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Identifying parameters of a discrete numerical model of soil from a geotechnical field test

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Abstract

A discrete numerical model of a pressuremeter test was developed in this paper. The model is constituted of spherical particles with a contact law involving an inter-particle rolling resistance in addition to the classical Coulomb friction model. The model is calibrated from a pressuremeter test performed in the Hostun sand. An adaptive size of discretization of the soil domain is considered to limit the computational cost. Different parameter sets are defined to identify the initial density of the numerical granular assembly. Then, each parameter set leads to a different calibration of the mechanical parameters of the model in order to reproduce the pressuremeter test results. Simulations show that experimental data from a pressuremeter test alone are not enough to fully calibrate the discrete model. Other laboratory test results need to be considered in the calibration methodology. Eventually, the prediction capability of the model is assessed by comparing the simulated constitutive responses to those measured experimentally on different triaxial loading paths.

Keywords:

Discrete element method, Pressuremeter test, Boundary value problem, Granular material

1. Introduction

Since the introduction of the Discrete Element Method (DEM) by Cundall and Strack (1979) to model the behavior of granular materials, this method has been widely applied in geomechanics. Numerical experiments have been done in order to study the constitutive behavior of geomaterials at a representative elementary volume (REV) scale (e.g. Bardet 1994; Ciantia et al. 2016) as well as the underlying mechanism at the grain scale (for instance Iwashita and Oda 2000; Rajaï et al. 1996). DEM models have also proven to be able to predict not only qualitatively but also quantitatively the behavior of soils even once subjected to complex loading paths (Sibille et al., 2019). Eventually, geotechnical engineering problems are addressed with a discrete approach which is rather efficient when large deformations are expected or failure and post failure responses are investigated (Li et al., 2019; Villard et al., 2016). When

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10 it comes to the calibration of a DEM model, this usually requires a characterization of the soil mechanical properties
11 through laboratory experiments such as triaxial or direct shear tests. However, such experiments are not necessarily
12 representative of the properties of the in-situ soil. Most often, in an engineering context, mechanical properties of
13 soils are deduced from field tests such as the static penetrometer or the pressuremeter tests. It is therefore important
14 to propose a methodology for calibrating DEM models based on field tests.

15 Identifying model parameters directly from field tests has been already investigated in the context of continuum
16 modeling with elasto-plastic constitutive relations (Boubanga, 1990; Bahar, 1992; Zentar et al., 2001). One of the
17 conclusion from these studies is that the set of model parameters to simulate the field test results is not unique as
18 found by Fahey and Carter (1993), and a choice should be made among the different parameter sets able to reproduce
19 the field test data. For an elastic-perfectly plastic model involving eight mechanical parameters, Biarez et al. (1998)
20 suggested to use information from laboratory tests, like the triaxial test, in addition to pressuremeter data in order to
21 be able to fully calibrate the model.

22 Concerning the DEM approach, numerical models of field tests as cone penetration tests and pressuremeter tests
23 have already been developed (Butlanska et al., 2014; Huang and Ma, 1994; Hosseini-Sadrabadi et al., 2019; Geng,
24 2010). However, the parameters of such numerical models are usually calibrated from laboratory test results and the
25 field test model is used as a tool to investigate and characterize the soil response to these field tests. Identification
26 of discrete model parameters from field tests has been poorly investigated and there is still no clear methodology to
27 achieve the parameter identification by this way. Therefore, the main objective of this paper is to study the relevance of
28 defining a calibration methodology of a DEM model from field tests. The pressuremeter test is chosen for this study
29 because it provides a wider range of information about the mechanical soil characteristics than a cone penetration
30 test. The pressure-volume curve resulting from the pressuremeter test provides information about the stress-strain
31 relationship before the soil failure and at the failure state, whereas cone resistance in conventional penetration state
32 informs on the soil shear strength (failure) only.

33 This paper is organized as follows : Section 2 presents the reference pressuremeter test and the used soil. Then,
34 in Section 3, the discrete element model, and in particular the inter-granular contact laws, are defined. Preparation of
35 the granular assembly as well as the DEM model of the pressuremeter test (boundary conditions and probe inflation)
36 are also discussed in this section. Thereafter, in Section 4, an investigation of the effects of the model parameters is
37 done through a parametric study. In particular, the study focuses mainly on the influence of the initial porosity and
38 the plastic parameters (contact friction angle φ_c and rolling friction coefficient, η_r), while the elastic parameters are
39 presumed as they have a minor impact on the results. Subsequently, in Section 5, the criteria for choosing the initial
40 porosity and the definition of the calibrating methodology are provided. The result obtained from the calibration of the
41 model parameters is eventually displayed. Section 6 is devoted to the definition of the triaxial loading paths considered
42 for the validation of the model. Prediction capability of the model, based on the suggested calibration, is assessed
43 by comparing the simulated constitutive responses to the responses measured experimentally on these loading paths.
44 Finally, main findings and conclusions are discussed in Section 7.

2. Reference pressuremeter test and soil

The reference pressuremeter test considered in this study is from a series of tests performed by Mokrani (1991) in Hostun sand RF within a calibration chamber under controlled conditions. The calibration chamber is 120 cm in diameter and 150 cm high and configured to apply oedometric boundary conditions (the vertical stress is imposed via an inflatable diaphragm at the top of the chamber while the vertical circular wall is fixed). The pressuremeter is constituted of a mono-cell probe with a diameter of 55 mm placed at the centre of the calibration chamber. The inflatable length of the probe is 160 mm. Soil sample is prepared by dry pluviation of the sand and the probe is buried during the sand deposition. Consequently, a direct contact between the membrane of the probe and the sand is established. After applying the specified axial stress on the soil deposit, the probe's membrane expands by fluid injection at a flow rate of $10 \text{ cm}^3/\text{min}$, which corresponds to a volume variation speed of 2.6 \%/min . The expansion stops when the probe's volume has doubled. A detailed description of the test procedure can be found in Mokrani (1991).

Hostun sand RF is composed of sub-angular to angular particles with a nearly uniform grain size distribution (Figure 1). The median grain size is $D_{50} = 0.35 \text{ mm}$ and the coefficient of uniformity is $C_U = 1.8$. The maximum and minimum void ratios are respectively $e_{max} = 0.961$ and $e_{min} = 0.624$ (Flavigny et al., 1990). The grain size distribution of Hostun sand is presented in Figure 1.

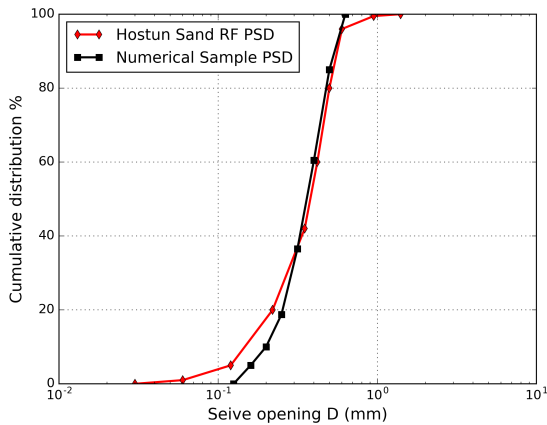


Figure 1: Particle size distribution of the Hostun sand RF used for the pressuremeter test in the calibration chamber. For comparison, the particle size distribution used for the discrete model of the pressuremeter test is also displayed

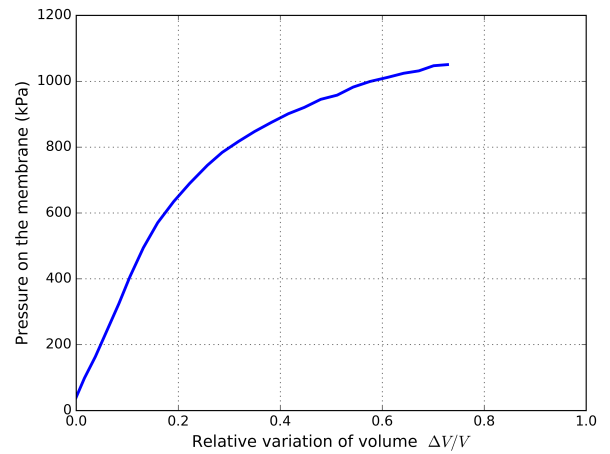


Figure 2: Pressuremeter curve of the experimental test n°11

In this study, a dense sand sample with a relative density $D_r = 83\%$ under a vertical stress equal to $\sigma_v = 100 \text{ kPa}$ (test n°11 from the series of Mokrani (1991)) is chosen to calibrate the discrete model. The pressure-volume curve obtained during the expansion of the probe is shown in Figure 2. In this figure, the pressure represents the pressure applied by the membrane of the probe on the soil and the volume is the volume of fluid injected in the probe (i.e.

65 the volume growth of the probe with respect to its initial volume). The absence of the re-establishing phase in the
 66 pressuremeter curve, representing the initial expansion of the probe until its full contact with the boundaries of the
 67 borehole when the test is performed *in situ*, is due to the direct contact between the probe and the soil thanks to the
 68 sand deposition around the probe.

