

A microstructure-based elastoplastic model to describe the behaviour of a compacted clayey silt in isotropic and triaxial compression

Original

A microstructure-based elastoplastic model to describe the behaviour of a compacted clayey silt in isotropic and triaxial compression / Musso, Guido; Azizi, Arash; Jommi, Cristina. - In: CANADIAN GEOTECHNICAL JOURNAL. - ISSN 0008-3674. - STAMPA. - 57:7(2020), pp. 1025-1043. [10.1139/cgj-2019-0176]

Availability:

This version is available at: 11583/2759730 since: 2021-01-09T19:33:34Z

Publisher:

nrcresearchpress

Published

DOI:10.1139/cgj-2019-0176

Terms of use:

This article is made available under terms and conditions as specified in the corresponding bibliographic description in the repository

Publisher copyright

GENERICO -- per es. Nature : semplice rinvio dal preprint/submitted, o postprint/AAM [ex default]

The original publication is available at <https://www.nrcresearchpress.com/doi/pdf/10.1139/cgj-2019-0176> / <http://dx.doi.org/10.1139/cgj-2019-0176>.

(Article begins on next page)

1
2
3
4
5
6
7
8
9
10
11
12
13
14
15
16
17
18
19
20
21
22
23
24
25
26
27
28
29
30
31
32
33

**A microstructure-based elastoplastic model to describe the behaviour
of a compacted clayey silt in isotropic and triaxial compression**

Guido Musso¹, Arash Azizi^{2*} and Cristina Jommi^{3,4}

1. Associate Professor
Department of Structural, Geotechnical and Building Engineering
Politecnico di Torino
Corso Duca degli Abruzzi, 24
10129 Torino, Italy
guido.musso@polito.it

2. Post-doc Researcher
Department of Structural, Geotechnical and Building Engineering
Politecnico di Torino
Corso Duca degli Abruzzi, 24
10129 Torino, Italy
Currently:
Research Fellow
Department of Engineering
Durham University
Lower Mountjoy, South Rd.
DH1 3LE Durham, UK
arash.azizi@durham.ac.uk

34 3. Professor
35 Department of Civil and Environmental Engineering,
36 Politecnico di Milano
37 Piazza Leonardo da Vinci, 32
38 20133 Milano, Italy
39 cristina.jommi@polimi.it

40 4. Department of Geosciences and Engineering
41 Delft University of Technology
42 Stevin weg 1
43 2628CN Delft, The Netherlands
44 c.jommi@tudelft.nl

45
46 *Corresponding author
47

48 **A microstructure-based elastoplastic model to describe the behaviour**
49 **of a compacted clayey silt in isotropic and triaxial compression**

50
51

52 **Abstract**

53

54 The paper focuses on the hydro-mechanical behaviour of an unsaturated compacted clayey silt, accounting for fabric
55 changes induced by drying-wetting cycles occurring at low-stress levels. The response along isotropic compression and
56 triaxial compression (shear) at constant water content was investigated by laboratory tests on both as compacted and
57 dried-wetted samples. Compaction induces a micro-structural porosity pertinent to clay peds and a macro-structural
58 porosity external to the peds. Drying-wetting cycles decrease the micro-porosity and increase the macro-porosity, which
59 reduces the water retention capacity, increases the compressibility and promotes higher peak strengths with more brittle
60 behaviour during triaxial compression. A coupled double porosity elastic-plastic model was formulated to simulate the
61 experimental results. A non-associated flow rule was defined for the macrostructure, modifying a stress-dilatancy
62 relationship for saturated granular soils to account for the increase in dilatancy with suction observed in the
63 experiments. The average skeleton stress and suction were adopted as stress variables. Consistently with model
64 predictions, the shear strength at critical state is not significantly influenced by the degree of saturation or by the
65 hydraulic history. On the other contrary, the higher peak strength, brittleness and dilatancy of the dried wetted samples
66 are mostly explained by their reduced water retention capacity.

67

68

69 **Key words: Compacted silt; drying-wetting cycles; hydro-mechanical behaviour; double-porosity formulation;**
70 **stress-dilatancy relationship**

71

72

73

74

75

76

Number of words: 7300

77

Number of figures: 15

78

Number of tables: 4

79

81 1. Introduction

82

83 According to the standard practice, the engineering requirements for earth constructions are guaranteed by compaction
84 at optimum density and water content, and the design of the earth construction is based on the properties of the soil
85 determined immediately after compaction. To reach the desired density, higher stresses than those imposed by service
86 loads are used, resulting in a material which is highly overconsolidated just after compaction. However, experimental
87 (Take and Bolton 2011) and numerical studies (Kovacevic et al. 2001) suggest the soil response changes over time
88 because of seasonal variations in water pressure causing progressive accumulation of volume strains and leading to a
89 dwindle of dilatancy. Therefore, the maximum shear strength that can be mobilised may reduce from peak values to
90 critical state values, with the consequence that design approaches based on soil parameters detected after compaction
91 might not be on the safe side for the long-term serviceability of the geo-structure.

92

93 More reliable design approaches should account for the fact that hydraulic conditions at the boundary of the earthwork
94 evolve continuously after the end of the construction, due to oscillations of both the water level and the relative
95 humidity of the air in contact with the embankment. These both cause oscillations of pore water pressure and suction,
96 which imply periodic drying-wetting cycles for large volumes of the unsaturated portion of the embankment. Daily and
97 seasonal cycles of relative humidity, associated to different sequences of rainy and dry days, can be relevant in
98 continental climates (e.g. Calabresi et al. 2013) and, because of global warming, such cycles are expected to become
99 more severe in the future (Rouainia et al. 2009). Increasing severity of suction oscillations over time, and the previous
100 experimental evidence on softening in the material response, justify why the effects of drying-wetting cycles on the
101 hydro-mechanical behaviour of compacted soils for earthworks deserve careful attention. Besides affecting the
102 maximum strength, drying-wetting cycles are known to increase the permeability and to reduce the water retention
103 capacity of compacted soils (Benson et al. 2007). These effects are unfavourable for the stability of embankments, since
104 higher permeability allows for the propagation of pore pressure changes from the surface to deeper layers, increasing
105 the chances of strain softening and failure (Nyambayo et al. 2004).

106

107 Changes in the hydraulic behaviour of clayey silts have been related to fabric changes, even occurring at constant
108 volume (Cuisinier and Laloui 2004). The clay fraction of these soils is organized into peds ('aggregates' in Ng et al.
109 2017) which plastically shrink over suction increase, while the total soil volume remains constant because of the
110 shielding effect provided by the 'skeleton' of silt particles (Romero et al. 2014). A larger macro-porosity emerges,
111 justifying the increase in permeability and the decrease in water storage capacity. This evolution of the soil fabric and of

112 the hydraulic behaviour during drying-wetting cycles can be reproduced adequately with double porosity hydro-
113 mechanical volumetric models such as the one proposed by Azizi et al. (2019). However, to the authors' knowledge,
114 limited attention has been paid to the influence of cyclic wetting and drying on the triaxial compression behaviour of
115 unsaturated compacted soils, although it seems reasonable to infer that the fabric changes responsible for the changes in
116 the hydraulic response will impact also on the mechanical behaviour. Experimental studies by Kemal et al. (2005) (on
117 sand samples), by Rojas et al. (2010) (on clayey silt samples cored from a river embankment a few years after
118 construction), and by Zhang et al. (2016) (on a slightly expansive silty soil) remark that drying-wetting cycles tend to
119 increase the peak soil strength and the post-peak soil brittleness.

120

121 In this work, we aim at broadening the understanding of the effects of previous drying-wetting cycles on the coupled
122 hydro-mechanical response of compacted clayey silts, including the pre-failure behaviour. The evolution of the
123 microstructure of a soil used in the construction of river embankments is analysed experimentally over different hydro-
124 mechanical paths, both immediately after compaction and after exposure to drying-wetting cycles. Stemming from the
125 premises on the water retention behaviour in Azizi et al. (2019), the comparison between experimental data on as-
126 compacted and dried-wetted samples is exploited to describe the mechanical response of the soil, within a double-
127 porosity elastic-plastic framework. The proposed model is used to simulate the results of drying, isotropic compression
128 and triaxial compression (shear) at constant water content tests on both as-compacted and dried-wetted samples, with
129 the purpose of showing and start quantifying the influence of natural drying-wetting cycles on the lifetime response of
130 embankments made of similar compacted clayey silts.

131

132

133 2. Soil characterization and sample preparation

134

135 The soil investigated is a clayey silt from Viadana, used in the construction of a full-scale model embankment built for
136 research purposes nearly twenty years ago (see Calabresi et al. 2013). The construction of the model embankment
137 promoted a few studies on this type of material, which is typically used in the reinforcement and construction of flood
138 defences along the Po river (e.g. Nocilla et al. 2006; Vassallo et al. 2007). The clay fraction (particle diameter $d < 2$
139 μm) is 20.40 % and the silt fraction ($2 \mu\text{m} \leq d < 60 \mu\text{m}$) is 79.60 %. The specific gravity is $G_s = 2.735$, the liquid limit
140 WL is 32.6 %, with a plasticity index PI equal to 8.3 %. According to ASTM D2487 Viadana clayey silt is a low
141 plasticity silt (ML), with an activity index $A = 0.4$.

142

143 The effects of drying-wetting cycles on both the fabric and the hydro-mechanical behaviour were investigated on
 144 samples that were statically compacted at a dry density $\rho_d = 1650 \text{ kg/m}^3$ and a water content $w = 20 \%$. This state
 145 replicates the construction specification for the model embankment (see e.g. Rojas et al. 2010). The fabric resulting
 146 from compaction (Original Fabric, OF) was investigated with Scanning Electron Microscopy (SEM) observations,
 147 which showed different aggregations of silt particles and peds of clay particles (Fig. 1). In the macrostructure, between
 148 silt particles and between silt particles and peds, the radius of the smallest pores appears to be around $1 \mu\text{m}$ or greater.
 149 Within the peds, the pore radii are clearly smaller than $1 \mu\text{m}$. Eight samples were prepared to investigate how the pore
 150 network changes as a result of different hydraulic and mechanical histories, using Mercury Intrusion Porosimetry
 151 (MIP). Four of them were analysed, respectively, at the compaction state (OF), after first drying (dry-OF) and after
 152 loading in oedometer at two different axial net stresses ($\sigma_{ax}^{net} = 98 \text{ kPa}$, LL-OF, and $\sigma_{ax}^{net} = 1.6 \text{ MPa}$, HL-OF). The
 153 other four samples were preliminarily exposed to 6 drying-wetting cycles (6Cyc samples). One of them referred to the
 154 dry state (dry-6Cyc), another to the wet one (6Cyc), and other two to loading at $\sigma_{ax}^{net} = 98 \text{ kPa}$ (LL-6Cyc) and
 155 $\sigma_{ax}^{net} = 1.6 \text{ MPa}$ (HL-6Cyc). The lower stress level, 98 kPa , was chosen to represent typical working stresses in the
 156 field, while the higher stress, 1.6 MPa , was chosen to investigate whether mechanical loads could (partially) erase the
 157 effects of previous drying-wetting cycles.

158
 159 Full details on sample preparation and on the cyclic hydraulic history simulating drying-wetting in the field are given in
 160 Azizi et al. (2019). In summary, drying was imposed by exposing the samples to the laboratory environment having a
 161 controlled temperature of 21°C and relative humidity of 38.5% , corresponding to a suction $s = 128.8 \text{ MPa}$, which
 162 brought the water content to a minimum of $w \approx 0.4\%$. After each drying stage, the samples were re-wetted by placing
 163 them in the compaction mould and injecting the volume of water needed to bring the water content back to its initial
 164 value ($w \cong 20\%$). After each drying-wetting cycle, the specimens were wrapped up in plastic bags and kept hanging
 165 over distilled water in a closed container for at least 5 days to ensure water content homogenisation. The state of the
 166 samples prepared for MIP analyses is reported in Table 1. Their hydraulic and mechanical histories are sketched in Fig.
 167 2.

168
 169 The volume of the samples decreased along drying and increased along wetting. By convention, volume decrease is
 170 associated to increasing volume strains and vice versa. Volume strains increased during the first three drying-wetting
 171 cycles (Fig. 3) and were about reversible for a larger number of cycles. Consistently, both the hydraulic behaviour

172 (water retention and hydraulic conductivity) and the fabric evolved during the first three cycles while remaining stable
173 afterwards (see Azizi et al. 2019 for a detailed discussion).

174

175 The hydro-mechanical behaviour of the OF and 6Cyc samples was studied along drying, isotropic and triaxial
176 compression at constant water content in a triaxial cell allowing for suction control or measurement. Another eight
177 samples were prepared to this scope, following the same procedure detailed above. The same stress paths were imposed
178 to the OF and the 6Cyc samples (Table 3) to allow addressing the effects of hydraulic cycles on the following hydro-
179 mechanical behaviour. According to Blight (1964), the pore pressure distribution within a sample sheared under
180 unsaturated conditions is uniform when the time to failure t_f is equal or greater than the time for consolidation t_{100} . On
181 the basis of the measured hydraulic conductivity (Azizi et al. 2019) and sample compressibility, t_{100} is expected to be of
182 the order of a few hours or less. Assuming that failure occurs when $\varepsilon_a = 20\%$, the axial strain rate imposed during
183 triaxial compression was $\dot{\varepsilon}_a = 0.25\% / \text{hour}$, which implies $t_f = 80$ hours.

184 The values of suction and isotropic net stress imposed during drying and isotropic compression are provided in Table 2.
185 The main details of the experimental procedures concerning both the microstructural characterization and the triaxial
186 tests are provided in Appendix 1, while the experimental results, which justify adopting a double porosity model, are
187 presented in section 4 (microstructural investigation) and 5 (triaxial tests).

188

189 **3. Double porosity formulation**

190

191 Given the observed fabric of the soil, a double porosity framework was chosen to simulate the results of the hydro-
192 mechanical tests. Double porosity formulations have been extensively adopted to reproduce the hydraulic (e.g.
193 Barenblatt et al. 1960; Gerke and van Genuchten 1993), hydro-mechanical (e.g. Alonso et al. 1999; Choo et al. 2016)
194 and chemo-hydro-mechanical behaviour (Musso et al. 2013) of geologic materials possessing two dominant families of
195 voids, such as fissures and matrix in reservoir rocks (Warren and Root 1963), or inter-aggregate and intra-aggregate
196 pores in compacted soils (e.g. Della Vecchia et al. 2013). Extended reviews of double porosity models are available in
197 Musso et al. (2013), Mašin (2013) and in Choo et al. (2016).

198

199 The basic concept underlying these models is that the porous medium can be modelled as two overlapping continua, or
200 structural levels, commonly named “microstructure” and “macrostructure”. The microstructure is identified with the
201 deformable solid aggregates, containing their “micropores”. The macrostructure is defined by the pore network made of
202 the voids between the aggregates and it is characterised by the spatial distribution of the aggregates. The two structural

203 levels deform according to independent constitutive laws, and they may exchange fluid masses if the fluid pressures in
 204 the two domains are different. However, in the following, the assumption is made that sufficiently slow hydro-
 205 mechanical processes occur. This assumption implies that the air and the water pressure are the same in the two
 206 structural levels, hence no explicit internal mass transfer conditions are needed for a complete description of the
 207 response.

208

209 3.1 Volumetric variables

210 In the definition of the two overlapping continua, porosity and water content – or void ratio and degree of saturation –
 211 of the soil are split between the two structural levels. The microstructure is made of the solid particles, having volume
 212 V_s , and of the voids within the clay peds, V_{vm} . Therefore, the microstructural void ratio e_m is defined as:

$$213 \quad e_m = \frac{V_{vm}}{V_s} \quad (1)$$

214 Since the peds are deformable, e_m will evolve with stress or suction (see section 3.3). The macro-structural void ratio e_M
 215 is the ratio of the volume of voids between peds (inter-peds, or macro-structural volume of voids V_{vM}) over the total
 216 volume of the peds, hence including the micropores:

$$217 \quad e_M = \frac{V_{vM}}{V_s(1 + e_m)} \quad (2)$$

218 The latter definition implies a reference “solid volume” for macroporosity which is not constant over time. Following
 219 the derivation given and discussed by Mašin (2013), the total void ratio, e , must be consistently written as:

$$220 \quad e = e_m + e_M + e_m e_M \quad (3)$$

221 where the third term accounts for the change of the volume of the reference solids considered in the definition of the
 222 macroscopic void ratio.

