<5%	30 a 55%	<10%
TD/m	<10%	20 a 50%	20 a 55%	10 a 15%	<0.5%	<10%

The superior layer, named TD/m, comprises clays and dolomites in similar levels. The inferior layer, named TD/y, has a gypsum mineral in almost a 50% proportion. This mineralogical composition reflects in its plastic features. The values obtained in the tested samples are shown in the plasticity chart below, Figure 2. In the case of gypsum marlstones (TD/y), as well as in the clays (TD/m), the samples have a wide level of liquid limits with values from 20 to 90. In both cases the points are mainly on line A. In addition, at the same liquid limit the clayey unit (TD/m) usually has a lower plasticity rate.

In a natural state, the density and humidity in both units are placed in rates of different values. The results from the different tests are depicted in Figure 3. A higher density and a lower humidity are observed in the samples of gypsumy marlstones unit (TD/y). The qualities of the resistance and the deformation in both units are also different from those obtained in the dilatometer tests. These results will be discussed afterwards.

4 HARDENING SOIL MODEL

The constitutive equation called the "Hardening Soil Model (HS-model)" has been developed [6]. It is a model formulated inside the framework of elastoplastic theory, which explains the behaviour of a simulated soil by pseudo-elastic models, and among them the hyperbolic model stands out as being the best known.

In this section only the most relevant features that allowed an explanation of the pressuremeter test are mentioned. As in any elastoplastic model, the HS-model comprises:

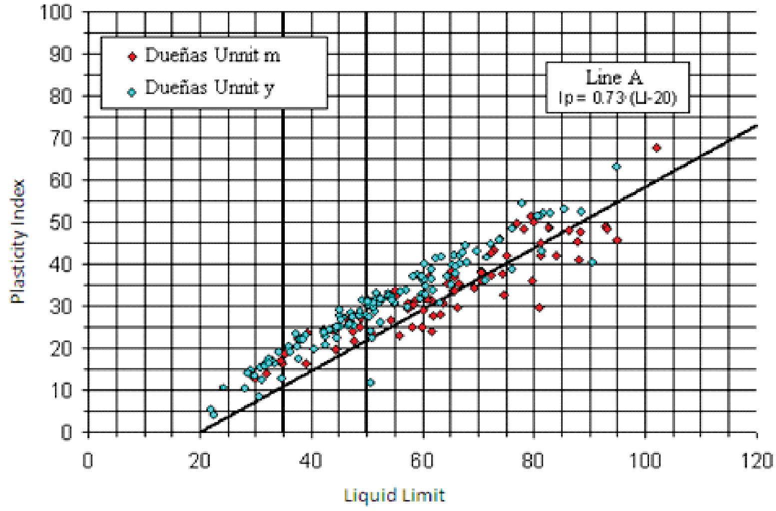

Figure 2. Graph of plasticity.

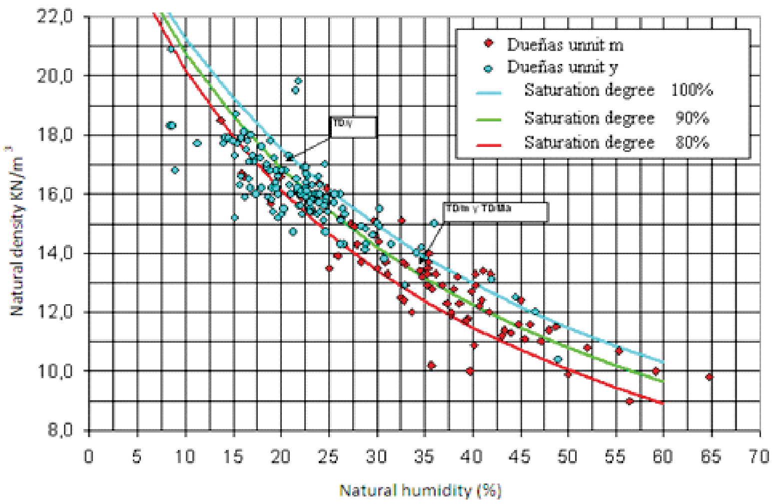

Figure 3. Dueñas Facies natural density and humidity.

- A defined failure criterion for effective pressures.
- An elastic behaviour for strain states under the failure level.
- A plastic power function that determines the direction of plastic deformations.
- A hardening law that modifies the failure criterion according to the previously achieved strain states and deformation tests.

