

1 An insight into the unloading/reloading loops on the compression curve of
2 saturated clays

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24 **Abstract**

25 Oedometer tests were carried out with loading/unloading/reloading on natural saturated
26 Ypresian clay taken from several depths. Common unloading/reloading loops were identified.
27 Further examination of the unloading or reloading curves shows that each path can be
28 satisfactorily considered as bi-linear with a small and a larger slopes separated by a threshold
29 vertical stress. This threshold stress can be considered as the swelling pressure corresponding
30 to the void ratio just before the unloading or reloading. Indeed, upon unloading, when the
31 applied stress is higher than the threshold stress or swelling pressure, the mechanical effect is
32 dominant and only small mechanical rebound is observed, corresponding to a small
33 microstructure change; by contrast, when the applied stress is lower than the swelling
34 pressure, physico-chemical effect becomes prevailing, and soil swelling occurs with a larger
35 microstructure change. Upon reloading, when the applied stress is lower than the swelling
36 pressure, the microstructure is not significantly affected thanks to the contribution of the
37 physico-chemical repulsive force, leading to a small volume change; on the contrary, beyond
38 the swelling pressure, the mechanical effect becomes dominant giving rise to larger volume
39 change corresponding to the microstructure collapse. Like unsaturated expansive soils, it is
40 found that there is a good relationship between the swelling pressure (threshold stress) and the
41 void ratio just before the unloading or reloading. This is confirmed by the results from the
42 data reported in the literature on Boom and London clays. It can be then deduced that the
43 unloading/reloading loop is rather due to the competition between the mechanical and
44 physico-chemical effects on the microstructure changes than the viscosity effect as commonly
45 admitted.

46 **Keywords:** clays; unloading/reloading loops; oedometer tests; mechanical effect, physico-
47 chemical effect; swelling pressure.

48

49 **Introduction**

50 It is well known that the soil compression curves show unloading/reloading loops. These
51 loops have been commonly explained by the soil viscosity effect: clayey soils have larger
52 loops because of their relatively higher viscosity, and sandy soils have narrow loops because
53 of their low viscosity, silty soils being in between. Indeed, Coop & Lee (1993) performed
54 compression tests on the Ham River sand and Dogs bay sand, and the results from
55 unloading/reloading paths show a negligible hysteresis without any marked loops. However,
56 the unloading/reloading loop is quite clear for silty soils (see Nasreddine 2004 for instance),
57 and becomes much more marked for clayey soils (Holtz et al. 1986), bentonite (Borgesson et
58 al. 1996), bentonite/sand mixtures (Tong & Yin 2011), kaolin/bentonite mixture (Di Maio et
59 al. 2004), and claystone (Mohajerani et al. 2011). When investigating the compressibility of a
60 soil, these loops are often ignored and an average slope is usually considered in the
61 calculation of soil volume change. Obviously, this practice is relatively easy for sandy soils
62 and silty soils, but difficult for clayey soils and even impossible for expansive soils.

63 From a fundamental point of view, explaining the unloading/reloading loops by viscosity
64 effect seems too simplistic and unclear. Further study on the corresponding mechanisms is
65 needed. But to the authors' knowledge, there is up to now no plausible mechanisms developed
66 allowing correctly explaining the soil volume change behaviour under unloading/reloading. In
67 this study, this aspect was investigated by performing oedometer tests with several
68 unloading/reloading cycles on natural Ypresian stiff clays. The results were analysed based on
69 the competition between the mechanical effect and physico-chemical effect occurred during
70 unloading or reloading. A new mechanism related to the soil swelling pressure was proposed.
71 This mechanism was verified by the results from the oedometer tests on other natural stiff
72 clays such as Boom clay and London clay.

73 **Materials and methods**

74 The materials studied are cores sampled from Ypresian formation at four different depths,
75 namely YP43 from 330.14 m - 330.23 m depth, YP64 from 351.20 m - 351.29 m depth, YP
76 73 from 361.30 m- 361.34 m depth, and YP95 from 382.35 m - 382.44 m depth. Their clay
77 mineralogy and physical properties are shown in Table 1 and Table 2, respectively. The four
78 depths have comparable clays fraction (between 48 and 59%) as well as similar smectite
79 content (between 26 and 34%), suggesting similar physical properties in terms of plasticity.
80 However, the values of plasticity index I_p and methylene blue VBS (see Table 2) shows a
81 much lower plasticity of YP43 as compared to the three other depths. YP43 has also the
82 lowest liquid limit ($w_L = 75$) and the highest carbonates content CaCO_3 (10.16%), its other
83 parameters (specific gravity G_s , plastic limit w_p , initial water content w_0 , initial void ratio e_0)
84 being similar to that of other depths. It is possible that the high carbonates content of YP43
85 gives rise a lower level of plasticity as indicated by the values of I_p and VBS in Table 2.
86 Further study is needed to clarify this point.

