

Impact of (Na, Ca)NO₃ concentration on the deviatoric mechanical behaviour of Boom Clay

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Abstract

The disposal of long-lived intermediate- and high-level radioactive waste is a major environmental concern. Deep geological disposal is widely regarded as the safest long-term solution. In Belgium, Boom Clay (BC) has been selected as the reference host formation due to its low permeability, self-sealing properties, and strong capacity to retain radionuclides. One specific waste type, Eurobitum—an intermediate-level bituminized waste—is stored in steel canisters within the repository. Over time, groundwater is expected to infiltrate the system and come into contact with these canisters. The bituminized waste swells upon water contact, releasing large amounts of (Na, Ca)NO₃ and generating a saline plume that diffuses into the BC. This plume, rich in sodium ions (Na⁺), can significantly impact the clay's physicochemical and hydro-mechanical behaviour. In this study, undrained triaxial tests under isotropic consolidation were conducted on intact BC samples pre-equilibrated with (Na, Ca)NO₃ solutions of varying concentrations. Solutions corresponded to sodium occupancies of 60% (1.0 mol/L) and 90% (2.0 mol/L), along with a reference Boom Clay synthetic water solution (0.015 mol/L NaHCO₃). Results showed an increase in shear strength and friction angle with sodium concentration, attributed to clay particle aggregation, shrinkage, and diffuse double layer contraction induced by salinity.

Key words: Boom Clay, triaxial test, ionic strength, sodium nitrate, diffuse double layer

Résumé

L'élimination des déchets radioactifs de niveau intermédiaire et élevé à longue durée de vie est une préoccupation environnementale majeure. Le stockage géologique profond est largement considéré comme la solution la plus sûre à long terme. En Belgique, l'argile de Boom (BC) a été choisie comme formation hôte de référence en raison de sa faible perméabilité, de ses propriétés d'auto-obturation et de sa forte capacité à retenir les radionucléides. Un type de déchet spécifique, l'Eurobitume, un déchet bituminisé de niveau intermédiaire, est entreposé dans des conteneurs en acier à l'intérieur du dépôt. Au fil du temps, l'eau souterraine est censée s'infiltrer dans le système et entrer en contact avec ces canisters. Les déchets bituminés gonflent au contact de l'eau, libérant de grandes quantités de (Na, Ca)NO₃ et générant une nappe saline qui diffuse dans l'argile Boom. Cette nappe, riche en ions sodium (Na⁺), peut avoir un impact significatif sur le comportement physico-chimique et hydromécanique de l'argile. Dans cette étude, des essais triaxiaux non drainés sous consolidation isotrope ont été effectués sur des échantillons intacts d'argile de Boom (BC) prééquilibrés avec des solutions de (Na, Ca)NO₃ à des concentrations variées. Les solutions correspondaient à des taux d'occupation en sodium de 60% (1,0 mol/L) et 90% (2,0 mol/L), ainsi qu'à une solution de référence d'eau synthétique de l'argile de Boom (0,015 mol/L de NaHCO₃). Les résultats ont montré une augmentation de la résistance au cisaillement et de l'angle de frottement avec la concentration en sodium, attribuée à l'aggrégation des particules d'argile, au rétrécissement et à la contraction diffuse de la double couche induite par la salinité.

Mots-clés : argile de Boom, essai triaxial, force ionique, nitrate de sodium, double couche diffuse

1. Introduction

Clay formations exhibit low permeability, high swelling capacity, and strong ability to adsorb radionuclides, which makes them suitable candidates for the geological disposal of radioactive waste (Delage et al. 2010). Under deep disposal conditions, interactions between clay minerals and the chem-

ical species released from certain radioactive waste forms through the cementitious engineered barriers can induce volume changes and modify shear strength, leading to various chemo-hydro-mechanical perturbations (Hellä et al. 2014). This study focuses on Eurobitum, an intermediate-level long-lived bituminized radioactive waste that contains mainly

precipitation sludges from spent nuclear fuel reprocessing. Eurobitum includes large quantities of soluble salts such as NaNO_3 (20–30 wt.%), CaSO_4 (4–6 wt.%), along with other compounds like CaF_2 and $\text{Ca}_3(\text{PO}_4)_2$ (Valcke et al. 2009). In Belgium, Boom Clay, a poorly indurated and plastic clay with low permeability and high radionuclide retention capacity, is considered a potential host formation for deep disposal. Over time, a saline plume composed of NaNO_3 , NaNO_2 , and CaSO_4 , along with ions such as Na^+ , K^+ , Ca^{2+} , and OH^- present in hyperalkaline pore water, is expected to diffuse from the waste through the cementitious barriers into the surrounding Boom Clay. This migration may result in locally elevated salt concentrations, potentially affecting hydro-mechanical behaviour within a zone extending up to approximately 3 m around the engineered barrier system (Weetjens et al. 2010; Bleyen et al. 2018). These chemical interactions may influence the long-term performance of Boom Clay by altering its favourable retention and mechanical characteristics. Understanding the coupled chemo-hydro-mechanical processes and quantifying the impact of salinity on mechanical response, particularly in terms of shear strength, is therefore essential.

Previous experimental studies have shown that the shear strength of clayey materials is sensitive to pore fluid composition, including ionic strength, pH, and ion type, due to changes in electrochemical interactions and structural rearrangements within the clay matrix (Di Maio and Fenelli 1997; Musso et al. 2003; Ayininuola et al. 2009; Ye et al. 2014; Manassero et al. 2016; Ying et al. 2021). These effects are linked to modifications in the thickness of the diffuse double layer (DDL) and the nature of interparticle forces. While the influence of pore fluid chemistry has been widely investigated for reconstituted clays (Di Maio et al. 2004; Deng et al. 2014, 2018), fewer studies have examined compacted or natural intact clays under chemically aggressive conditions (Castellanos et al. 2008; Siddiqua et al. 2011; Nguyen 2013; Al Mais et al. 2024). Several investigations have reported contrasting trends depending on mineral composition and salt type. For example, increased salt concentration was found to enhance undrained shear strength in kaolinitic soils (Sridharan et al. 2002), whereas montmorillonite-rich soils showed a reduction in strength. Ayininuola et al. (2009) observed that calcium-based salts enhanced interparticle bonding, increasing internal friction angle and cohesion. Moore (1991) and Di Maio and Fenelli (1997) investigated the residual shear strength of kaolinite and bentonite in various saline environments and reported that bentonite displayed increased residual friction angle with salt concentration, whereas kaolinite was less affected. Fang et al. (2019) found that higher sodium chloride concentrations increased friction angle but reduced cohesion in bentonite and bentonite-kaolin mixtures, due to thinner water films and enhanced mineral-to-mineral contact. Despite these findings, most studies rely on reconstituted specimens prepared by slurry consolidation or compaction (Deng et al. 2014, 2018), while only limited data are available on the mechanical behaviour of natural intact clays such as Boom Clay exposed to saline environments relevant to geological disposal.

The aim of the study is to investigate the effects of high-salinity (Na, Ca) NO_3 solutions and high sodium occupancy

on the BC mechanical behaviour, particularly the shear behaviour. A series of isotropically undrained triaxial tests were conducted on intact BC samples percolated with three different solutions: a synthetic BC solution (15 mmol/L NaHCO_3 , $\text{IS} = 0.015$ mol/L; De Craen et al. 2004), as well as two saline solutions with sodium occupancies of 60% at 1.0 mol/L and 90% at 2.0 mol/L of (Na, Ca) NO_3 , and under three different final effective confining pressures: 1.0, 2.25, and 4.5 MPa. The observed trends in stress-strain behaviour, stress paths (p' - q), pore pressure, and overall mechanical behaviour were analysed and are discussed in detail. Note that the cation occupancy refers to the proportion or fraction of a specific cation relative to the total amount of exchangeable cations. This metric indicates the extent to which the clay is saturated with this specific cation compared to the total number of adsorbed cations. Note that these high concentrations or sodium occupancies are not fully representative of the repository conditions but are inspired by the objective to have conditions that would enable to detect substantial differences for the studied responses. It is worth noting that only the saline effect is studied here, without consideration of alkalinity or pH. The concentration of the (Na, Ca)nitrate plume (in fact a Na, K, Mg, Ca—hydroxide, nitrate, sulfate plume) expected for the disposal of Eurobitum in a geological disposal facility is considerably lower than 1 mol/L, whereas high Na occupancies are predicted to occur only in a thin layer of the adjacent clay (Weetjens et al. 2010; Al Mais et al. 2024, 2025; Jacques et al. 2024).

2. Materials and methods

2.1. Materials

The material tested is the BC, obtained during a coring campaign in the HADES underground research laboratory at a depth of 223 m at Mol, Belgium. At this depth, the total vertical stress (σ_v) and the pore water pressure (u) are 4.5 and 2.2 MPa, respectively (Bernier et al. 2011). The clay fraction (mainly illite and smectite) represents 60% of the total weight; quartz is the predominant mineral (about 20%) in the non-argillaceous fraction, which also includes feldspars (10%), carbonates, and pyrite (Frederickx 2020). The cores were extracted from a horizontal borehole, oriented normal to the bedding plane, and were vacuum-wrapped in thermo-welded reinforced aluminium foil to limit water desaturation. The cores had diameters ranging from 90 to 100 mm and lengths between 600 and 1000 mm. Samples were prepared from three distinct cores: R78-79 W_Core 28.1, R78-79 W_Core 2.2, and CG76-77 W_Core 13.2.

Three different saline solutions were considered, corresponding to the diffused concentrations coming from bituminized waste and the cementitious barrier system into the near field based on repository scale simulations (Weetjens et al. 2010). In particular, the choice of higher sodium (Na^+) occupancies of 60% and 90%, along with the total concentrations of 1.0 and 2.0 mol/L, were considered based on the results of Weetjens et al. (2010) who utilized combined geochemical and transport calculations to model the evolution of Na^+ concentration and occupancy for the disposal of

Table 1. Composition of saline solutions.

Solution	Na ⁺ —occupancy (%)	NaNO ₃ (mmol/L)	Ca (NO ₃) ₂ (mmol/L)	Ionic strength (mol/L)
1	60	900	70	1.0
2	90	2000	–	2.0

Eurobitum in Boom Clay. These values should be viewed as just one of many possible conservative estimates of the near-field conditions.

Table 1 presents the concentration and composition of each solution, which consists of sodium nitrate (NaNO₃) and calcium nitrate tetrahydrate [Ca(NO₃)₂·4 H₂O]. Additionally, a natural BC system with a Na⁺ occupancy of approximately 35% (Baeyens and Henrion 1985) was included, representing the initial sodium occupancy in the clay materials of Boom Clay (Honty 2010) in contact with 15 mmol/L (IS = 0.015 mol/L) NaHCO₃. The most recent study by Honty et al. (2022) reveals, through the experimental investigation of Boom Clay pore water chemistry, that contemporary BC pore water is a dilute NaHCO₃ solution, with concentrations ranging from 10 to 20 mmol/L and an alkaline pH slightly above 8.