A typical feature of an HS model is that the hardening law is defined by the plastic deformation through the previous shear. The HS model's failure criterion is depicted in the equation:

$$f = q_a/E_{50}\, q/(q_a - q) - 2q/E_{ur} - \gamma_p \qquad (1)$$

where:

q: stress deviator ($q = \sigma_1 - \sigma_3$)
q_a: failure deviator asymptote obtained by the equation:

$$q_a = q_f/Rf \qquad (2)$$

q_f: failure deviator for an average effective pressure p, from:

$$q_f = 6\, sin(\phi)/(3 - sin(\phi))\, (p + c\, cotan(\phi)) \qquad (3)$$

where:

- c and ϕ are the Morh Coulomb's failure criterion parameters.
- The E_{50} and E_{ur} load module and download module obtained by the expressions:

$$E_{50} = E_{50}^{ref}\, (\sigma_3 + c\, cotan(\phi))/(\sigma_{ref} + c\, cotan(\phi)))^m \quad (4)$$

$$E_{ur} = E_{ur}^{ref}\, (\sigma_3 + c\, cotan(\phi))/(\sigma_{ref} + c\, cotan(\phi)))^m \quad (5)$$

where:

- E_{50}^{ref} and E_{ur}^{ref} are the reference modules for the confining pressure σ_{ref}.
- m is the influence exponent of the confining pressure in the deformation module.
- γ_p is the value of plastic shearing deformation obtained by the expression:

$$\gamma_p = \varepsilon_1^p - \varepsilon_2^p - \varepsilon_3^p \qquad (6)$$

The elastic behaviour below the failure level is an elastic equation with a deformation module E_{ur} and a Poisson's coefficient ν.

Finally, the plastic deformations are defined by the plastic potential function "g" with the expression:

$$g_{13} = (\sigma_1 - \sigma_3)/2 - ((\sigma_1 + \sigma_3)/2)\, sin(\psi) \qquad (7)$$

$$g_{12} = (\sigma_1 - \sigma_2)/2 - ((\sigma_1 + \sigma_2)/2)\, sin(\psi) \qquad (8)$$

where:

ψ is the dilatancy angle.

The second surface simulates the plastic deformation direction when two main strains match up (compression and traction biaxial states). To sum up, there are seven essential geotechnical parameters to define the constitutive equation:

c: the effective cohesion
ϕ: the effective friction angle
E_{50}^{ref}: the load module for a reference pressure
E_{ur}^{ref}: the reload module for a reference pressure
Rf: the failure relation
m: the influence exponent of the confining pressure in deformation module
ν: the Poisson's coefficient
ψ: the dilatancy angle.

5 THE INTERPRETATION MODEL OF THE PRESSUREMETER TEST

The interpretations of the pressuremeter tests were performed according to a concentric-rings model [7]. Figure 4 shows a schematic representation of the used model.

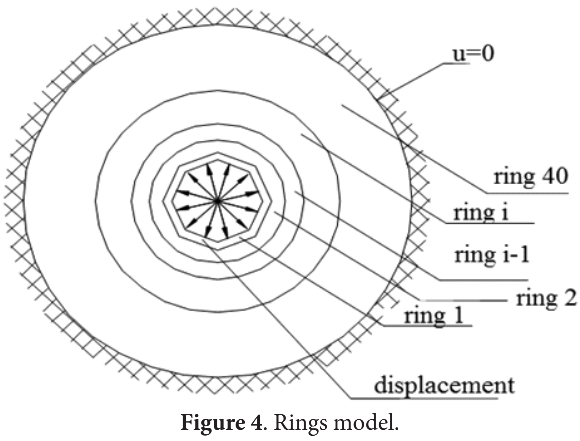

Figure 4. Rings model.

It is an axisymmetric model made of forty concentric cylindrical rings of variable thickness. This thickness grows with its radius so that the narrowest ring is close to the bore hole. The external radius of the model is approximately 10 metres. The internal one is the borehole radius, which is usually 5 cm in size. Initially, the internal radius of each ring is determined by the expression:

$$r_i = r_{i-1} + i^2/2000 \qquad (9)$$

where:

r_i is the internal radius of the "i" ring in the meter.

