Virtual fall cone tests using Discrete Element Method

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ABSTRACT
The Discrete Element Method (DEM) is an important tool to investigate soil-structure interaction problems involving dynamics and large strains. This technique is usually applied to reproduce the behaviour of discrete granular soils but can also be used to simulate the behaviour of continuous materials such as very soft clays, where the soil behaves as a viscous fluid. In that way, a DEM software called VISED, developed based on principles of equilibrium and interaction between the elements, was used to simulate soil behaviour on fall cone tests. Unlike the Finite Element Method (FEM), in the DEM the elements are discontinuous, allowing large strain simulations with no need for remeshing, which is a complex and time spending technique.

RÉSUMÉ
La méthode des éléments discrets (DEM) est un outil important pour étudier l'interaction sol-structure, les problèmes de la dynamique et de grandes déformations. Cette technique est généralement appliquée à reproduire le comportement des sols granulaires discrètes, mais peut être aussi utilisé pour simuler le comportement des matériaux en continu telles que les argiles très molles, où le sol se comporte plutôt comme un fluide visqueux. De cette façon, un logiciel de DEM que l'on appelle VISED, développé sur la base des principes d'équilibre et l'interaction entre les éléments, a été utilisé pour simuler le comportement du sol sur des essais au cône. Contrairement à la méthode des éléments finis (FEM), dans le DEM les éléments sont discontinus, permettant des simulations des grandes déformations sans avoir besoin changer le grid, qui est une technique complexe et qui dépense beacoup de temps.

1 INTRODUCTION

The numerical modelling of soil behaviour is a very important tool to solve engineering problems, although quite challenging regarding some types of soils and loading conditions. That is the case of dry non-cohesive soils, where the material is clearly discontinuous and constituted of independent particles. Another example is the case of very soft clays, where regular loadings are associated with large deformations.

The Finite Element Method (FEM) is by far the most frequent numerical method applied for soil simulation in the academy and industry. However, there are some restrictions associated with the simulation of problems involving discontinuous materials and large deformations.

In that way, the Discrete Element Method (DEM) shows up as a very promising tool due to some particular characteristics. The method is based on a number of non-deformable elements that constitute the material and these elements interact with each other through predefined bond conditions. When a limit condition is reached, the bond is broken and the elements are disconnected.

Thus, regarding situations where large deformations are expected, the numerical simulation is feasible with no need of any remeshing technique or equivalent procedure, once the elements are naturally rearranged in a new configuration.

The aim of this paper is to present a series of virtual tests simulating the free fall cone penetrometer in order to evaluate the performance of the program. Thus, input parameters such as dynamic friction and damping were changed in a specific range to evaluate the influence of this variation in the final results. The final cone penetration in the soil was compared with data from real tests in clays provided in the literature.

2 THE DISCRETE ELEMENT METHOD SOFTWARE – VISED 3.7

The Discrete Element Method visualization software – VISED 3.7, developed in C++ by Minato and Cantini (2009), uses the graphic library OpenGL© capable of generating graphics in real time with many effects such as texture and animation. The OpenGL Utility Tools (GLUT) was used to create special input and output features.

The software was initially developed to simulate materials that behave as discontinuous matter such as non-cohesive soils, but is also being used to emulate very soft soils in large strain conditions.

The numerical method adopted for the VISED 3.7 software was the discrete element method, which deals with each element as a rigid and independent particle. The elements interact with each other through predefined bond conditions.

The software uses the interaction between particles and equilibrium principles to calculate the numerical solution. The developed algorithm combines four main processes: collision between particles, analysis of contact forces between particles, fixations and external forces calculation, new velocities and position calculation.

The collision between elements uses the Munjiza-NBR algorithm, described on Munjiza (2004), which employs direct mapping to detect contact between elements.

