Revisiting the methods of determining hydraulic conductivity of saturated expansive clays in low-compressibility zone

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Abstract

The hydraulic conductivity of saturated clays is commonly determined either directly by monitoring water flux or indirectly based on Terzaghi's consolidation equation. Similar results are often obtained from the two methods; but sometimes there is significant difference between the two, in particular for expansive soils. In this study, the hydraulic conductivities determined by the two methods are first compared based on existing data in the literature. The indirect method is then revisited attempting to explain the difference identified. A modified effective stress, considering physic-chemical interaction between face-to-face oriented particles, is finally introduced to better describe the compressibility of expansive clays and to further improve the indirect method in determining hydraulic conductivity of such soils in the low-compressibility zone. Extra tests were performed on Gaomiaozi (GMZ) bentonite slurry and the results obtained allowed the modified indirect method to be verified.

Keywords: expansive clays; laboratory tests; hydraulic conductivity; Terzaghi's consolidation equation; modified effective stress

Notation

c_v: consolidation coefficient

m_v: coefficient of compressibility

m_v': modified coefficient of compressibility

γ_w: unit weight of water

e: void ratio

σ': effective stress

σ_v: vertical total stress

σ_v': vertical effective stress

σ_s: mineral to mineral contact stress

R: total interparticle repulsion divided by total interparticle area

A: total interparticle attraction divided by total interparticle area

σ_l: average net repulsive force acting on the bound water-film area divided by the total cross-sectional area

$\sigma^{R\text{-}A}$: net stress acting on water-films between clay particles

P_s: swelling pressure

k: hydraulic conductivity

k_d, k_{ind}: hydraulic conductivity determined directly and indirectly

k_{ind}': hydraulic conductivity calculated using modified effective stress

Introduction

The hydraulic conductivity k is one of the key parameters in geotechnical and geo-environmental engineering. Many efforts have been devoted to the measurement of this parameter in both field and laboratory conditions. In the laboratory, the hydraulic conductivity is commonly determined either directly by monitoring water flux or indirectly based on Terzaghi's consolidation equation. Tavenas et al. (1983) compared the direct and indirect measurement methods for natural clays and observed that the indirect method overestimated the values at relatively large void ratios while underestimated the values at smaller void ratios. The authors attributed the difference to the adopted assumptions of constant k, compressibility and coefficient of consolidation during each consolidation stage, as well as the ways of interpreting the consolidation-time curves on the basis of Terzaghi's consolidation theory. Mesri et al. (1994) reported that the indirect method typically underestimates the value of permeability by a factor of 2. Therefore, in the estimation of the settlement of Kansai International Airport Islands, the values of hydraulic conductivity of the involved Holocene marine clay determined by the indirect method were multiplied by 2 (Mesri and Funk, 2015).

This paper aims at better clarifying the indirect method of determining the hydraulic conductivity of expansive clays. First, reported k data for remoulded, undisturbed and compacted clayey soils, determined directly and indirectly in one-dimensional condition were collected. Then, the parameters used for indirectly determining k were analysed one by one in order to better understand the difference between the direct and the indirect methods. Thereby, the significance of physico-chemical effects between the bound water around clay particles in controlling the response of dense clay to external loading was identified. A modified effective stress concept based on this was postulated and introduced in the Terzaghi's consolidation theory, aiming to narrow the difference between the direct and the indirect methods. Finally, the modified method was verified based on existing data from the literature and complementary

data from tests on GMZ bentonite. This study suggests the necessity of introducing the modified effective stress concept into the Terzaghi's consolidation theory for active expansive clays in the low-compressibility zone.

Direct and indirect methods for *k* determination

Generally, in a conventional step-loading oedometer test, k can be indirectly calculated at each loading step based on Terzaghi's consolidation theory as follows:

$$\mathrm{k} = c_v m_v \gamma_w \quad (1)$$

where, k is the hydraulic conductivity (m/s), c_v is the consolidation coefficient (m^2/s), m_v is the coefficient of compressibility (kPa^{-1}), γ_w is the unit weight of water (taken equal to 10 kN/m^3 in this study).

Parameter c_v reflects the rate at which a saturated clay undergoes consolidation when subjected to an incremental load. In practice, the methods proposed by Casagrande and Fadum (1944) and Taylor (1948) are routinely used to estimate the c_v from the settlement-time curve at each loading step. It appears that this parameter shows mineral-dependence and varies with the stress state of soil (Retnamony et al., 1998; Sridharan and Nagaraj, 2004).

At the end of a loading step i, with corresponding void ratio e, m_v can be determined as follows:

$$m_{v,i} = \frac{\Delta e_i}{\Delta \sigma'_{v,i}(1+e_{i-1})} \quad (2)$$

Obviously, any factors involved in the determination of soil deformation and vertical effective stress increment $\Delta\sigma_v'$ can influence the value of m_v.