223 The microstructural degree of saturation S_{rm} is the ratio between the volume of water within the micro-pores V_{wm} and
 224 the volume of the micro-pores:

$$225 \quad S_{rm} = \frac{V_{wm}}{V_{vm}} \quad (4)$$

226 The macrostructural degree of saturation S_{rM} is:

$$227 \quad S_{rM} = \frac{V_{wM}}{V_{vM}} \quad (5)$$

228 where V_{wM} is the volume of water held outside the peds. The total degree of saturation S_r follows:

$$229 \quad S_r = S_{rM} + \frac{e_m}{e}(S_{rm} - S_{rM}) \quad (6)$$

230 The water ratio e_w expresses the ratio of the volume of water to the volume of solids. The microstructural water ratio
 231 e_{wm} is:

232
$$e_{wm} = \frac{V_{wm}}{V_s} = S_{rm}e_m \quad (7)$$

233 the macro-structural water ratio e_{wM} is:

234
$$e_{wM} = \frac{V_{wM}}{V_s(1 + e_m)} = S_{rM}e_M \quad (8)$$

235 and the relationship between the total water ratio and the water ratios of the two domains is:

236
$$e_w = e_{wm} + e_{wM}(1 + e_m) = S_{rm}e_m + S_{rM}e_M(1 + e_m) \quad (9)$$

237

238 **3.2 Water retention**

239 The total water ratio (eq. (9)) is a function of both the micro-and macro-degree of saturation and void ratio. Adopting
 240 two van Genuchten (1980) expressions for the degree of saturation over the main branches of the water retention
 241 functions of the micro and the macro porosities (e.g. Durner 1994; Casini et al. 2012; Della Vecchia et al. 2015), the
 242 water ratio can be written explicitly as a function of suction in the form:

243
$$e_w(s) = \left[\frac{1}{1 + (\alpha_m s)^{n_m}} \right]^{m_m} e_m + \left[\frac{1}{1 + (\alpha_M s)^{n_M}} \right]^{m_M} e_M(1 + e_m) \quad (10)$$

244 where n_m , m_m , α_m and n_M , m_M , α_M are model parameters describing the micro-structure and the macro-structure
 245 response, respectively. Infinitesimal variations of the total water ratio are given by:

246
$$de_w = de_{wm} + de_{wM} = [S_{rm}de_m + e_m dS_{rm}] + [S_{rM}(1 + e_m)de_M + e_M(1 + e_m)dS_{rM} + S_{rM}e_M de_m] \quad (11)$$

247 which measures the changes in water content as a function of changes in the void ratios and degree of saturations of the
 248 two domains. Also, the previous relationship shows that changes in the degree of saturation of the different fabric levels
 249 may occur even at constant water ratio, together with changes in the micro and macro void ratios. The transition
 250 between the wetting and the drying branches (and vice versa) is postulated as a linear law between the increment of
 251 degree of saturation and the increment of suction, independently for each structural level:

252
$$dS_r = -k_{sc}ds \quad (12)$$

253 where k_{sc} is a model parameter, describing the hydraulic stiffness of the soil over reversible drying-wetting cycles,
 254 bounded by the main wetting and drying branches.

255

256 Changes in the micro or macro void ratio impact mostly on the air entry value of the corresponding porous network. To
 257 account for this evidence, a dependency of the air entry value $1/\alpha$ on the void ratio is introduced. Simple relationships
 258 were chosen relating $1/\alpha_m$ and $1/\alpha_M$ to the micro-structural void ratio e_m and to the macro-structural void ratio e_M :

259
$$1/\alpha_m = (e_m/e_{m0})^{c_m}/\alpha_{m0} \text{ and } 1/\alpha_M = (e_{M0}/e_M)^{c_M}/\alpha_{M0} \quad (13)$$

260 where c_m and c_M are model parameters, e_{m0} and e_{M0} are the values of e_m and e_M at as-compacted conditions, and $1/\alpha_{m0}$
 261 and $1/\alpha_{M0}$ are the initial air-entry values. The empirical laws described by eq. (13) are assumed to hold for both the
 262 main drying and the main wetting curves.

263

264 3.3 Stress variables

265 Two stress variables are employed. The first one is the average skeleton stress, which depends on net stress (σ^{net}),
 266 effective degree of saturation (S_e) and matric suction (s):

$$\sigma' = \sigma^{net} + S_e s \mathbf{I} \quad (14)$$

267 Equation (14) defines the average skeleton stress for each structural level (macrostructural skeleton stress σ'_M or
 268 microstructural skeleton stress σ'_m), by using the corresponding effective degree of saturation. The second stress
 269 variable is matric suction. Similar stress variables have been adopted by different authors (e.g. Jommi 2000; Tamagnini
 270 2004; Romero and Jommi 2008; Zhang and Ikariya 2011; Zhou et al. 2012; Della Vecchia et al. 2013).

271 Both thermodynamic and experimental observations (Alonso et al. 2010) consistently show that the effective degree of
 272 saturation of interest for the macro-structure is given by the free water filling the macro-voids, and thus for the
 273 macrostructure $S_{eM} = S_{rM}$. For the microstructure, the relationship $S_{em} = S_{rm}$ is assumed to hold.

274

275 3.4 Mechanical model for the microstructure

276 The microstructure is assumed to behave isotropically. Many double porosity models formulated for unsaturated soils
 277 rely on the hypothesis of elastic microstructure (e.g. Gens and Alonso 1992; Alonso et al. 1999; Mašin 2013). However,
 278 the experimental data in Azizi et al. (2019) suggest that irrecoverable plastic strains of the peds take place during
 279 drying-wetting cycles, triggering changes in the water retention and permeability. Therefore, the elastoplastic model of
 280 Azizi et al. (2019) is adopted here. Two yielding mechanisms can be activated, one related to mechanical straining of
 281 the peds and the other related to irreversible changes of the microstructural water ratio. The former occurs when the
 282 stress path reaches the loading collapse (LC) curve, whereas the latter is triggered if the stress path reaches the suction
 283 increase (SI) curve during drying, or the suction decrease (SD) curve during wetting. These yield curves are expressed
 284 as:

$$\text{LC: } p'_m = p'_m, \text{ SI: } s = s_I, \text{ SD: } s = s_D \quad (15)$$

285

286 Within the elastic domain, the degree of saturation evolves along the scanning curves and the relationship between
 287 microstructural volume strain ε_m and stress increments is:

$$d\varepsilon_m^e = \frac{\kappa_m dp'_m}{(1 + e_m)p'_m} \quad (16)$$

288

289 where κ_m is the elastic compressibility of the microstructure.

290 If yielding occurs directly on the SI, it induces water ratio changes on the main drying WRC accompanied with
 291 hardening of the LC, whereas direct yielding on the SD induces water ratio changes on the main wetting WRC
 292 accompanied with softening of the LC. If yielding occurs on the LC, it produces plastic volumetric strains with a
 293 coupled outward movement of the SI and inward movement of the SD. The SI and the SD evolve together:

$$\frac{ds_I}{s_I} = \frac{ds_D}{s_D} \quad (17)$$

294 When yielding occurs due to SI or SD, the hardening law is:

$$dp'^*_m = h_{lc} p'^*_m \frac{ds_I}{s_I} \quad (18)$$

295 h_{lc} controls the coupled movement of LC due to SI or SD yielding and p'^*_m is the microstructural mean stress at yield.

296 The plastic volumetric strains ($d\varepsilon_m^p$) due to yielding of the LC curve are

$$d\varepsilon_m^p = \frac{(\lambda_m - \kappa_m) dp'^*_m}{(1 + e_m) p'^*_m} \quad (19)$$

297 where λ_m and κ_m are model parameters. In this case, the hardening law is given by

$$ds_I = h_s s_I \frac{dp'^*_m}{p'^*_m} \quad (20)$$

298 where h_s controls the coupled movement of SI and SD.

299 The general expression for plastic strain increment can be derived through eqs. (17) and (18):

$$d\varepsilon_m^p = \frac{(\lambda_m - \kappa_m)}{(1 + e_m)(1 - h_s h_{lc})} \left(\frac{dp'^*_m}{p'^*_m} - h_{lc} \frac{ds_I}{s_I} \right) \quad (21)$$

300

301 The flow rule for yielding on the SI and on the SD is:

$$\frac{d\varepsilon_m^p}{de_{wm}} = 0 \quad (22)$$

302 While the flow rule for yielding on the LC curve is:

$$\frac{de_{wm}}{d\varepsilon_m^p} = 0 \quad (23)$$

303

304 Changes of the microstructural void ratio are then written in the form:

$$de_m = -d\varepsilon_m(1 + e_m) = -(d\varepsilon_m^e + d\varepsilon_m^p)(1 + e_m) \quad (24)$$

305

306 When the suction or the effective stress change, the micro-structural void ratio changes and the WRC of the peds
 307 evolves accordingly with eq. (13). Further details on the model and the implications on the evolution of the WRC can
 308 be found in Azizi et al. (2019).

309

310 **3.5 Mechanical model for the macrostructure**

311 Silty soils show some recurrent specific behavioural trends, which make them different from ideal coarse or fine-
 312 grained soils (Cui and Delage 1996; Ma et al. 2016; Kim et al. 2016; Ng et al. 2017). While they have a mostly
 313 volumetric hardening similar to clays, they typically show non associative elasto-plastic response upon triaxial
 314 compression, which is more similar to those of coarser soils. When the over consolidation ratio, defined as the ratio
 315 between the maximum and the current net stress, is low (typically smaller than 2.5), shearing is ductile and the soil
 316 contracts. Shearing is brittle and accompanied by dilation for greater over-consolidation ratios. Peak strength typically
 317 occurs together with maximum dilatancy, and both the peak strength and dilatancy at failure have been found to
 318 increase when the degree of saturation decreases (e.g. Cui and Delage 1996; Cattoni et al. 2005), similarly to most types
 319 of soils (see, e.g., Zhan and Ng 2006; Yao et al. 2014; Zhou and Sheng 2015 on clays; Fern et al. 2016 on sands;
 320 Alonso et al. 2016 on rockfill). The finding is consistent with the original conclusions by Kohgo et al. (1993), who
 321 observed that one of the effects of suction is to inhibit sliding between particles.

322 In general, elasto-plastic models formulated for unsaturated silts adopt non-associative flow rules, but hydro-mechanical
 323 coupling has been either not introduced (Cui and Delage 1996; Chiu and Ng 2003) or introduced neglecting the water
 324 retention and mechanical role of the clay peds (Ma et al. 2016). Instead, the latter seems to be an important feature to
 325 understand and model the behaviour of Viadana silt (Azizi et al. 2019).

326 The yield function and the hardening rule adopted in the present formulation stem from the Modified Cam Clay Model
 327 (Roscoe and Burland 1968) extended to unsaturated states, as described in Jommi (2000), and used among others by
 328 Romero and Jommi (2008) and Della Vecchia et al. (2015). The yield function is:

$$f = q^2 - M^2 p'_M (p'_{0M} - p'_M) \quad (25)$$

329 where p'_{0M} is the pre-consolidation pressure of the macrostructure and M is the slope of the Critical State Line in the
 330 (p'_M, q) plane, which is assumed not to depend on suction. Following Jommi (2000) and Gallipoli et al. (2003), the
 331 preconsolidation pressure in unsaturated states is the sum of the saturated preconsolidation mean stress p'^*_M depending
 332 on the volumetric plastic strains, and a term which introduces the effects of the degree of saturation:

$$p'_{0M} = p'^*_M + (1 + b_1 (\exp(b_2(1 - S_{rM})) - 1)) \quad (26)$$

333 where b_1 and b_2 are model parameters describing the sensitivity of the pre-consolidation pressure to changes in the
 334 degree of saturation of the macrostructure.

335 The volumetric hardening law relates p_M^* to the plastic volume strains ε_{vM}^p :

$$\frac{dp_M^*}{d\varepsilon_{vM}^p} = \frac{(1 + e_M)p_M^*}{\lambda_M - \kappa_M} \quad (27)$$

336 where λ_M and κ_M are the elastic-plastic and the elastic volumetric compressibility of the macrostructure.

337

338 The flow rule is an original proposal of this work, which is formulated to take into account explicitly the evidence of
339 non-associative behaviour of the silt and the constraining effects of suction in the plastic range. The expression stems
340 from the original contribution of Li and Dafalias (2000) for saturated coarse soils:

$$\frac{\partial \varepsilon_v^p}{\partial \varepsilon_q^p} = d = d_0 \left[e^{m\psi} - \frac{\eta}{M} \right] \quad (28)$$

341 where ε_v^p and ε_q^p are the plastic components of the volumetric and deviatoric strains, respectively, d_0 and m are model
342 parameters, $\eta = q/p'$ is the stress ratio and ψ is the state parameter (Been and Jefferies 1985):

$$\psi = e - e_c(p') \quad (29)$$

343 where e is the current void ratio and e_c is the void ratio at critical state for the current mean effective stress.

344

345 As remarked, a few works point out that dilatancy in unsaturated conditions is higher than in saturated ones. However,
346 assuming dependency on suction only would imply very high (theoretically infinite) dilatancy for dry conditions.
347 Therefore, dilatancy is assumed to increase with the inner constraint induced by the hydraulic component of the
348 skeleton stress, through the product of the effective degree of saturation times suction. Li and Dafalias (2000)
349 expression is also modified so to account for pure volumetric plastic strains occurring along isotropic compression
350 paths. The proposed extension of eq. (27) to the unsaturated state for the macrostructure reads then:

$$d = \frac{d_0}{\eta^M} (\exp(\gamma S_{rMs})) \left[e^{m\psi} - \frac{\eta^M}{M} \right] \quad (30)$$

351 where $\eta_M = q/p'_M$ is the stress ratio of the macrostructure and γ is the additional model parameter weighting the
352 relevance of suction and degree of saturation on the deviatoric response.

353

354

355 4. Evolution of the pore size density in light of the double porosity framework

356

357 The description of the microstructural and macrostructural void ratio is based on the MIP measurements taken at the
358 different conditions outlined in Table 1. Bimodal Pore Size Density (PSD) functions were detected in all cases, and
359 their evolution with drying-wetting cycles and loading is presented in Fig. 4. Drying-wetting cycles affect the soil fabric

360 by shifting the size of the pores corresponding to the dominant peak to a larger pore radius ($r = 609$ nm for the OF
 361 sample, while $r = 917$ nm for the 6Cyc sample), by reducing the frequency of the pores of the dominant mode and by
 362 increasing the size and frequency of the pores belonging to the minor mode (Fig. 4a, see also Azizi et al. 2019).
 363 Mechanical loading of both OF and 6Cyc samples (Figs. 4b and 4c) reduces the frequency of the pores having radii
 364 larger than the one of the peak of the dominant mode. Increasing the load also leads to a progressive decrease of the size
 365 of the larger pores belonging to the minor mode. Pores on the left of the dominant peak were not affected by loading.
 366 Upon loading, the radius of the dominant peak of the OF samples remains fixed at $r = 609$ nm (Fig. 5b), while it
 367 progressively decreases from $r = 917$ nm to $r = 609$ nm for the 6Cyc samples (Fig. 4c). Interestingly, under the axial
 368 stress of 1.6 MPa the PSDs of the HL-OF and of the HL-6Cyc samples overlap very well (Fig. 4d), which suggests that
 369 the effects imparted on the fabric by the hydraulic history can be almost erased by high mechanical loads.

370

371 **4.1 Modelling the Pore Size Density data**

372 A criterion discriminating between intra-peds pores and inter-peds pores allows using the PSDs to evaluate the values of
 373 the microstructural void ratio e_m and of the macrostructural void ratio e_M , as observed in various previous work (e.g.
 374 Delage and Lefebvre 1984; Cuisinier and Laloui 2004; Monroy et al. 2010). Here, the threshold between intra-peds and
 375 inter-peds pores was set to correspond to the radius of the peak of the dominant mode, consistently with Azizi et al.
 376 (2019), which allowed reproducing the evolution of the water retention behaviour of compacted Viadana silt.

377 The microstructural void ratio is evaluated as

$$e_m = \int_{3.5r}^{R_t} \frac{PSD(r)}{3.5r \ln(10)} dr + 0.04 \quad (31)$$

378

379 where 3.5 nm is the smallest pore radius intruded by MIP, R_t is the threshold radius separating intra-peds from inter-
 380 peds pores and 0.04 is the void ratio corresponding to the very small non intruded pores, assumed to be equal to the
 381 residual water ratio obtained at very high suctions. The macrostructural void ratio e_M was evaluated applying eq. (3), by
 382 subtraction from the known total void ratio, e . The values of the e_m and e_M for each of the samples investigated are
 383 provided in Table 3.