The contact forces between elements have a normal and a tangential component. The normal component uses as parameters the normal stiffness (k_n) associated with the elements interpenetration (u_n), and the normal dumping (c_n) associated with the normal component of the relative velocity (v_n) as shown in Equation [1]. The tangential force uses as parameters the minimum value between the tangential stiffness (k_t) and the tangential damping (c_t), both associated with the tangential component of the relative velocity (v_t), and the dynamic friction (μ) associated with the normal force (F_n), as shown in Equation [2].

$$F_n = k_n u_n + c_n v_n \qquad [1]$$

$$F_t = min\left\{k_t \int v_t dt + v_t c_t ; F_n \mu\right\} \qquad [2]$$

The bond conditions are calculated using three properties: the Young modulus (E), yield stress (σ_E) and the strain at rupture (ε_r).

3 VISED 3.7 SOFTWARE INPUT PARAMETERS

The cone penetrometer was simulated using elements with conventional steel properties (Specific weight (γ) = 78,5 kN/m^3; Longitudinal Young modulus (E) = 200 000 MPa; Yield stress (σ_E) = 250 MPa; Strain at rupture (ε_r) = 20%; Dumping ($c_{n,t}$) = 100%; Dynamic friction (μ) = 0.7). These properties are not relevant for this soil-interaction problem once the steel is many times stiffer and stronger than the soil.

For the soil elements, the following parameters were adopted and are resumed in Table 1:

a) Specific weight (γ): constant of 18 kN/m^3;

b) Longitudinal Young modulus (E): this parameter was estimated based on a direct relationship with the undrained shear strength (s_U). Tests 2, 4, 6, 8, 10, 12, 14, 16 and 18 used $E=200s_U$ and the others used $E=100s_U$. The choice for variation between these two relationships had the purpose of evaluating the influence of this parameter in the analysis. The E values varied between 170 to 2,000 kPa;

c) Yield stress (σ_E): a classical relationship between the undrained shear strength and the vertical stress was adopted to calculate the yield stress $\sigma_E=s_U/0.22$

d) Strain at rupture (ε_r): tests 24 to 32 adopted the value 10% whilst the other tests adopted 5%;

e) Dumping ($c_{n,t}$): Iida (1999) proposed a range of values between 2% and 7% for soft clay deposits. Initially, a parametric study of the influence of this parameter in the results of the simulations was undertaken considering dumping values of 2.0%, 4.5% and 7.0% as suggested by Iida (1999). However, a higher series of values was adopted afterwards to help define more clearly the influence of the damping in the virtual tests. (5.0%, 12.5%, 20.0%, 27.5%, 30.0%, 35.0%, 40.0%, 42.5%, 50.0% and 80.0%);

f) Dynamic friction (μ): considering that the dynamic friction in soft clay deposits vary between 0.05 and 0.30, the values 0.05, 0.10, 0.20 and 0.30 were initially adopted. In a second phase, a complementary series of values 0.01, 0.06, 0.07, 0.08, 0.15 and 0.25 were used to allow a better comprehension of the role of this parameter in the results.

Table 1. Parameter values adopted for the numerical analysis.

Model	s_u (kPa)	E (MPa)	ε_r (%)	$c_{n,t}$ (%)	μ
1	10	1.0	5	2.0	0.10
2	10	2.0	5	2.0	0.10
3	10	1.0	5	2.0	0.20
4	10	2.0	5	2.0	0.20
5	10	1.0	5	2.0	0.30
6	10	2.0	5	2.0	0.30
7	10	1.0	5	4.5	0.10
8	10	2.0	5	4.5	0.10
9	10	1.0	5	4.5	0.20
10	10	2.0	5	4.5	0.20
11	10	1.0	5	4.5	0.30
12	10	2.0	5	4.5	0.30
13	10	1.0	5	7.0	0.10
14	10	2.0	5	7.0	0.10
15	10	1.0	5	7.0	0.20
16	10	2.0	5	7.0	0.20
17	10	1.0	5	7.0	0.30
18	10	2.0	5	7.0	0.30
19	20	2.0	5	80.0	0.01