It is important to clarify that in the context of the triaxial tests, only the sodium concentration is examined as a variable, with no pore volume percolation occurring in the BC samples (see Section 2.3.1).

2.2. Sample preparation

Prior to sample preparation, it is essential to identify the bedding plane due to its significant influence on the anisotropic behaviour. The bedding plane was easily visible to the naked eye upon opening the core. For the preparation of triaxial test samples, a cuboid section was first extracted from the core (Figs. 1a and 1b). The sample was then manually trimmed using a sharp blade cutter and a soil lathe (Fig. 1c) to achieve a cylindrical shape with a diameter of 38 mm and a height of 76 mm (Fig. 1d). To preserve moisture, the samples were wrapped in Parafilm, a semi-transparent, flexible film composed of wax blends, followed by sealing in high-density vacuum storage bags, and finally vacuum-packing them using a fast vacuum sealer. The basic properties of the intact BC were determined in the laboratory and are shown in Table 2. One can observe that the water content is in the range 21% < w₀ < 23.5%, and the saturation is in the range 99% < S_r < 100%. Note that the specific solid density (G_s) used for Boom Clay (BC) in this study is 2.65 Mg/m³ (De Craen et al. 2004). The void ratio was calculated based on the density of solids as well as the total volume and mass of each sample. Since the samples were hand-trimmed and prepared from different cores, there is some difference in the initial void ratio, varying between 0.539 and 0.618. To minimize the impact of sample variability, each sample was first isotropically loaded to 6 MPa under drained conditions and then unloaded to the final effective confining pressure before starting the undrained shearing phase (see details below).

2.3. Experimental set-up and test program

A self-compensating triaxial cell made by Geodesign called “HP3” (see Fig. 2a) was used (Nguyen 2013). The axial deviator stress is applied by a self-compensating hydraulic system without the effect of confining pressure on the piston. The measurement of the axial force is ensured by an internal force sensor with a capacity of 25 kN, corresponding to a maximum axial stress of 22 MPa. The self-compensating cell consists of three parts: cap, side wall, and base (Fig. 2b), and is supplied by a pore pressure controller of pressure and volume (CPV), a confining pressure CPV, a deviator stress CPV, and a solution converter. The solution converter is a small cell divided into two sections by an impermeable membrane. The upper section contains distilled water, where the applied water pressure from the CPV is transmitted. The lower section holds saline water, which is injected into the triaxial cell through the drainage system to saturate the sample. The primary function of the converter cell is to prevent corrosion of the CPV. The triaxial cell includes two axial LVDTs and three radial LVDTs to measure the displacement of the sample (Fig. 2c). An LVDT is a precise sensor used to measure linear displacement or movement, with an accuracy of 0.001 μm.

In this study, the loading direction during triaxial testing was oriented perpendicular to the bedding plane. The specimen is first covered with a geotextile membrane to accelerate the saturation process (Fig. 2d). The specimen is then placed between two dry metallic porous disks and encased within a neoprene membrane in the triaxial cell base, with LVDT rings positioned around it (Fig. 2c). To ensure that the neoprene membrane is not punctured and is watertight, a vacuum is applied to the specimen, and the pressure is monitored over time. If the pressure remains stable, the membrane can withstand the pressure, and the absence of leakage is confirmed.

The experimental protocol starts with the placement of the sample in the following order: installation of the side wall, filling the cell with silicone oil, and replacing the self-compensating cover or cap. The procedure followed during the undrained triaxial test is as follows:

- The specimen is isotropically consolidated in loading steps under drained conditions until the effective confining stress reaches the in situ effective stress ($\sigma'_3 = 2.25$ MPa; loading steps: 0.2 MPa → 0.4 MPa → 0.6 MPa → 0.8 MPa → 1 MPa → 1.2 MPa → 1.4 MPa → 1.6 MPa → 1.8 MPa → 2 MPa → 2.25 MPa). It is important to note that the initial vacuum was removed once the confining pressure is equal to 500 kPa ($\sigma'_3 = 500$ kPa).
- Once the volumetric deformation of the specimen stabilized at an effective confining stress of $\sigma'_3 = 2.25$ MPa, the saturation process was initiated using the desired solutions.

Fig. 1. Sample preparation for triaxial tests: (a) schematic representation of the triaxial sample; (b) rectangular block before shaping; (c and d) final cylindrical specimens prepared for triaxial testing.

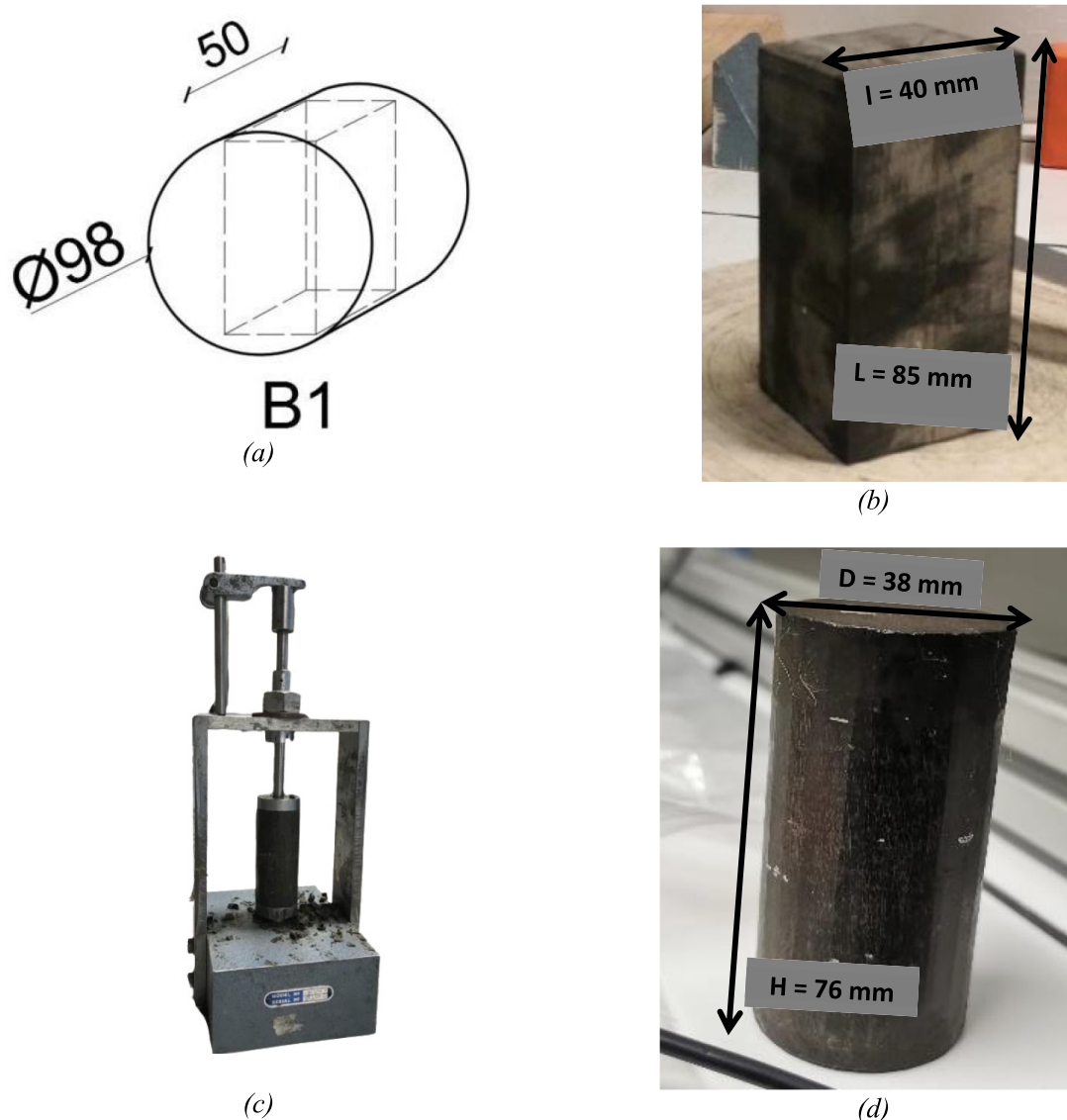


Table 2. Geometry, initial water content, and initial void ratio for the specimens tested.

Test	Sample localization	D_0 (mm)	H_0 (mm)	ρ (g/cm ³)	W_0 (%)	e_0	S_r
1	R78-79 W_Core 28.1 Block4	38.3	77	2.08	21	0.539	1
2	R78-79 W_Core 28.1 Block11	37	77	2.07	21.6	0.556	1
3	R78-79 W_Core 28.1 Block12	38.2	77	2.05	21.3	0.569	0.99
4	R78-79 W_Core 2.2 Block4	37.2	75	2.02	23.2	0.618	0.99
5	R78-79 W_Core 2.2 Block7	35.5	74.6	2.13	23.5	0.536	1
6	R78-79 W_Core 28.1 Block14	37.6	76.1	2.11	23.3	0.548	1
7	R78-79 W_Core 13.2 Block13	37.9	76.1	2.03	23.2	0.608	1
8	R78-79 W_Core 13.2 Block4	38.2	75.8	2.02	23.1	0.609	1
9	R78-79 W_Core 13.2 Block3	37	76.5	2.05	23.3	0.589	1

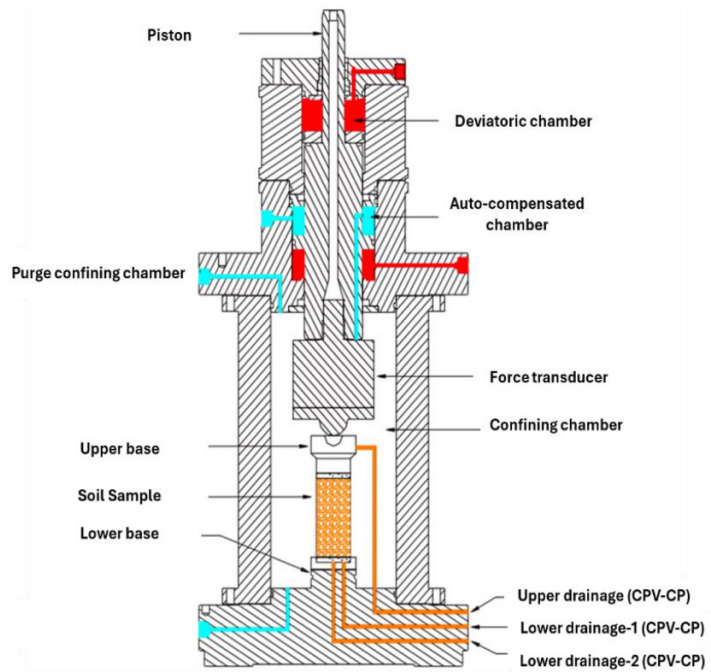
Saturation was achieved by applying a stepwise back pressure (BP) up to 1 MPa while maintaining a constant effective confining stress of 2.25 MPa ($\sigma'_3 = \sigma_3 - u = 2.25$ MPa) to prevent swelling of the specimen and to preserve any

microstructural modification. The desired solution was injected via CPV into the drainage system through one of the two outlets on the lower base for the upper and lower injections. The remaining outlet was left open to purge