An outline condition of no radial deformation is established on the external edge of the 40th ring of the model. On the internal edge of the first ring a "u" displacement takes place. In order to use the small deformation hypothesis, the magnitude of the "u" displacement is limited, so the variation in the radial deformation in any ring must be under 0.5%

For small deformations, each ring responds to the displacement according to an elastic and linear behavior. The used parameters are the unload deformation module E_{ur} and the Poisson's ratio v of the element when the displacement takes place. As a result, a "u" displacement is obtained in each ring in its internal outline.

According to the displacement of each ring, the average deformation ring is determined by the expressions:

$$\varepsilon_{ri} = (u_{i+1} - u_i)/(r_{i+1} - r_i) \qquad (10)$$

$$\varepsilon_{ti} = (u_i + u_{i+1})/(r_i + r_{i+1}) \qquad (11)$$

where:

ε_{ri} and ε_{ti} are the radial and circumferential deformations, respectively.

The vertical deformation is obtained by considering the total vertical pressure constant. Taking these deformations as a starting point and using a numerical integration procedure, the effective strains compatible with the constitutive equation are determined.

After each deformation increment, the ring model is updated with the new radii dimensions.

In the case of the undrained test, the deformation module is a transformation of the previous one, obtained by the condition of a constant shear modulus G.

$$G_u = G$$

$$E_u/(1 + v_u) = E_{ur}/(1 + v) \qquad (12)$$

The adopted undrained Poisson's module is 0.495.

The generated pore-water pressure in each element is a function of the volumetric deformation (ε_v) and the water-compression module. It is supposed to have a superior magnitude than the soil magnitude. The generated pore-water pressure in each interval and ring (v_i) is determined using the expression [8].

$$v_i = 300\ E_{ur}\ (v_u - v)\ /\ \{3\ (1 + v)\ (1 - 2 * v)\}\ \varepsilon_v \qquad (13)$$

With these effective pressures and the pore-water pressure, the total pressures are determined as:

$$\sigma = \sigma' + v \qquad (14)$$

The total radial pressure of the first ring corresponds to the one applied inside the borehole.

Additionally, the traction states are limited so that no negative effective stress could appear. No negative values are allowed for the interstitial pressure either. The model requires the definition of an initial state pressure, which must be similar to the corresponding geostatic one. The initial pore-water pressure will be a function of the phreatic level measured in the borehole.

6 MODEL CHARACTERISTIC RESULTS

The application of successive displacements inside the first ring of the model implies a relation between the internal pressure and the deformation of the borehole wall with a relatively hyperbolic shape [9]. Figure 5a shows the evolution of the total pressure on the borehole wall. The total radial pressure has two stages.

The first one with mainly elastic deformation is characterized by large increments of pressure for small deformations. The second one, in which the plastic deformation is increasingly more relevant and, the deformation grows quickly in increments of pressure similar to those found in the first phase. The radial pressure extends constantly versus the deformation, so that the shape of the curve is an inclined asymptote.

The circumferential pressure also has two stages, although in this case the pressure has lower values than the initial one. On the other hand, the total vertical pressure remains constant and close to the initial value.

Figure 5b depicts the same representation as the previous one with the three effective pressures close to the interstitial pressure. There is also an elastic initial stage followed by a plastic one for the radial and circular pressures. However, in the latter, the circumferential pressure reached a horizontal asymptotic value. This value reflects the total plastication of the internal rings.

The HS model is a constitutive equation whose function of breakage and plastication criterion can be represented in the plan p–q:

$$p = (\sigma_1 + \sigma_2 + \sigma_3)/3 \qquad (15)$$

$$q = \sigma_1 - \sigma_3 \qquad (16)$$

a)

b)

