

Desiccation Cracking in Fine-Grained Soils: Experimental Investigation, Constitutive Interpretation, and Numerical Modelling

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by

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Research Framework

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To my family,

To my sister,

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Publications Related to this Thesis

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Journal Manuscripts Ready for Submission

1. Ebrahimisadr H., François B. Development of an Instrumented Desiccation Testing Device and Image-Based Investigation of Tensile Stress Evolution, Deformation Localisation, and Cracking in Compacted Clayey Soils
2. Ebrahimisadr H., François B. Experimental Validation and Numerical Investigation of Drying-Induced Deformation and Desiccation Behaviour in Compacted Clayey Soils Using a Coupled THM Framework

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2. Ebrahimisadr H., François B. Tensile Strength and Crack Pattern During Desiccation of Compacted Clayey Soils. 3rd International Conference on Energy Geotechnics ICEGT, June 2025, Paris, France. [PDF](#)
3. Ebrahimisadr H., François B. Water Retention Curves and Tensile Strength for Studying Desiccation Cracking of Compacted Clay Soils. 8th International Conference on Unsaturated Soils UNSAT, May 2023, Milos, Greece. [PDF](#)

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- Ebrahimisadr H. Water Retention Curve to Study Desiccation Cracking of Compacted Clay Soils. Department/Research Unit ArGEnCo/UEE day, November 10, 2022, Liège, Belgium.

ABSTRACT

Desiccation-induced cracking in compacted clayey soils significantly affects the hydro-mechanical performance of geotechnical and geo-environmental structures. Despite its importance, the mechanisms governing stress development and crack initiation during drying remain only partially understood due to the strong coupling between hydraulic and mechanical processes. This thesis aims to improve the understanding of desiccation mechanisms through the development of an integrated experimental, constitutive, and numerical framework.

An original experimental setup was developed to directly measure tensile forces generated by restrained shrinkage under controlled boundary conditions. In parallel, Particle Image Velocimetry (PIV) techniques were implemented to quantify full-field surface displacements during drying, enabling a coupled mechanical–kinematic analysis. The results highlight the progressive and time-dependent development of tensile stresses, ultimately leading to crack initiation, and confirm the dominant role of boundary conditions in controlling stress build-up.

From a constitutive perspective, a methodology was proposed to calibrate the effective stress parameter as a function of the degree of saturation, based on shear strength considerations. This approach provides a consistent framework for describing the mechanical behaviour of unsaturated soils and is supported by a microstructural interpretation derived from pore size distribution analysis.

A coupled hydro-mechanical numerical model was developed to reproduce the drying process. The integration of experimentally derived parameters and calibrated constitutive relationships enabled the model to capture the evolution of suction, deformation, and stress. The comparison with experimental observations shows that the model reproduces the overall trends of the desiccation behaviour, while highlighting its limitations in representing localized deformation.

Overall, this work provides a consistent link between experimental observations, constitutive interpretation, and numerical modelling, contributing to an improved understanding of desiccation mechanisms in compacted clayey soils.

RÉSUMÉ

La fissuration induite par dessiccation dans les sols argileux compactés affecte de manière significative le comportement hydro-mécanique des ouvrages géotechniques et géo-environnementaux. Malgré son importance, les mécanismes régissant le développement des contraintes et l'initiation des fissures lors du séchage ne restent encore que partiellement compris en raison du fort couplage entre les processus hydrauliques et mécaniques. Cette thèse vise à améliorer la compréhension des mécanismes de dessiccation à travers le développement d'un cadre intégré expérimental, constitutif et numérique.

Un dispositif expérimental original a été développé afin de mesurer directement les efforts de traction générés par le retrait empêché sous des conditions aux limites contrôlées. Parallèlement, des techniques de Particle Image Velocimetry (PIV) ont été mises en œuvre pour quantifier les déplacements de surface au cours du séchage, permettant une analyse couplée mécanique-cinématique. Les résultats mettent en évidence le développement progressif et dépendant du temps des contraintes de traction, conduisant à l'initiation des fissures, et confirment le rôle prépondérant des conditions aux limites dans l'évolution des contraintes.

Du point de vue constitutif, une méthodologie a été proposée pour calibrer le paramètre de contrainte effective en fonction du degré de saturation, sur la base de considérations de résistance au cisaillement. Cette approche fournit un cadre cohérent pour décrire le comportement mécanique des sols non saturés et est appuyée par une interprétation microstructurale issue de l'analyse de la distribution des tailles de pores.

Un modèle numérique hydro-mécanique couplé a été développé afin de reproduire le processus de séchage. L'intégration des paramètres issus des essais expérimentaux et des relations constitutives calibrées permet au modèle de reproduire l'évolution de la succion, des déformations et des contraintes. La comparaison avec les observations expérimentales montre que le modèle reproduit les tendances globales du comportement de dessiccation, tout en mettant en évidence ses limites pour représenter les déformations localisées.

Dans l'ensemble, ce travail établit un lien cohérent entre observations expérimentales, interprétation constitutive et modélisation numérique, contribuant à une meilleure compréhension des mécanismes de dessiccation dans les sols argileux compactés.

CONTENTS

ABSTRACT.....	VII
RÉSUMÉ	VIII
1. INTRODUCTION	1
1.1 Background and Motivation.....	1
1.2 Scientific Context and Research Gap.....	1
1.3 Objectives and Scope of the Thesis.....	2
1.4 Main Contributions	3
1.5 Organisation of the Thesis.....	4
2. THEORETICAL BACKGROUND AND LITERATURE REVIEW	5
2.1 Introduction	5
2.2 Theory of Unsaturated Soil Mechanics	7
2.2.1 Stress definition	7
2.2.2 Two independent stress variables	8
2.2.3 Single effective stress approach for unsaturated soils	9
2.2.4 Soil–water characteristic curve and unsaturated soil property functions	15
2.3 Mechanical Behaviour of Unsaturated Soils	19
2.3.1 Shear strength behaviour	19
2.3.2 Wetting collapse behaviour.....	20
2.3.3 Behaviour during drying.....	21
2.4 Suction Measurement and Control Techniques	21
2.4.1 Components of soil suction	22
2.4.2 Suction measurement techniques.....	23
2.4.2.1 High-capacity tensiometers	23
2.4.2.2 Psychrometers and dew-point techniques.....	24
2.4.2.3 Filter paper method.....	24
2.4.3 Suction control techniques.....	26
2.4.3.1 Axis translation technique	26
2.4.3.2 Osmotic technique	27

2.4.3.3 Vapour equilibrium technique.....	28
2.5 Desiccation Testing and Cracking Mechanisms.....	31
2.5.1 Experimental approaches for desiccation cracking	31
2.5.2 Restrained shrinkage, tensile stress development and crack initiation.....	33
2.5.3 Direct measurement of tensile strength during desiccation.....	35
2.5.4 Positioning of the present research	38
2.6 Conclusion.....	39
3. MATERIALS AND EXPERIMENTAL CHARACTERISATION.....	41
3.1 Standard Characterization of the Material	41
3.1.1 Grain size distribution	41
3.1.2 Atterberg limits	42
3.1.3 USCS classification	43
3.1.4 Specific gravity	44
3.1.5 Standard proctor test.....	44
3.2 Sample Preparation	46
3.3 Initial Suction	47
3.4 Water Retention Capacity.....	48
3.4.1 Saline-solution method.....	48
3.4.2 Dynamic vapor sorption (DVS).....	53
3.4.3 Water retention curve.....	54
3.5 Volumetric Strain.....	58
3.6 Uniaxial Compression Test (UCS)	58
3.7 Triaxial Tests (CU)	61
3.8 Brazilian Tensile Test	63
3.9 Hydraulic Conductivity	66
3.10 Conclusion.....	66
4. CALIBRATION OF THE EFFECTIVE STRESS PARAMETER.....	69
4.1 Introduction	69
4.2 Generalized Effective Stress Framework	70
4.3. Methodology for c Parameter Calibration.....	70
4.4 Calibration Results and Discussion.....	72
4.5 Microstructural Interpretation	76
4.6 Conclusion.....	82
5. EXPERIMENTAL INVESTIGATION OF DESICCATION BEHAVIOUR.....	84
5.1 Introduction	84

5.2 Experimental Apparatus	85
5.2.1 Design concept	85
5.2.2 Aluminium desiccation box	86
5.2.3 Sliding rail system	88
5.2.4 Force measurement system.....	88
5.2.5 Evaluation and correction of system friction.....	89
5.2.6 Imaging and monitoring system	91
5.3 Testing Procedure	92
5.3.1 Soil preparation.....	92
5.3.2 Installation in the apparatus	93
5.3.3 Desiccation and data acquisition	94
5.3.4 Image processing and displacement analysis	95
5.4 Particle Image Velocimetry (PIV)	96
5.4.1 PIV history.....	98
5.4.2 Application of PIV in geomechanics	99
5.4.3 Image texture	100
5.4.4 Particle Image Velocimetry.....	100
5.4.5 Centroiding	101
5.5 Experimental Results.....	102
5.5.1 Overview of the testing programme	102
5.5.2 Evolution of tensile force during drying.....	103
5.5.3 Tensile stress–time behaviour and crack development.....	104
5.5.4 Crack initiation and surface observations.....	111
5.5.5 Full-field displacement and strain analysis using PIV	112
5.5.6 Displacement profiles along the specimen midline	120
5.5.6.1 Behaviour of soignies soil	121
5.5.6.2 Behaviour of kruibeke soil	125
5.5.6.3 Behaviour of tournai soil	129
5.6 Discussion	132
5.7 Conclusions	134
6. HYDRO-MECHANICAL MODELLING OF DESICCATION BEHAVIOUR	136
6. 1 Hydro-Mechanical Framework	136
6.2 Geometry and Mesh	138
6.3 Material Parameters.....	140
6.4 Boundary Conditions and Initial State	142

6.5 Numerical Modelling Results.....	143
6.5.1 Displacement fields under unconstrained conditions	143
6.5.2 Displacement fields under constrained conditions	146
6.5.3 Comparison of experimental and numerical results under unconstrained conditions	148
6.5.4 Comparison of experimental and numerical results under constrained conditions	151
6.6 Discussion	153
6.6.1 Comparison between experimental and numerical results	153
6.6.2 Limitations of the numerical model.....	154
6.7 Conclusions	154
7. CONCLUSIONS AND PERSPECTIVES	156
7.1 Conclusions	156
7.2 Perspectives	157
REFERENCES	i
List of Figures	xi
List of Tables.....	xvi

1. INTRODUCTION

1.1 Background and Motivation

Unsaturated soils are widely encountered in geotechnical and geo-environmental engineering applications, including earth structures, embankments, compacted clay liners, and engineered barriers for waste containment. In many of these systems, environmental conditions induce drying processes, leading to an increase in suction within the soil. This increase in suction is accompanied by volumetric shrinkage, which may generate tensile stresses when deformation is partially or fully restrained. Once the tensile strength of the material is exceeded, cracking occurs.

Desiccation-induced cracking significantly affects the hydro-mechanical behaviour of soils. From a mechanical perspective, cracking reduces stiffness and strength, while from a hydraulic point of view, it leads to a substantial increase in permeability due to the formation of preferential flow paths. As a result, desiccation cracking may compromise the integrity and long-term performance of geotechnical structures, particularly in applications where low permeability is essential.

Despite its importance, the mechanisms governing desiccation remain only partially understood. In particular, the development of tensile stresses during drying, their relationship with suction and soil state variables, and the conditions leading to crack initiation are still not fully clarified. This is mainly due to the strong coupling between hydraulic and mechanical processes in unsaturated soils, as well as the difficulty of reproducing and monitoring these processes under controlled laboratory conditions.

1.2 Scientific Context and Research Gap

Previous research has addressed desiccation from experimental, theoretical, and numerical perspectives, highlighting the key role of suction increase, boundary conditions, and soil microstructure. However, several important limitations remain.

From an experimental point of view, many studies have focused either on crack pattern observation or on global deformation measurements. While these approaches provide valuable insight into desiccation phenomena, they generally do not allow a direct quantification of the tensile stresses developing within the soil during drying. In addition, tensile strength is often evaluated using indirect methods or under constant suction conditions, which do not reproduce the actual stress path experienced during desiccation.

Furthermore, only a limited number of studies combine mechanical measurements with full-field deformation analysis at the laboratory scale. As a consequence, the link between stress development, strain localisation, and crack initiation remains insufficiently documented.

From a constitutive perspective, the mechanical behaviour of unsaturated soils strongly depends on the definition of an appropriate stress variable. The generalized effective stress framework provides a suitable framework for describing this behaviour by incorporating suction through an effective stress parameter. However, the determination of this parameter remains challenging, as many existing formulations rely on complex microstructural considerations or parameters that are difficult to identify experimentally. This limits their applicability in practical engineering problems.

From a numerical modelling point of view, several approaches have been proposed to simulate desiccation processes. Nevertheless, their predictive capability is strongly dependent on the consistency between experimental observations and constitutive modelling. In particular, the lack of experimentally calibrated relationships linking suction, saturation, and mechanical response remains a key limitation.

Overall, the current state of the art reveals a lack of an integrated framework combining direct experimental measurements, consistent constitutive interpretation, and physically based numerical modelling of desiccation processes.

1.3 Objectives and Scope of the Thesis

The main objective of this thesis is to improve the understanding of desiccation mechanisms in compacted clayey soils through the development of an integrated experimental, constitutive, and numerical approach. The study considers three different soils with varying plasticity and microstructural characteristics. As a preliminary step, the three soils are characterized experimentally from an hydro-mechanical point of view, including water retention properties,

tensile and compressive strengths at various suctions and triaxial compression behaviour under saturated conditions.

Then, as a first main contribution, an experimental framework is developed to directly characterise the mechanical response of soils during desiccation. A specific device is designed to measure the tensile forces generated by restrained shrinkage under controlled boundary conditions. In parallel, an image analysis system combined with Particle Image Velocimetry (PIV) techniques is implemented to quantify full-field surface displacements and strain localisation. This coupled mechanical–kinematic approach allows a detailed investigation of the relationship between shrinkage, stress development, and crack initiation.

Second, a constitutive interpretation is proposed within the framework of generalized effective stress. A methodology is developed to calibrate the effective stress parameter as a function of the degree of saturation, based on shear strength considerations. This approach provides a simple and robust relationship, ensuring a consistent framework to describe the mechanical behaviour of unsaturated soils.

Third, a coupled hydro-mechanical numerical modelling strategy is implemented to reproduce the drying process. The model integrates the experimentally derived parameters and the calibrated constitutive relationships, enabling a consistent description of suction evolution, deformation, and stress development.

1.4 Main Contributions

The originality of this work lies in the integration of experimental, constitutive, and numerical approaches within a consistent and unified framework.

The main contributions are summarized as follows:

- Development of an experimental device enabling direct measurement of tensile forces during desiccation.
- Implementation of a coupled monitoring system combining force measurements and full-field displacement analysis using Particle Image Velocimetry (PIV).
- Experimental investigation of tensile stress evolution in relation to suction, shrinkage, and crack initiation.

- Proposal of a methodology for calibrating the effective stress parameter based on shear strength considerations.
- Microstructural interpretation of the evolution of the effective stress parameter.
- Development of a coupled hydro-mechanical numerical model incorporating experimentally calibrated parameters.
- Demonstration of consistency between experimental observations and numerical simulations.

1.5 Organisation of the Thesis

The thesis is organised as follows.

Chapter 2 presents the theoretical background and literature review related to unsaturated soil mechanics.

Chapter 3 describes the experimental characterisation of the studied soils.

Chapter 4 focuses on the calibration of the effective stress parameter.

Chapter 5 presents the experimental investigation of desiccation processes.

Chapter 6 is dedicated to the hydro-mechanical numerical modelling.

Finally, Chapter 7 summarises the main findings and provides perspectives for future research.

2. THEORETICAL BACKGROUND AND LITERATURE REVIEW

2.1 Introduction

Unsaturated soils are soils in which the pore space is occupied simultaneously by water and air (Fredlund and Morgenstern 1977). These conditions are commonly encountered in near-surface soils above the groundwater table as well as in compacted geomaterials used in several geotechnical and geo-environmental engineering applications, such as embankments, earth dams, compacted liners, engineered barriers, and slopes (Fredlund 2006). In contrast to saturated soils, the simultaneous presence of water and air within the pore network introduces additional hydraulic variables that significantly influence the mechanical behaviour of the soil.

Classical soil mechanics was initially developed for saturated conditions through the effective stress principle proposed by (Terzaghi 1936). However, experimental studies performed on partially saturated soils demonstrated that their behaviour differs significantly from that observed under saturated conditions, particularly with respect to shear strength, volume change, and hydraulic behaviour (Bishop et al. 1960). The need to describe the behaviour of soils under partially saturated conditions, therefore, led to the development of unsaturated soil mechanics.

In unsaturated soils, the interaction between the solid skeleton, pore water, and pore air governs the hydro-mechanical response of the material. Two main theoretical frameworks have been developed to describe the stress state of unsaturated soils. (Fredlund and Morgenstern 1977) proposed a theoretical framework for unsaturated soils based on independent stress state variables, whereas (Bishop 1959) introduced the generalized effective stress approach, in which the contribution of suction is incorporated through an effective stress parameter. (Fredlund and Morgenstern 1977) also considered the air–water interface as an important component influencing soil behaviour. The interaction at the air–water interface plays an important role in the stress-transfer mechanisms governing the behaviour of partially saturated soils. One of the main characteristics of unsaturated soils is the presence of suction, generally associated with negative pore-water pressure relative to pore-air pressure. Suction influences

the interaction between the pore fluid and the soil skeleton, thereby affecting the material's mechanical response. Capillary effects at the air–water interface generate interparticle forces within the soil structure (Fisher 1926). As a consequence, suction directly influences the hydro-mechanical behaviour of unsaturated soils.

The behaviour of unsaturated soils is intrinsically coupled because hydraulic changes directly affect the mechanical response of the soil skeleton. Variations in suction and degree of saturation influence stiffness, compressibility, shear strength, permeability, and deformation behaviour (Alonso et al. 1990). In general, drying processes increase suction and tend to increase shear strength and stiffness, whereas wetting processes reduce suction and may induce collapse or instability.

The coupled hydro-mechanical behaviour of unsaturated soils plays an important role in many geotechnical problems subjected to environmental actions such as rainfall infiltration, evaporation, and drying. Variations in suction conditions may strongly influence the stability and performance of geotechnical structures. Rainfall infiltration may reduce suction and contribute to slope instability in residual soils (Rahardjo et al. 2019). Conversely, drying processes in compacted clayey soils may induce volumetric shrinkage and tensile stress development. When shrinkage is restrained, tensile stresses progressively develop within the soil mass, and cracking may occur once the tensile strength of the material is exceeded.

Desiccation cracking is one of the most important consequences of hydro-mechanical coupling in unsaturated fine-grained soils. Crack formation modifies both the hydraulic and mechanical behaviour of the material by increasing permeability and altering the integrity of the soil structure. Consequently, understanding the mechanisms governing suction evolution, deformation, stress development, and cracking is essential for the analysis of unsaturated geomaterials and their engineering applications.

Over the past decades, developments in unsaturated soil mechanics progressively enabled the incorporation of suction effects into laboratory characterization, constitutive modelling, and numerical analyses for geotechnical and geo-environmental engineering applications (Fredlund 2006; Gens 2010). The theoretical aspects related to these formulations are presented in the following section.

2.2 Theory of Unsaturated Soil Mechanics

2.2.1 Stress definition

The formulation of classical saturated soil mechanics is mainly based on the effective stress principle proposed by (Terzaghi 1936), in which the mechanical behaviour of saturated soils is governed by the difference between total stress and pore-water pressure. Under saturated conditions, the effective stress represents the stress transmitted through the soil skeleton and constitutes the basis for the interpretation of strength and deformation behaviour.

However, the direct extension of the effective stress principle to unsaturated soils is not straightforward because the pore space is occupied simultaneously by water and air (Fredlund and Morgenstern 1977). Experimental observations showed that partially saturated soils exhibit a behaviour significantly different from that observed under saturated conditions, particularly regarding strength, deformation, and hydraulic behaviour (Bishop et al. 1960). The simultaneous presence of two fluid phases within the pore network, therefore, introduces additional variables influencing the hydro-mechanical response of the soil.

One of the main characteristics of unsaturated soils is the presence of suction, generally associated with the difference between pore-air pressure and pore-water pressure (Fredlund 2006). Suction reflects the interaction between the fluid phases and the soil skeleton and plays an important role in the mechanical behaviour of unsaturated soils. Capillary effects at the air–water interface generate interparticle forces within the soil structure (Fredlund 2006). These interactions contribute to the increase in stiffness and shear strength generally observed during drying processes (Bishop et al. 1960).

The hydro-mechanical behaviour of unsaturated soils is intrinsically coupled because changes in the hydraulic state directly affect the mechanical response of the soil skeleton. In classical consolidation theory, the interaction between stresses and pore-water pressures already leads to coupled hydraulic and mechanical behaviour (Biot 1941; Terzaghi 1936). In unsaturated soils, this coupling becomes more complex because variations in suction and degree of saturation influence both hydraulic transfer processes and mechanical response (Gens 2010). Consequently, deformation processes and hydraulic changes cannot be considered independently.

The introduction of suction as an additional variable led to important developments in unsaturated soil mechanics. Early approaches attempted to extend the effective stress concept

for partially saturated conditions through the use of a single effective stress variable (Bishop et al. 1960). However, other formulations proposed the use of independent stress state variables to describe the behaviour of unsaturated soils (Fredlund and Morgenstern 1977). The main aspects related to these formulations are presented in the following sections.

2.2.2 Two independent stress variables

(Fredlund and Morgenstern 1977) proposed a theoretical framework for unsaturated soils based on independent stress state variables within the context of multiphase continuum mechanics.

The formulation proposed by (Fredlund and Morgenstern 1977) showed that the stress state of an unsaturated soil can be represented using two independent stress variables. Among the possible combinations, the pair composed of net normal stress and matric suction became the most commonly adopted framework in unsaturated soil mechanics (Rahardjo et al. 2019)

Net normal stress is defined as the difference between total stress and pore-air pressure:

$$\sigma - u_a \quad (2.1)$$

where σ is the total stress and u_a is the pore-air pressure. Matric suction is expressed as the difference between pore-air pressure and pore-water pressure:

$$u_a - u_w \quad (2.2)$$

where u_w is the pore-water pressure. In this framework, net stress mainly represents the mechanical loading applied to the soil skeleton, whereas matric suction characterizes the hydraulic state of the material.

The use of two independent stress variables provided an important basis for the interpretation of the coupled hydro-mechanical behaviour of unsaturated soils. Variations in suction modify the soil's hydraulic state and influence stress transfer mechanisms within the soil skeleton. Consequently, changes in suction may significantly affect stiffness, compressibility, shear strength, and deformation behaviour (Gens 2010). At the same time, deformation processes may alter the soil's hydraulic response by changing the void ratio and degree of saturation.

The evolution of suction during drying or wetting processes directly influences the material's mechanical behaviour. As a consequence, hydraulic and mechanical processes in unsaturated soils are intrinsically coupled and cannot be considered independently (Gens 2010).

The framework based on independent stress state variables also enabled the development of constitutive formulations for unsaturated soils (Fredlund et al. 1993). The constitutive relationships commonly used for saturated soils were subsequently extended to unsaturated conditions by incorporating suction effects (Rahardjo et al. 2019). These developments constituted an important extension of classical saturated soil mechanics to partially saturated conditions. Although the framework based on two independent stress variables has been widely adopted in unsaturated soil mechanics, alternative formulations based on single effective stress concepts were also proposed. In these approaches, the contribution of suction to the stress state is incorporated through an effective stress parameter generally denoted by χ . The theoretical aspects related to this formulation are presented in the following section.

2.2.3 Single effective stress approach for unsaturated soils

To consider this three-phase material as an equivalent continuum, single effective stress approach for unsaturated soils encompasses the complexity of micro-mechanics occurring at the pore scale into a macroscopic formulation that models the multi-phase porous medium into an equivalent, single-phase, single-stress state continuum (Khalili et al. 2005).

Clearly, the use of suction, or a modified version of it, is required to define the stress state of the medium accurately (Gens et al. 2006; Nuth and Laloui 2008). In addition, the relative amounts of the pore air and pore water phases play a key role in the mechanical properties of the unsaturated soil. This proportion, usually defined by the degree of saturation, should be considered, directly or indirectly, in the complete description of the hydro-mechanical soil state.

In the sense of Terzaghi's definition (Terzaghi 1936), the effective stress should be such that all the measurable effects of a change in stress, such as compaction, distortion and a change in shearing resistance, are exclusively due to a change in the effective stress. Consequently, the effective stress enters the elastic and elasto-plastic constitutive equations of the soil matrix. In other words, the effective stress formulation in combination with an elasto-plastic framework should be able to tackle stress-strain response and failure of porous materials upon external stress and suction variations.

Because effective stress formulation requires considering soil suction and relative amounts of the pore air and pore water phases, the water retention curve, which relates suction to the degree of saturation, plays an important role in the geomechanical response of soils.

In the development and the use of a hydro-mechanical constitutive model, that aims to describe stress-strain behaviour upon varying suction, the consideration of a generalized effective stress framework offers the advantage that all the constitutive equations are expressed as a function of this unique stress variable. However, the complexity stands in the generalized effective stress formulation, for which the effect of suction must be weighted by a parameter taking into account the relative amount of fluid phases in the pores. Bishop (Bishop 1959) named this parameter, χ , as the effective stress parameter.

Bishop (Bishop 1959) first attempted to extend the concept of effective stress, as defined by Terzaghi (Terzaghi 1936) in saturated conditions, to unsaturated porous media (Eq. (2.3)).

$$\sigma'_{ij} = \sigma_{ij} + \chi s \delta_{ij} \quad (2.3)$$

where s is the suction and χ is the effective stress parameter. σ_{ij} is the total stress tensor (i.e., the stress applied externally) and δ_{ij} is the Kronecker symbol ($=0$ when $i \neq j$ and $=1$ when $i = j$). In this formulation, χ , the effective stress parameter, varies with the degree of saturation, from zero for dry soil to unity for fully saturated conditions. This expression enables a simple transition from partially to fully saturated states, recovering Terzaghi's expression for $S_r = 1$ (i.e. $\chi = 1$). Several authors (Gallipoli et al. 2003, 2018; Khalili and Khabbaz 1998; Öberg and Sällfors 1997; Pham 2022; Zhang and Houston 2024) attempted to identify the relationship between the effective stress parameter and the degree of saturation, based on various criteria, such as the volume change and shear strength (Bishop et al. 1960), the comparison between soil response upon the change in applied external stress and applied suction (Jennings and Burland 1962) or critical state (Vanapalli and Fredlund 2000). Recent studies have highlighted the need to incorporate microscale interactions in the effective stress framework. (Zhang and Lu 2019) discussed that the classical Bishop's χ formulation lacks the capacity to reflect both capillary and adsorptive interactions within the pore network. (Lu and Likos 2006) introduced the concept of suction stress as a more comprehensive representation of interparticle forces, combining van der Waals attraction, double-layer repulsion, cementation, capillary tension, and air–water interfacial effects. These forces were shown to contribute significantly to the effective stress even in dry or high-suction states (Lu et al. 2010; Lu and Griffiths 2004). (Zhang and Lu 2018) also demonstrated that suction stress depends on both capillary water and adsorptive water, with a proposed formulation involving dual components (adsorptive and capillary suction stress), providing a physical interpretation for χ based on water content and pore structure (Zhang and Lu 2020).

Various authors have proposed different equations to relate the χ parameter with the degree of saturation or suction, or a modified version of them, and eventually some micro-structural features. Table 2.1 reports some of those expressions.

Table 2.1 Various expressions of χ parameter as a function of the degree of saturation or suction, and some micro-structural features

Expressions of χ parameter	Reference	Explanation
$\chi = \begin{cases} \left(\frac{s}{s_e}\right)^{-0.55} & \text{if } s > s_e \\ 1 & \text{if } s \leq s_e \end{cases}$	(Khalili and Khabbaz 1998)	s_e : air-entry suction
$\chi = S_r$	(Schrefler 1984)	
$\chi = \frac{S_r - S_{r0}}{1 - S_{r0}}$	(Kim et al. 2010)	S_{r0} : the residual degree of saturation
$\chi = \frac{S_r - S_r^m}{1 - S_r^m} + \frac{1}{n_{smooth}} \ln \left[1 + \exp \left(-n_{smooth} \frac{S_r - S_r^m}{1 - S_r^m} \right) \right]$	(Alonso et al. 2013)	S_r^m : microstructural degree of saturation (*) n_{smooth} defines the degree of smoothing of the function $\chi(S_r)$
$\chi = S_{rM} + (e - e_{wm}) \frac{\delta S_{rM}}{\delta(e - e_{wm})} \Big _{s=cst} \quad \text{with } S_{rM} = \frac{e_w - e_{wm}}{e - e_{wm}}$	(Vaunat and Casini 2017)	e : void ratio $e_w = \frac{V_w}{V_s}$: water ratio $e_{wm} = \frac{V_{wm}}{V_s}$: microstructural water ratio (**)
$\chi = \left(\frac{S_r - S_r^m}{1 - S_r^m} \right)$	(Alonso et al. 2010)	S_r^m : microstructural degree of saturation (*)
$\chi = (S_r)^\alpha$	(Alonso et al. 2010)	α : material parameter

$\chi = \begin{cases} \frac{e_w - e_{wm}}{e - e_{wm}} = \frac{S_r - S_r^m}{1 - S_r^m} & \text{if } S_r > S_r^m \\ 0 & \text{if } S_r \leq S_r^m \end{cases}$	(Tarantino and Tombolato 2005)	S_r^m : microstructural degree of saturation (*)
$\chi = \frac{S_r - S_{r0}}{1 - S_{r0}} + \frac{k^{aw} a_{aw}}{s}$	(Nikooee et al. 2013)	a_{aw} : specific air–water interfacial area (the air–water interfacial area per total volume). k^{aw} : a material parameter related to air–water surface tension
$\chi = (1 - n)x_{sw} + nS_r$	(Gray et al. 2009)	x_{sw} : the fraction of wetted solid surface area n : porosity
$\chi = S_r \left(\frac{\beta_1}{S_r^{\beta_2}} \right)$	(Ghorbani and Kodikara 2024)	β_1, β_2 : material parameters
$\chi = \begin{cases} \frac{s_e + \frac{(s_c - s_e)(s - s_e)}{(s - s_e) + a_e}}{s} & \text{if } s > s_e \\ 1 & \text{if } s \leq s_e \end{cases}$	(Kohgo et al. 1993)	a_e : material parameter s_c : a critical suction s_e : air-entry suction

(*) Microstructural degree of saturation: Degree of saturation required to saturate micro-voids and let macro-voids empty.

(**) Microstructural water ratio: Water ratio required to saturate micro-voids and let macro-voids empty.

(Nikooee et al. 2013) and (Gray et al. 2009) include the effect of surface tension at the air–water interface. However, this information is usually not available from conventional experimental characterization. The expression in Eq.2.4 was originally proposed by (Tarantino and Tombolato 2005) :

$$\chi = \frac{S_r - S_r^m}{1 - S_r^m} \quad (2.4)$$

And subsequently adapted by (Alonso et al. 2013), (Vaunat and Casini 2017) and (Alonso et al. 2010). It includes the pore size distributions as a key factor to calibrate the c parameter. S_r^m is the microstructural degree of saturation that refers to the degree of saturation needed to

fully saturate the micro-porosity while the macro-porosity remains dry. In this concept, the ratio between micro-porosity and total porosity governs the evolution of c with the degree of saturation. Upon a low degree of saturation, the water is essentially stored inside the aggregates and this “intra-aggregate” water does not contribute to the macroscopic stress (i.e., remains negligible), while, upon a higher degree of saturation, water floods the “inter-aggregate” voids which contributes more to the increase of the internal stress (i.e., becomes significant). This approach is particularly well-adapted for the double structure materials with two characteristic sizes of pores (large pores between aggregates and small pores inside aggregates).

Following this idea, the power law of the degree of saturation is expressed in Eq. (2.5).

$$\chi = (S_r)^\alpha \quad (2.5)$$

with an α exponent higher than one, proposed by (Alonso et al. 2010), makes the c parameter increases slowly upon a low degree of saturation, while the increase is faster upon a higher degree of saturation. So, the α exponent is related to the ratio between micro-porosity and total porosity. A high value of α exponent corresponds to a soil that has a large ratio of micro-porosity considering the total porosity, such that the water stored in this micro-porosity does not contribute to the inter-aggregate skeleton stress (i.e., the generalized effective stress). This concept is illustrated in Fig. 2.1 where Eqs. (2.4) and (2.5) are compared.

Recent developments incorporated more advanced features in the formulations of effective stress for unsaturated soils. In the study of (Haeri et al. 2014) they observed that the yield suction in collapsible loess soils increases with net shear stress ratio under anisotropic conditions, highlighting the need to include initial stress state in the χ calibration process. (Zhan et al. 2014) and (Wang et al. 2019) further showed that soil deformation during wetting is governed not only by suction but also by anisotropic net stresses, which influence the soil-water retention behaviour and collapse potential. These results underscore the importance of developing suction-controlled, anisotropic test procedures for accurate modelling of unsaturated soils (Haeri et al. 2017; Zhou et al. 2014).

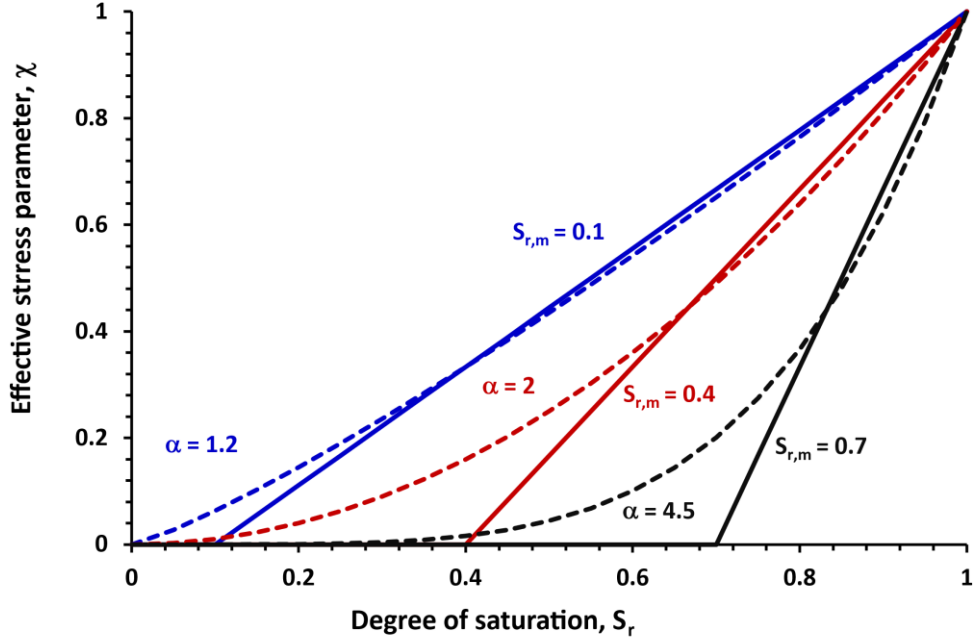


Fig. 2.1 Comprehensive justification of the power law based on microstructural features, as proposed by (Alonso et al. 2010). Full lines and dashed line correspond to Eqs. 2.4 and 2.5, respectively.

Moreover, (Zhang and Lu 2020) proposed a unified suction stress concept that decomposes suction into two components: capillary suction stress and adsorptive suction stress. This framework captures the full range of water–particle interactions, offering a better link between suction and the mechanical response in constitutive models (Lu and Likos 2006; Zhang and Lu 2018). The adsorptive contribution, as they argue, becomes dominant at low water contents and should be explicitly considered in defining χ .

Despite the relative consensus on the applicability of effective stress to unsaturated soils, recent discussions in the literature questioned the (in)definability of effective stress for unsaturated soils. (Zhang and Houston 2024) argued that a single effective stress, strictly analogous to Terzaghi’s, cannot represent all mechanical responses of unsaturated soils, and therefore its use should be restricted to specific stress paths. Their closure to the subsequent discussions (Zhang and Houston 2025) reinforced this position, emphasising the risk of misusing the term “effective stress” in the unsaturated soil context. In contrast, (Khoshghalb and Shahbodagh 2025) and (Morvan and Perić 2025) defended the effective stress framework, showing that with appropriate formulations, it can still capture both shear strength and volume change behaviour. (Rojas 2025) further demonstrated that χ can be defined as a function of the microstructural water distribution, strengthening the need for experimental calibration.

2.2.4 Soil–water characteristic curve and unsaturated soil property functions

The soil–water characteristic curve (SWCC), also called water retention curve (WRC), constitutes one of the fundamental relationships in unsaturated soil mechanics because it describes the relationship between suction and the amount of water retained within the pore structure of the soil (Fredlund 2006; Fredlund and Rahardjo 1993). Since suction directly controls the hydraulic state of the material, the SWCC provides the basis for interpreting several unsaturated soil property functions, including permeability evolution, hydraulic conductivity, stiffness variation, and hydro-mechanical coupling processes.

For compacted clayey soils, the SWCC plays a particularly important role because these materials exhibit strong interactions between hydraulic and mechanical behaviour during drying and wetting processes. Variations in suction influence the degree of saturation, capillary interactions, pore-water distribution, and moisture migration kinetics. Consequently, the SWCC represents a key constitutive relationship for understanding desiccation-induced behaviour in compacted geomaterials.

The general shape of a SWCC is commonly characterized by three main regions: the near-saturated zone, the transition zone, and the residual zone. At low suction values, the soil remains close to saturation and only limited desaturation occurs. Once the air-entry value (AEV) is reached, air progressively enters the largest pores, leading to a significant decrease in water content with increasing suction. At higher suction levels, the soil progressively approaches residual conditions where additional increases in suction produce comparatively small changes in water content. Figure 2.2 shows a typical plot of a soil-water characteristic curve, along with some of its key characteristics.

The shape of the SWCC depends on several factors, including pore-size distribution, mineralogy, density, soil fabric, and compaction conditions. Fine-grained compacted soils generally exhibit higher air-entry values and broader transition zones due to their smaller pore sizes and stronger physico-chemical interactions between water and clay particles. Typical soil-water characteristic curves for different soils are shown in Fig. 2.3.

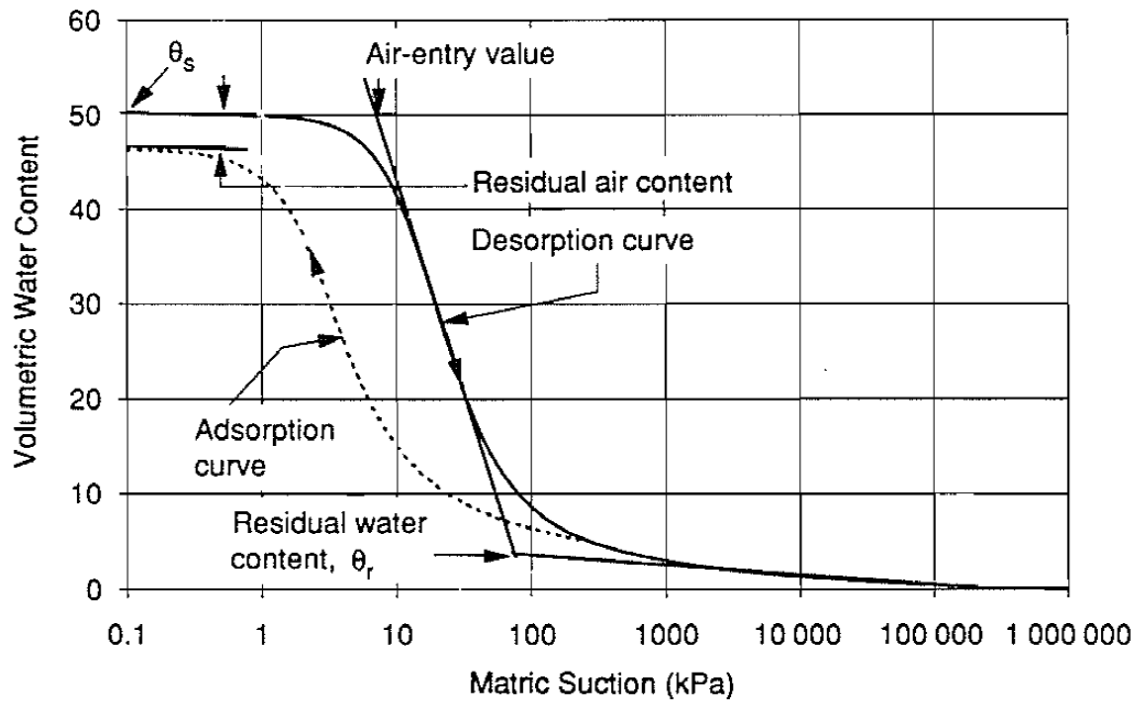


Fig. 2.2 Typical soil-water characteristic curve of soil (Fredlund and Xing 1994).

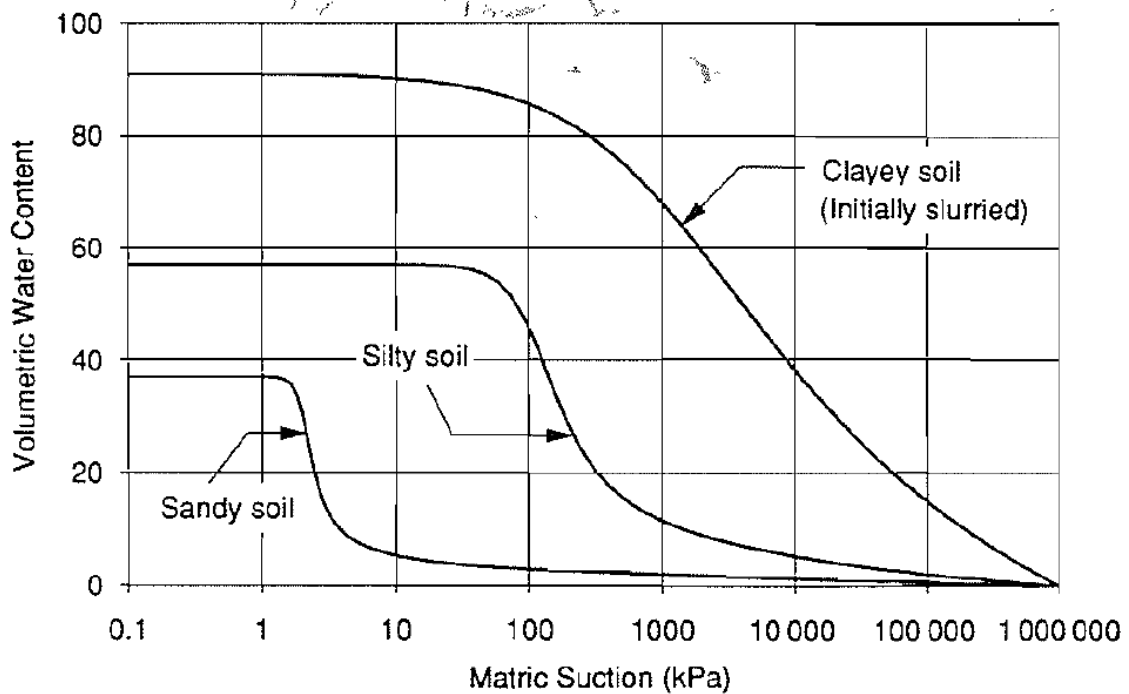


Fig. 2.3 Soil-water characteristic curves for a sandy soil, a silty soil, and a clayey soil (Fredlund and Xing 1994).

The SWCC also exhibits hysteresis between drying and wetting paths because the hydraulic response depends on the hydraulic history of the soil. Drying and wetting curves, therefore, do not coincide due to pore-network effects, contact-angle variations, and air entrapment mechanisms (Fredlund 2006).

Several empirical and semi-empirical equations have been proposed to reproduce the SWCC. Among the most widely used formulations are the models proposed by (Brooks and Corey 1964), (Van Genuchten 1980), and (Fredlund and Xing 1994). These formulations introduce fitting parameters associated with the air-entry value, pore-size distribution, and residual water content behaviour of the soil.

The Brooks and Corey (1964) formulation is commonly expressed as:

$$S_e = \left(\frac{\psi_b}{\psi} \right)^\lambda \quad (2.6)$$

where S_e is the effective degree of saturation, representing the fraction of pore water that is available for capillary flow and hydraulic processes, excluding the residual water that remains strongly adsorbed to the soil particles; ψ_b is the air-entry suction, and λ is the pore-size distribution index.

The Van Genuchten (1980) formulation may be written as:

$$S_e = [1 + (\alpha\psi)^n]^{-m} \quad (2.7)$$

where α , n , and m are fitting parameters controlling the desaturation behaviour of the soil.

Fredlund and Xing (1994) later proposed an alternative equation capable of reproducing the SWCC over a wider suction range, particularly at high suction values:

$$\theta_w = C(\psi) \frac{\theta_s}{\left\{ \ln \left[e + \left(\frac{\psi}{a} \right)^n \right] \right\}^m} \quad (2.8)$$

where a , n , and m are fitting parameters and $C(\psi)$ is a correction factor accounting for high suction conditions.

The SWCC furthermore constitutes the basis for estimating hydraulic conductivity functions in unsaturated soils. As desaturation progresses, the continuity of the liquid phase progressively decreases, resulting in a substantial reduction in hydraulic conductivity. In compacted clayey soils, this reduction strongly influences moisture migration kinetics during drying.

The relative hydraulic conductivity associated with the van Genuchten–Mualem formulation (Van Genuchten 1980; Mualem 1976) may be expressed as:

$$k_r = S_e^{1/2} \left[1 - (1 - S_e^{1/m})^m \right]^2 \quad (2.9)$$

where k_r is the relative hydraulic conductivity and S_e is the effective degree of saturation.