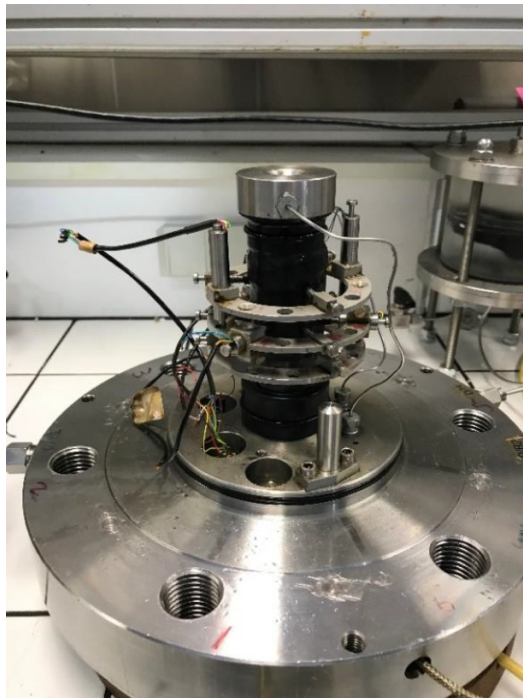
Fig. 2. Triaxial cell HP3 (a and b), LVDTs (c), and sample covering (d).



(a)



(b)



(c)



(d)

air from the tubes and porous stones. Once the air was purged, the remaining outlet was closed, and the saturation began.

- The saturation time was standardized across all tests at 35 days. Following the saturation period, the Skempton coef-

ficient $\left(B = \frac{\Delta u}{\Delta \sigma_3}\right)$ was verified by (i) isolating the specimen from the CPV system to create undrained conditions with an initial pore pressure of $u = 1$ MPa; and (ii) slightly increasing the confining pressure by an increment of $\Delta \sigma_3 =$

Table 3. Initial conditions of undrained triaxial tests.

Test No.	BP (MPa)	σ'_3 (MPa)	IS (mol/L)
1	1	4.5	2
2	1	2.25	2
3	1	1	2
4	1	4.5	1
5	1	2.25	1
6	1	1	1
7	1	4.5	0.015
8	1	2.25	0.015
9	1	1	0.015

200 kPa. To achieve a degree of saturation $S_r = 99\%$ and with a pore pressure of 1 MPa, the Skempton coefficient B must be equal to or greater than 0.85 (Afnor 1994). Nearly all specimens tested in this study have reached or exceeded this value, except tests 2 and 3, which yielded slightly lower values near 0.8. This effect is reflected in the evolution of pore water pressure during shearing (see further discussion below). Once the measurement of the Skempton coefficient is completed, the confinement pressure was reduced to $\sigma'_3 = 2.25$ MPa ($\sigma_3 - 1$ MPa), and the drainage system was opened.

- After the saturation, all specimens were isotropically consolidated to 6 MPa, representing the preconsolidation pressure of Boom Clay under in situ conditions (Horseman et al. 1987; Awarkeh 2023). This step was applied to minimize the effects of initial anisotropy, as well as potential heterogeneities or small fractures that might have resulted from hand trimming during sample preparation, ensuring that all specimens shared the same initial conditions. Subsequently, the specimens were unloaded to three final effective confining pressures: 1, 2.25, and 4.5 MPa, corresponding to three overconsolidation ratios (OCR) of 6, 2.6, and 1.3, respectively. The selection of these stress levels was motivated by the need to cover the two main volumetric strain domains—an effective confining pressure of 1 MPa (OCR = 6) was used to examine the dilatancy behaviour of Boom Clay, while 4.5 MPa (OCR = 1.3) enabled the study of its contraction behaviour. The intermediate confining pressure of 2.25 MPa represents the in situ effective stress, with an OCR of 2.6, capturing the transition between contraction and dilation. Finally, the water valves were closed, and the specimens were sheared at a constant rate of 5 $\mu\text{m}/\text{min}$ (Afnor 1994) with the stress and pore pressure continuously measured. The test was stopped when the axial strain reached between 5% and 10% ($\varepsilon_a = 5\text{--}10\%$).

The triaxial tests were conducted following the French standards (Afnor 1994). In total, nine undrained triaxial tests were conducted, with all details summarized in Table 3. These tests investigated the effects of effective confining pressure, solute concentration of (Na, Ca)NO₃. Table 3 provides the values for the applied BP, effective confining pressure (σ'_3), and the ionic strength (IS) of the percolated solution. The ionic

strength of 0.015 mol/L (0.015 mol/L of NaHCO₃ (De Craen et al. 2004)) corresponds to the synthetic BC solution, which was used as a reference in the analyses.

All the tests were performed at a controlled ambient temperature of 20 ± 1 °C.

2.3.1. Numerical simulation of the saturation phase

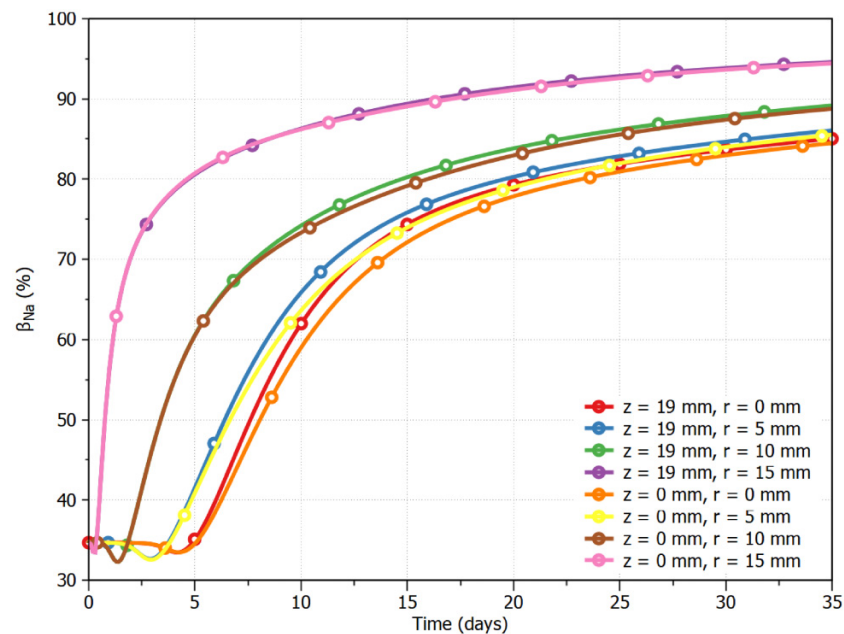
As stated above, the saturation process was maintained for 35 days for all specimens to achieve the target concentration and sodium occupancy required for the triaxial tests. However, since no measurements of either concentration or sodium occupancy were conducted at the end of the saturation phase to confirm whether the target values had been uniformly achieved across all specimens, reactive transport simulations were used. These simulations were based on a previously validated reactive transport model for Boom Clay. As an example, Al Mais (2025) determined the number of pore volumes needed to reach similar target concentrations and sodium occupancies in oedometer samples using a geochemical model for Boom Clay. This geochemical model had been validated through laboratory measurements on Boom Clay specimens. In addition, the transport component of the reactive transport model relies only on the effective diffusion coefficient and tortuosity, both of which are based on established values for Boom Clay. Thus, the validated reactive transport model was applied to predict the evolution of concentration profiles and sodium occupancy within the Boom Clay specimens. These simulations were conducted using the HPx framework (Jacques et al. 2018) implemented within HYDRUS 5 (Šimůnek et al. 2024). The HPx framework integrates PHREEQC (Parkhurst and Appelo 2013) as the geochemical solver with the HYDRUS flow and transport model, enabling detailed assessment of coupled hydrogeochemical processes. In this study, an axisymmetric two-dimensional (2D) geometry is used. Correspondingly, due to the symmetry of the problem, only half of the sample is modelled (height: 38 mm; radius: 19 mm). The model applies a fixed concentration boundary condition at the top and along the outer edges of the cylindrical specimen, corresponding to the boundary conditions of the experimental procedure during the saturation phase (see Section 2.3).

The key properties of Boom Clay were assumed as follows: porosity of 0.35, saturated hydraulic conductivity of 4×10^{-12} m/s (Al Mais et al. 2024), and an effective diffusion coefficient of 10^{-10} m²/s. The boundary solution comprised a 2 mol/L NaNO₃ solution, while reactions involving Na⁺ and the Boom Clay matrix were described using the cation exchange model from Wang et al. (2023). Table 4 summarizes the key parameters of the geochemical exchange model, which serves as the basis for simulating the cation exchange processes in Boom Clay under the defined experimental conditions.

The initial water composition of Boom Clay was defined in equilibrium with conditions outlined by Wang et al. (2023), characterized by a pH of 8.2 and ionic concentrations of Na, K, Ca, and Mg set at 1.34×10^{-2} , 2.07×10^{-4} , 7.02×10^{-5} ,

Table 4. Parameters of the exchange model (Wang et al. 2023) for the Boom Clay.

Capacity	0.88 eq/L
Cation exchange—Gaines-Tomas—written in PHREEQC format (Bc^- represents the cation exchanger)	
	LogK
$Bc^- + Na^+ = NaBc$	0
$Bc^- + K^+ = KBc$	1.4815
$2Bc^- + Ca^{2+} = CaBc$	0.9618
$2Bc^- + Mg^{2+} = MgBc$	0.9344

Fig. 3. Numerical evolution of sodium occupancy (β_{Na}) in Boom Clay samples during the saturation phase with $Na = 90\%$ ($IS = 2.0$ mol/L) at different locations (z : length, r : radius).

and 4.94×10^{-5} mol/L, respectively. The initial cation exchange complex occupancy of Boom Clay was approximately 34.6%, 16.4%, 28.9%, and 20.1% for Na, K, Ca, and Mg, respectively (Frederickx et al. 2018). These parameters collectively ensured the accurate representation of chemical and hydraulic processes during the simulation and experimental phase.