87 Oedometer tests were performed on soil samples of 50-mm diameter and 20-mm height by
88 hand-trimming from cores that have 1000-mm length and 100-mm diameter. After installing
89 soil sample in the oedometer cell, step loading up to the in situ vertical effective stress (up to
90 point A in Figure 1a) was carried out without contact with water in order to avoid any
91 swelling which would affect the initial soil microstructure (Delage et al., 2007; Cui et al.
92 2009, Deng et al., 2011a, 2011b, 2011c). Note that the in-situ vertical effective stresses (σ'_{v0})
93 were estimated by taking an average mass density of overburden soils equal to 1.9 Mg/m^3
94 (Van Marcke & Laenen, 2005) with an underground water level assumed to be at the ground
95 surface. For a reason of convenience, σ'_{v0} were rounded to 3.2 MPa for all the four depths.

96 The degree of saturation at point A, calculated using the initial degree of saturation (see S_{r0} in
97 Table 2) and the volume change recorded, was found to be 100% for all samples tested.

98 Under σ'_{v0} , the bottom porous stone and the drainage tubes were saturated with the in-situ
99 synthetic water that has the same chemical composition as the field water (A-B). Afterwards,
100 step unloading to 0.125 MPa (B-C), reloading up to 16 MPa (C-D), unloading again to 0.125
101 MPa (D-E), reloading up to 32 MPa (E-F) and finally unloading to 0.125 MPa (F-G) were
102 conducted. The stabilization of volume change was considered as achieved when the vertical
103 strain rate is lower than $5 \times 10^{-4}/8$ h (AFNOR, 1995, 2005).

104

105 **Test results**

106 Figure 1 shows the compression curve for the four depths. For YP43 (Figure 1a), YP64
107 (Figure 1b) and YP73 (Figure 1c), two full unloading/reloading cycles and one extra single
108 unloading were applied, while for YP95 (Figure 1d), only one full cycle and one extra single
109 unloading were applied. Figure 1a shows that the application of the in-situ vertical stress
110 (3.2 MPa) led the soil sample to point A. When saturating the bottom porous stone and the
111 drainage tubes, a negligible volume change (A to B) was observed, confirming that the
112 sample was fully saturated. Upon unloading from B to C, a nearly bi-linear curve was
113 observed: when the stress was higher than a threshold stress σ_{s1} the slope was small, and
114 when the stress was lower than σ_{s1} the slope is significantly larger. Upon reloading from C to
115 D, a tri-linear curve was identified. Below the stress at point B, a nearly bi-linear curve was
116 again observed, with a small slope below a threshold stress σ_{s4} and a larger slope beyond σ_{s4} .
117 Further loading beyond point B gave rise to a larger slope, certainly related to the plastic
118 volume changes. Examination of the unloading/reloading curve B-C-B shows a hysteretic

119 loop. The same phenomenon can be observed when unloading the sample from D to E and
120 reloading the sample from E to F: there is a nearly bi-linear curve from D to E with a small
121 slope beyond a threshold stress σ_{s2} and larger slope below σ_{s2} , there is also a nearly bi-linear
122 curve from E to D with a threshold stress σ_{s3} , and when the stress is beyond point D, the slope
123 is increased because of the plastic volume change. The unloading from F to G confirmed the
124 bi-linearity of the curve with a threshold stress σ_{s3} . Comparison of the unloading slopes shows
125 that both the small slope and large slope were increasing with the maximum stress applied
126 prior to unloading.

127 The same observation can be made on the results from the tests on the other three depths
128 (YP64 – Figure 1b, YP73 – Figure 1c and YP95 – Figure 1d). There are also more or less well
129 defined bi-linear curves for both unloading and reloading paths with threshold stresses.

130 It must be mentioned that the bi-linearity observed in this study should be considered as a
131 particular case because for most soils non-linear curves are often observed. It is likely that
132 there is relatively well defined bi-linearity for natural stiff clays like the studied natural
133 Ypresian clay. Basically, the shape of the curves depends on both the soil nature and soil
134 microstructure.

135 To further analyse the results, in Figure 2, the values of threshold stress in Figure 1 for each
136 depth are presented as a function of the void ratio just before each unloading or reloading (σ_{s1}
137 with e_{i1} ; σ_{s2} with e_{i2} ; σ_{s3} with e_{i3} ; σ_{s4} with e_{i4} ; σ_{s5} with e_{i5}). A good linear relationship is
138 obtained in a semi-logarithmic plane for all the four depths. In order to verify this
139 observation, some results from oedometer tests on other stiff clays from the literature are
140 collected, and shown in Figure 3 together with the results of Ypresian clay. It is observed that
141 the variations of swelling pressure with the initial void ratio before unloading or reloading for
142 Boom clay from Essen taken at different depths (Ess 75, Ess 83, Ess 96, Ess 104, Ess112) and

143 for Boom clay from Mol (Deng et al. 2011a, Horseman et al. 1987) show also good linear
144 functions. Obviously, the relationship between σ_s and e_i is soil nature dependent: the slope is
145 different for different soils.

146 Figure 4 shows the results obtained on the basis of the data reported by Gasparre and Coop
147 (2008) on London clay at Heathrow Airport Terminal 5. It involves six samples from
148 different depths: 7 m (C7), 10 m (B10), 25 m (B25), 28 m (B28), 36 m (A36) and 51 m
149 (A51). Again, a good linear relationship is observed for all samples with unloading/reloading.