(Fredlund 2006) showed that hydraulic conductivity may decrease by several orders of magnitude during desaturation. As permeability decreases, moisture transfer becomes progressively slower, promoting the development of non-uniform suction fields and localized deformation patterns within compacted soils.

The SWCC, therefore, represents a central constitutive relationship in unsaturated soil mechanics because it governs the hydraulic state of the soil and indirectly controls several hydro-mechanical processes, including permeability evolution, shrinkage behaviour, stiffness changes, and suction redistribution. Consequently, accurate characterization of the SWCC constitutes an essential component in the interpretation and modelling of desiccation-induced behaviour in compacted clayey soils.

2.3 Mechanical Behaviour of Unsaturated Soils

The mechanical behaviour of unsaturated soils is strongly influenced by suction, degree of saturation, hydraulic history, and soil structure. Variations in suction therefore directly influence stiffness, shear strength, compressibility, deformation behaviour, and hydraulic response. In compacted fine-grained soils, these coupled processes become particularly important during drying and wetting cycles. Changes in suction may induce shrinkage, swelling, collapse, tensile stress generation, and cracking, which significantly affect the long-term hydro-mechanical behaviour of engineered earth structures and compacted clay barriers.

2.3.1 Shear strength behaviour

One of the most characteristic mechanical features of unsaturated soils is the increase in shear strength associated with suction. Experimental observations showed that partially saturated soils generally exhibit higher shear resistance compared to saturated soils subjected to similar stress conditions because capillary stresses contribute to interparticle attraction within the soil skeleton (Fredlund and Rahardjo 1993).

The contribution of suction to shear strength may be interpreted using the extended shear strength framework. At relatively low suction levels, the increase in shear strength is generally significant because capillary forces remain effective within the continuous water phase. However, as desaturation progresses, the rate of strength increase becomes nonlinear due to the reduction of hydraulic continuity within the pore network (Vanapalli et al. 1996). This behaviour is schematically illustrated in Fig. 2.4, which shows the upward shift of the shear strength envelope with increasing matric suction.

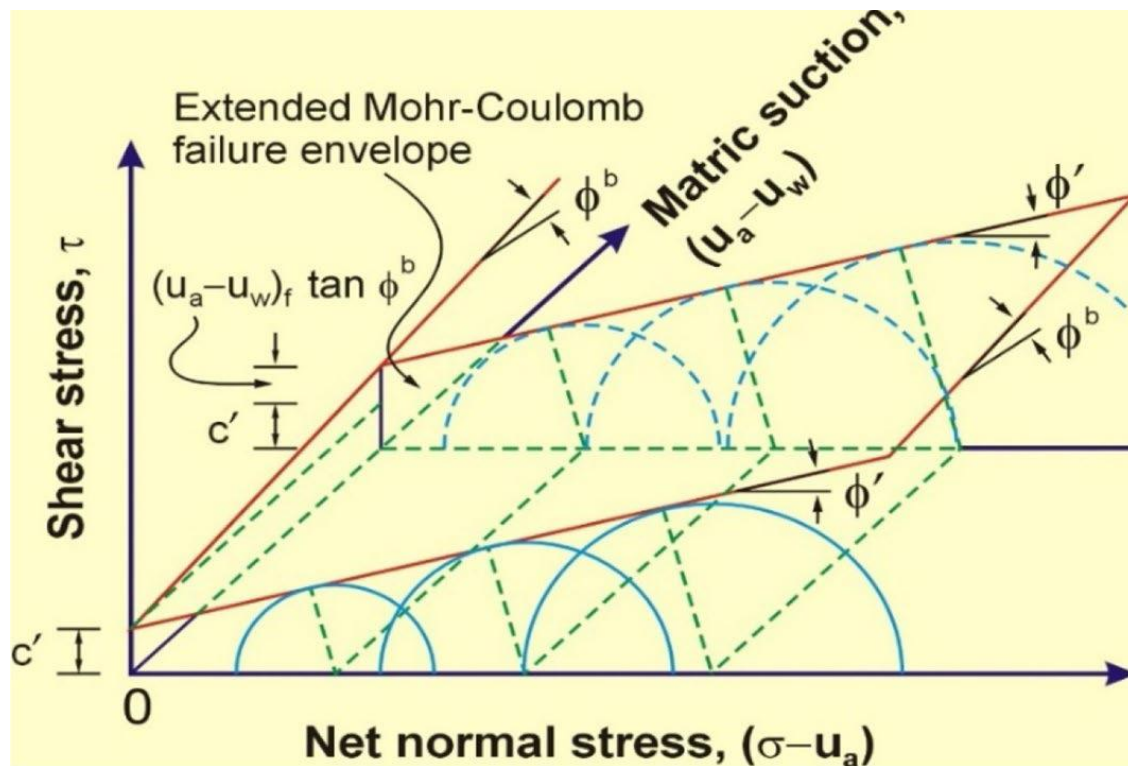


Fig. 2.4 Schematic illustration of shear strength envelopes at different matric suction levels based on the extended Mohr–Coulomb failure criterion (Fredlund et al. 1978).

The evolution of shear strength is closely related to the hydraulic state of the soil and therefore depends on the soil–water characteristic curve previously presented in Fig. 2.3. Fine-grained compacted soils commonly exhibit a more pronounced suction-induced increase in strength because of their higher water retention capacity and smaller pore sizes.

2.3.2 Wetting collapse behaviour

Wetting-induced collapse represents one of the most important hydro-mechanical responses of unsaturated soils. This phenomenon occurs when an initially unsaturated soil subjected to

external loading is wetted, causing a collapse of the soil structure and resulting in irreversible volumetric contraction (Jennings and Knight 1957).

Under partially saturated conditions, capillary forces contribute to the apparent stability and stiffness of the soil structure. During wetting, the progressive reduction of suction weakens these capillary interactions and allows structural rearrangement to occur. As a result, significant collapse strains may develop even without changes in external loading conditions.

The magnitude of collapse depends on several factors, including soil fabric, density, stress state, mineralogy, and hydraulic history. Experimental observations showed that compacted soils prepared dry of optimum are generally more susceptible to collapse because of their aggregated structure and larger inter-aggregate pores.

Numerical and experimental studies have shown that the coupling between suction changes and mechanical deformation plays a fundamental role in the development of collapse behaviour in compacted clay materials. This phenomenon is particularly relevant for compacted clay barriers and engineered earth structures subjected to environmental wetting–drying cycles (Collin et al. 2002).

2.3.3 Behaviour during drying

Drying processes induce progressive desaturation and suction increase within the soil mass, leading to shrinkage deformation and internal stress redistribution. In compacted fine-grained soils, evaporation causes water loss from the pore network and generates capillary stresses that progressively modify the mechanical response of the soil skeleton.

Continued drying induces differential shrinkage and progressive stress redistribution within the soil mass. Under restrained conditions, tensile stresses may develop and eventually lead to crack initiation. The resulting cracks can significantly modify the hydraulic and mechanical behaviour of the material by creating preferential flow paths and altering stress transfer mechanisms. The detailed processes governing tensile stress development, strain localisation, crack initiation, and crack propagation are discussed in Section 2.5.

The hydro-mechanical mechanisms governing drying-induced behaviour are particularly relevant for compacted clay barriers, landfill covers, embankments, and other engineered earth structures exposed to atmospheric drying conditions. These mechanisms constitute a major aspect of the experimental and numerical investigations presented in this thesis.

2.4 Suction Measurement and Control Techniques

The characterization of unsaturated soils requires reliable techniques capable of measuring and controlling suction under laboratory conditions. Over the past decades, numerous techniques have been developed to measure or impose suction over different operational ranges. The following sections present the fundamental components of soil suction, the most commonly used suction measurement techniques, and the principal methods employed for suction control in unsaturated soil mechanics.

2.4.1 Components of soil suction

Soil suction represents one of the fundamental variables governing the hydraulic and mechanical behaviour of unsaturated soils. In partially saturated geomaterials, the coexistence of pore air and pore water within the pore network generates capillary and physico-chemical interactions that strongly influence the stress state and hydro-mechanical response of the soil skeleton (Fredlund and Morgenstern 1977; Fredlund and Rahardjo 1993).

In unsaturated soil mechanics, suction is generally divided into two main components: matric suction and osmotic suction. The combination of these two components defines the total suction of the soil:

$$\psi = (u_a - u_w) + \pi \quad (2.11)$$

where ψ is the total suction, $(u_a - u_w)$ is the matric suction, and π represents the osmotic suction component.

Matric suction corresponds to the pressure difference between pore-air pressure u_a and pore-water pressure u_w . This component originates primarily from capillary effects associated with the air–water interface and from adsorption mechanisms occurring within the soil pore structure. In geotechnical engineering applications, matric suction generally represents the dominant hydraulic variable controlling the mechanical behaviour of compacted soils, including stiffness evolution, shear strength increase, collapse behaviour, shrinkage, and desiccation cracking (Fredlund and Morgenstern 1977; Fredlund and Rahardjo 1993).

Osmotic suction is associated with differences in solute concentration between pore water and pure free water. This component mainly results from dissolved salts and physico-chemical interactions within the pore fluid. Although osmotic suction may become important in highly active clays, chemically contaminated soils, or saline environments, its influence on the

mechanical behaviour of compacted geomaterials is generally less significant than that of matric suction in conventional geotechnical applications (Fredlund and Rahardjo 1993).

The total suction of a soil, therefore, reflects the combined contribution of capillary and osmotic effects. Depending on the experimental technique employed, either matric suction or total suction may be measured or controlled. Techniques such as the axis translation method and osmotic control technique are generally associated with matric suction control, whereas vapour equilibrium methods are commonly related to total suction control through relative humidity regulation (Delage et al. 1998, 2008) .

The relationship between suction and relative humidity is commonly expressed using Kelvin's law, which establishes the thermodynamic link between pore-water potential and vapour equilibrium conditions (Delage et al. 1998).

$$s = \frac{RT}{Mg} \ln RH \quad (2.12)$$

where s is the suction; R is the perfect gas constant ($R=8.3143 \text{ J mol K}^{-1}$); T indicates the absolute temperature ($T=293 \text{ K}$ at 20°C); M stands for the molar mass of water ($M=18.016 \text{ g mol}^{-1}$); g is the gravity acceleration ($g=9.81 \text{ m s}^{-2}$); RH is the relative humidity (Delage et al. 1998).

2.4.2 Suction measurement techniques

Reliable measurement of soil suction constitutes a fundamental requirement for the experimental characterization of unsaturated soils. Because suction directly governs the hydraulic and mechanical behaviour of partially saturated geomaterials, several laboratory and field techniques have been developed to measure suction across different operational ranges and environmental conditions (Fredlund 2006; Fredlund and Rahardjo 1993). However, no single technique is capable of accurately covering the entire suction range encountered in unsaturated soils. Consequently, the selection of a suitable measurement technique depends on the expected suction range, soil type, testing objective, equilibration time, and required measurement accuracy.

Among the most widely used suction measurement techniques are tensiometers, psychrometers, and filter paper methods. These approaches differ in their measurement principles, applicable suction domains, and experimental limitations.

2.4.2.1 High-capacity tensiometers

Tensiometers are direct suction measurement devices used to determine matric suction in unsaturated soils. The technique is based on the measurement of negative pore-water pressure through a saturated high air-entry ceramic interface connected to a pressure transducer (Hilf 1956; Ridley and Burland 1993). Conventional tensiometers are generally limited to relatively low suction values because cavitation occurs when the water pressure becomes sufficiently negative. Recent developments in high-capacity tensiometers have considerably extended the measurable suction range and improved the reliability of direct suction measurements (Delage et al. 2008).

The principal advantage of tensiometers lies in their ability to provide direct matric suction measurements with relatively rapid response times. However, their use remains limited by cavitation risks, saturation requirements, and long-term stability issues associated with air diffusion through the ceramic disk (Delage et al. 2008; Fredlund 2006).

2.4.2.2 Psychrometers and dew-point techniques

Psychrometers and dew-point devices are commonly used for the measurement of high suction ranges in unsaturated soils. These techniques are based on the thermodynamic relationship between relative humidity and total suction established through Kelvin's law. Unlike tensiometers, psychrometers do not directly measure pore-water pressure but instead infer suction from the equilibrium vapour pressure surrounding the soil specimen (Delage et al. 2008; Romero et al. 2001).

Psychrometric methods are particularly suitable for highly compacted clayey soils and expansive materials subjected to high suction conditions where conventional tensiometers become ineffective. High-range psychrometers considerably expanded the accessible suction range in unsaturated soil testing and enabled more accurate characterization of water retention behaviour at elevated suction levels (Fredlund 2006).

Despite their broad measurement range, psychrometers require careful temperature control because relative humidity measurements are highly sensitive to thermal fluctuations. Equilibration time and vapour transfer conditions may also significantly affect measurement accuracy.

2.4.2.3 Filter paper method

The filter paper method is an indirect suction measurement technique widely used for determining the initial suction of soils due to its simplicity, low cost, and applicability over a broad suction range. The method was standardized in (ASTM Designation D 5298-03 2008), which specifies the use of calibrated laboratory-grade filter papers as passive sensors for the determination of matric and total suction.

The technique is based on the equilibration of moisture between the soil specimen and the filter paper under controlled sealed conditions. Depending on the experimental arrangement, either matric suction or total suction may be determined. For matric suction measurements, the filter paper is placed in direct contact with the soil specimen, allowing liquid and vapour exchange between the soil and the paper. For total suction measurements, the filter paper is separated from the soil to prevent direct contact, thereby allowing only vapour transfer to occur (Fondjo et al. 2020).

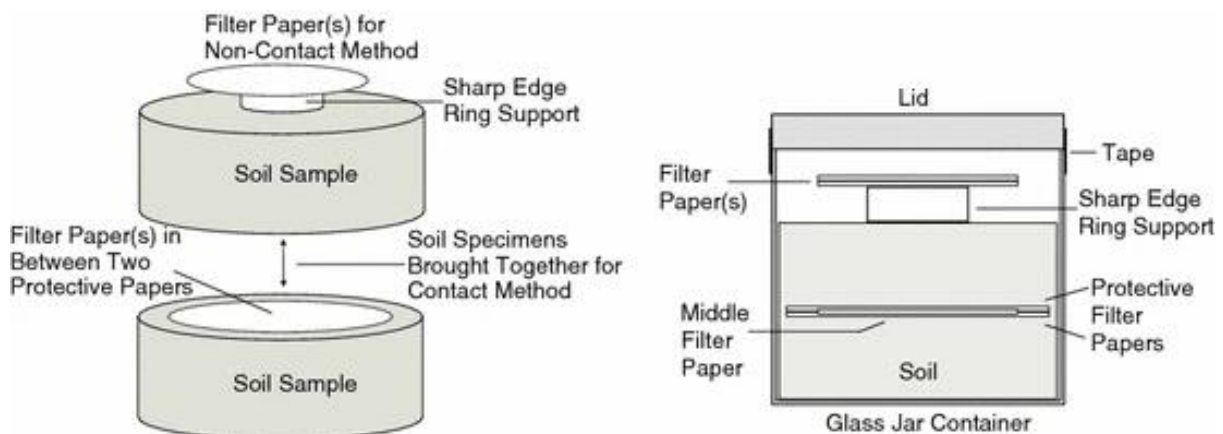


Fig. 2.5 Schematic drawings of soil total and matric suction measurements (Bulut and Leong 2008)

Commonly used filter papers include Whatman No. 42 and Schleicher & Schuell No. 589-WH papers because of their relatively uniform pore structure and the availability of well-established calibration relationships between filter paper water content and suction (Bulut et al. 2001; Bulut and Leong 2008).

In typical laboratory applications, the soil specimen and filter paper are placed inside airtight containers and allowed to equilibrate under constant temperature conditions for several days. After equilibration, the water content of the filter paper is determined gravimetrically and converted into suction using previously established calibration curves. The method allows the

determination of suction over a relatively large range, generally extending from approximately 10 kPa to several hundreds of MPa.

The widespread use of the filter paper method in unsaturated soil mechanics is primarily attributed to its simplicity, relatively low cost, and ability to cover a considerably wider suction range than many direct measurement techniques. Unlike tensiometers, which are generally restricted to relatively low matric suction values, the filter paper method can be applied over a broad range extending from low suction levels to several hundreds of MPa. Furthermore, the method can be used for the determination of both matric and total suction in laboratory as well as field applications, making it one of the most versatile suction measurement techniques available (Fondjo et al. 2020).

Although the filter paper method is widely adopted because of its experimental simplicity and broad suction range, the technique remains sensitive to handling procedures, equilibration conditions, contamination, and temperature variations. Careful specimen preparation and strict control of experimental conditions are therefore essential to obtain reliable suction measurements (ASTM D5298-10; Fondjo et al. 2020).

Among the various suction measurement techniques available in unsaturated soil mechanics, the filter paper method remains particularly suitable for the determination of initial suction conditions in compacted soils before drying or wetting paths. In the present study, this method was used for the determination of the initial suction state of the investigated compacted clayey soils before desiccation testing.

2.4.3 Suction control techniques

Suction control techniques are used to impose a prescribed hydraulic state on unsaturated soil specimens under laboratory conditions. Unlike suction measurement methods, which aim to determine the suction already present in the soil, suction control methods are designed to apply a target suction and allow the specimen to reach hydraulic equilibrium. The choice of technique depends mainly on the suction range, the type of suction to be controlled, the required equilibration time, and the compatibility of the method with the mechanical test being performed.

2.4.3.1 Axis translation technique

The axis translation technique is the most widely used method for controlling matric suction in laboratory testing of unsaturated soils. The method originated from the pressure plate technique

developed by (Richards 1942) and was later adapted for geotechnical applications through the work of (Hilf 1956) and (Bishop et al. 1960). It is based on the artificial increase of pore-air pressure while maintaining hydraulic continuity between the pore water in the soil specimen and an external water reservoir through a saturated high air-entry value (HAEV) interface, typically a ceramic disk. Under these conditions, matric suction is controlled as the difference between pore-air pressure and pore-water pressure (Delage et al. 2008; Hilf 1956; Richards 1942).

A key advantage of the technique is that it allows negative pore-water pressures to be translated into the positive pressure range, thereby enabling their measurement and control without cavitation. This development represented a breakthrough in unsaturated soil testing and facilitated the adaptation of conventional geotechnical devices, including oedometers, triaxial apparatuses, and direct shear systems, for suction-controlled testing (Bishop et al. 1960; Fredlund and Morgenstern 1977; Hilf 1956).

The axis translation technique has been extensively employed to investigate the hydraulic and mechanical behaviour of unsaturated soils, including water retention, volume change, and shear strength characteristics. Owing to its reliability and compatibility with conventional laboratory equipment, it remains the reference technique for controlling low to intermediate matric suctions. Furthermore, studies have demonstrated a good continuity between results obtained using axis translation and those derived from vapour-control techniques over wider suction ranges (Delage et al. 2008).

Despite its widespread use, several practical limitations have been identified. These include air diffusion through the saturated interface, evaporation and condensation effects within the pressure chamber, difficulties associated with air pressurisation at high degrees of saturation, and the potentially long time required to reach hydraulic equilibrium. Nevertheless, when appropriate experimental precautions are adopted, the axis translation technique provides a reliable and well-established framework for suction-controlled laboratory testing (Delage et al. 2008; Tarantino and Tombolato 2005).

2.4.3.2 Osmotic technique

In the osmotic method, the specimen is hermetically enclosed in a semi-permeable membrane that is permeable to water. The specimen and the membrane are immersed in an aqueous solution containing large-sized soluble polyethylene glycol molecules (PEG). Because the PEG

molecules cannot pass through the membrane, an osmotic suction is applied to the specimen through the membrane (Delage et al. 2008). PEG molecules come in a variety of sizes, which are defined by their molecular weight. PEG solutions with molecular weights of 6000 and 20000 g/mol are the most frequently used (Delage et al. 1998). The suction value in the osmotic method is determined by the concentration of the solution; The suction increases as the concentration does.

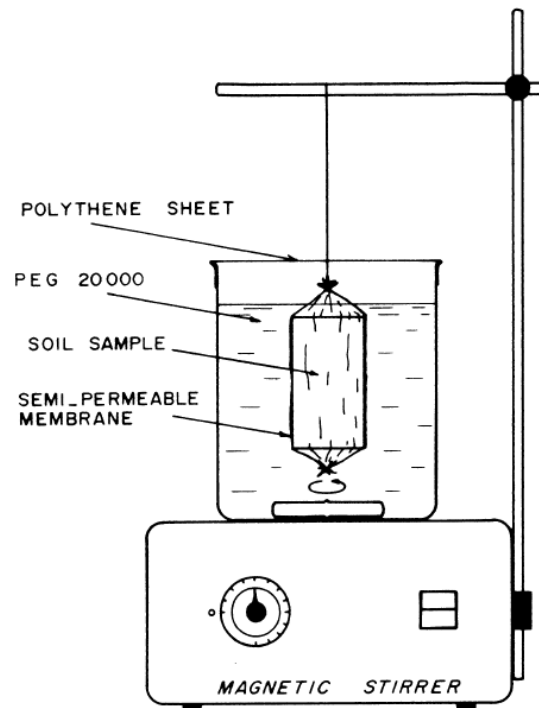


Fig. 2.6 Schematic use of the osmotic technique (Cui and Delage 1996).

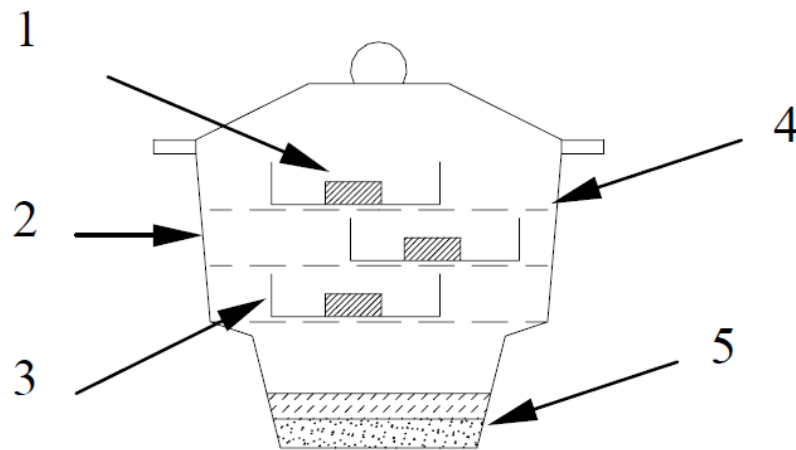
Compared with the axis translation technique, the osmotic method allows higher matric suctions to be imposed without the need for air pressurisation.

While the axis translation technique is generally limited to suction values in the range of several hundreds of kPa, the osmotic technique can conventionally achieve suctions up to approximately 1.5 MPa. Further developments of the method have extended its application to suction levels approaching 10 MPa (Delage et al. 1998).

The technique is particularly suitable for the intermediate suction range, typically between the operating ranges of axis translation and vapour equilibrium techniques. However, its reliability depends on the integrity of the semi-permeable membrane and the stability of the PEG solution, which may influence the imposed suction during long-term testing (Delage et al. 1998, 2008).

2.4.3.3 Vapour equilibrium technique

The vapour equilibrium technique is commonly used to impose high suction values by controlling the relative humidity of the air surrounding the soil specimen. Samples are placed in a sealed desiccator containing a saturated solution of a given salt. A specific relative humidity is imposed inside the desiccator according to the physicochemical properties of the solution. Water exchanges occur between the solution and the soil specimen through vapour transfer until equilibrium is reached. Once the vapour equilibrium is established, the specimen is subjected to a suction corresponding to the imposed relative humidity (Delage et al. 1998). A schematic representation of the vapour equilibrium technique is presented in Fig. 2.7.



- 1 – Soil sample
- 2 – Desiccator
- 3 – Glass cup
- 4 – Support
- 5 – Over saturated salt solution

Fig. 2.7 Schematic representation of the vapour equilibrium technique used for suction control through a saturated salt solution (Tang and Cui 2005).

The relationship between relative humidity and suction is described by Kelvin's law, previously introduced in Eq. (2.12). This relationship provides the theoretical basis for converting the imposed relative humidity into the corresponding suction value. Because very high suctions can be generated under dry atmospheric conditions, the vapour equilibrium technique is particularly suitable for the investigation of compacted clayey soils and for the determination of soil-water retention behaviour over a wide suction range (Delage et al. 1998).

One of the main advantages of the vapour equilibrium technique is its ability to cover suction ranges that are difficult to achieve using axis translation or osmotic techniques. Furthermore, the method is relatively simple to implement and can be applied to a wide variety of soils. However, the establishment of hydraulic equilibrium may require relatively long periods, particularly for low-permeability clayey soils, and strict temperature control is generally required to ensure stable relative humidity conditions (Delage et al. 1998, 2008).

Beyond suction control applications, vapour equilibrium techniques have also been widely used to investigate evaporation processes and soil–atmosphere interactions under controlled laboratory conditions. (Gerard et al. 2019) demonstrated that relative humidity imposed through saturated saline solutions provides a reliable framework for analysing evaporation kinetics and drying behaviour in compacted soils.

Dynamic Vapour Sorption (DVS) is a gravimetric technique that measures the amount of water absorbed or desorbed by a specimen as well as the rate of mass change. The method consists of exposing a specimen to a sequence of controlled relative humidity steps and continuously monitoring its mass until equilibrium is reached at each humidity level. The water retention curve is subsequently established from the equilibrium water contents corresponding to the imposed relative humidities. A schematic representation of a typical DVS apparatus is presented in Fig. 2.8 (Hill et al. 2010).

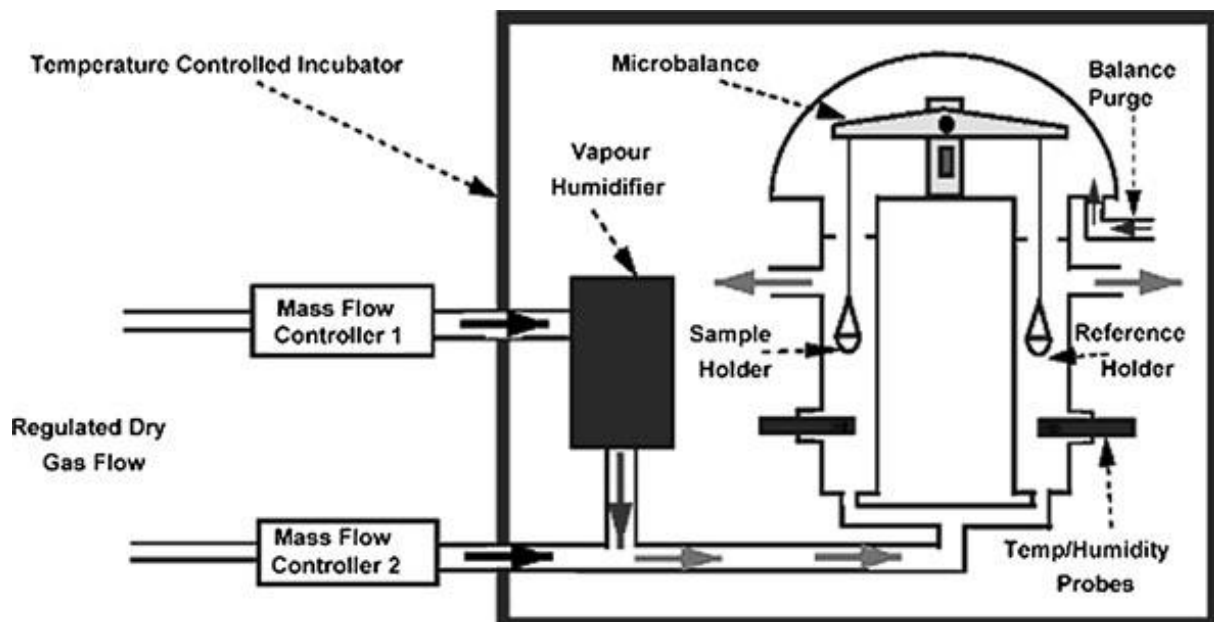


Fig. 2.8 Schematic representation of a Dynamic Vapour Sorption (DVS) apparatus (Hill et al. 2010).

Compared with conventional desiccator-based vapour equilibrium methods, DVS provides automated humidity control and continuous monitoring of specimen mass, allowing a more detailed characterization of adsorption and desorption processes over a broad suction range. Relative humidity is imposed in predefined steps, and the specimen mass is continuously recorded until equilibrium is reached at each stage (Hill et al. 2010). The technique has therefore become increasingly used for the determination of water retention properties of fine-grained soils and compacted clayey materials.

In the present study, suction control for water retention testing was achieved using saturated saline solutions and Dynamic Vapour Sorption (DVS) measurements. The specific experimental procedures and the suction ranges adopted are presented in Chapter 3.

2.5 Desiccation Testing and Cracking Mechanisms

Desiccation cracking has been the subject of extensive experimental research due to its significant implications for the hydro-mechanical behaviour of fine-grained soils. The formation of desiccation cracks may substantially increase hydraulic conductivity, reduce stiffness and strength, and compromise the long-term performance of geotechnical and geo-environmental structures. As a result, numerous experimental and theoretical studies have been devoted to understanding the mechanisms governing drying, shrinkage, tensile stress development, crack initiation, and crack propagation in clayey soils.

Although considerable progress has been achieved over the past decades, desiccation cracking remains a complex phenomenon involving the interaction of hydraulic, mechanical, environmental, and microstructural processes. Previous investigations have approached the problem from different perspectives, including observation of crack patterns, monitoring of shrinkage behaviour, measurement of tensile strength, and numerical modelling of crack initiation and propagation. These studies have progressively improved the understanding of desiccation processes while also highlighting the limitations associated with conventional testing methodologies.

The following sections review the principal experimental approaches used to investigate desiccation cracking, the mechanisms responsible for stress generation and crack initiation, and the development of testing concepts aimed at directly quantifying tensile strength during drying.

2.5.1 Experimental approaches for desiccation cracking

Experimental investigations of desiccation cracking initially focused on the observation of shrinkage and crack formation in drying soil layers. Early studies commonly employed circular, rectangular, or prismatic specimens exposed to controlled or natural drying conditions. The primary objective was generally to characterize crack geometry, crack spacing, crack depth, water loss, and shrinkage behaviour. Seminal investigations demonstrated that desiccation cracking can significantly alter the hydraulic and mechanical properties of soils and established the importance of understanding the conditions leading to crack formation (Konrad and Ayad 1997; Morris et al. 1992). These studies also highlighted the influence of soil type, initial water content, density, and boundary conditions on the resulting crack patterns.

Subsequent experimental programmes introduced more sophisticated monitoring techniques and focused on understanding the mechanisms governing crack initiation rather than solely describing the final crack pattern. Experimental studies performed on compacted clay liners, laboratory soil beds, and restrained soil specimens showed that cracking develops as a consequence of shrinkage restraint during drying and is strongly influenced by suction evolution within the soil mass (Miller et al. 1998; Nahlawi and Kodikara 2006). These investigations demonstrated that desiccation should be considered as a coupled hydro-mechanical process in which evaporation induces suction increase, volumetric contraction, stress redistribution, and ultimately tensile failure.

A major contribution to the understanding of desiccation mechanisms was provided by (Péron 2008), who conducted a comprehensive experimental study involving free and constrained desiccation tests on fine-grained soils. His results showed that the stresses responsible for crack initiation originate primarily from restraining boundary conditions and moisture gradients developing during drying. The study further demonstrated that cracking generally occurs at relatively high degrees of saturation and non-zero suction, highlighting the importance of simultaneously considering hydraulic and mechanical processes during desiccation. Péron also emphasized that crack formation cannot be interpreted solely from the final crack geometry, but must be analysed through the progressive development of stresses and deformations occurring throughout the drying process.

Further advances were achieved through the work of (Lakshmikantha 2009), who developed large-scale and highly instrumented laboratory experiments specifically designed to investigate the evolution of cracking during drying. By combining environmental control, image

acquisition systems, and quantitative image analysis techniques, the study enabled continuous monitoring of crack initiation, crack propagation, crack interaction, and final crack network development. These experiments provided valuable insight into the temporal evolution of desiccation cracking and revealed the existence of size effects in soil cracking behaviour, supporting the application of fracture mechanics concepts to the interpretation of desiccation-induced fracture processes. The work also highlighted the importance of controlled environmental conditions and detailed monitoring for understanding the mechanisms governing crack formation.

In addition to crack pattern characterisation, several studies have focused on the transient evolution of shrinkage during drying. (Prime et al. 2016) investigated drying-induced shrinkage in Boom Clay under controlled laboratory conditions and highlighted the importance of monitoring shrinkage propagation and hydro-mechanical coupling during non-uniform drying. Their observations demonstrated that shrinkage development is a progressive process that evolves before equilibrium conditions are reached.

Recent advances in desiccation cracking research have increasingly relied on non-contact monitoring techniques such as Digital Image Correlation (DIC), Particle Image Velocimetry (PIV), image analysis methods, and X-ray Computed Tomography. These techniques provide detailed information on displacement localisation, strain development, and crack evolution, enabling a more comprehensive investigation of the hydro-mechanical mechanisms governing desiccation cracking. Such approaches have significantly improved the understanding of crack initiation and propagation processes compared with traditional visual observations (El Hajjar et al. 2023; Tang et al. 2021).

Despite these significant developments, most conventional desiccation tests remained primarily focused on monitoring water loss, shrinkage, and crack geometry. Although such approaches provide valuable information regarding the phenomenology of desiccation cracking, they generally do not allow direct quantification of the tensile stresses developing within the soil during drying. Consequently, the relationship between stress development, tensile strength mobilisation, and crack initiation has remained only partially understood. This limitation motivated the development of more advanced experimental methodologies aimed at directly measuring tensile stress evolution during desiccation and establishing a more explicit link between hydraulic changes, mechanical response, and crack initiation.

2.5.2 Restrained shrinkage, tensile stress development and crack initiation

The formation of desiccation cracks is generally recognised as the result of a progressive hydro-mechanical process in which drying induces shrinkage while boundary conditions restrict the free deformation of the soil mass. Although evaporation is the primary driving mechanism, crack initiation itself is fundamentally a mechanical phenomenon resulting from the development of tensile stresses within the soil.

During drying, water progressively evaporates from the soil surface, leading to an increase in suction and a reduction in water content. The increase in suction modifies the stress state within the soil skeleton and induces volumetric contraction. Under completely free shrinkage conditions, the resulting deformation can largely occur without the development of significant tensile stresses. However, in most practical situations, shrinkage is partially or fully restrained by external boundary conditions, internal moisture gradients, or differential deformation within the soil mass. Under such conditions, the tendency of the soil to contract is opposed by the surrounding material or by external constraints, resulting in the progressive accumulation of tensile stresses.

Several authors have identified shrinkage restraint as one of the principal mechanisms responsible for desiccation cracking in fine-grained soils. (Konrad and Ayad 1997) proposed an idealised framework describing the development of tensile stresses during drying, while subsequent experimental studies demonstrated the importance of restraint conditions in controlling crack initiation and propagation. Laboratory observations reported by (Morris et al. 1992a), (Nahlawi and Kodikara 2006), and (Rodríguez et al. 2007) showed that cracking generally occurs when shrinkage-induced tensile stresses exceed the tensile resistance of the material. Similar conclusions were drawn from experimental investigations on compacted clay liners and engineered barrier materials, where desiccation cracking was consistently associated with restrained shrinkage conditions.

The role of hydraulic gradients during drying has also been widely recognised. Evaporation primarily occurs at exposed surfaces, resulting in non-uniform moisture distributions within the specimen. Consequently, differential shrinkage develops between regions experiencing different drying rates. These moisture gradients generate additional internal restraints that contribute to stress redistribution and tensile stress development. As discussed by (Péron 2008), the stresses responsible for crack initiation arise not only from external constraints but also from moisture gradients developing within the drying soil mass. His experimental observations

demonstrated that desiccation cracking is closely associated with the combined effects of suction increase, shrinkage deformation, and restraint-induced stress generation. Furthermore, the study showed that cracking often occurs at relatively high degrees of saturation and non-zero suction, indicating that crack initiation generally takes place before the soil reaches a highly desaturated state.

Once tensile stresses reach a critical value corresponding to the tensile strength of the soil, localised deformation begins to develop, and a crack is initiated. Experimental observations indicate that crack initiation commonly occurs near exposed surfaces where drying is most intense and tensile stresses are highest. Following initiation, stress redistribution takes place within the surrounding material, influencing subsequent crack propagation and the development of the final crack pattern. (Lakshmikantha 2009) demonstrated that crack evolution is a progressive process involving crack initiation, propagation, interaction, and hierarchical subdivision of the soil domain. These observations emphasise that the final crack network is the consequence of a continuous sequence of hydro-mechanical processes rather than a single failure event.

Following crack initiation, crack propagation and interaction progressively govern the evolution of the desiccation pattern. Experimental observations have shown that desiccation cracks rarely develop as isolated features. Instead, primary cracks are commonly followed by secondary and tertiary cracks, leading to the formation of hierarchical crack networks. The final crack pattern therefore, results from a progressive process involving crack initiation, propagation, intersection, and subdivision of soil domains. This hierarchical behaviour has been extensively documented in large-scale desiccation experiments and is now recognised as a characteristic feature of desiccation cracking in fine-grained soils (Lakshmikantha 2009; Tang et al. 2021).

The understanding that desiccation cracking results from the interaction between suction evolution, shrinkage deformation, tensile stress generation, and tensile strength mobilisation has provided the foundation for many experimental and numerical studies. Consequently, accurate characterisation of tensile strength under representative drying conditions has become a central aspect of desiccation research, motivating the development of experimental methodologies capable of directly quantifying tensile stress evolution during drying. This aspect is discussed in the following section.

2.5.3 Direct measurement of tensile strength during desiccation

Since desiccation cracking is initiated when tensile stresses exceed the tensile strength of the soil, reliable determination of tensile strength is essential for understanding and modelling crack initiation. Consequently, numerous laboratory techniques have been developed to evaluate the tensile resistance of geomaterials. These methods can generally be classified into indirect and direct testing approaches.

Indirect techniques estimate tensile strength from alternative loading configurations, such as flexural tests, penetration tests, fracture tests on notched specimens, or Brazilian-type tests. Several researchers have successfully applied such methods to fine-grained soils and compacted geomaterials (Hanson et al. 1994; Kim et al. 2012; Konrad and Ayad 1997). Although these techniques are relatively simple to perform, the tensile stress field is not directly imposed, and the interpretation of the measured strength often relies on simplifying assumptions regarding stress distribution and failure mechanisms. As a result, the measured values may not fully represent the actual tensile conditions experienced during desiccation.

Direct tension tests were subsequently developed to overcome some of these limitations. In these approaches, tensile forces are applied directly to the specimen until failure occurs, allowing a more explicit determination of tensile strength. Examples include the testing methods proposed by (Nahlawi et al. 2004), (Tamrakar et al. 2005), (Rodríguez et al. 2007), and (Vesga 2009). While these techniques provide a more direct assessment of tensile resistance, the tests are generally conducted under nearly constant water content or controlled suction conditions, with tensile loads applied externally until failure is reached. Consequently, the measured strength does not necessarily correspond to the stress path experienced during natural desiccation, where suction evolves continuously, and tensile stresses develop progressively as drying proceeds.

This limitation became increasingly recognised as experimental evidence accumulated regarding the stress-path dependency of unsaturated soils. During desiccation, evaporation induces a continuous increase in suction while shrinkage restraint progressively generates tensile stresses within the soil mass. Therefore, crack initiation results from the simultaneous evolution of hydraulic and mechanical conditions rather than from the application of an externally imposed tensile load at constant moisture conditions. Several authors consequently highlighted the need for experimental methodologies capable of reproducing the actual drying path experienced by the soil prior to cracking.

An advancement in this direction was achieved by (Varsei et al. 2016) who developed a novel experimental apparatus specifically designed to measure tensile strength during desiccation. The device consists of a split mould connected through load cells, allowing direct measurement of tensile forces generated by restrained shrinkage while simultaneously monitoring moisture loss and crack formation. By carefully controlling the restraint conditions, the desiccation problem was reduced to an approximately one-dimensional configuration in which a nearly uniform tensile stress field develops within the specimen. This approach enabled direct quantification of tensile force at the moment of crack initiation and represented a major departure from conventional boundary-value desiccation tests, where stress fields are generally unknown and tensile strength must be inferred indirectly.

The experimental setup proposed by (Varsei et al. 2016) is presented in Fig. 2.9. The apparatus combines a desiccation box, load cells, a digital imaging system, and continuous monitoring of specimen mass, thereby enabling simultaneous observation of drying, stress development, and crack formation under controlled laboratory conditions.

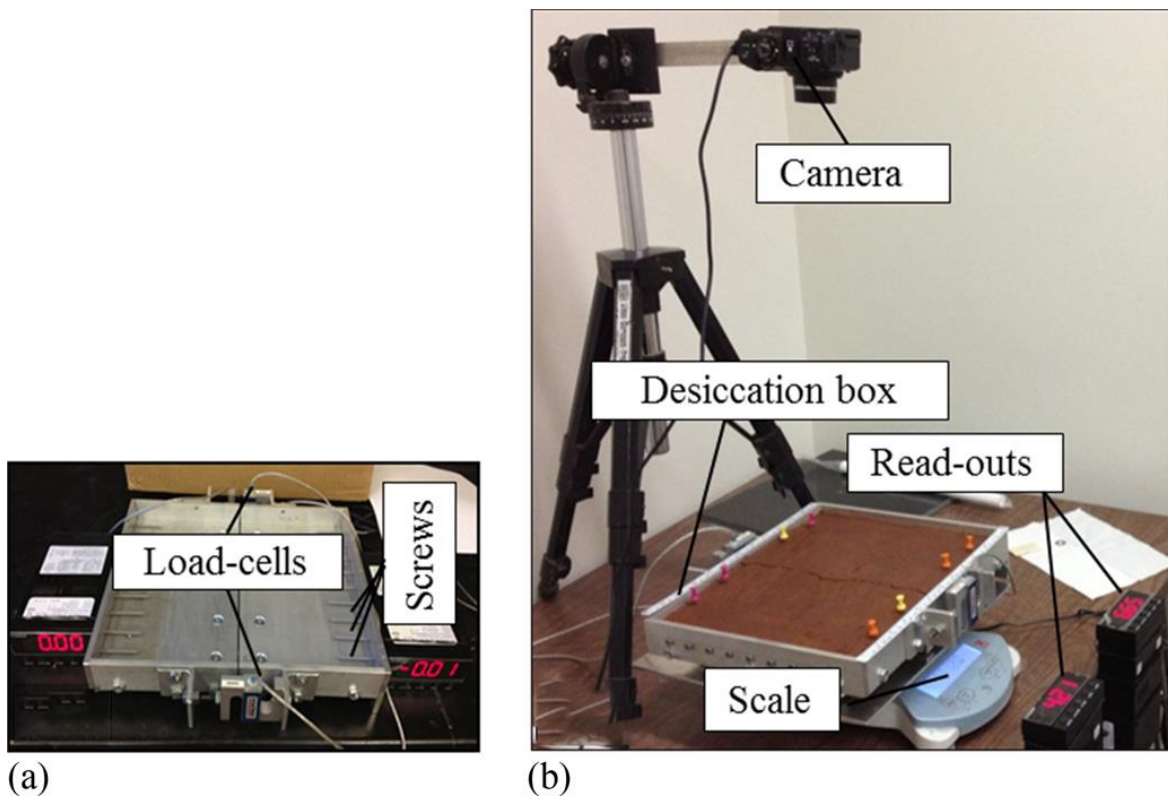


Fig. 2.9 Experimental apparatus proposed by (Varsei et al. 2016) for direct measurement of tensile forces generated during restrained desiccation of compacted soils: (a) desiccation box equipped with load cells; (b) complete experimental setup including the imaging system, weighing scale, and data acquisition units.

The significance of this methodology lies in its ability to characterise tensile strength along the actual desiccation path. Instead of imposing failure through an external loading system, tensile stresses are generated naturally by restrained shrinkage, reproducing the physical mechanisms responsible for crack initiation in the field. The measured tensile strength reflects the coupled influence of suction evolution, water loss, soil structure, and stress development occurring during drying. This concept provided a new framework for investigating desiccation-induced failure and established a direct link between tensile strength mobilisation and crack initiation.

(Salimi et al. 2021) further extended this experimental philosophy by investigating the evolution of tensile strength during desiccation under different environmental conditions. Their results confirmed the importance of measuring tensile strength under representative drying conditions and demonstrated that the tensile resistance of compacted clayey soils evolves continuously throughout the desiccation process. These observations reinforced the limitations of conventional constant-suction testing approaches and highlighted the value of direct monitoring techniques capable of capturing the coupled hydro-mechanical processes governing crack initiation.

2.5.4 Positioning of the present research

The literature reviewed in the previous sections demonstrates that significant progress has been achieved in understanding the mechanisms governing desiccation cracking in fine-grained soils. Experimental investigations have improved the understanding of shrinkage behaviour, crack initiation, crack propagation, and the influence of environmental and boundary conditions on desiccation processes. In parallel, advances in tensile strength testing and constitutive modelling have provided valuable insights into the relationship between suction evolution, stress development, and cracking.

Nevertheless, several limitations remain. Many conventional desiccation studies primarily focus on crack pattern observations or global shrinkage measurements without direct quantification of the tensile stresses responsible for crack initiation. Conversely, most tensile strength tests are performed under controlled or constant suction conditions and therefore do not fully reproduce the actual hydro-mechanical stress path experienced during desiccation. Although the experimental concepts proposed by (Varsei et al. 2016) and further developed by (Salimi et al. 2021) represent important advances towards direct measurement of tensile strength during drying, relatively limited information remains available regarding the

simultaneous evolution of tensile stresses, displacement fields, and strain localisation preceding crack formation.