Figure 3 illustrates the temporal evolution of sodium occupancy (β_{Na} , %) at fixed elevations ($z = 0$ mm and $z = 19$ mm) with varying radial distances ($r = 0, 5, 10$, and 15 mm). It can be observed that the sodium occupancy at the center ($z = 0$) and radial distances of 10 and 15 mm approaches 95%. In contrast, at $z = 0$ and $r = 0$, it was found to stabilize at around 85%. These results indicate that the chosen saturation protocol, which involves saturating the specimen from both the top and bottom boundaries, is effective in achieving near-uniform sodium occupancy within 35 days.

At the center of the specimen, the potassium (K^+) occupancy was found to be approximately 14%, while the calcium (Ca^{2+}) and magnesium (Mg^{2+}) occupancies were found to be less than 1%, reflecting the dominance of sodium (Na^+) exchange under the experimental conditions.

Figure 4 presents the spatial distribution of sodium occupancy across the BC samples, highlighting the gradient of β_{Na} from high-concentration boundaries to the centre. Near the boundaries with elevated concentrations, the sodium occupancy is close to 100%, confirming the effective cation exchange and the full saturation.

3. Experimental results

3.1. Effect of confining pressure

In the following section, the test results are discussed with reference to the mean effective stress (p') and the deviator stress (q), defined as

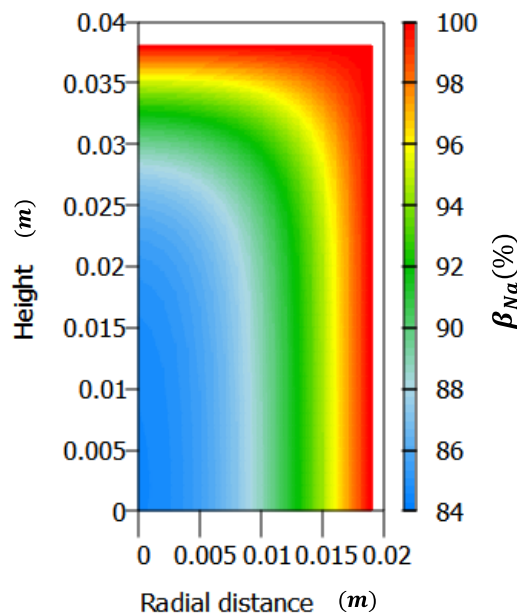
$$(1) \quad p' = \frac{\sigma'_1 + 2\sigma'_3}{3}$$

$$(2) \quad q = \sigma'_1 - \sigma'_3$$

where σ'_1 and σ'_3 represent the vertical and lateral effective stresses, respectively.

Failure (peak) is defined as the point at which the maximum value of q is reached during compression, with the

Fig. 4. Spatial distribution of sodium occupancy (β_{Na}) in Boom Clay after 35 days.



corresponding values of mean effective stress and deviator stress indexed as p'_f and q_f . In contrast, the residual deviator stress corresponds to the final value of q after reaching a plateau, specifically in tests conducted at lower effective confining pressures (1 and 2.25 MPa) that exhibit dilative behaviour. The corresponding values of q and p' at this residual state are denoted as q_r and p'_r , respectively.

Figure 5 illustrates the results of the nine undrained triaxial tests, showing the variations of deviator stress (q) and pore water pressure (Δu) as function of axial strain (ε_a) under different initial effective confining pressures. Several common features can be observed:

- The stress–strain curves indicate that BC exhibits a typical elasto-plastic behaviour. Initially, the curves display a limited linear elastic region, which continues until yielding occurs, marked by a noticeable deviation from linearity. The yield stress represents the transition between the elastic and plastic regions. After reaching the peak deviator stress (q_f), softening or hardening behaviour may occur, depending on the initial conditions of each test, particularly in terms of effective confining pressure. The limited elastic zone falls in $\varepsilon_a < 1\%$.
- Regardless of the (Na, Ca)NO₃ concentration, it can be seen that the soil stiffness increases with increasing effective confining stress σ'_3 . As illustrated in Fig. 5a, all samples were percolated with (Na, Ca)NO₃ at an ionic strength of 2.0 mol/L. Test 1, conducted at $\sigma'_3 = 4.5$ MPa, is characterized by a peak deviator stress of 2.8 MPa, which is higher than in test 2 (2.25 MPa) and test 3 (1.4 MPa), conducted at $\sigma'_3 = 2.25$ MPa and $\sigma'_3 = 1$ MPa, respectively. This trend is consistent with the results from other tests conducted with varying (Na, Ca)NO₃ concentrations at the same effective

confining stress, where higher effective confining stresses correlate with higher peak deviator stresses.

- The pore water pressure curves show that higher pore water pressures correspond to increased effective confining stresses. As the effective confining stress increases, the void ratio decreases, leading to a more rapid increase in pore pressure and overall higher values. Test 1 gives an evolution of pore pressure of 1.9 MPa at $\sigma'_3 = 4.5$ MPa, with $\Delta u = 0.51$ MPa for samples confined at 2.25 MPa, and $\Delta u = 0.25$ MPa for samples confined at 1 MPa as it is presented in Fig. 5b.
- The stress–strain curves of the tests conducted at the highest overconsolidation ratios (OCR = 6 for tests 2, 3, 5, and OCR = 2.6 for tests 6, 8, and 9) show a peak followed by softening behaviour. Such strain softening is more pronounced in tests performed at lower effective confining stresses ($\sigma'_3 = 1.0$ MPa). A similar trend is also reflected in the pore water pressure evolution, where a rapid increase is followed by a decrease, particularly for samples with OCR = 6 and $\sigma'_3 = 1.0$ MPa. Samples with an OCR of 1.3 (tests 1, 4, and 7) exhibit contraction behaviour, meaning that under loading, the clay tends to decrease in volume. This behaviour is characterized by a rapid increase in deviator stress and pore water pressure, followed by a plateau where further deformation results in only minor changes in stress.

3.2. Effect of solute concentration

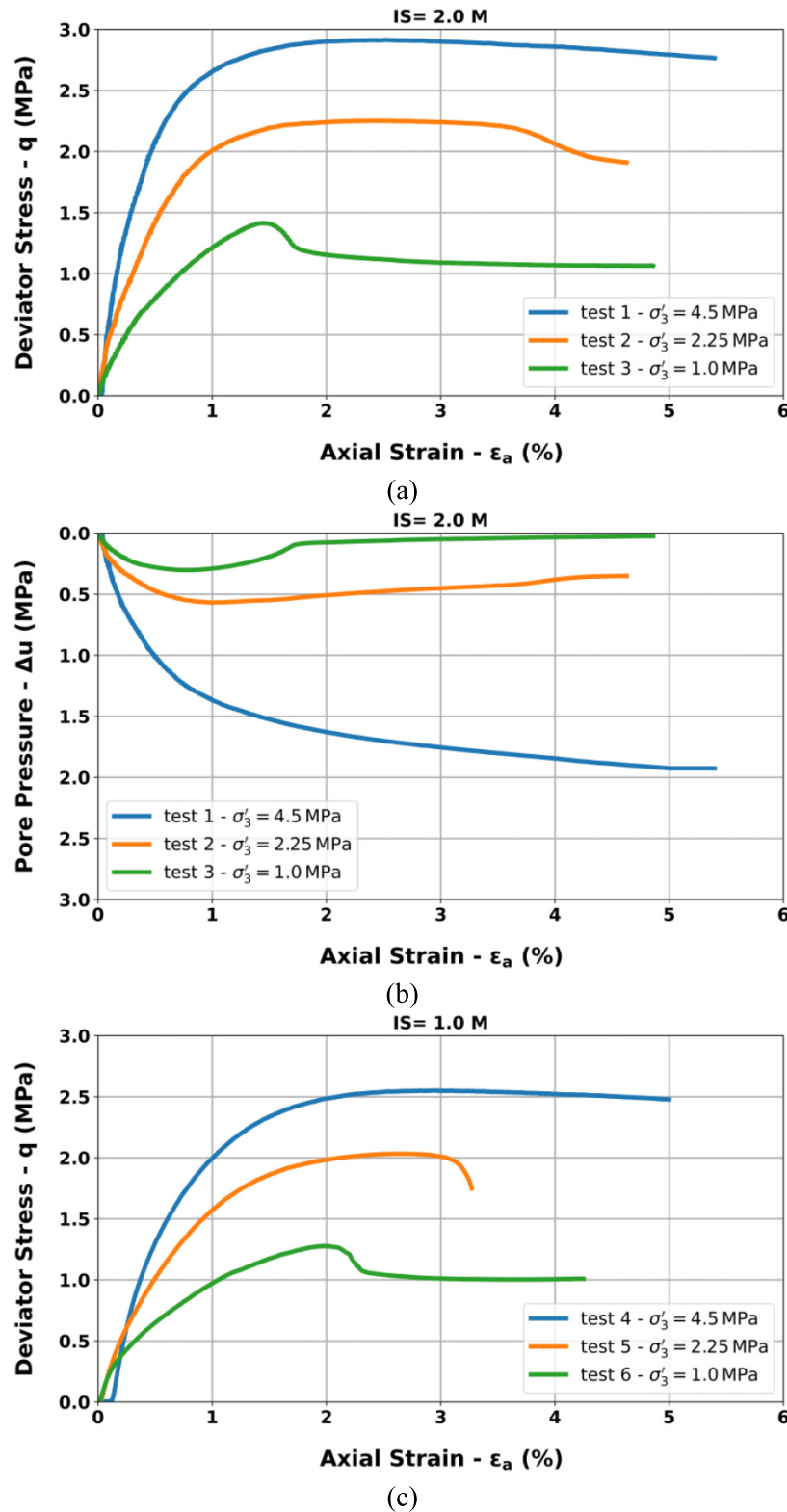
Figure 6 shows typical results of consolidated undrained triaxial compression shear tests percolated with three different saline solutions (0.015 mol/L; 1.0 mol/L; 2.0 mol/L) and subjected to three different effective confining pressures (1, 2.25, and 4.5 MPa). Note that percolation of the clay with 1.0 and 2.0 mol/L is theoretically shown to reach 60% and 90% sodium occupancy, respectively (Section 2.3.1).

For the three different effective confining stresses σ'_3 , the peak deviator stress (q_f) increases with higher ionic strength of (Na, Ca)NO₃ and hence also with higher sodium occupancy. As shown in Fig. 6a, under $\sigma'_3 = 4.5$ MPa, the stress–strain curve reveals that the sample percolated with 2.0 mol/L (Na, Ca)NO₃ (IS = 2.0 mol/L) exhibits a higher deviator stress of 2.92 MPa, compared to 2.55 and 2.25 MPa for samples percolated with 1.0 mol/L (Na, Ca)NO₃ (IS = 1.0 mol/L) and 0.015 mol/L NaHCO₃ (IS = 0.015 mol/L), respectively. Similar trend is observed for samples under 2.25 and 1 MPa, as shown in Figs. 6c and 6e, respectively. Table 5 summarizes, for each triaxial compression test, the peak deviator stress (q_f), residual deviator stress (q_r), and maximum pore water pressure (Δu_{max}) in relation to the initial effective confining stress and ionic strength.