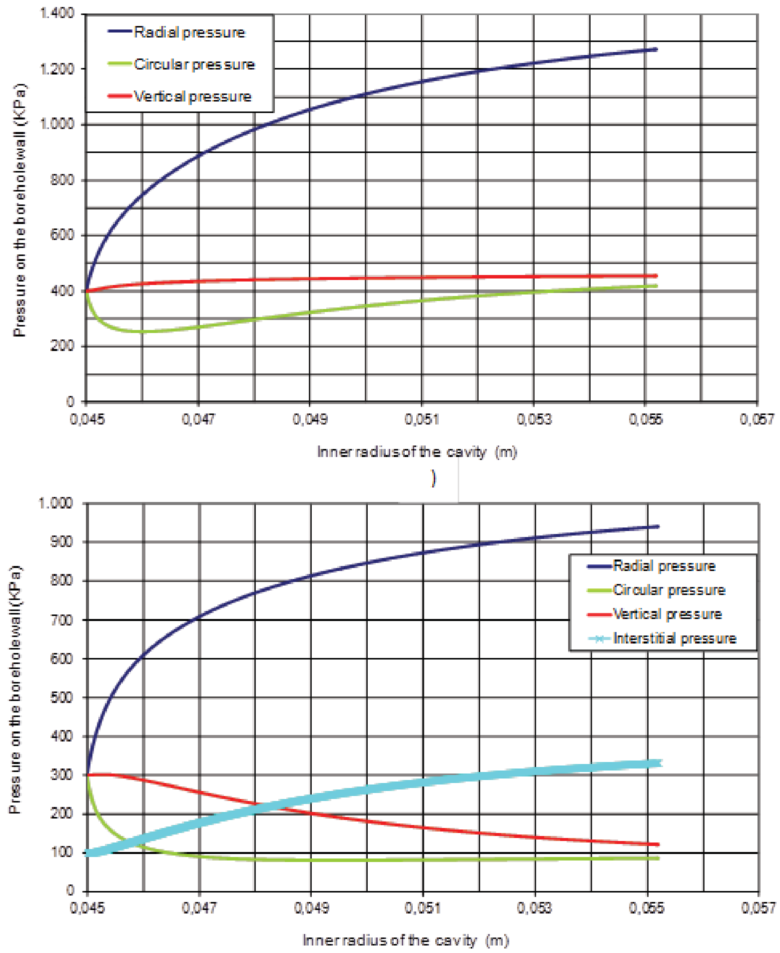

Figure 5. Pressures on the borehole drilling. Total pressures (a) and effective and interstitial pressures (b).

As the function that defines the breakage criterion depends on the previous shear plastic deformation, its representation is a group of curves, the superior limit of which is the q_a value. Figure 6 shows the p and q evolution in the test, together with the q_a limit.

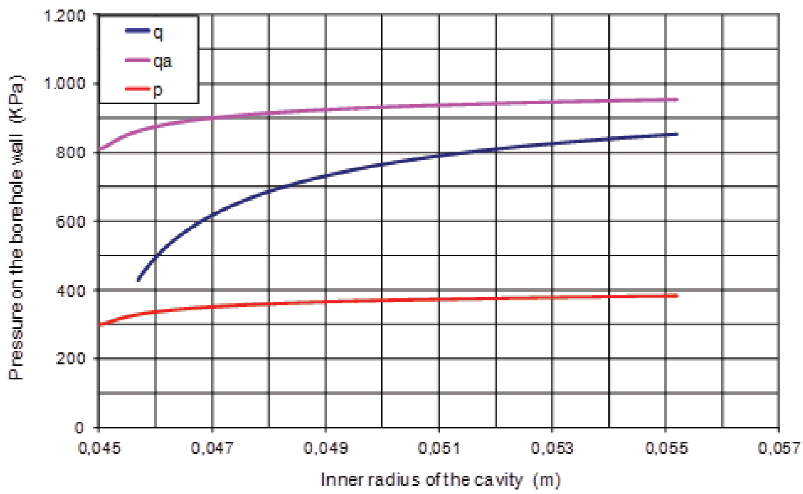

Figure 6. Test p–q graph.

The graph shows how the strain deviator q grows until it becomes asymptotic with the line defined by q_a . The plastic deformations take place along the whole loading process. It must be made clear that the dilatometer test is basically a test of p=constant.

This model allows a study of the evolution of pressures inside the soil. Figure 7 shows this evolution. It is clear that the volume of soil affected by the test does not expand by more than 2.0 m around the borehole. The strain variations between 2.0 m and 10 m of the model are almost null.

The circumferential effective pressure decreases quickly close to the borehole wall. As it moves away from the borehole wall, the circumferential pressure decreases slowly, until it reaches an asymptotic value corresponding to its initial value.

A similar behaviour is observed in the circular and vertical pressure. In the same way, the farther we are from the borehole, the lower the pore-water pressure becomes as it reaches its initial value.

7 COMPARISON BETWEEN THE TEST RESULTS AND THE MODEL

The results of the model have been compared with in-situ tests in the Dueñas unit. Among the thirty-five pressuremeter tests carried out, three have been distinguished. In the first group gypsumy marlstones (TD/y) are tested, in the second one there are clays from hard to very hard from Dueñas clayey layers (TD/m) and, finally, in the third group there are clays of high consistency, also from Dueñas clayey layers (TD/m).