The present research builds upon these previous developments and adopts the general experimental philosophy introduced by (Varsei et al. 2016), in which tensile stresses are generated naturally by restrained shrinkage during drying. However, the scope of the investigation extends beyond the direct measurement of tensile force. Particular emphasis is placed on the combined analysis of mechanical response and full-field kinematics through the integration of tensile force measurements and image-based displacement monitoring. This approach enables the identification of deformation patterns and strain localisation processes occurring before crack initiation.

Furthermore, the experimental observations are interpreted within a broader hydro-mechanical framework that combines desiccation testing, effective stress characterisation, and numerical modelling. By establishing links between suction evolution, tensile stress development, deformation localisation, and crack initiation, this integrated approach provides a more comprehensive framework for investigating desiccation mechanisms in compacted clayey soils and addresses several limitations identified in previous experimental studies.

2.6 Conclusion

This chapter reviewed the fundamental principles governing the behaviour of unsaturated soils, with particular emphasis on soil suction, water retention behaviour, and effective stress concepts. The theoretical framework associated with the generalized effective stress approach and the effective stress parameter χ was presented, highlighting its relevance for the interpretation of the hydro-mechanical behaviour of compacted clayey soils.

The chapter also examined the mechanisms associated with drying-induced shrinkage and desiccation cracking. Particular attention was given to the development of tensile stresses resulting from restrained shrinkage, the initiation and evolution of cracks, and the principal experimental methodologies used to investigate these phenomena. Recent advances in monitoring techniques and direct tensile strength measurements were also reviewed.

The literature survey highlighted the strong coupling between suction evolution, deformation processes, tensile stress development, and crack initiation during desiccation. Furthermore, it demonstrated the importance of experimental approaches that are capable of reproducing

realistic drying conditions while simultaneously monitoring the mechanical and kinematic response of the soil.

The concepts and experimental developments reviewed in this chapter provide the theoretical and methodological basis for the experimental investigations presented in the following chapters. In particular, they support the interpretation of the hydro-mechanical behaviour, effective stress characterisation, tensile stress development, and desiccation-induced deformation processes investigated throughout this research.

3. MATERIALS AND EXPERIMENTAL CHARACTERISATION

This chapter presents the methodological framework and the experimental advancements accomplished and the experimental results obtained during the course of this research. The study begins with a comprehensive geotechnical description and characterisation of the material, followed by a detailed explanation of the experimental procedures and analytical methods used to investigate its mechanical and hydraulic properties.

3.1 Standard Characterization of the Material

Three types of soil are studied in this research. They are called “Soignies”, “Kruibeke”, and “Tournai”, corresponding to the locations where they were extracted in Belgium. These three soils exhibit different characteristics to cover a range of soil behaviour from silt to clay. For the standard characterization of the materials, particle size analysis, Atterberg limits, specific gravity measurement, and the standard Proctor tests were performed.

3.1.1 Grain size distribution

Grain size distribution curves of the three soils were obtained from sieve analysis in accordance with (ASTM D6913 2017), and sedimentation (hydrometer) analysis in accordance with (ASTM D7928 2016). According to the particle size distribution curves (Fig. 3.1), a large portion of the grains has a diameter smaller than 80 μm . This fraction represents 85.6% for the Soignies soil, 83.7% for the Kruibeke soil, and 88.5% for the Tournai soil. Their grain size distribution is relatively similar, but they exhibit different geomechanical properties due to different mineralogical compositions. Table 3.1 reports the mineralogical composition of the tested soils, focusing exclusively on the clay minerals. In particular, the Tournai soil is very plastic ($I_p = 98.3 \%$) because of the presence of the illite-smectite mixed layer.

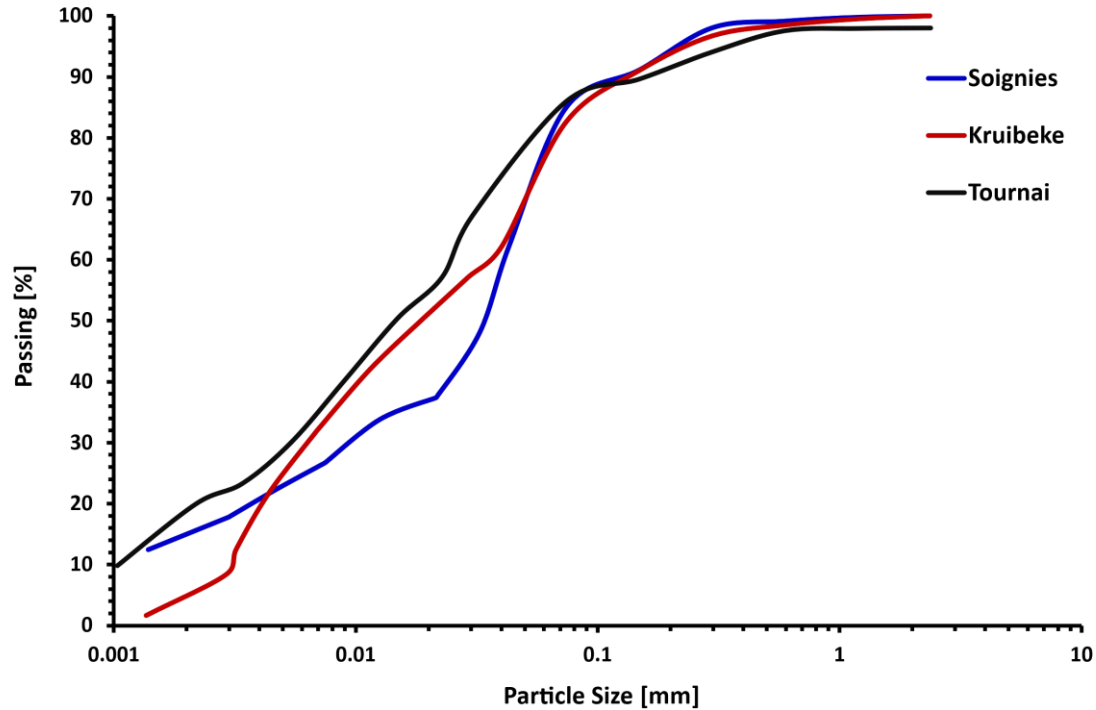


Fig. 3.1 Particle size distribution curve for the three tested soils

Table 3.1 Mineralogical composition of the tested soils, focusing on clay minerals (Schmitz et al. 2004)

Soil	Illite (%)	Illite- chlorite mixed layer (%)	Illite- smectite mixed layer (%)	Smectite (%)	Kaolinite (%)	Chlorite (%)	Total clay mineral (%)
<i>Soignies</i>	17	8	8	3	10	5	52
<i>Kruibeke</i>	17	11	10	3	8	12	61
<i>Tournai</i>	10	5	44	6	-	-	65

3.1.2 Atterberg limits

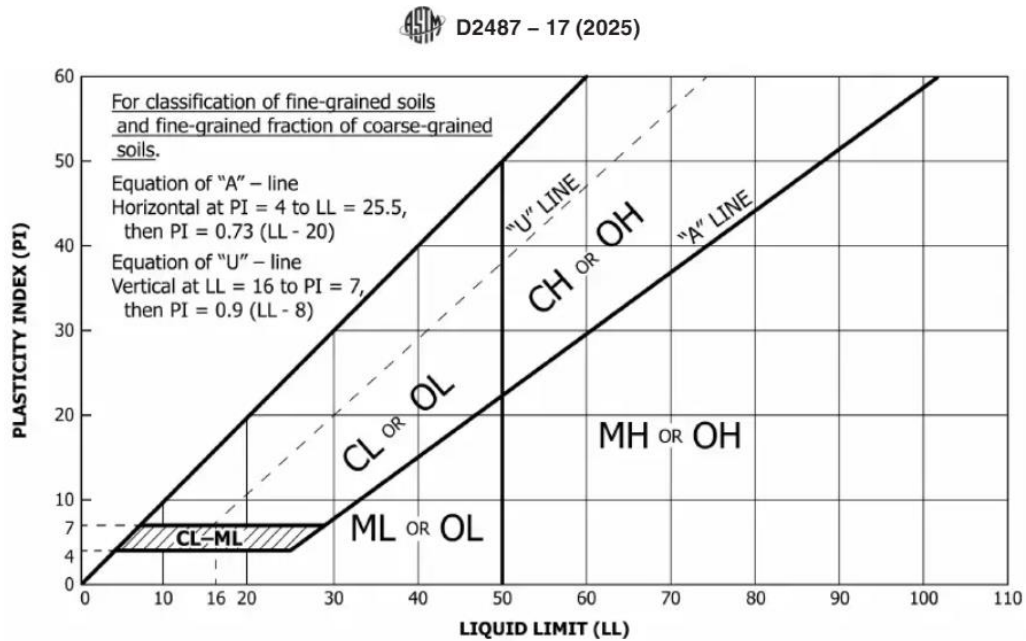
The Atterberg limits of the tested material were determined in accordance with the (ASTM. D4318 1998) standard. Table 3.2 summarizes the obtained values of the liquid limit and the plastic limit. The plasticity index, calculated directly from these parameters, serves as an important criterion for classifying fine-grained soils within the framework of the Unified Soil Classification System (USCS) (Casagrande 1948; Howard 1977).

Table 3.2 Atterberg limit of the studied materials

Soil	<i>Soignies</i>	<i>Kruikeke</i>	<i>Tournai</i>
Liquid Limit (%)	36.4	56	141.1
Plastic Limit (%)	18.6	29.7	42.76
Shrinkage Limit (%)	9	14.4	27
Plasticity Index (%)	17.8	26.3	98.3

3.1.3 USCS classification

According to the Unified Soil Classification System (USCS) (ASTM D2487 2025) (Fig. 3.2), the soil from Soignies corresponds to low plasticity clay (CL), while the soils from Kruikeke and Tournai correspond to high plasticity clay (CH).

**Fig. 3.2** Plasticity chart (ASTM D2487 2025)

3.1.4 Specific gravity

The specific gravity of the soil particles was determined using a helium gas pycnometer in accordance with (ASTM D5550–14 2014). The corresponding results are presented in Table 3.3.

Table 3.3 Specific gravity of the studied materials

Soil	<i>Soignies</i>	<i>Kruikeke</i>	<i>Tournai</i>
Specific Gravity, G_s (g/cm ³)	2.63	2.81	2.52

3.1.5 Standard proctor test

Standard proctor tests were performed on the three soils, according to the (ASTM D698-12 2021) standard. Figure 3.3 presents the soil sample for the standard proctor test. The results of the tests are presented in Fig. 3.4. Table 3.4 summarizes the obtained results in terms of optimum water content and maximum dry unit weight.

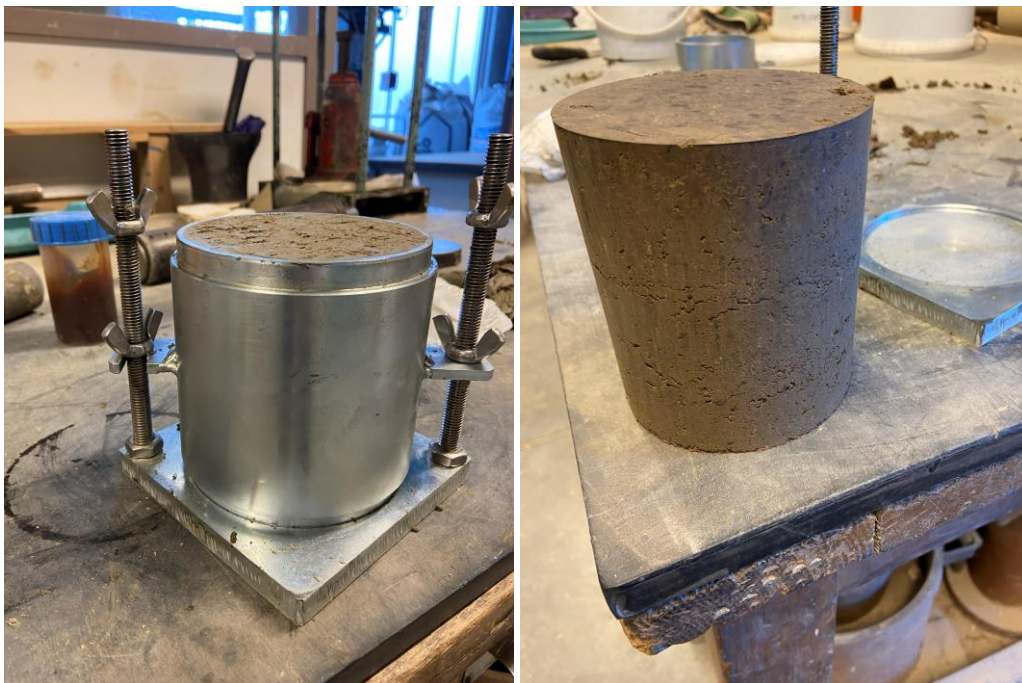
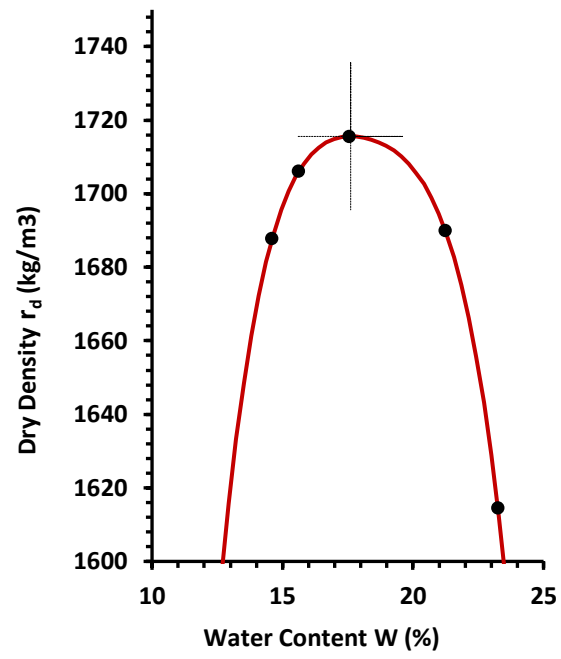
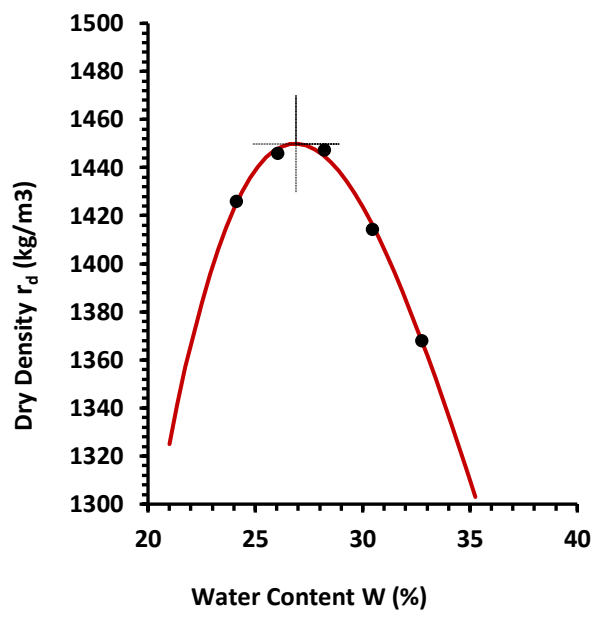


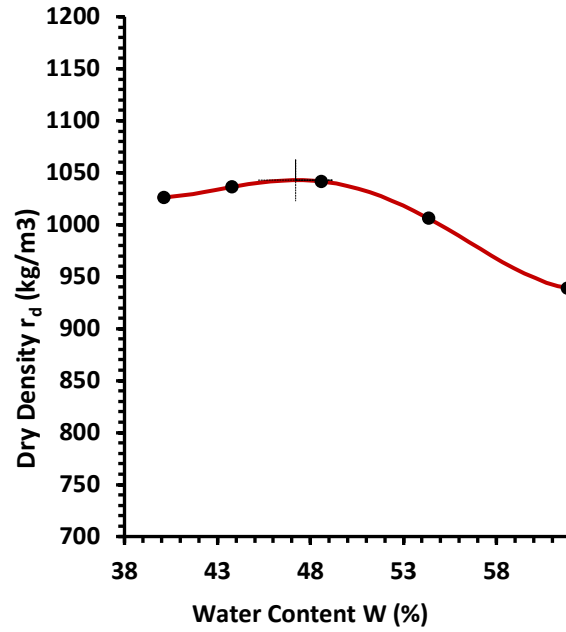
Fig. 3.3 (a) molded and (b) unmolded soil sample for standard proctor test



(a)



(b)



(c)

Fig. 3.4 Standard Proctor Test Results for three tested soils (a) Soignies, (b) Kruibeke, (c) Tournai

Table 3.4 Standard Proctor Test Results

Soil	<i>Soignies</i>	<i>Kruibeke</i>	<i>Tournai</i>
Maximum dry Unit Weight under standard proctor compaction (g/cm³)	1.72	1.45	1.04
Optimum Water Content under standard proctor compaction (%)	18	27	47.6

3.2 Sample Preparation

All the samples of this study are prepared by static compaction under optimum standard proctor conditions, in terms of water content and dry density. First, the soil is mixed with the amount of water required to achieve the optimum water content. Then, the wet soil is left to rest for at least 24 hours in a plastic bucket to ensure that the water is evenly distributed throughout the soil. To prevent any moisture loss, a plastic film is placed between the lid and the bucket. To further minimize potential water loss, the bucket is inverted to rest on its lid. Finally, the samples are prepared by compacting the soil into a cylindrical mold (see the dimensions in Table 3.5). The mass of soil inserted in the mold is adjusted to correspond to the optimum standard Proctor density. All the samples are compacted from top to bottom. Specimens have

been compacted statically in three layers (except for the small specimen, used for water retention curve, in one layer) using a hydraulic jack.

Table 3.5 Dimensions of the samples for each test (WRC= Water retention curve, UCS= Uniaxial compression test, Triax CU = Consolidated undrained triaxial test)

Test	WRC	UCS	Triax CU
Height (mm)	7.5	50	99
Diameter (mm)	33.2	25	51

3.3 Initial Suction

The initial suction of the investigated soils was determined using the filter paper method in accordance with (ASTM D5298-10 2010) . Schleicher & Schuell No. 589-WH filter papers were used together with the calibration relationship proposed by (Bulut et al. 2001). Depending on the experimental configuration, both matric suction and total suction measurements were performed. For matric suction measurements, the filter paper was placed in direct contact with the soil specimen, whereas for total suction measurements the filter paper was separated from the soil to allow only vapour transfer.

The soil specimens and filter papers were sealed inside airtight containers and maintained under controlled temperature conditions until equilibrium was reached. After equilibration, the water content of the filter paper was determined gravimetrically and converted into suction using the adopted calibration curve.

Table 3.6 summarizes the initial matric and total suction of the three soils under as-compacted conditions.

Table 3.6 Initial matric and degree of saturation of the soils under as-compacted conditions

Soil	<i>Soignies</i>	<i>Kruibeke</i>	<i>Tournai</i>
Initial Degree of Saturation under as-compacted state (%)	87	81	85
Matric suction under as-compacted state (*) (MPa)	0.785	1.67	1.063
Total suction under as-compacted state (*) (MPa)	1.291	2.104	1.351

3.4 Water Retention Capacity

To establish the water retention curve, it is necessary to control the drying process. For this purpose, suction control tests are performed. Two techniques of suction control are applied. For suction values between 4 MPa and 110 MPa, the saline-solution method was used to impose various relative humidities, leading to different suction levels. Furthermore, the Dynamic Vapour sorption (DVS) method was applied for suction ranges from 14 MPa to 670 MPa.

3.4.1 Saline-solution method

Two specimens of each soil were prepared for the suction values of 4.2, 14.5, 22.4, 38.7, 63.7, and 110 MPa, for a total of 36 samples (12 of each soil). Figure 3.5 illustrates the experimental setup in which soil samples were placed inside different chambers corresponding to the varied relative humidities. In addition, Fig. 3.6 depicts monitoring of the relative humidity during saline-solution suction control tests. The Soil samples are extracted from the chambers to be weighed and measured after reaching the equilibrium and target values for the relative humidities. Fig 3.7 represents the evolution of relative humidity over time, and Table 3.7 shows the target relative humidity and suction values for different chambers in the vicinity of various salts. The suction value of 110 MPa corresponds to the soil samples placed directly in the laboratory environment without putting them in the vicinity of any salt.

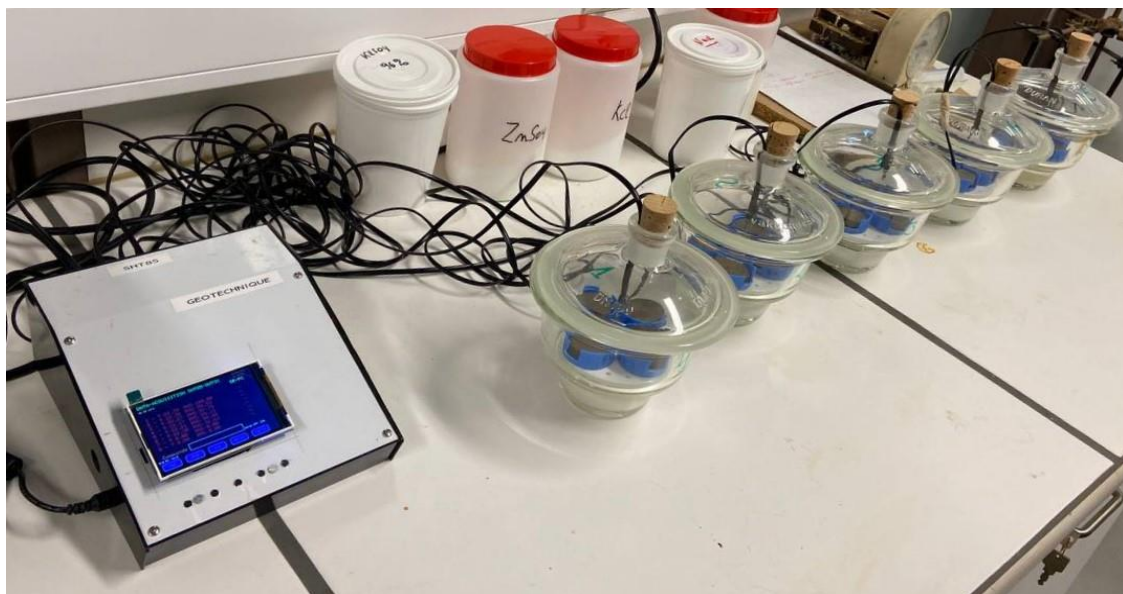


Fig. 3.5 Experimental setup of the saline-solution suction control technique.

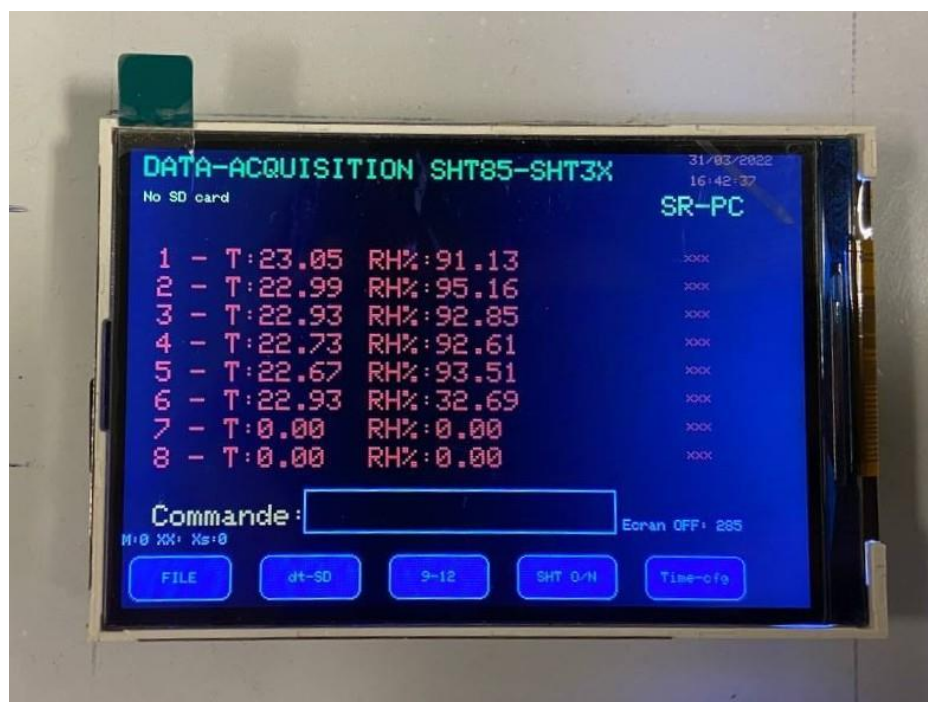


Fig. 3.6 Monitoring evolution of relative humidity and temperature inside the chambers during the saline-solution suction control test.

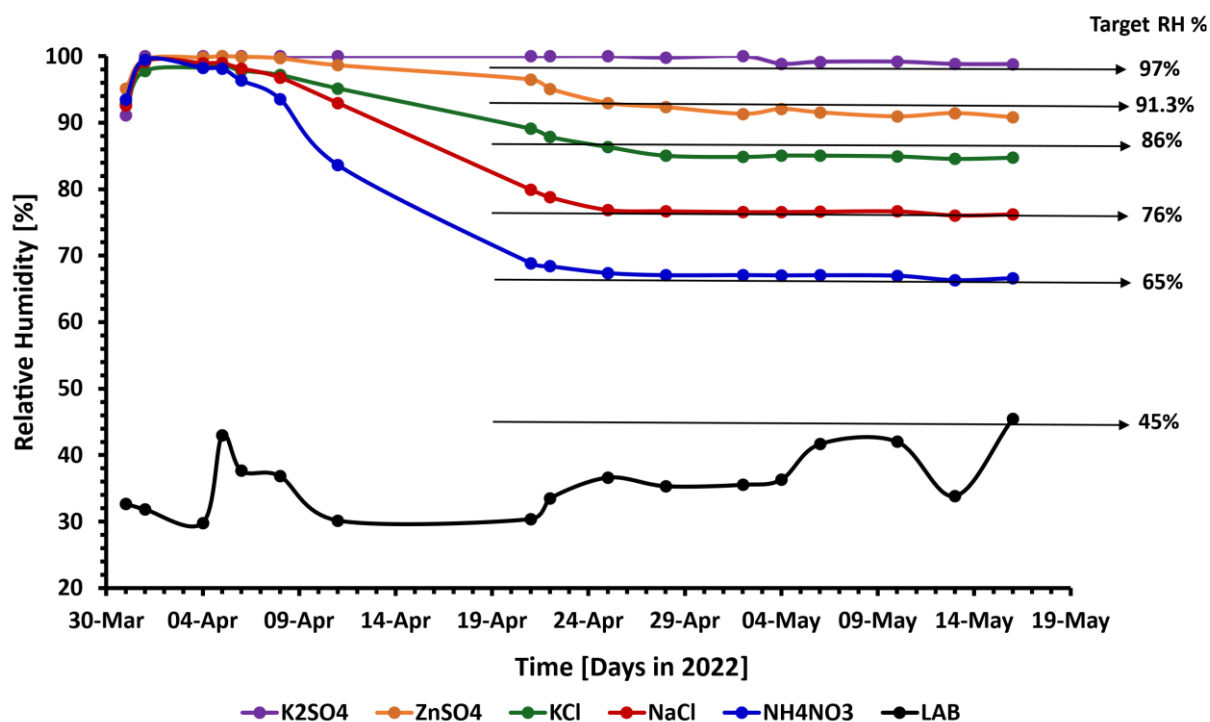
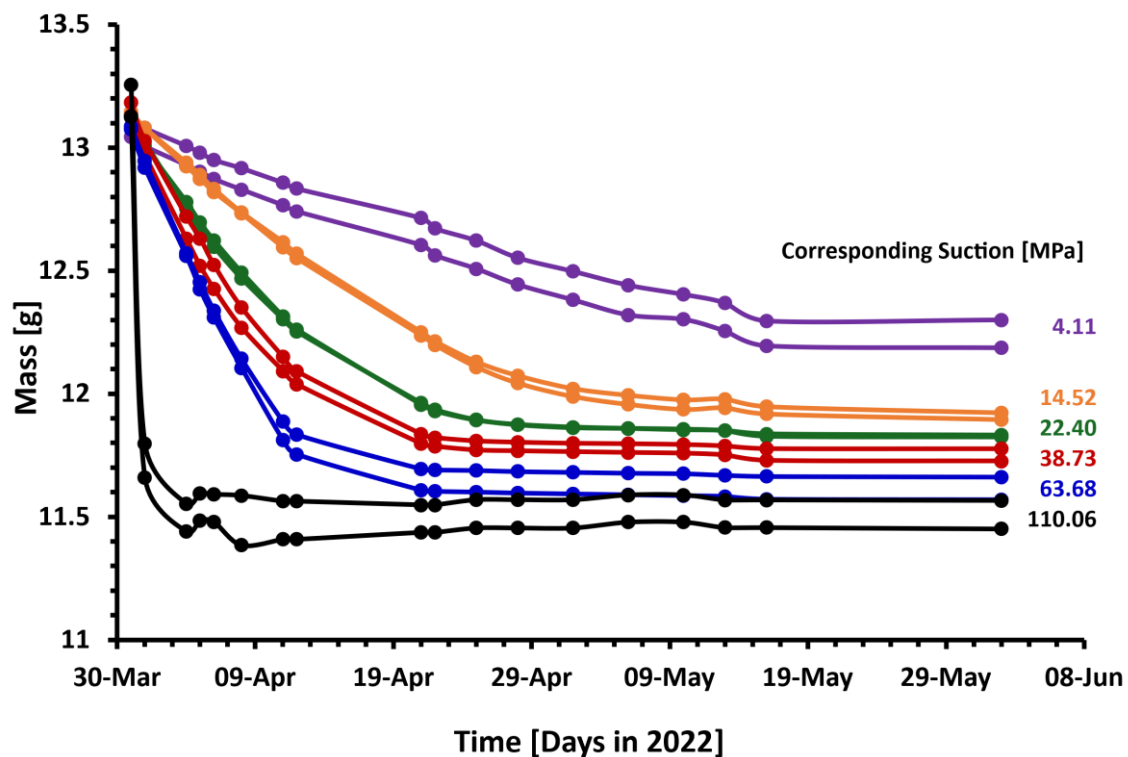


Fig. 3.7 Evolution of relative humidity during the saline-solution suction control test.

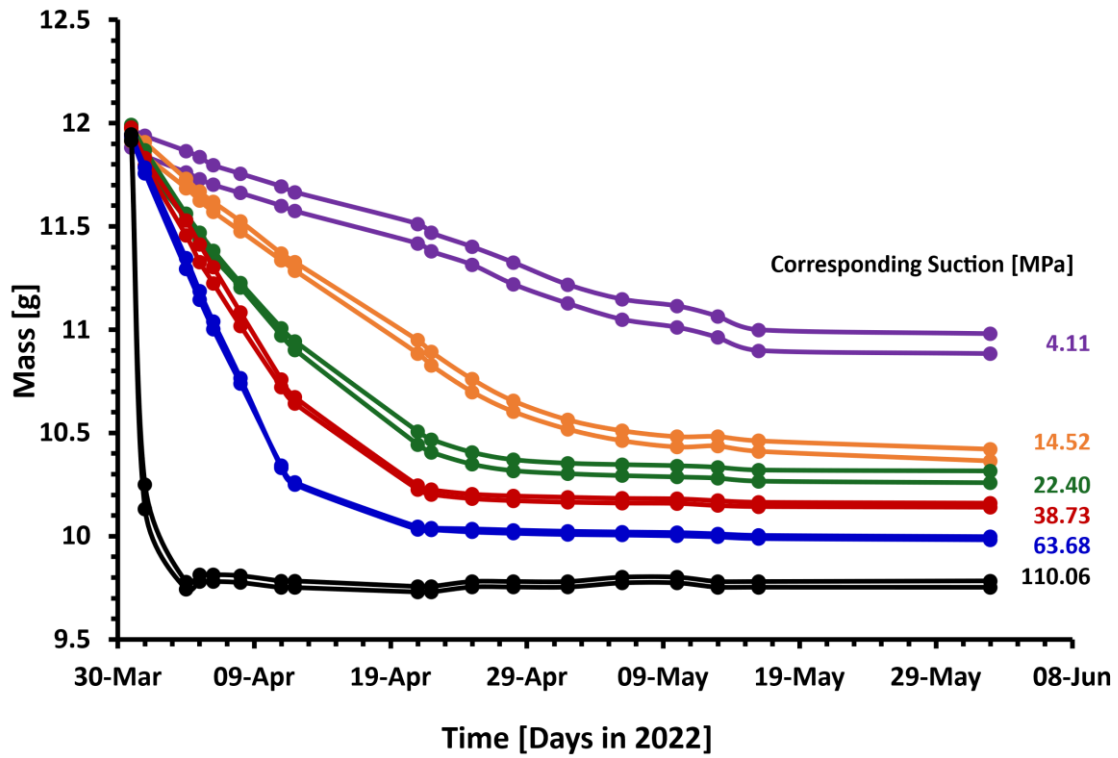
Table 3.7 Imposed relative humidity and equivalent suction for the saline-solution method

Solution	Target RH %	Target suction (MPa)
K ₂ SO ₄	97	4.11
ZnSO ₄	91.3	14.52
KCl	86	22.4
NaCl	76	38.73
NH ₄ NO ₃	65	63.68
LAB	45	110.06

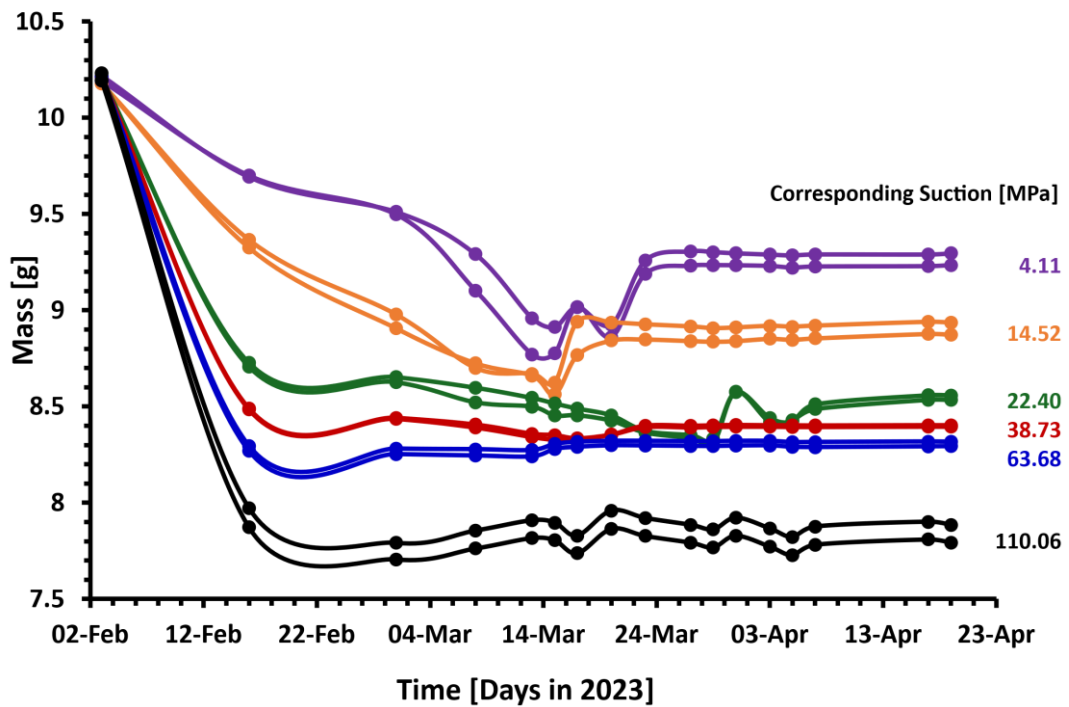
The evolution of mass over time for the three tested soils is presented in Fig. 3.8. In Fig. 3.8(c), the weight of soil samples of Tournai soil first decreased and then rose slightly again. This happened because, at some point, the condition is adjusted by adding more salt and removing a bit of water to help the samples reach equilibrium at the intended relative humidity. That adjustment caused the weight to drop and then stabilize again as the samples found their new balance. The degree of saturation–time relationship obtained from the saline-solution tests is presented in Fig. 3.9.



(a)

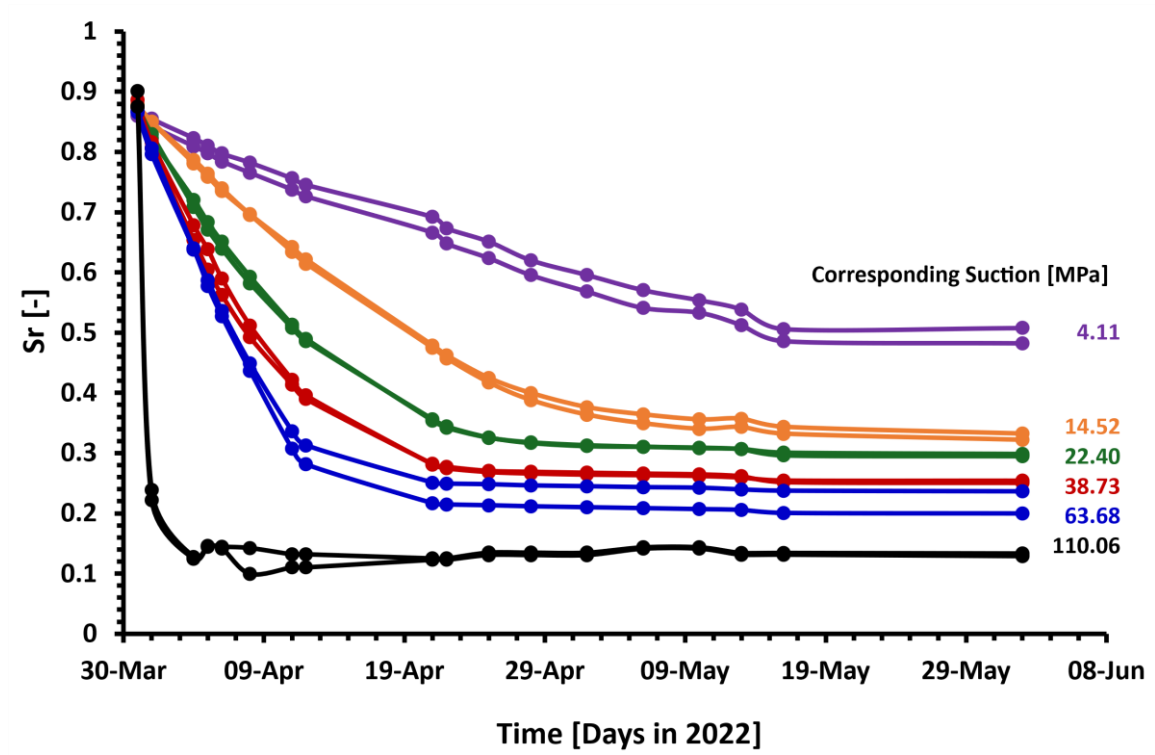


(b)

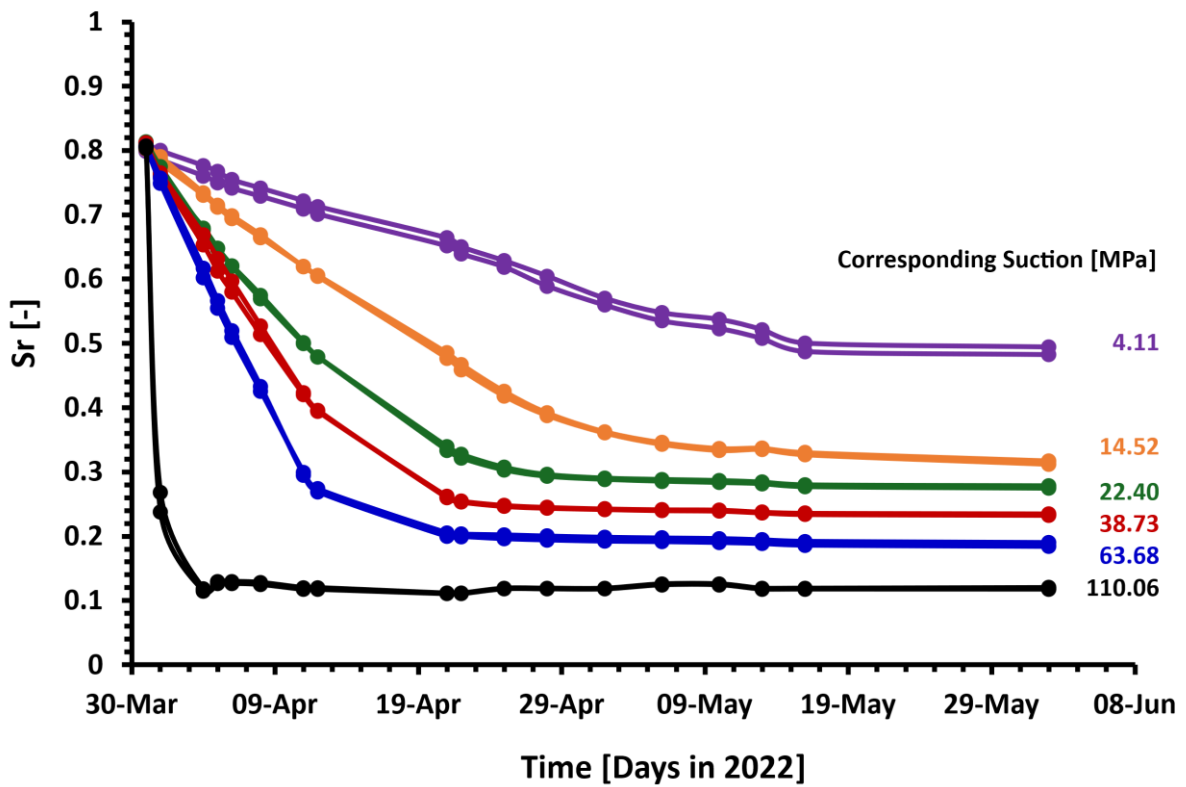


(c)

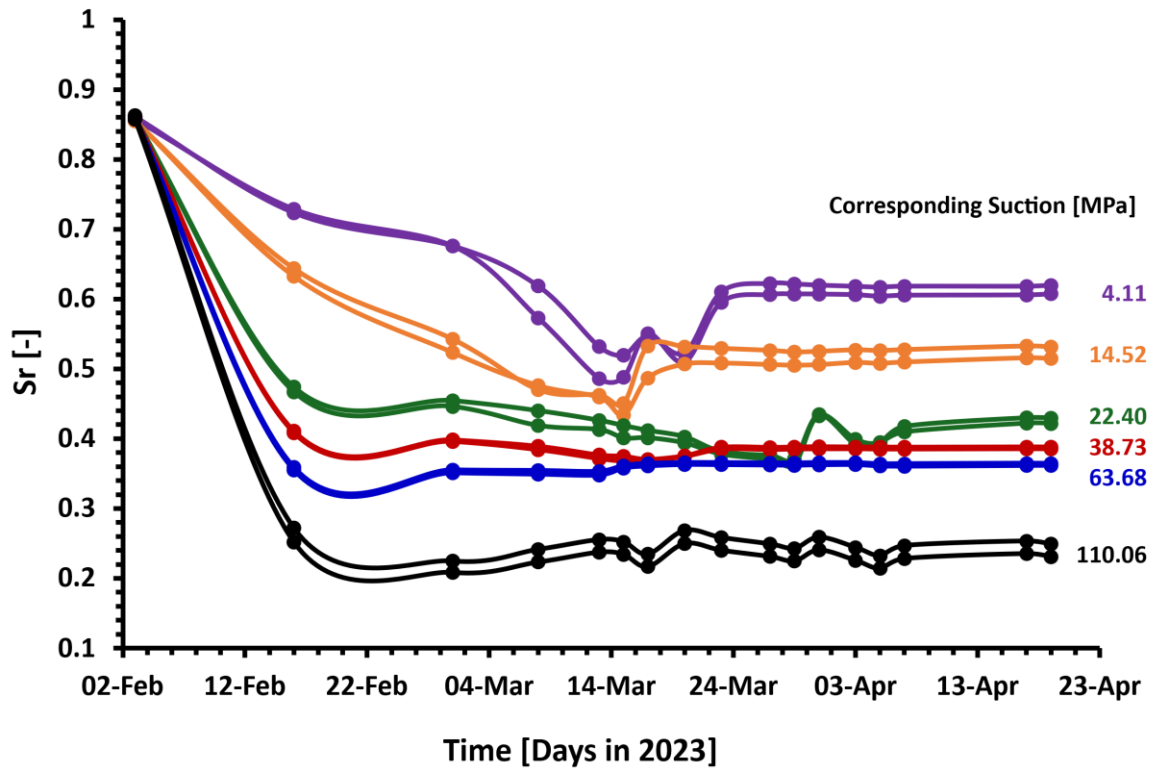
Fig. 3.8 Evolution of mass vs. time of specimen tested with the saline-solution technique (a) Soignies, (b) Kruibeke, (c) Tournai soil.



(a)



(b)



(c)

Fig. 3.9 Evolution of Degree of Saturation vs. time of specimen tested with the saline-solution technique (a) Soignies, (b) Kruibeke, (c) Tournai Soil.

3.4.2 Dynamic vapor sorption (DVS)

In this research, the three tested soils were analyzed by the Dynamic Vapour Sorption (DVS) method (Fig. 3.10). With this technique, the applied suction ranges from 14 MPa to 670 MPa. The principles of the DVS technique are presented in Section 2.4.3.3.