384

385 The experimental evolution of the total void ratio e , of the micro-structural void ratio e_m and of the macro-structural
 386 void ratio e_M along the drying-wetting cycles and the mechanical loading is provided in Fig. 5. Drying-wetting cycles
 387 reduced the micro-structural void ratio leaving the total void ratio substantially unaffected. As a result, the
 388 macrostructural void ratio increased. Note that the 6Cyc samples were more compressible than the OF samples when

389 loaded to 98 kPa, suggesting that the larger macro-porosity developed during the preliminary drying-wetting cycles was
 390 prone to collapse under small mechanical loads. However, the void ratio of both types of samples under the stress of 1.6
 391 MPa is about the same.

392

393

394 **5. Hydro-mechanical behaviour of Viadana silt in unsaturated conditions and model predictions**

395

396 The hydro-mechanical behaviour of Viadana silt in unsaturated conditions, as detected through drying, isotropic and
 397 triaxial compression tests run in a suction controlled triaxial cell, is presented together with the predictions obtained
 398 with the double-porosity model introduced in Section 3. The calibration of the parameters of the model is described
 399 first.

400

401 **5.1 Calibration of model parameters**

402 The procedure for the calibration of the parameters of the double porosity water retention model and of the mechanical
 403 model for the micro-structure is explained in detail in Azizi et al. (2019). Water retention parameters for both the
 404 microstructure and the macrostructure were determined from independent sets of measurements and from back-analysis
 405 of the water retention curves of the OF and 6Cyc samples.

406 The air entry value of the microstructure was determined by introducing the value of the threshold pore radius
 407 separating the microstructure from the macrostructure in the Washburn-Laplace equation, which relates the pore size to
 408 the suction at which their desaturation takes place. Knowing the evolution of the air entry values and of e_m and e_M along
 409 the drying-wetting cycles (see Table 3) allowed calibrating c_m and c_M in eq. (13). For the tested soil, c_m and c_M were
 410 found to be 16.5 and 4, respectively, as provided in Tables 4. The parameters n_M , m_M , n_m , and m_m of the van Genuchten
 411 expression were calibrated upon the data of the first drying. The scanning parameter k_{sc} was evaluated on the basis of
 412 suction cycles run in a suction controlled oedometer (Azizi et al. 2017).

413 The compression behaviour of compacted samples of Viadana silt in saturated conditions presented in Nocilla et al.
 414 (2006) was interpreted to determine λ_M and κ_M , under the assumption that the volume strains of the microstructure can
 415 be neglected if compared to those of the macrostructure. Data in the same work were also interpreted to determine the
 416 slope M of the critical state line in the (p'_M, q) plane, which was found to be $M = 1.29$. The parameters b_1 and b_2 (eq.
 417 (26)), describing hardening due to the macroscopic degree of saturation S_{rM} , and the parameters for the flow rule γ and
 418 d_0 (eq. (30)), were calibrated on the results of the tests carried out on the OF samples, while they were used to predict
 419 the behaviour of the 6Cyc samples.

420 An oedometer test was performed to determine the preconsolidation stress, assuming that this was the same for the
 421 microstructure and the macrostructure. An OF specimen was compacted, saturated under a net axial stress
 422 $\sigma_{ax}^{net} = 10\text{kPa}$ and then loaded. The axial preconsolidation stress was found to range about $\sigma_{ax}^* = 400\text{ kPa}$. The radial
 423 stress at preconsolidation was estimated through Jaky's expression $K_0 = 1 - \sin\varphi'_{cs}$ (with φ'_{cs} as the critical state
 424 friction angle and $\sin\varphi'_{cs} = \frac{3M}{6+M}$), which provided $K_0 = 0.47$, giving $\sigma_r^* = K_0\sigma_{ax}^* \cong 212\text{ kPa}$. Eventually, the value of
 425 the mean preconsolidation stress in saturated conditions resulted in $p'^* = p'_M = p'_m \cong 360\text{ kPa}$.

426

427 **5.2 Drying**

428 Drying took place in the suction - controlled triaxial cell, where target suction values $s_\theta = 50\text{ kPa}$ or $s_\theta = 300\text{ kPa}$ were
 429 imposed while keeping the mean net stress constant at 5 kPa (stress path in Fig. 6a). The water ratio of all samples
 430 decreased as shown in Fig. 6b. When subjected to the same suction, 6Cyc samples expelled more water than OF
 431 samples. Figures 6c and 6d show the experimental results and the model predictions for the OF and 6Cyc samples in
 432 terms of $e-e_w$. A small contraction occurred when imposing $s_\theta = 300\text{ kPa}$, but the effect of the suction increase was
 433 mostly a decrease of the total degree of saturation. At the end of the drying process the water ratio and degree of
 434 saturation of the 6Cyc samples was noticeably smaller than the one of the corresponding OF samples.

435

436 The main drying and wetting curves and the predictions of the evolution of the water ratio with suction are presented in
 437 Fig. 6e. The model allows explaining the different water ratios of the two samples at the end of drying. The hydraulic
 438 states of the OF samples were initially inside the reversible domain, very close to their main drying curve. Upon suction
 439 increase, they approached ($s_\theta = 50\text{ kPa}$) or reached ($s_\theta = 300\text{ kPa}$) the main drying curve. The 6Cyc samples were
 440 initially closer to their main wetting curve than the OF samples, since they had underwent wetting during the last
 441 preparatory stage. However, they also reached their main drying curve when subjected to increasing suction. According
 442 to eqs. (10) and (12), the 6Cyc samples ($e_M = 0.20$) have a lower water retention capacity than OF samples ($e_M = 0.17$)
 443 because of their larger macrostructural void ratio.

444

445 **5.3 Isotropic Compression**

446 OF and 6Cyc samples were isotropically compressed to mean net stress values $p^{net} = 100, 200$ and 400 kPa (samples
 447 with $s_\theta = 50\text{ kPa}$) or to a mean net stress value $p^{net} = 100\text{ kPa}$ (samples with $s_\theta = 300\text{ kPa}$) while keeping suction constant
 448 (Fig. 7a). At $p^{net} = 100$ and 200 kPa , the 6Cyc samples were more compressible than the OF samples. However, the void
 449 ratios of the OF and of the 6Cyc samples were about the same at $p^{net} = 400\text{ kPa}$, which is consistent with the

450 microstructural observations in Section 4. For both types of samples, the compressibility reduced with suction and axial
451 stress.

452

453 The model provides very reasonable predictions of void ratio changes along isotropic compression, considering that the
454 compressibility parameters of the macrostructure, κ_M and λ_M , used in the simulations, were determined on different
455 samples tested in saturated conditions. Also, the model correctly predicts that the void ratios of the 6Cyc samples are
456 smaller than the void ratios of the OF samples at the same mean net stress, although this difference is overestimated at
457 high stresses.

458

459 Figure 8a shows the changes in void and water ratios during isotropic compression. The water ratio decreased slightly in
460 all cases, however, changes in the degree of saturation were negligible since the effects of changes in volume and water
461 content counterbalanced each other. Figures 8b and 8c show the results in terms of e_w-s . Because of the small decrease
462 in the water ratio, the hydraulic states of all samples moved slightly towards the corresponding main wetting curve, still
463 remaining rather close to the main drying curve. According to the model, both the main drying and the main wetting
464 curves evolved due to the decrease of void ratios during isotropic compression, but the changes in the suction range of
465 interest were very small (in the figure only the final position of the WRCs is shown).

466

467 **5.4 Triaxial compression**

468 Triaxial compression (shearing) at constant water content started from the conditions achieved after isotropic
469 compression. The experimental results and model predictions of the triaxial compression phase for the tests performed
470 at a net confining stress of 100 kPa (OF_1 and 6Cyc_1 with initial suction $s_0 = 50$ kPa; OF_4 and 6Cyc_4 with initial
471 suction $s_0 = 300$ kPa) are plotted in Fig. 9. The peak strength and the post-peak softening of both OF and 6Cyc samples
472 were larger at the highest suction (Fig. 9a). However, the 6Cyc samples showed a higher peak strength and a more
473 pronounced softening than the OF samples when sheared at the same initial suction. The model predicts reasonably well
474 the peak strength and the brittleness of these samples. Volumetric strains are plotted against deviatoric strains in Fig. 9b.
475 All samples initially contracted and afterwards dilated. Dilatancy, which was larger in the case of the 6Cyc samples,
476 increased with suction. The slope of volumetric strains changes with deviatoric strains, as predicted by the model in the
477 plastic range (i.e. when the volume starts to increase), is quite similar to the experimental one, especially for the OF_4
478 and 6Cyc_4 samples. Suction decreased during all tests, more markedly the higher its initial value, and the decrease in
479 suction was larger for the OF samples compared to the 6Cyc samples (Fig. 9c). Because of the constant water content

480 condition, the total degree of saturation changed very slightly, increasing when the volume decreased and decreasing
 481 when the volume increased (Fig. 9d).

482

483 Figure 10 shows the results of the triaxial compression phase of the OF and 6Cyc samples confined at 200 kPa and 400
 484 kPa net stress and with an initial suction $s_0 = 50$ kPa (samples OF_2 and OF_3 and samples 6Cyc_2 and 6Cyc_3). At
 485 these higher confinement stresses, the deviatoric stress increased monotonically during triaxial compression, and the
 486 strength of the 6Cyc samples was slightly higher than the one of the OF samples (Fig. 10a). Monotonic compressive
 487 strains occurred in all tests (Fig. 10b) with the exception of 6Cyc_2 (the 6Cyc sample confined at 200 kPa radial net
 488 stress), which showed moderate softening and little dilatancy. The decrease in suction experienced by the OF samples
 489 was larger than the one of the corresponding 6Cyc samples (Fig. 10c). Because of the constant water content constraint,
 490 and of contraction during triaxial compression, the total degree of saturation increased for all samples (Fig. 10d).

491

492 Figure 11 shows the evolution of suction during triaxial compression, together with the evolution predicted for the main
 493 drying and main wetting retention curves. For contracting samples, such as OF2 and 6Cyc_2 in Fig. 11a, the model
 494 predicts an increase of the water retention capacity, with the state of both samples remaining inside the reversible
 495 domain. On the contrary, for dilating samples such as OF_4 and 6Cyc_4 in Fig. 11b, the model predicts a decrease of
 496 the water retention capacity, and the final hydraulic state of these samples lays on the corresponding main drying
 497 curves.

498

499

500 6. Discussion

501

502 The model predictions can be exploited to provide an insight into the hydro-mechanical behaviour of the compacted
 503 Viadana silt.

504 At the end of the triaxial compression stage, the samples reached or approached (samples OF_1, OF_4 and 6Cyc_1)
 505 critical state conditions. Figure 12a compares the experimental points at the end of triaxial compression with the stress
 506 paths predicted by the model, together with results of undrained triaxial compression tests on saturated compacted
 507 samples from Nocilla et al. (2006), which are used to the sake of comparison. In the (p'_M, q) plane, the critical state for
 508 all the compacted samples (both saturated and unsaturated and regardless of the previous hydraulic history) is very well
 509 fitted with a single line having a slope $M = 1.29$. This substantiates the use of the average skeleton stress as stress
 510 variable, which is made possible by the correct identification of the macro-structural degree of saturation. Also, the

511 results in Fig. 12a validate the assumptions made on both the position of the threshold between the micro and the
 512 macro-pores, and the coupled hydro-mechanical model governing the water retention behaviour of the two structural
 513 domains.

514

515 The void ratio at critical state in Fig. 12b appears to be influenced by suction, as previously remarked for instance by
 516 Gallipoli et al. (2003), suggesting that the locus of the critical state conditions should be fitted by a Critical State
 517 Surface in the (p'_M, q, e, s) hyperspace rather than by a line in the traditional (p'_M, q, e) space. However, for given
 518 suction, the OF and 6Cyc samples seem to approach the same line, suggesting that the Critical State Surface does not
 519 depend substantially on the previous hydraulic history.

520

521 Figure 13 introduces the position of the yield curves of the macrostructure at the beginning of triaxial compression for
 522 the highly overconsolidated samples, tested at 100 kPa confining stress (OF_1, OF_4 and 6Cyc_1, 6Cyc_4), together
 523 with the stress paths predicted by the model. All these samples dilated and softened. The preconsolidation pressure p'_{0M} ,
 524 which defines the size of the yield curves (eq. (25)), depends on the macrostructural degree of saturation S_{rM} (eq. (26)):
 525 thus it increases with suction and, for the same suction, it is larger for the 6Cyc samples compared to the OF samples
 526 because of the reduced capacity to retain water of the former. This implies the sequence $p'_{0M}(\text{OF}_1) < p'_{0M}(\text{6Cyc}_1) <$
 527 $p'_{0M}(\text{OF}_4) < p'_{0M}(\text{6Cyc}_4)$. The stress paths intercept the yield surface on the dry side, where $\eta_M/M > 1$, and the
 528 model predicts negative d values (which means volume increase, eq. (30)) for all of them. Since the hardening rule of
 529 the Modified Cam Clay is used, the interception between the stress paths and the yield surface provides the peak
 530 deviatoric stress, q_{peak} . Therefore, the larger the p'_{0M} , the larger q_{peak} , which is consistent with the sequence of peak
 531 strengths detected in the experiments.

532

533 The stress paths of all the samples sheared at 200 kPa and 400 kPa confining stress and suction $s_0 = 50$ kPa intercepted
 534 the yield surface on the wet side ($\eta_M/M < 1$), which implied instead positive d values, and then contraction and
 535 hardening. Figures 14a and 14b show the experimental data and model predictions of the stress-dilatancy relationships
 536 for OF and 6Cyc samples, respectively. The state parameter of the samples sheared at 100 kPa confining stress (OF_1,
 537 OF_4, 6Cyc_1 and 6Cyc_4) is negative, since their void ratios are smaller than the ones at critical state for the same
 538 stress conditions (see Fig. 12). These samples yield when $\eta_M/M > 1$, with a predicted negative value of d (hence,
 539 dilating). The opposite holds for the samples sheared at 200 kPa and 400 kPa of confining stress. Predictions compare
 540 very reasonably with the experimental data although the dilatancy of sample OF_4 appears to be underestimated.

541

542 Relevant model predictions of the evolution of suction during triaxial compression are presented in Fig. 15. An air entry
 543 value of the microstructure higher than 200 kPa (Table 4) ensured that the microstructure remained saturated during all
 544 the tests performed at $s_0 = 50$ kPa. In these cases, the evolution of the WRC is governed by the changes occurring in the
 545 macrostructure. An example is given by the OF_1 sample (Fig. 15a). At the beginning of triaxial compression, the
 546 hydraulic state lays in the reversible domain. Here, according to eq. (11), suction decreases when the macrostructural
 547 degree of saturation S_{rM} increases and suction increases when S_{rM} decreases. Along triaxial compression, an elastic
 548 contraction occurs first, which in light of the constant water content constraint implies an increase in saturation and then
 549 a decrease in suction. Afterwards, for $\varepsilon_q > 0.02$, the sample dilates, thus S_{rM} decreases and suction increases again. Note
 550 that this monotonous relationship between suction and volume changes occurs only because the hydraulic state always
 551 moves within the reversible domain, and not on the main drying curves plotted as grey lines in Fig. 15a. Similarly, the
 552 hydraulic state moved within the reversible domain also in the other samples tested at $s_0 = 50$ kPa, for which suction
 553 changes and volume strains had the opposite sign.

554

555 On the contrary, when $s_0 = 300$ kPa (as for test OF_4 in Fig. 15a), very little water is held by the macrostructure. The
 556 bimodal WRC is then dominated by the microstructure, which reduces its water retention capacity when the
 557 microstructural void ratio decreases, and the value of suction associated to the given water content on the main branches
 558 of the WRC reduces (eqs. (10) and (13)). Note that this occurs also along the whole triaxial compression process,
 559 following eq. (21). As shearing progresses, the main drying curve reaches the hydraulic state of the sample when $\varepsilon_q =$
 560 0.02, which constrains the evolution of suction for the rest of the test to the main drying curve.