150 **Interpretation and discussion**

151 Delage and Lefebvre (1984) showed that when compressing a clayey soil in oedometer,
152 macro-pore collapse occurs first during loading path under stresses higher than the
153 preconsolidation pressure or yield stress of soil, leading to irrecoverable volume changes; this
154 process results in microstructure changes characterized by more and more orientated particles.
155 As Le et al. (2011) indicated with a more orientated microstructure the physico-chemical
156 interaction between clay particles and adsorbed water is enhanced. Indeed, Olson and Mesri
157 (1970) pointed out that the competition between the mechanical effect and physico-chemical
158 effect on soil volume change behaviour depends strongly on the particles geometric
159 arrangement defined by the contact angle between particles: the mechanical effect is more
160 pronounced when the contact angle is large, and on the contrary, the physico-chemical effect
161 becomes dominant when the particles are more parallel with a small contact angle. Thereby,
162 upon mechanical loading in oedometer the contact angle is decreasing, leading to increase of
163 the physico-chemical effect.

164 Basically, the increase of physico-chemical effect due to loading can be well evidenced
165 during unloading: the more orientated particles undergo higher repulsion forces and thus show
166 more significant swell: the unloading curve could have a much greater slope than the initial

167 loading slope before the preconsolidation pressure is reached, and the void ratio after full
168 unloading could even be higher than the initial void ratio. Examination of the unloading
169 slopes in Figure 1 confirms this reasoning: the unloading slope is indeed increasing with
170 increasing yield stress.

171 The description above shows that when unloading from the previous maximum stress
172 (commonly termed as yield stress), the compression behaviour is mainly governed by the
173 competition between the physico-chemical effect and mechanical effect. When the external
174 stress is higher than the repulsive force related to the soil particles-water interaction (physico-
175 chemical effect), low swelling volume change occurs; otherwise higher swelling volume
176 change can be expected. Similarly, upon reloading, when the external stress is lower than the
177 repulsive force related to the physico-chemical effect, this external stress is balanced by the
178 repulsive force, leading to a small volume decrease; on the contrary, when the external stress
179 becomes higher, the mechanical effect becomes dominant giving rise to larger volume
180 decrease. This conceptual description implies that it is possible to determine some
181 characteristic stress that separate the zone with prevailing physico-chemical effect from the
182 zone with prevailing mechanical effect. It is indeed what is observed in Figure 1 with the
183 threshold stress. Upon unloading, when the vertical stress is higher than the threshold stress
184 σ_s , the volume change is characterized by the mechanical rebound; however, in the lower
185 stress range the volume change is characterized by the physico-chemical swelling.

186 Furthermore, if we admit that the physico-chemical effect mentioned above corresponds to
187 the matric suction as we do commonly for expansive soils (saturated or unsaturated), the
188 threshold stress identified should correspond to the swelling pressure. Indeed, the swelling
189 pressure of a soil is commonly defined as the stress under which no volume change occurs
190 upon wetting. According to this definition, swelling takes place when wetting a soil under a

191 stress lower than the swelling pressure; on the contrary, collapse takes place when wetting the
192 soil under a stress higher than the swelling pressure. Thus, the swelling pressure can
193 experimentally be determined by loading the soil samples in oedometer to different vertical
194 stress σ_v' and then wetting them (see Figure 5a). Various other methods can be used for this
195 purpose (see Figure 5b). The “constant-volume” method (path OA) is based on the use of a
196 relatively rigid cell with total pressure measurement (Tang et al., 2011, Wang et al. 2012).
197 The value of pressure after stabilisation is the swelling pressure of soil. For the “zero-swell”
198 method (path OBB’) the equipment employed is a conventional oedometer (Basma et al.,
199 1995; Nagaraj et al., 2009). Firstly, a low initial load (0.1 MPa for example) is applied on the
200 specimen prior to water flooding. As the specimen wets up it attempts to swell. When the
201 swell exceed a certain value (0.1% for example), additional pressure is added in small
202 increment to bring the volume of soil specimen back to its initial value (Basma et al., 1995;
203 Attom et al., 2001). This operation is repeated until the specimen ceases to swell. The
204 swelling pressure is defined as the stress under which no more swelling occurs. The “swell-
205 consolidation” method (path OCC’) consists of re-saturating the soil under a low pressure (0.1
206 MPa for example). After swell completion, standard consolidation test is performed. The
207 pressure required to bring the soil specimen back to its original void ratio is defined as the
208 swelling pressure (Basma et al., 1995; Agus, 2005).

209 The small mechanical rebound upon unloading implies an insignificant microstructure
210 change, i.e., the microstructure pattern remains rather orientated with dominated face-to-face
211 particle contacts (see Figure 6). When the external stress is lower than the threshold stress or
212 swelling pressure σ_s , the prevailing physico-chemical effect leads to a significant
213 microstructure change characterized by formation of more and more face-to-edge particle
214 contacts. This microstructure change corresponds to a larger soil swelling. From this
215 description, the threshold stress or swelling pressure σ_s separates the zone with insignificant

216 microstructure change from the zone with significant microstructure change. The increase of
217 this swelling pressure with loading ($\sigma_{s1} < \sigma_{s2} < \sigma_{s3}$) observed in Figure 1 is also consistent
218 with the common results on unsaturated expansive soils: a higher stress causes lower void
219 ratio or higher density, increasing the swelling pressure (Villar et al., 2008; Siemens et al.,
220 2009; Wang et al., 2012).