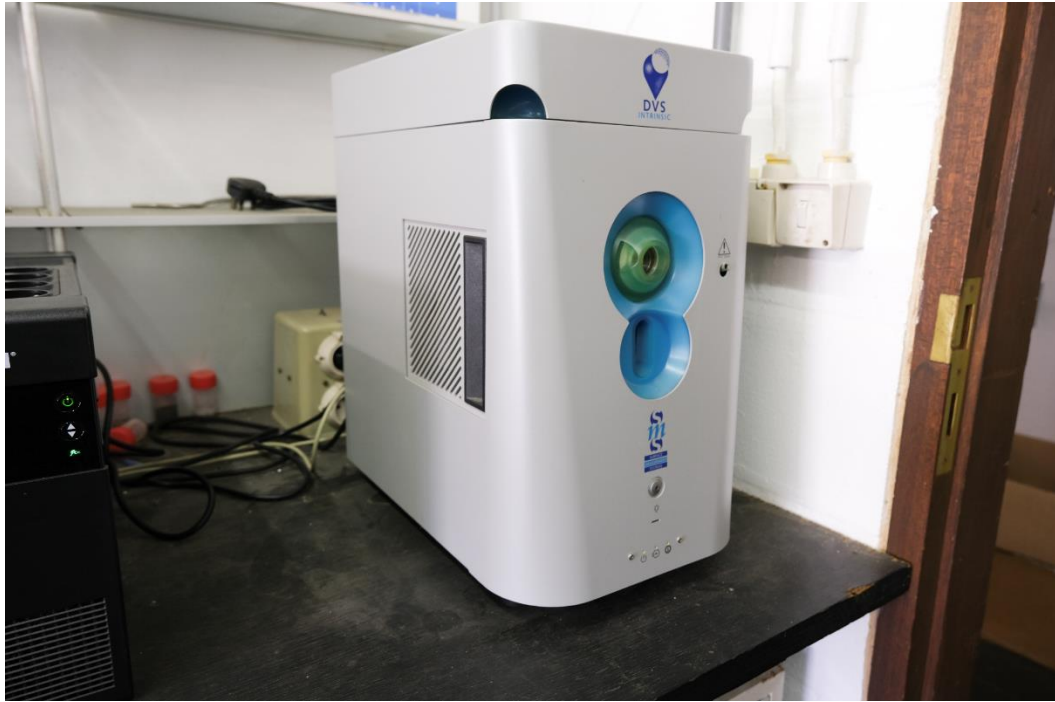


Fig. 3.10 Dynamic vapor sorption (DVS) device (www.carpor.uliege.be).

3.4.3 Water retention curve

The Water Retention Curve (WRC) of the soils has been determined to observe their retention behaviour as a function of the applied suction. For the saline solution method, four samples of each soil were prepared for each suction value, ranging from 4 MPa to 110 MPa. Two small cylindrical samples with dimensions of 33.2 mm in diameter and 7.6 mm in height, and two bigger cylindrical samples with dimensions of 50.75 mm in diameter and 40.8 mm in height. A total of 24 samples of each soil were prepared for 6 levels of suction values and underwent this process. Once equilibrium is reached, the water content and degrees of saturation at equilibrium are measured. The water content measurement corresponds to the last recorded measurement over time, the one taken at equilibrium. The dimensions of the cylindrical specimen were measured with a caliper, after compaction and after reaching hydric equilibrium, to evaluate the deformation upon drying. To ensure consistency, the measurement process follows a specific protocol. Three height measurements are taken to divide the end surfaces into three equal parts, and three diameter measurements are taken at three different heights (top, middle, and bottom). Then, an arithmetic mean is calculated to obtain a single height value and a single diameter value for each sample. This approach is used due to the non-planarity of the surfaces, which causes slight variations in dimensions from one point to

another. Furthermore, the mass was measured after compaction, after hydric equilibrium, and after full drying in an oven at 105 °C to deduce the water content and the degree of saturation and plot the water retention curve.

The measurements allow for the determination of the final total volume of the samples and, consequently, the degree of saturation at equilibrium. This is done using Equation:

$$S_{r(t_i)} = \frac{V_w}{V_v} = \frac{\left(\frac{\left(M_{Tot}(t_i) - \frac{M_{Tot,init}}{1 + w_{opt}} \right)}{\rho_w} \right)}{\left(V_{Tot,init} - \frac{M_{Tot,init}}{\frac{(1 + w_{opt})}{\rho_s}} \right)} \quad (3.1)$$

Where V_w is the volume of water, V_v is the volume of voids, M_{Tot} is the total mass, V_{Tot} is the total volume, ρ_w is the density of water, and ρ_s is the density of soil.

For each equilibrium state, the final specimen volume was used instead of the initial volume, together with the total mass recorded at the corresponding equilibrium time. The water retention curves were established by determining the degree of saturation S_r and the corresponding matric suction sat equilibrium.

The experimental points obtained from the DVS technique are only reported as water content because the volume variation of the specimen upon drying was not measured, so the degree of saturation could not be deduced.

Then, the water retention curve was fitted with the Van Genuchten Eq. (3.2) (Van Genuchten 1980)

$$S_r = \left[1 + \left(\frac{s}{P_r} \right)^n \right]^{\left(\frac{1}{n} - 1 \right)} \quad (3.2)$$

where P_r and n are fitting parameters associated with the material. The curve is calibrated on the experimental points to obtain the best match in the sense of the least squares method.

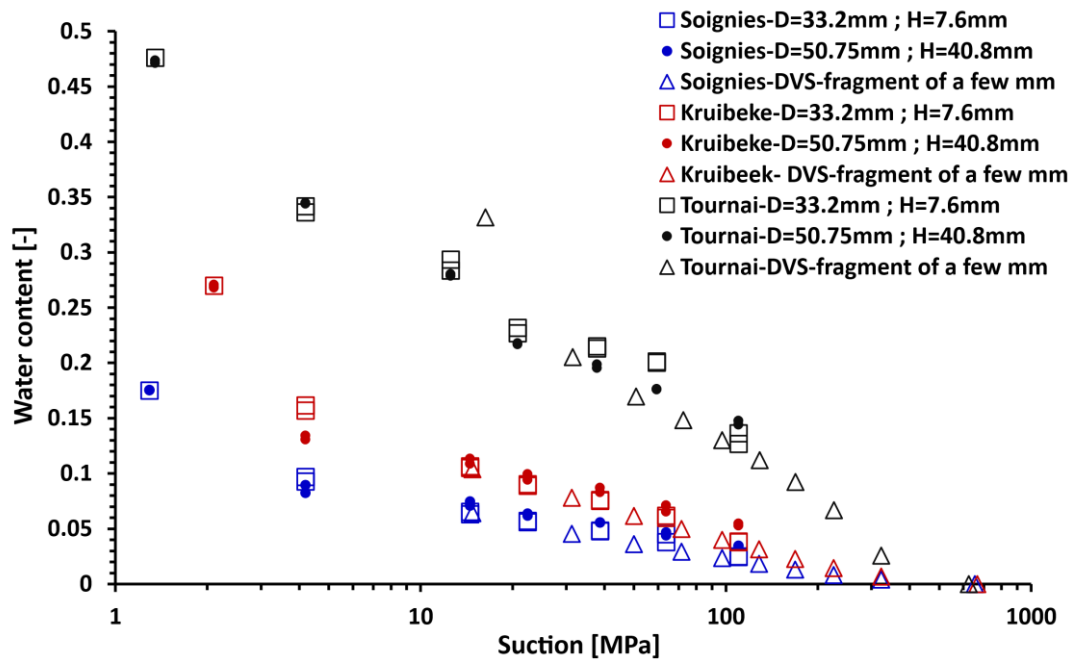
As shown in Fig. 3.11(b), the fitting curve, according to the van Genuchten equation, adapts well to the experimental results. Table 3.8 reports the van Genuchten calibration parameters for the three tested soils.

The water retention curve of the Tournai soil is above that of the Kruibeke soil, which in turn is above that of the Soignies soil. This result is entirely expected, coherent, and consistent with the nature of clayey soils. Indeed, the higher the clay content and plasticity of a soil, the greater its water retention capacity due to its fine structure and strong water adsorption capacity on clay particle surfaces. Concerning the slope of these curves, it was observed that the Soignies curve has a slightly lower slope than that of Kruibeke, which itself has a lower slope than that of Tournai. This means that water is extracted more rapidly from the soil when the curve slope is steeper. Therefore, the Tournai soil loses less water at the same suction compared to the other two soils.

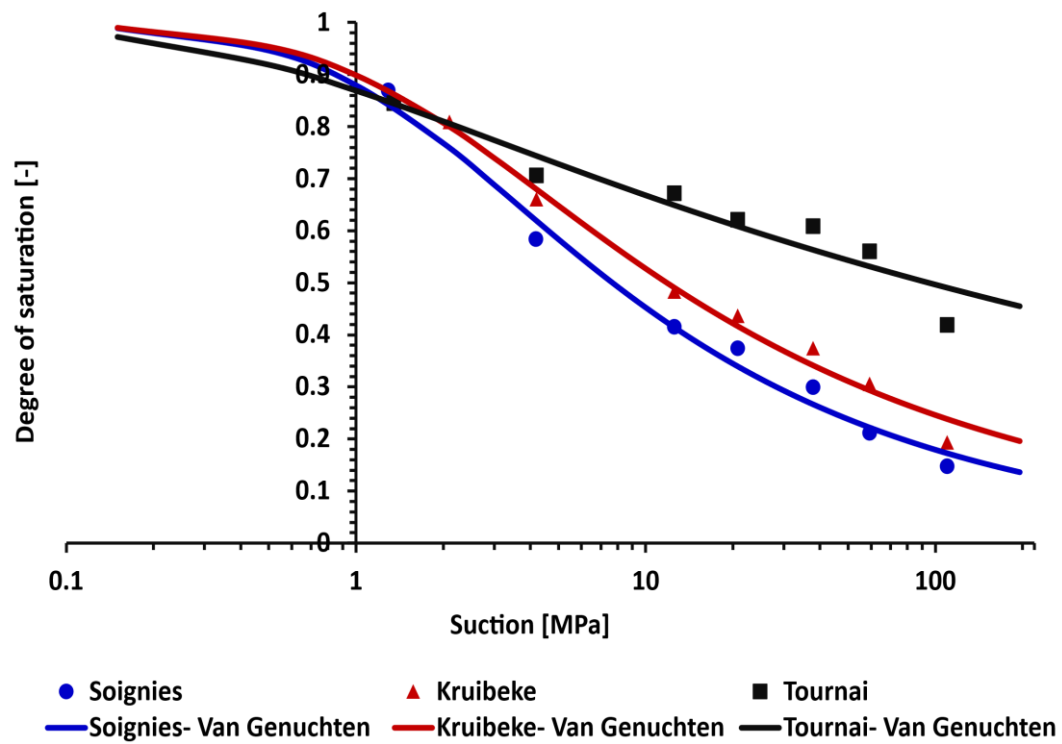
This observation can be linked to the parameter n . A lower value of n leads to a flatter slope of the water retention curve, indicating a more gradual desaturation process. Accordingly, the Tournai soil, which exhibits the lowest n , shows the flattest curve. Fig. 3.11 illustrates the water retention curves in terms of water content and degree of saturation. For the water retention curve in terms of degree of saturation (Fig. 3.11(b)), dimensional measurements and volume change evaluations were performed only on the larger specimens, as the small specimens showed greater measurement variability, whereas the larger ones provided more consistent and reliable results due to their greater size and mass. Consequently, a single volume determination and one degree of saturation value were obtained for each suction level.

Table 3.8 Calibration Parameters (P_r and n) Derived from the van Genuchten Model

Soil	P_r (MPa)	n
Soignies	1.51872	1.410
Kruibeke	1.62026	1.34
Tournai	0.46177	1.130



(a)



(b)

Fig. 3.11 Water retention curve in terms of (a) water content, and (b) degree of saturation

3.5 Volumetric Strain

The volumetric deformation corresponding to each applied suction is reported in Fig. 3.12 for the three soils. Every reported point being taken on a different specimen may explain the relative variability of the obtained points. The three soils initially undergo significant shrinkage during drying at a relatively low suction level, followed by less or nearly no shrinkage during drying at the higher suction level. This is due to the progressive densification of the material upon drying that induces a stiffening, leading to less shrinkage propensity. The Tournai soil exhibits the largest volumetric strain upon drying, due to its high initial water content and low dry density, leading to significant shrinkage upon drying.

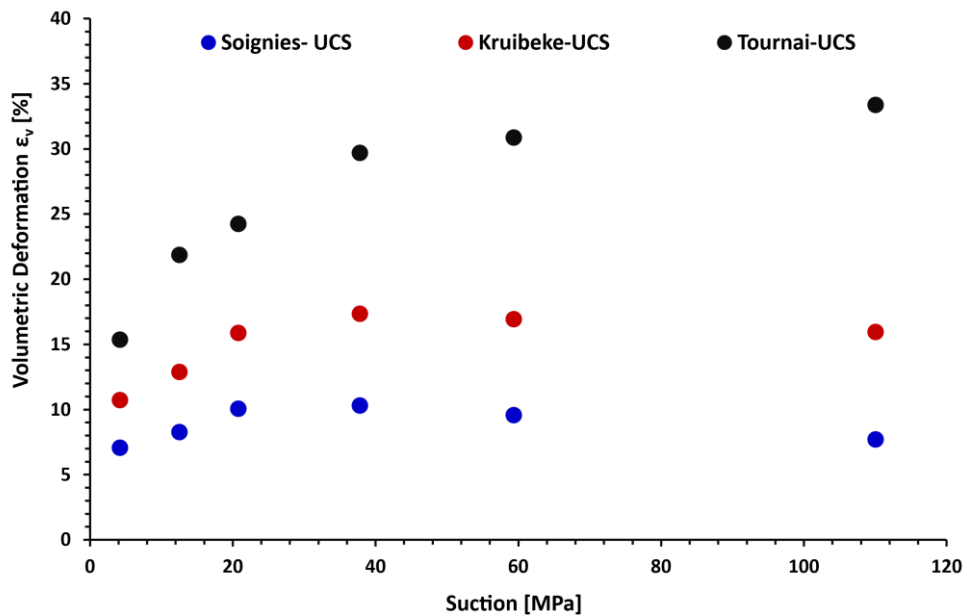


Fig. 3.12 Volumetric deformation (ε_v) versus suction under UCS conditions

3.6 Uniaxial Compression Test (UCS)

Uniaxial compression tests were performed at a rate of 0.3 mm/min on the specimen under six suction level conditions (five controlled relative humidity and the laboratory condition). For each uniaxial compression test, the elastic stiffness was determined as the maximum slope of the axial stress vs axial strain curve on an axial strain interval of 0.2%, while the uniaxial compression strength (UCS) is the maximum axial stress reached at the peak of the curve.

Figure 3.13 shows the axial stress vs axial strain curves for the three soils tested under uniaxial compression state at various suctions (from 4.2 MPa to 110 MPa of suction). From those

results, Young's modulus and uniaxial compression strengths (UCS) are plotted as a function of suction in Fig. 3.14.

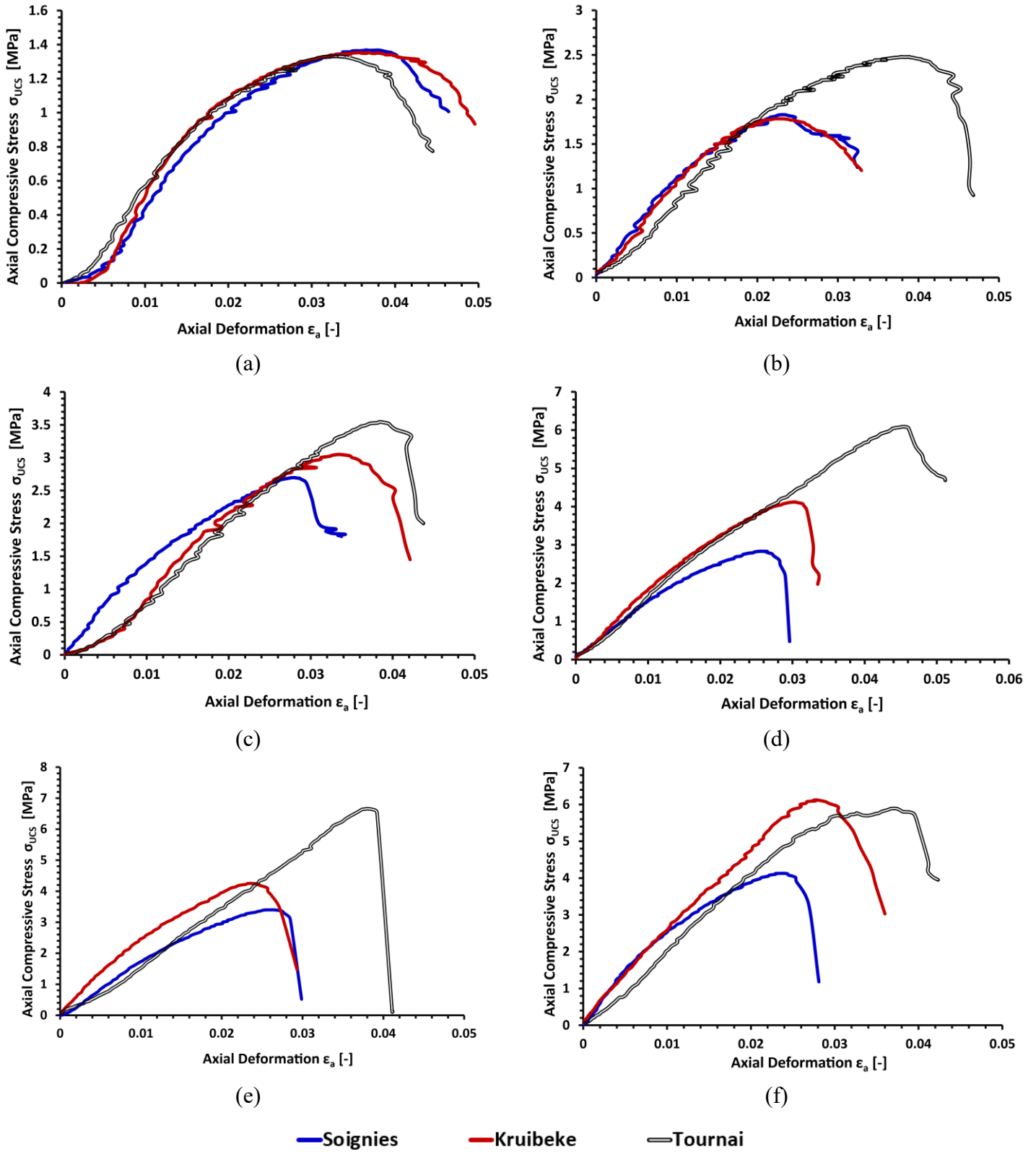
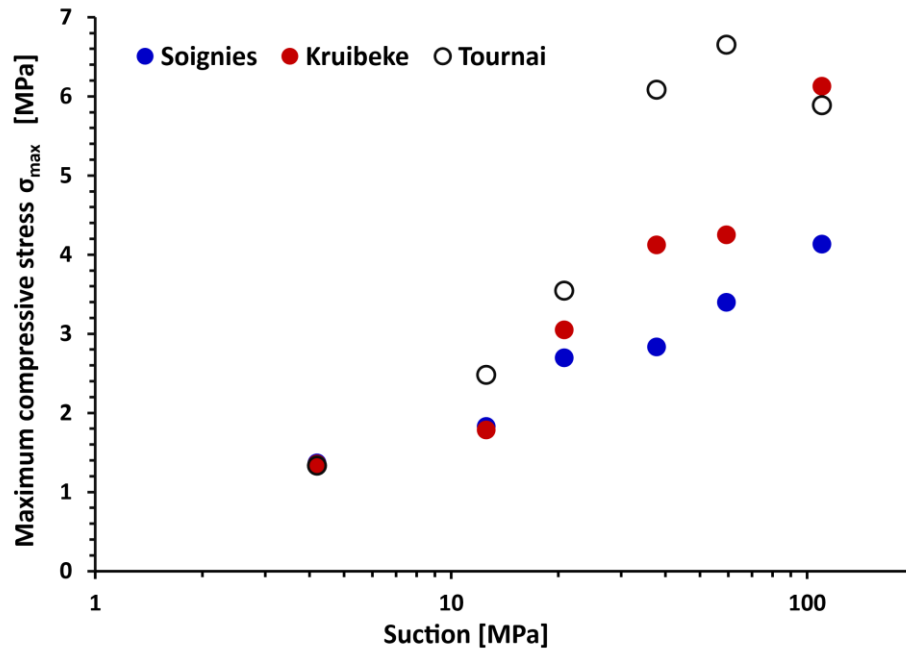
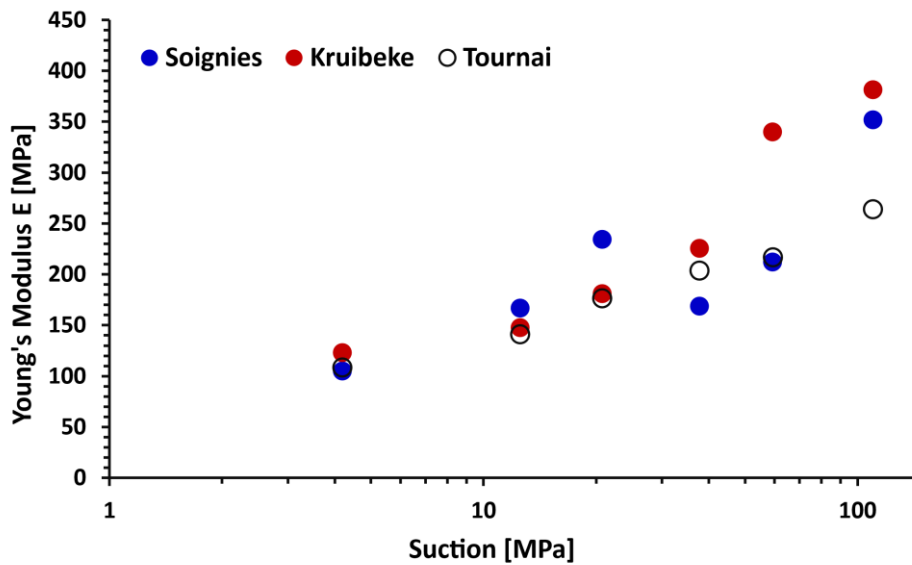


Fig. 3.13 Uniaxial compression tests at various suction levels: **a** $s = 4.2$ MPa, **b** $s = 12.55$ MPa, **c** $s = 20.8$ MPa, **d** $s = 37.8$ MPa, **e** $s = 59.4$ MPa, **f** $s = 110.1$ MPa



(a)



(b)

Fig. 3.14 Evolution of (a) uniaxial compressive strength and (b) Young's modulus, as a function of suction

3.7 Triaxial Tests (CU)

Triaxial compression tests were performed on saturated soil specimens under consolidated undrained conditions, with the measurement of pore water pressure to allow for the deduction of intrinsic shear failure criterion (i.e., effective cohesion and friction angle). Confining pressures of 100 kPa, 200 kPa, and 400 kPa were imposed. The results of consolidated undrained triaxial compression tests are presented in Figs. 3.15 to 3.17 through the evolution

of the deviatoric stress q in function of the axial strain ε_1 , the water pressure u_w in function of the axial strain ε_1 and the deviatoric stress q in function of the mean effective stress p' for the three soils. Mohr circles at failure are also plotted together with the deduced failure criterion. The data reflect the behaviour of soil samples under confining pressures of 100 kPa, 200 kPa, and 400 kPa.

The failure criterion selected corresponds to the conditions at which the deviatoric stress q is maximum. From these results, effective cohesion c and the friction angle ϕ have been deduced as summarized in Table 3.8.

Table 3.8 Effective cohesion and friction angle obtained from consolidated undrained (CU) triaxial compression tests under saturated conditions

Soil	c [kPa]	ϕ [°]
Soignies	15.5	24.3
Kruikeke	9.2	26.6
Tournai	21.8	21.6

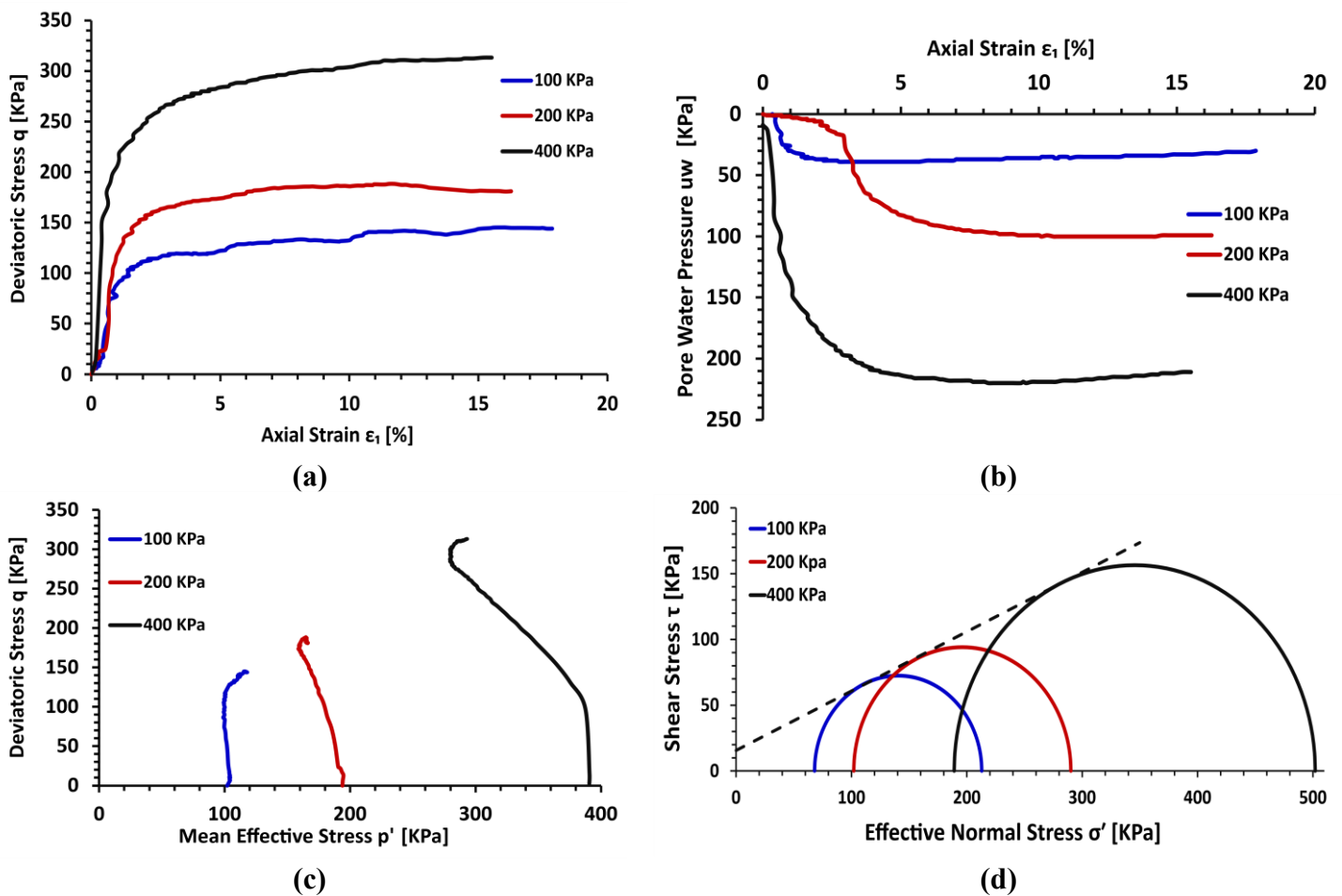
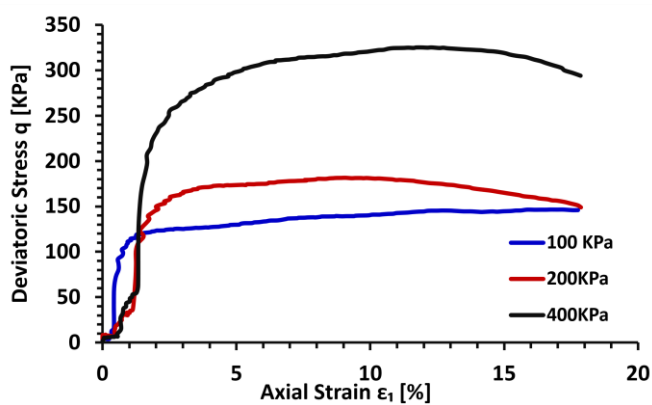
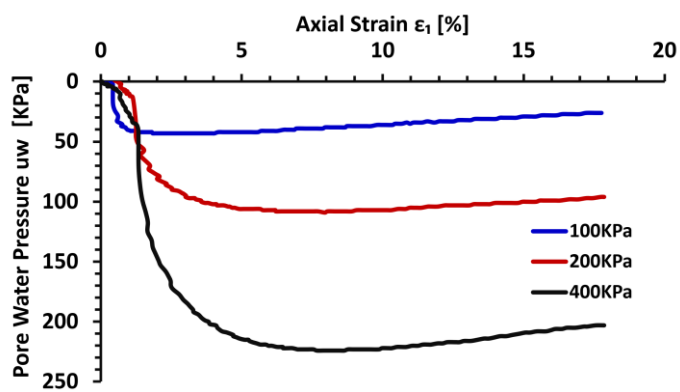


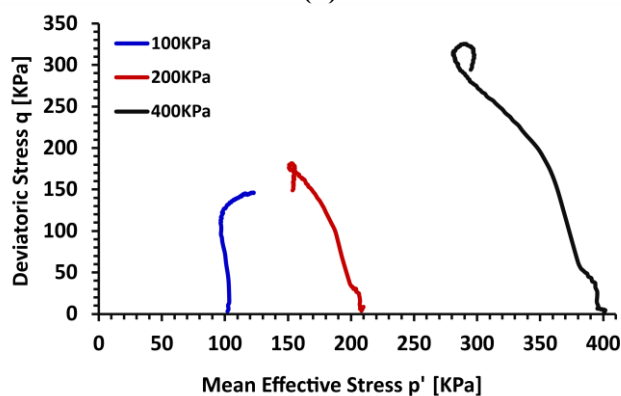
Fig. 3.15 Stress-strain and pore pressure responses under varying confining pressures of Soignies soil: **a** Deviatoric stress vs. axial strain; **b** Pore water pressure vs. axial strain; **c** Stress paths in p' - q space; **d** Shear stress vs. effective normal stress.



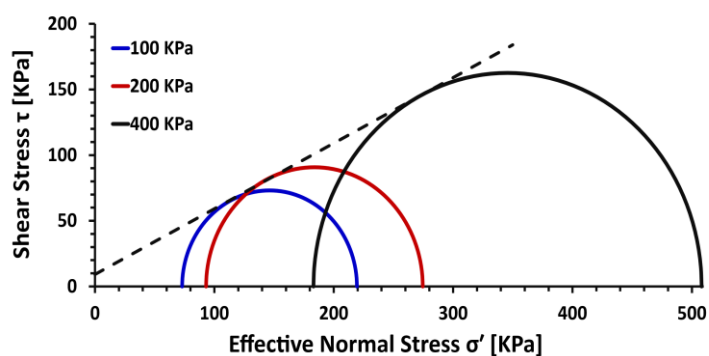
(a)



(b)

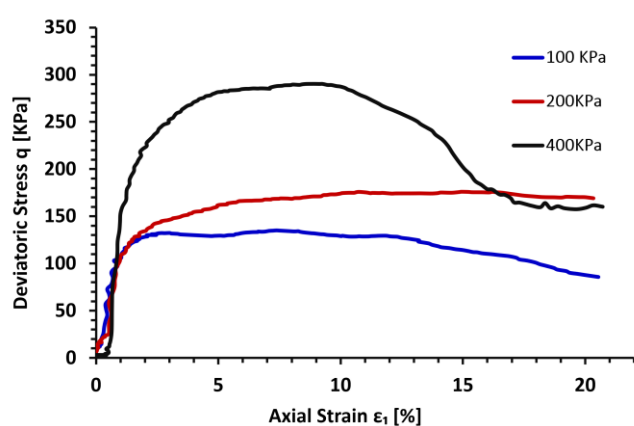


(c)

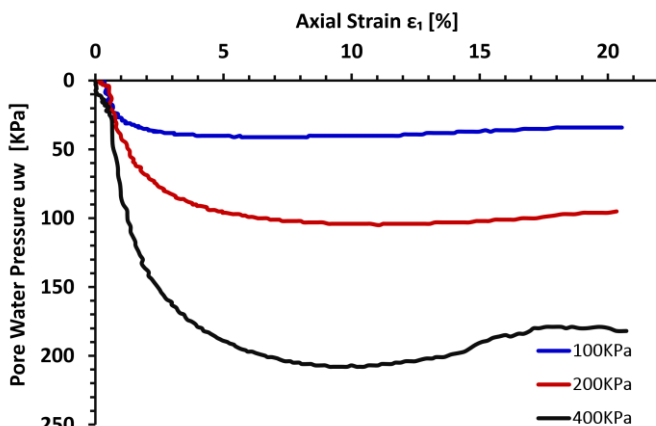


(d)

Fig. 3.16 Stress-strain and pore pressure responses under varying confining pressures of Kruikebeke soil: **a** Deviatoric stress vs. axial strain; **b** Pore water pressure vs. axial strain; **c** Stress paths in p' - q space; **d** Shear stress vs. effective normal stress.



(a)



(b)

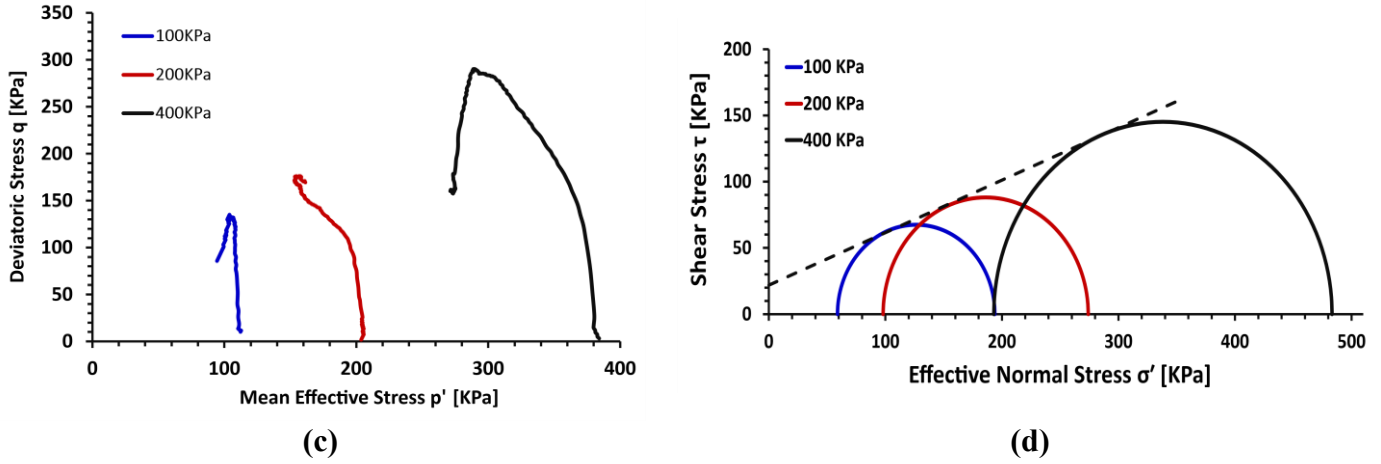


Fig. 3.17 Stress-strain and pore pressure responses under varying confining pressures of Tournai soil: **a** Deviatoric stress vs. axial strain; **b** Pore water pressure vs. axial strain; **c** Stress paths in p' - q space; **d** Shear stress vs. effective normal stress.

3.8 Brazilian Tensile Test

During desiccation, tensile cracking in the soil occurs when tensile stress overpasses tensile strength. Consequently, it is essential to study the soil tensile strength in cracking-related research. The Brazilian tensile test (also called the indirect tensile test) is considered in this study.

The tensile strength of the soils was evaluated using the Brazilian (indirect tensile) test method. According to , the cylindrical specimen is placed in a specially designed cell that consists of two steel parts that are assembled so that they make contact with the sample in two diametrically opposite sides (Stirling et al. 2015). The test involves applying a compressive load to a horizontal cylindrical disk sample via two rigid and opposed platens. The compressive load creates a perpendicular tensile stress until the specimen fails (Deshpande 2016). According to the elastic theory, soil stress is considered to be constant along its diameter. The tensile strength can be measured by the following equation (al Hour et al. 2020).

$$\sigma_t = \frac{2P}{\pi D L} \quad (3.3)$$

Where σ_t is the maximum tensile stress; P is the load at failure; D is the diameter of the sample, and L is the thickness of the sample.

Brazilian tensile tests (Fig. 3.18) were applied to the soil samples under as-compacted conditions (i.e., just after compaction under standard proctor conditions) and at various suctions

ranging from 4.2 MPa to 110 MPa (applied through the saline solution method). The sample's dimensions are 50.75 mm in diameter and 40.8 mm in height. The average tensile strength values obtained from the Brazilian test are presented in Fig. 3.19

The results indicate that applying suction to the samples significantly increases the tensile strength of all tested soils. Tensile strength exhibits a clear increasing trend with increasing suction. This relationship can be reasonably described by a logarithmic function for all three soils, as shown by the fitted trend lines.

Among the tested materials, Tournai clay shows the highest tensile strength at high suction levels and the steepest rate of increase, reflecting a stronger sensitivity of tensile behaviour to suction. Kruibeke soil exhibits an intermediate response, while Soignies soil shows the lowest tensile strength and the smallest increase with suction. This ranking is consistent with the plasticity characteristics of the soils, where Tournai is the most plastic, followed by Kruibeke, and Soignies is the least plastic.

The stronger tensile response of more plastic soils can be attributed to their higher clay content and greater capacity to develop capillary and physico-chemical bonding under suction. As suction increases, interparticle forces intensify, leading to a more pronounced improvement in tensile strength, particularly in highly plastic soils.

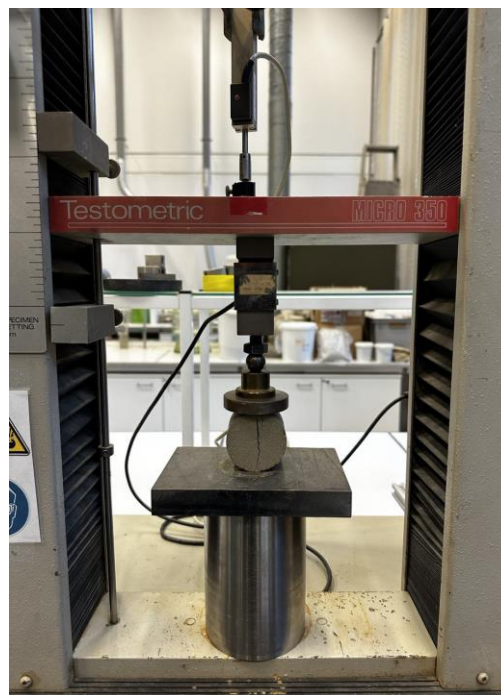


Fig. 3.18 Experimental setup for the Brazilian tensile test using a universal testing machine (Testometric Micro 350).

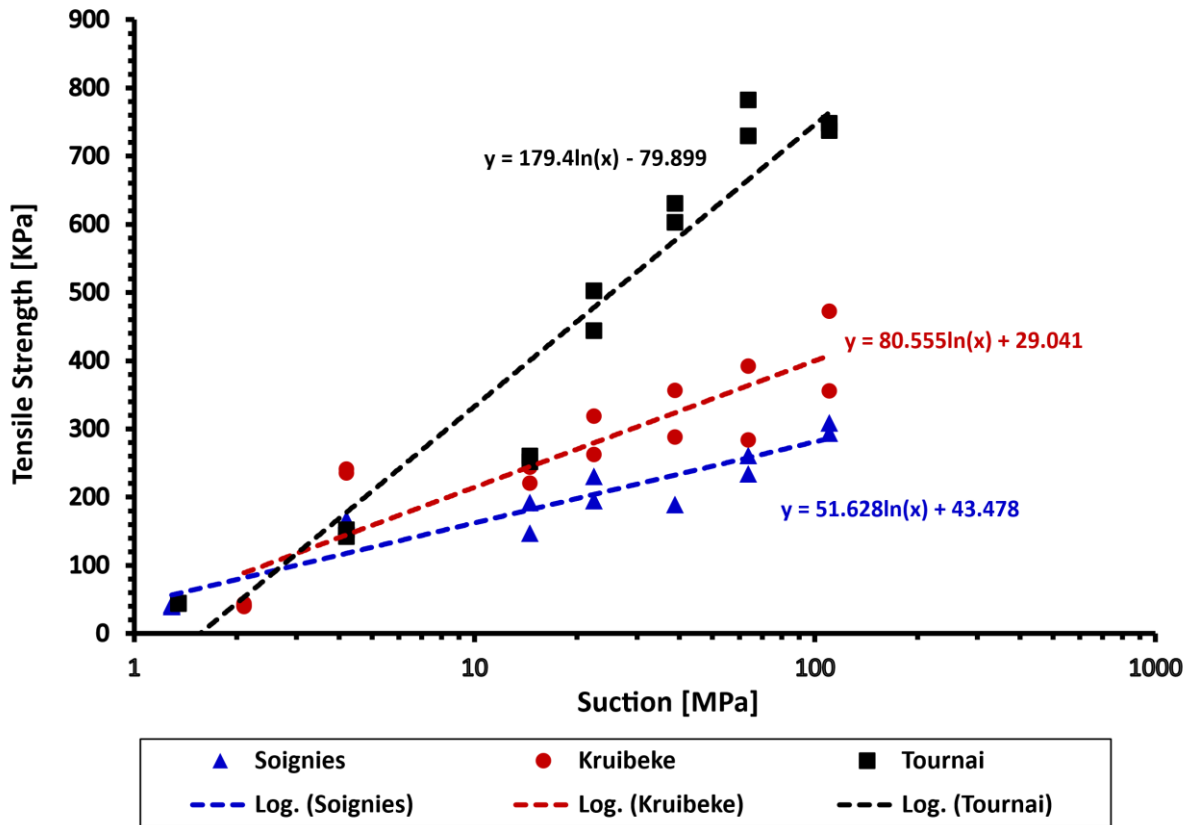


Fig. 3.19 Tensile strength vs suction.

3.9 Hydraulic Conductivity

Hydraulic conductivity tests were conducted on the three clayey soils in accordance with (ASTM D5084-24 2024). The soil samples were prepared under Standard Proctor conditions as cylindrical specimens with a height of 100 mm and a diameter of 50 mm.

The hydraulic conductivity was determined using the constant-head permeability method. The compacted specimens were saturated before testing, and a constant hydraulic head was applied at one side of the specimen through the control of the water pressure from a pressure-volume controller, while the other side of the specimen is at atmospheric pressure. The volume of water flowing through the specimen was measured over a specified time interval. The hydraulic conductivity (k) was calculated using the following equation:

$$k = \frac{Q L}{A h} \quad (3.4)$$

where Q is the volumetric flow rate, L is the specimen height, A is the cross-sectional area of

the specimen, and h is the applied hydraulic head. Table 3.9 represents the results of hydraulic conductivity for the studied soils.

Table 3.9 Hydraulic Conductivity (k) of studied soils.

Soil	k (m/s)
Soignies	5.79×10^{-10}
Kruikeke	1.02×10^{-10}
Tournai	2.00×10^{-11}

3.10 Conclusion

In this chapter, a comprehensive experimental characterization of three Belgian clayey soils (Soignies, Kruikeke, and Tournai) was carried out in order to establish their physical, hydraulic, and mechanical properties under both saturated and unsaturated conditions. The investigated soils cover a wide range of plasticity and clay mineralogy, from low plasticity clay (Soignies) to highly plastic clay (Tournai), allowing a representative assessment of different clay behaviours.

Index properties, including grain-size distribution, Atterberg limits, specific gravity, and compaction characteristics, highlighted a strong contrast in plasticity and water affinity among the tested materials, with Tournai clay exhibiting the highest clay content and plasticity index. The initial state of the compacted specimens was further characterised in terms of suction and degree of saturation, providing a consistent reference condition for subsequent hydromechanical testing.

Water retention behaviour was investigated using both the saline-solution technique and dynamic vapour sorption (DVS), enabling the determination of water retention curves over a wide suction range. The results confirmed that water retention capacity increases with clay content and plasticity, with Tournai soil showing the highest retention and Soignies the lowest. The van Genuchten model provided a satisfactory fit of the experimental data, and the associated parameters reflected the different desaturation rates of the three soils.

Mechanical behaviour under varying suction was assessed through uniaxial compression and Brazilian tensile tests. Both compressive strength and tensile strength were found to increase markedly with suction, highlighting the crucial role of suction in enhancing interparticle

bonding and apparent cohesion. The most plastic soil (Tournai) systematically exhibited the highest stiffness and strength, as well as the strongest sensitivity to suction. In addition, saturated consolidated undrained triaxial tests allowed the determination of the intrinsic shear strength parameters, while hydraulic conductivity tests confirmed the very low permeability of the compacted clays, decreasing with increasing plasticity.

For clarity and synthesis, all the main physical, hydraulic, and mechanical properties of the three soils are summarized in Table 3.10, which provides a global comparison of their index properties, retention parameters, strength characteristics, and permeability. This table constitutes a reference dataset for the interpretation and modelling presented in the following chapters.

Overall, the results presented in this chapter provide a consistent experimental basis for analysing the coupled hydromechanical behaviour and cracking susceptibility of compacted clayey soils under drying conditions, which will be further exploited in the subsequent parts of this study.