561

562

563 7. Conclusions

564

565 The hydro-mechanical behaviour of a compacted clayey silt in unsaturated conditions, at the as-compacted state and
 566 after exposure to drying-wetting cycles, was investigated by means of laboratory tests and then interpreted with a
 567 coupled double-porosity elasto-plastic model. Drying-wetting cycles at low confinement stresses alter the soil fabric
 568 promoting volumetric shrinkage of the microstructure, made of clay peds, and a consequent increase of the macro-
 569 porosity. This porosity exchange significantly reduces the capacity of the soil to retain water for values of suction
 570 smaller than about 400 kPa. Because of its larger macro-porosity, the dried-wetted soil is more compressible than the
 571 as-compacted soil when small loads are applied. The response to triaxial compression at constant water content

572 conditions is also affected. At the same initial values of mean net stress and suction, the dried-wetted soil shows higher
573 shear strength and more brittle behaviour than the 'as-compacted' soil. This is a consequence of different effects of the
574 fabric changes induced by the previous drying-wetting cycles. Firstly, because of the differences in the water retention
575 behaviour, the suction decrease occurring during triaxial compression is more contained in the dried-wetted material
576 compared to the as-compacted one, which keeps the stress path on a drier side. Secondly, higher suction implies higher
577 dilatancy, hence, higher peak strength.

578

579 The proposed elasto-plastic coupled hydro-mechanical double porosity framework allowed reproducing all the relevant
580 aspects observed during the tests. The model, which adopts the average skeleton stress of the macrostructure as a
581 constitutive stress for the macro-porosity domain, permits a natural transition from saturated to unsaturated conditions.
582 The slope of the critical state line in the mean - deviatoric stress plane is clearly the same for the saturated and
583 unsaturated conditions, and for any of the two fabrics of the unsaturated material, although the void ratio at critical state
584 depends on the suction. The volumetric and hydraulic behaviour along isotropic compression in unsaturated conditions
585 is reasonably reproduced using the elastic and elasto-plastic compliance parameters deduced from tests on saturated
586 samples to characterize the behaviour of the macrostructure.

587

588 The mechanical behaviour of the macrostructure is nicely captured with an elasto-plastic model which adopts the yield
589 surface of the Modified Cam Clay, where the preconsolidation pressure increases when the degree of saturation
590 decreases. The different response to triaxial compression is triggered by the different water retention behaviour for the
591 two fabrics. Since preliminary drying-wetting cycles reduce the volume the water retention capacity of the macro-
592 structure, the higher peak strength of the dried-wetted samples can be explained accounting for their smaller macro-
593 structural degree of saturation, which implies a larger preconsolidation pressure. A non-associated flow rule, which
594 takes into account the role of the state parameter, of the degree of saturation of macropores and of suction, further
595 contributes to nicely reproducing the increasing dilatancy and higher peak strength of the dried-wetted samples.

596

597

598 **References**

- 599 Alonso, E.E., Pereira, J.M., Vaunat, J. and Olivella, S. 2010. A microstructurally based effective stress for unsaturated
600 soils. *Géotechnique*, 60(12), 913–925.
- 601 Alonso, E.E., Romero E. and Ortega, E. 2016. Yielding of rockfill in relative humidity-controlled triaxial experiments.
602 *Acta Geotechnica*, 11, 455–477. doi:10.1007/s11440-016-0437-9.
- 603 Alonso, E.E., Vaunat, J. and Gens, A. 1999. Modelling the mechanical behaviour of expansive clays. *Engineering
604 Geology* 54, 173–183.
- 605 Azizi, A., Jommi, C. and Musso, G. 2017. A water retention model accounting for the hysteresis induced by hydraulic
606 and mechanical wetting-drying cycles. *Computers and Geotechnics*, 87, 86–98.
- 607 Azizi, A., Musso, G. and Jommi, C. 2019. Effects of repeated hydraulic loads on microstructure and hydraulic
608 behaviour of a compacted clayey silt. *Canadian Geotechnical Journal*, *Accepted*.
- 609 Barenblatt, G.I., Zheltov, Iu.P. and Kochina, I.N. 1960. Basic concepts in the theory of seepage of homogeneous liquids
610 in fissured rocks. *J. Appl. Math. Mech.*, 24(5), 1286–1303.
- 611 Been, K. and Jefferies, M.G. 1985. A state parameter for sands. *Géotechnique*, 35(2), 99–112.
- 612 Benson, C.H., Sawangsuriya, A., Trzebiatowski, B. and Albright, W.H. 2007. Postconstruction changes in the hydraulic
613 properties of water balance cover soils. *Journal of Geotechnical and Geoenvironmental Engineering*, 133(4), 349–359.
- 614 Blight, G.E. 1964. The Effect of non-uniform pore pressure on laboratory measurements of the shear strength of soils.
615 *ASTM symposium on laboratory shear testing of soils*, STP 361, 173–191.
- 616 Calabresi, G., Colleselli, F., Danese, D., Giani, G. P., Mancuso, C., Montrasio, L., Nocilla, A., Pagano, L., Reali, E. and
617 Sciotti A. 2013. A research study of the hydraulic behaviour of the Po river embankments. *Canadian Geotechnical
618 Journal*, 50, 9, 947–960.
- 619 Casini, F., Vaunat, J., Romero, E. and Desideri, A. 2012. Consequences on water retention properties of double-porosity
620 features in a compacted silt. *Acta Geotechnica*, 7(2), 139–150.
- 621 Cattoni, E., Cecconi, M and Jommi, C. 2005. Soil dilatancy and suction: some remarks on their mutual effect on the on
622 the shear strength of granular soils. In: *Proceedings of the eleventh international conference on computers methods and
623 advances in geomechanics*, Torino, Italy, pp 19–26.
- 624 Chiu, C.F. and Ng, C.W.W. 2003. A state-dependent elasto-plastic model for saturated and unsaturated soils
625 *Géotechnique*, 53, No. 9, 809–829.
- 626 Choo, J., White, A.J. and Borja, R.I. 2016. Hydromechanical Modeling of Unsaturated Flow in Double Porosity Media.
627 *Int. J. Geomech.*, D4016002. doi:10.1061/(ASCE)GM.1943-5622.0000558.
- 628 Cui, Y. and Delage, P. 1996. Yielding and plastic behaviour of an unsaturated compacted silt. *Géotechnique*,
629 46(2):291–311.
- 630 Cuisinier, O. and Laloui, L. 2004. Fabric evolution during hydromechanical loading of a compacted silt. *Int. J. Numer.
631 Anal. Meth. Geomech.*, 28, 483–499, doi:10.1002/nag.348.
- 632 Delage, P. and Lefebvre, G. 1984. Study of the structure of a sensitive Champlain clay and its evolution during
633 consolidation. *Canadian Geotechnical Journal*; 21:21–35.
- 634 Delage, P. and Pellerin, F.M. 1984. Influence de la lyophilisation sur la structure d'une argile sensible du Québec. *Clay
635 Minerals*, 19, 151–160.
- 636 Della Vecchia, G., Dieudonne, A.C., Jommi, C. and Charlier, R. 2015. Accounting for evolving pore size distribution in
637 water retention models for compacted clays. *International Journal for Numerical and Analytical Methods in
638 Geomechanics*, 39 (7), 702–723.
- 639 Della Vecchia, G., Jommi, C. and Romero, E. 2013. A fully coupled elastic–plastic hydromechanical model for
640 compacted soils accounting for clay activity. *Int. J. Numer. Anal. Meth. Geomech.*, 37, 503–535.

- 641 Durner, W. 1994. Hydraulic conductivity estimation for soils with heterogeneous pore structure. *Water Resources*
642 *Research*, 30(2), 211–223.
- 643 Fern, E.J., Robert, D.J., and Soga, K. 2016. Modeling the Stress-Dilatancy Relationship of Unsaturated Silica Sand in
644 Triaxial Compression Tests. *J. Geotech. Geoenviron. Eng.*, 142(11): 04016055.
- 645 Gallipoli, D., Gens, A., Sharma, R., and Vaunat, J. 2003. An elasto-plastic model for unsaturated soil incorporating the
646 effects of suction and degree of saturation on mechanical behaviour. *Géotechnique*, 53(1), 123– 135.
- 647 Gens, A. and Alonso, E.E. 1992. A framework for the behaviour of unsaturated expansive clays. *Canadian*
648 *Geotechnical Journal*, 29:1013-1032, <https://doi.org/10.1139/t92-120>.
- 649 Gerke, H.H. and van Genuchten, M.T. 1993. A dual-porosity model for simulating the preferential movement of water
650 and solutes in structured porous media. *Water Resour. Res.*, 29(2), 305– 319.
- 651 Jommi, C. 2000. Remarks on the constitutive modelling of unsaturated soils, in Proc. Int. Workshop on Unsaturated
652 Soils: Experimental Evidence and Theoretical Approaches in Unsaturated Soils, edited by A. Tarantino and C.
653 Mancuso, pp. 139–153, A.A. Balkema, Rotterdam, Netherlands.
- 654 Kemal, S., Kiyama, S., Aoyama, S. and Kobayashi A. 2005. Laboratory Study on Shear Behavior of Unsaturated
655 Granular Soil During Cyclic Suction Loading. *Transactions of The Japanese Society of Irrigation, Drainage and*
656 *Reclamation Engineering*, 2005, 238, 329-335.
- 657 Kim, B.S., Park S.W., Takeshita Y. and Kato S. 2016. Effect of suction stress on critical state of compacted silty soils
658 under low confining pressure. *Int. J. Of Geomechanics*, 16 (6): D4016010.
- 659 Kohgo, Y., Nakano, M. and Mihazaki, T. 1993. Theoretical aspects of constitutive modelling for unsaturated soils. *Soils*
660 *and Foundations* 33, 4, 49-63. doi.org/10.3208/sandf1972.33.4_49.
- 661 Kovacevic, N., Potts, D.M. and Vaughan, P.R. 2001. Progressive failure in clay embankments due to seasonal climate
662 changes. *Proc. 15th Int. Conf. Soil Mech. Geotech. Engng*, Istanbul, Turkey 3, 2127–2130.
- 663 Li, X.S. and Dafalias, Y.F. 2000. Dilatancy for cohesionless soils. *Géotechnique* 50, 4, 449–460
- 664 Ma, T., Wei, C., Wei, H. and Li, W. 2016. Hydraulic and Mechanical Behavior of Unsaturated Silt: Experimental and
665 Theoretical Characterization. *Int. J. Geomech.*, 16(6): D4015007.
- 666 Mašin, D. 2013. Double structure hydromechanical coupling formalism and a model for unsaturated expansive clays.
667 *Engineering Geology*, 165: 73–88.
- 668 Monroy, R., Zdravkovic, L. and Ridley, A. 2010. Evolution of microstructure in compacted London Clay during
669 wetting and loading, *Géotechnique* 60, 2, 105–119, doi: 10.1680/geot.8.P.125.
- 670 Musso, G., Romero, E. and Della Vecchia, G. 2013. Double-structure effects on the chemo-hydro-mechanical
671 behaviour of a compacted active clay. *Géotechnique*, 63(3), 206–220.
- 672 Ng, C.W.W., Mu, Q.Y. and Zhou, C. 2017. Effects of soil structure on the shear behaviour of an unsaturated loess at
673 different suctions and temperatures. *Can. Geotech. J.* 54: 270–279.
- 674 Nocilla, A., Coop, M.R. and Colleselli, F. 2006. The mechanics of an Italian silt: an example of “transitional”
675 behaviour. *Géotechnique* 56, 4, 261-271.
- 676 Nyambayo, V.P., Potts, D.M. and Addenbrooke, T.I. 2004. The influence of permeability on the stability of
677 embankments experiencing seasonal cyclic pore water pressure changes. *Advances in Geotechnical Engineering:*
678 *Proceedings of the Skempton Conference* (Jardine R. J., Potts D. M. and Higgins K. G. (eds)). Thomas Telford, London,
679 2004, 2, 898–910.
- 680 Rojas, J.C, Mancuso, C. and Danese, D. 2010. Pre and post-construction characterization of an embankment fill
681 material. In “*Unsaturated soils. Experimental studies in unsaturated soils and expansive soils*”. *Proceedings of the 4th*
682 *Asia Pacific conference on unsaturated soils*, Newcastle, Australia, 23-25 November 2009. Buzzi O. Fytus S. and
683 Sheng D. editors, CRC press.
- 684 Romero, E. 1999. Thermo-hydro-mechanical behaviour of unsaturated Boom clay: an experimental study. PhD thesis,
685 Universidad Politècnica de Catalunya, Barcelona, Spain.

- 686 Romero, E. and Jommi, C. 2008. An insight into the role of hydraulic history on the volume changes of anisotropic
687 clayey soils. *Water Resources Research*, 44, W12412, doi:10.1029/2007WR006558.
- 688 Romero, E., Vaunat, J. and Merchan, V. 2014. Suction effects on the residual shear strength of clays. *Journal of Geo-*
689 *Engineering Sciences* 2 (2014) 17–37, doi:10.3233/JGS-141320.
- 690 Roscoe, K.H. and Burland, J.B. 1968. *Engineering Plasticity*, Cambridge University Press, Evanston, IL, p. 535, J.
691 Heyman, F.A. Leckie (Eds.).
- 692 Rouainia, M., Davies, O. O'Brien, T. and Glendinning, S. 2009. Numerical modelling of climate effects on slope
693 stability. *Proceedings of the Institution of Civil Engineers - Engineering Sustainability*, 162, 2, 81-89.
- 694 Take, W.A. and Bolton, M.D. 2011. Seasonal ratcheting and softening in clay slopes, leading to first-time failure.
695 *Géotechnique* 61, 9, 757–769.
- 696 Tamagnini, R. 2004. An extended Cam-clay model for unsaturated soils with hydraulic hysteresis. *Géotechnique* 54, 3,
697 223-28.
- 698 Van Genuchten, M.T. 1980. A closed-form equation for predicting the hydraulic conductivity of unsaturated soils. *Soil*
699 *Science Society of America Journal*, 44, 892–898.
- 700 Vassallo, R., Mancuso, C. and Vinale, F. 2007. Effects of net stress and suction history on the small strain stiffness of a
701 compacted clayey silt. *Canadian Geotechnical Journal*, 44(4), 447-462.
- 702 Warren, J. and Root, P. 1963. The behavior of naturally fractured reservoirs. *SPEJ.*,3(3),245–255.
- 703 Yao, Y.P., Niu, L. and Cui, W.J. 2014. Unified hardening (UH) model for overconsolidated unsaturated soils. *Canadian*
704 *Geotechnical Journal*, 51:810-821 <https://doi.org/10.1139/cgj-2013-0183>.
- 705 Zhan, T.L.T. and Ng, C.W.W. 2006. Shear strength characteristics of an unsaturated expansive clay. *Canadian*
706 *Geotechnical Journal*, 43(7): 751–763. doi:10.1139/t06-036.
- 707 Zhang, F. and Ikariya T. 2011. A new model for unsaturated soil using skeleton stress and degree of saturation as state
708 variables. *Soils and foundations*. 51 (1), 67-81.
- 709 Zhang, J., Sun, D., Zhou, A. and Jiang, T. 2016. Hydromechanical behaviour of expansive soils with different suctions
710 and suction histories. *Canadian Geotechnical Journal*, 53:1-13, <https://doi.org/10.1139/cgj-2014-0366>.
- 711 Zhou, A.N. and Sheng, D. 2015. An advanced hydro-mechanical constitutive model for unsaturated soils with different
712 initial densities. *Computers and Geotechnics*, 63, 46-66, doi.org/10.1016/j.compgeo.2014.07.017.
- 713 Zhou, A.N., Sheng, D., Sloan, S.W. and Gens, A. 2012. Interpretation of unsaturated soil behaviour in the stress–
714 saturation space: II: Constitutive relationships and validations. *Computers and Geotechnics*, 43, 111-123
715 <https://doi.org/10.1016/j.compgeo.2012.02.009>.
- 716
- 717
- 718
- 719
- 720
- 721
- 722
- 723
- 724
- 725
- 726

727 **List of symbols**

- 728 Void ratio: e
- 729 Microscopic void ratio: e_m
- 730 Macroscopic void ratio: e_M
- 731 Degree of saturation: S_r
- 732 Microscopic degree of saturation: S_{rm}
- 733 Macroscopic degree of saturation: S_{rM}
- 734 Water ratio: e_w
- 735 Water ratio of the microstructure: e_{wm}
- 736 Water ratio of the macrostructure: e_{wM}
- 737 Initial value of air entry pressure of the microstructure: $1/\alpha_{m0}$
- 738 Initial value of the air entry pressure of the macrostructure: $1/\alpha_{M0}$
- 739 Parameters of the van Genuchten expression of the microstructure: n_m and m_m
- 740 Parameters of the van Genuchten expression of the macrostructure: n_m and m_m
- 741 Model parameter controlling the dependency of microstructure on air entry value: c_1
- 742 Model parameter controlling the dependency of macrostructure on air entry value: c_2
- 743 Skeleton stress of the microstructure: σ'_m
- 744 Skeleton stress of the macrostructure: σ'_M
- 745 Preconsolidation stress of the microstructure: p^*_{0m}
- 746 Saturated preconsolidation stress of the macrostructure: p^*_{0M}
- 747 Suction increase of the microstructure: s_I
- 748 Suction decrease of the microstructure: s_D
- 749 Slope of the critical state line in the (p'_M, q) plane: M
- 750 Elastic logarithmic compliance of the microstructure: κ_m
- 751 Elastic logarithmic compliance of the macrostructure: κ_M
- 752 Elasto-plastic logarithmic compliance of the macrostructure: λ_m
- 753 Elasto-plastic logarithmic compliance of the macrostructure: λ_M
- 754 Poisson coefficient of the macrostructure: ν
- 755 Parameters of the modified Li and Dafalias flow rule: d_0, γ, m
- 756
- 757
- 758
- 759
- 760
- 761
- 762
- 763