69 3. Discrete numerical model

70 3.1. Inter-particle contact law

71 The soil is represented by an assembly of spherical particles. The interaction contact law includes Coulomb
 72 friction (Figure 3) characterized by three parameters: the normal stiffness, k_n ; the tangential stiffness, k_t ; and the
 73 contact friction angle, φ_c . When two spheres are in contact, normal contact force, \vec{F}_n , and tangential contact force, \vec{F}_t ,
 74 to the contact plane, are calculated such that:

$$75 \quad \vec{F}_n = k_n \delta_n \vec{n} \quad (1)$$

$$\Delta \vec{F}_t = -k_t \Delta \vec{u}_t \quad \text{with} \quad \|\vec{F}_t\| \leq \|\vec{F}_n\| \tan \varphi_c \quad (2)$$

76 where \vec{n} is the normal unit vector to the plane of contact. Contact detection depends on the overlap, δ_n , between
 77 two particles. So when there is no overlap, the contact is lost. The normal contact force is directly proportional
 78 to the overlap, δ_n , through the normal stiffness, k_n . \vec{F}_t is computed in an incremental form because of the non-
 79 linearity introduced by the Coulomb friction. In the elastic regime, it is directly proportional to the relative tangential
 80 displacement, $\Delta \vec{u}_t$, at contact point through the tangential stiffness, k_t .

81 Rolling resistance is introduced in addition to Coulomb friction in order to limit the excessive rolling of the
 82 oversimplified spherical particles and to reflect the soil grain's angularity. Representation of real grain shape, with
 83 for instance polyhedral particles, requires an important additional computational cost. Spherical particles with rolling
 84 resistance constitutes an alternative solution to describe correctly the plastic macroscopic behaviour of soils as shown
 85 in Sibille et al. (2019) while keeping a reasonable computational cost. Rolling resistance at contact is defined by the
 86 rolling stiffness, k_r , and the rolling friction coefficient, η_r , as represented in Figure 3B. The rolling moment \vec{M}_r to limit
 87 the relative rolling rotation of particles at contact, $\vec{\theta}_r$, is then determined as follows:

$$\Delta \vec{M}_r = -k_r \Delta \vec{\theta}_r \quad \text{with} \quad \|\vec{M}_r\| \leq \|\vec{F}_n\| \eta_r \min(R_1, R_2) \quad (3)$$

88 where R_1 and R_2 are the radii of the two particles involved in the contact respectively.

89 The contact stiffnesses k_n , k_t and k_r are defined in such a way that the macroscopic elastic characteristics of the
 90 particle assembly are independent of the particle size. k_n is proportional to the harmonic mean of the two particles'
 91 radii, R_1 and R_2 via the stiffness modulus, E_c . k_t and k_r are computed depending on k_n and the dimensionless tangential
 92 and rolling stiffness coefficients, α_t and α_r , respectively:

$$93 \quad k_n = 2E_c \frac{R_1 R_2}{R_1 + R_2}; \quad k_t = \alpha_t k_n; \quad k_r = \alpha_r R_1 R_2 k_t \quad (4)$$

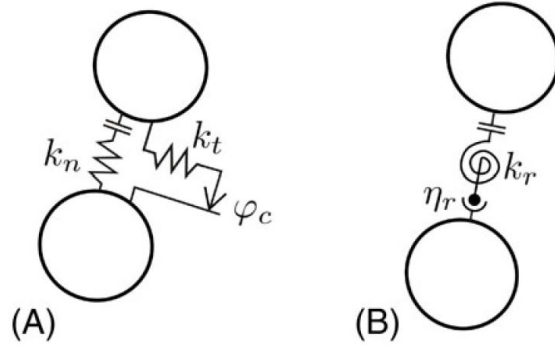


Figure 3: Inter-particle contact model with Coulomb friction (A) and rolling resistance (B).

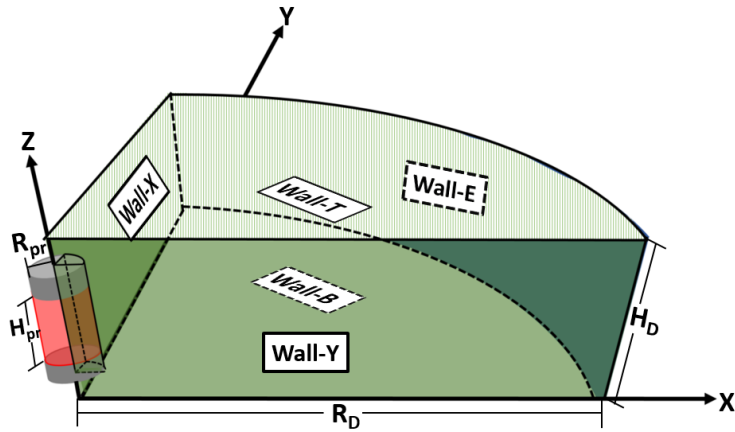


Figure 4: Schematic diagram of the pressuremeter model, the red zone represents the inflatable cell of the pressuremeter and the green zone is filled with the particles representing the soil.

94 3.2. Boundary conditions and probe inflation

95 The numerical model of the pressuremeter test has been developed with the open-source software YADE (Šmilauer
 96 et al., 2015). The geometry of the pressuremeter probe in the calibration chamber presents a rotational symmetry with
 97 respect to the vertical axis. Consequently, the size of the simulation domain could be limited to a given angular sector
 98 of the real problem in order to reduce the computational cost. However, a previous study (Hosseini-Sadrabadi, 2019)
 99 shown that the simulation domain should not be smaller than a quarter of the real problem to avoid artefact on the
 100 interaction between the granular assembly and the intruder (as a penetrometer tip or here the pressuremeter probe).
 101 Therefore, only a quarter of the domain is represented numerically as shown in Figure 4. This choice is validated
 102 in the next section by comparison with a model where the whole domain is represented. Rigid walls (walls X, Y,
 103 B, T and the cylindrical wall E as labelled in Figure 4) are used for modeling the outer boundaries of the simulation
 104 domain. Limit conditions related to the calibration chamber are reproduced by fixing the walls X, Y and E, while
 105 the bottom and top walls, B and T respectively, can translate vertically to impose the vertical stress on the granular

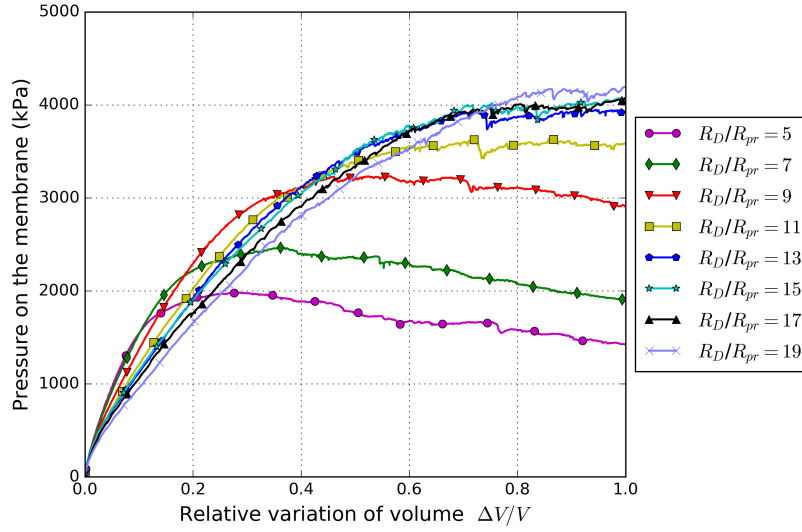


Figure 5: Simulated pressuremeter curves from a dense initial granular packing for different ratios of the radius of the simulation domain to the probe radius (R_D/R_{pr}).

106 assembly thanks to a closed-loop control.

107 The model of the pressuremeter probe is constituted of a top and a bottom cylindrical parts (in grey in Figure 4)
 108 which are fixed, and a cylindrical wall (in red in Figure 4) representing the inflatable membrane of the probe. The
 109 probe's expansion is simulated by increasing the radius of this cylindrical wall up to doubling its internal volume, as
 110 for the experimental test.

111 Contact between soil particles and external walls is considered perfectly smooth and contact friction angle and
 112 rolling friction are set to zero. By this way the impact of the external boundaries on the simulated response is limited.
 113 For the contact between the probe and the soil, contact friction angle is also set to zero but rolling friction is kept at
 114 the same value as in the soil bulk.

115 Modeling a boundary value problem requires a critical choice of the size of the simulation domain. On one hand,
 116 if the size is huge, a significant number of particles is needed, which is computationally expensive; on the other hand if
 117 the size is too small the impact of virtual boundaries on the simulated response may not be negligible. This is typically
 118 the case of the radius R_D of the external cylindrical wall (Wall-E). Therefore, results of simulations of probe inflation
 119 for different values of the ratio R_D/R_{pr} (where R_{pr} is the radius of the probe) are presented in Figure 5. Simulations
 120 were performed with the parameters detailed further in the article in Table 1 (with φ_c and η_r fixed at 19° and 0.55
 121 respectively) and from a very dense initial state of the granular assembly. It is worth saying that the effect of the ratio
 122 R_D/R_{pr} is more important in dense samples than a loose one (Wesley, 2002). Pressuremeter curves, and in particular
 123 the limit pressure p_l (i.e. the pressure reached for a doubling of the probe volume), are almost independent of the ratio
 124 R_D/R_{pr} when it is greater than 13. A ratio of 17 is considered in the following of this study.

125 Effect of the height of the simulation domain and of the slenderness of the probe were also investigated through
126 the ratios H_D/H_{pr} and H_{pr}/R_{pr} respectively (where H_D is the height of the simulation domain and H_{pr} the height of
127 the probe). Again, simulated pressuremeter curves are insensitive to these ratios for $H_D/H_{pr} \geq 3.25$ and $H_{pr}/R_{pr} \geq 2$
128 (For the sake of concision plots of the results are not displayed). These values are the one used in the following steps
129 (even if experimentally the probe slenderness is 2.9).