221 The reasoning above applies also for the reloading path. When the external stress is lower
222 than the swelling pressure, the matric suction related to the physico-chemical effect is high
223 enough to balance the effect of external stress, and the current microstructure changes
224 insignificantly (with the face-to-edge particles preserved). As a result, the slope of the
225 compression curve is small. By contrast, when the external stress is higher than the swelling
226 pressure, the mechanical effete becomes dominant and larger volume change occurs by
227 collapse of large-pores (the particles arrangement tends towards face-to-face type), leading to
228 a larger slope of the compression curve. This is also consistent with the swelling pressure
229 definition shown in Figure 5.

230 It should be mentioned that the model used above for microstructure changes during
231 unloading/reloading is totally conceptual. It should be confirmed by experimental evidences
232 provided in further studies implying appropriate techniques. Theoretically, the more plastic
233 the soil is (high smectite content, high plasticity index I_p , large methylene blue value, etc.),
234 the more microstructure changes from face-to-face pattern to face-to-edge pattern can be
235 expected.

236 Figures 2-4 show that there is a linear relationship between swelling pressure and the void
237 ratio before unloading or reloading in a semi-logarithmic scale for natural stiff clays as
238 Ypresian clay, Boom clay and London clay, although this relationship is soil nature
239 dependent. This is consistent with the results from the tests on unsaturated expansive soils,

240 thereby bring another evidence for the identified threshold stress as swelling pressure. Note
241 that for other soils there is not necessarily bi-linear unloading/reloading curves, thus, difficult
242 to determine the relationship between the swelling pressure and void ratio before unloading or
243 reloading. However, it is believed that the conceptual model developed in this study would be
244 still applicable.

245 **Conclusion**

246 Oedometer tests were carried out with loading/unloading/reloading on saturated natural
247 Ypresian clay from different depths. The unloading/reloading loops of compression curves of
248 Ypresian clay have been explained by the competition between the mechanical and physico-
249 chemical effects. Upon unloading, when the applied stress is higher than the swelling
250 pressure, the mechanical effect is dominant and only small mechanical rebound is observed;
251 by contrast, when the applied stress is lower than the swelling pressure, physico-chemical
252 effect becomes dominant, giving rise to soil swelling with a larger slope. Upon reloading,
253 when the applied stress is lower than the swelling pressure, the microstructure is more or less
254 preserved thank to the contribution of the matric suction, leading to a small volume change;
255 on the contrary when the applied stress is higher than the swelling pressure, the mechanical
256 effect prevails and larger volume change occurs by collapse of large-pores. The good linear
257 relationship between the swelling pressure and the void ratio just before unloading or
258 reloading confirms this interpretation. Further examination of Boom clay from both Essen and
259 Mol sites as well as London clay brings further confirmation to this concept. It can be then
260 concluded that the unloading/reloading loop is rather due to the competition between the
261 mechanical and physico-chemical effects than the viscosity effect as commonly admitted.
262 Note however that this interpretation was done only based on the oedometer tests on natural

263 stiff clays. Further studies are needed to make more clarification, especially in terms of
264 microstructure investigation.

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266 **References**

267 AFNOR, 1995. Sols : reconnaissance et essais: essai de gonflement à l'oedomètre,
268 détermination des déformations par chargement de plusieurs éprouvettes. XP P 94-091.

269 AFNOR, 2005. Geotechnical investigating and testing, Laboratory testing of soils, Part 5:
270 Incremental loading Oedometer test. XP CEN ISO/TS 17892-5.

271 Agus, S., 2005. An Experimental study on hydro-mechanical characteristics of compacted
272 bentonite-sand mixtures. PhD thesis. Weimar.

273 Attom, M., Abu-Zreig, M. & Obaidat, M., 2001. Changes in clay swelling and shear strength
274 properties with different sample preparation techniques. Geotechnical Testing Journal,
275 ASTM, 24, 157-163.

276 Basma,A.A., Al-Homoud, A.S., Husein, A., 1995. Laboratory assessment of swelling
277 pressure of expansive soils. Applied Clay Science, 9(5), 355–368.

278 Borgesson, L., Karnland, O., Johannesson, L. E., 1996. Modelling of the physical behaviour
279 of clay barriers close to water saturation. Engineering Geology, 41, 127-144.

280 Coop, M. R., Lee, I. K., 1993. The behaviour of granular soils at elevated stresses. Proc.
281 Wroth Memorial Symposium: Predictive soil mechanics, pp. 186 - 198.

282 Cui Y.J., Le T.T., Tang A.M., Delage P., Li X.L., 2009. Investigating the time dependent
283 behaviour of Boom clay under thermo-mechanical loading. Géotechnique, 59 (4), 319-
284 329.