Meanwhile, at the end of each consolidation stage, the hydraulic conductivity k can be determined directly based on Darcy's law by connecting the base of the soil specimen to a burette using falling head method or to a pressure/volume controller using constant head method (ASTM D5856; ASTM D7100).

Comparison of the two methods based on the literature data

In this section, the direct and indirect results reported in the literature are collected and compared. The sources of data are summarized in Table 1. The reliability of indirect k values is evaluated by the ratio k_d/k_{ind}, where k_d and k_{ind} are the direct and indirect hydraulic conductivities, respectively. The k_d/k_{ind} results in the high-compressibility zone are plotted in Fig. 1, together with the e-lgσ_v' curves. It is observed that the values of k_d/k_{ind} exhibit a large scatter. However, most of them are close to unity (mostly from 0.5 to 1.1). This suggests that at this stage, the indirect method is reasonably valid. Nevertheless, in the case where very high stress is applied and the soil compressibility changed to very small value, k_d/k_{ind} turns to be much larger than unity (Fig. 1b,c), suggesting that the indirect method becomes less valid. Further examination of Eq. 1 shows that the reliability of indirect value k mainly depends on the accuracy of c_v and m_v. As pointed out by Tavenas et al. (1979), clays in the normally consolidated state exhibit significant variations in terms of c_v and m_v with changes in void ratio. This is confirmed by the results of three bentonite specimens shown in Fig. 2 and Fig. 3: a decreasing trend is identified for c_v (Fig. 2) and m_v (Fig. 3). Tavenas et al. (1983) also reported that the reduction of c_v near the drainage boundary is much faster than in other part of specimen. Obviously, this finding is contradictory to the assumption of constant c_v at each loading step. Thereby, the indirect value k is affected by this assumption, explaining the fluctuation observed in Fig. 1a.

As the consolidation process steps into the low-compressibility zone, small strain is recorded and the variations in c_v and m_v become moderate (Figs. 2 and 3). These observations are more consistent with the Terzaghi's assumptions, theoretically making this theory more applicable in this zone. However, it is observed that k_d/k_{ind} starts to be larger than unity (see Fig. 1b,c). It is commonly admitted that in this zone all macro-pores have collapsed and further loading gives rise to compression of well orientated face-to-face particles. In such orientated microstructure,

79 the physic-chemical interaction between clay particles and absorbed water is enhanced and
80 starts to govern the global volume change behaviour of specimen (Cui et al., 2013). Mesri and
81 Olson (1971) also pointed out that while investigating the hydraulic conductivity of soils, it is
82 important to consider not only the mechanical variables (mainly the size, shape and geometrical
83 arrangement of the clay particles), but the physico-chemical variables (surface charge density
84 and distribution, valence of the adsorbed cations as well as the properties of the involved fluid).
85 From this point of view, the stress induced by the physico-chemical interactions should be
86 accounted for when calculating the effective stress, which could impact the determination of m_v
87 by Eq. 2.
88
89 **Introduction of a modified effective stress in the indirect method**
90 For saturated soils subjected to an external loading, Terzaghi (1936) introduced an effective
91 stress in form of Eq.3 and stated that mechanical responds, such as compression, distortion and
92 changes in shearing resistance, etc., are exclusively due to changes in effective stress:
93 $$\sigma' = \sigma_t - u_w \quad (3)$$
94 where σ' is the effective stress ($ML^{-1}T^{-2}$), σ_t is the total external stress ($ML^{-1}T^{-2}$) and u_w is the
95 pressure ($ML^{-1}T^{-2}$) of the free bulk water.
96
97 However, for expansive clayey soils, in addition to free pore water, there are another two kinds
98 of water: 1) crystalline water which is strongly adsorbed and attached to clay sheets; 2) double
99 layer water which is well adsorbed to clay particles. Under mechanical loading, free pore water
100 was compressed, generating pore water pressure. The dissipation of such pressure leads to the
101 volume change of soil and this process is commonly known as soil consolidation. However, for
102 the adsorbed water, the changes of its pressure is controlled by the physico-chemical
103 interactions between the bound water around clay particles and this pressure is interparticle
104 distance-independent.
105

Depending on the circumstances, these two kinds of water pressure could act separately or together to control the volumetric behaviour of clayey soils. In a consolidating clay with high compressibility, a large number of large pores exist and the drainage of "free" water in the large pores is responsible for the volume change. This is accompanied by the development of particles reorientation and thus the progressive enhancement of physico-chemical interactions between bound water and clay particles (Bolt, 1956; Mašín and Khalili, 2015); as the consolidation keeps proceeding in the low compressibility zone, the large pores disappear as all clay particles are normally well orientated, the "free" water is thoroughly drained out and all water can be considered as adsorbed water. Thus, the common pore water pressure can be taken equal to zero and the physico-chemical effects will control the response of the dense clay to applied loading. However, water can still flow under very high gradients (Pusch et al., 1987). In such case, the physico-chemical stress can be taken equal to the swelling pressure (Zhang, 2017).