130 3.3. Preparation of the granular assembly

131 In order to limit the number of particles and to optimize the computational cost, an adaptive discretization
132 (Hosseini-Sadrabadi et al., 2019) of the soil medium into discrete particles is implemented. A refine discretization
133 with a relatively low mean size of particles is performed in the neighborhood of the pressuremeter probe where de-
134 formations, at the macroscopic scale, are important and relative displacements and rearrangements of the particles, at
135 the microscopic scale, play a key role. Then, the mean particle size increases as the distance from the probe increases,
136 such that the coarser particles are defined close to the cylindrical wall E, where almost reversible deformations are
137 expected, represented by an elastic response of the inter-particle contacts. The spatial rate of change of the mean
138 particle size is limited such that the Terzaghi filter criterion holds at any point of the domain in order to avoid the
139 migration of the finer particles close to the probe towards the outer cylindrical wall E where particles are coarser. It
140 is worth noting that such a variable size of discretization of the medium requires a definition of the contact stiffnesses
141 relative to the radii of the particles involved in a contact (cf. Equation 4), making the effective macroscopic properties
142 of the granular assembly independent of the particle size.

143 At any point the numerical particle size distribution follows the one of the Hostun sand, except that particles smaller
144 than 0.125 mm and greater than 0.63 mm (with respect to the real soil grading) are discarded in order to reduce the
145 computational time. The numerical particle size distribution, scaled with respect to the mean particle size of the Hos-
146 tun sand is compared with the experimental one in Figure 1.

147 Eventually, the soil domain is discretized using 33,000 particles with a ratio $R_{pr}/R_p = 18$ (where R_p is the local mean
148 particle radius) at the vicinity of the probe, decreasing down to $R_{pr}/R_p = 4.8$ close to the cylindrical wall E, after the
149 compaction process. The resulting discrete model, after compaction of the granular assembly detailed here after, is
150 presented in Figure 6.

151
152 The granular assembly is generated from the creation of a loose cloud of particles with a relative distribution of
153 radii as explained above. The compaction is composed of two steps as represented in Figure 7. There is an initial
154 compaction by particle growing which is rather isotropic, followed by a vertical compression as performed experi-
155 mentally. The first compaction by expanding the radius of the particles (Chareyre et al., 2002) is used to compact the
156 assembly until reaching a relatively low mean pressure of 30 kPa (Figure 7a). During this phase the contact friction
157 angle is tuned in order to reach the desired porosity after compaction (Figure 7b). Lower contact friction angles re-
158 sults in denser granular assembly. At the end of this step, the contact friction angle is changed to the nominal one (i.e.

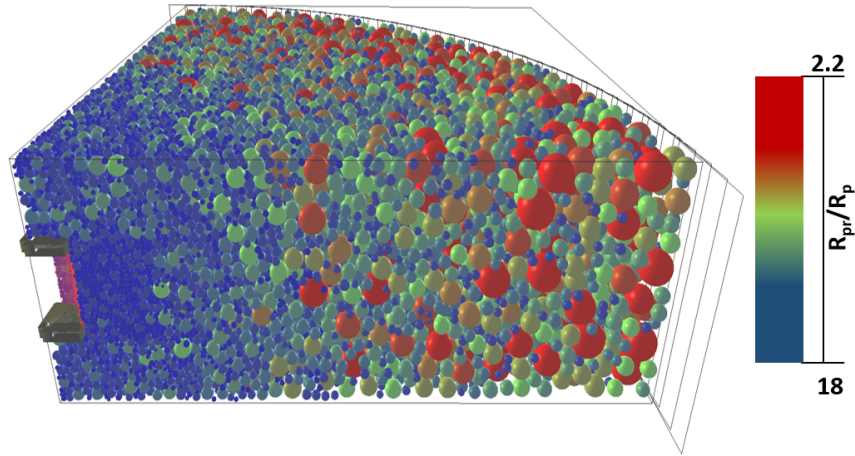


Figure 6: Discrete element model of the pressuremeter test with an adaptive size of discretization of the soil domain.

159 the contact friction angle used to simulate probe inflation). Experimentally there was no isotropic confining pressure
 160 applied by the calibration chamber on the sand but only a vertical stress. Therefore, the confining pressure generated
 161 by the expansion method is removed by reducing the size of the particles at a very slow rate until the mean pressure is
 162 almost nil (1 kPa), and the assembly is compressed vertically up to the desired vertical stress of 100 kPa, as imposed
 163 experimentally (Figure 7a) and until reaching an unbalanced force ratio (ratio of the mean resultant particle force to
 164 the mean contact force) less than 10^{-3} (Figure 7b). This state is considered sufficiently close to the static equilibrium
 165 and constitutes the initial state for the simulation of the probe inflation.

166
 167 Two validations are presented here after regarding respectively the adaptive discretization implemented in the
 168 model and the representation of a quarter of the domain only. First, the homogeneity of the porosity of the granular
 169 assembly built with the adaptive discretization is compared with the porosity obtained from the same model, but
 170 with a constant mean value of the particle radii, such that $R_{pr}/R_p = 4$ in the whole simulation domain. Figure 8
 171 shows the local porosity versus the dimensionless distance from the rotational axis of symmetry R/R_{pr} (where R is
 172 the radial distance from the axis of symmetry) for the two kinds of discretization. The local porosity is calculated in
 173 a given volume of investigation centered on the point of measurement. There is a similar low scattering of porosity
 174 as the distance from the probe increases for both cases. This scattering tends to be slightly more important far from
 175 the probe with the adaptive discretization as particles are coarser. This induces a stronger variability of the porosity
 176 measurement for the same size of volume of investigation. However, the homogeneity of the granular assembly made
 177 with the adaptive discretization remains satisfactory when compared to the non adaptive case.

178 Concerning the representativeness of considering a quarter of the domain instead of the whole one, the pressure-
 179 volume curve simulated from a quarter of the domain is compared in Figure 9 with the one simulated from the whole
 180 numerical description of the domain. In the latter case 130,000 particles were used to discretize the soil instead of

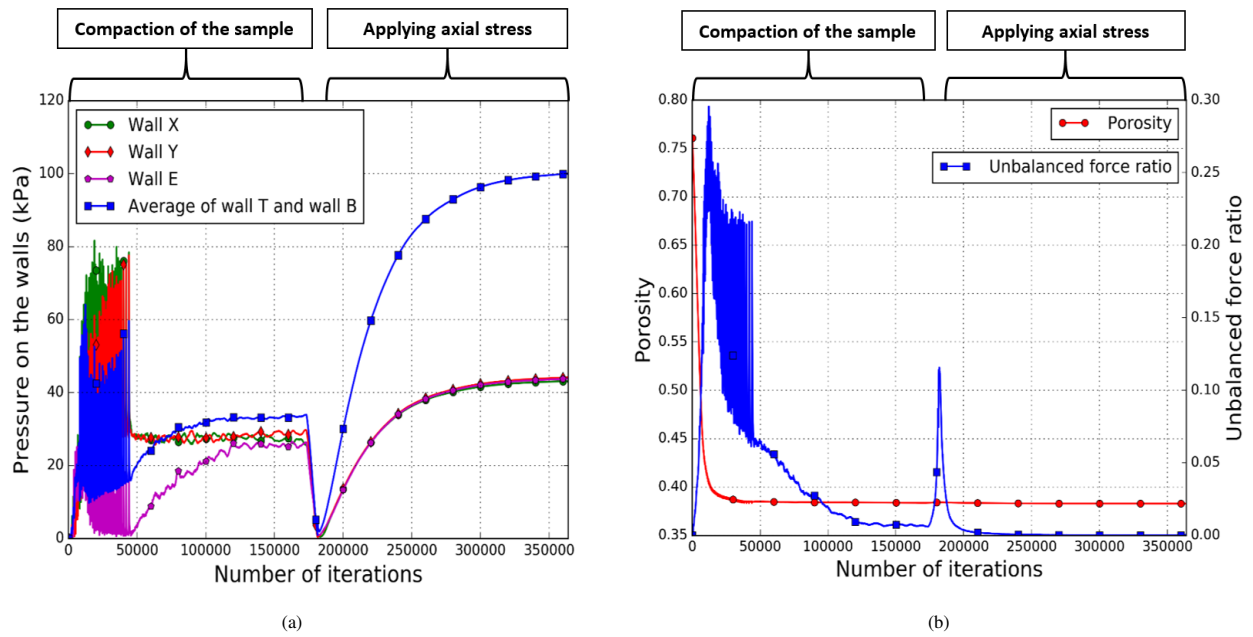


Figure 7: Preparation of the granular assembly by an initial compaction of the granular assembly followed by the application of the vertical stress: pressures on the boundary wall of the simulation domain (a) and porosity and unbalanced force ratio (b).

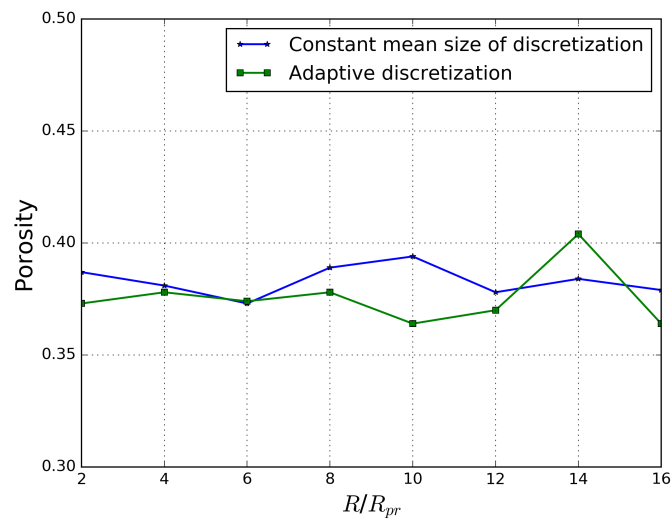


Figure 8: Comparison of the local porosity of granular assemblies built with and without the adaptive discretization technique.