Model	s_u (kPa)	E (MPa)	ε_r (%)	$c_{n,t}$ (%)	μ
20	20	2.0	5	30.0	0.01
21	20	2.0	5	40.0	0.05
22	20	2.0	5	30.0	0.05
23	20	2.0	5	30.0	0.10
24	20	2.0	10	30.0	0.05
25	20	2.0	10	30.0	0.10
26	20	2.0	10	30.0	0.01
27	20	2.0	10	40.0	0.01
28	20	2.0	10	40.0	0.05
29	20	2.0	10	40.0	0.10
30	20	2.0	10	50.0	0.01
31	20	2.0	10	50.0	0.05
32	20	2.0	10	50.0	0.10
33	20	2.0	5	50.0	0.01
34	20	2.0	5	30.0	0.05
35	20	2.0	5	40.0	0.01
36	20	2.0	5	80.0	0.05
37	20	2.0	5	40.0	0.10
38	20	2.0	5	40.0	0.07
39	20	2.0	5	40.0	0.08
40	20	2.0	5	40.0	0.06
41	10	1.0	5	40.0	0.06
42	30	3.0	5	40.0	0.06
43	40	4.0	5	40.0	0.06
44	10	1.0	5	40.0	0.07
45	20	2.0	5	35.0	0.05
46	20	2.0	5	35.0	0.10
47	20	2.0	5	35.0	0.15
48	20	2.0	5	35.0	0.20
49	20	2.0	5	35.0	0.25
50	20	2.0	5	35.0	0.30
51	20	2.0	5	5.0	0.05
52	20	2.0	5	12.5	0.05
53	20	2.0	5	20.0	0.05
54	20	2.0	5	27.5	0.05
55	20	2.0	5	35.0	0.05
56	20	2.0	5	42.5	0.05
57	1.70	0.170	5	35.0	0.05
58	3.36	0.336	5	35.0	0.05
59	5.02	0.502	5	35.0	0.05
60	6.68	0.668	5	35.0	0.05
61	8.34	0.834	5	35.0	0.05
62	10	1.0	5	35.0	0.05
63	20	2.0	5	35.0	0.05
64	22	2.2	5	35.0	0.05
65	24	2.4	5	35.0	0.05
66	26	2.6	5	35.0	0.05
67	28	2.8	5	35.0	0.05
68	30	3.0	5	35.0	0.05
69	12	1.2	5	35.0	0.05
70	14	1.4	5	35.0	0.05
71	16	1.6	5	35.0	0.05
72	18	1.8	5	35.0	0.05
73	1.70	0.170	5	35.0	0.30
74	3.36	0.336	5	35.0	0.30
75	5.02	0.502	5	35.0	0.30
76	6.68	0.668	5	35.0	0.30
77	8.34	0.834	5	35.0	0.30
78	10	1.0	5	35.0	0.30
79	12	1.2	5	35.0	0.30
80	14	1.4	5	35.0	0.30
81	16	1.6	5	35.0	0.30
82	18	1.8	5	35.0	0.30
83	20	2.0	5	35.0	0.30
84	22	2.2	5	35.0	0.30
85	24	2.4	5	35.0	0.30
86	26	2.6	5	35.0	0.30
87	28	2.8	5	35.0	0.30
88	30	3.0	5	35.0	0.30

4 THE FALL CONE PENETRATION TEST

4.1 The laboratory fall cone penetration test

The fall cone penetration is a basic laboratory test where the penetration of a free falling steel cone in a soil sample is measured and related to the water content of the soil when the test is carried out. Thereafter, the Liquidity Limit can be determined as the water content associated with a penetration of 20mm.

The equipment consists of a metallic cone tip, a device to measure the penetration of the cone and a container (55mm diameter by 40mm high) where the sample is placed. The cone point angle α is typically 30°, 60° or 90° and its weight can be 60g or 80g.