Table 3.10 Physical parameters of the three tested soils.

Soil	<i>Soignies</i>	<i>Kruibeke</i>	<i>Tournai</i>
Liquid Limit (%)	36.4	56	141.1
Plastic Limit (%)	18.6	29.7	42.76
Shrinkage Limit (%)	9	14.4	27
Plasticity Index (%)	17.8	26.3	98.3
Specific Gravity, G_s (g/cm ³)	2.63	2.81	2.52
Soil type (USCS)	CL	CH	CH
Maximum dry Unit Weight under SPT (g/cm ³)	1.72	1.45	1.04
Optimum Water Content under SPT (%)	18	27	47.6
Hydraulic Conductivity, k (m/s)	5.79×10^{-10}	1.02×10^{-10}	2.0×10^{-11}
Initial Degree of Saturation under as-compacted state (%)	87	81	85
Matric suction under as-compacted state (MPa)	0.785	1.67	1.063
Total suction under as-compacted state (MPa)	1.291	2.104	1.351

4. CALIBRATION OF THE EFFECTIVE STRESS PARAMETER

The effective stress parameter (c) in the generalized effective stress formulation for unsaturated soils is intended to weight the contribution of suction to the magnitude of effective stress. It is usually expressed as a function of the degree of saturation. This chapter presents a comprehensive methodology to calibrate this c parameter, based on the uniqueness of the shear failure criterion when expressed in a generalized effective stress reference. The method requires knowing the intrinsic strength parameters (effective cohesion and friction angle), from triaxial compression tests upon saturated conditions, and the uniaxial compression strength (UCS) at various suctions. The efficiency of the methodology is demonstrated by applying it to the three compacted clayey soils, characterized experimentally in Chapter 3 of this thesis. Then, the microstructural significance of the obtained relationship between the c parameter and the degree of saturation is discussed from pore size distribution analyzed from a bi-modal perspective.

4.1 Introduction

The proposed approach relies on experimental results obtained from triaxial compression tests under saturated conditions to determine the intrinsic shear strength parameters, and on uniaxial compression tests performed at different suction levels to evaluate the corresponding unsaturated strength. For each test condition, a value of χ is determined and subsequently related to the degree of saturation for the three studied soils. Existing formulations for the effective stress parameter χ in unsaturated soils often rely on complex descriptions of the soil microstructure and require parameters that are difficult to determine experimentally. In this context, the present study adopts a simple and robust calibration approach, in which χ is expressed as a function of the degree of saturation using a power-law relationship, based on standard mechanical tests.

4.2 Generalized Effective Stress Framework

The generalized effective stress concept extends Terzaghi's effective stress definition (Terzaghi 1936) to unsaturated conditions by incorporating suction through a weighting parameter χ (Bishop 1959). The effective stress is expressed as:

$$\sigma' = \sigma - \chi s \quad (4.1)$$

where σ' is the effective stress, σ is the total stress, s is the suction, and χ is the effective stress parameter. In this formulation, χ varies with the degree of saturation, from zero for dry soil to unity for fully saturated conditions, enabling a transition from partially to fully saturated states.

Various expressions have been proposed to relate the χ parameter to the degree of saturation or suction. Among them, the expression proposed in Eq. (4.2), originally by (Tarantino and Tombolato 2005), introduces a microstructural interpretation of χ .

$$\chi = \frac{S_r - S_r^m}{1 - S_r^m} \quad (4.2)$$

where S_r is the degree of saturation and S_r^m is the microstructural degree of saturation, corresponding to the degree of saturation required to fill the micro-porosity, while the macro-porosity remains dry. In this framework, χ remains equal to zero when water is stored within the micro-porosity and increases when water progressively invades the inter-aggregate voids.

Following this microstructural interpretation, a simplified power-law relationship between χ and the degree of saturation, as proposed by (Alonso et al. 2010), is adopted for calibration purposes:

$$\chi = (S_r)^\alpha \quad (4.3)$$

where S_r is the degree of saturation and α is a material parameter controlling the evolution of χ .

The detailed theoretical background and existing formulations of χ are presented in Chapter 2. Based on this framework, the calibration methodology is introduced in the following section.

4.3. Methodology for χ Parameter Calibration

The objective of developing equations for effective stress in unsaturated soils is to derive a generalized effective-stress expression that provides a unique failure criterion. This failure

criterion should not be influenced by suction and the degree of saturation when expressed in terms of effective stress (Gerard et al. 2015). Combining generalized effective stress formulation (Eq. (4.1)) with the conventional Mohr-Coulomb shear failure criterion reveals that the effect of suction and degree of saturation on the shear resistance is indirectly considered through the χ parameter (Eq. (4.4)).

$$\tau_f = c + \chi s + \sigma'_N \tan \phi \quad (4.4)$$

where c is the cohesion, ϕ the friction angle, σ'_N the effective normal stress, σ_N the total normal stress and τ_f the shear stress at failure.

Accordingly, the term $\chi s \tan \phi$ can be considered as an additional strength provided by soil suction, usually called capillary cohesion. In other words, the product χs plays a similar role than an external normal stress. It exerts internal stress within a soil specimen, even in the absence of any external stress. This internal stress depends on the water conditions of the sample, which are expressed through the product of the effective stress parameter and suction. So, suction can be considered as an internal confining stress. The stress state expressed in generalized effective stress can be represented on a Mohr circle diagram by shifting the Mohr circles expressed in total stress to the right by an amount equivalent to χs (Fig.4.1) (Gerard et al. 2015).

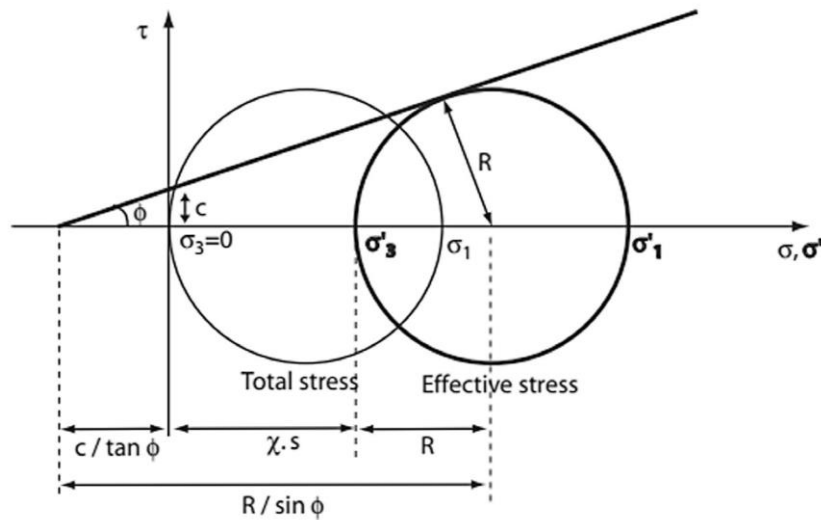


Fig. 4.1 Mohr circles experienced during the uniaxial compression test expressed in terms of total stress and generalized effective stress (Gerard et al. 2015)

When starting from uniaxial compression strength at various suctions, the calibration of the χ parameters requires finding an appropriate expression for χ as a function of the degree of saturation (or eventually suction), which allows horizontal shifting of the Mohr circles obtained

at failure to align them with the failure criterion expressed in terms of effective stress. The horizontal distance between the Mohr circle expressed in total stress and the failure criterion corresponds to the product χs and can be obtained through a geometric relationship based on stresses and intrinsic parameters, as illustrated in Fig. 4.1.

The value of the effective stress parameter is then given by Eq. (4.5) (Gerard et al. 2015).

$$\chi = \frac{\frac{R - c \cos \phi}{\sin \phi} - R}{s} \quad (4.5)$$

where R is the radius of the Mohr circle, defined as $R = \frac{(\sigma_1 - \sigma_3)}{2}$. For uniaxial compression tests, no external confining stress is applied, leading to $\sigma_3 = 0$.

Considering the power law of the degree of saturation (Eq. (4.3)) expressed in a double logarithmic plane in Eq. (4.6),

$$\log \chi = \alpha \log S_r \quad (4.6)$$

the α exponent is a material parameter, which is the slope of the regression line in the $(\log S_r ; \log \chi)$ plane.

For every single test result (at one suction and one degree of saturation), one value of χ can be deduced. So, it is possible to obtain a set of points in the $(\log S_r ; \log \chi)$ plane and then find the best regression line, with the slope being the α parameter.

The full methodology requires shear tests (triaxial compression tests or direct shear tests) under saturated conditions to obtain the intrinsic (effective) shear failure criterion, defined by the effective cohesion and friction angle. Then, a series of uniaxial compression tests at various suctions (corresponding to various degrees of saturations) will serve to find one value of χ per value of S_r . The corresponding experimental results are presented in Chapter 3. At the end, the α exponent is calibrated to find the best fit between Eq. (4.3) and $\chi(S_r)$ through the slope of the regression line in the $(\log S_r ; \log \chi)$ plane, as expressed in Eq. (4.6).

4.4 Calibration Results and Discussion

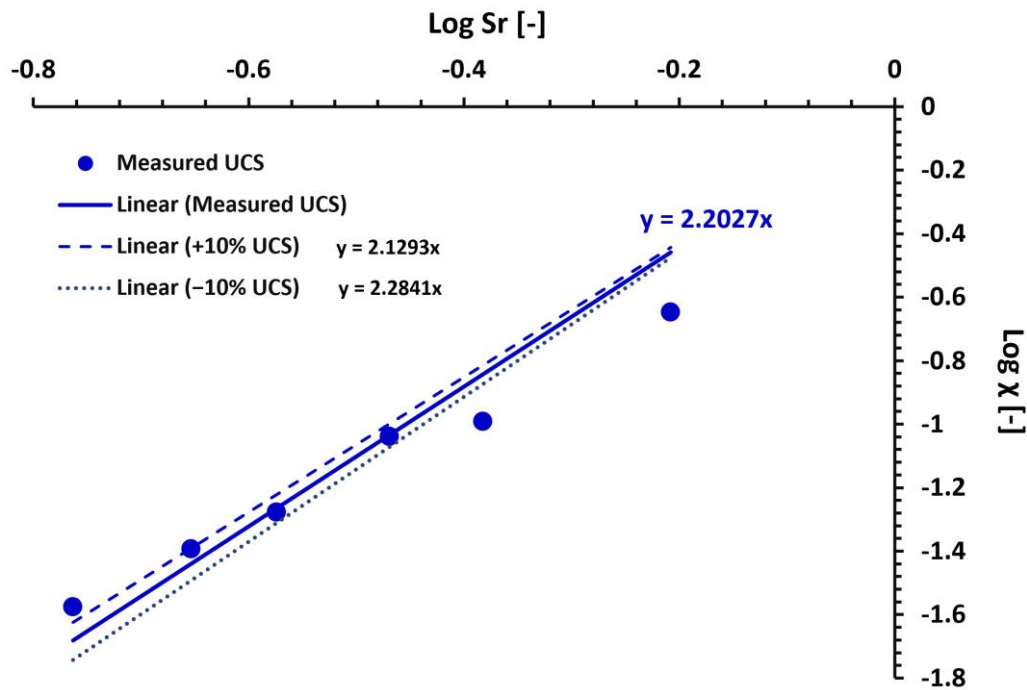
According to the methodology exposed in Section 4.3, the expression of the χ parameter as a function of the degree of saturation can be fitted through the calibration of the α exponent of Eq. (4.3). The main principle is to target the uniqueness of the shear failure criterion, whatever the suction level, when the stress state is expressed in the generalized effective stress reference. According to Eq. (4.6), α is the slope of the $(\log S_r ; \log \chi)$ linear regression curve passing through the origin of the graph (because when $S_r = 1$, $\chi = 1$, i.e. when $\log S_r = 0$, $\log \chi =$

0).

To evaluate the influence of measurement uncertainty in the unconfined compression (UCS) test on the calibration of the χ parameter, a sensitivity analysis was performed.

In addition to the measured UCS value, two additional calibration curves were constructed by considering $\pm 10\%$ variation in UCS strength. Figure 4.2 illustrates these three calibration curves for each soil. The results show that the $\pm 10\%$ variation in UCS induces only a minor shift in the χ - S_r relationship, confirming the robustness of the proposed calibration approach with respect to possible experimental errors in UCS testing.

As shown in Fig. 4.2, the best fit is obtained for $\alpha = 2.20$ for Soignies soil, $\alpha = 2.62$ for Kruibeke soil and $\alpha = 3.97$ for Tournai soil.



(a)

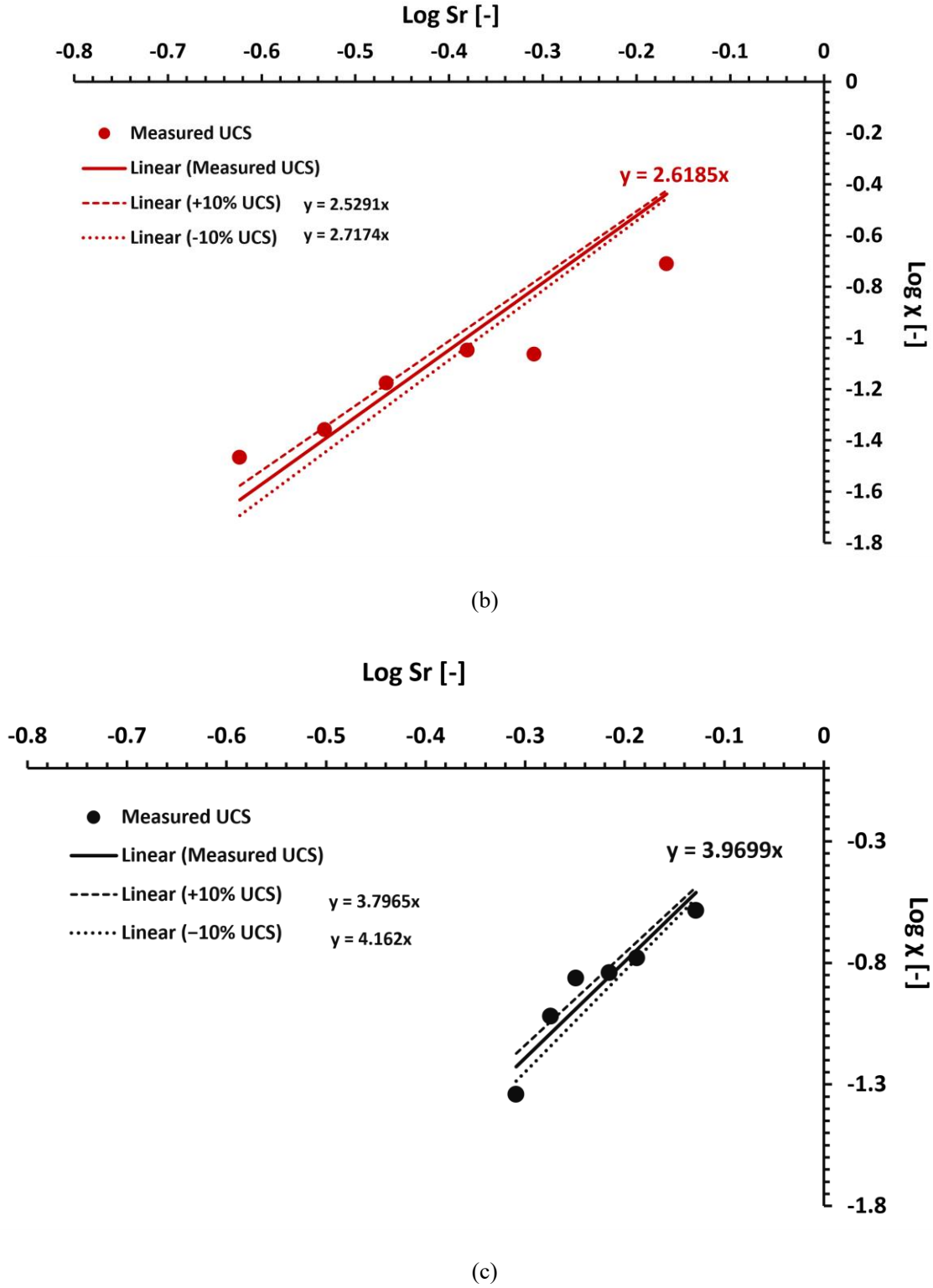
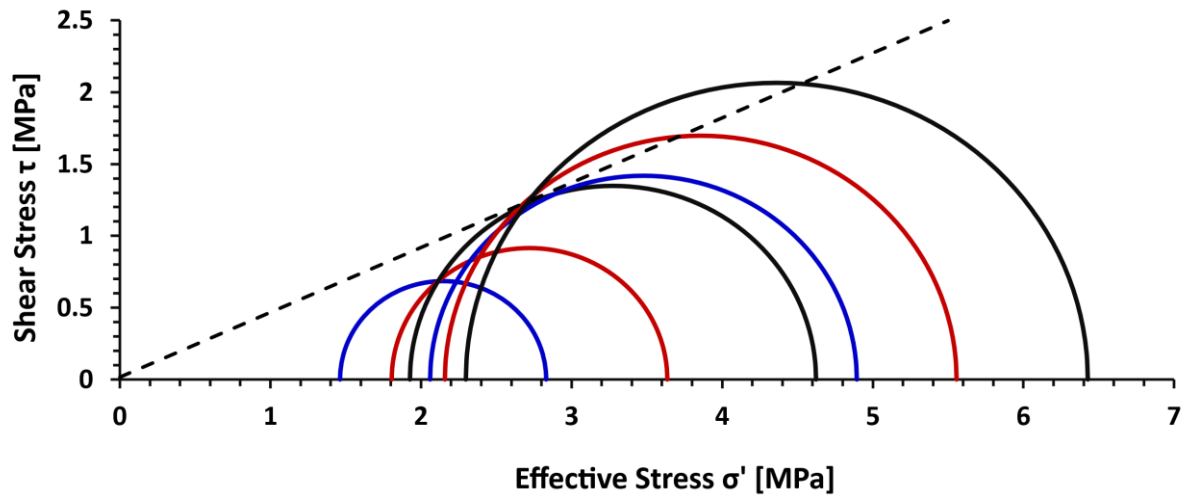
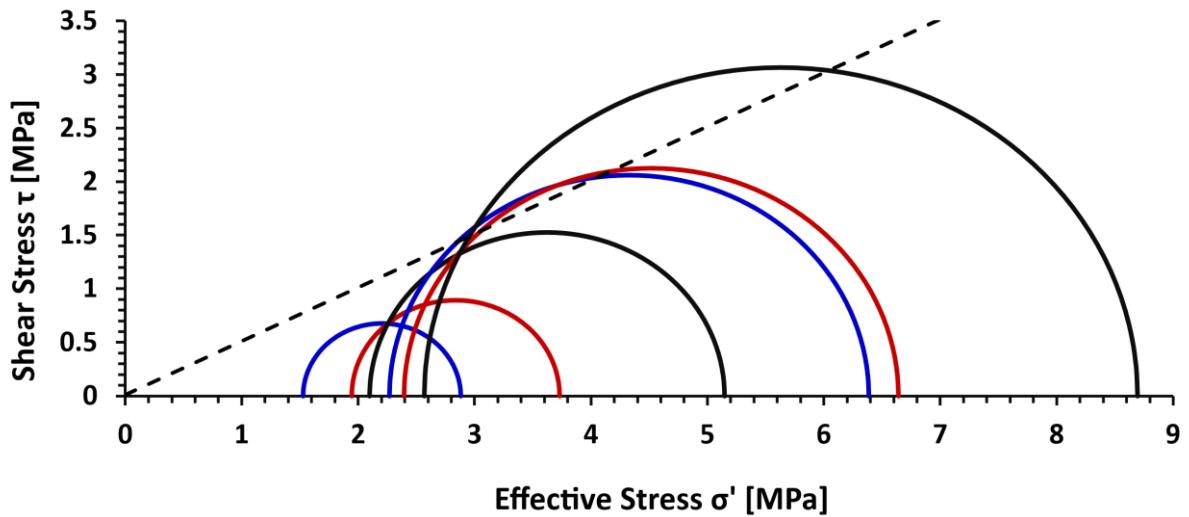


Fig. 4.2 Sensitivity of χ calibration to $\pm 10\%$ variation in UCS for the tested soils: (a) Soignies, (b) Kruibeke, and (c) Tournai. Solid, dashed, and dotted lines represent the measured UCS, $+10\%$ UCS, and -10% UCS cases, respectively.

Finally, the set of Mohr circles at failure obtained from uniaxial compression at various suctions is reported in Fig. 4.3, using the generalized effective stress reference for the three soils. As expected, by the essence of the methodology, the obtained Mohr circles at failure tend to align on a unique failure criterion based on the cohesion and the friction angle deduced from triaxial tests under saturated conditions. Even if the alignment is not perfect, it corresponds to the best fit in Fig. 4.2.



(a)



(b)

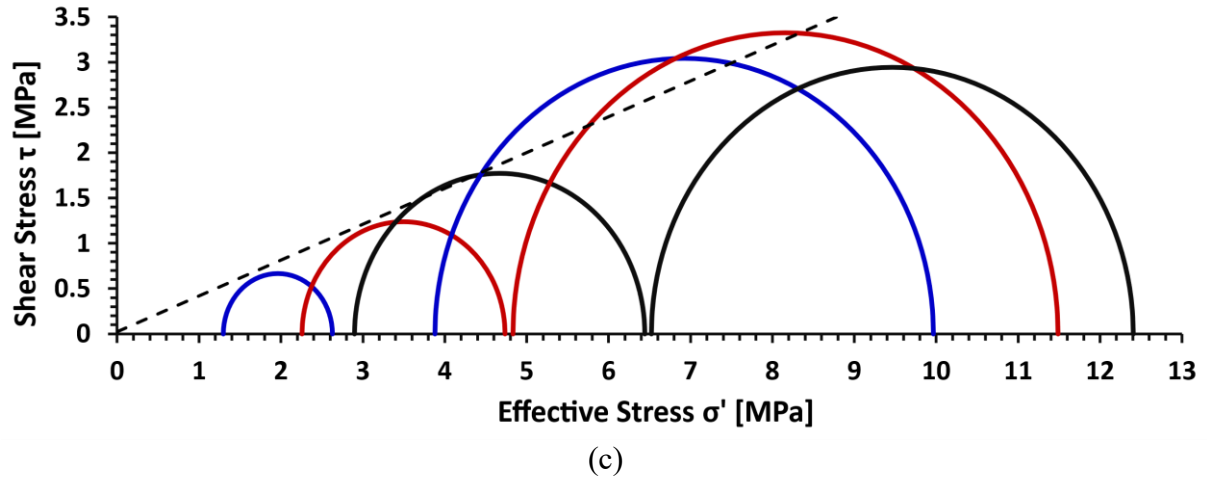


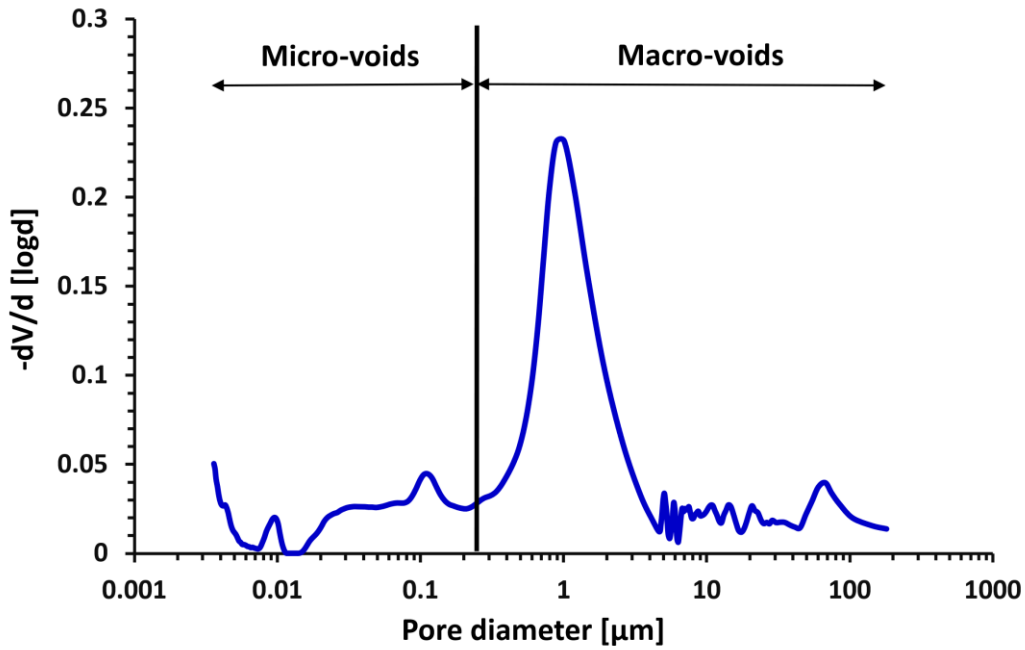
Fig. 4.3 Mohr circles at failure expressed in terms of generalized effective stress for uniaxial compression tests performed at different initial suctions: (a) Soignies soil, (b) Kruibeke soil, (c) Tournai soil

4.5 Microstructural Interpretation

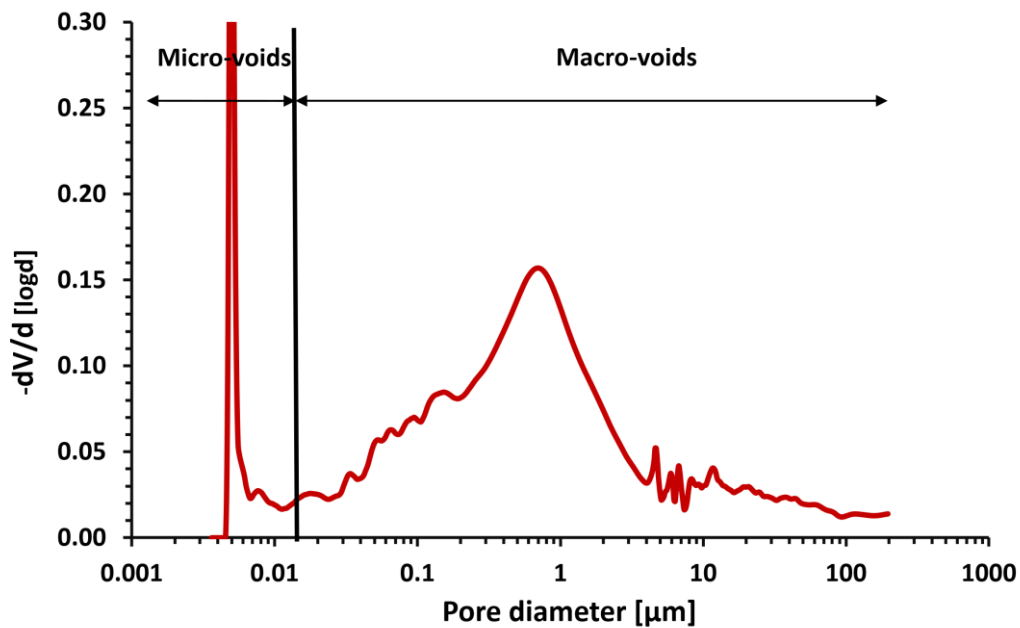
The evolution of the effective stress parameter χ in unsaturated soils is fundamentally linked to the underlying microstructural changes occurring within the pore network. According to the concept developed by (Tarantino and Tombolato 2005), subsequently adapted by (Alonso et al. 2010), (Alonso et al. 2013) and (Vaunat and Casini 2017), only the “inter-aggregate” water (located in macro-voids) contributes to the macroscopic effective stress, while the “intra-aggregate” water (located in micro-voids) is neutral regarding this χ parameter. As long as the water is confined within aggregates, there is no evolution of χ . Once aggregates become fully saturated, additional water migrates into larger voids, leading to capillary effects between aggregates that causes an increase in χ .

Consequently, the pore size distribution (PSD) of the soil is a good indicator for the relation between the χ parameter and the degree of saturation. Mercury intrusion porosimetry (MIP) was used to characterize the pore size distribution of the soils. Figure 4.4 shows the pore size distribution of the three soils at the initial state (before any suction application) obtained from mercury intrusion porosimetry (MIP). Differentiation between micro- and macro-porosity is delimited by the lowest point of the PSD at intermediate pore diameter. The cumulative volume of intruded mercury for voids larger than this threshold is the macro-voids, while the cumulative volume for smaller voids is the micro-voids. On that basis, the macro- and micro-porosity, as obtained from MIP, are reported in Table 4.1. The total porosity, as determined from external volumes and dry masses, is slightly higher than the total intruded porosity. The

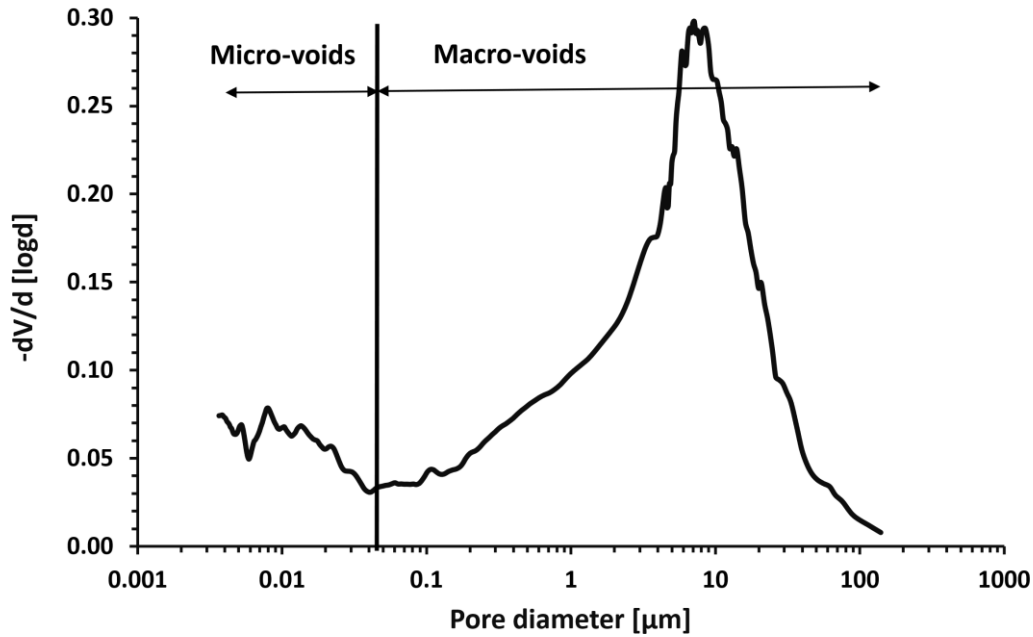
difference between total porosity and intruded porosity is attributed to the micro-porosity because it is considered to correspond to very small pores not intruded by mercury during MIP. Instead of porosities, it is more convenient to work with void ratios because the void space is reported with respect to a constant value (i.e., the volume of solid particles), in opposition to porosity, which is reported with respect to a changing parameter (i.e., the total volume). Table 4.1 also reports the total, micro, and macro void ratios in addition to porosities.



(a)



(b)



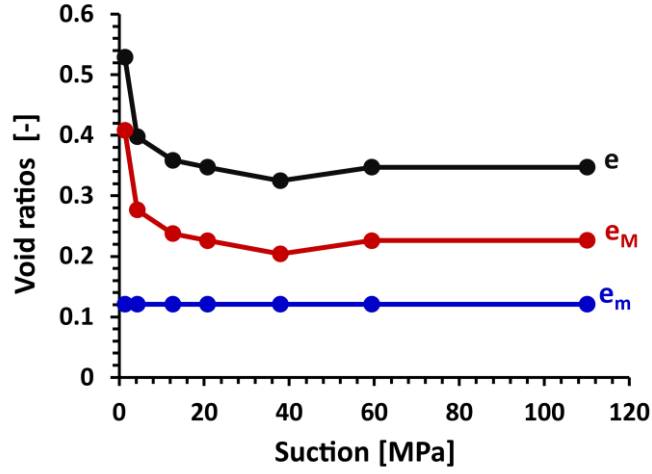
(c)

Fig. 4.4 Pore size distribution (PSD) of the three soils obtained from mercury intrusion porosimetry (MIP); Differentiation between micro and macro voids is delimited by the lowest point of the PSD at intermediate pore diameter; (a) Soignies soil, (b) Kruibeke soil, (c) Tournai soil

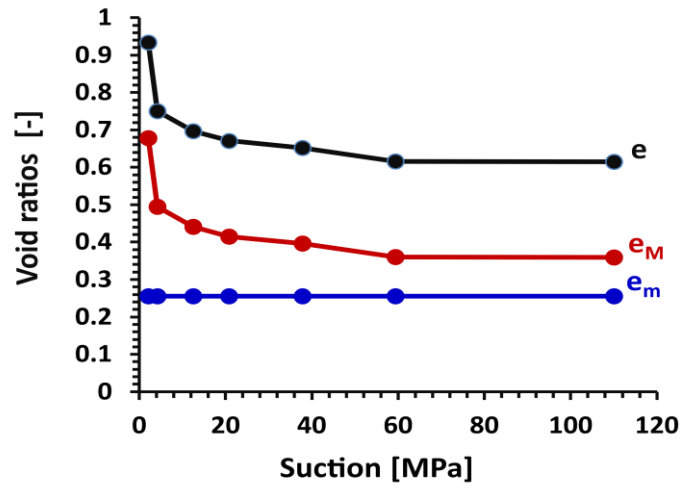
Table 4.1 Microstructural parameters of the three soils as deduced from mercury intrusion porosimetry

Soil	Soignies	Kruibeke	Tournai
Intruded porosity	0.327	0.395	0.472
Intruded micro-porosity	0.060	0.044	0.069
Intruded macro-porosity	0.267	0.351	0.402
Total porosity	0.346	0.483	0.587
Total micro-porosity	0.079	0.132	0.185
Total macro-porosity	0.267	0.351	0.402
Total void ratio	0.529	0.934	1.421
Total micro-void ratio	0.121	0.256	0.447
Total macro-void ratio	0.408	0.678	0.974
Initial microstructural degree of saturation $S_{r,0}^m$	0.314	0.274	0.229

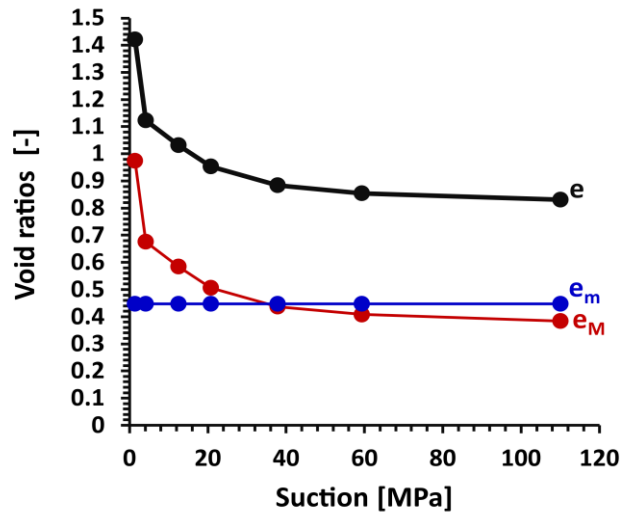
Then, during suction application, the shrinkage is assumed to be fully attributed to macro-void ratio reduction. In this concept, the aggregates (including micro-voids) are assumed incompressible while the closure of macro-voids induces macroscopic shrinkage. Consequently, upon drying, macro-voids decrease while micro-voids remain constant. Figure 4.5 reports the evolution of total, micro, and macro void ratios, based on the volumetric strain as a function of suction, as reported in Fig. 3.12.



(a)



(b)



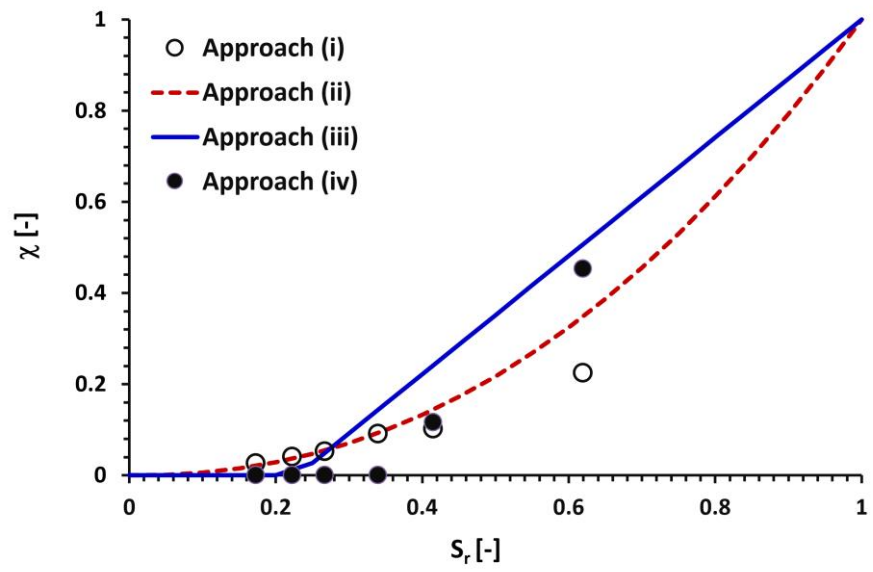
(c)

Fig. 4.5 Evolution of total void ratio (e), macro void ratio (e_M) and micro-void ratio (e_m) during drying, assuming that the micro-void ratio (intra-aggregate) remains constant; (a) Soignies soil, (b) Krübeke soil, (c) Tournai soil

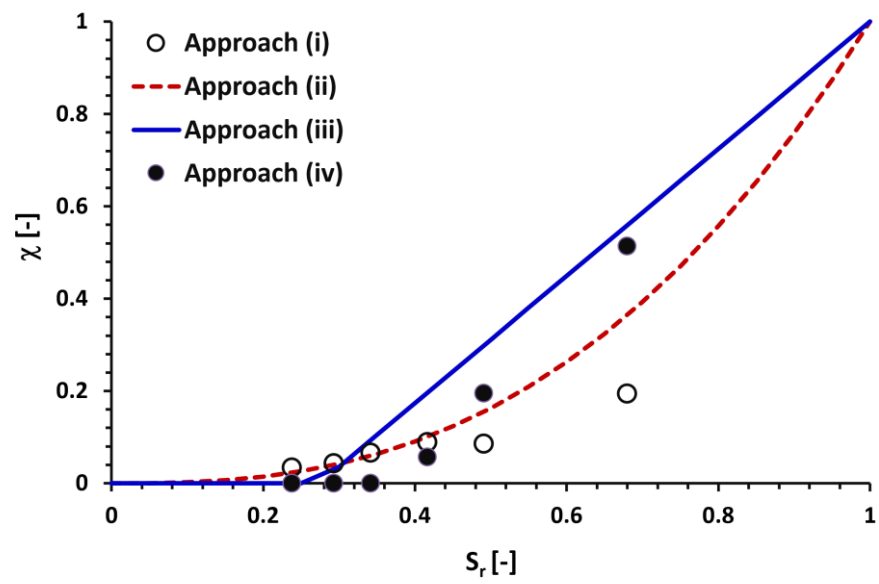
When total, micro, and macro void ratios are known at various suctions, it is then possible to deduce the χ parameter according to the concept of Eq. (4.2). Figure 4.6 reports the χ parameter as a function of the degree of saturation for the three soils, following four approaches:

- (i) The value of χ such that the shear failure obtained from uniaxial compression tests at various suctions aligns with the failure criterion under saturated conditions.
- (ii) The power law (Eq. (4.3)), with the α exponent calibrated according to the methodology developed in Section 4.3.
- (iii) Eq. (4.2), assuming that the void ratio remains constant upon drying (i.e. $S_r^m = S_{r,0}^m = \text{constant}$)
- (iv) Eq. (4.2), considering evolving void ratio upon drying as shown in Fig. 4.5, the reduction of the void ratio is fully attributed to the reduction of macro void ratios (i.e. S_r^m increases upon drying).

When the χ parameter is obtained experimentally from uniaxial compression tests at various suctions to fulfill a unique failure criterion (Approach (i)), the relation of χ with S_r is in relatively good agreement with the conceptual model of (Tarantino and Tombolato 2005), based on microstructure. The χ parameter remains negligible at low degrees of saturation, as water is primarily retained within the intra-aggregate microporosity. The power law (Approach (ii)) fits relatively well the experimental point of χ parameter for Tournai soil (the most plastic soil), while for Kruikebeke and Soignies soils, the matching is less accurate for the lower suction levels (i.e., the higher degree of saturation). But still, the trend of the curve agrees with the experimental points. For Approach (iii), which neglects the evolution of the microstructure upon drying, the transition between dry macro-voids and water invading macro-voids is probably too sharp. In reality, the transition between those two states is smoother because water starts to invade the macro-voids progressively, while micro-voids are not yet fully saturated. Also, assuming a constant microstructure upon drying is a strong assumption because it underestimates the S_r^m parameters at high suction. Indeed, at high suction, the macro voids have been significantly reduced, due to shrinkage, leading to a larger proportion of micro-voids over the total voids. Finally, Approach (iv) gives a better trend than Approach (iii), even if the transition between dry and water invading macro-voids is still too sharp for the same reason as Approach (iii) (i.e., water starts to invade macro-voids before full saturation of micro-voids).



(a)



(b)

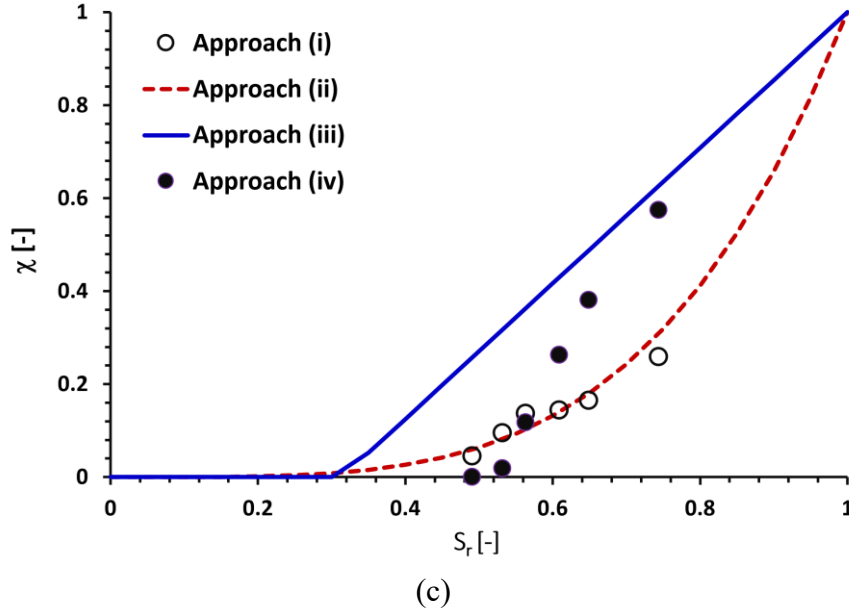


Fig. 4.6 χ parameter as a function of the degree of saturation of the three soils: (a) Soignies, (b) Kruibeke, (c) Tournai according to four different approaches: (i) From uniaxial compression strength at various suction; (ii) Fitting through a power law of the point obtained by approach (i); (iii) from Eq. (2) assuming the void ratio remains constant upon drying; (iv) from Eq. (2) with varying macro void ratio upon drying.

4.6 Conclusion

Using a generalized effective stress formulation for unsaturated soils requires the calibration of the effective stress parameter, the so-called χ parameter. In this chapter, the χ parameter is expressed as a power law of the degree of saturation. The calibration protocol used triaxial compression tests under saturated conditions to obtain the intrinsic (effective) shear failure criterion, defined by the effective cohesion and friction angle. Then, a series of uniaxial compression tests at various suctions (corresponding to various degrees of saturation) were used to find one value of χ per value of degree of saturation, to shift the uniaxial compression strength onto the intrinsic failure criterion expressed in effective stress. The experimental results for these tests are presented in Chapter 3. Finally, the power law has been calibrated to find the best fit between the χ parameters and the degrees of saturation.

The exponent of the power law controls the rate at which the χ parameter increases with the degree of saturation. Upon a low degree of saturation, when the water is essentially stored in the micro-porosity, the χ parameter is insignificant because this “intra-aggregate” water does not contribute to the macroscopic stress. On the contrary, upon a higher degree of saturation, water floods the “inter-aggregate” voids, contributing more to the increase of the effective stress (i.e., becomes significant).

To investigate this microstructural perspective, pore size distributions obtained from mercury intrusion porosimetry were analyzed considering their bi-modal character. When the degree of saturation remains below the microstructural degree of saturation, required to saturate the micro-porosity, the χ parameter is negligible, while when the degree of saturation overpasses the microstructural degree of saturation, water invades the macro-porosity and the χ parameter increases more significantly.

This method has been applied to three compacted clayey soils studied in this thesis. At the end, the proposed methodology provides a simple but efficient and comprehensive protocol to calibrate the χ parameter based on the assumption of the uniqueness of the shear failure criterion when expressed in the generalized effective stress reference.

5. EXPERIMENTAL INVESTIGATION OF DESICCATION BEHAVIOUR

5.1 Introduction

Desiccation cracking of clayey soils is a widespread phenomenon encountered in many geo-environmental and geotechnical systems such as compacted liners, embankments, earth dams, and engineered barriers. During drying, evaporation induces an increase in suction and volumetric shrinkage. When shrinkage is restrained by boundary conditions, tensile stresses develop within the soil mass, and cracking initiates once these stresses exceed the soil tensile strength. The formation of desiccation cracks significantly alters both the mechanical integrity and the hydraulic performance of the soil by reducing stiffness and strength while increasing permeability (Costa et al. 2013; Morris et al. 1992b; Tang et al. 2015).