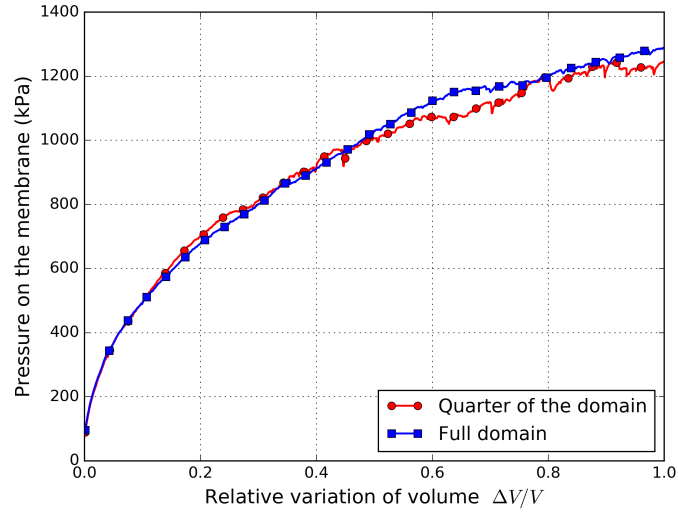


Figure 9: Comparison of pressuremeter curves simulated from the numerical representations of a quarter of the soil domain and of the full domain respectively.

181 33,000 particles for the representation of the quarter of the domain as shown in Figure 6. The model parameters are
 182 the same for both configurations (with φ_c and η_r fixed at 19° and 0.55 respectively, and initial porosity n_0 equal to
 183 0.380). Both simulations give quasi identical results showing that representing a quarter only of the problem with
 184 rotational symmetry, and the associated boundary conditions, is enough to model the pressuremeter test.

185
 186
 187
 188
 189

190 4. Parametric study

191 In this section, the effects of the parameters of the discrete model on the simulated pressure-volume curve are
 192 investigated with the aim of defining a methodology for the calibration of the numerical model from the pressuremeter
 193 test. Both mechanical parameters related to the inter-particle interaction law and the initial density of the granular
 194 assembly are considered. Table 1 gives the values of the initial porosity n_0 and the contact parameters used for this
 195 parametric study.

196 For the sake of conciseness, the parametric study focuses on the role played by the “plastic” contact parameters
 197 (i.e. the contact friction angle φ_c and the rolling friction coefficient η_r) and the initial porosity. Elastic contact pa-
 198 rameters are chosen here sufficiently high to approach the limit of rigid particles (Roux and Combe, 2010; Roux and

Table 1: Contact parameters and initial porosity used for the parametric study.

	E_c (MPa)	α_t	α_r	φ_c (deg)	η_r	n_0
Case 1	500	0.3	5	10-40	0.55	0.368
Case 2	500	0.3	5	19	0.01-0.9	0.352
Case 3	500	0.3	5	19	0.55	0.329-0.401

199 Chevoir, 2005) and avoid by this way an interdependence between local elastic stiffnesses and macroscopic plastic
 200 properties of the granular assembly (Aboul Hosn et al., 2017; Zhao et al., 2018). In these conditions, the impact of
 201 the elastic contact parameters on the pressure-volume curve is rather limited and is not displayed here. Note that the
 202 value of E_c could be tuned to reproduce the slope of the pressure-volume curve at small deformations as long as it
 203 stays close to the rigid particle limit. Initial slope of the pressure-volume curve could also be affected by the initial
 204 connectivity of the granular assembly depending on its preparation methodology (Sibille et al., 2021).

205
206

207 4.1. Sliding and rolling friction

208 Contact friction angle, φ_c , and rolling friction, η_r , representing the dry friction at contact and the shape of the parti-
 209 cles respectively, are considered as the primary parameters of the model that control the macroscopic plastic response
 210 of the granular assembly. Their impact on the pressure-volume curve is investigated by performing simulations of
 211 probe inflation with the parameters identified as "Case 1" in Table 1, to test values of φ_c ranging from 10° to 40° , and
 212 as "Case 2" to test values of η_r from 0.01 to 0.9.

213 Besides, contrary to the physical pressuremeter test, where the volumetric deformation of the soil cannot be
 214 measured during the probe inflation, the discrete numerical model provides the access to this information. Hence, the
 215 mean porosity of the granular assembly in the direct vicinity of the probe is calculated in a 3D window of investigation
 216 with the same height as the probe with a length and width equal to $1.6 R_{pr}$.

217 Figures 10 and 11 present the pressure-volume curves and the changes in porosity near the probe for the parametric
 218 studies on φ_c and η_r respectively. The contact friction angle impacts both the limit pressure and the volumetric
 219 deformation of soil, with an important increase in the limit pressure from 600 kPa (for $\varphi_c = 10^\circ$) to 7000 kPa (for
 220 $\varphi_c = 40^\circ$) (Figure 10a) and of the soil dilatancy (Figure 10b) with φ_c . The rolling friction affects the limit pressure
 221 similarly to φ_c , with a higher limit pressure (from 800 kPa to 3300 kPa) for a larger rolling friction (from 0.01 to
 222 0.5). However a saturation is observed for $\eta_r > 0.5$ where the limit pressure is independent of η_r beyond this threshold
 223 (Figure 11a). Contrary to φ_c , the volumetric response of the soil depends only weakly on η_r (Figure 11b). These results
 224 are in agreement with the effects of the rolling friction on the simulated responses to drained triaxial compressions
 225 as described for instance in Aboul Hosn et al. (2017): the peak shear strength increases with η_r but then tends to

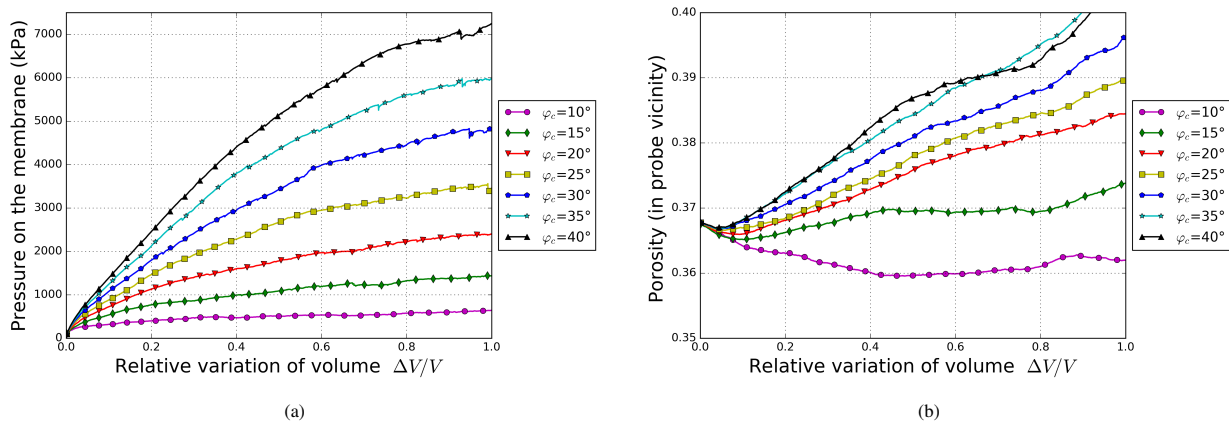


Figure 10: Pressure-volume curves (a) and porosity variations nearby the probe (b) simulated for different values of the contact friction angle (Case 1 in Table 1).

226 stabilizes for values of η_r above 0.3 to 0.7. In addition, a wider range of volumetric responses (dilatancy) is reachable
 227 by changing φ_c rather than η_r .

228 4.2. Initial porosity

229 The constitutive behaviour of a "real" soil is strongly dependent on its initial density, affecting directly the volu-
 230 metric deformation and the maximum shear strength. Thus, the initial density of the granular assembly is an important
 231 parameter to investigate and probe inflation has been simulated with the parameters of the "Case 3" in Table 1, consid-
 232 ering a range of initial porosities, n_0 , from 0.329 to 0.401. Figure 12 shows a significant effect of the initial porosity on
 233 the limit pressure and the volumetric deformation. As the initial porosity decreases, the granular assembly is denser
 234 and present a higher resistance to the probe inflation together with a more dilatant response.

235 To resume, the contact friction angle and the initial porosity affect similarly the simulated soil response to the
 236 probe inflation, affecting significantly both the pressure-volume curve (with among others the limit pressure) and the
 237 volumetric deformation. Consequently, there is no independent relationship between the model response to the probe
 238 inflation and n_0 or φ_c . Nevertheless, the volumetric response could be assumed as independent of the rolling friction
 239 coefficient η_r (even it is not totally true), in order to isolate the effect of η_r from n_0 and φ_c . However, there is still an
 240 interplay between η_r , n_0 and φ_c with respect to the limit pressure and the pressure-volume curve.