The fall cone penetration test was chosen to be modeled because it is a simple and very well known soil laboratory test that correlates directly the water content of the sample with the penetration of the cone. As the strength of clays is a function of the water content, a relationship between the soil strength and the cone penetration can be obtained. This test was also chosen because, as a conventional test, the amount of available data is high and the level of uncertainties is low.

4.2 Relationship between undrained strength and penetration in conventional fall cone tests

Koumoto and Houlsby (2001) presented Equation [3] for clays where the undrained shear strength (s_u) is a function of the liquidity index (I_L).

$$s_u = e^{\frac{1.070 - I_L}{0.217}} \qquad [3]$$

Additionally, Muntohar and Hashim (2002) presented a series of tests correlating the fall cone penetration (*d*) with the liquidity index (I_L) for an artificial clay made of a mixture of bentonite and kaolin.

Based on Equation [3] and the data presented by Muntohar e Hashim (2002) it was possible to derive a direct relationship between the undrained shear strength (s_u) and cone penetration (*d*) presented in Figure 1. Equation [4] shows the logarithm function that best fit the data. Certainly this equation is only valid for the mixture studied by the authors, but it gives a good idea of the behavior of a typical clay.

$$d = -3.933 ln(s_u) + 20.851 \qquad [4]$$

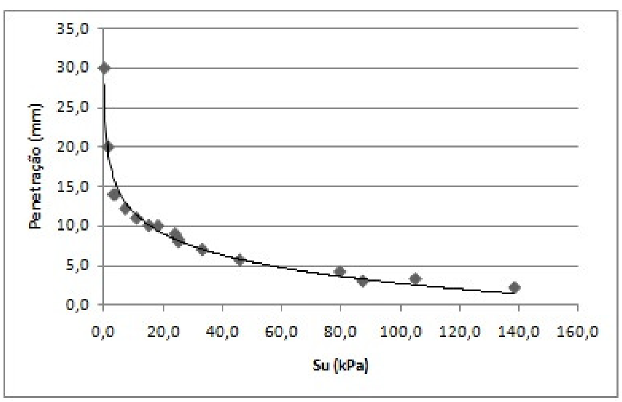

Figure 1. Direct relationship between s_U and *d*.

Apart from Equation [4], Silveira (2001) also proposed another relationship, described on Equation [5], where the product of the undrained shear strength (s_u) by the square of the cone penetration (*d*) is a constant (*α*) for the same soil.

$$s_u = \frac{\alpha}{d^2} \qquad [5]$$

Therefore, it is possible to find, for each soil, the value of the constant *α*. In that way, this equation is a good tool to evaluate the results of the DEM analysis once any series of tests can be checked in terms of consistency by calculating the product of the undrained shear strength by the square of the penetration and verifying how close it is from a constant value.

4.3 The numerical modelling of the cone test

Due to the small size of the test (with an order of magnitude of a few centimeters) VISED was unable to properly process the model. To avoid any divergence between interactions, the required time step should be of the order of nanoseconds, with high computational cost. Thus, it was necessary to adopt an overall scale ten times greater then the real one, thus obtaining a soil sample of 55cm diameter and 40cm in height and a cone 35cm high and α = 30º, as shown in Figure 2.

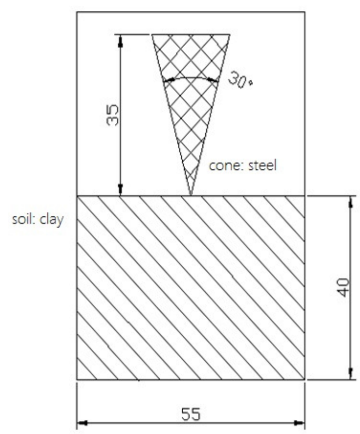

Figure 2. Numerical fall cone penetration test dimensions in cm.

The soil sample used for the virtual test was composed of discrete elements spheres of 4 mm radius, while the cone was composed of discrete elements spheres of 5 mm radius. Figure 3 shows the final prototype model.