Figure 6a shows that lightly overconsolidated specimens with an OCR of 1.3 exhibit contraction behaviour, characterized by a plateau following the peak deviator stress. This plateau indicates a stabilization in the stress response after reaching the peak. Such behaviour is typical for lightly overconsolidated clays (Baldi et al. 1991; Sultan et al. 2010).

In contrast, for the overconsolidated specimens (OCR = 2.6 and 6) shown in Figs. 6c and 6e, a different pattern emerges.

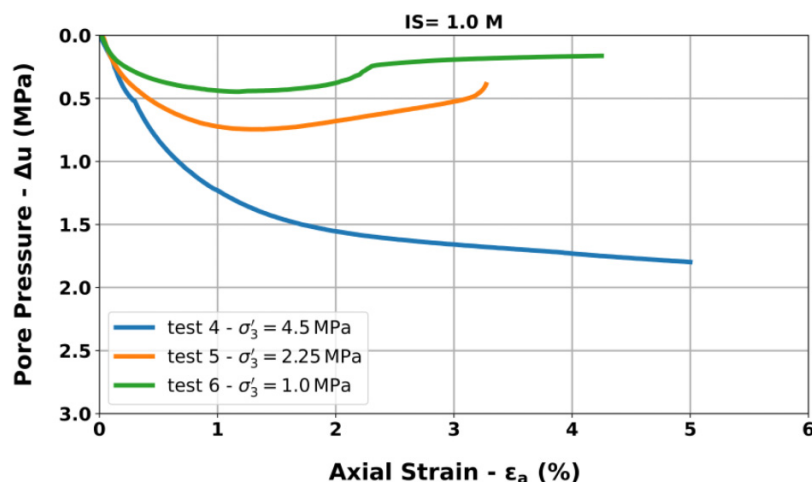
Fig. 5. Stress–strain and pore pressure curves evolutions under different effective confining stress.



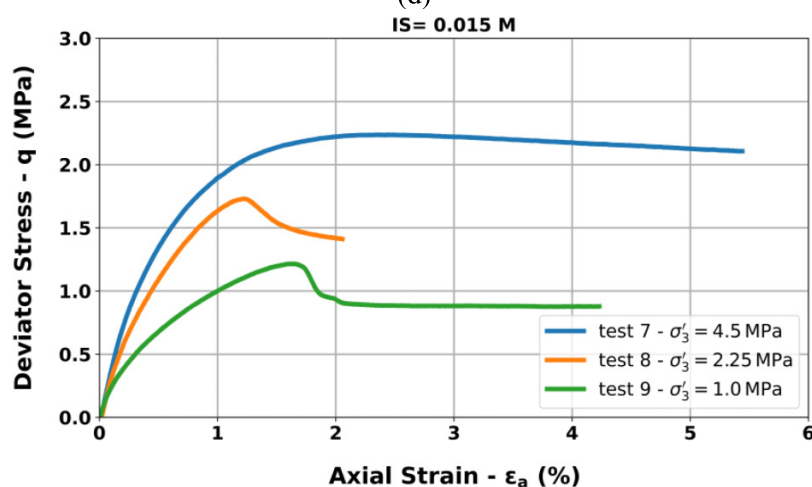
These samples reach a distinct peak deviator stress, followed by a softening phase that represents a loss of strength, often associated with the initiation of failure in a clay structure. This softening response is more pronounced for highly overconsolidated clays, where accumu-

lated stress from previous loading cycles makes them more susceptible to failure once the peak stress threshold is exceeded. The transition from peak to softening marks a shift from stable deformation to a weakening phase, during which the clay structure begins to deteriorate under

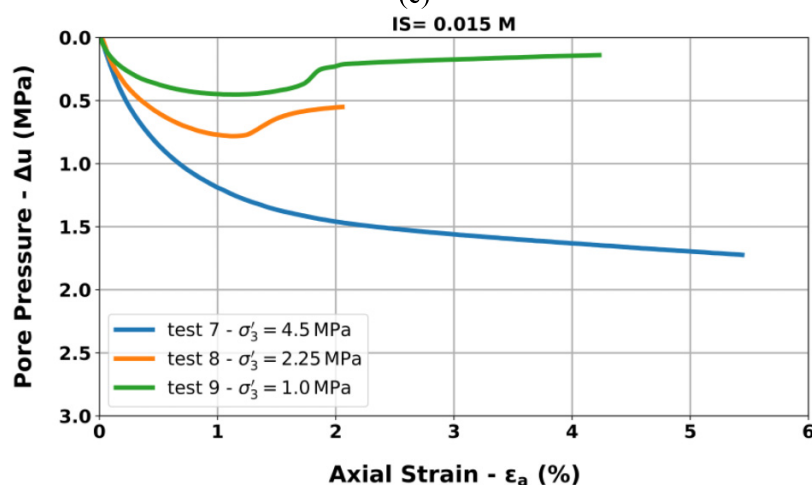
Fig. 5. (concluded).



(d)



(e)



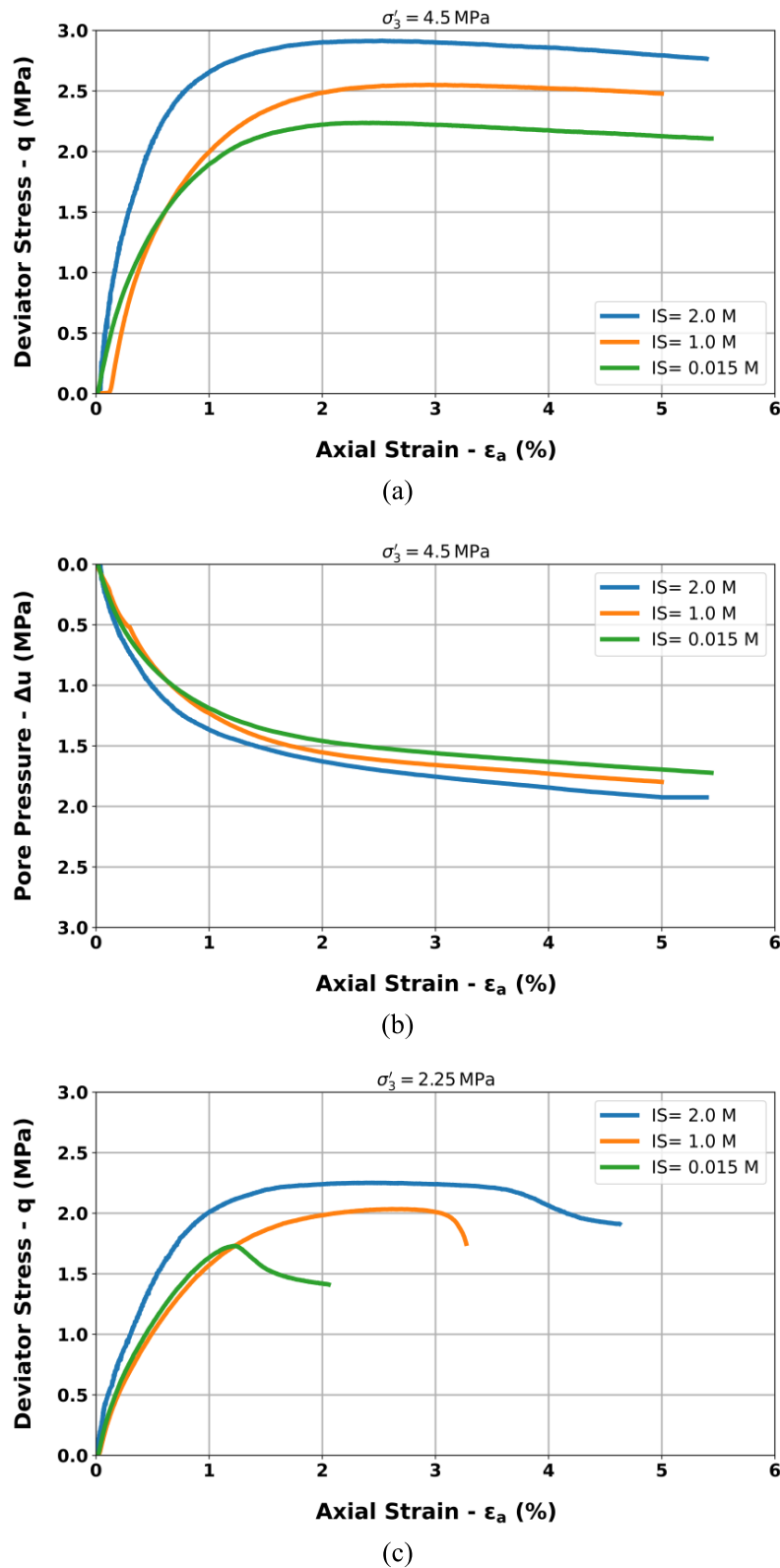
(f)

given stress, leading to the formation of the shear failure plane.

This distinction in behaviour between lightly and highly overconsolidated specimens underscores the impact of OCR on the mechanical response of BC. As it is widely reported

in the literature, higher OCR values are associated with a greater tendency for strength loss and structural failure, whereas lower OCR values generally favour stabilization after initial deformation (Holtz et al. 1981; Burland 1990).

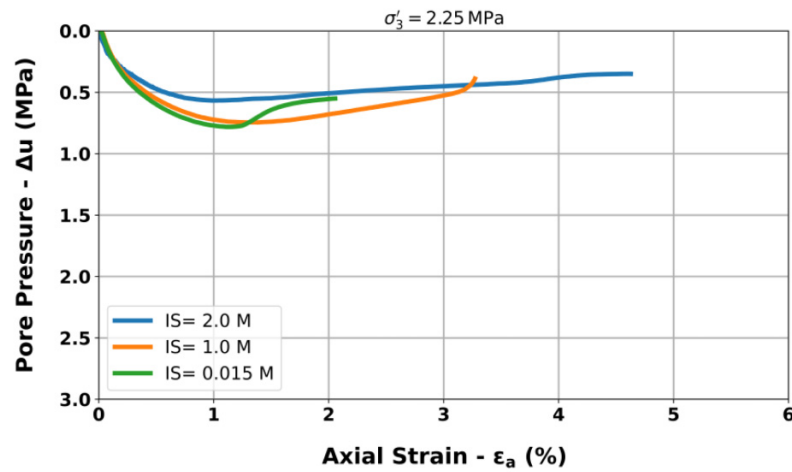
Fig. 6. Stress–strain curve and pore water pressure evolution under different ionic strength (IS).