285 Delage P., Lefebvre G., 1984. Study of the structure of a sensitive Champlain clay and of its
286 evolution during consolidation. Canadian Geotechnical Journal, 21 (1), 21-35.

287 Delage P., Le T.T., Tang A.M., Cui Y.J., Li X.L., 2007. Suction and in-situ stresses of deep
288 Boom clay samples. *Géotechnique*, 57(1), 239-244.

289 Deng Y. F., Tang A. M., Cui Y. J., Nguyen X. P., Li X. L., Wouters L., 2011a. Laboratory
290 Hydro-mechanical Characterisation of Boom Clay at Essen and Mol. *Physics and
291 Chemistry of the Earth*, 36 (17-18), 1878–1890.

292 Deng Y.F., Tang A.M., Cui Y.J., Li X.L., 2011b. A study on the hydraulic conductivity of
293 Boom clay. *Canadian Geotechnical Journal*, 48, 1461-1470.

294 Deng Y. F., Cui Y. J., Tang A. M., Nguyen X. P., Li X. L., Van Geet M., 2011c. Investigating
295 the pore-water chemistry effects on the volume change behaviour of Boom clay. *Physics
296 and Chemistry of the Earth*, 36 (17-18), 1905–1912.

297 Di Maio, C., Santoli, L., Schiavone, P., 2004. Volume change behaviour of clays: the
298 influence of mineral composition, pore fluid composition and stress state. *Mechanics of
299 Materials*, 36, 435–451.

300 Gasparre, A., Coop, M. R., 2008. Quantification of the effects of structure on the compression
301 of a stiff clay. *Canadian Geotechnical Journal*, 45(9), 1324-1334.

302 Holtz, R. D., Jamiolkowski, M. B., Lancellotta, R., 1986. Lessons from oedometer tests on
303 high quality samples. *Journal of Geotechnical Engineering*, 112(8), 768-776.

304 Horseman, S.T., Winter M.G., Entwistle D.C., 1987. Geotechnical characterization of Boom
305 clay in relation to the disposal of radioactive waste. Commission of the European
306 Communities, EUR 10987, 87 p.

307 Le T.T., Cui Y.J., Munoz J.J., Delage P., Tang A.M., Li X., 2011. Studying the stress-suction
308 coupling in soils using an oedometer equipped with a high capacity tensiometer. *Front.
309 Archit. Civ. Eng.China*, 5(2), 160–170.

310 Mohajerani M., Delage P., Monfared M, Tang A.M., Sulem J., Gatmiri, B. 2011. Oedometer
311 compression and swelling behaviour of the Callovo-Oxfordian argillite. *International*

312 Journal of Rock Mechanics and Mining Sciences, 48(4), 606 – 615.
313 (doi:10.1016/j.ijrmms.2011.02.01)

314 Nagaraj, H. MOHAMMED Munnas, M. Sridharan,A., 2009. Critical Evaluation of
315 Determining Swelling Pressure by Swell-Load Method and Constant Volume Method,
316 Geotechnical Testing Journal, ASTM, 32, 305-314.

317 Nasreddine, K. 2004. Effet de la rotation des contraintes sur le comportement des sols
318 argileux. PhD thesis, Ecole Nationale des Ponts et Chaussées, Paris.

319 Olson R. O., Mesri G., 1970. Mechanisms controlling compressibility of clays. Journal of Soil
320 Mechanics and Foundation Division, Proceeding of the American Society of Civil
321 Engineers. SM6, pp. 1863-1878

322 Siemens, G., Blatz, J.A., 2009. Evaluation of the influence of boundary confinement on the
323 behaviour of unsaturated swelling clay soils. Canadian Geotechnical Journal, 46(3), 339–
324 356.

325 Tang C. S., Tang A. M., Cui Y. J., Delage P., Sshroeder C., De Laure E., 2011. Investigating
326 the Swelling Pressure of Compacted Crushed-Callovo-Oxfordian Argillite. Physics and
327 Chemistry of the Earth, 36 (17-18), 1857–1866.

328 Tong, F., Yin, J. H., 2011. Nonlinear Creep and Swelling Behavior of Bentonite Mixed with
329 Different Sand Contents Under Oedometric Condition. Marine Georesources &
330 Geotechnology, 29(4), 346-363.

331 Vandenberghe N., 2011. Qualitative & quantitative mineralogical analyses on the Ypresian
332 clay. Report of Applied Geology & Mineralogy Group of K. U. Leuven to NIRAS-
333 ONDRAF.

334 Van Marcke Ph., Laenen B. 2005. The Ypresian clays as possible host rock for radioactive
335 waste disposal: an evaluation. Report of Belgian agency for radioactive waste and
336 enriched fissile materials, 149p.