Sridharan and Rao (1973) suggested accounting the electrical forces acting in the water-films around clay particles into the common Terzaghi's effective stress equation. Lambe (1960) included the electrical attractive and repulsive forces between water-films around clay particles as follows:

$$\sigma' = \sigma_s + R - A \tag{4}$$

where σ_s is the mineral to mineral contact stress ($ML^{-1}T^{-2}$), R is the total interparticle repulsion divided by total interparticle area ($ML^{-1}T^{-2}$), A is the total interparticle attraction divided by total interparticle area ($ML^{-1}T^{-2}$). R-A is a general term representing all possible attractive and repulsive stresses between clay particles.

By examining the effective stress in a stiff indurated clay rock theoretically and experimentally, Zhang (2017) concluded that in a dense clay-water system, the effective stress is transferred

through the solid-solid contact between non-clay mineral grains and for the most part, the bound pore water in narrow pores, i.e.:

$$\sigma' = \sigma_s + \sigma_l \quad (5)$$

where σ_l represents the average net repulsive force acting on the bound water-film area divided by the total cross-sectional area ($ML^{-1}T^{-2}$). The experimental results of Zhang (2017) suggested that for stiff indurated clay rock, the swelling pressure is almost equal to the effective stress. Mašín and Khalili (2015) referred the net electrical stress acting on water-films between clay particles as $\sigma^{R\text{-}A}$ and argued that this kind of stress mainly controls the mechanical behaviour of soils with prevailing face-to-face particle arrangement. In this regard, the σ_l and $\sigma^{R\text{-}A}$ refer to the same kind of stresses acting in the dense clay-water system.

Therefore, for the low-compressibility zone where the particle arrangement is dominated by the face-to-face feature, the term $\Delta\sigma_v'$ in Eq. 2 should be revisited. At each loading step the $\sigma^{R\text{-}A}$ is assumed to be equal to the swelling pressure P_s at the corresponding density. The vertical effective stress $\sigma_{v,i}'$ at a given step i is determined by subtracting the $P_{s,i}$ value from the applied vertical stress $\sigma_{v,i}$ basd on Eq. 6. Then, a new $m_{v,i}'$ is determined using the modified σ_v' following Eq. 7. Thus, a modified indirect value k_{ind}' can be further calculated by substituting the new $m_{v,i}'$ in Eq. 1. With such modifications, the new k_{ind}' is expected to be closer to the direct value.

$$\sigma'_{v,i} = \sigma_{v,i} - P_{s,i} \quad (6)$$

$$m'_{v,i} = \frac{\Delta e_i}{(\sigma'_{v,i} - \sigma'_{v,i-1})(1+e_{i-1})} \quad (7)$$

It is worth noting that the modification is proposed by incorporating the physico-chemical effects into the effective stress equation for dense expansive clays. In other words, the proposed approach is an extension of Terzaghi's consolidation theory.

Complementary experiment

To evaluate the validity of the proposed modification, an oedometer test was performed on GMZ bentonite slurry. The bentonite, originated from Inner Mongolia, China, is a Na^+ bentonite with a montmorillonite fraction of 75.4%. The total cation exchange capacity is 77.30 meq/100g with 43.36, 29.14, 12.33 and 2.51 meq/100g for Na, Ca, Mg and K, respectively. The bentonite powder, with particles smaller than 0.2 mm and a solid particles density of 2.66 Mg/m^3, has a liquid limit of 276% and a plastic limit of 37%.

The slurry was prepared by mixing de-aired distilled water with the bentonite powder to reach a water content of 1.5 times its liquid limit. Care was taken to avoid trapping air bubbles inside. After 24 h sealing for water homogenization, the slurry was carefully poured into the oedometer cell with 50 mm inner diameter to the marked height of 30 mm. The oedometer has a steel porous disk and a filter paper previously placed at the base. Afterwards, a saturated filter paper, a steel porous disk and the piston were placed at the top of the specimen in sequence. The slurry was allowed to pre-consolidate under a stress of 0.013 MPa, which corresponds to the piston weight. The positions of the piston during the pre-consolidation were monitored using a cathetometer to determine the void ratio change.

When the pre-consolidation was completed, the oedometer cell was placed in a high-pressure load frame, which enables a maximum vertical pressure of 50 MPa to be applied on the specimen. More details about this loading system can be found in Ye et al. (2012). Conventional step loading was applied with ultimate load equal to 41.25 MPa.