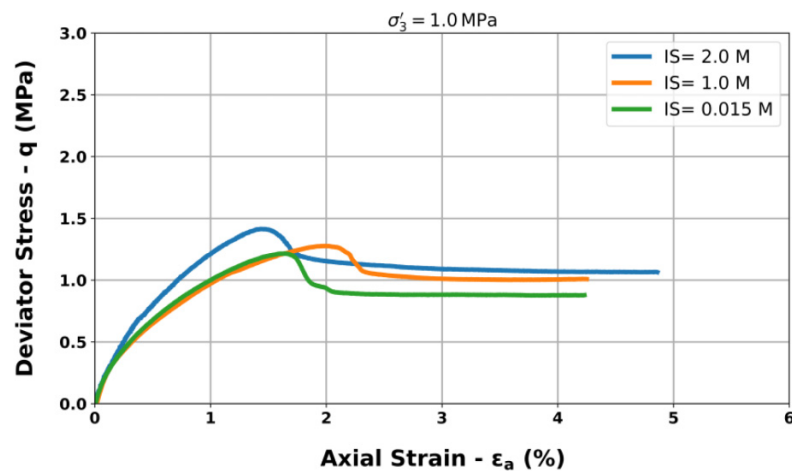
The stress paths in (p' , q) space are plotted in Fig. 7. They are consistent with the classical undrained stress paths typically observed for overconsolidated and lightly overconsolidated clay (Parry and Wroth 1981; Horseman et al. 1987; Giger et al.

2018). However, tests 4 and 9 exhibit an unusual verticality, which may be attributed to either load-control issues, where the applied load was not varied as intended, or from a technical limitation of the pore pressure transducer, which may

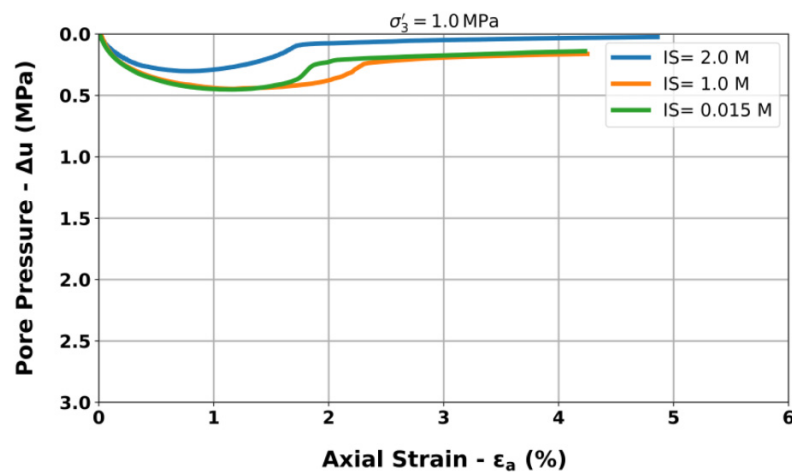
Fig. 6. (concluded).



(d)



(e)



(f)

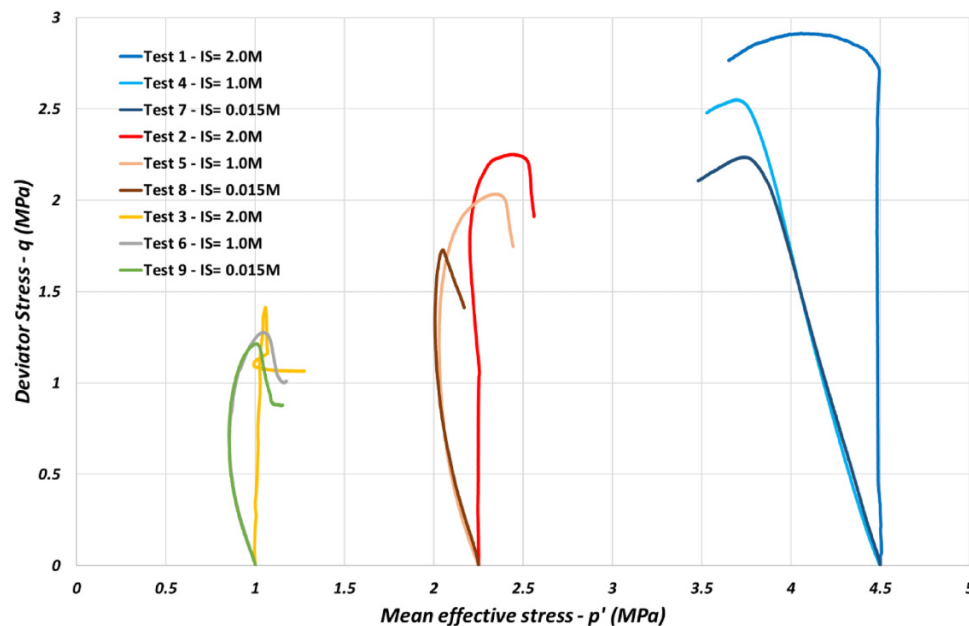
not have fully responded to the pressure fluctuations during testing.

In a common undrained triaxial test on an isotropic material, the mean effective stress (p') remains constant during the

elastic phase, leading to a linear stress path in the q - p' plane (Wood 1990). This occurs because, under undrained conditions, volume change is restricted, and the effective stress distribution remains uniform.

Table 5. Summary of test data.

Test No.	σ_3' (MPa)	IS (mol/L)	q_f (MPa)	q_r (MPa)	$\Delta u_{\max}$ (MPa)
1	4.5	2	2.92	2.92	1.92
4	4.5	1	2.55	2.55	1.85
9	4.5	0.015	2.25	2.25	1.72
2	2.25	2	2.25	1.92	0.56
5	2.25	1	2	1.75	0.75
8	2.25	0.015	1.7	1.42	0.75
3	1	2	1.41	1.06	0.3
6	1	1	1.27	1	0.45
7	1	0.015	1.2	0.87	0.45

Fig. 7. Stress paths in (p' , q) plane.

However, for an anisotropic material like Boom Clay, this behaviour deviates due to its inherent fabric and structural anisotropy. Specifically, the stress paths are not perfectly vertical, indicating that p' does not remain constant within the elastic zone. This deviation is particularly pronounced in samples percolated with lower ionic strength solutions (0.015 and 1.0 mol/L). This behaviour reflects the inherent anisotropy of Boom Clay, as it responds differently under varying stress conditions. In contrast, samples percolated with a higher ionic strength solution (2.0 mol/L) exhibit a more pronounced vertical stress path with constant p' . This behaviour is likely due to the increased osmotic suction associated with the contraction of the percolating solution, which enhances the clay structural rigidity and reduces soil's compressibility.

The general leftward slope of the stress paths reflects the net increase in pore pressure, as observed in previous analyses. In the overconsolidated samples (tests 2, 3, 5, 6, 8, and 9), the stress paths display a subtle rightward curvature both before and after failure. This behaviour indicates a tendency of dilatancy under high OCR conditions (OCR = 6 and

2.6). This rightward curvature is characteristic of overconsolidated clays and reflects internal resistance to deformation after peak stress is reached.

In contrast, the samples with a lower overconsolidation ratio (OCR = 1.3) show a leftward curvature in the post-failure region, suggesting a contractive response after the peak stress. This means that once failure occurs, the soil structure collapses, leading to a continued increase in pore pressure and further volume reduction. This divergence in stress path behaviour between overconsolidated and lightly overconsolidated specimens highlights the influence of OCR on the failure mechanisms and pore pressure responses.

3.3. Yielding

Various approaches have been proposed in the literature to determine the yield stress. For soft, structured clays sheared at constant confining pressure (e.g., Tavenas and Leroueil 1980), yield is often identified at the maximum deviator stress or the peak of the stress–strain curves. In isotropic compression tests, yield is commonly taken as the intersection of two best-fit linear segments in the (ε_v : $\ln p'$) plot. For denser

soils like compacted clays, where no peak is observed, a volumetric criterion may be applied (Cui et al. 1995; Delage and Cui 1996). Work criteria are also used to identify yield points (Leroueil et al. 1979). Sultan et al. (2010) determined yield stress based on a change in slope in linear (ε_v : $\ln p'$) plots, demonstrating that this method can reliably determine the yield stress. For Opalinus clay, Popp and Salzer (2007) and Zhang (2016) noted that the yield point could indicate the onset of microcracking, detected through shear wave velocity. However, while some shear wave velocity measurements on BC have been conducted (Piriyaikul 2006), there is no conclusive evidence that microfracture formation occurs beyond the yield point. Yielding in BC likely relates to modifications in microstructure or micro-fissuring, marking the transition to plastic deformation. After this yield point, BC enters a plastic deformation region, with stress increasing nonlinearly until peak stress (q_f), known as peak shear strength. After reaching the peak, all samples demonstrate a strain-softening response, with the stress-strain curve descending as shear resistance decreases due to continued plastic shear strain. This softening response, observed by other researchers (Horseman et al. 1987; De Bruyn and Thimus 1996; Coll 2005), continues until the shear resistance stabilizes at the residual shear strength (q_r). Limited studies were found in literature for the determination of the yield stress and elastic zone for the undrained triaxial shear test.

In this study, a new approach is introduced to define the elastic zone and yield stress (q_y) of BC under undrained conditions. Traditionally, yield stress is often identified based on the stress-strain curve, particularly at the point of maximum deviator stress or the change in slope of the stress-strain relationship. However, this method can sometimes be ambiguous, especially for stiff clays like Boom Clay. The proposed method focuses on the evolution of pore water pressure (Δu) in relation to the mean effective stress (p') during the triaxial tests. As seen in Fig. 8a, the pore water pressure initially increases in a nearly linear fashion up to ($\Delta u = 0.25$ MPa) with respect to p' , reflecting the elastic deformation. However, as the stress increases, an inflection point occurs where the slope of the curve changes. This change in slope is indicative of the point at which the clay transitions from elastic to plastic behaviour, marking the yield stress (q_y). This method is consistent with the volumetric criterion (Delage and Cui 1996; Sultan et al. 2010). The inflection point is crucial because it signifies the onset of microstructural changes within the clay, such as the initiation of microcracking or the reorganization of clay particles. To identify the yield stress more accurately, the inflection point in the $\Delta u - p'$ curve is projected onto the stress space, specifically the ($p':q$) plot (Fig. 8b) and subsequently onto the ($\varepsilon_a:q$) plot (Fig. 8b).

By using this method, the yield stress is determined based on the observable changes in the pore water pressure response, rather than just the deviator stress, offering a more comprehensive understanding of the behaviour under undrained conditions. This approach also provides a more reliable means to differentiate the elastic and plastic deformation regions.