241 5. Initial state and calibration of the discrete model from the pressuremeter test

242 The parametric study showed that the simulated response to the probe inflation depends mainly on the plastic
 243 contact parameters and the initial porosity of the granular assembly. The contact elastic parameters are considered as
 244 secondary parameters, and their effect can be significantly reduced as long as the chosen values are close to the rigid

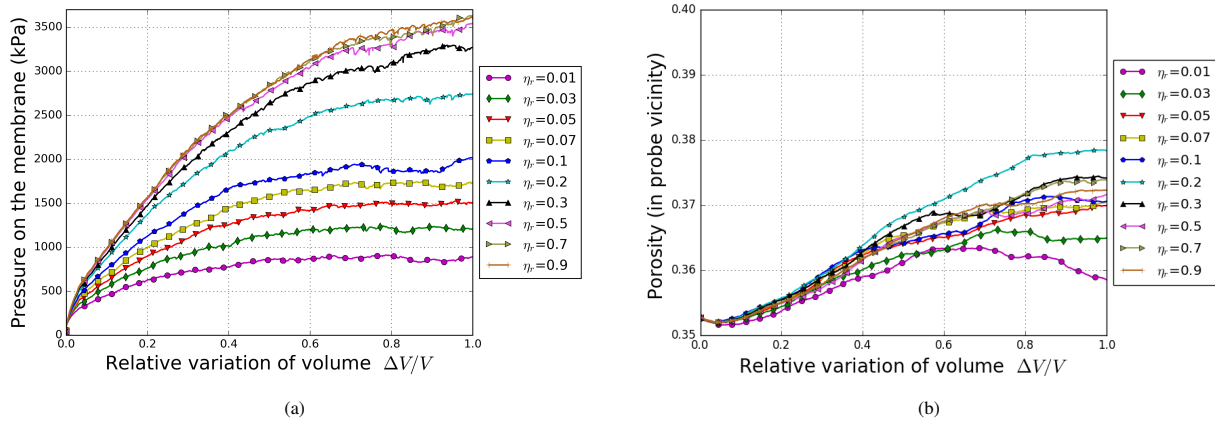


Figure 11: Pressure-volume curves (a) and porosity variations nearby the probe (b) simulated for different values of the rolling friction coefficient (Case 2 in Table 1).

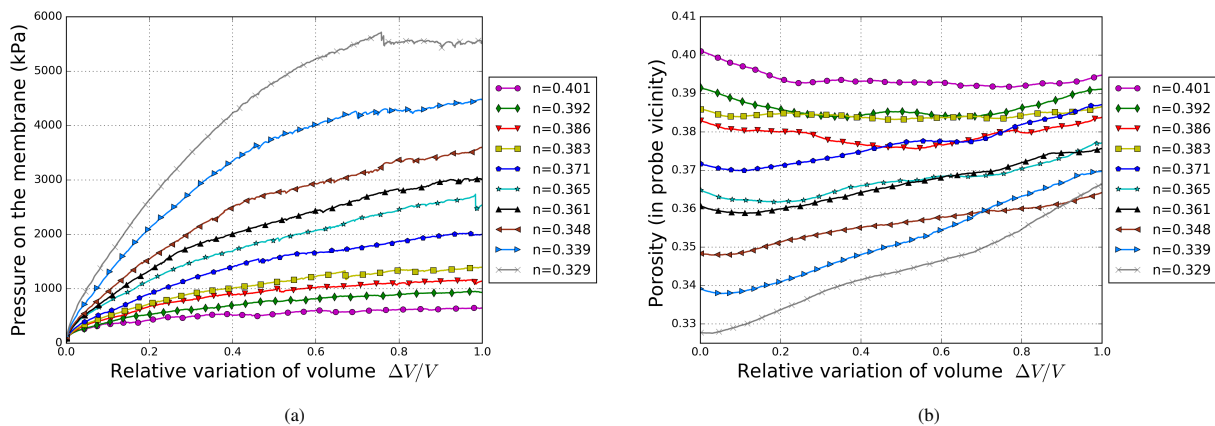


Figure 12: Pressure-volume curves (a) and porosity variations nearby the probe (b) simulated for different initial porosities.

245 particle limit. Therefore, the calibration process can be limited to plastic parameters: contact friction angle, φ_c , and
246 rolling friction coefficient, η_r , once the soil's initial porosity n_0 is fixed.

247 However, as there is no uncoupled relation between the characteristics of the pressure-volume curve and each of
248 these three parameters, there is no possibility to calibrate both φ_c and η_r from a pressuremeter test only.

249 5.1. Choice of the initial porosity

250 The initial porosity of the model could be fixed independently of the pressure-volume curve by following one of
251 the following three approaches (Sibille et al., 2019):

- 252 i The first approach is to assign the porosity of the real soil to the numerical granular packing. This solution is
253 quite straightforward to implement but it assumes that for a given initial porosity the real soil and the numerical
254 packing should undergo similar volumetric deformations, which may not holds if the grain shape is different;
- 255 ii The second approach is to assign the relative density of the real soil (here $D_r = 83\%$) to the numerical packing.
256 This option can be implement by considering the maximum and minimum void ratios achieved in the numerical
257 assembly. Again, this approach assumes that the numerical assemblies at minimum and maximum relative
258 densities simulate the same mechanical responses as the real soil at the same minimum and maximum relative
259 densities.
- 260 iii The third approach consists of adjusting the initial porosity of the numerical sample in a way to reproduce
261 the same volumetric deformation of the real soil. This actually depends on the difference between the initial
262 porosity and the one at the critical state for the numerical model and the real soil respectively

263 Regarding the calibration of the discrete model from the pressuremeter test, the third option cannot be considered as
264 the test does not provide any information about the soil's volumetric response. Consequently, the first and the second
265 options only will be considered as two different scenarios. The determination of the relative density for the numerical
266 model requires to define minimum and maximum void ratios, e_{max} and e_{min} (or porosities). e_{min} is obtained after the
267 compaction of the granular assembly with $\varphi_c = 0$, whereas e_{max} is reached by prohibiting the tangential sliding at
268 contacts.

269 5.2. Rolling friction coefficient, η_r , calibration

270 Aboul Hosn et al. (2017) has shown that the shear strength at critical state depends only on the rolling friction
271 coefficient (if $\varphi_c \geq 15^\circ$). Consequently, η_r can be calibrated independently of the other parameters if the information
272 about the critical state is available. Such information cannot be deduced from the pressuremeter test and we made the
273 choice to take advantage of the knowledge of the shear strength at large deformation for the Hostun sand (deduced
274 from triaxial compressions) and the corresponding value of the rolling friction coefficient. Hence from previous
275 studies, a first value of $\eta_r = 0.55$ has been determined from triaxial compressions performed on a dense Hostun sand
276 (Sibille et al., 2019). Then, thanks to a refine assessment of the shear strength at large deformation based on triaxial

Table 2: Parameters of the discrete model deduced from the calibration from the experimental pressuremeter test.

Parameter set	Approach	n	η_r	φ_c (deg)	E_c (MPa)	α_t	α_r
1	Same absolute density	0.405	0.55	23.5	500	0.3	5
2	Same relative density	0.359	0.55	12.0	500	0.3	5
3	Same absolute density	0.405	0.25	26.5	500	0.3	5
4	Same relative density	0.353	0.25	10.5	500	0.3	5
5	Same absolute density	0.405	0.10	43.0	500	0.3	5
6	Same relative density	0.348	0.10	10.5	500	0.3	5

277 compressions on both dense and loose Hostun sand, the rolling friction coefficient has been reevaluated at $\eta_r = 0.25$
 278 (Sibille et al., 2021). Both values of η_r are intended to be representative of Hostun sand regardless its initial density.
 279 Nevertheless, the second value can be considered as a more representative calibration.

280 These two values of η_r are considered in this study in addition to another one, $\eta_r = 0.1$, chosen arbitrarily to widen
 281 the range of investigation. For every value of η_r the two different approaches of identifying the initial porosity will be
 282 used. Therefore, six different parameter sets have been identified, as detailed in Table 2. For the parameter sets with
 283 an initial numerical relative density identical to the real one, the minimum and maximum void ratios of the numerical
 284 assemblies have to be calculated for each value of η_r as they may depends on the latter, in particular the looser state
 285 (e_{max}) which is the less stable one. Table 3 shows the different maximum and minimum void ratios (or porosities) for
 286 the different values of η_r and the respective void ratios (or porosities) corresponding to a relative density of 83%.

287 5.3. Contact friction angle, φ_c , calibration

288 After defining the rolling friction coefficient, the only parameter to be calibrated is the contact friction angle φ_c .
 289 For each parameter set, different values of contact friction angle, φ_c , are tested using the pressuremeter model. The
 290 pressure-volume curves simulated with these different values of φ_c are compared visually with the experimental curve
 291 to identify which value can best fit the experimental response. This parameter has a direct influence on the plastic part
 292 of the response and much less on the pseudo-elastic phase ($\Delta V/V < 0.2$) which is mainly controlled by the elastic
 293 contact parameters and the initial state of the granular assembly. Therefore, the choice of φ_c is mainly based on the
 294 comparison of the plastic part of the pressure-volume curves (at large change of the probe volume). Table 2 shows the
 295 values of the contact friction angle obtained for each parameter set.

297 5.4. Results and discussion

298 The resulting calibrations are presented in Figure 13 according to each parameter set and compared with the
 299 experimental pressure-volume curve. The corresponding parameters of the models deduced from these calibrations
 300 are given in Table 2.

Table 3: Maximum and minimum void ratios (porosities) obtained for different values of η_r and the respective void ratios (porosities) corresponding to a relative density of 83%, as for the sand used for the pressuremeter test.