The values of the coefficient of consolidation c_v at the last three consolidation steps, where specimen was expected to be in the low-compressibility zone, were estimated using Casagrande's method for indirectly determining the hydraulic conductivity. Constant head

182 permeability tests were also carried out after consolidation completion at these three steps under
183 1 MPa water injection pressure, by using a volume/pressure controller connected to the
184 oedometer cell. In addition, swelling pressure tests using constant-volume method were
185 performed on statically compacted GMZ bentonite specimens with void ratios taken from the
186 compression curve.

187 **Application to existing data**

188 The attempts of introducing a modified effective stress into the indirect method are made on
189 the tested GMZ bentonite slurry, as well as three bentonites in the low-compressibility zone
190 from literatures. Results are summarized in Table 2 and graphically shown in Fig. 4. It is worth
191 noting that the values of swelling pressure P_s of GMZ bentonite were deduced from the swelling
192 pressure-dry density relationship determined by Ye et al. (2007), while those for Kunigel and
193 Fourges bentonites were deduced from the correlations established by Wang et al. (2012) based
194 on the literature data. All these swelling pressure-dry density expressions are given in Table 2.
195 As expected, since the external stress is partially supported by the physico-chemical forces
196 between inter-particles, the modified compressibility coefficient $m_v^{'}$ increases as less
197 incremental stress is required to cause a certain decrease in void ratio in each loading step
198 according to Eqs. 6 and 7. In other words, from the perspective of permeability, as compared
199 to the circumstance before modification, it is easier for the specimen in loading step i to drain
200 water with volume change of Δe_i since less incremental effective stress is needed. Therefore,
201 the modified indirect value $k_{ind}^{'}$ increases and becomes much closer to the direct value k_d, making
202 a significant decrease of $k_d/k_{ind}^{'}$ ratio. This, in turn, supports the idea of $\sigma^{R\text{-}A} = P_s$ in the case of
203 compression of orientated face-to-face particles. Note that parameter c_v is determined based on
204 the settlement-time curves at each loading step using the common Casagrande method. As
205 negligible free water is expected to be involved in the low-compressibility zone, the
206 consolidation process must mainly depend on adsorbed water, thus physically much more
207 complicated. However, as the void ratio changes a little in this zone, the values of c_v should

change a little too. This is evidenced by the similar c_v values at different stresses for a given soil in Table 2. Further examination shows that the improvement of $k_d/k_{ind}^{'}$ results of GMZ bentonite, range from 1.2 to 1.9, is more significant than that of other soils ranging from 1.7 to 3.8 (Table 2). It is normally due the different accuracies of obtaining swelling pressures: tested on compacted specimens for GMZ bentonite slurry, deduced from different expressions by Ye et al. (2007) for compacted GMZ bentonite and Wang et al. (2012) for the rest two slurries. Furthermore, it is important to note that the swelling pressures used for the three slurries are determined/estimated from the experiments on compacted soils. As for GMZ bentonite, better improvement in $k_d/k_{ind}^{'}$ is found in the compacted specimens than the slurry ones (see also Table 2), indicating the idea that in highly compacted bentonite, the net stress $\sigma^{R\text{-}A}$ acting on water-films between clay particles is equal to its swelling pressure, also, the necessity of determining the $\sigma^{R\text{-}A}$ in slurry at different void ratios. Better results of $k_d/k_{ind}^{'}$ are expected when the $\sigma^{R\text{-}A}$ is obtained with the soils tested in slurry state.

Conclusions

The data of hydraulic conductivity k of undisturbed, remoulded and compacted expansive soils, determined in the laboratory by direct and indirect methods are collected and compared, showing that in the primary compression zone which is characterised by a high compressibility the two methods give similar results, while in the low-compressibility zone the indirect method gives much lower hydraulic conductivity.

In the low-compressibility zone, c_v is found to be almost constant and the difference between the direct and indirect methods is attributed to the effect of physic-chemical interaction. The results of the first attempt with consideration of a modified effective stress accounting for such interaction show that much closer hydraulic conductivity can be obtained between the direct and indirect methods, showing the relevance of such an approach.

It is worth noting that while directly measuring the hydraulic conductivity (k_d), there must be a certain range of error in the test results even though the test is carried out by strictly following the testing standards. However, as the data obtained here are quite rich, showing the same variation trend, it is believed that this kind of test error does not affect the general conclusion drawn in this paper. More data about more expansive clays are of course needed to further verify this approach.