The yield stress and undrained tangent Young's modulus are summarized in Table 6. The results clearly indicate that

both parameters depend on confining stress and salt concentration. Figure 9 illustrates the evolution of the undrained Young's modulus at three different confining pressures as a function of ionic strength. Regardless of the ionic strength, the undrained Young's modulus increases with increasing confining stress. Additionally, for each confining stress level, the undrained Young's modulus varies with ionic strength, showing a slight increase as ionic strength rises. This increase can be attributed to the shrinkage of the DDL and the densification of the Boom Clay material, leading to higher stiffness. This trend underscores the combined influence of confining stress and ionic strength on the mechanical properties of the material.

The undrained Young's modulus values reported in the literature for Boom Clay vary considerably. Mair and Taylor (1992) determined an undrained elastic modulus of approximately 1000 MPa, whereas Rousset et al. (1989) reported a slightly higher value of 1430 MPa. These higher values were measured using the bender element method, which captures the undrained Young's modulus at very low strain levels. Djeran-Maigre et al. (1994), who conducted a series of triaxial tests under both drained and undrained conditions, found a much lower short-term elastic modulus around 500 MPa. The differences among these values may be attributed to variations in the testing conditions and protocols, which were not thoroughly detailed in each study, as well as the heterogeneity of the samples, since the tests were performed on specimens from different BC cores.

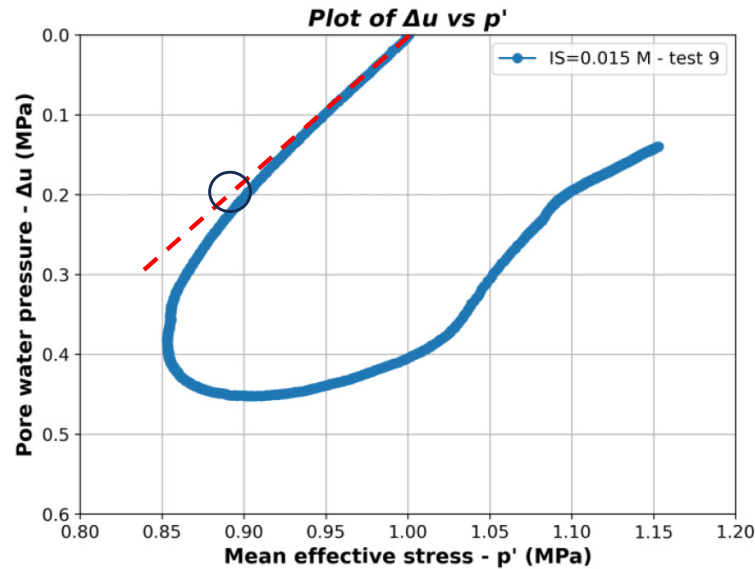
3.4. Critical state envelope

According to the French standard NF P 94-070 (Afnor 1994), soil failure in triaxial tests is defined at the point of the peak deviator stress (q_f). For the specimens with a low overconsolidation ratio (OCR = 1.3), which display contraction behaviour, the failure is characterized by a horizontal plateau in the stress-strain curve; so, for tests 1, 4, and 9, the failure is interpreted as the point where this plateau begins. In contrast, highly overconsolidated clay samples (OCR = 6 and 2.6) exhibit a peak stress followed by softening behaviour. These samples reach failure as the deviator stress gradually decreases to a residual value. This residual stress, observed at the plateau following the peak, is often used to characterize the ultimate shear strength along the critical state plane for overconsolidated specimens. The resulting set of stress states at failure can be used to construct the failure envelope, offering a comprehensive understanding of shear strength characteristics under different initial conditions.

The critical state, meanwhile, represents a stable condition where further deformation does not result in additional changes in volumetric strain (for drained triaxial tests) or pore water pressure (for undrained triaxial tests). At this stage, the soil reaches a balance, continuing to deform without any further internal stress adjustments. Identifying this state is important for better understanding the effect of salt concentration on the shear behaviour of BC.

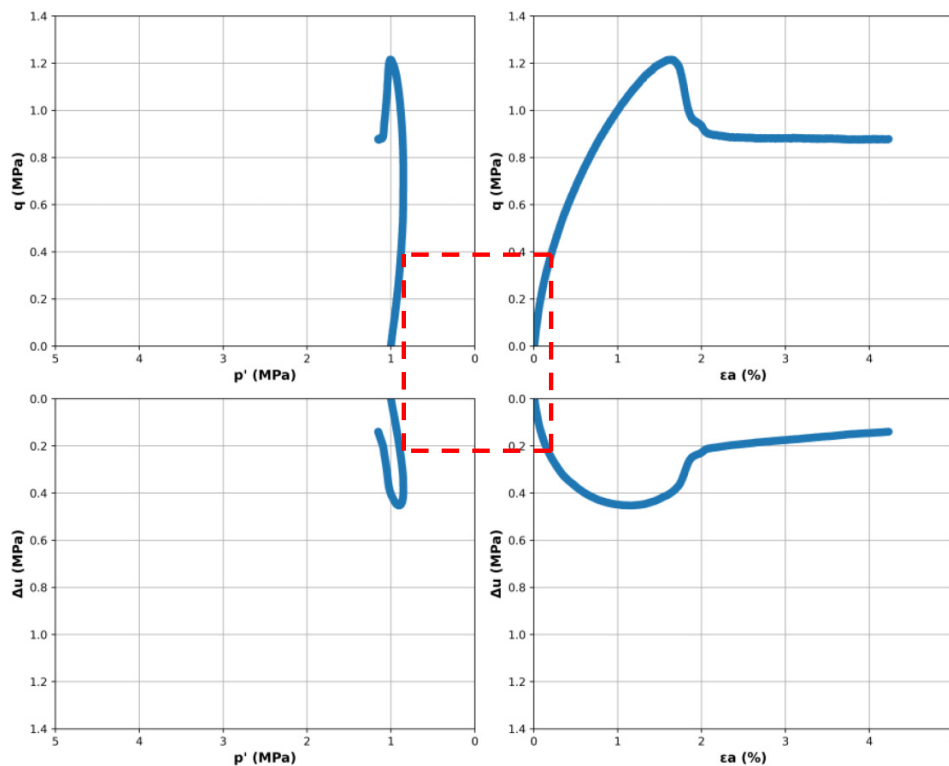
Regarding the chemical effects on residual shear strength, the stress states at failure for BC are presented in Fig. 10 in the $p':q$ plane. It can be observed that the failure plane for

Fig. 8. (a) Yield stress determined from a semi-log plot projected in a $(\Delta u:p')$ plot and (b) synthetic presentation of triaxial test 9.



(a)

IS = 0.015 M - test 9



(b)

samples with higher IS lies above those with lower IS. The failure plane shifts upwards as salt concentration increases, corresponding to a higher peak deviator stress at higher salt concentrations.

It should be noted that the failure planes determined in this study display a linear trend. However, the literature reports some bilinear trends for BC (Coll 2005; Bernier et al.

2011; Lima 2011). Other authors, such as Petley (1999), have suggested that for certain clays and mudrocks, a nonlinear failure criterion may be more appropriate, as for London clay.

These failure planes provide valuable insights into the key geomechanical parameters, such as effective cohesion and internal friction angle, for varying salt concentrations and thus also sodium occupancies. Table 7 presents the

Table 6. Yield deviator stress, effective stress, and undrained Young’s modulus obtained from this study.

Test No.	σ'_3 (MPa)	IS (mol/L)	q_y (MPa)	p'_y (MPa)	E_U (MPa)
1	4.5	2	1.9	4.48	600
4	4.5	1	1.25	4.1	435
7	4.5	0.015	1.0	4	340
2	2.25	2	1.0	2.25	340
5	2.25	1	0.7	2.05	290
8	2.25	0.015	0.5	2.025	280
3	1	2	0.5	1.0	200
6	1	1	0.425	0.87	195
9	1	0.015	0.40	1.0	190

Fig. 9. Young’s modulus evolution versus ionic strength (IS) under different effective confining stresses.

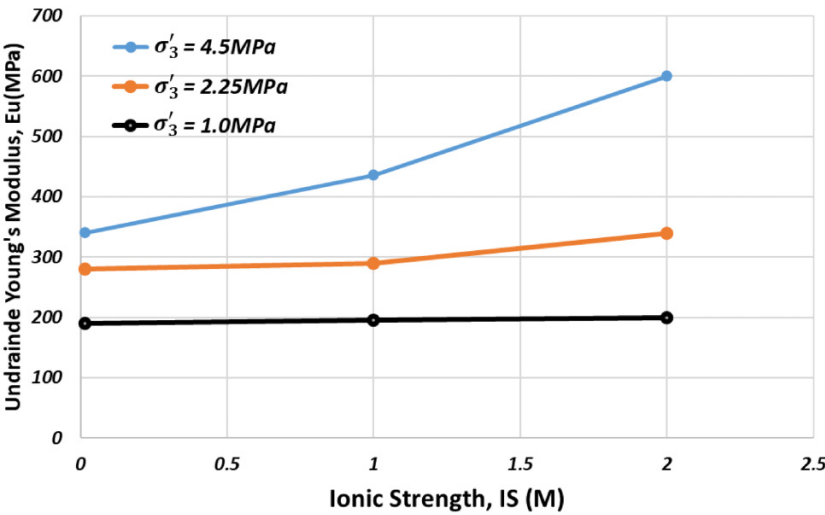


Fig. 10. Failure envelopes in the p' – q plane for Boom Clay at ambient temperature.

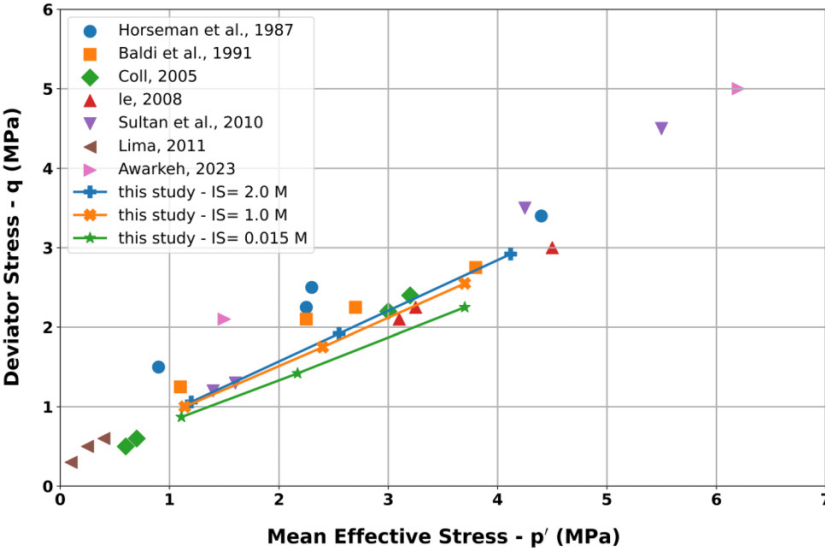


Table 7. Main characteristics corresponding to critical slope, effective friction angle, and cohesion at different ionic strength.