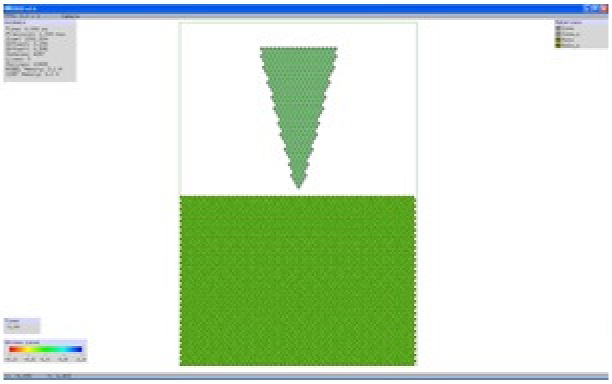

Figure 3. DEM mesh for the fall cone penetration test.

5 NUMERICAL FALL CONE CALIBRATION TESTS

The input parameters used by VISED are based in basic physics and are not directly related to any specific material. These physical parameters are not usually used to model soil behavior. Therefore it was necessary to evaluate the influence of these parameters and compare the numeric results with the real laboratory tests.

Initially, 56 models were carried out with the adopted values already presented in Table 1. The first series of models (even numbers between 1 and 18) assumed the parameter values suggested by Iida (1999) and a Young modulus calculated using $E=200.s_u$. These parameters led to high penetration results which proved not to simulate adequately the real tests.

Regarding the second series of tests (odd numbers between 1 and 18), the value of the Young modulus was calculated using $E=100.s_u$. These results were considered better than the first set of tests since the values were closer to the expected behaviour.

Even though the results were enhanced, further improvement was still necessary to make the displacements closer to reality. Thus, a third series of models (19 to 44) was carried out to determine optimal values of parameters to be used in VISED.

The damping and dynamic friction values were changed, while the s_u values were kept virtually constant in all models, except in models 41 to 44. For those tests, the undrained strength was changed in order to evaluate whether the results obtained in the model 40 was maintained over this variation.

Also very high values (up to 80%) were considered for the damping, with the purpose of evaluating the results in these extreme conditions. The dynamic friction was also varied in the tests, but with values not exceeding the limit of 0.3 (typically between 0.05 and 0.20), once soft clays were the focus of this research. Figure 4 to 5 show the variation range for the final penetration with change in damping, friction and strain at rupture values.

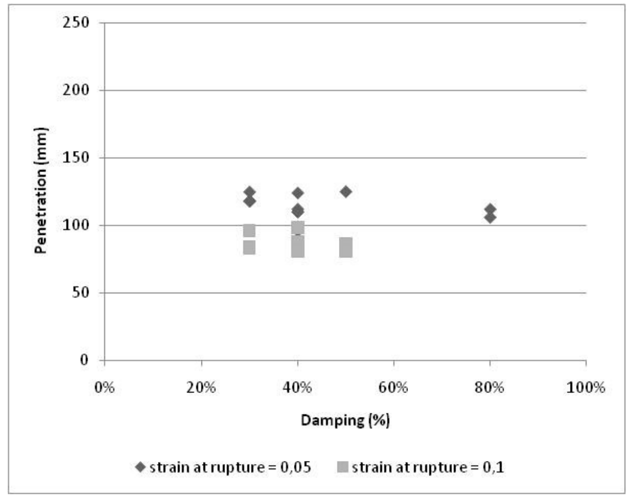

Figure 4. Variation of penetration with damping (models 19 to 40).

The fourth set of test (models 45 to 50 – Figure 6), was carried out in order to evaluate the variation of friction values, when considering the damping (35%) and s_u (20kPa) values constant. The range of friction values adopted varied from 0.05 to 0.30, with a progression of 0.05 per model. It was noticed, however, that the variation of penetration was very small (less than 10%).