- 337 Villar, M.V., Lloret, A., 2008. Influence of dry density and water content on the swelling of a
338 compacted bentonite. *Applied Clay Science*, 39(1-2), 38–49.
- 339 Wang Q., Tang A.M., Cui Y.J., Delage P., Gatmiri B. 2012. Experimental study on the
340 swelling behaviour of bentonite/claystone mixture. *Engineering Geology*, 124, 59-66.
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358 Figure 6. Representation of microstructure changes during unloading/reloading

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Table 1. Mineralogy of Ypresian clay (Vandenberghe et al., 2011)

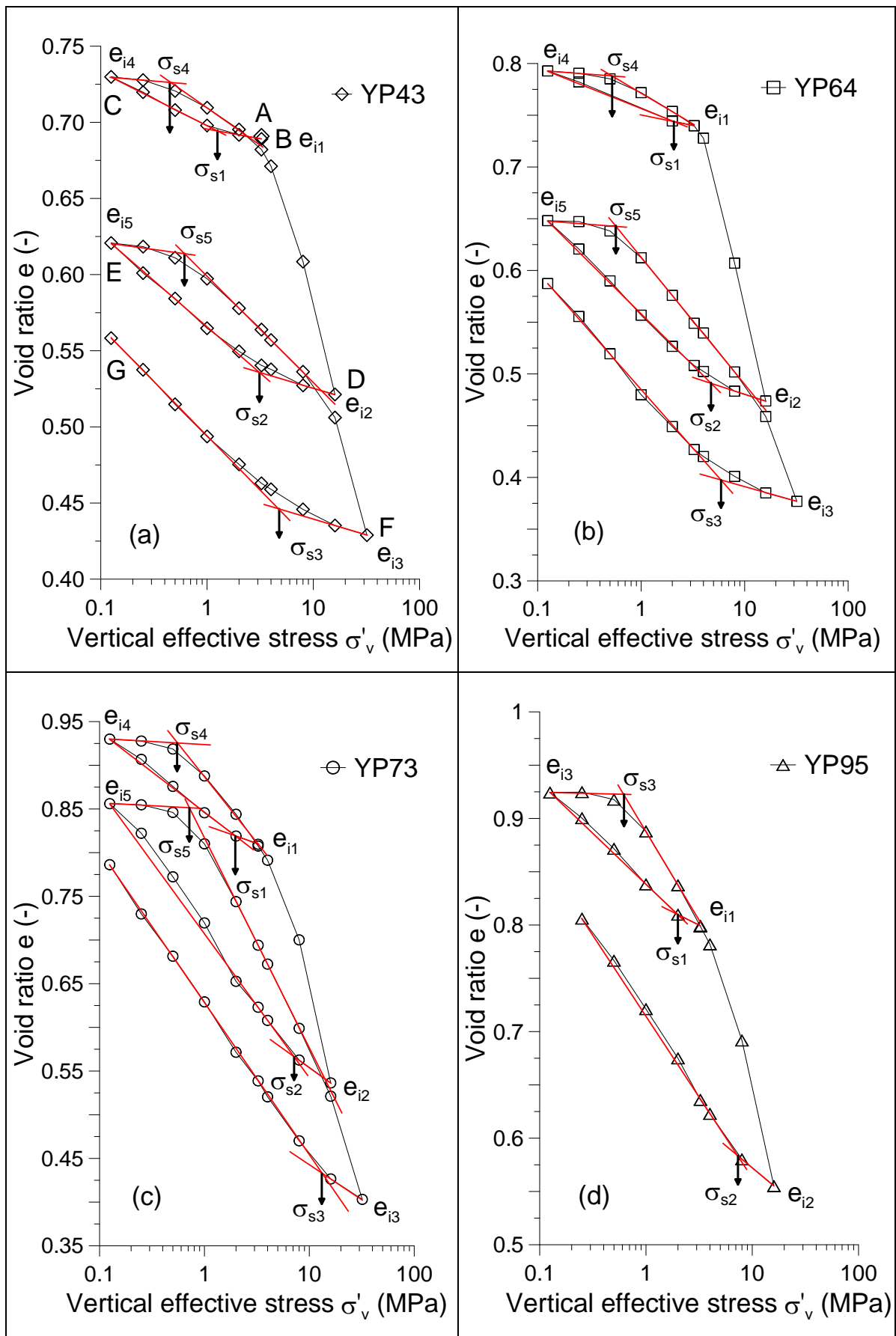
Mineral	YP43	YP64	YP73	YP95
Depth (m)	330.14-330.23	351.20-351.29	361.30-361.34	382.35-382.44
Σ Clay (% wt)	54	48	57	59
Illite	5	4	5	6
Kaolinite	2	5	3	3
Smectite	32	26	33	34
Illite/Smectite	12	10	12	13
Chlorite/others	3	3	4	3
Σ Non-clay (% wt)	46	52	43	41
Quartz	32	36	31	27
K-feldspar	6	7	6	7
Plagioclase	6	6	5	3
Others	2	3	1	4

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366

Table 2. Physical properties of Ypresian clay

Soil	G_s (-)	w_L (-)	w_P (-)	I_P (-)	w_0 (%)	e_0 (-)	S_{r0} (%)	VBS (g/100g)	CaCO ₃ (%)
YP43	2.78	75	34	42	26	0.81	89	3.95	10.16
YP64	2.79	114	34	80	26	0.79	92	7.09	1.38
YP73	2.80	137	36	101	31	0.94	93	13.12	0.88
YP95	2.80	132	44	88	30	0.95	87	12.72	3.82