Because tensile strength controls the initiation and propagation of cracks, its accurate measurement is essential for understanding and modelling desiccation behaviour. Nevertheless, most conventional laboratory techniques determine tensile strength either indirectly (e.g., flexure or Brazilian tests) or directly under nearly constant water content or suction conditions. These approaches do not reproduce the actual stress path experienced during natural desiccation, where suction increases continuously, and tensile stresses develop progressively. Consequently, the measured strength may not be representative of field conditions (Kodikara and Costa 2013; Nahlawi et al. 2004; Rodríguez et al. 2007).

To address these limitations, (Varsei et al. 2016) developed a first-generation desiccation apparatus that enables direct measurement of tensile forces generated in a compacted soil layer during drying. By dividing the mould into two halves connected through load cells and constraining the specimen to contract predominantly in one direction, the desiccation problem can be approximated as a one-dimensional elemental test with a nearly uniform tensile stress field. This configuration allows direct quantification of tensile force at the moment of crack initiation, representing a significant improvement over previous boundary-value tests.

More recently, similar concepts have been used to directly investigate tensile strength evolution during desiccation under different environmental conditions, further highlighting the importance of measuring strength along the actual drying path rather than under constant suction states (Salimi et al. 2021). Despite these advances, many studies still rely either on global force measurements without detailed deformation tracking or on image-based crack observations without simultaneous stress measurements. A combined approach that quantifies both the mechanical response and the full-field kinematics is therefore required to better interpret the mechanisms governing crack initiation and propagation. From an experimental perspective, this philosophy aligns with the use of well-controlled elemental testing and detailed monitoring advocated in previous large-scale desiccation studies (Lakshmikantha 2012).

Accordingly, the objective of this chapter is to present an improved experimental methodology for characterising tensile strength and crack development of compacted clayey soils during desiccation. Building upon the concept proposed by (Varsei et al. 2016), an aluminium desiccation box was designed consisting of a fixed half and a low-friction sliding half mounted on a rail system and connected through force sensors to directly measure tensile forces generated by restrained shrinkage. In addition, an overhead imaging system was employed to acquire time-lapse photographs throughout the test, and Particle Image Velocimetry (PIV) techniques were implemented to quantify full-field surface displacements and strain localisation. This coupled mechanical–kinematic monitoring enables simultaneous evaluation of force–time response, tensile stress evolution, and crack initiation and propagation under controlled boundary conditions.

The following sections describe the experimental apparatus, testing procedure, image analysis methodology, and the resulting mechanical and deformation responses observed during desiccation.

5.2 Experimental Apparatus

5.2.1 Design concept

The experimental device developed in this study is designed to directly measure the tensile forces generated in a compacted clayey soil layer during desiccation. The design is based on the principle that, if soil shrinkage is restrained in one direction, tensile stresses progressively develop and can be quantified from the reaction force required to prevent contraction. By

reducing the desiccation problem to an approximately one-dimensional configuration, the tensile stress at crack initiation can be directly determined.

This concept follows the constrained desiccation approach initially proposed by (Varsei et al. 2016), in which the soil specimen is placed inside a split mould connected through force sensors, enabling direct measurement of tensile force during drying. The present apparatus extends this concept by incorporating a low-friction sliding system and synchronized force–image acquisition to improve measurement accuracy and enable simultaneous mechanical and kinematic monitoring.

5.2.2 Aluminium desiccation box

The main component of the setup is a rigid rectangular aluminium box composed of two independent halves. Aluminium was selected to ensure high stiffness, dimensional stability, and minimal deformation during testing.

One half of the box is fixed to the supporting frame, while the other half is free to translate horizontally. The soil specimen is compacted across the entire box, creating a continuous layer that bridges the two halves. During drying, the shrinkage of the soil induces tensile forces that act to pull the two halves toward each other. These forces are transferred directly to the force sensors installed between the two sections.

To promote a controlled stress state and prevent detachment at the boundaries, mechanical anchors (screws) are embedded at the end walls, ensuring that shrinkage results in internal tensile stress development rather than slippage along the boundaries (Fig. 5.1). This configuration favours the formation of a dominant tensile crack near the mid-length of the specimen, where the tensile stress is expected to be maximum. To ensure proper restraint, screws were embedded into the end walls to anchor the soil and prevent boundary slippage during shrinkage.

The internal dimensions of the aluminium box are 303 mm in length, 220 mm in width, and a specimen thickness of 30 mm, defining the specimen geometry and the cross-sectional area used for stress calculations. Unless stated otherwise, all reported dimensions correspond to the internal mould dimensions defining the soil specimen geometry. The aluminium wall thickness is 10 mm, resulting in external dimensions that are 20 mm larger in both plan directions. The internal and external geometrical dimensions of the desiccation box used in this study are summarised in Table 5.1.



Fig. 5.1 Constrained desiccation box showing the end-wall screws used to anchor the soil.

Table 5.1 Geometrical characteristics of the desiccation box

Parameter	Symbol	Value
Internal length	L	303 mm
Internal width	W	220 mm
Specimen thickness	H	30 mm
Aluminium wall thickness	t	10 mm
External length	L_e	323 mm
External width	W_e	240 mm
Cross-sectional area used for stress calculation ($W \times H$)	A	6600 mm ²

5.2.3 Sliding rail system

To minimize parasitic friction and ensure that measured forces correspond primarily to soil tensile resistance, the movable half of the box is mounted on a rail system equipped with roller or ball bearings.

Reducing base friction is essential to approximate a uniaxial stress condition and to avoid overestimation of tensile forces. The rail system, therefore, improves the reliability and repeatability of force measurements compared with configurations in which the box slides directly on a rigid surface.

5.2.4 Force measurement system

Tensile forces generated during desiccation are measured using two inline load cells, each with a capacity of 50 kg, installed between the fixed and movable halves of the box. The sensors continuously record the reaction force required to restrain soil contraction. A schematic view of the force measurement system, along with the location of the load cells, is shown in Fig. 5.2.

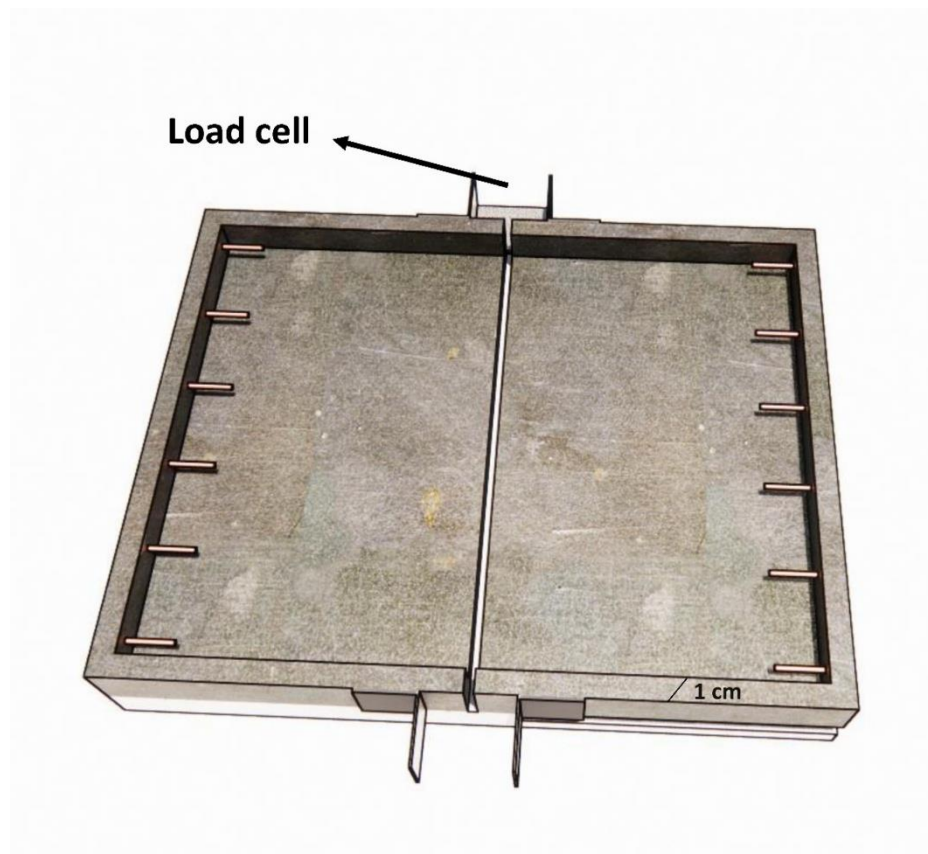


Fig. 5.2 Schematic illustration of the split desiccation box showing the position of the inline load cells and end-wall screws.

The total tensile force is obtained by summing the readings of both sensors. It is subsequently converted into average tensile stress by dividing by the cross-sectional area of the soil specimen. This direct force measurement approach enables identification of the onset of tensile stress development, the peak tensile stress, and the stress at crack initiation.

Compared with indirect back-analysis methods commonly used in conventional desiccation tests, this configuration provides a more direct and physically meaningful estimation of soil tensile strength during drying (Salimi et al. 2021; Varsei et al. 2016).

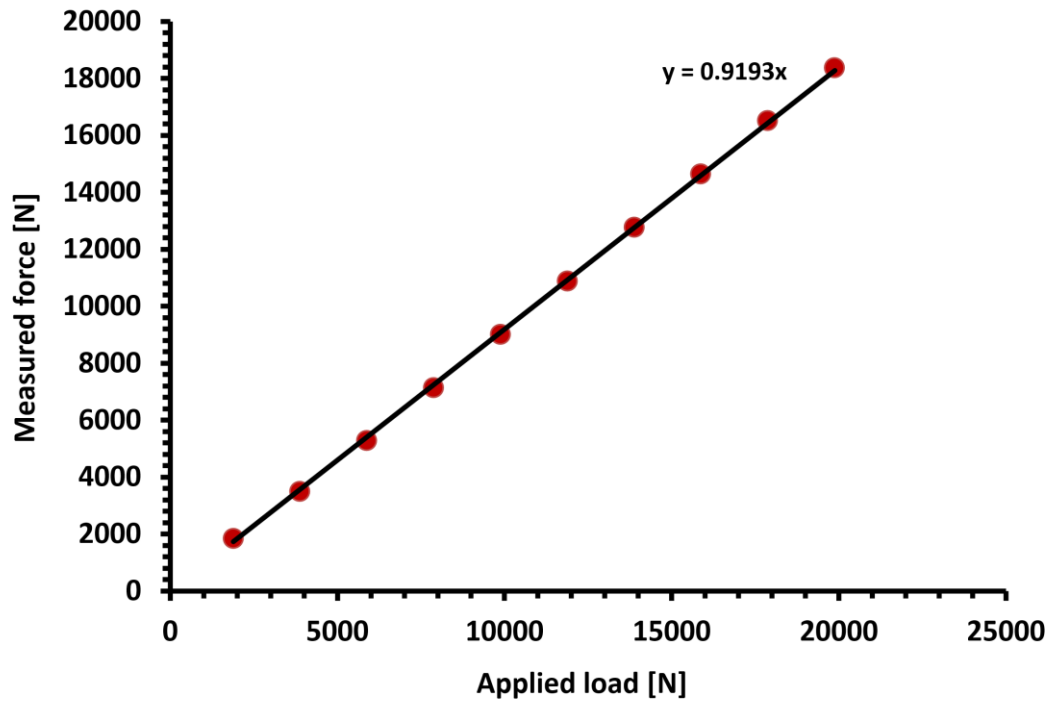
5.2.5 Evaluation and correction of system friction

To assess the friction induced by the mechanical setup (rail and pulley system), a series of blank tests was carried out without soil. Known masses (approximately 2 kg increments) were progressively applied, and the corresponding forces were recorded using the two sensors. The measured force corresponds to the sum of both sensors, while the difference between the applied load and the measured force represents the friction of the system.

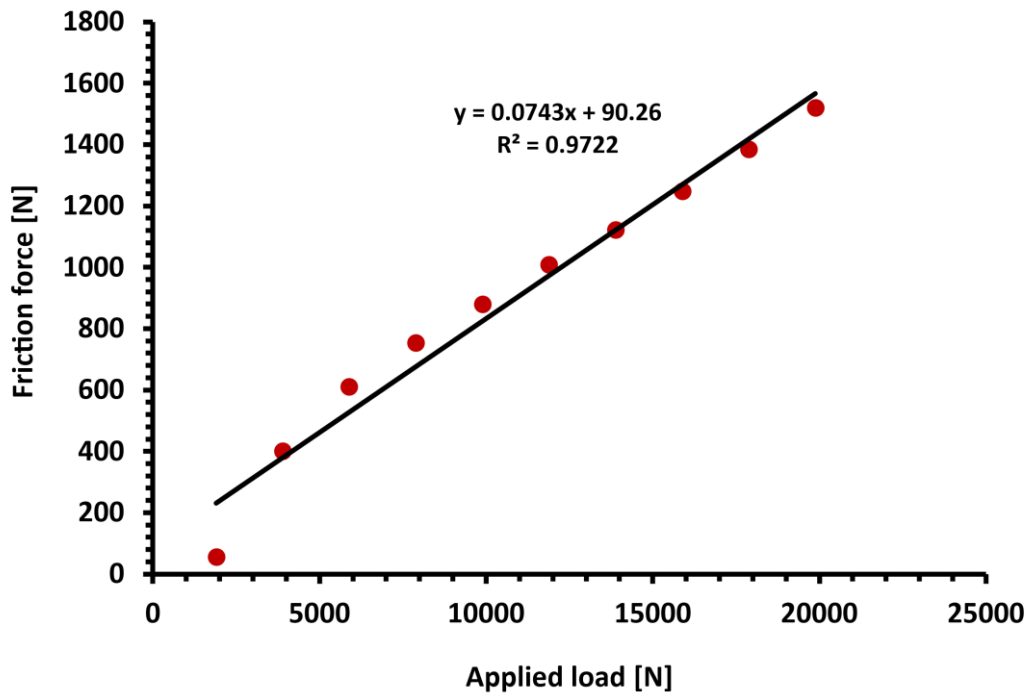
The results indicate that the friction is not constant but increases proportionally with the applied load. A nearly linear relationship was observed between the applied load and the friction force, with a high correlation ($R^2 \approx 0.96$). The friction represents approximately 5–10% of the applied load, highlighting a load-dependent behaviour rather than a constant offset. Figure 5.3 (a) shows the relationship between the applied load and the measured force recorded by the two load cells. The corresponding friction force, calculated as the difference between the applied and measured forces, is shown in Figure 5.3(b).

In addition, a difference between loading and unloading paths was observed, reflecting a hysteresis effect in the mechanical system. Since the experimental tests correspond to a loading configuration, only the loading branch was considered for calibration.

Based on these observations, a friction correction was applied to all measured forces. The calibration was performed using the loading curve, and the corrected values were used to compute both the tensile force–time and tensile stress–time responses presented in the following sections. This procedure ensures that the reported results are not overestimated by the setup's mechanical resistance.



(a)



(b)

Fig. 5.3 Calibration of the loading system: (a) relationship between the applied load and the measured force recorded by the load cells; (b) relationship between the applied load and the corresponding friction force used for force correction.

5.2.6 Imaging and monitoring system

An overhead imaging system is installed above the apparatus to monitor surface deformation and crack development throughout the test. A digital camera is rigidly mounted on a fixed frame with the optical axis oriented perpendicular to the soil surface to avoid geometric distortion. Images are captured automatically at regular time intervals during drying.

Uniform overhead lighting was provided using a fixed light panel to ensure homogeneous illumination of the soil surface and consistent image quality for PIV analysis. The acquired images are later analysed using Particle Image Velocimetry (PIV) techniques (see section 5.4) to quantify full-field displacement and strain localisation associated with shrinkage and crack initiation.

This combined mechanical–optical configuration enables simultaneous measurement of:

- force–time response,
- shrinkage deformation, and
- crack evolution.

Such synchronized monitoring provides a more comprehensive characterization of the desiccation cracking process than approaches relying solely on either force or visual observations. A schematic overview of the desiccation apparatus and its main components is presented in Fig. 5.4.

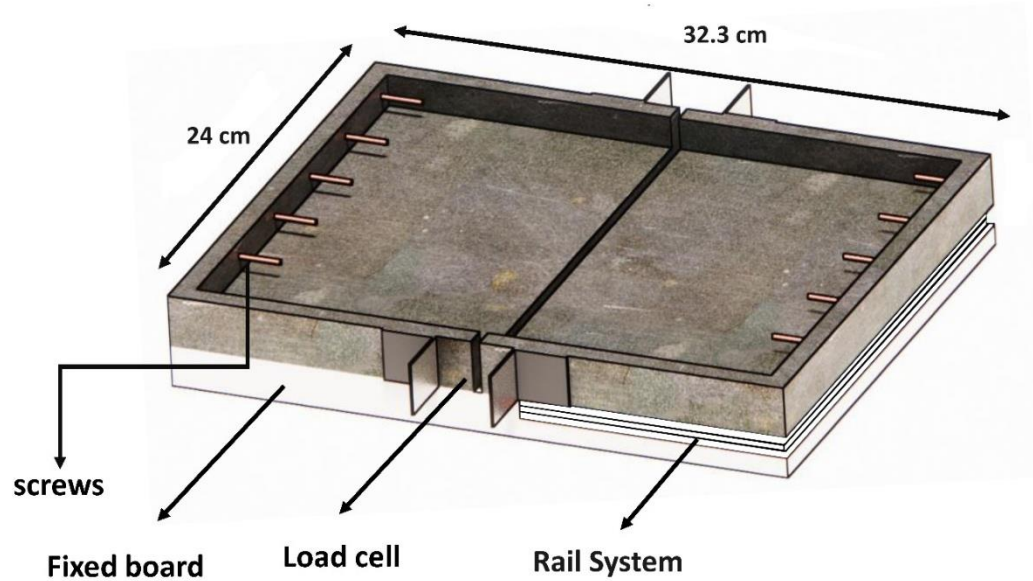


Fig. 5.4 Schematic illustration of the constrained desiccation box showing the split aluminium mould, external dimensions, end-wall screws used to anchor the soil, sliding rail system, and inline load cells.

5.3 Testing Procedure

The desiccation tests were conducted following a standardized and repeatable procedure to ensure consistency between specimens and reliable comparison of results. Each test comprised four main stages: soil preparation, specimen installation, data acquisition during drying, and post-processing.

5.3.1 Soil preparation

Before testing, the soil was air-dried, gently pulverised, and sieved to remove large aggregates and ensure uniformity. The required amount of water was then added to reach the optimum water content determined from the Standard Proctor compaction tests. The wetted soil was thoroughly mixed and sealed in airtight containers for at least 24 hours to allow moisture equilibration and homogeneous water distribution.

Based on the target dry density obtained from the Proctor results and the known specimen volume, the required soil mass was calculated to ensure accurate control of density. The pre-weighed soil was placed directly inside the aluminium desiccation box and statically compacted in successive layers using a rigid steel plate connected to a hydraulic jack (Fig.

5.5). This controlled compaction procedure ensured uniform density, good contact with the box boundaries, and high repeatability between specimens. The mass of soil placed in the mould was adjusted to match the target Standard Proctor dry density.

The final specimen formed a continuous rectangular layer spanning both halves of the split mould, with a nominal thickness of 30 mm.



Fig. 5.5 Static compaction of the soil specimen inside the aluminium desiccation box using a hydraulic jack and a rigid steel plate to achieve the target dry density.

5.3.2 Installation in the apparatus

Following compaction, the surface of the specimen was carefully levelled to obtain a flat and uniform geometry suitable for optical measurements (Fig. 5.6). The movable half of the box was positioned on the rail system, and the force sensors were connected and checked.

Before the start of each test, the load cells were zeroed to eliminate any initial bias. The imaging system was then aligned such that the camera optical axis was perpendicular to the soil surface. Lighting conditions were adjusted to provide uniform illumination and avoid shadows or reflections that could affect image analysis.

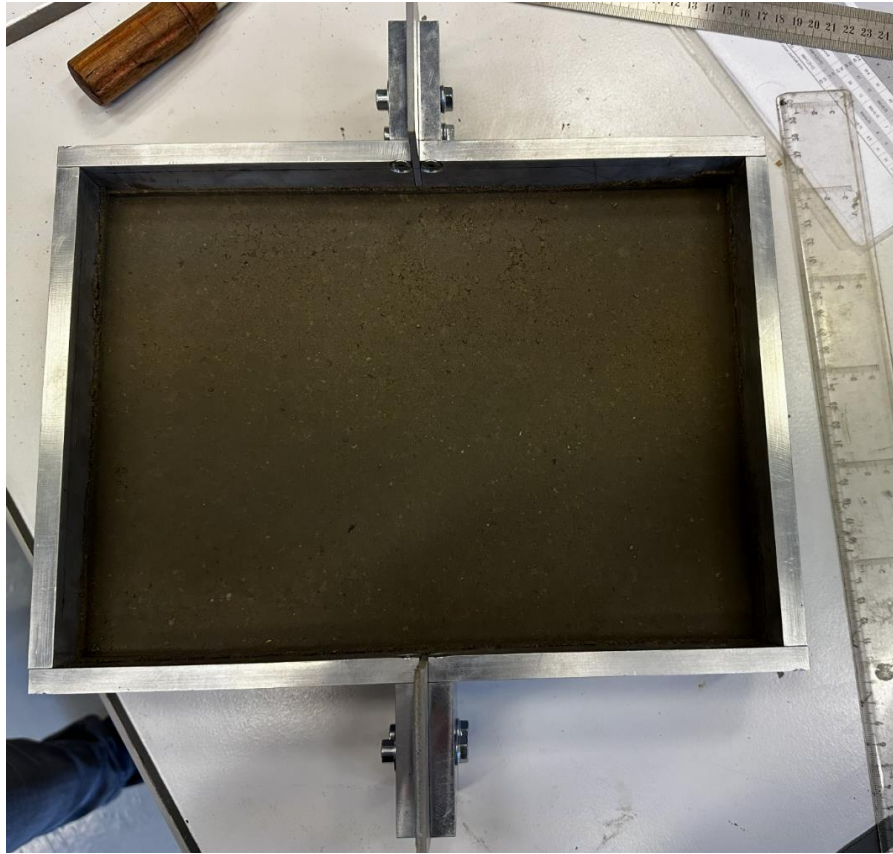


Fig. 5.6 Compacted soil specimen inside the aluminium desiccation box after preparation and surface levelling before testing.

5.3.3 Desiccation and data acquisition

Desiccation was initiated by exposing the specimen to laboratory ambient conditions. No external mechanical load was applied; tensile stresses developed naturally as a result of restrained shrinkage.

During drying, measurements were recorded continuously:

- tensile force from the load cells,
- surface images captured automatically at regular time intervals.

Time-lapse images were acquired every two minutes throughout the test to provide sufficient temporal resolution for displacement tracking and crack detection. Data acquisition continued until either a clear tensile crack developed, indicating that the specimen had reached a near-dry state.

During the test, the load cells and the overhead camera were connected to a PC for continuous data acquisition. Force readings and time-lapse images were recorded automatically and could be monitored in real time to ensure proper operation throughout the

drying stage. A photograph of the complete experimental setup during testing is shown in Fig. 5.7.



Fig. 5.7 Desiccation test setup showing the installed specimen, aluminum box, camera, and lighting system used for continuous monitoring and data acquisition.

5.3.4 Image processing and displacement analysis

The captured images were subsequently processed using Particle Image Velocimetry (PIV) implemented in MATLAB (GeoPIV). Consecutive images were correlated to compute incremental displacement fields across the soil surface.

This analysis enabled quantification of:

- global shrinkage,
- local strain localisation, and
- the timing and position of crack development.

The calibration was performed before testing to convert pixel displacement into physical units, ensuring that measured deformations corresponded to actual specimen movements.

Displacement magnitudes are expressed in millimetres (mm) based on calibration of the image scale.

Before displacement analysis, a calibration procedure was performed to convert image-space measurements (pixel units) into object-space coordinates (millimetres). The calibration was based on the known internal dimensions of the aluminium desiccation box (303 mm × 220 mm), which fully defined the region of interest captured in the images. The number of pixels corresponding to these known distances was measured from the reference image, allowing the pixel-to-millimetre conversion factor to be determined in both horizontal and vertical directions.

Since the camera optical axis was carefully aligned perpendicular to the soil surface and the specimen remained planar during testing, a constant image scale was assumed across the field of view. This assumption is consistent with standard practice in image-based deformation measurements when out-of-plane distortion is negligible (White et al. 2003). More advanced photogrammetric corrections may be required when significant perspective effects or lens distortions are present; however, given the limited field size and central positioning of the specimen within the image, distortion effects were considered negligible in the present setup.

The conversion from image-space to object-space coordinates therefore followed a uniform scaling approach, as commonly adopted in geotechnical PIV applications where planar deformation and controlled imaging geometry are ensured (White et al. 2003; White and Take 2002).

5.4 Particle Image Velocimetry (PIV)

Several image-based techniques have been developed to characterise soil deformation and desiccation cracking across multiple spatial scales. Early approaches relied on direct photographic monitoring combined with image thresholding and segmentation to quantify crack morphology and network evolution (Horgan 1998; PRESTON et al. 1997; Puppala et al. 2004; Sarmah et al. 1996; Velde 1999; Vogel et al. 2005).

Morphological image processing methods based on Minkowski functionals and erosion–dilation operations have also been employed to describe crack topology and connectivity (Holden 2001; Mecke n.d.; Serra 1983). In addition, X-ray microtomography and three-dimensional imaging techniques have been used to investigate internal crack development

during drying (Léonard et al. 2002, 2005), while CCD camera systems have enabled time-lapse monitoring of crack initiation and propagation at the surface (Lecocq and Vandewalle 2002).

For laboratory desiccation experiments, digital camera systems provide a practical and cost-effective solution for continuous crack monitoring (Tang et al. 2008) whereas non-contact laser scanning systems have been used to obtain detailed surface profiles of cracked soils. Although these techniques primarily describe crack morphology, they do not directly quantify displacement and strain fields.

To obtain full-field kinematic measurements, correlation-based methods such as Digital Image Correlation (DIC) and Particle Image Velocimetry (PIV) have been increasingly adopted in geotechnical testing. Originally developed for experimental fluid mechanics (Adrian 1991) these techniques were later adapted for soil deformation measurements through dedicated tools such as GeoPIV (White and Take 2002). Extensive validation studies have demonstrated their high precision and robustness for both small- and large-strain problems (Stanier et al. 2016; Sutton et al. 2009; White et al. 2003, 2001). Among the available techniques, PIV was selected in the present study due to its non-intrusive nature and ability to provide continuous full-field displacement measurements over the entire specimen surface.

Although PIV provides robust and reliable full-field displacement measurements, some limitations should be acknowledged. The technique performs optimally when sufficient surface texture is available for correlation. For the compacted fine-grained clay specimens tested in this study, the relatively smooth and homogeneous surface locally reduced image contrast, making particle tracking more challenging at the pixel scale and potentially introducing minor uncertainties in some regions. Consequently, an optimal mesh (subset) size was selected as a compromise between spatial resolution and correlation stability, as excessively small subsets resulted in increased noise and unreliable displacement estimates. With appropriate calibration and controlled lighting conditions, satisfactory measurement accuracy was nevertheless achieved.

It is also worth noting that the spatial resolution of the imaging system is determined by the pixel density and field of view. While sub-pixel displacement interpolation improves measurement precision, the object-space resolution remains limited by the pixel-to-millimetre scale factor. Consequently, the technique is not intended to resolve very fine crack apertures at sub-millimetre scale directly, but rather to identify displacement discontinuities and strain localisation associated with macroscopic crack initiation and propagation. This limitation is

consistent with established discussions on image-based deformation measurement in geotechnical applications (Stanier et al. 2016; White et al. 2003).

5.4.1 PIV history

The first quantitative measurements of particles by cinematography imaging were conducted by (Nayler and Frazer 1917) to investigate a stormy flow by tracking a model immersed in water. They recorded particle images sequentially on filmstrips. They marked particle points on a piece of paper under a film using a pin to examine the film. They determined streaks by using multi-frame photos. They applied this process to 80 frames of filmstrips. The first work on Particle Tracing Velocimetry (PTV) was conducted manually, which has a dynamic range of 80:1. Figure 5.8 provides an image of this study that is similar to modern articles' results qualitatively and in detail. It shows that most of the advances in PTV are due to the computerized tracking process.

Many studies are conducted on measuring particle velocity to estimate the fluid flow field or multi-phase flow. Particle Streak Velocity (PSV), Laser Speckle Velocimetry (LSV), Particle Image Velocimetry (PIV), and Particle Tracing Velocimetry are specified names of this technique.

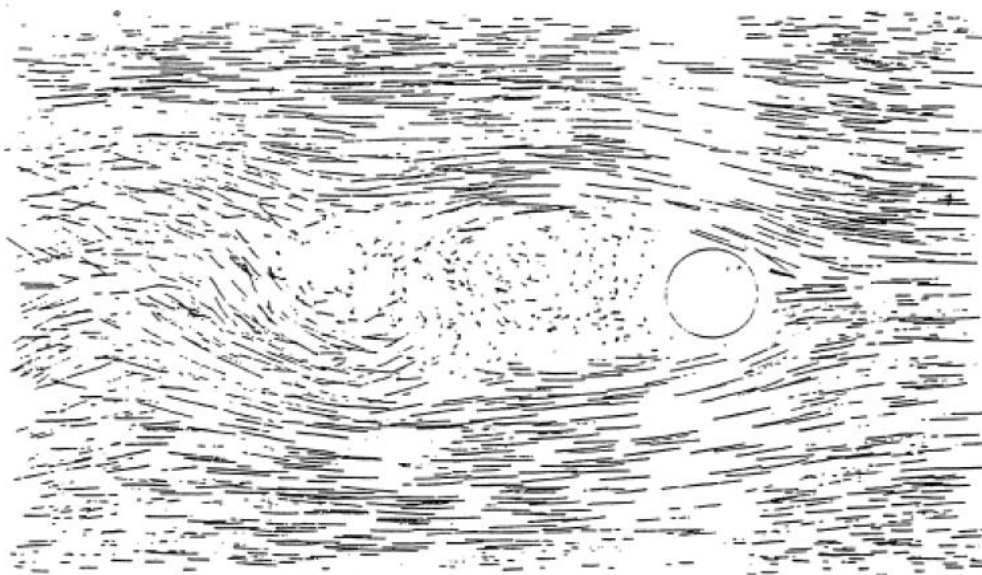


Fig. 5.8 Measurements of particle tracking of 2D unsteady flow around a cylinder by using cinematography (Nayler and Frazer 1917)

Other names such as Pulsed Light Velocimetry (PLV), Particle Image Displacement Velocimetry (PIDV), Particle Displacement Velocimetry (PDV) and Digital Particle Image

Velocimetry (DPIV) refer to similar content with minimal changes. Holographic Particle Image Velocimetry (HPIV) and Photogrammetric Particle Tracking Velocimetry (PPTV) are 3D methods that can be used with LSV, PIV and PTV, (Ansaripour 2014). Particle Image Velocimetry was applied for the first time by (Adrian 1991) on fluid mechanics in order to measure velocity fields of 2D and 3D flows (Ansaripour 2014). Adrian states that PIV is used in measuring the displacement of instantaneous fields of gas and fluid flows in laboratory scale with high quality and precision. Also, he states that this technique is used in millimeter domains per second to a few hundred meters per second.

5.4.2 Application of PIV in geomechanics

Particle Image Velocimetry was originally developed for experimental fluid mechanics (Adrian 1991). Its application to geotechnical testing was significantly advanced by (White et al. 2003, 2001), who adapted the technique for measuring soil deformation in laboratory model and element tests. In geotechnical applications, soil deformation can be interpreted as a low-velocity flow process, allowing PIV principles to be applied to track the movement of soil patches across successive digital images (White et al. 2001; White and Take 2002).

Unlike fluid mechanics experiments, where artificial tracer particles and laser illumination are often required, soil specimens generally possess sufficient natural texture due to grain contrast and surface roughness. This characteristic simplifies image acquisition and reduces the need for intrusive markers or complex optical setups. The development of GeoPIV software further facilitated systematic displacement measurement in geotechnical testing (White and Take 2002).

Since its introduction, PIV has been widely used in geotechnical research to investigate deformation mechanisms in model foundations, retaining structures, penetration problems, and seismic loading scenarios. For example, (Yamamoto and Kusuda 2001) employed image analysis to study reinforced foundation behaviour, while (Knappett et al. 2006) used PIV to examine shallow foundation failure under seismic loading. These studies demonstrate the capability of PIV to provide detailed full-field displacement data that would be difficult to obtain using conventional instrumentation.

Successful application of PIV relies on the presence of sufficient surface texture to ensure reliable correlation between successive images. Texture refers to spatial variations in pixel intensity within each subset, which enable unique identification of soil patches during tracking

(White et al. 2001). For granular soils, natural grain contrast generally provides adequate texture. In contrast, compacted fine-grained clays may exhibit relatively smooth and homogeneous surfaces, potentially reducing correlation quality. Therefore, appropriate subset size selection is essential to balance spatial resolution and measurement stability.

5.4.3 Image texture

In geotechnical PIV applications, the image is divided into a mesh of small regions referred to as patches (or subsets). Each patch represents a group of neighbouring pixels whose intensity pattern is tracked between successive images. The displacement of each patch is determined by identifying, in the subsequent image, the location that maximizes its correlation with the reference patch (White et al. 2001; White and Take 2002). This patch-based tracking approach forms the core of the adaptation of PIV for geotechnical testing, where soil deformation is interpreted as a low-velocity flow process.

Successful application of PIV requires sufficient texture within each patch to ensure reliable correlation. Texture refers to spatial variations in pixel intensity that allow each patch to be uniquely identified during tracking (White et al. 2001).

5.4.4 Particle Image Velocimetry

In the process of image analysis by PIV, images are taken using a digital camera from soil during deformation, and then soil deformation is determined using particle image velocimetry between a pair of consecutive images.

Image processing that is done during PIV for measuring displacement between a pair of digital images is shown in Fig. 5.9. First, the image is divided into a network of test patches and each test patch ($I_{\text{test}}(U)$) includes an image matrix of $L \times L$ pixel ($I(U)$). A search patch is selected in second image ($I_{\text{search}}(U+S)$) to find test patch displacement between images 1, 2; the search patch defines an area that test patch searches through, expanding the test patch up to S_{max} in the u, v directions (test patch (first rectangular with L size) plus a margin that is equal to the maximum vertical and horizontal displacement vector). Then it searches for the test patch location in the next image using a function that is defined as the fitness function and selects a patch of the second image as the second location of the first image test patch, for which the fitness function is maximized; also, it introduces a motion vector between the two patch centers as the displacement vector. (White et al. 2003; White and Take 2002). Image processing will be done using MatPIV and GeoPIV software based on MATLAB.

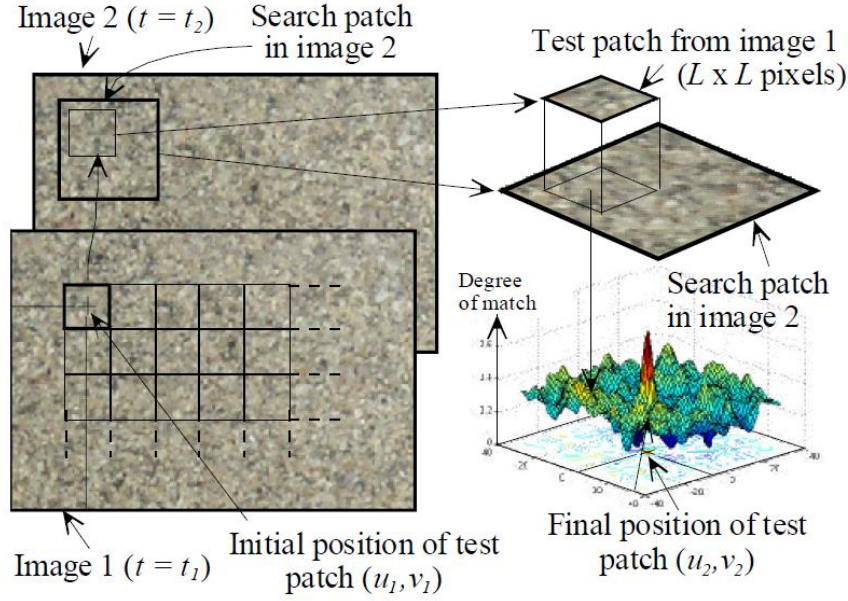


Fig. 5.9 Principles of PIV analysis (White and Take 2002)

The software outputs are a 2D matrix of u , v ; the first one presents the horizontal component, and the second one presents the vertical component of each patch displacement vector.

System precision in PIV depends on patch size and image texture. Smaller patches improve spatial discretization of the displacement field but may reduce correlation robustness due to limited intensity information, whereas larger patches enhance correlation stability at the expense of spatial resolution (White et al. 2003).

5.4.5 Centroiding

Centroiding is one of the image processing steps and determines the special location of reference points. Corresponding pixels and reference points are specified by selecting the threshold intensity level, and the target location is determined as the selected pixel's center surface in both directions. Determining reference points' location in real space and the points' location on the image, image pixel size is determined for each test, and measurements on image space in pixels are converted into length in point units (Stanier et al. 2016).

5.5 Experimental Results

5.5.1 Overview of the testing programme

To assess repeatability and account for experimental variability, multiple tests were performed under identical preparation and boundary conditions. All specimens were prepared

using the same compaction procedure, geometry, and measurement settings to ensure consistent comparison between tests.

Two replicate tests were performed for each boundary condition, resulting in four tests per soil. A total of twelve desiccation tests were conducted on three different clayey soils. In the constrained configuration, screws installed along the side walls of the desiccation box remained in contact with the soil throughout drying, restricting horizontal shrinkage and promoting the development of tensile stresses. In contrast, the unconstrained configuration allowed free shrinkage without mechanical restraint.

For each test, tensile force and time-lapse images were continuously recorded. The measured force was subsequently converted into tensile stress, while the captured images were analysed using Particle Image Velocimetry (PIV) to quantify full-field displacements and strains. The main characteristics of the desiccation test programme, including soil type, boundary condition, and specimen density, are summarised in Table 5.2.

The measured tensile strengths and observed crack patterns for each test are presented and discussed in the following sections.

Table 5.2 Summary of desiccation test programme

Test name	Soil	Boundary condition	Bulk density (g/cm ³)
<i>S1</i>	Soignies	Constrained	2.03
<i>S2</i>	Soignies	Constrained	2.03
<i>S3</i>	Soignies	Unconstrained	2.03
<i>S4</i>	Soignies	Unconstrained	2.03
<i>K1</i>	Kruikebe	Constrained	1.84
<i>K2</i>	Kruikebe	Constrained	1.84
<i>K3</i>	Kruikebe	Unconstrained	1.84
<i>K4</i>	Kruikebe	Unconstrained	1.84
<i>T1</i>	Tournai	Constrained	1.54
<i>T2</i>	Tournai	Constrained	1.54
<i>T3</i>	Tournai	Unconstrained	1.54
<i>T4</i>	Tournai	Unconstrained	1.54

5.5.2 Evolution of tensile force during drying

The tensile forces generated by restrained shrinkage were continuously monitored using two inline load cells installed between the fixed and movable halves of the desiccation box. The total tensile force was calculated as the sum of the forces recorded by both sensors.

As an example, Fig. 5.10 shows the evolution of the individual forces measured by each load cell (F1 and F2), together with the corresponding total force, for the test S2. The two sensors exhibited comparable magnitudes and similar trends throughout the test, indicating symmetric load transfer and proper alignment of the apparatus. For clarity, subsequent analyses are based solely on the total tensile force.

During drying, the tensile force generated by restrained shrinkage progressively increases as the soil undergoes volumetric contraction. This increase reflects the gradual development of tensile stresses within the specimen due to the restriction of shrinkage imposed by the boundary conditions. In all tests, a distinct peak in tensile force is observed, followed by a sudden drop corresponding to crack initiation and the associated release of tensile stress within the soil mass.

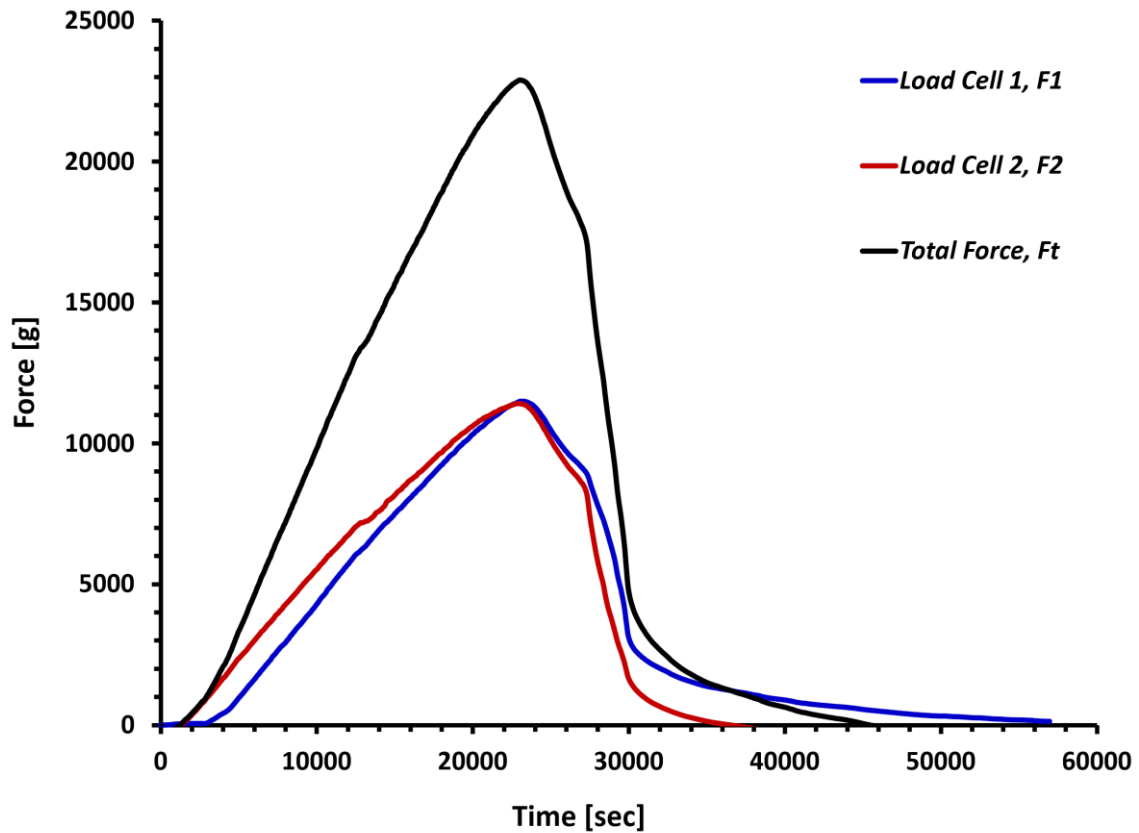


Fig. 5.10 Evolution of the forces measured by the two load cells (F1 and F2) and the corresponding total tensile force during a desiccation test of S2.

5.5.3 Tensile stress–time behaviour and crack development

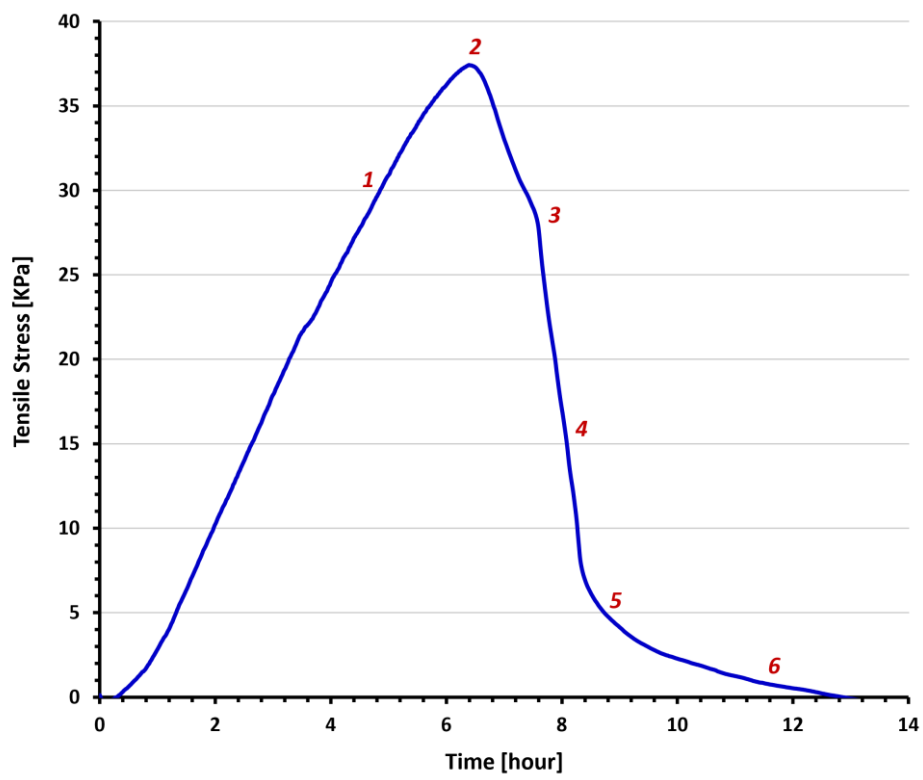
The tensile stress evolution and peak tensile strengths obtained from the load cell measurements are presented in this section. To enable comparison between tests with identical geometry, the measured tensile force was converted into average tensile stress by dividing the total force by the cross-sectional area of the soil specimen. The cross-sectional area was calculated based on the internal specimen width and thickness (220 mm × 30 mm), resulting in an area of 6600 mm².