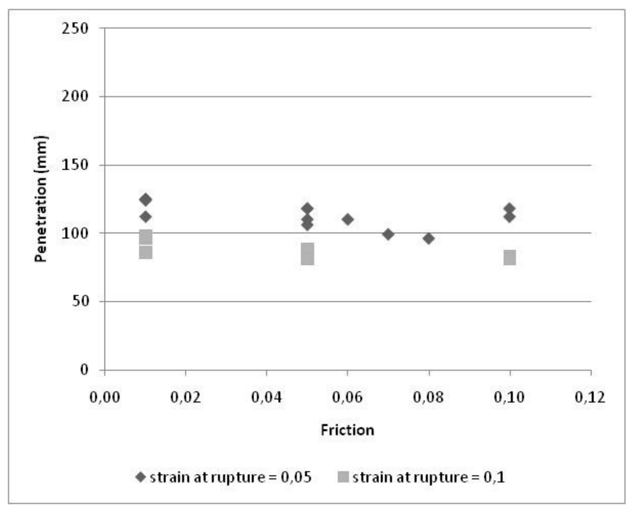

Figure 5. Variation of penetration with dynamic friction (models 19 to 40).

For the fifth series (models 51 to 56 – Figure 7) the values of the damping was changed from 5.0% to 42.5%, keeping constant the values of friction (0.05) and strength (20kPa). There was little variation of the results with the exception of model 51 ($c_{n,t}$=5%), showing a tendency of penetration increase with damping decrease.

Based on the results found on these series of models, it seemed that the most appropriate value for the damping was around 35%. This value was kept constant for the remaining simulations.

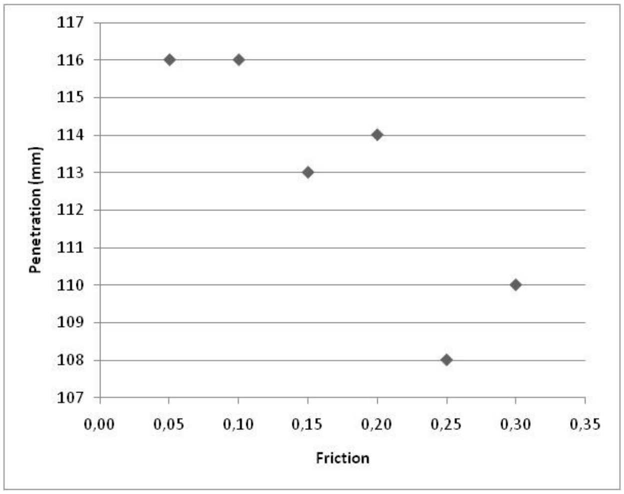

Figure 6. Variation of penetration with friction (models 45 to 50).

The strain at rupture (ε_r) value remained fixed in 5% on most models, once this is an acceptable value for soft soils. Only a small number of models (24 to 32) were studied with a value of 10% to exam whether there was any significant change in behavior.

The specific weight remained constant at 18kN/m^3, as well as the properties of the steel cone, which were adopted as standard parameters considered in the literature for this specific material.

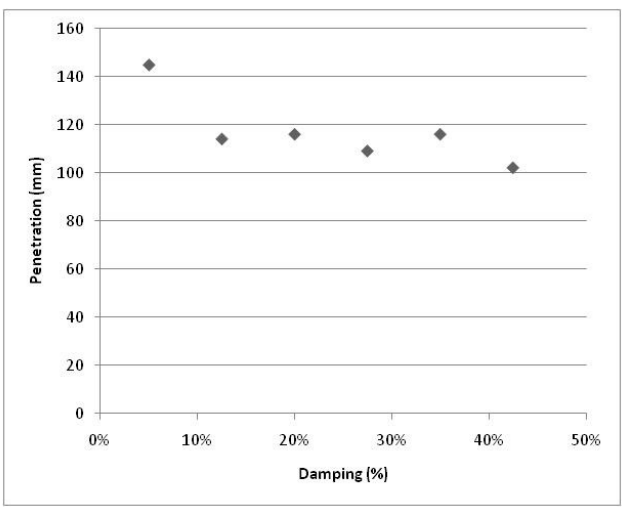

Figure 7. Variation of penetration with damping (models 51 to 56).