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348

349 Table 1. Main information of consolidation-permeability tests from the literature

Type	*Soil*	w_L	*Loading range*	c_v *determination method*	*Permeability test*	*Reference*
Slurry	Kunigel bentonite	474%	0.13-30 MPa	Casagrande's	constant-head	Marcial et al. (2002)
	Fourges clay	112%				
	Carbon-amended bentonite-sand	27%-42%	24-1532 kPa	Casagrande's	falling-head	Malusis et al. (2009)
	Nagoya clay-sand	/	9.8-1256 kPa	Taylor's	falling-head	Watabe et al. (2011)
	Nagoya clay-bentonite	/				
Undisturbed soil	Boom clay	67.2%	0.05-3.2 MPa	Casagrande's	constant-head	Deng et al. (2010)
	Boom clay	67.2%	0.05-3.2 MPa	Casagrande's	constant-head	Ngyuen et al. (2013)
Compacted soil	GMZ bentonite	276%	0.5-42 MPa	Casagrande's	constant-head	Zhu et al. (2013) Ye et al. (2014)

350

Table 2. Information of indirect *k* before and after modification

Soil	*e*	σ_v' *(MPa)*	P_s *(MPa)*	*Modified* σ_v' *(MPa)*	c_v *(m^2/s)*	m_v *(kPa^{-1})*	m_v' *(kPa^{-1})*	k_d *(m/s)*	k_{ind} *(m/s)*	k_{ind}' *(m/s)*	k_d / k_{ind}	k_d / k_{ind}'	*Data resource*
GMZ bentonite slurry	0.64	11.57	3.67	7.90	/	/	/	/	/	/	/	/	this work
	0.52	17.32	7.12	10.20	7.1×10^{-10}	1.2×10^{-5}	3.0×10^{-5}	2.4×10^{-13}	8.4×10^{-14}	2.1×10^{-13}	2.9	1.2	
	0.46	29.29	14.90	14.39	9.7×10^{-10}	3.3×10^{-6}	9.5×10^{-6}	1.2×10^{-13}	3.2×10^{-14}	9.3×10^{-14}	3.6	1.3	
	0.42	41.25	22.75	18.50	6.5×10^{-10}	3.9×10^{-6}	7.6×10^{-6}	9.2×10^{-14}	2.6×10^{-14}	4.9×10^{-14}	3.6	1.9	
Compacted GMZ bentonite	0.74	5.21	1.66	3.55	1.6×10^{-9}	1.8×10^{-5}	2.5×10^{-5}	[a]5.8×10^{-13}	3.0×10^{-13}	4.8×10^{-13}	2.0	1.2	Zhu et al (2013) Ye et al (2014)
	0.54	20.98	7.49	13.49	1.2×10^{-9}	5.9×10^{-6}	9.8×10^{-6}	2.1×10^{-13}	8.3×10^{-14}	1.4×10^{-13}	2.5	1.5	
	0.47	38.42	13.39	25.03	1.6×10^{-9}	2.5×10^{-6}	3.8×10^{-6}	[a]1.1×10^{-13}	4.8×10^{-14}	7.2×10^{-14}	2.3	1.5	
Kunigel bentonite	0.47	8.38	2.03	6.35	6.7×10^{-11}	1.4×10^{-5}	1.7×10^{-5}	3.1×10^{-14}	9.7×10^{-15}	1.3×10^{-14}	3.2	2.3	Marcial et al (2002)
	0.32	31.23	4.09	27.14	1.0×10^{-10}	1.4×10^{-6}	1.5×10^{-6}	6.4×10^{-15}	1.5×10^{-15}	1.9×10^{-15}	4.3	3.4	
Fourges clay	0.82	8.38	0.59	7.79	4.8×10^{-10}	2.0×10^{-5}	2.2×10^{-5}	2.0×10^{-13}	9.8×10^{-14}	1.3×10^{-13}	2.5	1.6	
	0.74	16.59	1.17	15.42	4.9×10^{-10}	5.9×10^{-6}	6.3×10^{-6}	1.4×10^{-13}	2.9×10^{-14}	3.8×10^{-14}	4.2	3.8	
	0.65	31.23	2.51	28.72	5.3×10^{-10}	3.5×10^{-6}	3.8×10^{-6}	4.1×10^{-14}	1.9×10^{-14}	2.5×10^{-14}	2.5	1.7	

* *P_s of compacted GMZ bentonite from Ye et al (2014) with distilled water is calculated based on the prediction equation by Ye et al. (2007). $P_s = 1.94\times10^{-5}exp(7.419\ \rho_d)$* (MPa), where ρ_d is the dry density of bentonite (Mg/m^3).

P_s of Kunigel and Fourges bentonite are calculated based on the prediction equations by Wang et al. (2012). Kunigel: $P_s = 3.67\times10^{-3}exp(3.32\rho_d)$; Fourges: $P_s = 7.83\times10^{-7}exp(9.24\rho_d)$.