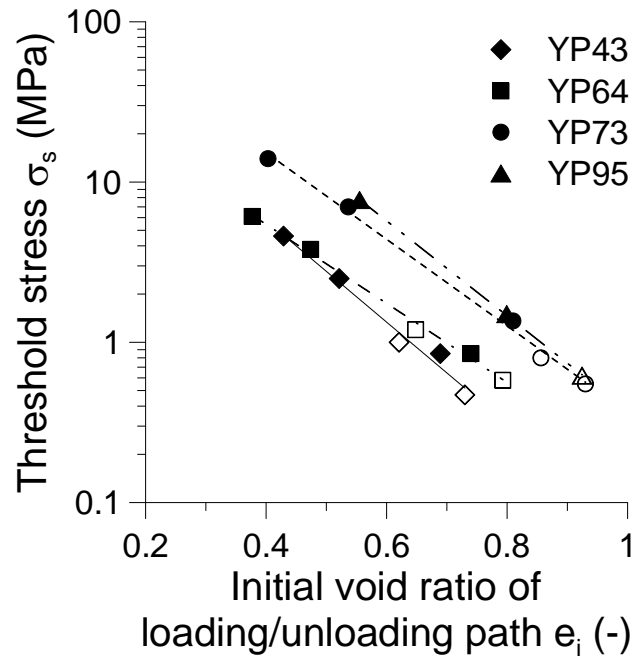


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Figure 1. Oedometer test with unloading/reloading on Ypresian clays

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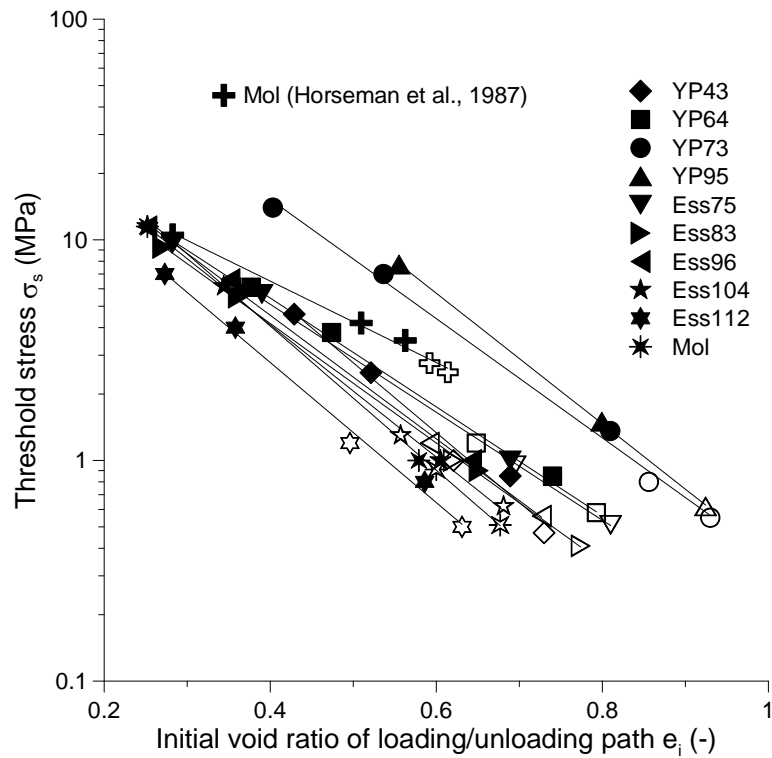


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372 Figure 2. Swelling stress versus the void ratio just before unloading or reloading for Yprecian
373 clays - Closed symbols are for unloading path, open symbols for loading path

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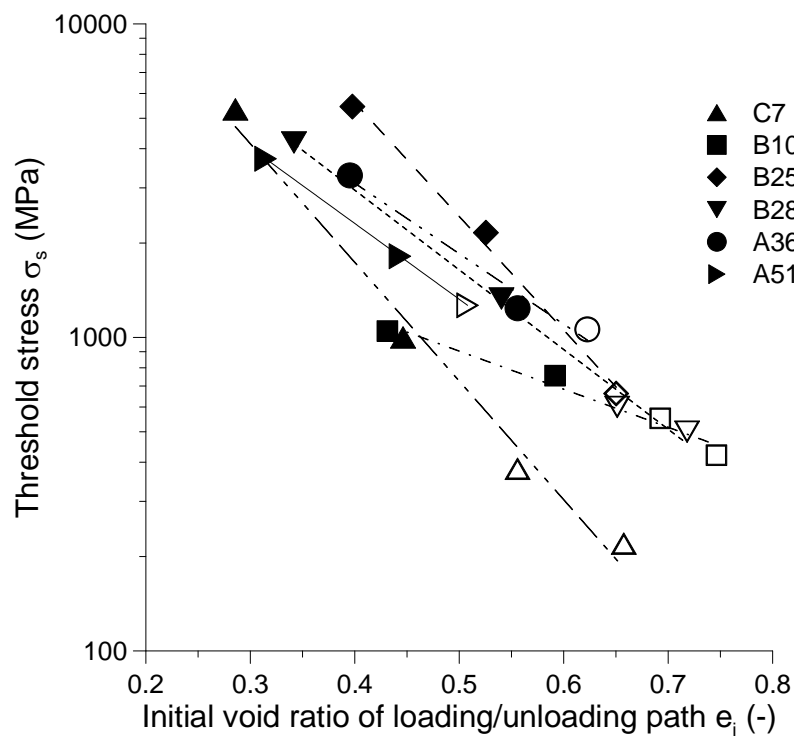
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377 Figure 3. Swelling stress versus the void ratio just before unloading or reloading for Boom
 378 (from both Essen and Mol sites) and Ypresian clays. - Closed symbols are for unloading path,
 379 open symbols for loading path