The graphs in Figure 8a and 8b below show the four main tests from those carried out with gypsumy marlstones (TD/y). This material is classified as soft rock. This graph depicts the adjustment with the rings model.

Three of the tests were adapted to the radial total pressures curve and the fourth one is adjusted to the radial effective pressures curve, both in the same calculation. The parameters used in this calculation are presented in the chart below (Table 3).

Table 3. Parameters used in this calculation are gathered in the chart below to a depth of 20 m.

HS-Model Parameters for gypsumy marlstones in Dueñas unit	
Effective cohesion	0.5 Mpa
Internal friction angle	45º
Dilatancy angle	3º
Loading reference module E_{50}^{ref}	400 MPa
Downloading reference module E_{ur}^{ref}	600 MPa
m	0.8
R_f	0.7

Initial state equal to 20 m deep. (*) reference pressure 100 KPa

The test results are analogous to those of the proposed model. Nevertheless, two different behaviours are observed. In the three tests of Figure 10 the radial pressure grows to an inclined asymptotic straight line. This behaviour should belong to a model where the pore-water pressure does not disappear, but it is added to the effective pressure to support the pressure on borehole walls. The adjustment of these experimental curves takes place with the total pressure of the model.

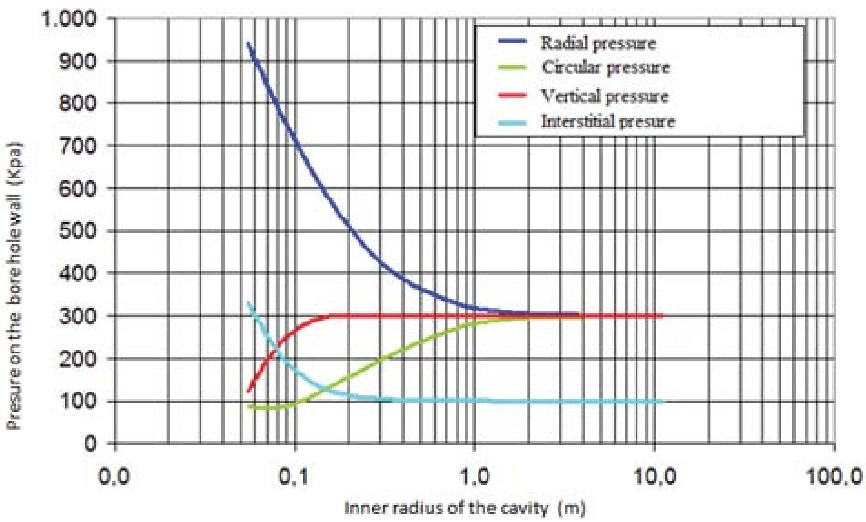

Figure 7. Evolution of pressures inside the soil.

a)

b)