[a] *data from personal communication*

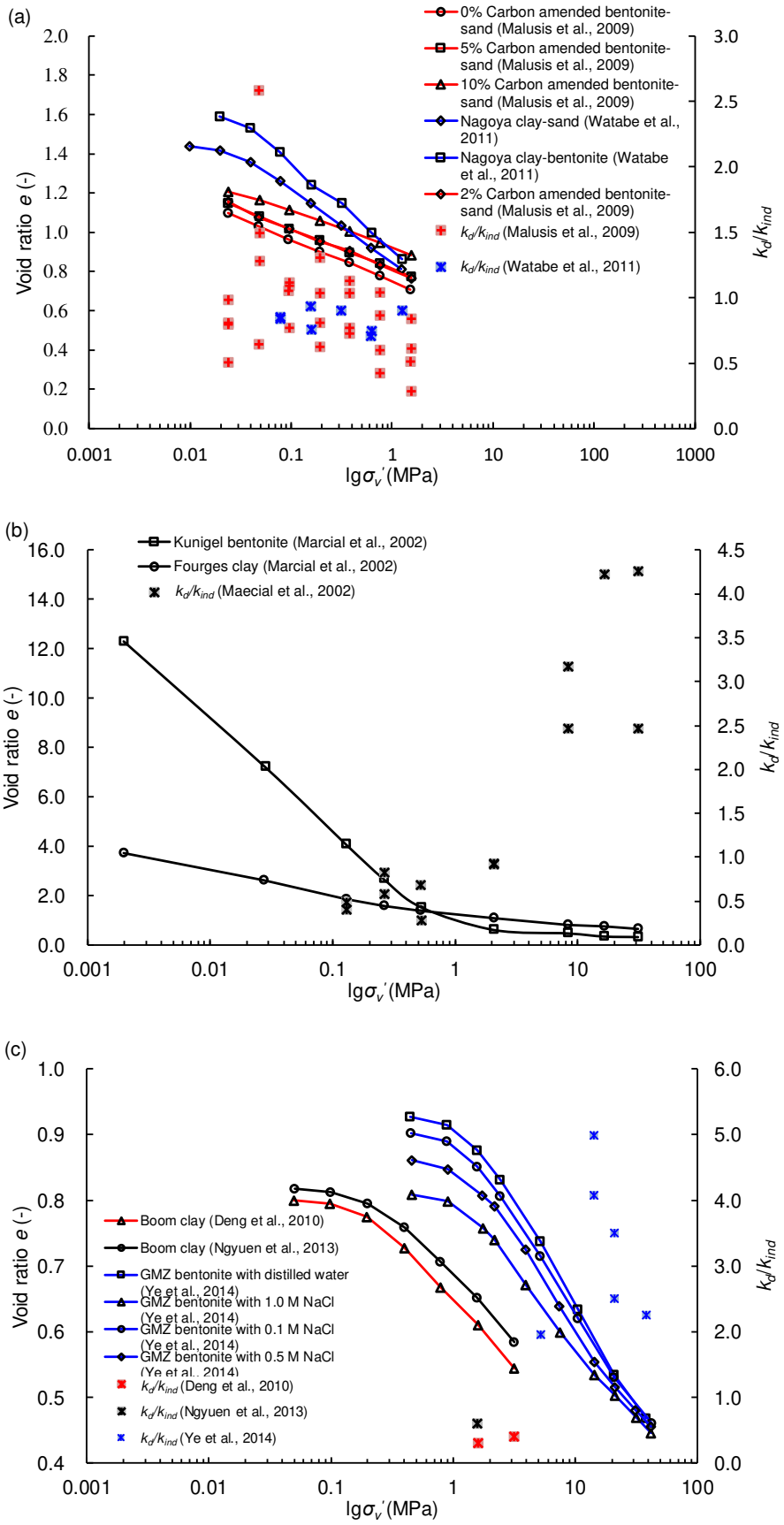


Figure 1. Evolution of k_d/k_{ind} during consolidation, together with the compression curves; (a) slurry in the loading range from 1-2000 kPa, (b) slurry in the loading range from 0.001-100 MPa, (c) compacted and undisturbed clays in the loading range from 0.01-100 MPa