764 List of Tables

765 Table 1. State of the samples for MIP analyses

766 Table 2. Samples for hydro-mechanical characterization with values of suction and isotropic net stress applied during
767 isotropic compression in the triaxial tests

768 Table 3. Macro-structural, micro-structural and total void ratios from interpretation of MIP analyses

769 Table 4. Values of the parameters used in the numerical simulations

770

771

772

773

774

775

776

777

778

779

780

781

782

783

784

785

786

787

788 **List of Figures**

- 789 Fig. 1. SEM images of OF sample: (a) detail with macropores, clay peds and silt particles; (b) detail with micropores
790 within a clay ped.
- 791 Fig. 2. Hydraulic and mechanical histories of samples prepared for MIP analysis.
- 792 Fig. 3. Volumetric strains during the drying-wetting cycles. Positive volume strains indicate volume decrease, negative
793 volume strains indicate volume increase.
- 794 Fig. 4. Evolution of the PSD with drying-wetting cycles and with mechanical loading: (a) evolving the PSD with drying
795 and wetting; (b) evolving the PSD of OF samples with mechanical loading; (c) evolving the PSD of 6Cyc samples with
796 mechanical loading; (b) the PSDs of OF and 6Cyc samples being subjected to 1.6 MPa.
- 797 Fig. 5. Evolution of void ratio e , micro-structural void ratio e_m and macro-structural void ratio e_M : (a) with hydraulic
798 loads; (b) with mechanical loads.
- 799 Fig. 6. Behaviour of samples along drying to 50 and 300 kPa of suction: a) stress and hydraulic paths; b) evolution of
800 water ratio with time; c) void ratio-water ratio OF samples; d) void ratio-water ratio 6Cyc samples; e) suction- water
801 ratio.
- 802 Fig. 7. Isotropic compression at mean net stress $p^{net} = 100, 200$ and 400 kPa; a) stress path; b) void ratio- mean net
803 stress.
- 804 Fig. 8. Behaviour of OF and 6Cyc samples along isotropic compression to mean net stress of $p^{net} = 100, 200$ and 400
805 kPa: a) void ratio-water ratio; b) suction-water ratio.
- 806 Fig. 9. Behaviour of OF and 6Cyc samples ($s_0 = 50$ and 300 kPa, $p^{net} = 100$ kPa) along triaxial compression at constant
807 water content: a) deviatoric stress – deviatoric strains; b) volume strain – deviatoric strain; c) suction – deviatoric strain;
808 d) degree of saturation – deviatoric strain.
- 809 Fig. 10. Behaviour OF and 6Cyc samples ($s_0 = 50$ kPa, $p^{net} = 200$ kPa and 400 kPa) along triaxial compression at
810 constant water content: a) deviatoric stress – deviatoric strains; b) volume strain – deviatoric strain; c) suction –
811 deviatoric strain; d) degree of saturation – deviatoric strain.
- 812 Fig. 11. Behaviour of OF and 6Cyc samples ($s_0 = 50$ kPa and $s_0 = 300$ kPa, $p_{net} = 100, 200$ and 400 kPa) during the
813 triaxial compression phase: (a) void ratio-water ratio for samples with $s_0 = 50$ kPa; b) suction-water ratio for samples
814 with $s_0 = 300$ kPa.
- 815 Fig. 12. Interpretation of final testing conditions and model predictions in terms of macroscopic average skeleton stress
816 of the macrostructure: (a) deviatoric – mean stress plane; (b) compression plane.
- 817 Fig. 13. Stress paths and initial position of the yield curve: (a) OF_1 and 6Cyc_1 ($s_0 = 50$ kPa); (b) samples OF_4 and
818 6Cyc_4 ($s_0 = 300$ kPa).
- 819 Fig. 14. Relationship between stress ratio and dilatancy: (a) OF samples; (b) 6Cyc samples.
- 820 Fig. 15. Changes in suction and evolution of the points lying on the main drying and main wetting curves: (a) test OF_1
821 ($s_0 = 50$ kPa); (b) OF_4 ($s_0 = 300$ kPa).
- 822
- 823
- 824

Table 1. State of the samples for MIP analyses.

Sample	Axial stress σ_{ax} (kPa)	Hydraulic state	Void ratio e (–)	Water content w (%)	Degree of saturation S_r (–)
OF	-	-	0.66	20.0	0.83
dry- OF	-	Dry (first drying)	0.64	1.8	0.08
dry-6Cyc	-	Dry (6th drying)	0.63	0.4	0.07
6Cyc	-	Wet (6 th drying- wetting cycle)	0.65	19.9	0.83
LL-OF	98	-	0.61	18.9	0.85
HL-OF	1600	-	0.51	18.5	0.93
LL-6Cyc	98	Wet 6 th drying- wetting cycles	0.59	18.2	0.86
HL-6Cyc	1600	Wet 6 th drying- wetting cycles	0.51	17.5	0.95

Table 2. Samples for hydro-mechanical characterization with values of suction s and mean net stress p^{net} applied during drying and isotropic compression in the triaxial tests

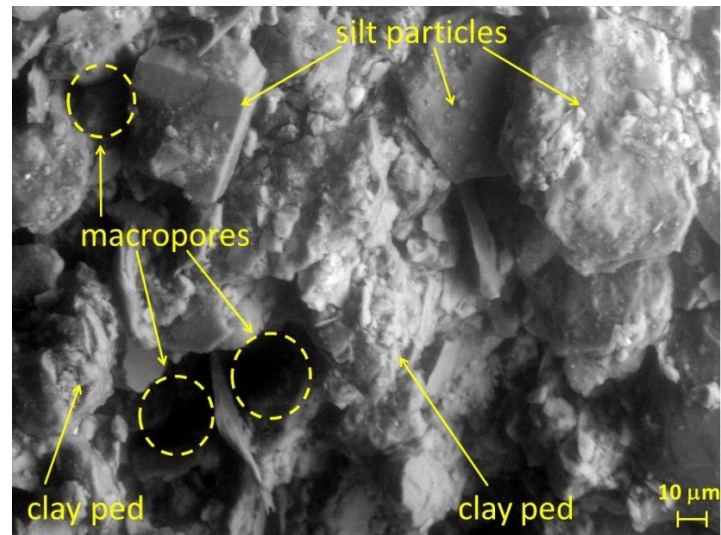
Test	Drying		Isotropic compression	
	Suction, s (kPa)	Mean net stress p^{net} , (kPa)	Suction, s (kPa)	Mean net stress p^{net} , (kPa)
OF_1	50	5-10	50	100
OF_2	50	5-10	50	200
OF_3	50	5-10	50	400
OF_4	300	5-10	300	100
6Cyc_1	50	5-10	50	100
6Cyc_2	50	5-10	50	200
6Cyc_3	50	5-10	50	400
6Cyc_4	300	5-10	300	100

Table 3. Macro-structural, micro-structural and overall void ratios from interpretation of MIP analyses

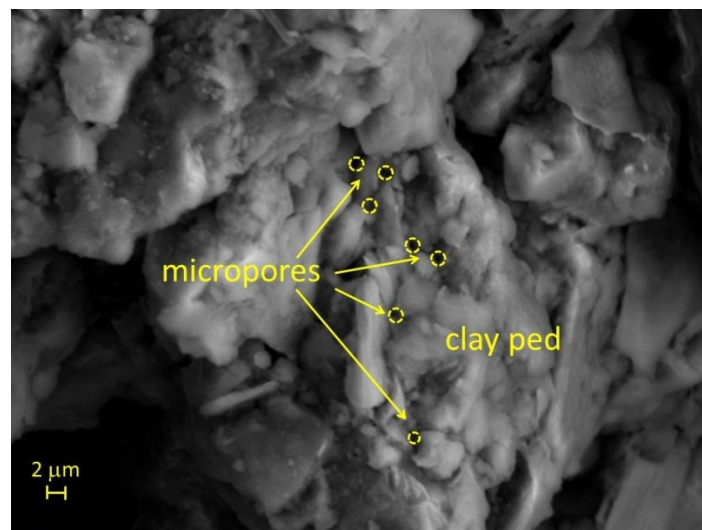
Void ratio	OF	Subjected to hydraulic load			Subjected to mechanical load		Subjected to hydraulic and mechanical load	
		dry-OF	dry-6Cyc	6Cyc	LL-OF	HL-OF	LL-6Cyc	HL-6Cyc
e	0.66	0.63	0.63	0.66	0.61	0.51	0.59	0.51
e_m	0.42	0.41	0.38	0.38	0.44	0.42	0.40	0.41
e_M	0.17	0.16	0.18	0.20	0.12	0.06	0.14	0.07

Table 4. Values of the parameters used in the numerical simulations

Hydraulic Parameters									
Hydraulic path	Micro-structure				Macro-structure				Both domains
	$1/\alpha_{m0}$ (kPa)	n_m (-)	m_m (-)	c_m (-)	$1/\alpha_{M0}$ (kPa)	n_M (-)	m_M (-)	c_M (-)	k_{sc} (kPa ⁻¹)
Drying	236	2.86	0.14	16.5	64	1.75	0.83	4	$3 \cdot 10^{-4}$
Wetting	34	2.61	0.12		8	2.37	0.97		
Mechanical Parameters, Micro-structure									
κ_m (-)	λ_m (-)	h_{IC} (-)	h_s (-)	p'^*_m (kPa)	s_1 (kPa)	s_D (kPa)			
0.009	0.056	4.3	0.25	360	236	33 (OF) 5 (6Cyc)			
Mechanical Parameters, Macro-structure									
κ_M (-)	ν (-)	λ_M (-)	p'^*_M (kPa)	b_1 (-)	b_2 (-)	M (-)	d_0 (-)	γ (kPa ⁻¹)	m (-)
0.025	0.2	0.13	360	0.14	2.2	1.29	1.7	0.03	1.2



(a)



(b)

Fig. 1. SEM images of OF sample: (a) detail with macropores, clay peds and silt particles; (b) detail with micropores within a clay ped.

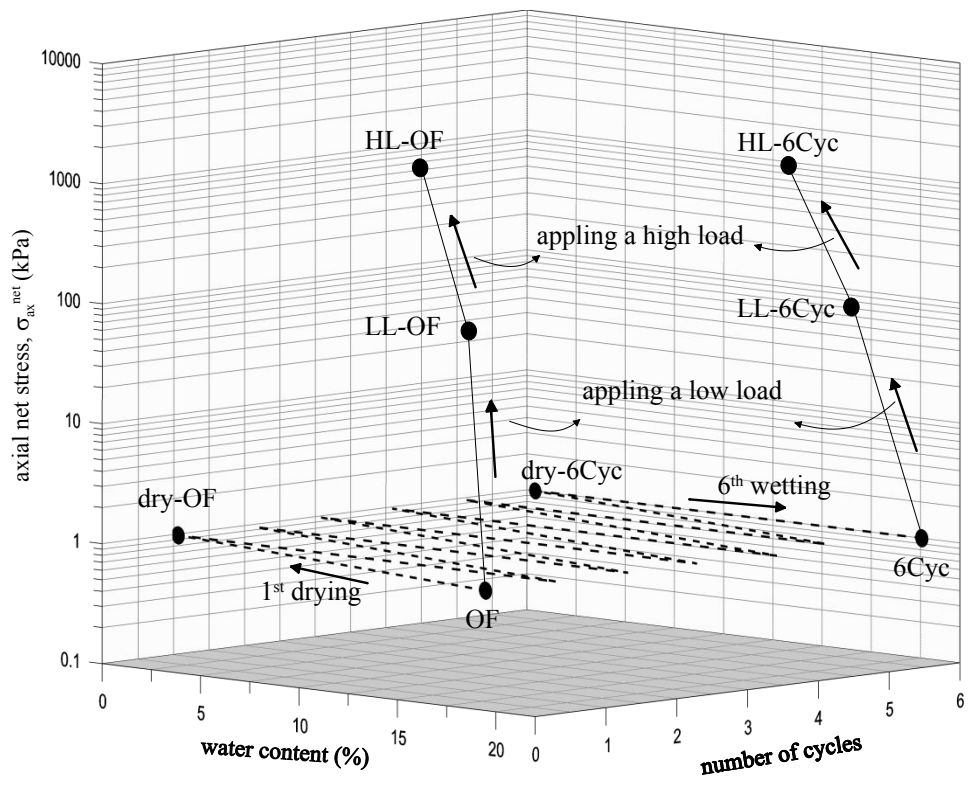


Fig. 2. Hydraulic and mechanical histories of samples prepared for MIP analysis.

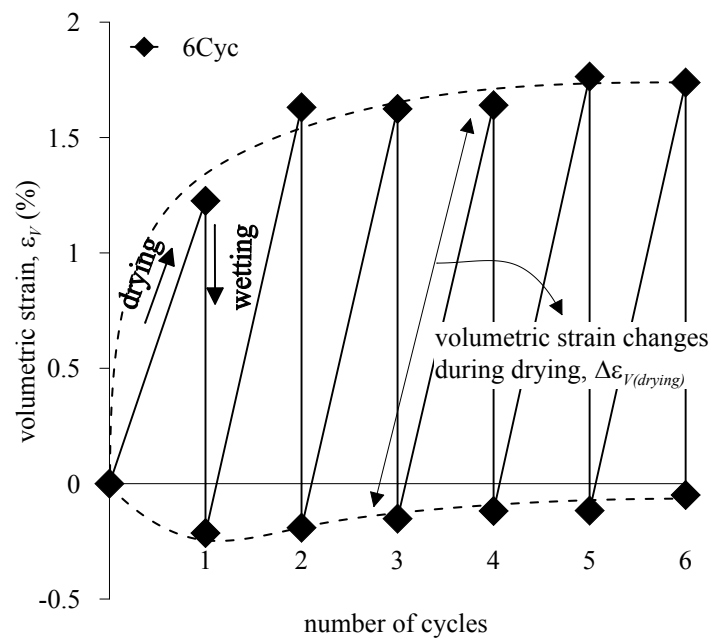
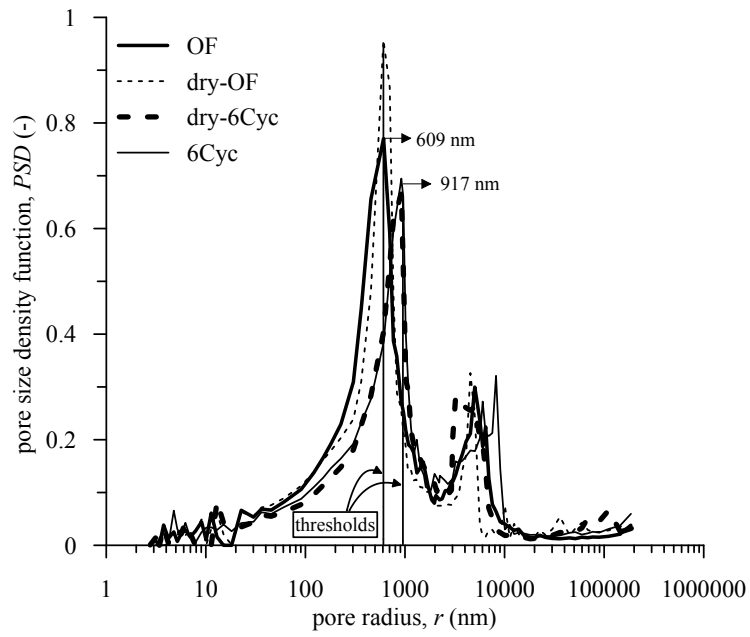
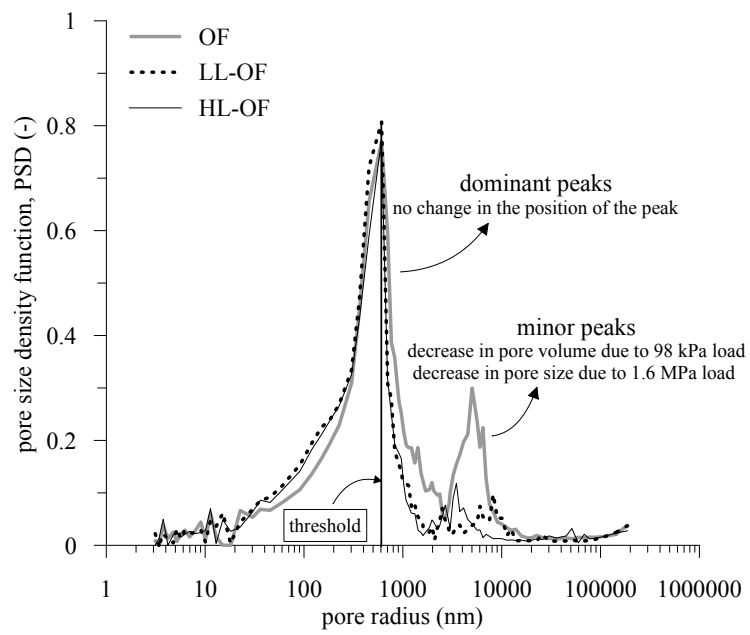