At this point it is important to recall that the value of undrained strength is not used directly in the models. It is used only as a basis for calculating the Young modulus and the yield stress.

Figure 8 and 9 exemplify the visual output of models 01 and 10 respectively.

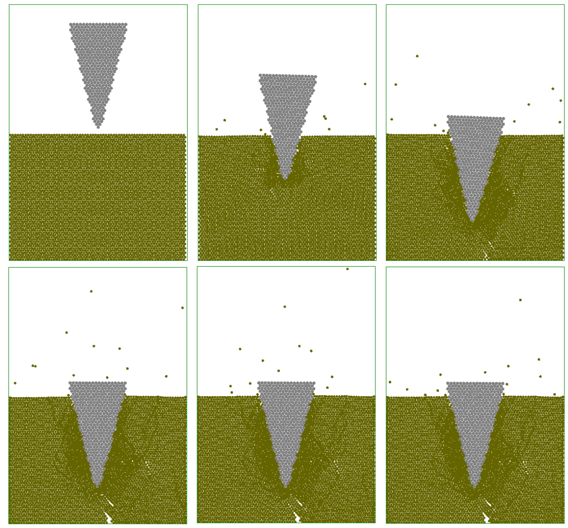

Figure 8. Visual output for model 01.

6 NUMERICAL FALL CONE FINAL TESTS

After the first 5 series of tests to carry out the calibration of the friction, damping and strain at rupture values, thirty-two new models were undertaken and analyzed (57 to 88).

The analysis of the previous calibration models showed that friction and damping have little influence on the variation of penetration for s_u values lower than 20kPa. For the first set of final tests (models 57 to 72) the damping and friction values were set at 35% and 0.05, respectively, while the s_u value varied in the range between 1.7kPa and 30kPa.

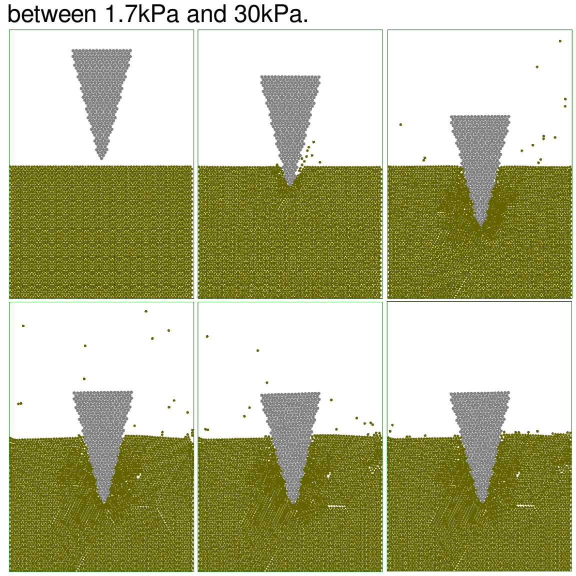

Figure 9. Visual output for model 10.

The tests results, presented in Figure 10, showed that the variation of penetration values with s_u has a logarithmic behaviour, which is in accordance with Equation [4].