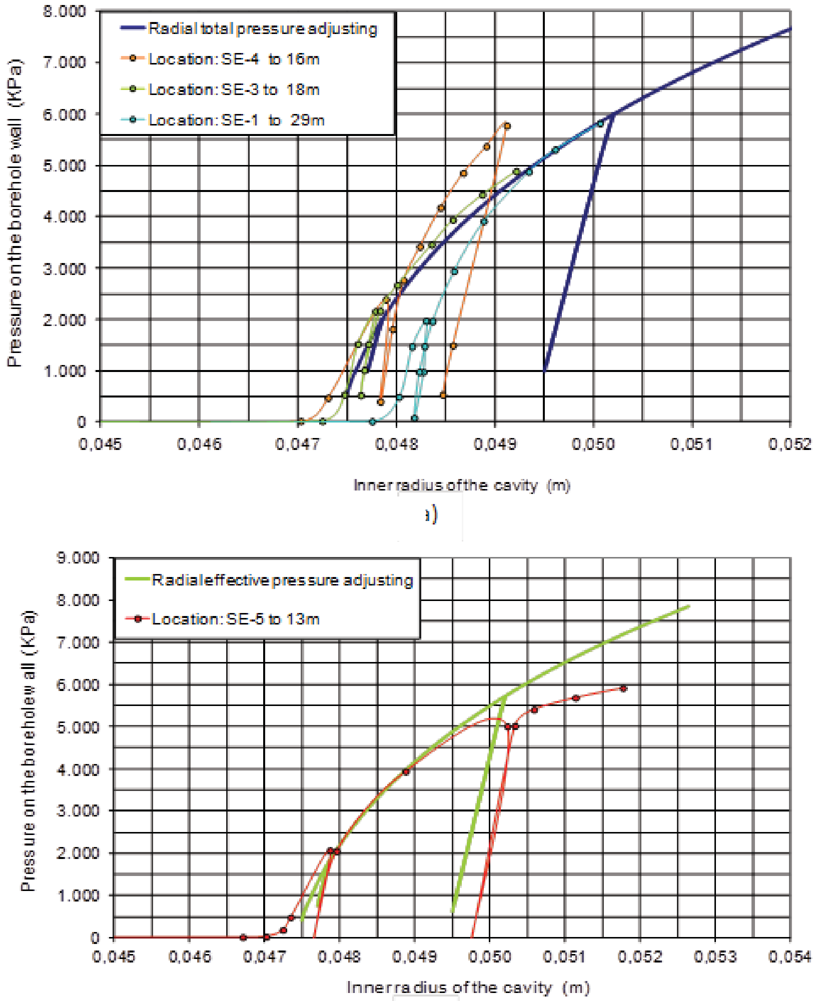

Figure 8. Pressuremeter tests with gypsum marlstones. Total pressures (a) and effective pressures (b).

The test in Figure 8b shows how for pressures over 6 MPa the curve begins to bend to a horizontal straight line. In this case, the test has a drained behavior. The drainage can occur along the loading of the rifting of the rock or a limit to the generated interstitial pressure due to the exhaustion of the confining vertical pressure.

Figure 9 also shows a sequence of pressure pressuremeter tests, which represents those carried out in soil from a

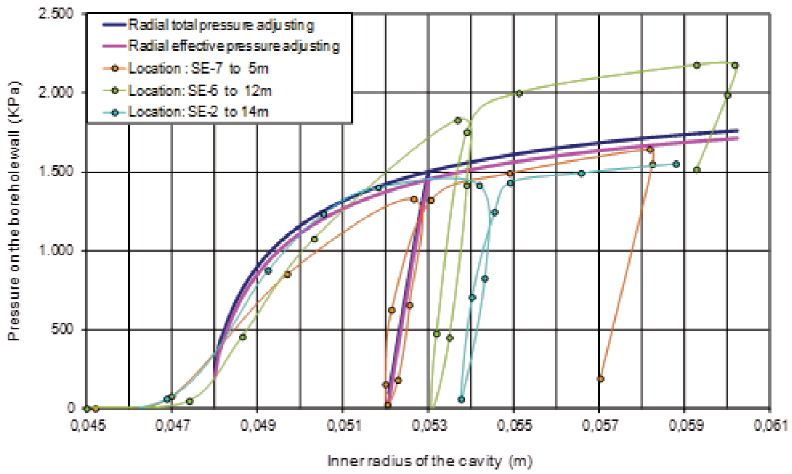

Figure 9. Pressuremeter tests with clays from hard to very hard consistency.

hard to a very hard consistency of the clayey unit (TD/m). Their results are compared to the model adjustment.

The parameters used in this analysis are gathered in the chart below (Table 4).

Table 4. Parameters used in this calculation are gathered in the chart below to a depth of 10 m.

HS-Model Parameters for hard to very hard clays in Dueñas Facies	
Effective cohesion	0.25 Mpa
Internal friction angle	30º
Dilatancy angle	3º
Loading reference module $E_{50}{}^{ref}$	60 MPa
Downloading reference module $E_{ur}{}^{ref}$	90 MPa
m	0.8
R_f	0.7

Initial state equal to 10 m deep. (*) reference pressure 100 KPa

Finally, Figure 10 depicts the tests with soil of firm consistency of the clayey unit (TD/m). Their results are compared to the model adjustment. The parameters used in this analysis are gathered in the chart in next column (Table 5).

8 DISCUSSION

The pressuremeter is a test of horizontal load, which allows us to obtain a detailed account of the main geotechnical parameters of the soil behavior [9]. To obtain these parameters, however, it is advisable to define a soil constitutive equation and a mathematical model

Table 5. Parameters used in this calculation are gathered in the chart below to a depth of 10 m.

HS-Model Parameters for hard to very hard clays in Dueñas Facies	
Effective cohesion	30 Kpa
Internal friction angle	28º
Dilatancy angle	3º
Loading reference module $E_{50}{}^{ref}$	8 MPa
Downloading reference module $E_{ur}{}^{ref}$	15 MPa
m	0.8
R_f	0.7

Initial state equal to 10 m deep. (*) reference pressure 100 KPa

able to carry out the test. The mathematical model was realized with the commercial program Plaxis 8.1 [10].