To illustrate in more detail the relationship between tensile stress evolution and crack development during drying, a detailed analysis was carried out for test *S2*. Figure 5.11 presents the evolution of tensile stress over time, together with a sequence of surface images captured at different stages of the test. The numbered points on the stress–time curve correspond to the times at which the surface photographs were taken.

At the early stages of drying (points 1–2), the tensile stress increases progressively due to restrained shrinkage while the soil surface still appears visually intact. At point 3, shortly after the peak tensile stress is reached, a first hairline crack becomes visible near the upper boundary of the specimen. As drying progresses (point 4), this crack propagates downward. Subsequently, an additional crack initiates from the lower part of the specimen (point 5) and propagates upward. At the final stage (point 6), these cracks extend further and form a nearly continuous fracture path across the specimen.

This sequence clearly shows that crack initiation occurs before the crack becomes clearly visible at the soil surface, and that visible crack propagation takes place after the peak tensile stress has been reached.

The observed relationship between tensile stress evolution and crack development indicates that crack initiation is mechanically triggered when the shrinkage-induced tensile stress reaches the tensile resistance of the soil. As the stress approaches its peak value, localised deformation and microcracking develop, although they may not yet be clearly visible at the surface. Once the peak stress is attained, unstable crack propagation occurs, leading to a rapid release of tensile stress and a loss of tensile continuity. The post-peak response is characterised by stress relaxation and progressive crack opening, with the extent of stress drop depending on the boundary conditions and the degree of stress accumulation before failure.



(a)



(b)



(c)



(d)



(e)



(f)

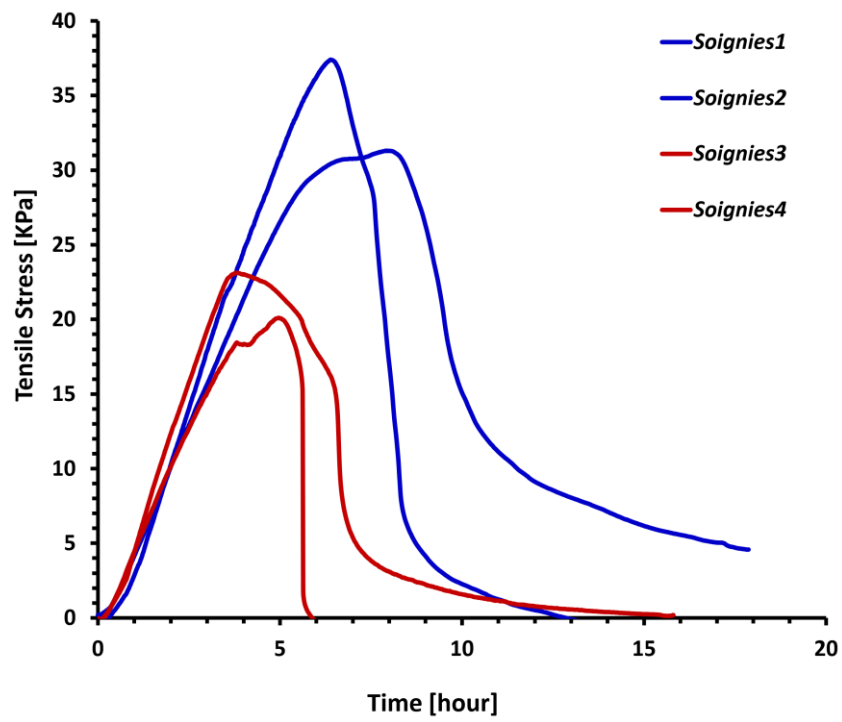


(g)

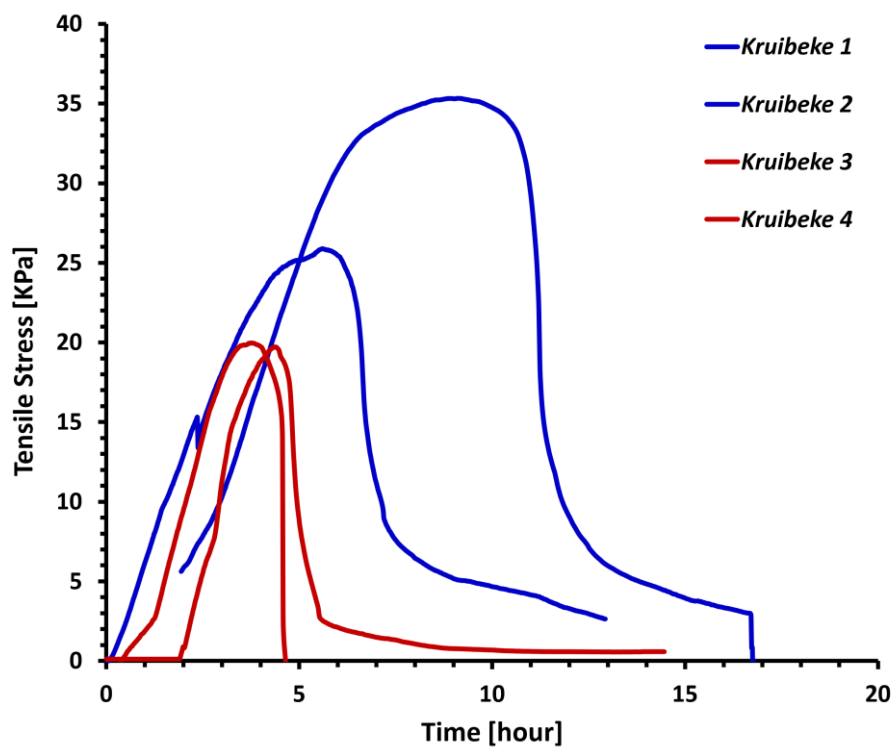
Fig. 5.11 Relationship between tensile stress evolution and crack development during a representative constrained desiccation test. (a) Tensile stress–time curve obtained from the load cell measurements. The numbered points indicate the times at which the surface photographs were captured. (b–g) Sequential images of the soil surface corresponding to the numbered points on the stress curve, showing the progressive development of cracking during drying.

The evolution of tensile stress with time for the different tests is shown in Fig. 5.12. For all specimens, the tensile stress increased progressively during drying until reaching a maximum value, which corresponds to the tensile strength of the soil under desiccation conditions. After crack initiation, the stress rapidly decreased as the specimen lost its tensile continuity.

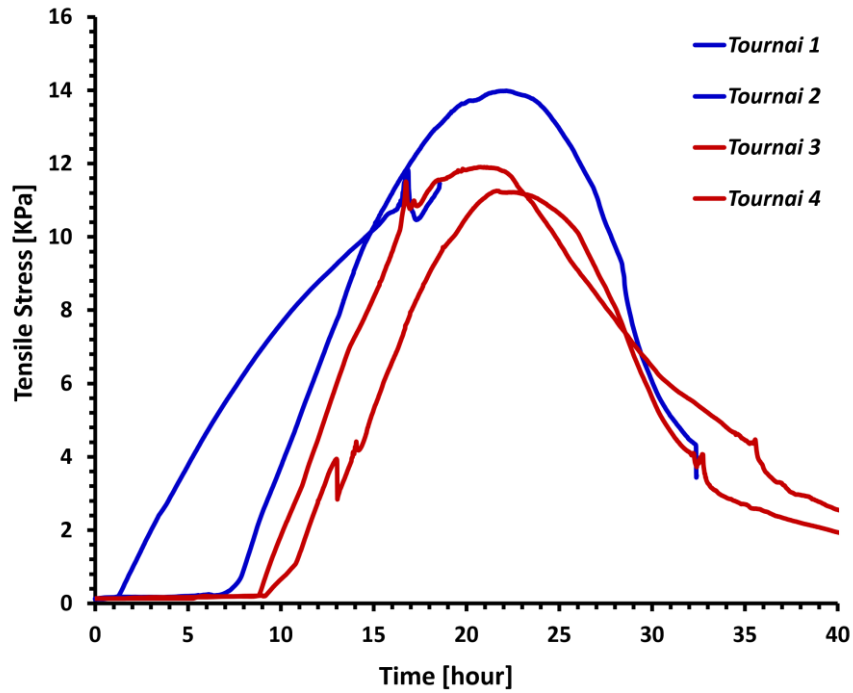
The peak tensile stress values obtained for each test are summarized later in Table 5.3. Differences in tensile strength were observed between soils and boundary conditions, with constrained specimens generally exhibiting higher stress levels due to the imposed restriction of shrinkage.



(a)



(b)



(c)

Fig. 5.12 Evolution of the Tensile Stress during the desiccation test for three tested soils (a) Soignies, (b) Kruibeke, (c) Tournai, for two boundary conditions; Blue and red graphs represent the constrained and unconstrained conditions, respectively.

The tensile stress evolution highlights two fundamentally different cracking mechanisms governing the three materials. For Soignies and Kruibeke, a marked influence of boundary conditions is observed. Under constrained conditions, peak tensile stresses reach approximately 35 kPa for Soignies and 33 kPa for Kruibeke, whereas under unconstrained conditions, significantly lower maxima are recorded (≈ 23 kPa and ≈ 18 kPa, respectively). This substantial reduction demonstrates that external restraint is the primary driver of tensile stress accumulation in these materials. When lateral contraction is restricted, shrinkage strains are converted into tensile stresses, promoting stress build-up until cracking occurs. In the unconstrained configuration, partial stress relaxation is permitted through lateral deformation, limiting the attainable tensile stress. The fact that tensile stress remains non-zero even in the unconstrained case indicates that complete stress release does not occur, most likely due to frictional interaction at the base of the container, which provides the only remaining mechanical restraint.

In contrast, Tournai exhibits comparable tensile stress evolution under both boundary conditions, with peak stresses in the range of 10–13 kPa. The limited sensitivity to external restraint suggests that crack initiation in Tournai is predominantly governed by internal

constraint mechanisms rather than boundary effects. The lower permeability of Tournai promotes stronger moisture gradients and delayed homogenization of suction, leading to differential shrinkage within the specimen thickness. These internal incompatibilities generate tensile stresses independently of lateral boundary restraint. Consequently, the cracking mechanism in Tournai appears to be controlled by hydro-mechanical heterogeneity rather than by externally imposed confinement.

A comparison with independently measured tensile strengths provides additional insight into the fracture process. For Soignies, the maximum tensile stress developed during drying (≈ 35 kPa) is of the same order as the tensile strength obtained from Brazilian tests at initial water content, indicating that crack initiation coincides with the attainment of the intrinsic material tensile resistance. A similar correspondence is observed for Kruibeke. This agreement strongly supports the interpretation that cracking is mechanically triggered once the shrinkage-induced tensile stress reaches the tensile strength threshold, rather than being governed by progressive microcrack accumulation or creep-like relaxation. In Tournai, the peak stress cannot be correlated with the tensile strength as deduced from Brazilian tests, because tensile cracking occurs due to the development of internal stresses, not measured by the external force sensors.

The kinetics of stress development further differentiate the materials. Soignies and Kruibeke reach peak tensile stress within approximately 4–8 hours, whereas Tournai requires nearly 20 hours to attain maximum stress. This delayed response is consistent with the reduced permeability of Tournai, which limits moisture transport and slows suction redistribution. The slower development of hydraulic gradients results in a more gradual accumulation of shrinkage strain and associated tensile stress. Therefore, permeability governs not only the temporal evolution of drying but also the rate at which tensile stresses accumulate, ultimately influencing the onset of cracking.

Overall, the results demonstrate that crack initiation during drying is controlled by a combination of mechanical restraint and intrinsic tensile strength, with the relative contribution of boundary versus internal constraint being strongly material-dependent. Soignies and Kruibeke behave as boundary-controlled systems, whereas Tournai exhibits internally controlled fracture behaviour governed by hydro-mechanical coupling.

5.5.4 Crack initiation and surface observations

Surface observations revealed distinct cracking behaviours depending on soil type and boundary condition. Representative crack patterns obtained for the three soils are presented

in Fig. 5.13.

For the Soignies and Kruibeke soils, visible cracking was observed only under constrained conditions. In these tests, a single dominant crack typically initiated near the centre of the specimen and propagated approximately transverse to the direction of restraint. The crack developed rapidly once the peak tensile stress was reached, leading to a sudden loss of tensile resistance. Under unconstrained conditions, both soils mainly exhibited uniform shrinkage without the formation of distinct surface cracks.

In contrast, the Tournai soil exhibited cracking under both constrained and unconstrained conditions. Instead of a single localized fracture, multiple cracks developed progressively over the surface, forming a distributed crack network without a clear preferential orientation. Crack initiation occurred at several locations, and the pattern evolved gradually during drying. Overall, these observations demonstrate that both soil type and boundary condition strongly influence crack morphology and distribution during desiccation. A more detailed interpretation of the cracking mechanisms and their relationship with tensile stress evolution is provided in the Discussion section (see Section 5.6). A consolidated comparison of cracking behaviour for all specimens is provided in Table 5.3.

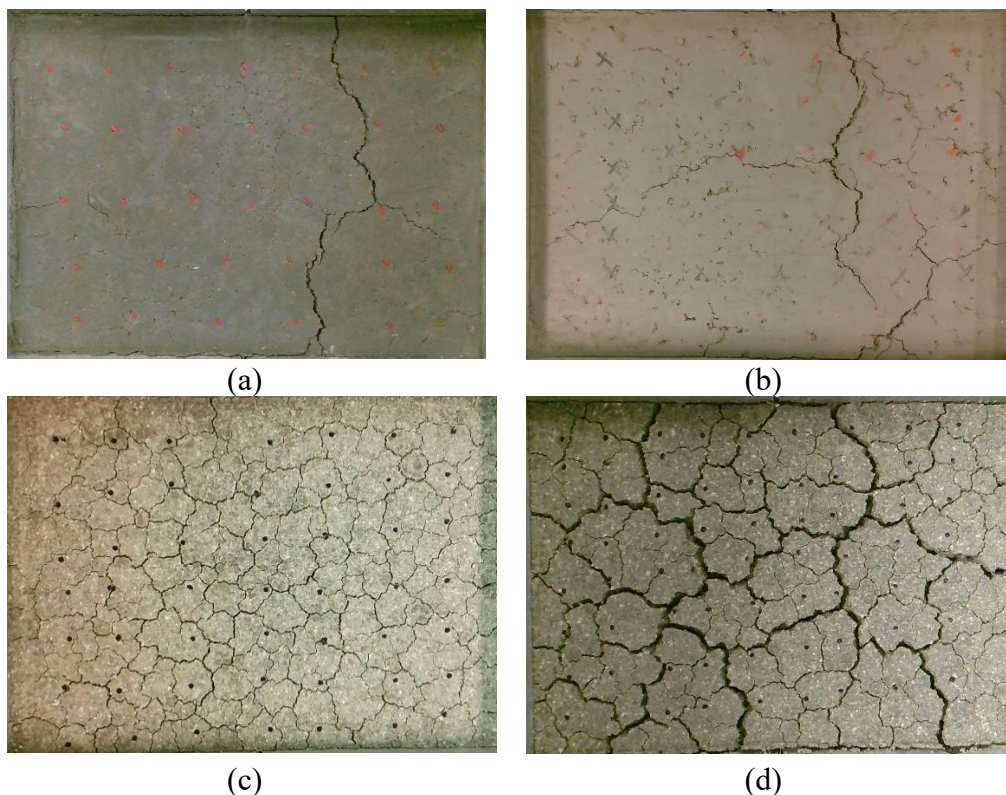


Fig. 13.5 Crack pattern obtained from (a) Soignies Constrained condition (b) Kruibeke Constrained Condition (c) Tournai Constrained Condition (d) Tournai Unconstrained Condition

5.5.5 Full-field displacement and strain analysis using PIV

Full-field surface deformations were quantified for all twelve tests using Particle Image Velocimetry (PIV). The PIV analysis was conducted using both the classical square-subset implementation and the reliability-guided (RG) circular subset approach, depending on the test configuration. For the square implementation, subset sizes ranged between 50×50 and 65×65 pixels. For the RG approach, circular subsets with diameters of 20–25 pixels were employed.

The subset dimensions were selected through preliminary sensitivity analyses for each test to ensure stable correlation and minimize noise while maintaining adequate spatial resolution. Smaller subsets resulted in increased correlation instability, whereas excessively large subsets reduced the ability to capture localized strain gradients. The final subset size adopted for each test corresponded to the optimum compromise between spatial resolution and correlation robustness.

For each experiment, the results are presented using four panels: (a) soil surface before cracking, (b) surface after cracking, (c) horizontal displacement field, and (d) horizontal linear strain field. This representation allows direct comparison between visible crack morphology and the corresponding deformation patterns.

The displacement and strain maps are displayed using a colour scale from dark blue to dark red. Blue indicates extension, whereas red indicates compression. The image analysis was calibrated using a geometric reference, and displacement magnitudes are therefore expressed in millimetres (mm), corresponding to the actual physical movements at the soil surface.

For the Soignies and Kruibeke soils under constrained conditions, the displacement fields show clear non-uniform shrinkage with strong horizontal gradients oriented toward the restrained boundaries. The strain maps reveal a distinct and narrow band of concentrated extension developing progressively near the center of the specimen. This localisation coincides with the position of the single visible crack observed in the post-cracking images. The deformation, therefore, concentrates within one dominant zone before fracture.

Under unconstrained conditions, both soils exhibit comparatively uniform displacement distributions. The strain fields remain diffuse, and no clear localisation bands are observed, which is consistent with the absence of visible cracks and the predominance of homogeneous shrinkage.

A markedly different behaviour is observed for the Tournai soil. For both constrained and unconstrained conditions, the displacement fields display heterogeneous and multi-

directional movements across the entire surface. Instead of a single dominant localisation band, the strain maps show multiple regions of extension distributed throughout the specimen. These zones correspond to the formation of a network of interconnected cracks rather than one localised fracture.

This distributed kinematic response is consistent with the progressive and spatially widespread cracking observed in the surface images, indicating that deformation is not concentrated in a single region but develops simultaneously at several locations.

Overall, the PIV analysis confirms that deformation localisation strongly depends on soil type and boundary condition. While Soignies and Kruibeke exhibit localised strain leading to a single dominant crack only under constrained conditions, the Tournai soil shows distributed strain and cracking even without external restraint. The full set of PIV results for the twelve tests is presented in Figures 5.14 to 5.25.

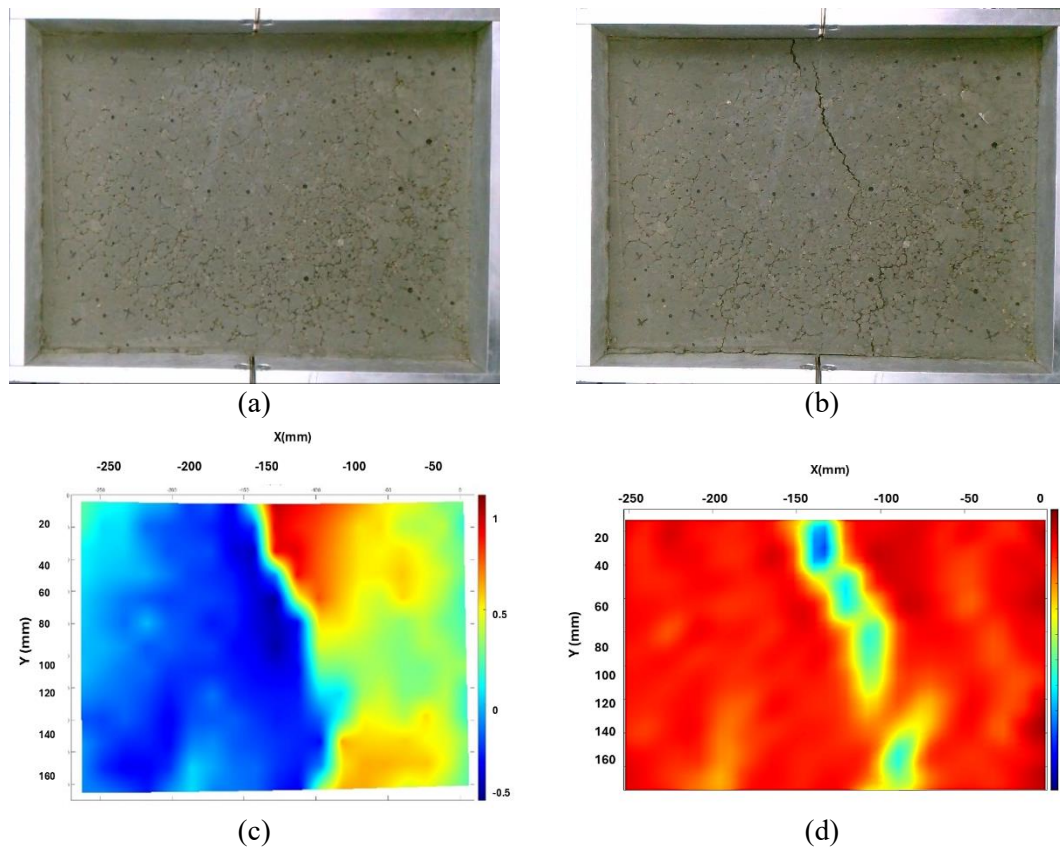


Fig. 5.14 Desiccation test results for *SI* under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

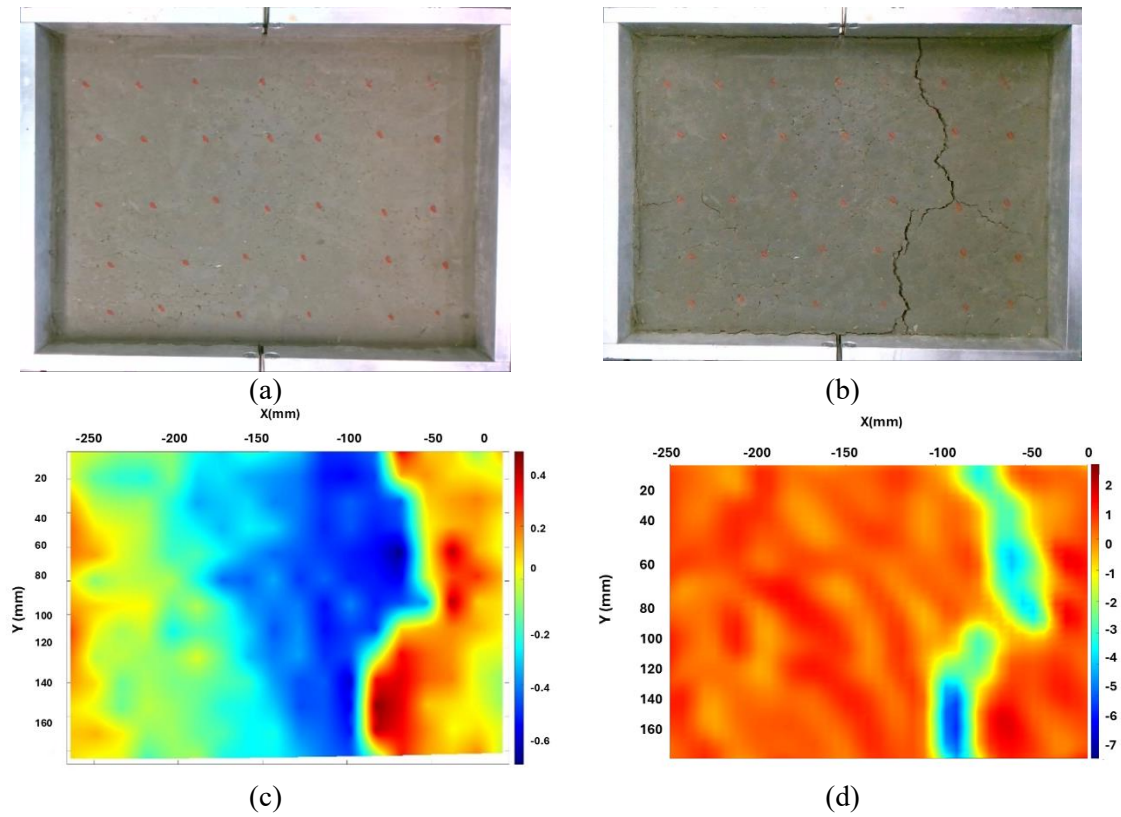


Fig. 5.15 Desiccation test results for *S2* under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

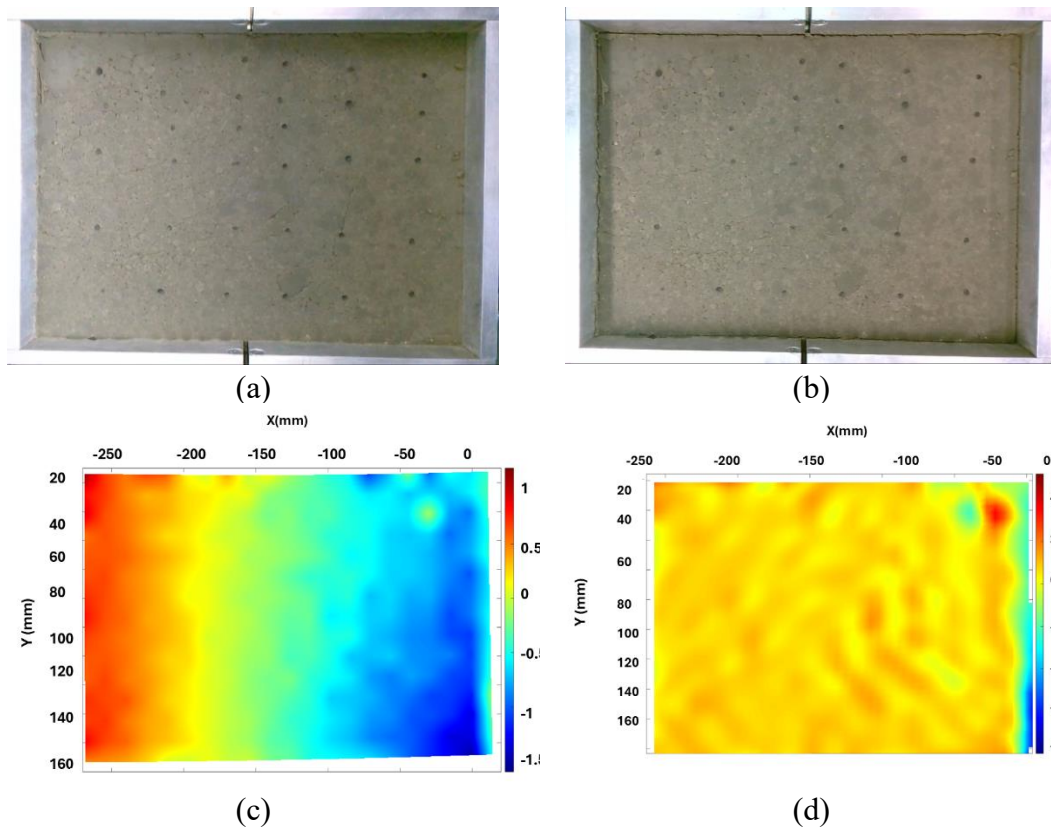


Fig. 5.16 Desiccation test results for *S3* under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

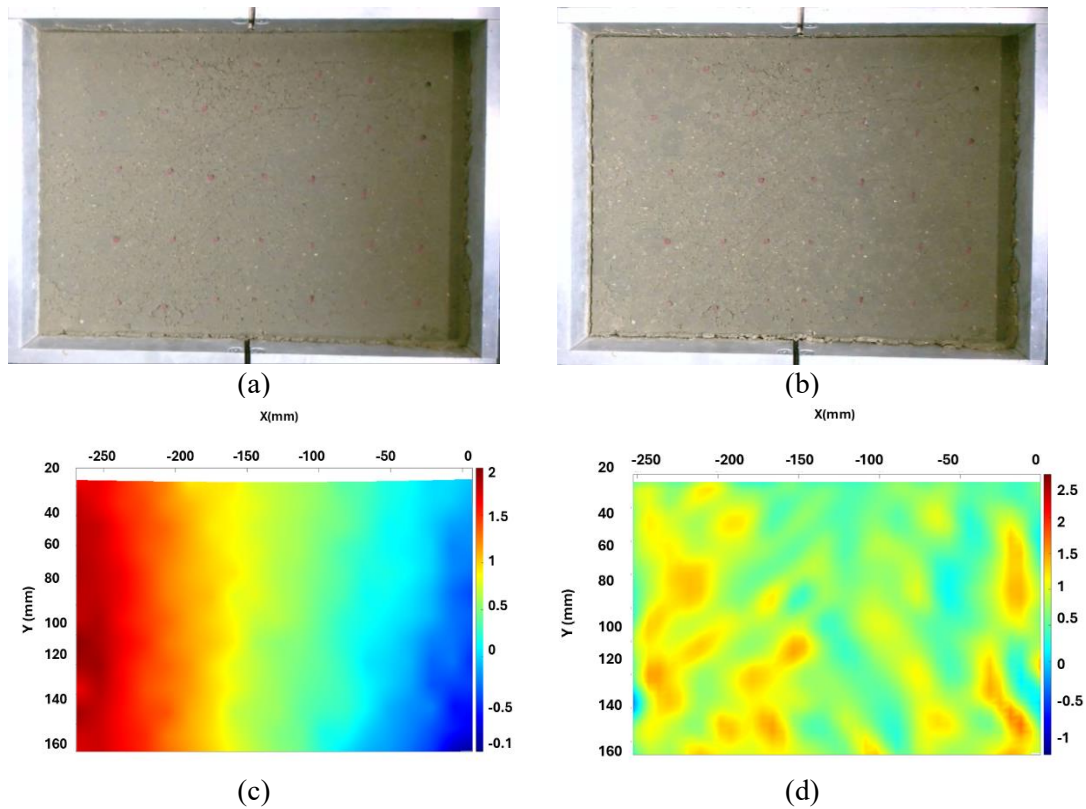


Fig. 5.17 Desiccation test results for *S4* under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.



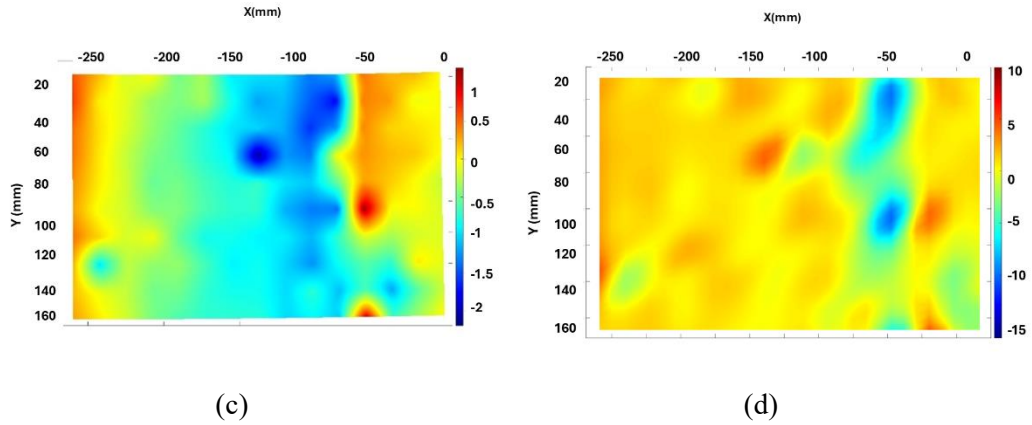


Fig. 5.18 Desiccation test results for $K1$ under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

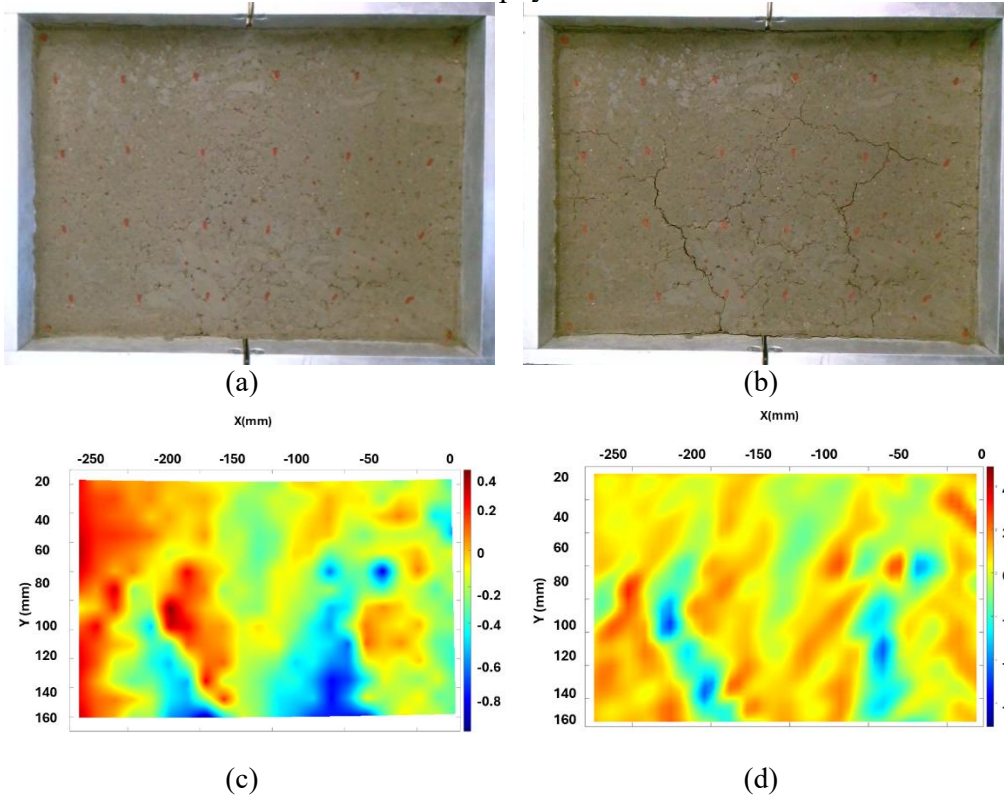


Fig. 5.19 Desiccation test results for $K2$ under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

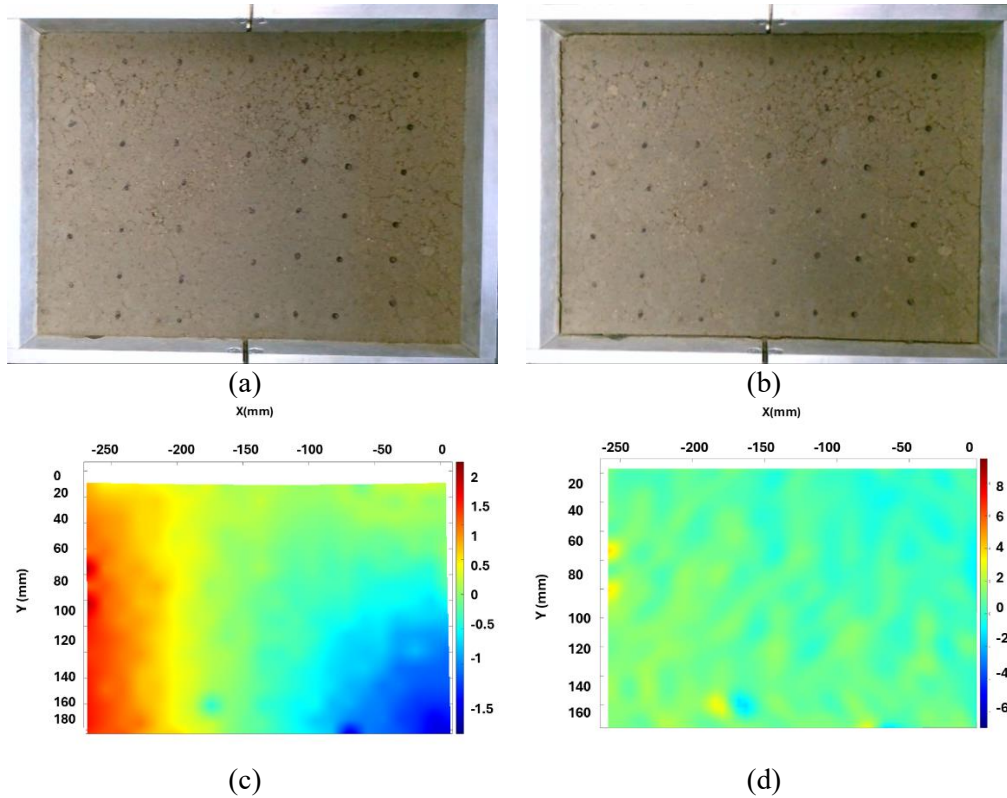


Fig. 5.20 Desiccation test results for *K3* under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

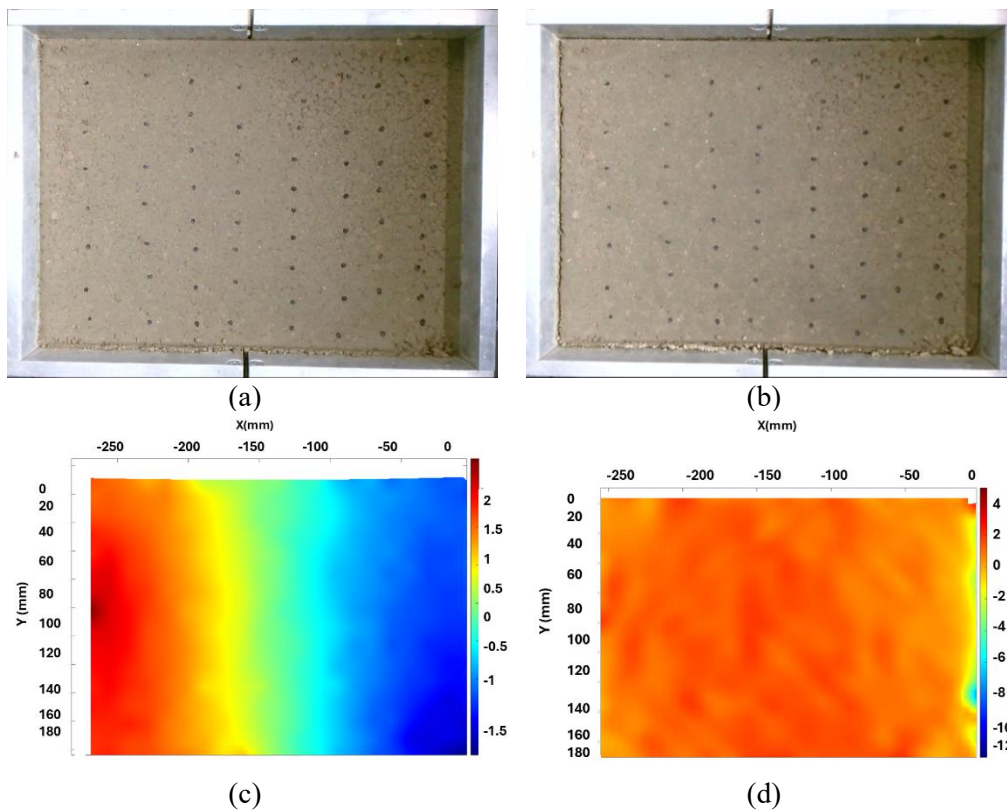


Fig. 5.21 Desiccation test results for *K4* under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

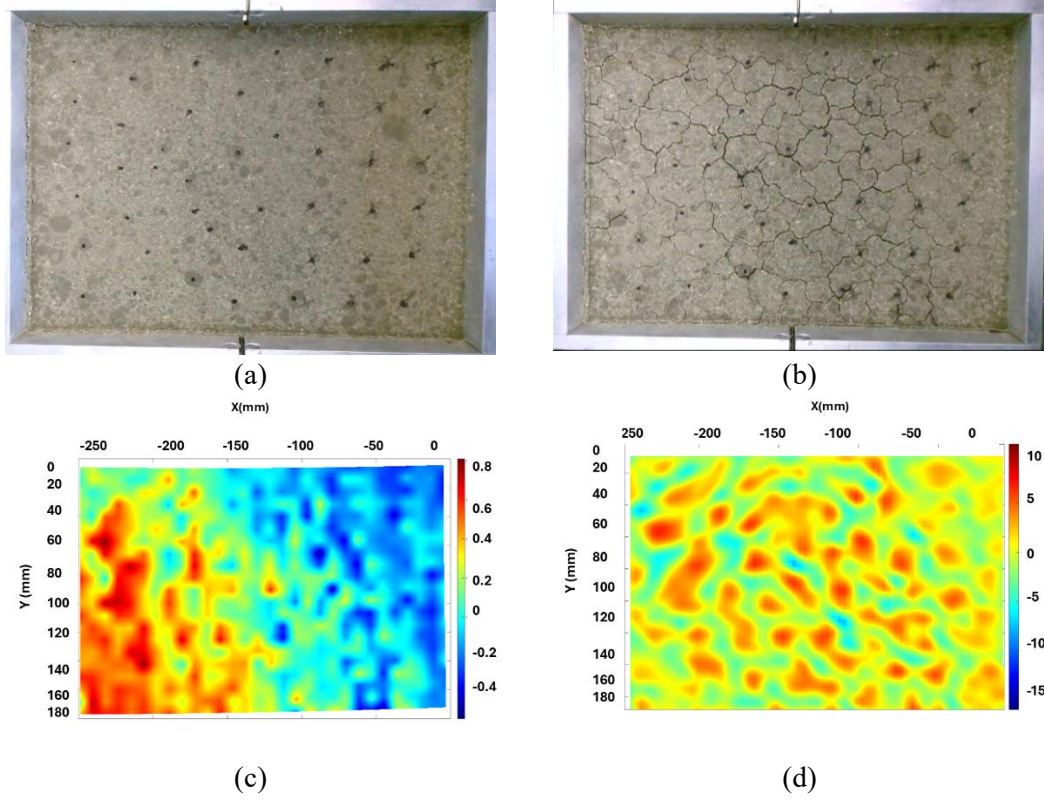
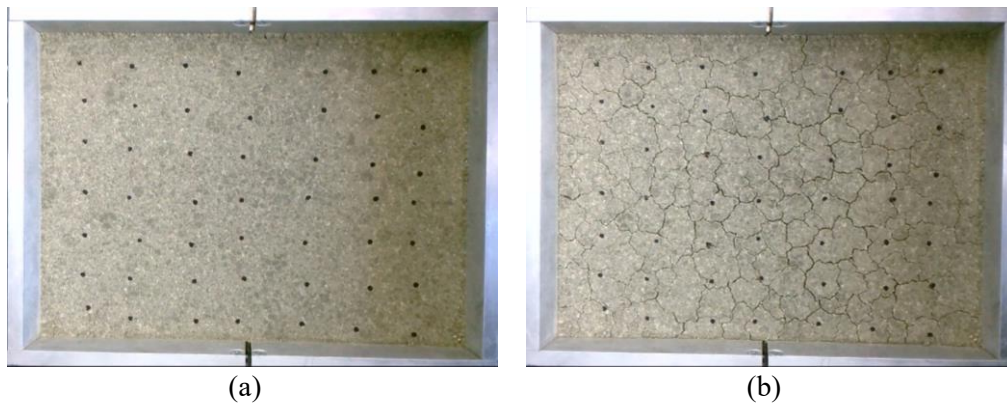


Fig. 5.22 Desiccation test results for *T1* under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.