η_r	n_{max}	n_{min}	e_{max}	e_{min}	e	n
0.55	0.475	0.329	0.904	0.490	0.560	0.359
0.25	0.456	0.329	0.838	0.490	0.549	0.354
0.1	0.437	0.329	0.776	0.490	0.538	0.350

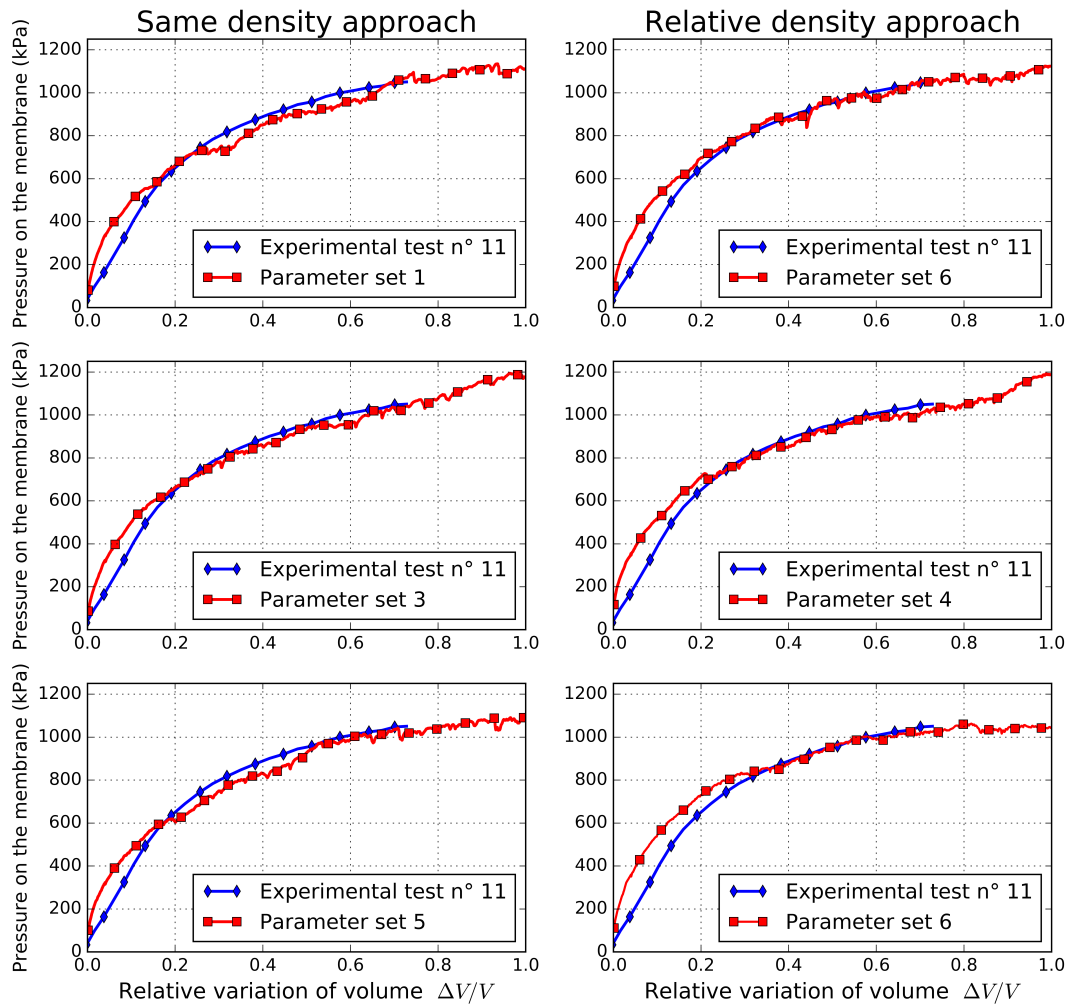


Figure 13: Simulations of the pressuremeter test according to the six different parameter sets of calibration compared to the experimental test results.

301 For all the parameter sets the limit pressure is correctly reproduced by the model. Nevertheless, in all the cases
302 the model slightly overestimates the stiffness (i.e. the slope of the pressure-volume curve) in the pseudo-elastic phase
303 for $\Delta V/V < 0.2$. This could be fitted, at least partially, by tuning the contact stiffness E_c . However, the experimental
304 curve presents itself in this pseudo-elastic phase a shape not strictly convex (the slope of the pressure-volume curve is
305 not strictly decreasing with the inflation of the probe) which may be attributed to a slight experimental drawback. The
306 presence of probe during the sand pluviation may affect the density of the sand in the direct vicinity of the probe and
307 impact by this way the pressure-volume curve at the beginning of the probe inflation (Mokrani, 1991). Consequently,
308 there was no clear reason to attempt to fit more closely the experimental curve in this phase.
309 In all cases, these results show clearly that a unique set of parameters of the discrete model cannot be identified from
310 data of a pressuremeter test. Fahey and Carter (1993) and Biarez et al. (1998) reached the same conclusion concern-
311 ing the calibration of elasto-plastic constitutive relations since the latter were able to describe a given experimental
312 pressure-volume curve with different mechanical input parameters. Nevertheless, the parameter sets 2, 4 and 6 leads
313 to the identification of very low contact friction angles, lower than 15° . Such weak values may not be representative
314 of the contact of silica particles (Kawamoto et al., 2018) and may constitute a drawback for the discrete model itself
315 (Aboul Hosn et al., 2017). Therefore, parameter sets 1, 3 and 5 should a priori be preferred to parameter sets 2, 4 and 6.

316
317 Furthermore, the coefficient of earth pressure at rest, K_0 , was deduced experimentally from measurements of the
318 vertical, σ_v , and horizontal, σ_h , stresses with “Glötzl” stress gauges. K_0 was equal to 0.30 once the soil sample is
319 built and subjected to the vertical stress (i.e. before the inflation of the probe). In the simulations, this coefficient is
320 determined with σ_h calculated as the average stress on the vertical walls (Walls X, Y and E) and σ_v as the average
321 stress on the bottom and top wall (walls B and T). After the application of the vertical stress, $K_0 = 0.39, 0.43,$ and
322 0.51 for the parameter sets 1, 3, and 5 respectively and $K_0 = 0.52, 0.53,$ and 0.55 for parameter sets 2, 4 and 6. These
323 results show that parameter sets 1, 3 and 5 lead also to earth pressure coefficients at rest closer to the value obtained
324 experimentally (the closest being the parameter set 1).

325 **6. Validation from laboratory tests**

326 Validation tests, consisting of different kinds of loadings than the one induced by the pressuremeter test, are
327 considered to assess the relevance of the sets of parameters identified previously and to possibly discriminate them.
328 For that purpose, three different loading paths performed with a true triaxial apparatus on cubical samples of Hostun
329 sand are considered. The side length of the sand samples is initially 10 cm and they are compacted in a dense state
330 ($D_r = 100\%$). Compaction and loading are all performed in dry condition. The first loading path is a monotonous
331 drained axisymmetric triaxial compression under a confining pressure of 200 kPa. The second kind of path is a cyclic
332 stress proportional loading with a really 3D stress state due to the proportionality fixed between the three principal
333 stresses. Finally, the third loading is a circular stress path in the deviatoric stress plane. This loading path was used as

334 a reference path to benchmark the prediction ability of phenomenological constitutive relations during the Cleveland
335 Workshop in 1987 (Saada and Bianchini, 1988). For this loading, the mean pressure and deviatoric stress are kept
336 constant while the Lode angle is changed continuously. First and third tests were performed by Lanier and Zitouni
337 (1988), and second test by Zitouni (1988).

338 These loading cases constitute homogeneous problems, contrary to the model of the pressuremeter test which is
339 a boundary value problem with heterogeneous fields of strains and stresses. Consequently, these 3 triaxial problems
340 are modeled with a parallelepipedic periodic cell including 10 000 particles. The particle size distribution is identical
341 to the one used for the pressuremeter model (Figure 1). Besides, the initial relative density of the sand samples used
342 for these triaxial tests is higher than the relative density of the sand used for the pressuremeter test ($D_r = 83\%$).
343 Consequently, for the simulations of the validation tests, the mechanical parameters of the contact law are of course
344 the one identified in Table 2, but the initial porosity of the numerical samples is chosen in order to be consistent with
345 the approach applied for the identification of the parameters. Hence, it is either the same porosity as the one of the
346 sand sample in the true triaxial apparatus ($n = 0.385$), or the same relative density $D_r = 100\%$. Therefore, the absolute
347 value of the initial numerical porosity is different from the one indicated in Table 2 relative to the pressuremeter tests.

348 6.1. Axisymmetric triaxial compression

349 Simulated responses to the triaxial compression are compared with the experimental results plotted with a thick
350 magenta line in Figure 14. The experimental test includes one cycle of unloading-reloading which is not simulated
351 (prediction ability of the model with respect to cyclic loadings is investigated with the next validation case). Each
352 simulation is performed with a set of mechanical parameters calibrated according to one of the six parameter sets
353 respectively, as displayed in Table 2. Parameter sets defined with the same approach for the determination of the
354 initial sample porosity (either absolute density, or relative density identical to the physical sand samples) are grouped
355 together in Figure 14.

356 Concerning the "absolute density approach" (odd parameter sets), the shear strength at large deformation decreases
357 from parameter set 1 to parameter set 5. This is in agreement with the fact that the critical shear strength depends only
358 on the rolling friction (if the contact friction angle is high enough) (Aboul Hosn et al., 2017) since the rolling friction,
359 η_r , decreases from parameter set 1 to 5. Conversely, the dilatant behaviour, strongly dependent on the contact friction
360 angle, is more pronounced for parameter set 5 where the contact friction angle is the highest, than for parameter set
361 1 with the lowest contact friction angle (among parameter sets 1, 3 and 5).