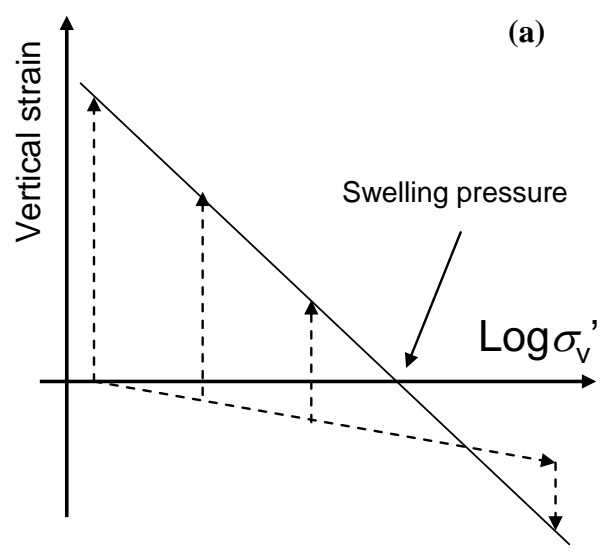
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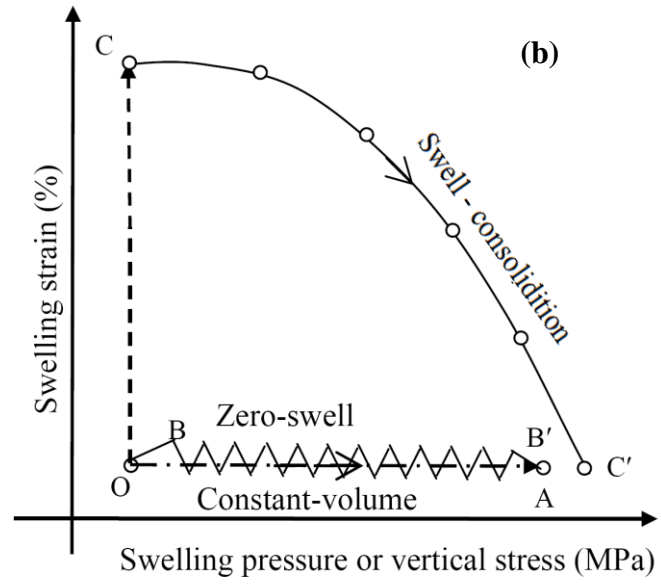
381

382 Figure 4. Swelling stress versus the void ratio just before unloading or reloading for London
 383 clay. - Closed symbols are for unloading path, open symbols for loading path

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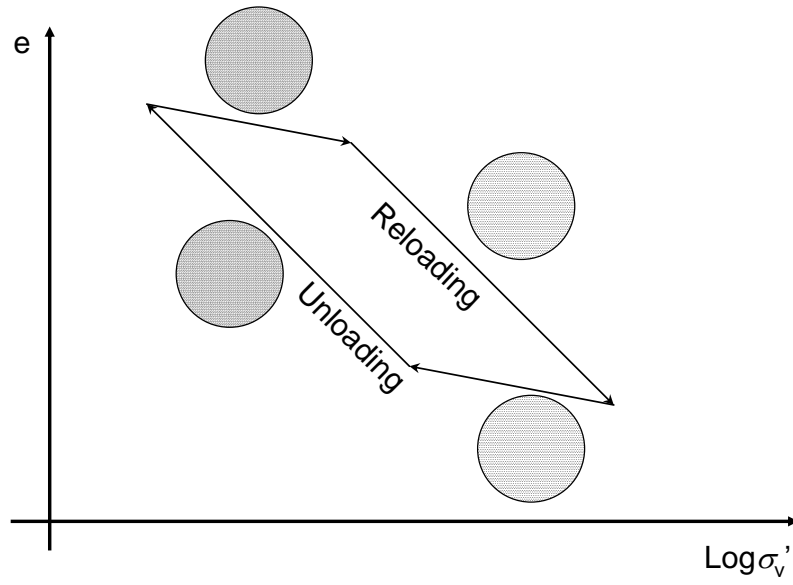


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Figure 5. Methods for swelling pressure determination. (a) loading-wetting method, (b) constant-volume, zero-swell and swell-consolidation methods



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Figure 6. Representation of microstructure changes during unloading/reloading