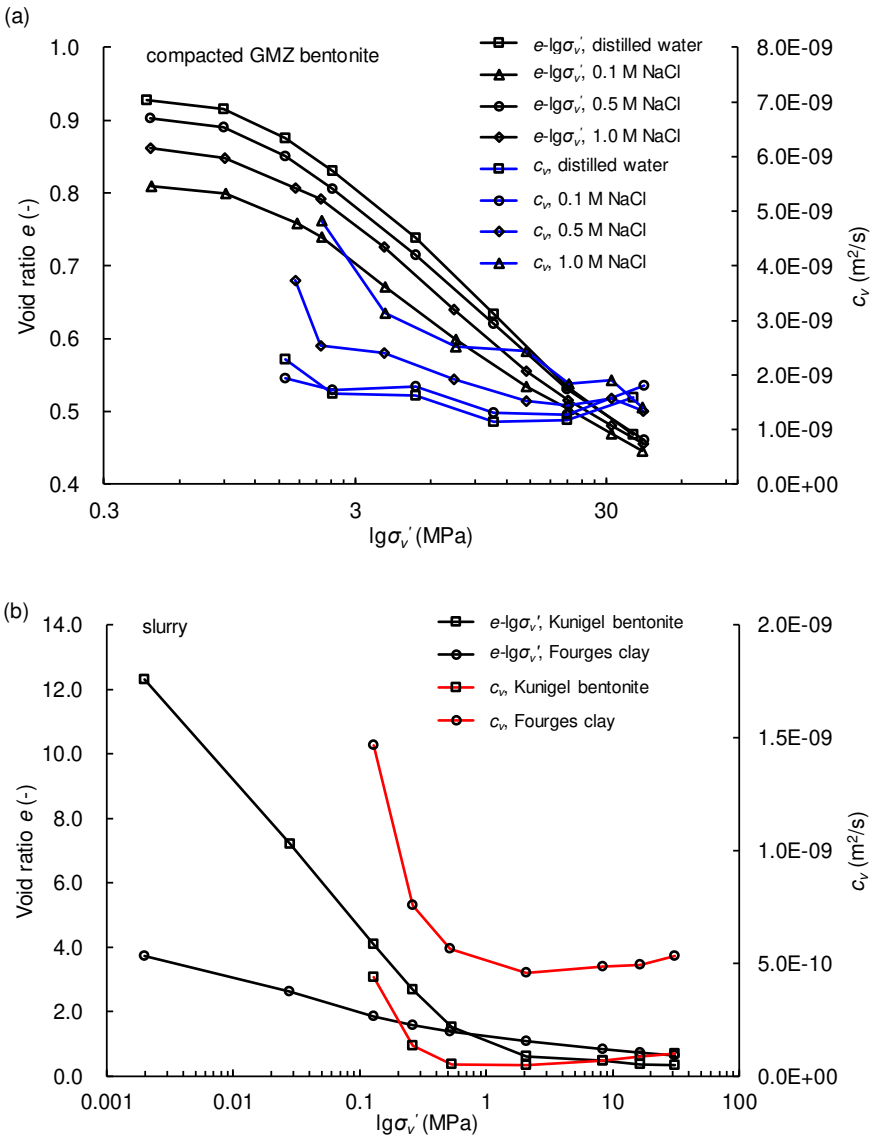


Figure 2. Evolution of c_v during consolidation of three bentonites, together with the compression curves; (a) compacted GMZ bentonite, (b) Kunigel bentonite slurry and Fourges slurry