362 Concerning the "relative density approach" (even parameter sets), the shear strength at large deformation is only
363 weakly depending on the rolling friction. Moreover it is apparently not in agreement with the shear strength at large
364 deformation simulated with the absolute density approach. For instance, the rolling friction for parameter sets 1 and
365 2 ($\eta_r = 0.55$) is the same, thus the critical shear strength should be the same for these two parameter sets.

366 However, the values of the contact friction angles calibrated with the relative density approach are all very low (φ_c
367 ranges from 10.5 to 12 deg). Such values may be not representative of the friction properties between two silica sand

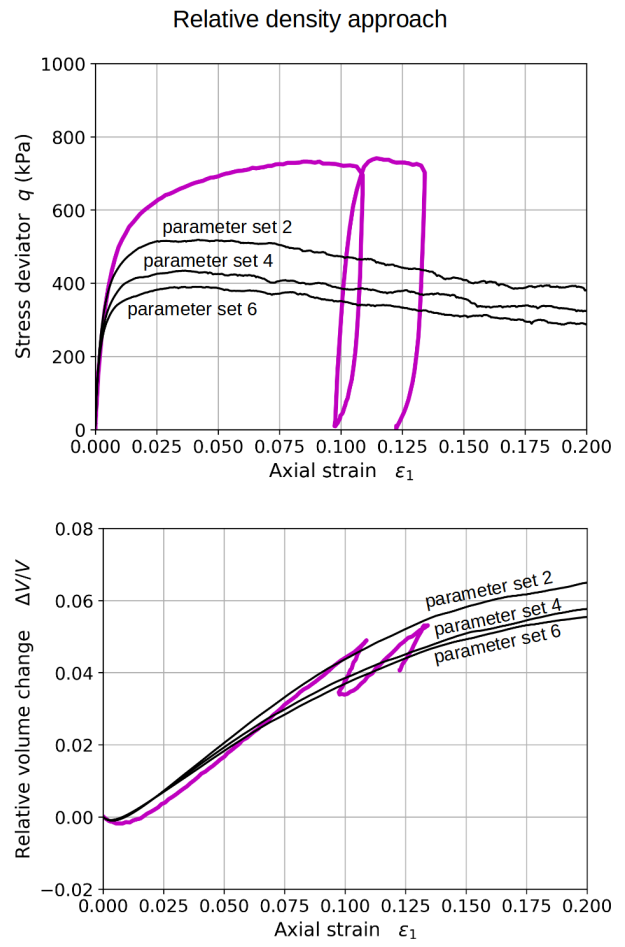
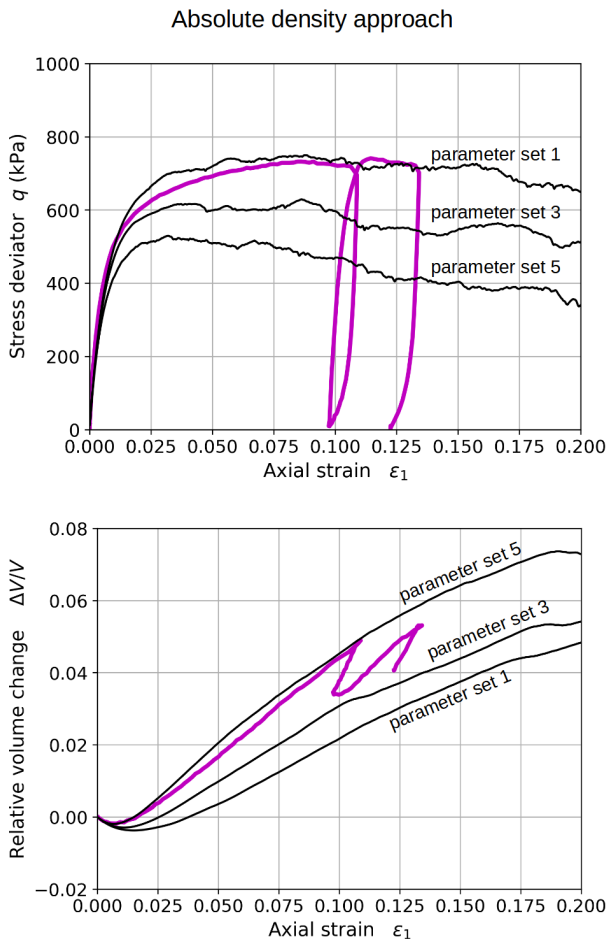


Figure 14: Axisymmetric triaxial compression on dense Hostun sand: comparison of the experimental results with thick magenta (or grey in black and white printings) line and simulated predictions with thin black lines.

Table 4: Final set of parameters for the pressuremeter model

Approach	n	η_r	φ_c (deg)	E_c (MPa)	α_t	α_r
Same absolute density	0.405	0.55	23.5	500	0.3	5

368 grains (Kawamoto et al., 2018) and below the threshold of 15 deg identified by Aboul Hosn et al. (2017) to avoid an
 369 interplay between the rolling friction and the contract friction angle on the critical shear strength. Consequently, the
 370 parameter sets identified with the relative density approach could be discarded for this reason. The dilatant behaviour
 371 is almost the same for parameter sets 2, 4 and 6 since the contact friction angles and the initial porosities are very
 372 close for these sets of parameters.

373
 374 Finally, all the simulated responses to the drained compression are consistent with the values of the respective sets
 375 of parameters used. However, only the parameter set 1 gives a satisfying prediction of the constitutive response to this
 376 loading path, even if the dilatancy is a little underestimated. Consequently, parameter sets 1 only is considered for the
 377 next validation cases.

378 To sum up, the final mechanical parameters of the model and its initial density state (parameter set 1) have been
 379 defined as follows:

- 380 • The initial porosity was assigned equal to the porosity of the real soil;
- 381 • The three elastic parameters E_c , α_t and α_r were presumed with sufficiently high values to avoid the interdepen-
 382 dence between local elastic stiffnesses and macroscopic plastic properties;
- 383 • The rolling friction coefficient, η_r , was calibrated from a rough estimation of the shear strength of Hostun sand
 384 at the critical state based on a triaxial drained compression (Sibille et al., 2019);
- 385 • The contact friction angle, φ_c , was calibrated by tuning this value in the pressuremeter model until the simulated
 386 pressure-volume curve best fits the experimental one, in particular for a large inflation of the probe ($\Delta V/V >$
 387 0.2).

388 Table 4 summarizes the validated set of parameters of the pressuremeter model.

389 6.2. Cyclic stress proportional loading paths

390 For stress proportional loading paths the intermediate principal stress σ_2 is fixed with respect to the major and
 391 minor principal stresses, σ_1 and σ_3 respectively, thanks to the parameter b kept constant for a given loading:

$$b = \frac{\sigma_2 - \sigma_3}{\sigma_1 - \sigma_3} = cst. \quad (5)$$

392 For the test considered here $b = 2/3$ and the mean pressure is kept constant and equal to 500 kPa. In addition,
 393 loading/unloading cycles are performed by imposing the amplitude of the strain in direction 1 such that $\Delta\varepsilon_1 = 4.9\%$,

394 except for the first cycle with a larger amplitude. Experimental results are compared with the response simulated
 395 according to the parameter set 1 in Figure 15. The stress-strain response is rather well predicted by the model, but
 396 the volumetric deformations are slightly overestimated. In particular, for the initial compression, strain localization
 397 occurred experimentally quite early (around $\varepsilon_1 = 2.5$ to 3 %) (Zitouni, 1988) deviating the measured soil response
 398 from what should be the constitutive response. Consequently, experimental response cannot be compared rigorously
 399 with the predicted one for the initial compression, beyond this state. In all cases, it can be considered as a good per-
 400 formance for a model calibration performed on a single monotonous tests (conventional constitutive models dedicated
 401 to cyclic soil responses are usually calibrated from cyclic tests).

402 6.3. Circular stress path

403 The circular stress path is performed by keeping constant the mean pressure p , equal to 500 kPa, and the second
 404 stress invariant $I_{2\sigma}$, equal to 420 kPa (representing the intensity of the deviatoric stress), while the Lode angle φ_σ is
 405 progressively increased from -120 deg to +600 deg ($\varphi_\sigma = 0$ correspond to the direction of the axis s_1 which is the
 406 projection of the σ_1 axis in the deviatoric stress plane). Consequently, the mechanical state of the sand sample is
 407 controlled by imposing the stress state such that:

$$\begin{aligned}
 \sigma_1 &= p + \sqrt{2/3} I_{2\sigma} \cos(\varphi_\sigma) \\
 \sigma_2 &= p + \sqrt{2/3} I_{2\sigma} \cos(\varphi_\sigma - 120^\circ) \\
 \sigma_3 &= p + \sqrt{2/3} I_{2\sigma} \cos(\varphi_\sigma + 120^\circ)
 \end{aligned} \tag{6}$$

408 The initial state of the circular stress path is reached by realizing first an isotropic compression up to 500 kPa, followed
 409 by a compression at constant mean pressure in direction '3' (i.e. such that $\varphi_\sigma = -120$ deg) until reaching $I_{2\sigma} =$
 410 420 kPa.