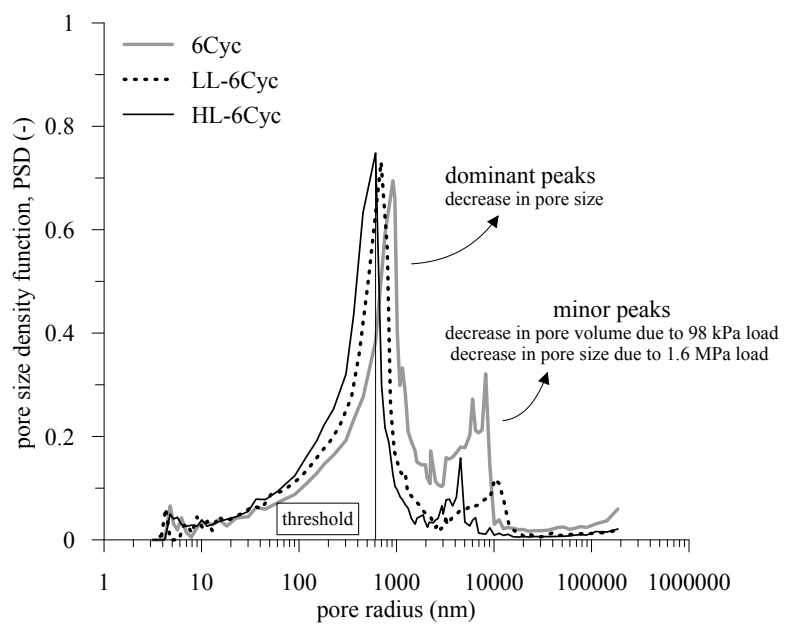
Fig. 3. Volumetric strains during the drying-wetting cycles. Positive volume strains indicate volume decrease, negative volume strains indicate volume increase.



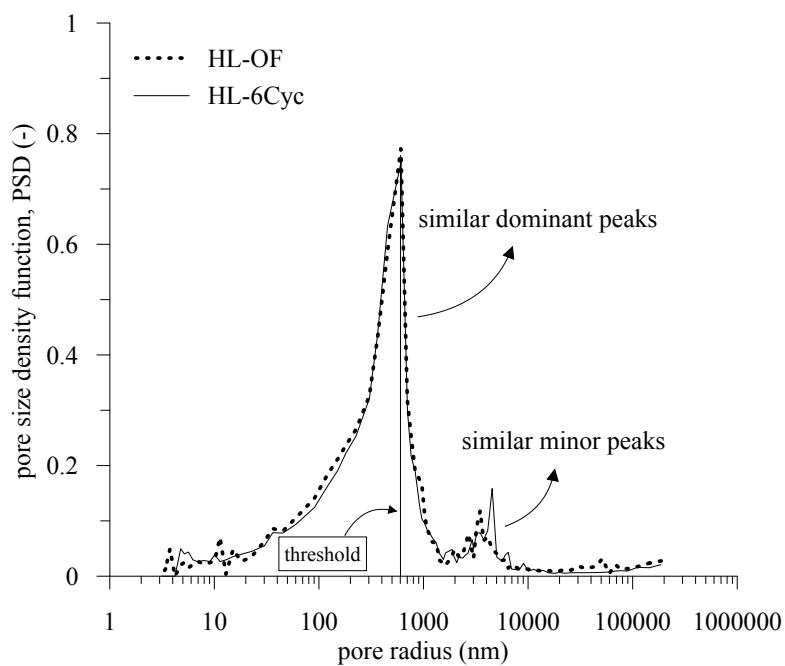
(a)



(b)

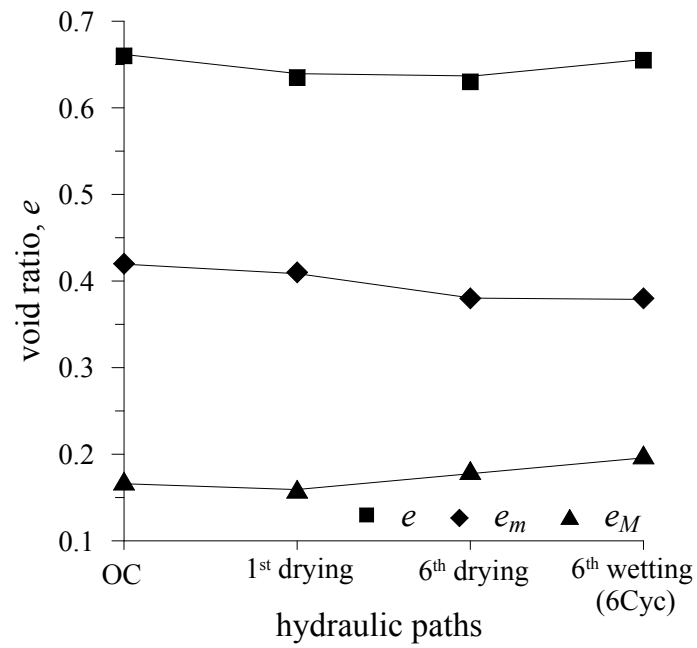


(c)

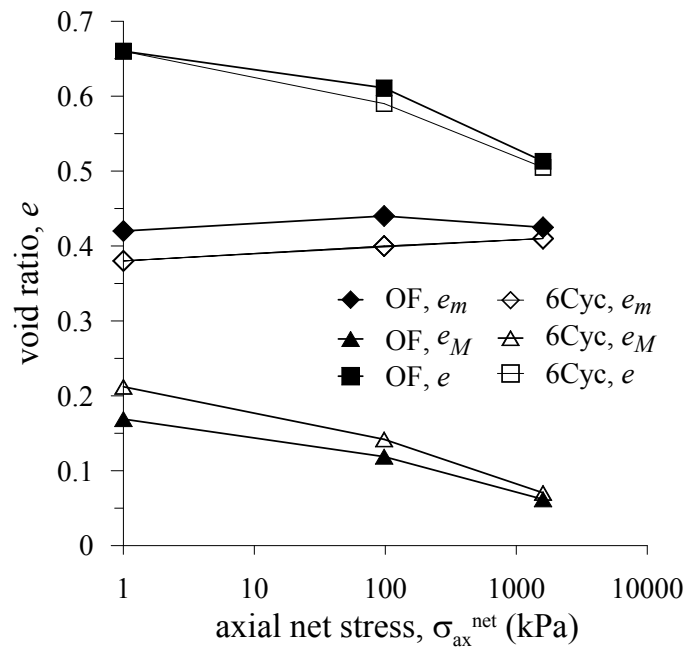


(d)

Fig. 4. Evolution of the PSD with drying-wetting cycles and with mechanical loading: (a) evolving the PSD with drying and wetting; (b) evolving the PSD of OF samples with mechanical loading; (c) evolving the PSD of 6Cyc samples with mechanical loading; (b) the PSDs of OF and 6Cyc samples being subjected to 1.6 MPa.

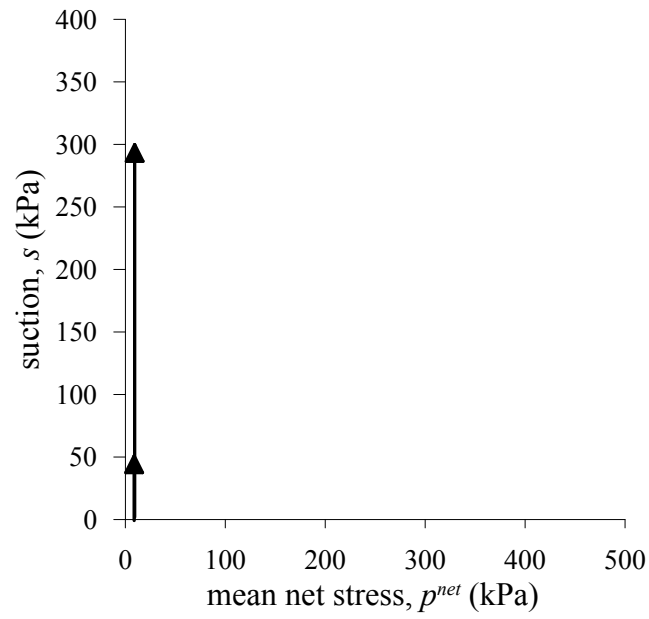


(a)

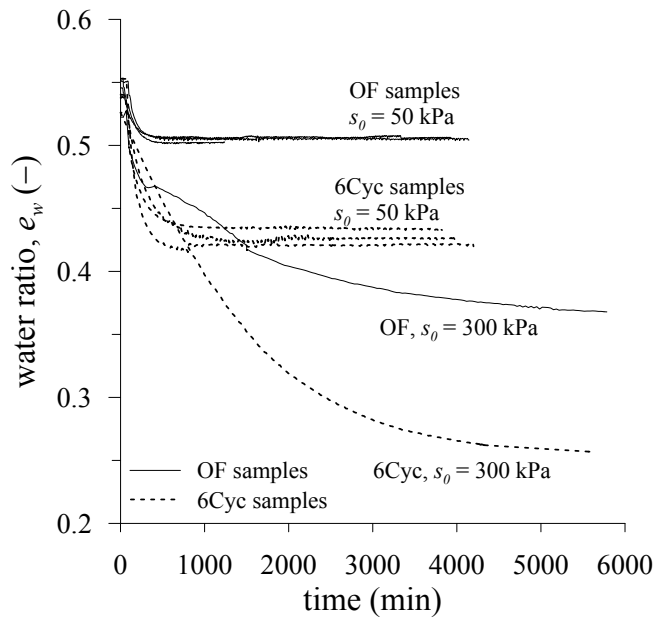


(b)

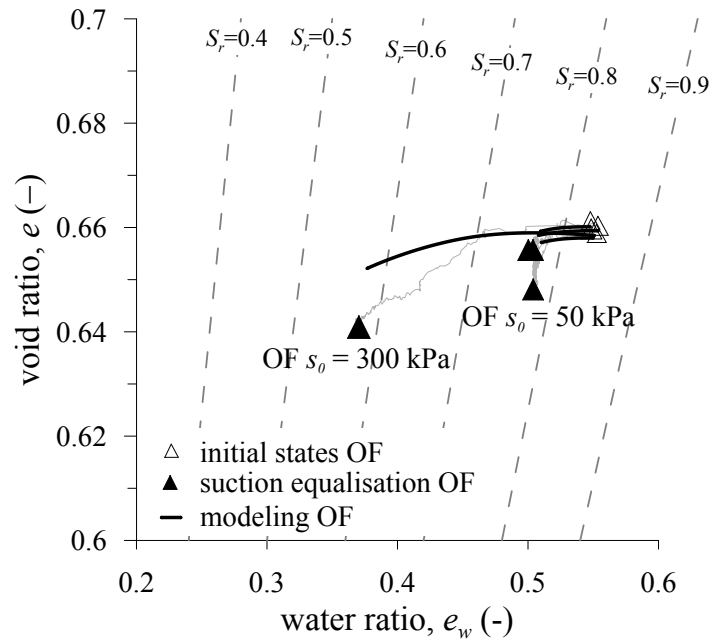
Fig. 5. Evolution of void ratio e , micro-structural void ratio e_m and macro-structural void ratio e_M : (a) with hydraulic loads; (b) with mechanical loads.



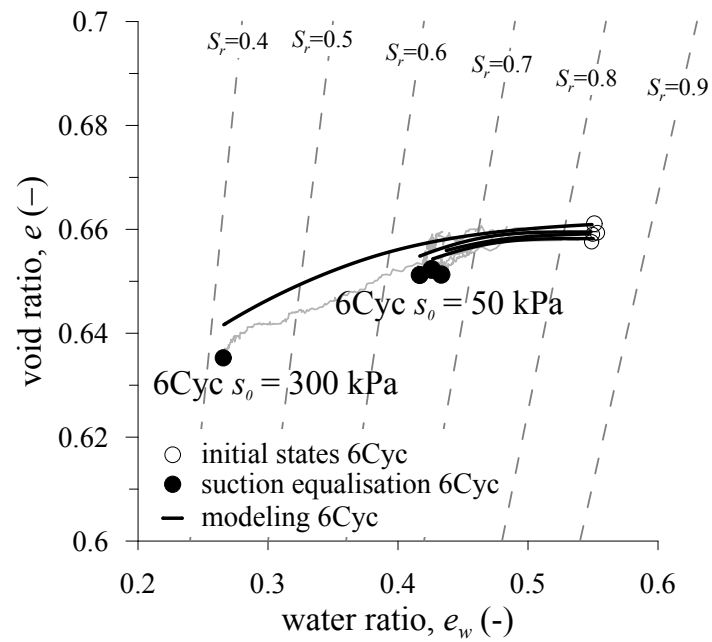
(a)



(b)



(c)



(d)

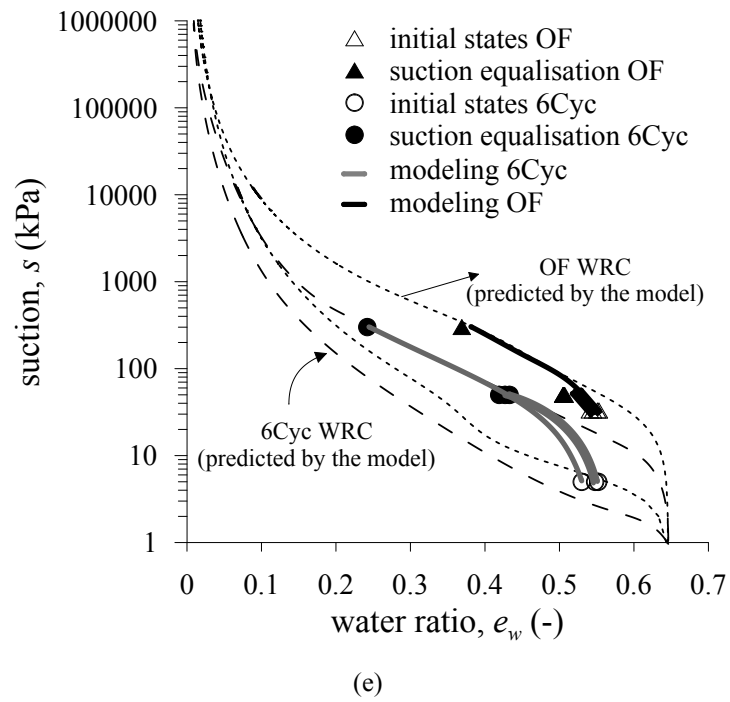
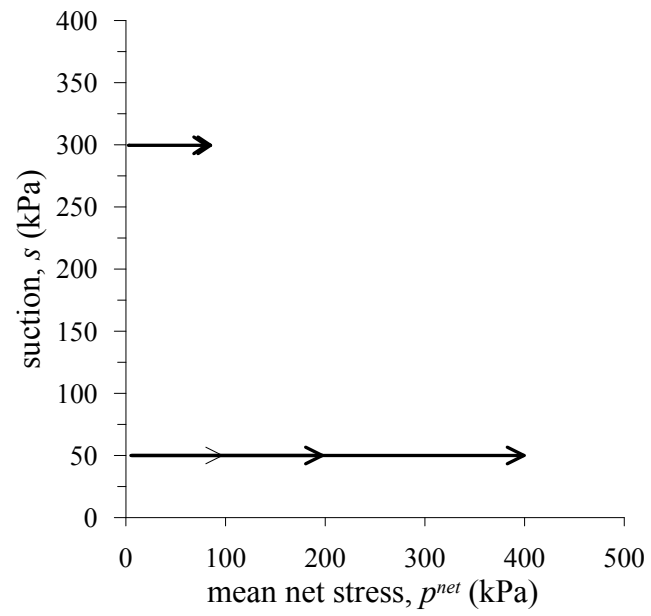
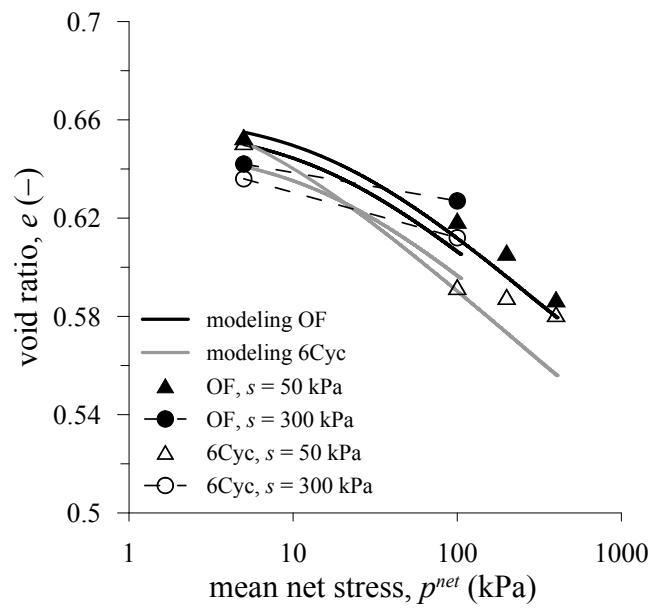


Fig. 6. Behaviour of samples along drying to 50 and 300 kPa of suction: a) stress and hydraulic paths; b) evolution of water ratio with time; c) void ratio-water ratio OF samples; d) void ratio-water ratio 6Cyc samples; e) suction- water ratio.