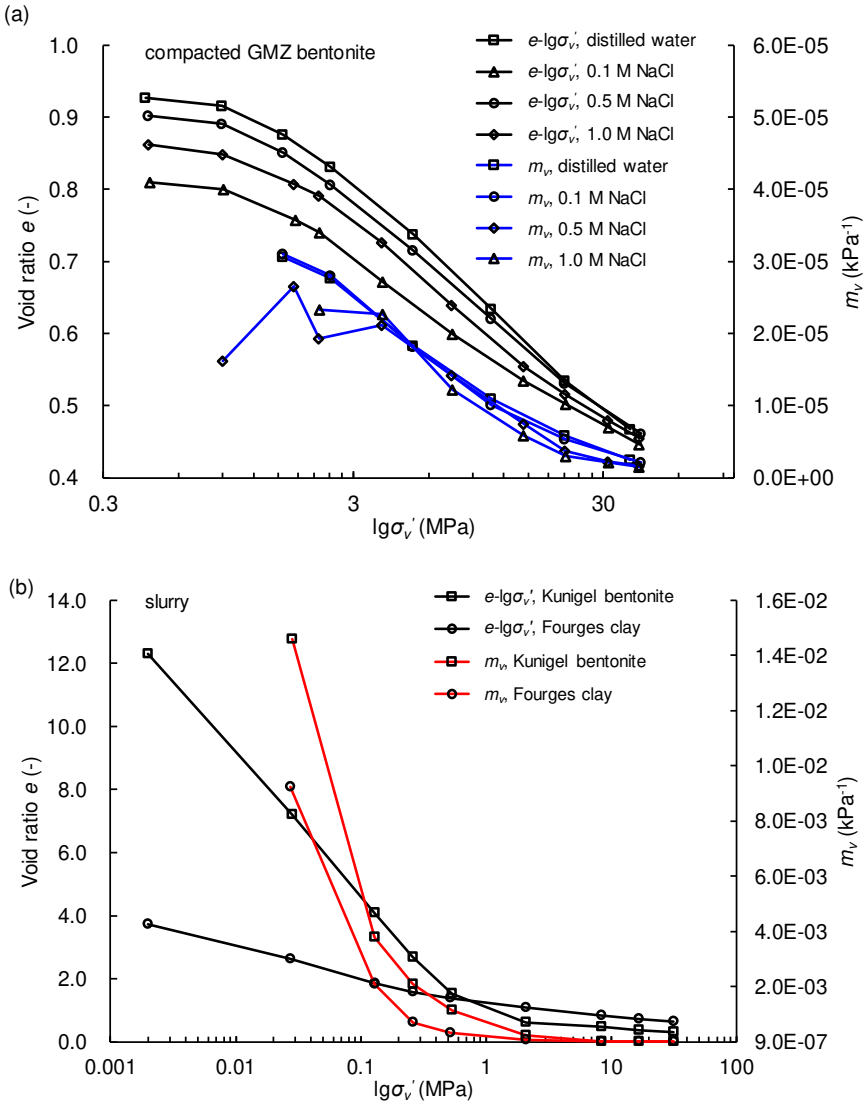
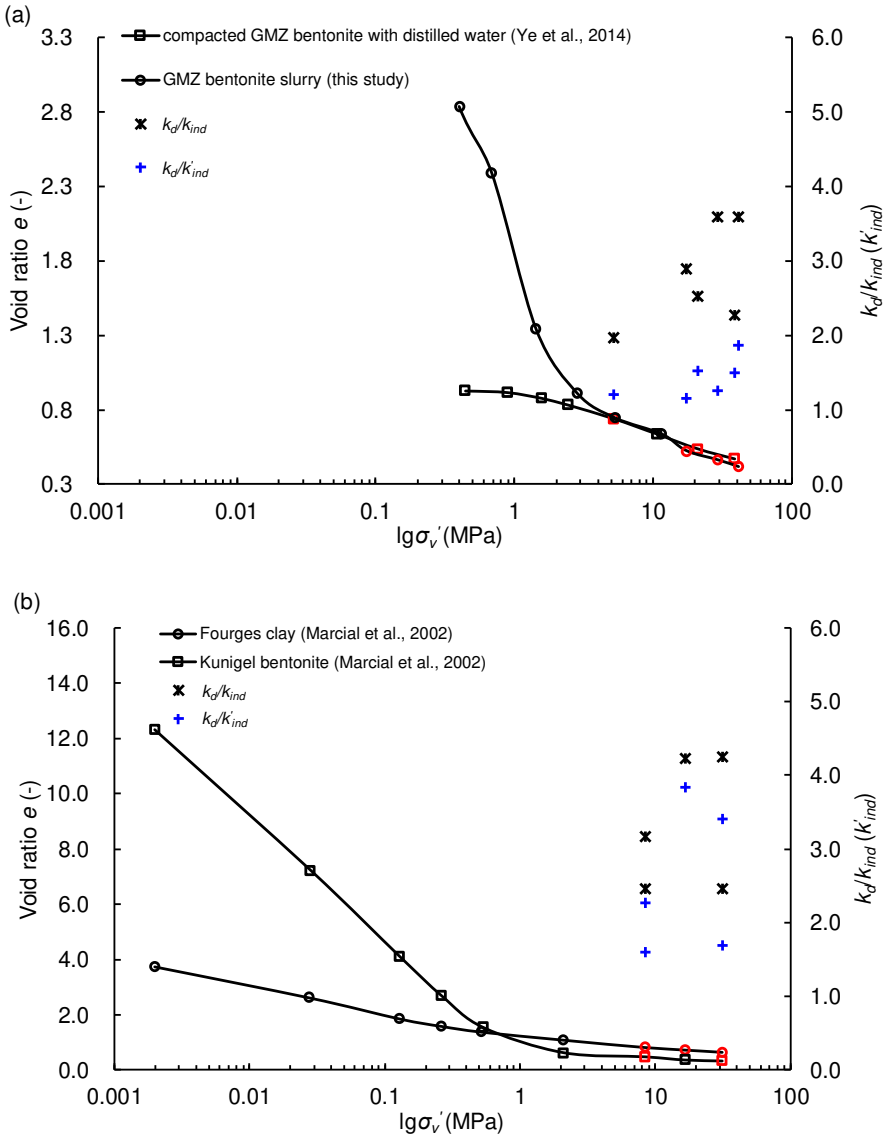


Figure 3. Evolution of m_v together with the compression curves; (a) compacted GMZ bentonite, (b) Kunigel bentonite slurry and Fourges slurry



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376 Figure 4. Comparisons of k_{ind} and modified k'_{ind}, together with the compression curves; (a) GMZ

377 bentonite, (b) Kunigel bentonite slurry and Fourges slurry