411 The response predicted by the model (parameter set 1) is compared with the experimental one in Figure 16. The
 412 strain response path is displayed as its projection in the deviatoric strain plane from one hand, and as the relative
 413 volume change (representing the component of the strain path out of the deviatoric strain path) on the other hand.
 414 Concerning the initial compression at constant mean pressure along direction '3', the model describes a purely de-
 415 viatoric deformation (visible in the deviatoric strain plane) whereas experimentally both deviatoric and volumetric
 416 deformations have been observed. Then, regarding the circular stress path itself, the model globally underestimates
 417 the amplitude of the deformations both deviatoric and volumetric, but gives an acceptable and realistic prediction of
 418 the sand response. It is worth noting that the shift in the volumetric strain at the end of the circular path (i.e. for
 419 $\varphi_\sigma = 600$ deg) results only partially from the circular loading, the other part coming from the initial compression.
 420 Consequently, the relative shift in the volumetric strain between the experimental and the simulated results due to the
 421 circular loading is 38%.

422 Furthermore, it is important to keep in mind that conventional elasto-plastic constitutive relations are not able to
 423 describe a realistic soil response (even qualitatively) for this kind of loading path (because the plastic mechanisms in

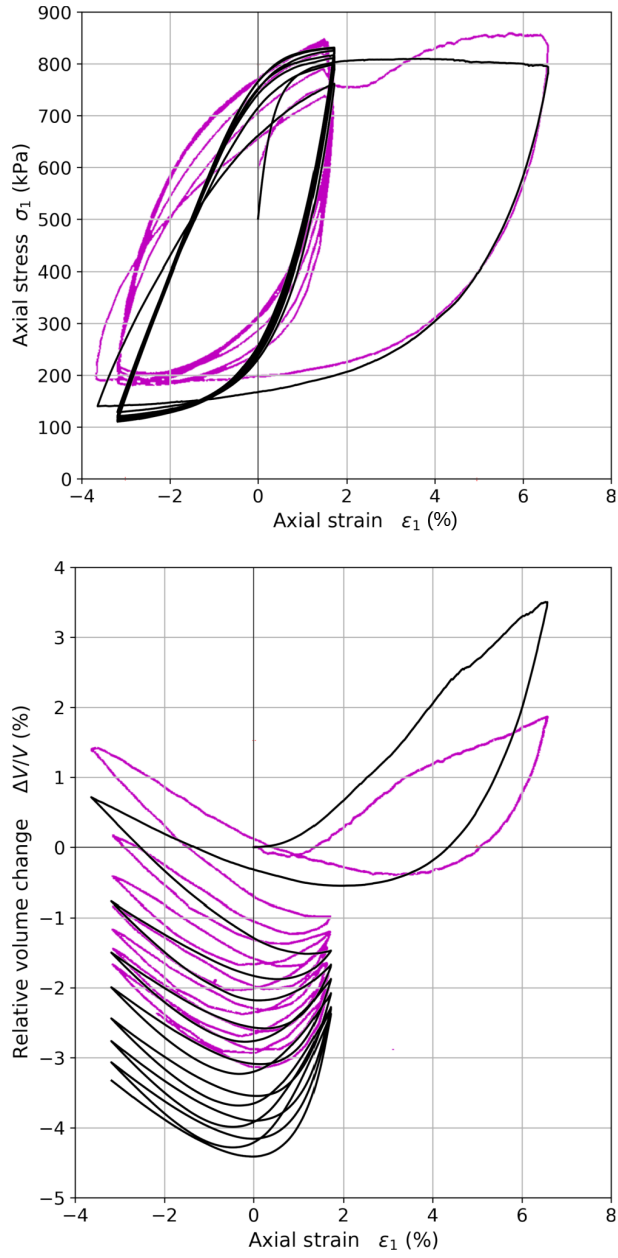


Figure 15: Cyclic proportional stress loading path at constant mean pressure: comparison of the experimental results with magenta (or grey in black and white printings) line and simulated prediction with black line.

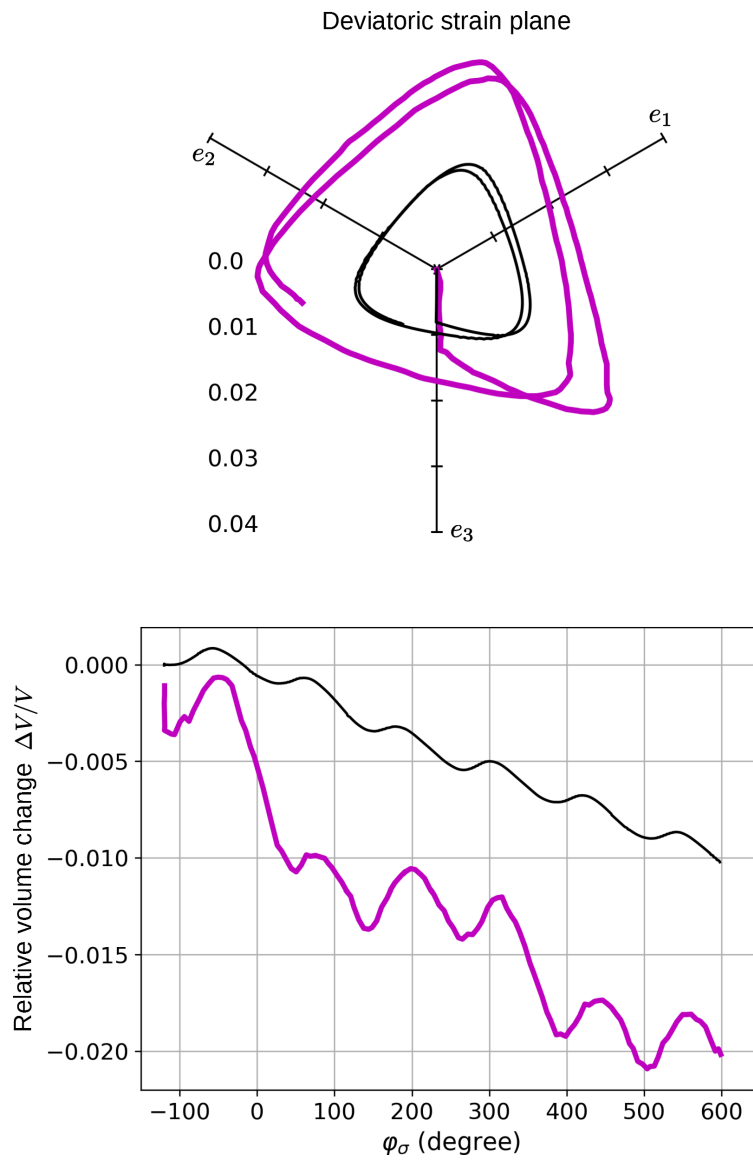


Figure 16: Strain response to a circular stress loading path in the deviatoric stress plane: comparison of the experimental results with thick magenta (or grey in black and white printings) line and simulated predictions with thin black line.

424 such relations are usually related to changes in the mean pressure, or of the deviatoric stress which do not occur along
425 the circular stress path) (Saada and Bianchini, 1988). Only advanced incrementally non-linear constitutive relations
426 have shown their ability to produce a good prediction of the soil response in these conditions (Darve and Dendani,
427 1988). However these models involve an important number of parameters requiring several axisymmetric triaxial tests
428 in compression and extension to be calibrated. Therefore, the prediction of the discrete model is here relatively good
429 compared to the amount of experimental data necessary to calibrate it (a single pressuremeter test and the knowledge
430 of the mobilized internal friction angle at the critical state).

431 **7. Conclusion**

432 A discrete numerical model of pressuremeter test has been developed. To limit the computational cost the model
433 takes advantage of the rotational symmetry of the problem by representing numerically only one quarter of the whole
434 domain. Moreover, the discretization of the soil into discrete particles is made dependent on the distance to the
435 probe of the pressuremeter. In the vicinity of the probe, where important soil deformations, or even the soil fail-
436 ure, are expected a refine discretization is performed by representing the soil with relatively small (and numerous)
437 discrete particles. Further from the probe, the relative mean diameters of the particles is gradually increased since
438 particle rearrangement is less important. Hence, relatively coarse (and few) particles are enough to describe reversible
439 deformations. By this way, the number of particles involved in the model is reduced.

440 The simulation of the pressuremeter test shows that the volumetric response of the soil close to the probe is rather
441 independent (but not totally) of the rolling friction between particles. Besides, there is a cross dependency of the
442 soil response, and in particular the limit pressure, on the contact friction angle, the rolling friction and the initial soil
443 porosity. In other words, essentially because the dilatancy of the soil is not a measurable data from a pressuremeter
444 test, the number of typical soil properties provided by this field test is lower than the number of model parameters.
445 Consequently, the mechanical parameters of the discrete model and the initial density of the granular assembly cannot
446 be identified separately from a pressure-volume curve coming from a pressuremeter test only. This conclusion is in
447 agreement with previous studies about the calibration of elasto-plastic constitutive relations from the pressuremeter
448 tests.

449 The suggested calibration methodology requires an a priori assessment of the critical shear strength of the con-
450 sidered soil. In addition, different parameter sets have been investigated by assigning either the initial porosity of the
451 model, or its initial relative density, equal respectively to the initial soil porosity, or to the initial relative density of the
452 soil. Following these different strategies, the discrete model has been calibrated from a pressuremeter test performed
453 on Hostun sand. Then, a validation step was performed by simulating different triaxial loading paths (asymmetric and
454 truly triaxial paths, monotonous and cyclic paths) for comparison with the experimental responses of the Hostun sand
455 measured from triaxial loading devices. The model calibrated by assuming the initial numerical porosity identical to
456 the initial soil porosity leads to a rather good prediction of the Hostun sand responses on the validation triaxial path.

457 These predictions are quite similar to what can be expected with a model calibrated from an axisymmetric triaxial
458 compression. However the calibration relies only partially on the pressuremeter test, the assessment of the critical
459 shear strength may either be empirical or require a specific test in itself.

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