IS (mol/L)	M (–)	φ' (°)	c' (MPa)
0.015	0.55	14.6	0.13
1.0	0.60	15.8	0.135
2.0	0.65	17	0.14

evolution of the critical slope (M), effective cohesion, and internal friction angle for different IS levels. As indicated, the slope of the critical state line, which coincides with the failure plane, increases with increasing ionic strength. Specifically, the internal friction angle increases from 14.6° at IS = 0.015 mol/L (representing the in situ pore water of BC (De Craen et al. 2004)) to 17° for samples with IS = 2.0 mol/L, which corresponds to the theoretically estimated Na⁺ occupancy of 90%. In contrast, the effective cohesion remains relatively unaffected by higher ionic strength, exhibiting only a slight increase from 0.13 MPa at IS = 0.015 mol/L to 0.14 MPa at IS = 2.0 mol/L. This marginal change suggests that while the friction angle is notably influenced by ionic strength, the effective cohesion remains stable with varying salt concentrations.

Figure 10 presents the critical state strength envelope for Boom Clay at ambient temperature, incorporating the findings from this study alongside the results from various authors (Horseman et al. 1987; Baldi et al. 1991; Coll 2005; Lê 2007; Sultan et al. 2010; Lima 2011; Awarkeh 2023). All results appear consistent. The data scatter likely stems from variations in experimental procedures, particularly during the saturation phase and sample preparation, leading to differing interpretations of the strength relationship.

A single linear fit of the relationship between mean effective stress and deviator stress at the critical state suggests a slope of approximately 0.67 (Yu et al. 2018; Awarkeh 2023). Deng et al. (2011), however, proposed a bilinear model where an intersection point near the in situ effective stress divides the envelope into two linear sections: a low-stress and a high-stress segment. In the high-stress region, a slope of 0.48 was reported. Excluding the data from the current study, the literature suggests a general friction angle of approximately 17° and an effective cohesion of 0.2 MPa, which is close to the results from this study (14.6° and 0.55 MPa).

4. Discussion

Higher concentrations of salt ions in pore water influence the clay structure by shrinking the DDL around clay particles, which results in closer particle arrangements and increases the interparticle bonding. This process generally enhances soil stiffness (Van Olphen 1963; Barbour and Fredlund 1989; Mitchell and Soga 2005; Thyagaraj and Rao 2010, 2013; Rao et al. 2013).

At the clay fabric scale, initial suction in unsaturated clays primarily consists of matric suction ($u_a - u_w$) and inherent osmotic suction, which arises from the initial pore water chemistry. In fully saturated clayey soils, matric suction is negligible. When (Na, Ca)NO₃ solutions infiltrate, salts migrate

into the soil pores, raising the concentration of pore fluid. This process induces osmotic suction due to the concentration difference between the infiltrating solution and the original pore water. As osmotic suction increases, the stiffness of the soil is enhanced. At the particle level, the behaviour of clays is governed by the DDL, which controls both hydraulic and mechanical responses. The DDL forms around the negatively charged clay particle surface and the surrounding layer of dispersed cations (Mitchell and Soga 2005). According to Gouy–Chapman DDL theory, the thickness of this layer inversely varies with solute concentration and cation valency, with cation exchange altering the DDL thickness (Van Olphen 1963; Mitchell and Soga 2005). It has been observed that under fully saturated conditions, an increase in salinity causes a slight increase of shear strength for overconsolidated and more significant increase for slightly overconsolidated soils. When a higher concentration of (Na, Ca)NO₃ infiltrates the soil, the equilibrium between the clay, water, and electrolyte is disturbed. This alteration in pore fluid concentration triggers the displacement of exchangeable cations on the clay particle surfaces by the dominant cations present in the pore fluid; in this instance, Na⁺ ions. This displacement results in a modification of the DDL thickness. As Na⁺ ions infiltrate through the interlayer space of montmorillonite sheets, they can cause a shrinkage of these spaces, as demonstrated in previous studies (Di Maio and Fenelli 1997; Di Maio et al. 2004; Al Mais et al. 2024, 2025). This shrinkage further contributes to the increase in soil stiffness. These trends are consistent with findings from the literature. According to Sridharan and Prakash (1999), an increase in pore water concentration reduces the interparticle repulsive forces between clay particles while enhancing the attractive forces. This results in an increase in shear strength at the particle level. It is likely that the swelling of clays is inhibited as salt concentration increases, which in turn affects the DDL. As the DDL becomes suppressed, the distance between particles decreases, leading to the increase of stiffness. Consequently, this suppression of DDL generally causes a decrease in void ratio (i.e., a reduction in water content), contributing to the hardening of soil. Di Maio and Fenelli (1997), Di Maio et al. (2004), Barbour and Yang (1993), among others, showed that when clay soils are permeated with saline solutions, an increase in salt concentration results in decreased soil compressibility and higher strength. It is worth noting that these tests were performed under fully saturated conditions, i.e., suction is zero. Additionally, most of the specimens used in these studies were prepared by initially mixing the soil with distilled water and then permeating it with brine. This approach reflects a scenario where the original soil structure remains partly intact while the pore fluid is replaced, representing short-term exposure to the salt solution. However, Di Maio and Fenelli (1997) and Di Maio et al. (2004) also conducted shear tests on samples initially reconstituted with brine, and an increase in shear strength was observed in these cases as well. Observations showed that the soil prepared with brine exhibited a lower void ratio, which likely contributed to the increase in strength, associated with both the increase in pore solution concentration and the compaction effect resulting from the brine exposure.

Mokni (2011) performed a series of direct shear tests on Boom Clay powder mixed with NaNO_3 solution. The results demonstrated that under partially saturated conditions, an increase in salinity caused a slight decrease in both compressibility and shear strength. Note that the samples prepared with NaNO_3 solution exhibited a reduction in water content at a given matric suction. This reduction in shear strength could be attributed to the lower water content of soil, which is equilibrated with the electrolyte at the given matric suction. Therefore, the observed decrease in strength may be primarily due to the change in water content, which influences the mechanical response of the material under these specific conditions. Barbour and Yang (1993) summarized osmotic consolidation tests conducted by Yang (1990), Ho (1985), and Barbour (1986) on remoulded and undisturbed samples of glacial till and glaciolacustrine clay, exposed to various solution concentrations. The samples were subjected to specific stress levels and then equilibrated with NaCl solution through diffusion. Following this, they were reloaded to higher stresses. The results showed that when the water-saturated clay was exposed to NaCl brine, it underwent a decrease in volume. This volume change was accompanied by an apparent increase in preconsolidation stress and a decrease in compressibility, until the new preconsolidation stress was exceeded. Afterwards, the samples continued to consolidate along the same virgin compression line. Similar observations were reported by Barbour and Fredlund (1989). Ho (1985), as cited by Barbour and Yang (1993), also showed results for the samples remoulded with brine and then tested in water. A noteworthy observation was that submersion of the samples in water caused a decrease in volume under constant effective stress. Di Maio and Fenelli (1997) investigated the effects of exposure of water-saturated specimens of Ponza bentonite, alternately to pure water and saturated NaCl, KCl, or CaCl_2 solutions. Exposure to these three electrolytes produced consolidation of the specimens, large volumetric deformations, and an increase in residual shear strength. The effects of NaCl exposure were reversible in terms of ion exchange reactions when the samples were re-exposed to pure water, while those for KCl and CaCl_2 were not. The irreversibility observed with KCl and CaCl_2 is mainly due to the stronger electrostatic interactions between the clay particles and the potassium (K^+) and calcium (Ca^{2+}) ions. K^+ ions have a lower hydration energy and are more tightly bounded to the clay particles than sodium (Na^+) ions, making their displacement less likely. Additionally, Ca^{2+} ions, being divalent, form even stronger bonds with the clay particles, further reducing the reversibility of the effects. These stronger ion-clay interactions lead to more permanent changes in the clay structure when exposed to KCl or CaCl_2 solutions.

5. Conclusion

The influence of salt concentration and effective confining stress on the hydro-mechanical properties of clayey formations is an important input for the assessment of the functioning of underground repositories for radioactive waste disposal. This study presents a series of undrained triaxial laboratory tests conducted on Boom Clay, which is studied as

one of the reference host formations in Belgium, to investigate the chemo-hydro-mechanical response of the material. All tests were performed on saturated samples under various stress and chemical conditions. From the salt concentration and mechanical conditions considered in this study, the following conclusions can be drawn:

1. The OCR significantly affects the contraction/dilatancy behaviour of Boom Clay percolated with the same saline solutions. The samples with $\text{OCR} = 1.3$ exhibited a contraction behaviour, characterized by a rapid increase in deviator stress and pore water pressure, followed by a plateau. In this case, the pore water pressure rose sharply due to the undrained conditions and the compressive nature of the deformation, stabilizing once the stress state approached steady conditions. In contrast, overconsolidated samples ($\text{OCR} = 6$ and 2.6) showed a peak followed by strain softening associated with failure. Such strain softening was more pronounced at lower effective confining stresses. The evolution of pore water pressure in these cases differed significantly. Initially, the pore pressure increases sharply, reaching a peak, before starting to drop and eventually approaching zero or negative values as the shear plane develops.
2. An increase in shear strength was observed with increasing salt concentration. This hardening was interpreted by the increased critical state plane, internal friction angle, and elastic modulus, attributed to the shrinking of the DDL. This suggests tighter interparticle spacing due to suppressed DDLs. However, the chemical effect on the effective cohesion remains insignificant.
3. The pore pressure build-up during undrained shearing was found to be dependent on the initial mean effective stress and chemical concentration. Higher pore water pressure was generated with higher salt concentration under the same initial confining stress.

This study advances the understanding of chemical effects on the mechanical behaviour of Boom Clay. Further research is needed to investigate the thermal impact on its hydro-chemo-mechanical properties, with particular emphasis on time-dependent behaviour under coupled thermo-chemo-hydro-mechanical conditions.

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Data availability

The data supporting the findings of this study are available from the corresponding author upon reasonable request.

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Formal analysis: HAS, YC

Investigation: HAS, YC, FC

Methodology: HAS, YC, EV, DJ

Project administration: TG

Supervision: YC, XL, EV, FC

Validation: SL, FC

Visualization: HAS

Writing – original draft: HAS

Writing – review & editing: HAS, YC, SS, EV, TG, SL

Competing interests

The authors declare that there is no conflict of interest.

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