The natural soil is heterogeneous enough so that the tests done with similar materials are not identical, but they vary within a range of values. In this way the adjustments made with the rings model aim to simulate the average behaviour of the tests carried out with the same material. The *in-situ* tests reflect that the response of the natural soil goes from a drained behaviour without generated interstitial pressures to an undrained one [11]. The adjustment is achieved by choosing a set of parameters that leads to an analytical curve. The set of parameters is modified until the analytical curve matches with the experimental one. Each parameter accounts for a part of the curve. At the same time, typical values of the main parameter for this kind of soil, cohesion and friction internal angles are used.

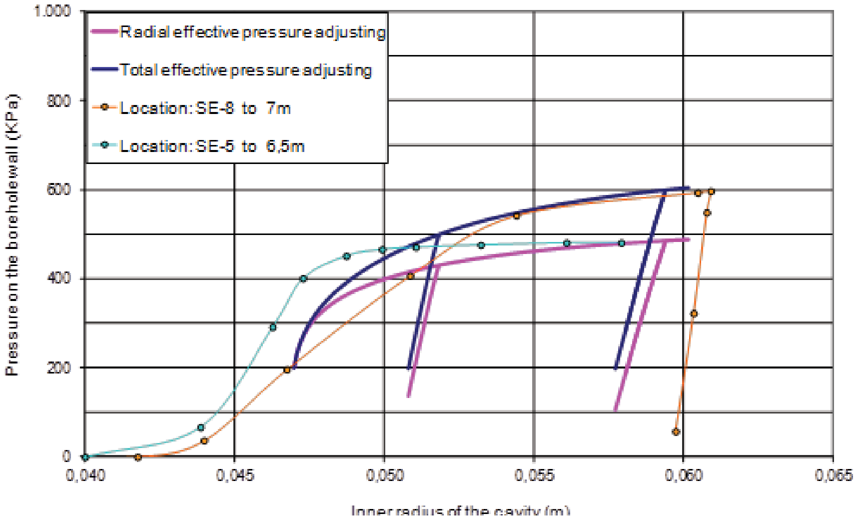

Figure 10. Pressuremeter tests with clays of firm consistency.

The vertical deformation was determined by the hypothesis of constant vertical pressure, which seems to be more appropriate for tests near to the surface. Other hypotheses may be used, such as the null vertical deformation. This model could achieve a better adjustment to the experimental curve.

All these considerations mean that the use of parameters obtained by the test should not be unique, but it should be considered as a range of values for each parameter. The range depends on both the experimental curves and the set of parameters. Small variations of a parameter could also achieve a valid adjustment. Each soil requires a sensitivity analysis of the parameter.

The test must be complemented with other mechanical tests as well as a certain caution in their expected magnitude. This means that the engineer has to use a certain geotechnical common sense when choosing the parameters.

The pressuremeter test develops large deformations close to the borehole wall. The used model must employ a definition of deformation according to its magnitude. The use of the hypothesis for a small deformation requires the modification of the model's geometry as the deformation advances. Because the strain is greater than 1%, and the inner ring deforms a lot, the model has to modified with the ring mesh at each step

The tests carried out in Dueñas Facies have allowed us to test a variety of clayey materials with a variable consistency from firm to very hard. The materials classified as soft rocks have also been tested. This range of materials allows us to quantify the geotechnical parameters of the hardening soil model.

The main parameters in the adjustment are the cohesion, the friction angle and the modules of deformation in load and download. The other three parameters, i.e., exponent, dilatancy angle and *Rf* value, permit a small adjustment in the curve shape defined by the previous parameters.

9 CONCLUSIONS

The pressuremeter test has proven to be a useful tool that completes the geotechnical survey of the soil [10, 11]. As a result of it the geotechnical parameters applied to advanced constitutive equations are also possible. This article shows an application that adjusts the test results with those obtained by the model ruled by the "Hardening Soil Model" constitutive equation. This model was studied [12]. The model adjusts to the *in-situ* tests so it is possible to find geotechnical parameters for the different levels of soil that form Dueñas Facies.

In this way the parameters obtained can be applied to computer programmes that develop these kinds of constitutive equations, as it is an advanced model of soil behaviour designed for foundations and structures developed in Dueñas Facies.

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