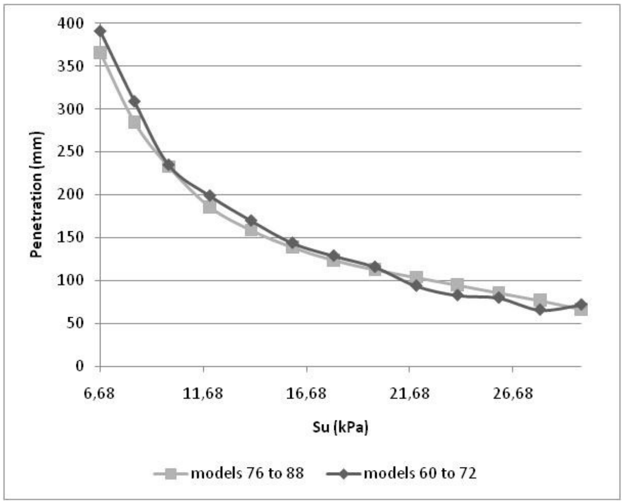

Figure 10. Variation of penetration with undrained strength (models 57 60 to 72 and 73 76 to 88).

The second set of final tests (models 73 to 88) was carried out with the same parameters as the first set fo final tests, except for friction, which had its value set at 0.30. A similar behavior to what was observed in the first set of runs was found. The relationship between cone penetration and undrained strength follows a logarithmic trend.

7 ANALYSIS OF THE RESULTS

The penetration versus s_u curve, obtained by the numerical analysis, presented the same format as the curves found in real laboratory tests. It is important to mention that for very low s_u values, the cone reached the bottom of the container giving a final constant penetration value of 400 mm. This was expected once the falling cone test is not suitable for testing clays with very high water content.

In order to analyze the results from the numerical DEM simulations under the light of Equation [5], proposed by Silveira (2001), the product $\alpha=s_u d^2$ was calculated. The results presented in Table 2 show an α value of around 4,000 for both cases, but with a very high standard deviation, which means some discrepancy from the behavior proposed by Silveira (2001).

Table 2. Analysis of $\alpha=s_u d^2$ value for tests 57 to 88.

Models	Mean α	Standard deviation	Standard deviation (%)
57 to 72	4,029	2,642	66%
73 to 88	3,916	2,307	59%

Figure 11 shows the results of the numerical models 57 to 72 and 73 to 88 in comparison with Equation [4] and Equation [5] with an α constant proposed by Silveira (2001).

Models with undrained strength higher than 20kPa showed better agreement with Equation [4] than for lower values of s_u.

For lower values of s_u, a softer response of the soil behavior produced very high penetrations. While the cone reached the bottom of the container, in the numerical analysis, with undrained strength values around 7kPa, in Equation [4] the bottom is never reached, even for a soil with s_u=0.

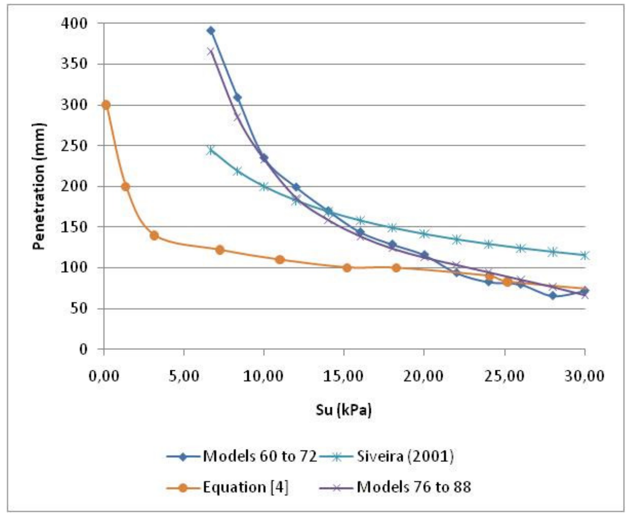

Figure 11. Comparison between the numerical models 57 to 88 and literature references.

These numerical results need more analysis and comparison with other laboratory results, including clays from different sites with different characteristics.

8 CONCLUSIONS

The DEM proved to be a feasible tool to simulate numerically the behaviour of soft soils, particularly in problems where large deformations are expected.

In that way, the data from numerical analysis of the fall cone test show promising results with an overall behavior similar to the real test.

Certainly much more analysis and adjustments are required to turn this tool into a reliable and powerful way to simulate soft clay behavior.

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