(a)



(b)

Fig. 7. Isotropic compression at mean net stress $p^{net} = 100, 200$ and 400 kPa; a) stress path; b) void ratio- mean net stress.

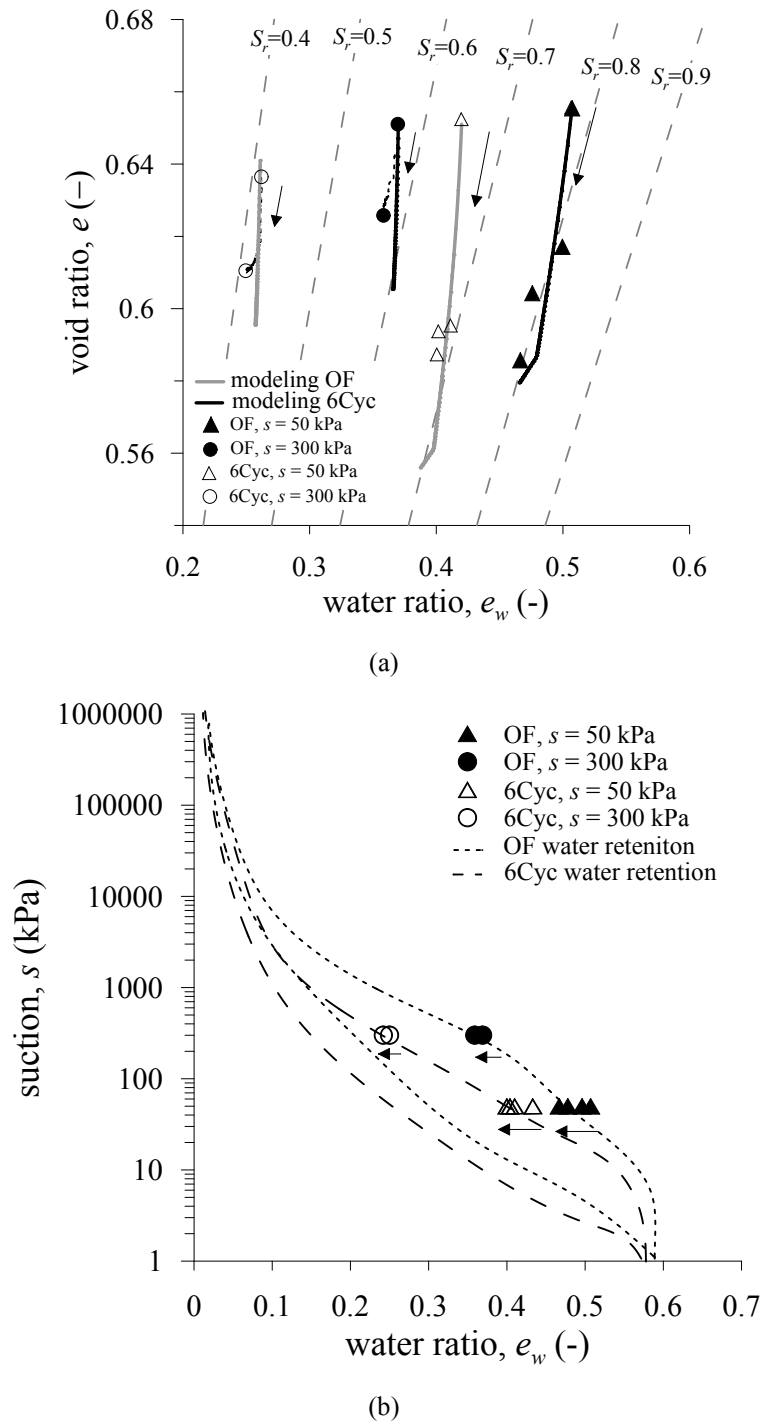


Fig. 8. Behaviour of OF and 6Cyc samples along isotropic compression to mean net stress of $p^{net} = 100, 200$ and 400 kPa: a) void ratio-water ratio; b) suction-water ratio.

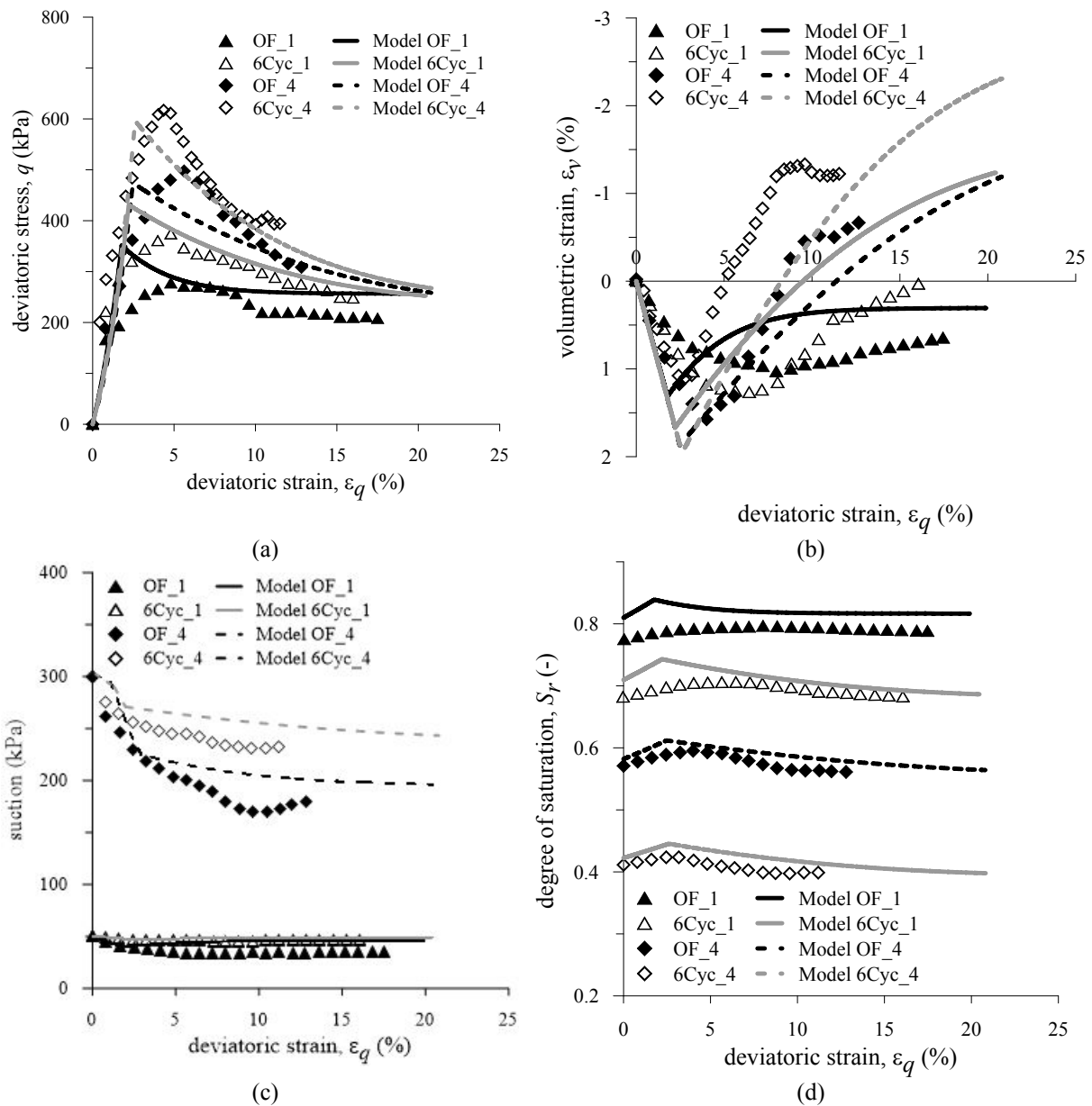


Fig. 9. Behaviour of OF and 6Cyc samples ($s_0 = 50$ and 300 kPa, $p^{net} = 100$ kPa) along triaxial compression at constant water content: a) deviatoric stress – deviatoric strains; b) volume strain – deviatoric strain; c) suction – deviatoric strain; d) degree of saturation – deviatoric strain.

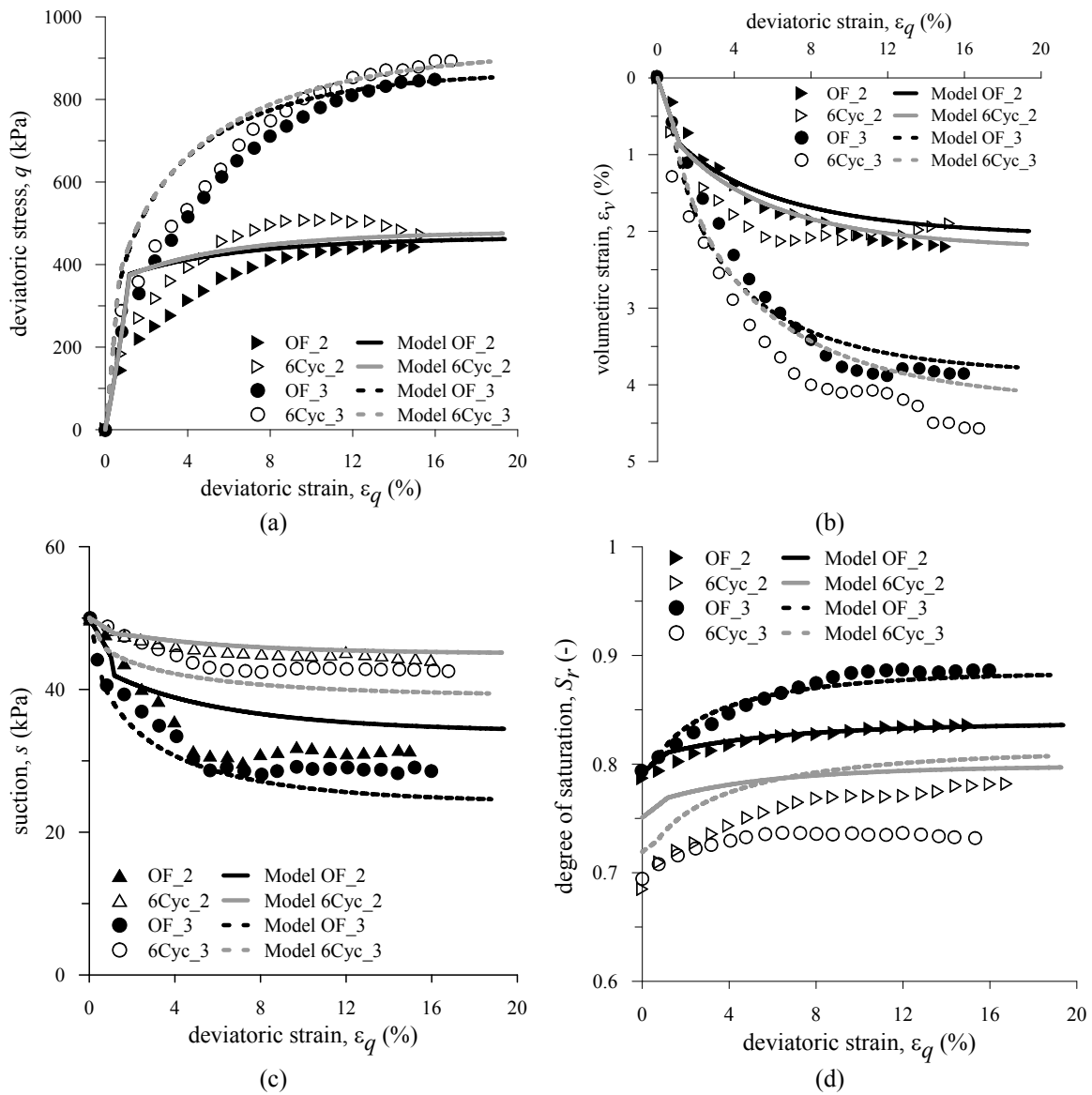
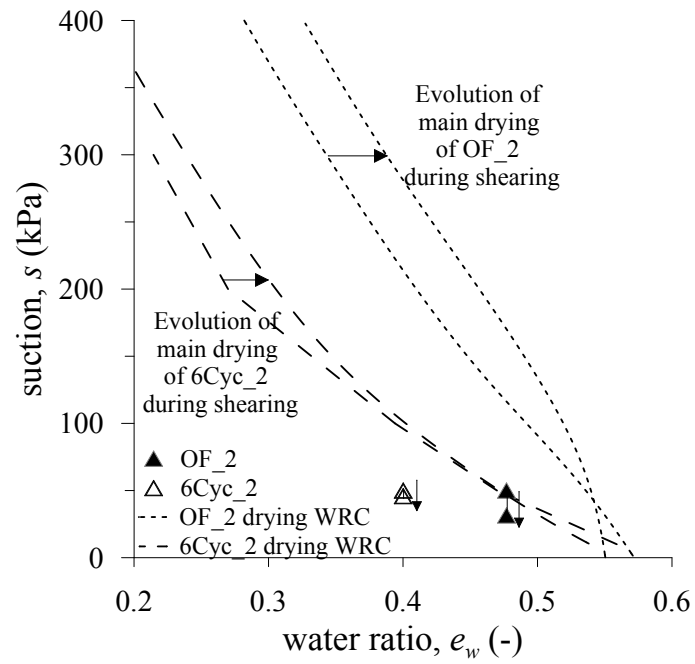
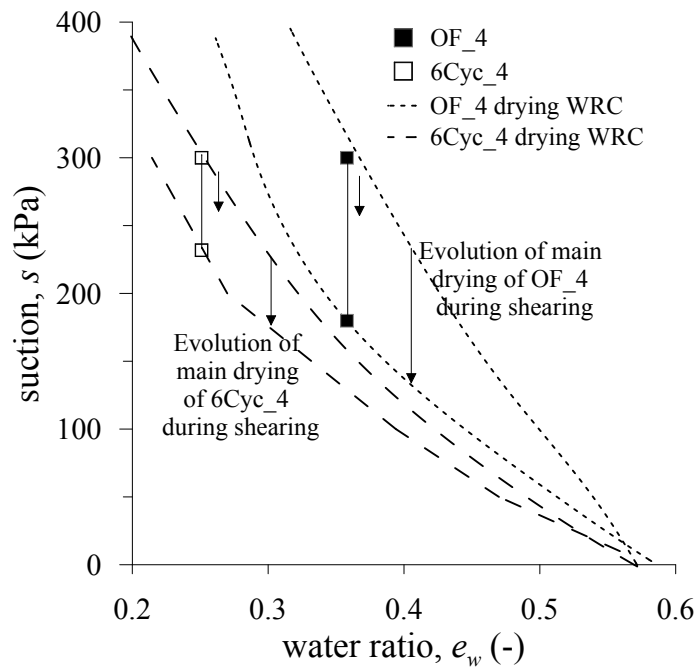


Fig. 10. Behaviour OF and 6Cyc samples ($s_0 = 50$ kPa, $p^{net} = 200$ kPa and 400 kPa) along triaxial compression at constant water content: a) deviatoric stress – deviatoric strains; b) volume strain – deviatoric strain; c) suction – deviatoric strain; d) degree of saturation – deviatoric strain.

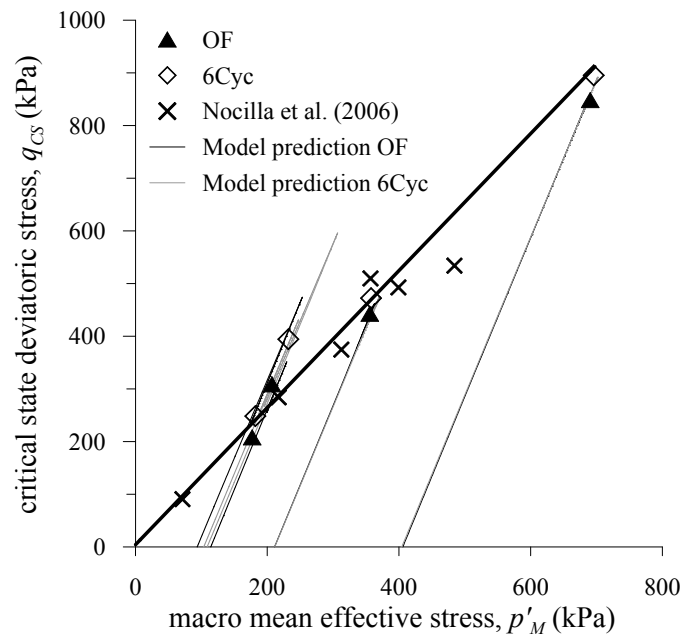


(a)

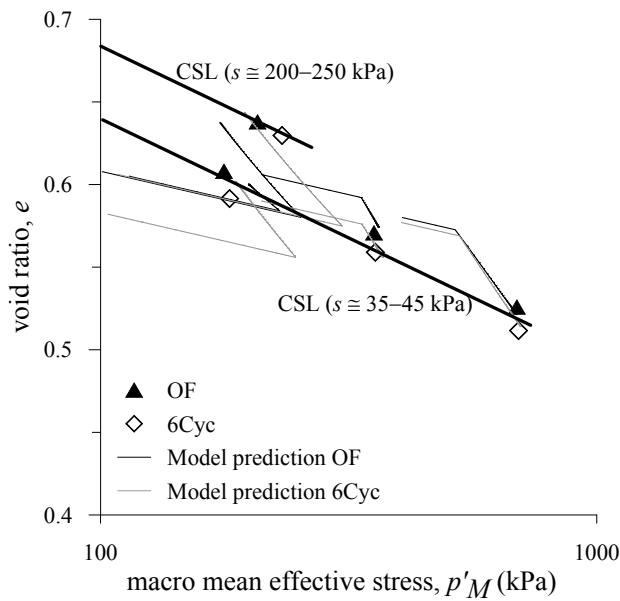


(b)

Fig. 11. Behaviour of OF and 6Cyc samples ($s_0 = 50$ kPa and $s_0 = 300$ kPa, $p_{net} = 100, 200$ and 400 kPa) during the triaxial compression phase: (a) void ratio-water ratio for samples with $s_0 = 50$ kPa; b) suction-water ratio for samples with $s_0 = 300$ kPa.

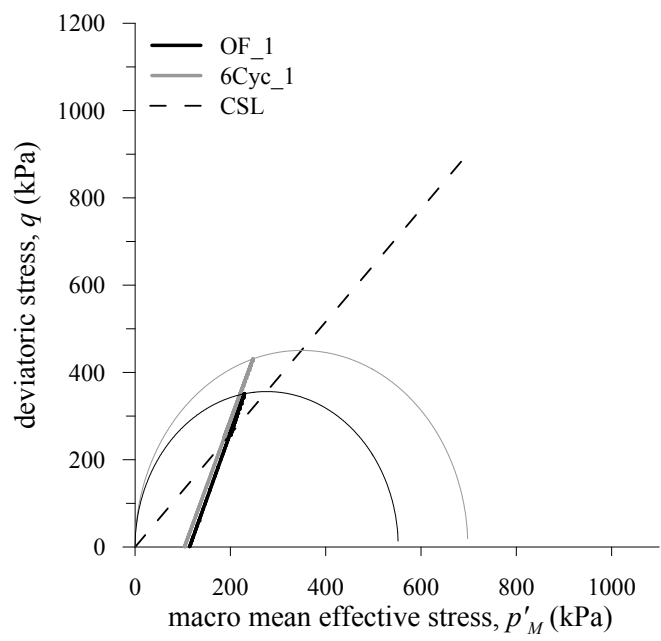


(a)

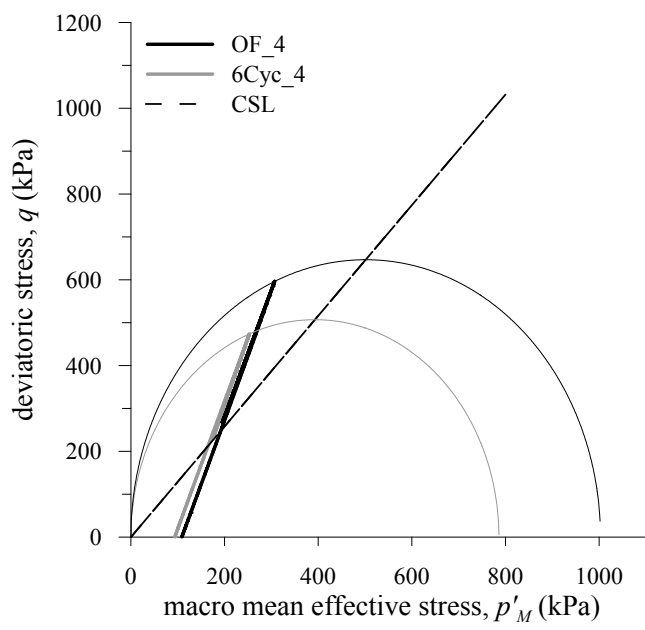


(b)

Fig. 12. Interpretation of final testing conditions and model predictions in terms of macroscopic average skeleton stress of the macrostructure: (a) deviatoric – mean stress plane; (b) compression plane.

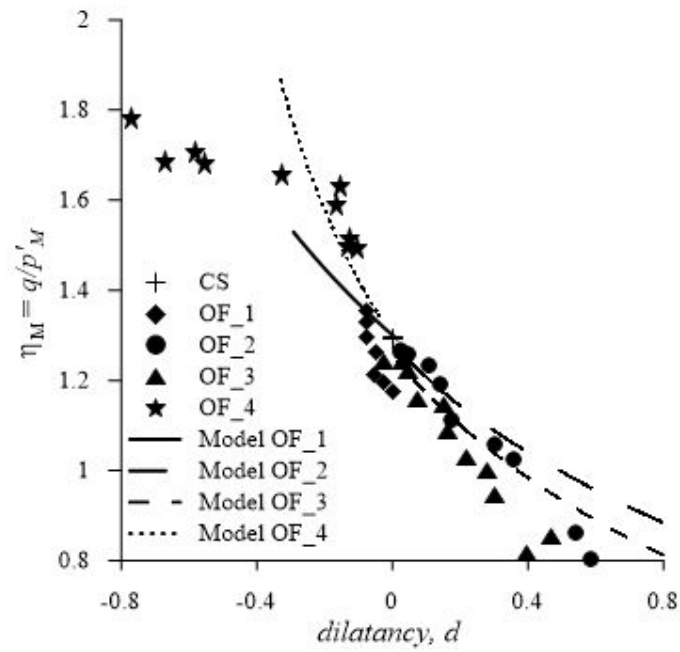


(a)

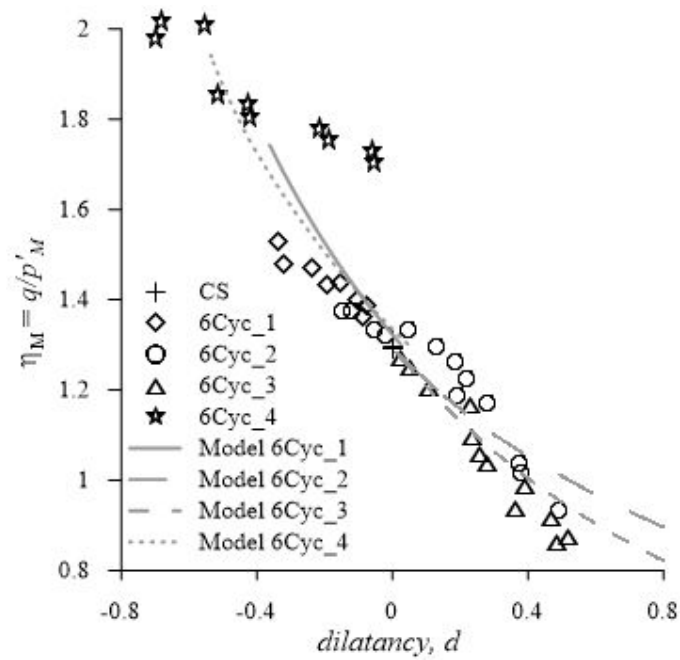


(b)

Fig. 13. Stress paths and initial position of the yield curve: (a) OF_1 and 6Cyc_1 ($s_0 = 50$ kPa); (b) samples OF_4 and 6Cyc_4 ($s_0 = 300$ kPa).

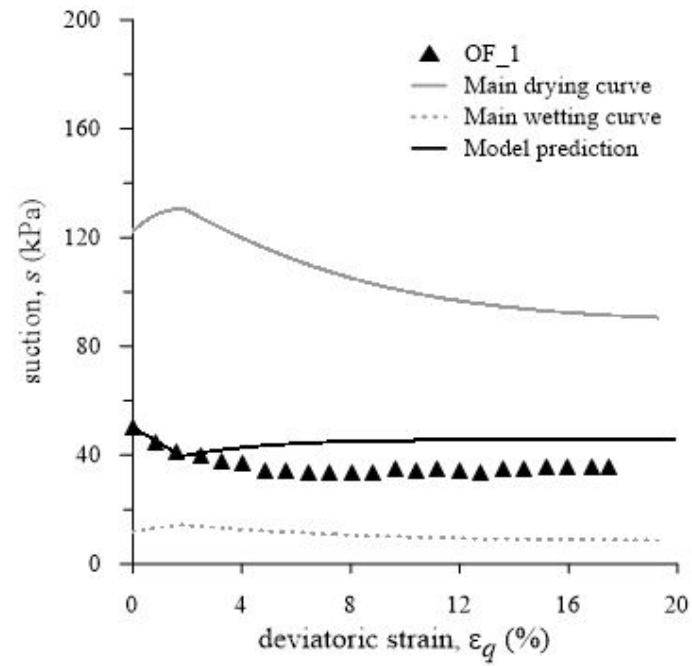


(a)

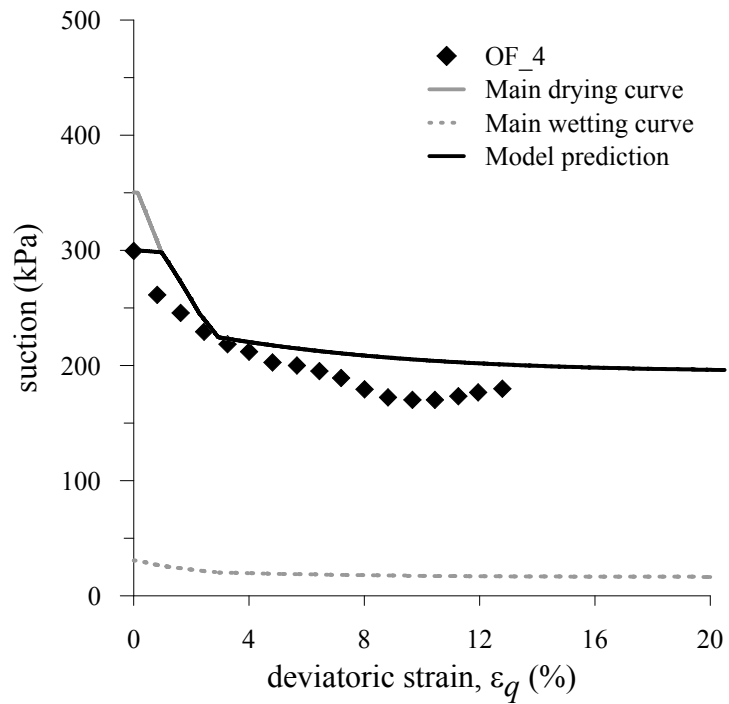


(b)

Fig. 14. Relationship between stress ratio and dilatancy: (a) OF samples; (b) 6Cyc samples.



(a)



(b)

Fig. 15. Changes in suction and evolution of the points lying on the main drying and main wetting curves: (a) test OF_1 ($s_0 = 50$ kPa); (b) OF_4 ($s_0 = 300$ kPa).

Appendix 1 - Experimental techniques

Drying was imposed by exposing the samples to the laboratory environment, having a controlled temperature of 21 °C and relative humidity of 38.5% (suction $s = 128.8$ MPa). The average water content at the end of drying was $w \approx 0.4\%$. Wetting was imposed by placing the samples in the compaction mould with a basement having a plastic porous disc and water conduits. The water was injected at a rate of about 500 mm³/h to bring the water content back to its initial value ($w \cong 20\%$). Vertical deformations were allowed during wetting while radial ones were constrained by the steel frame. At the end of wetting the samples were dismantled, wrapped up in plastic bags and held in a sealed humid container for at least 5 days to ensure water equalisation.

All the MIP specimens (height and diameter of about 10 mm) were previously freeze-dried to preserve the soil fabric at its natural water content (Delage and Pellerin 1984). A Micromeritics AutoPore IV 9500 was used, injecting mercury into to pore network under vacuum condition. The relationship between the apparent pore radius (r) and the absolute injection pressure (p) was obtained through Washburn's equation:

$$r = - \frac{2\sigma^{Hg} \cos\theta_{nw}}{p} \quad (A1)$$

where $\sigma^{Hg} = 0.484$ N/m at 25 °C is the surface tension of the mercury and $\theta_{nw} = 140^\circ$ is the contact angle between the mercury and the pore wall.

The MIP tests were interpreted in terms of pore size density function (PSD) defined at $\log r$:

$$PSD = f(\log r) = \frac{p}{\log e} \frac{d(V_{v0} - V_v)}{V_{v0} dp} \quad (A2)$$

where e is void ratio, V_{v0} is the total volume of pores and $(V_{v0} - V_v)$ is the volume of intruded mercury or the volume of pores with radius equal or greater than r .

The hydromechanical response along drying, isotropic and triaxial compression was studied with a suction-controlled triaxial cell, adopting the axis translation technique. The top and bottom caps at both sides of the specimen were equipped with two concentric porous stones, one for each fluid of concern (air and water). The inner porous stone, a ceramic disc having an Air Entry Value of 500 kPa in the $s_\theta = 300$ kPa tests and an Air Entry Value of 100 kPa in the $s_\theta = 50$ kPa tests, was connected to the water line and it was used to impose the water pressure. The external porous stone, a stainless steel coarse porous ring, was used to impose the air pressure. Local axial displacements were measured on the side of the samples by two miniature LVDTs. An external LVDT contrasted to

the loading ram measured large and necessary displacements. Radial displacements were measured using electro-optical laser sensors mounted outside of the cell, on opposite sides of the sample (as e.g. in Romero 1999).

The triaxial tests consisted of three stages including suction equalization, isotropic and triaxial compression. First the desired suction ($s_0 = 50$ kPa or $s_0 = 300$ kPa) was imposed through the axis translation technique while applying a small net mean pressure p^{net} . The sample was being held to equalize at constant suction until the changes in water content and volume were stabilized.

During isotropic compression, the net mean pressure was increased to the predefined values (100, 200 or 400 kPa) under a free drainage condition while the suction was kept constant. The net pressure was increased at the rate of 50 kPa/h while the deviatoric stress was maintained smaller than 5 kPa, providing isotropic conditions.

During triaxial compression at the constant water content condition, the water drainage was prevented and the pore water pressure was measured by means of a pressure transducer connected to the water porous stone, while the air pressure was kept constant. A constant axial strain rate of 0.25%/h was imparted to ensure that the measured water pressure was in equilibrium with the sample. The cell pressure was kept constant and the axial force was measured.