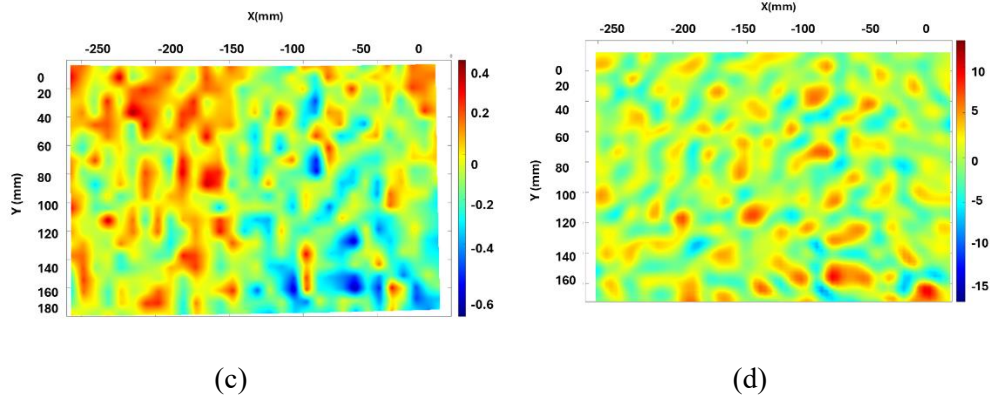


Fig. 5.23 Desiccation test results for $T2$ under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

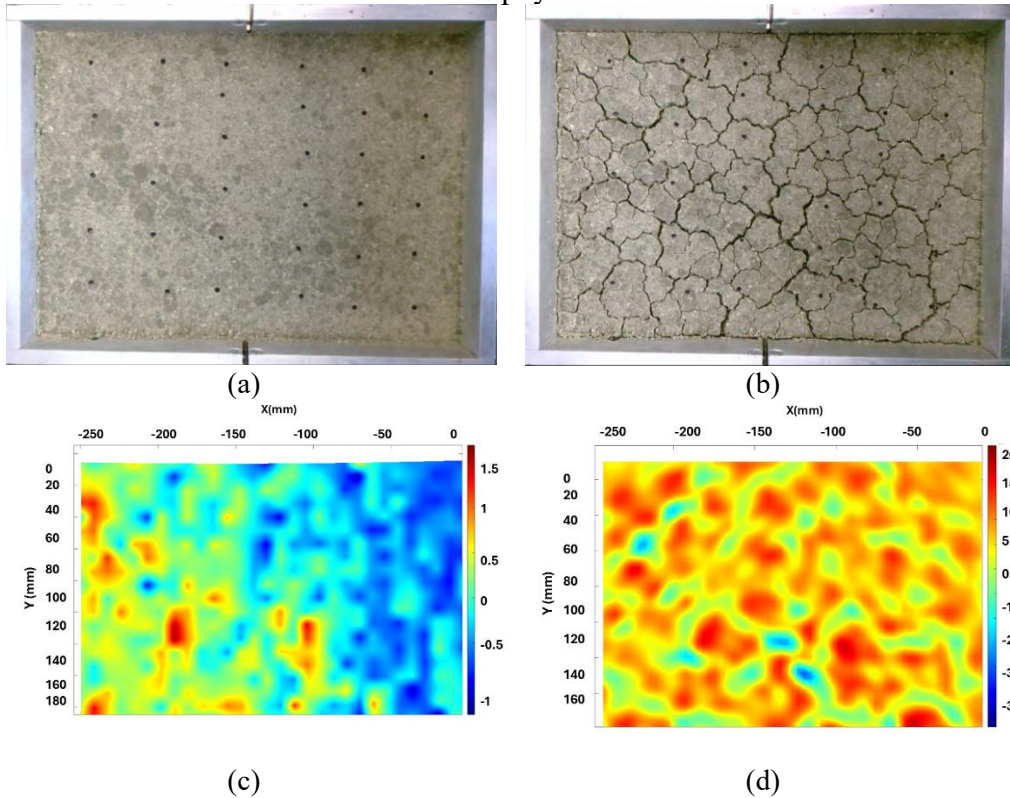


Fig. 5.24 Desiccation test results for $T3$ under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

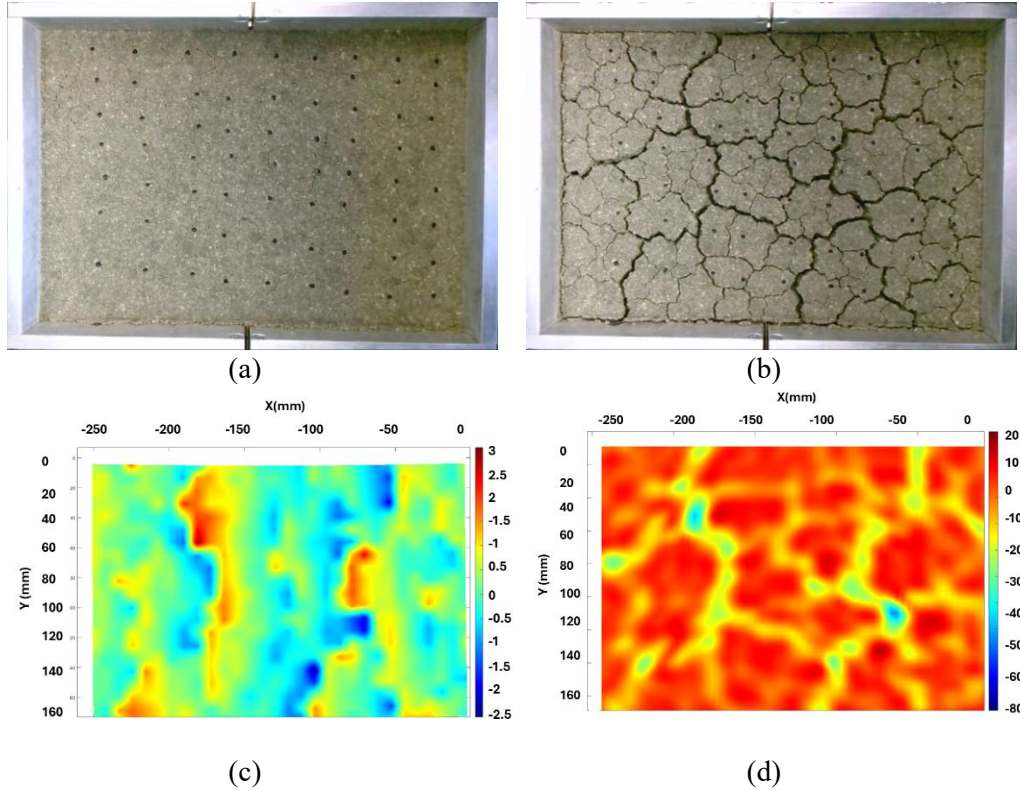


Fig. 5.25 Desiccation test results for *T4* under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.

5.5.6 Displacement profiles along the specimen midline

To further quantify the deformation patterns identified from the PIV analysis, horizontal displacement profiles were extracted along the midline of the specimen surface. This additional analysis allows the spatial distribution of deformation along the specimen length to be examined and provides a direct link between the measured displacement field and the crack locations observed on the specimen surface.

The displacement profiles were obtained along a horizontal cross-section located at $Y = 110$ mm, corresponding to the mid-width of the specimen. Since the internal width of the desiccation box is 220 mm, this location represents the centreline of the soil surface. For each test, the horizontal displacement component (U_x) obtained from the PIV analysis was extracted for all points along this line. The horizontal coordinate (X) represents the distance along the specimen length measured from the left side of the box.

Displacements are expressed in millimetres (mm) following calibration of the image scale, ensuring that the measured values correspond to the actual surface movements of the specimen.

For each test, displacement profiles were plotted at four different times during drying: 3 h, 6 h, 12 h, and the failure time, where the failure time corresponds to the instant at which the peak tensile stress was recorded by the load cells. These profiles allow the progressive evolution of deformation along the specimen to be examined during the drying process.

It should be noted that the displacement profiles extend only to approximately 250–260 mm along the X direction, although the internal length of the box is 303 mm. This difference results from the PIV processing procedure. To ensure reliable correlation results, the analysis region was slightly cropped near the specimen boundaries. Displacements close to the edges often exhibit reduced correlation reliability due to image distortion and boundary effects associated with the subset size and search window used in the PIV algorithm. Consequently, only the central portion of the specimen was retained for displacement extraction, resulting in an effective analysis domain slightly smaller than the physical specimen length.

The displacement profiles obtained from the PIV analysis for the twelve desiccation tests are presented in Figs. 5.26–5.37. In each figure, the horizontal displacement component u_x is plotted along the specimen midline ($y = 110$ mm) as a function of the longitudinal coordinate x for four selected times (3 h, 6 h, 12 h, and the failure time corresponding to the peak tensile stress). These profiles are used to identify deformation localisation and displacement discontinuities associated with crack development.

5.5.6.1 Behaviour of soignies soil

The displacement profiles for the Soignies soil (tests *S1–S4*) clearly highlight the influence of boundary conditions on deformation localisation (Figs. 5.26–5.29).

For the constrained tests *S1* and *S2*, the displacement profiles exhibit a clear displacement discontinuity, which appears as a sharp jump in the curve. This discontinuity corresponds to the location of the dominant crack observed in the surface images in Figs. 5.14 (b) and 5.15 (b). In both tests, the displacement discontinuity becomes clearly visible in the 12 h profiles, which correspond to a stage after crack initiation for this soil.

At earlier times (3 h and 6 h), the displacement profiles remain relatively smooth, suggesting that although localised deformation may already begin to develop at the future crack location, the displacement discontinuity is still too small to produce a clearly visible crack at the soil surface. As drying progresses, deformation becomes concentrated near the crack position, resulting in a sudden displacement jump along the profile.

In contrast, the unconstrained tests *S3* and *S4* show smooth and continuous displacement profiles without abrupt discontinuities. This behaviour is consistent with the surface

observations presented in Figs. 5.16 (b) and 5.17 (b), where no dominant crack developed and shrinkage remained relatively uniform across the specimen surface.

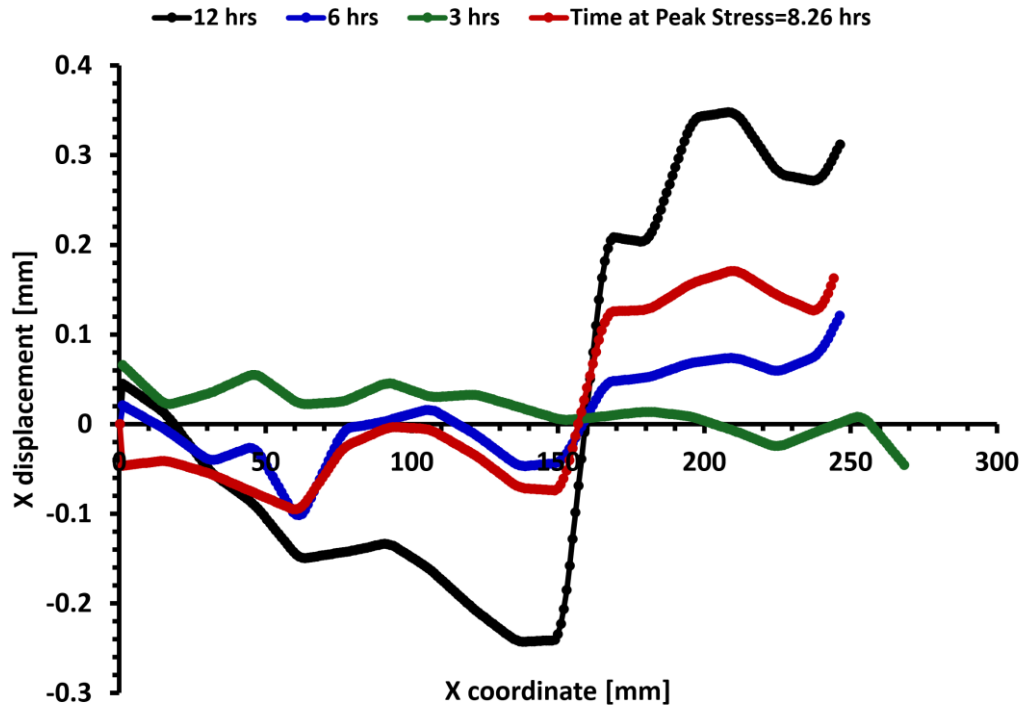


Fig. 5.26 Horizontal displacement profiles along the specimen midline for test *S1* at 3 h, 6 h, 12 h, and time at peak stress.

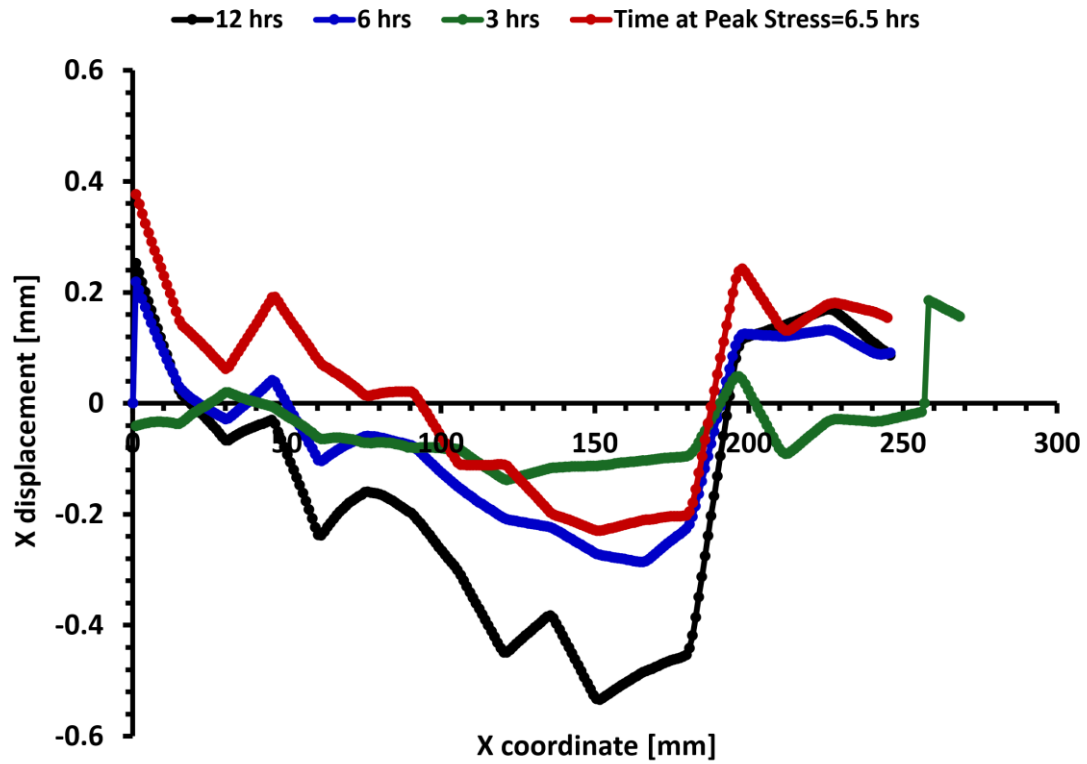


Fig. 5.27 Horizontal displacement profiles along the specimen midline for test *S2* at 3 h, 6 h, 12 h, and time at peak stress.

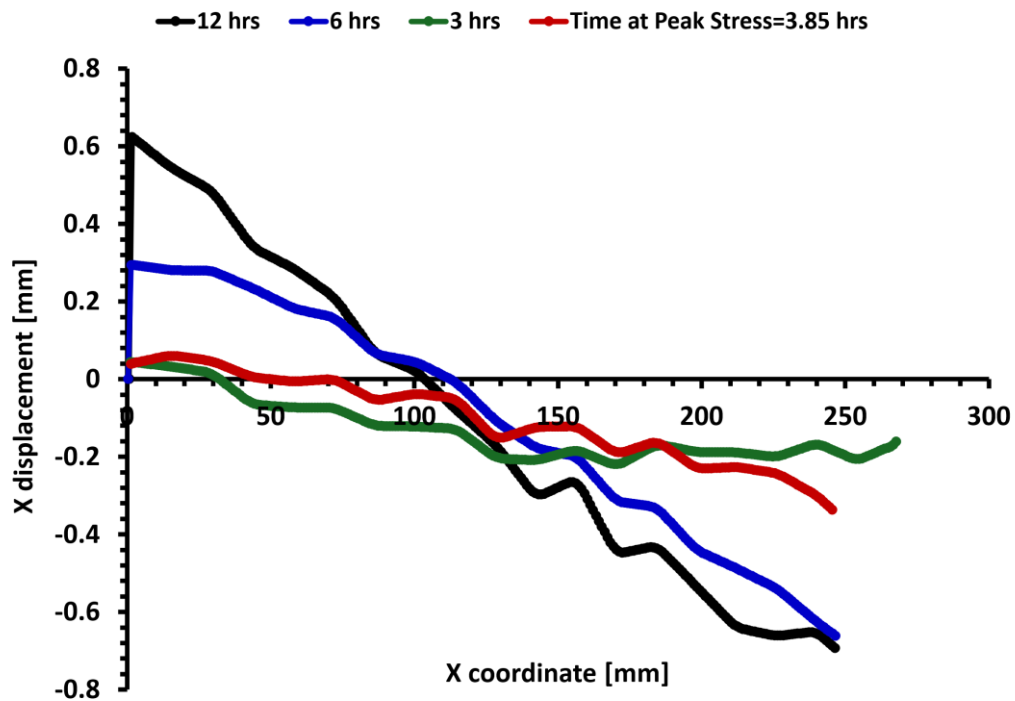


Fig. 5.28 Horizontal displacement profiles along the specimen midline for test *S3* at 3 h, 6 h, 12 h, and time at peak stress.

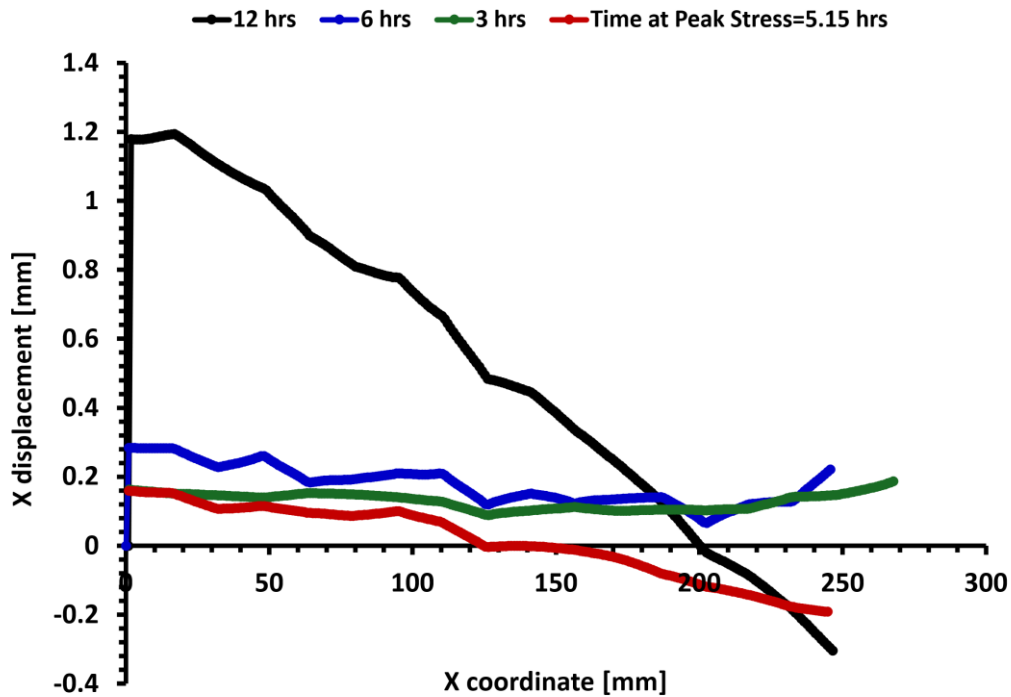


Fig. 5.29 Horizontal displacement profiles along the specimen midline for test *S4* at 3 h, 6 h, 12 h, and time at peak stress.

5.5.6.2 Behaviour of kruibeke soil

A similar behaviour is observed for the Kruibeke soil. For the constrained tests *K1* and *K2* (Figs. 5.30 and 5.31), the displacement profiles reveal distinct displacement discontinuities corresponding to the locations of the main cracks visible in Figures 5.18 (b) and 5.19 (b).

These displacement jumps become most evident in the 12 h profiles, indicating that crack formation occurred before this stage of drying. At earlier times (3 h and 6 h), the displacement profiles remain relatively smooth, suggesting that although localised deformation may already begin to develop at the future crack location, the displacement discontinuity is still too small to produce a clearly visible crack at the soil surface.

For the unconstrained tests *K3* and *K4* (Figs. 5.32 and 5.33), the displacement profiles remain continuous and nearly linear along the specimen length. No abrupt displacement discontinuities are observed, which is consistent with the surface images (Figs. 5.20 (b) and 5.21 (b)), where no dominant crack developed during drying.

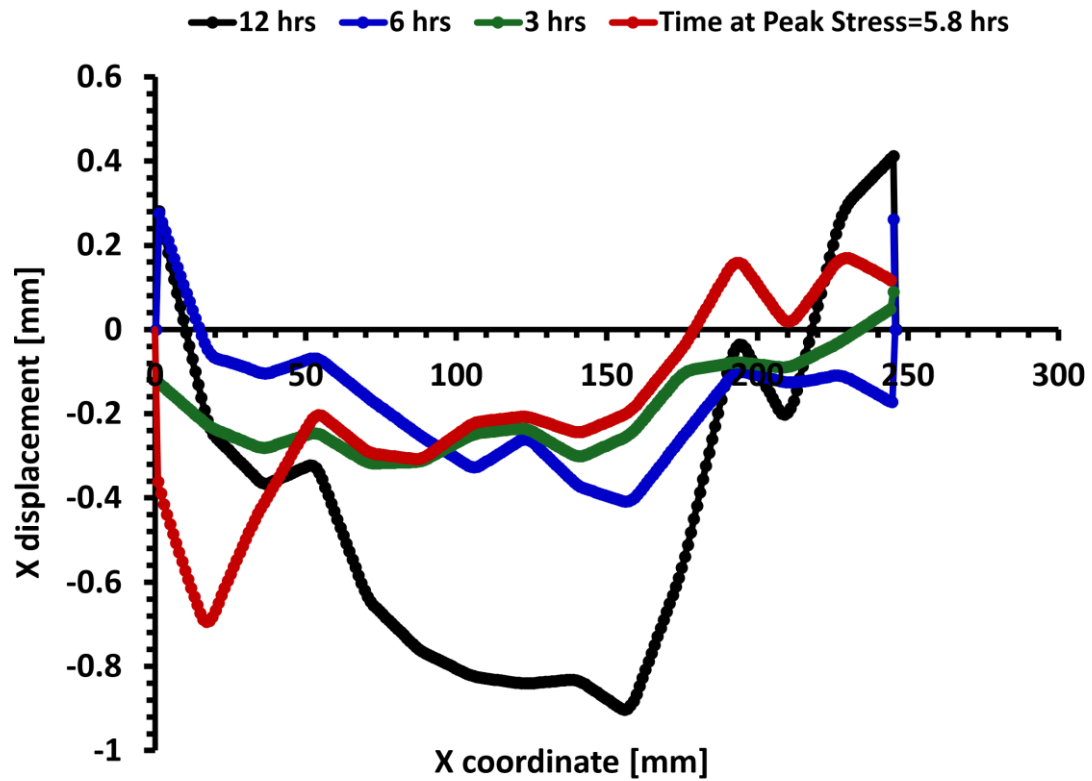


Fig. 5.30 Horizontal displacement profiles along the specimen midline for test *K1* at 3 h, 6 h, 12 h, and time at peak stress.

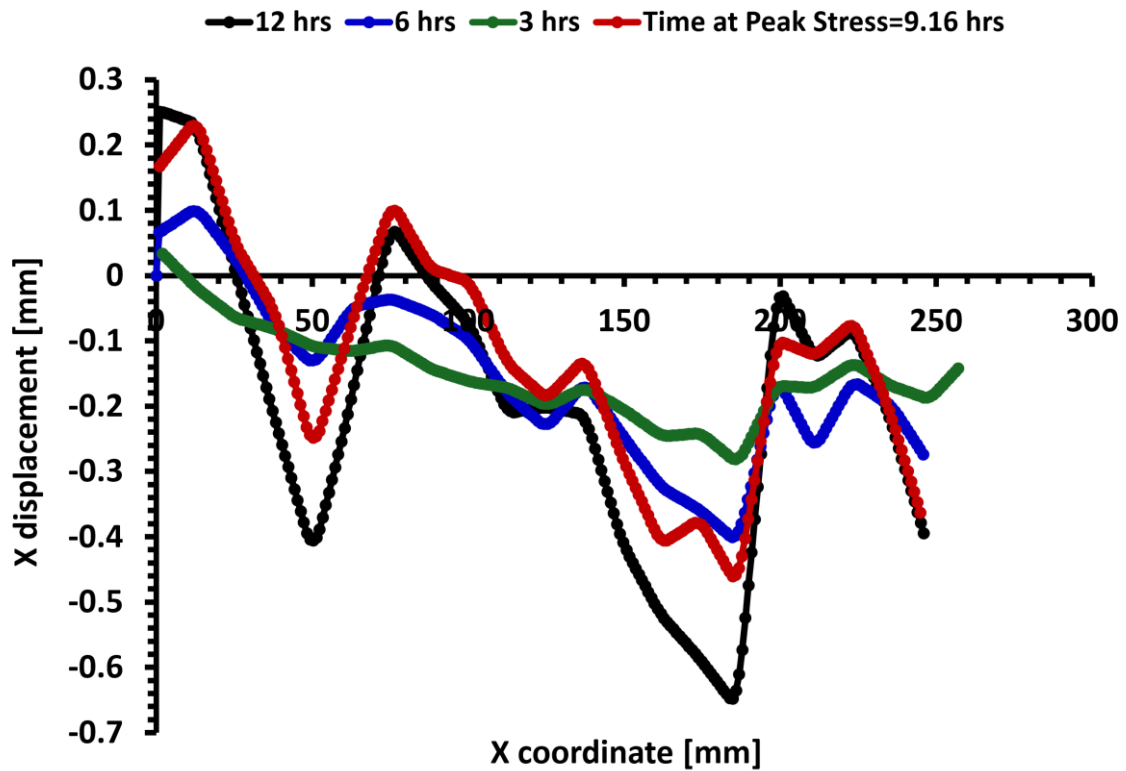


Fig. 5.31 Horizontal displacement profiles along the specimen midline for test *K2* at 3 h, 6 h, 12 h, and time at peak stress.

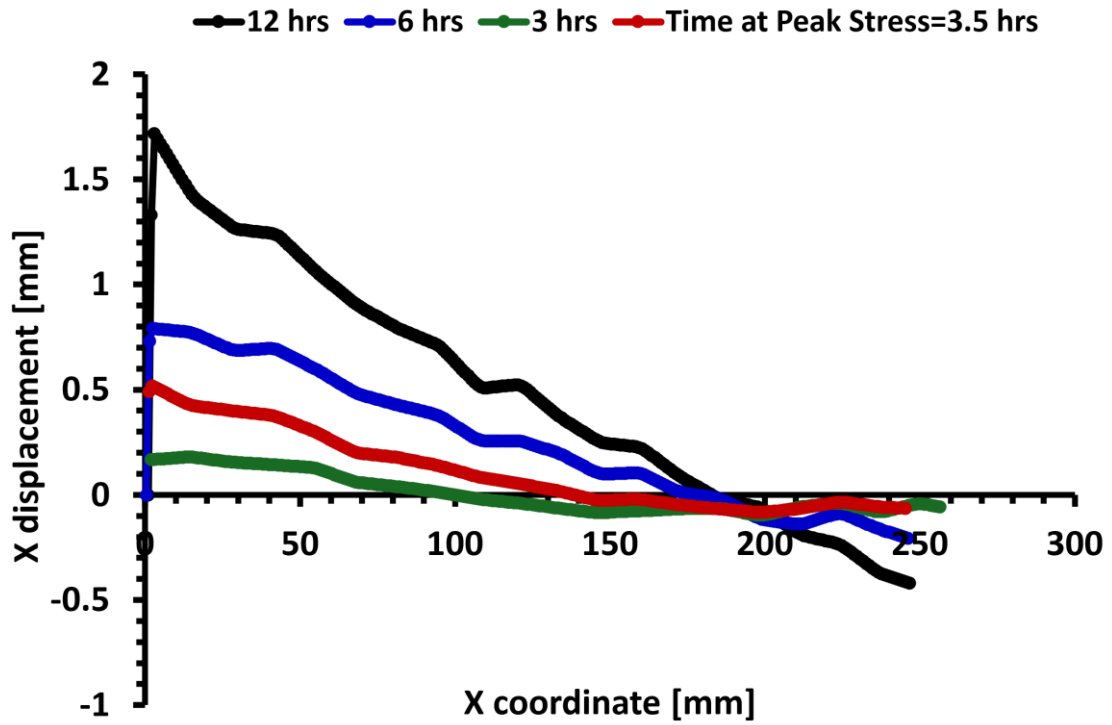


Fig. 5.32 Horizontal displacement profiles along the specimen midline for test *K3* at 3 h, 6 h, 12 h, and time at peak stress.

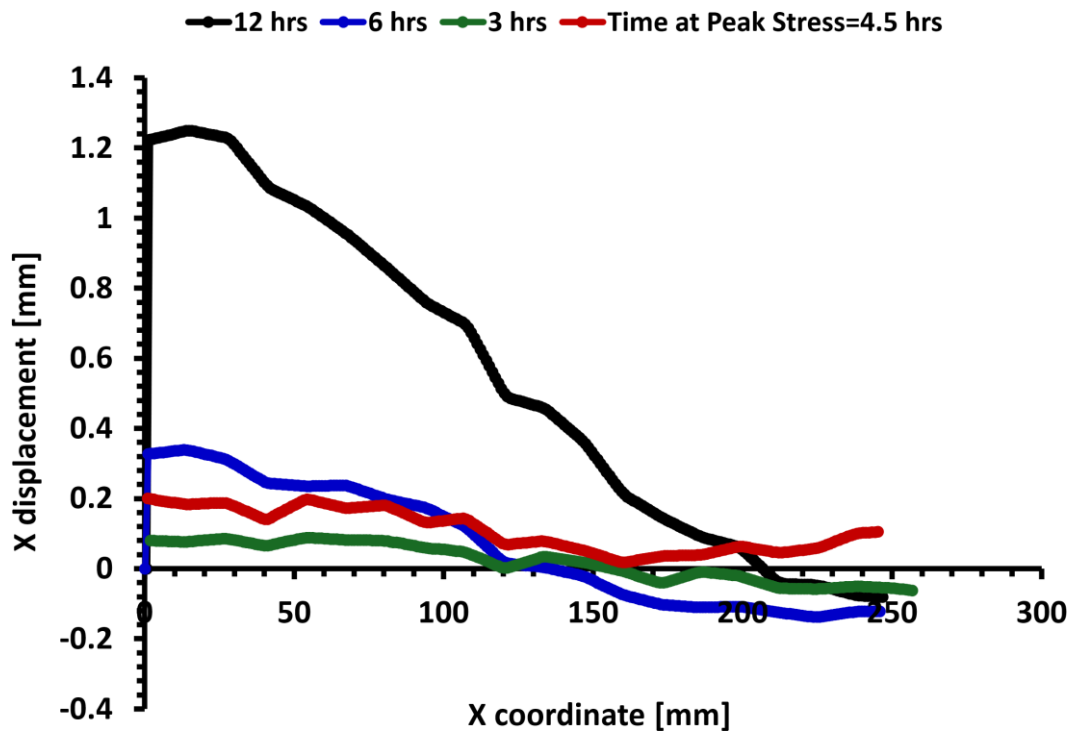


Fig. 5.33 Horizontal displacement profiles along the specimen midline for test *K4* at 3 h, 6 h, 12 h, and time at peak stress.

5.5.6.3 Behaviour of tournai soil

The behaviour of the Tournai soil differs significantly from that of the other two soils. For tests T1–T4 (Figs. 5.34–5.37), the displacement profiles do not exhibit a single dominant displacement discontinuity. Instead, the curves display multiple smaller displacement variations distributed along the specimen length.

These irregular displacement patterns correspond to the distributed crack networks observed in the surface images (Figs. 5.22–5.25(b)). Rather than a single localised fracture, several cracks develop progressively across the specimen surface, leading to deformation occurring at multiple locations.

For the Tournai soil, crack initiation occurs later during the drying process. Consequently, the displacement irregularities associated with crack formation are most clearly visible in the profiles corresponding to the failure time, defined by the peak tensile stress measured by the load cells. At earlier times (3 h, 6 h, and sometimes 12 h), the displacement profiles also show some discontinuities but with smaller extent. It demonstrated that even before the peak stress, some hairline cracks initiate at the surface of the specimen even if full failure is not yet developed.

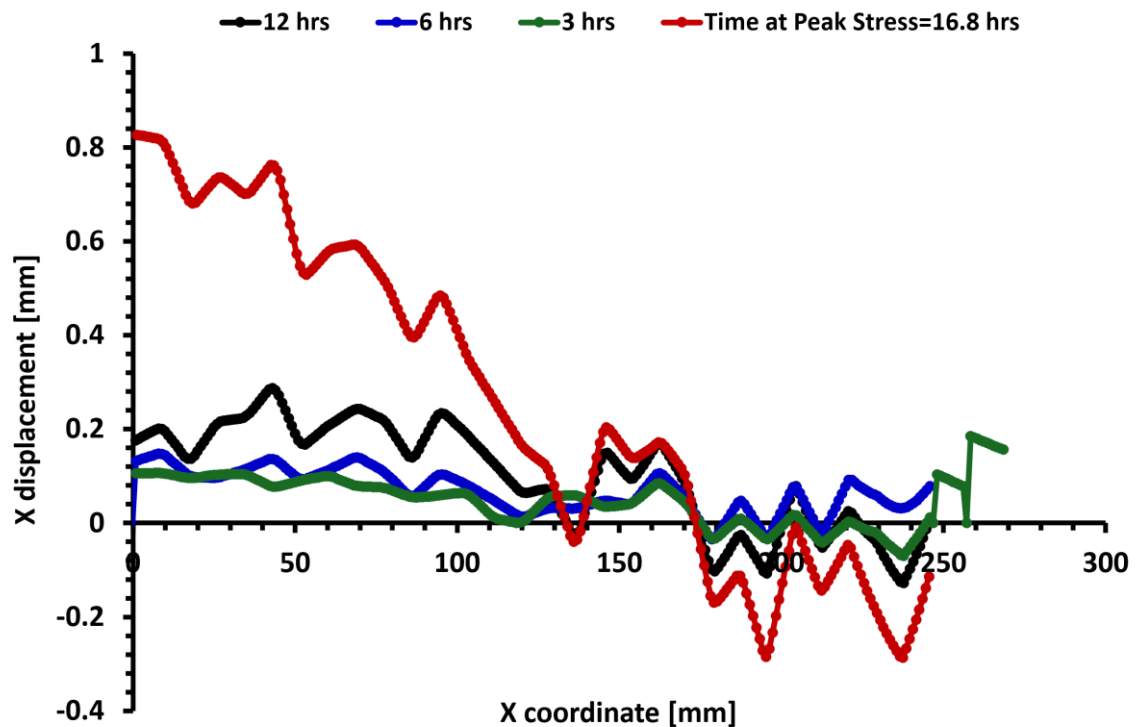


Fig. 5.34 Horizontal displacement profiles along the specimen midline for test *T1* at 3 h, 6 h, 12 h, and time at peak stress.

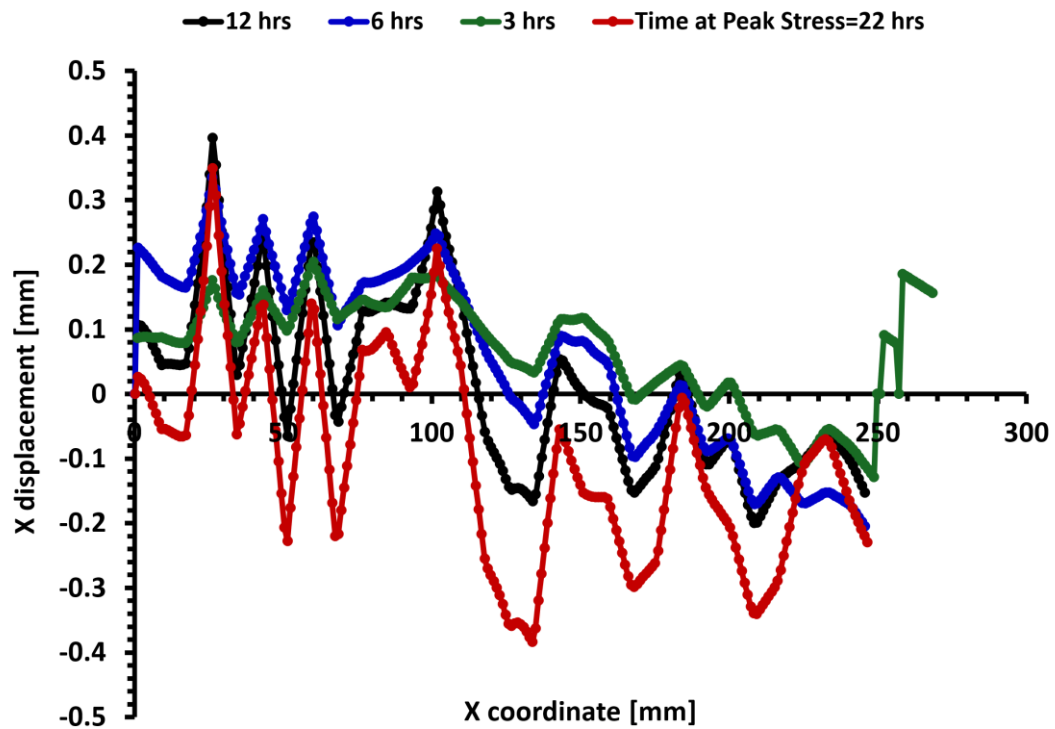


Fig. 5.35 Horizontal displacement profiles along the specimen midline for test *T2* at 3 h, 6 h, 12 h, and time at peak stress.

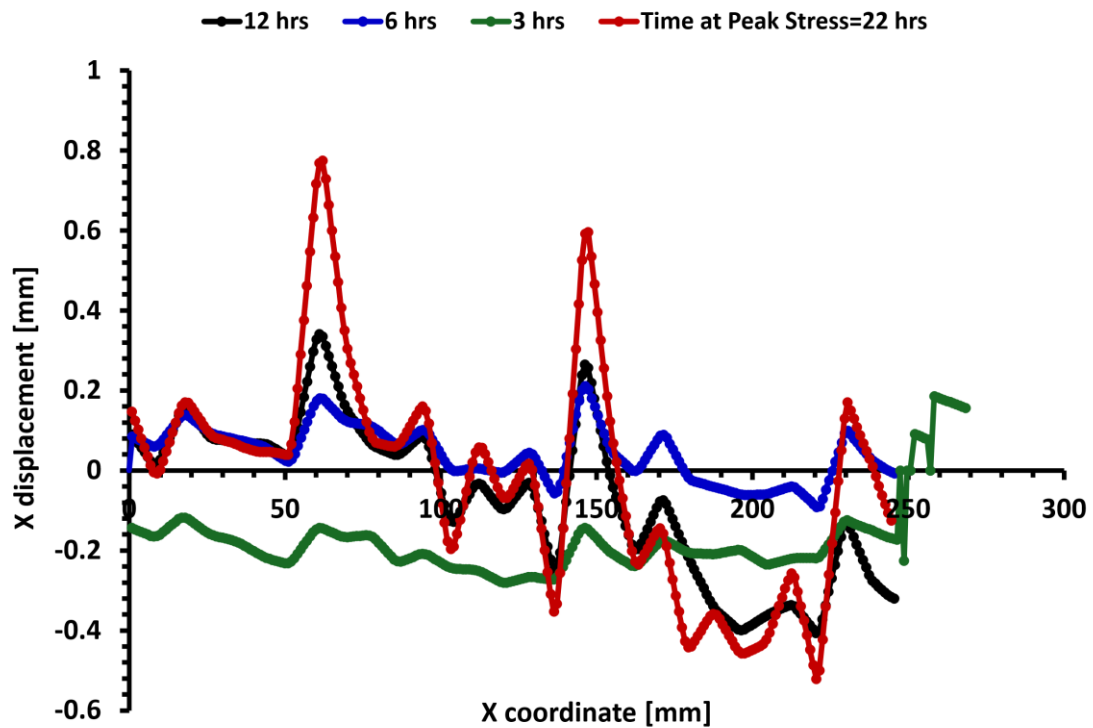


Fig. 5.36 Horizontal displacement profiles along the specimen midline for test *T3* at 3 h, 6 h, 12 h, and at failure time.

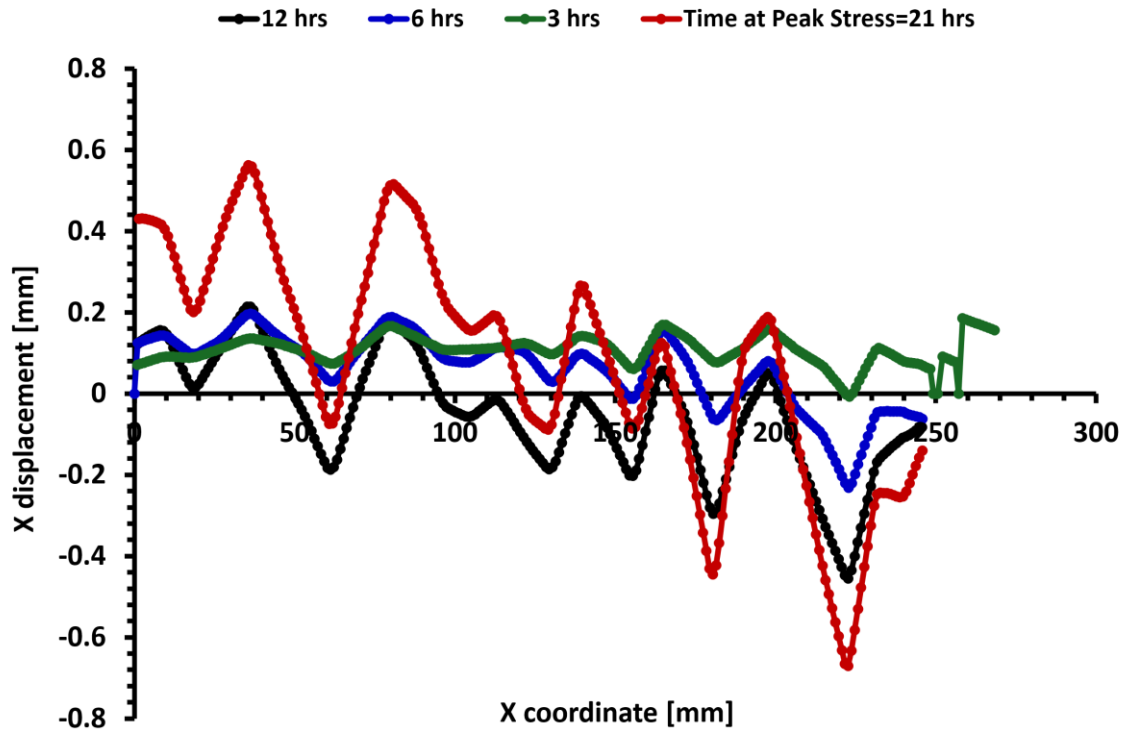


Fig. 5.37 Horizontal displacement profiles along the specimen midline for test *T4* at 3 h, 6 h, 12 h, and time at peak stress.

5.6 Discussion

The experimental results highlight two clearly distinct cracking mechanisms depending on soil type and boundary condition. While Soignies and Kruibeke exhibit cracking only when external restraint is imposed, the Tournai soil develops cracks even in the absence of boundary constraint, indicating fundamentally different deformation responses during drying.

Under constrained conditions, all three soils exhibited cracking; however, the crack morphology differed markedly. Soignies and Kruibeke developed a single crack oriented transverse to the direction of restraint, whereas Tournai formed a distributed crack network without a preferential direction. This contrast indicates that cracking in Soignies and Kruibeke is primarily governed by external boundary restraint, while in Tournai it is largely controlled by internally generated stresses.

For the Soignies and Kruibeke soils, shrinkage is mainly controlled by the external restraint provided by the box and screws. The restriction of global contraction promotes a progressive build-up of tensile stresses across the specimen. The PIV fields show strong strain localization within a narrow band, which subsequently develops into a single dominant

crack. Once this crack forms, tensile stress drops rapidly, explaining the sharp peak observed in the stress–time curves. In contrast, when no external restraint is applied, shrinkage remains relatively uniform, and tensile stresses are insufficient to initiate visible fractures. A markedly different behaviour is observed for the Tournai soil. Cracking occurs under both constrained and unconstrained conditions and is characterized by distributed fractures over the surface. This behaviour suggests that cracking is not primarily controlled by boundary restraint but rather by internal mechanisms developing within the soil layer. A plausible explanation is the development of moisture and suction gradients through the thickness during drying, which induce differential shrinkage between the upper and lower parts of the specimen. Such internal constraints generate tensile stresses near the surface and promote distributed cracking even without external restraint. The schematic illustration in Fig. 5.38 conceptually represents this mechanism.

Overall, the combined force measurements, surface observations, and PIV analysis demonstrate that desiccation cracking may result either from externally imposed restraint or from internally generated stress gradients, depending on soil characteristics.

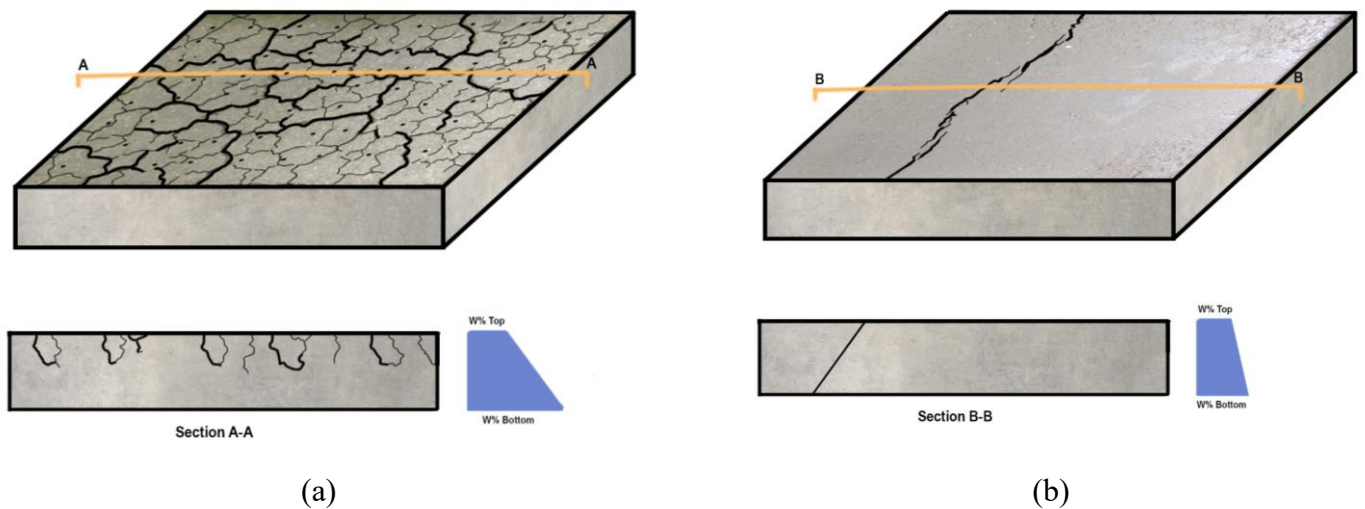


Fig. 5.38 Schematic illustration of internal moisture gradient and associated stress distribution for (a) and (b) localized cracking patterns

5.7 Conclusions

This chapter presented an experimental investigation of desiccation cracking in three clayey soils using a custom-designed constrained–unconstrained drying device combined with force measurements and PIV-based image analysis. The combined interpretation of

mechanical and kinematic data provides a consistent and comprehensive understanding of the mechanisms governing tensile stress development and crack formation during drying.

The results demonstrate that both soil type and boundary condition play a fundamental role in controlling desiccation behaviour. Two distinct cracking responses were identified. For the Soignies and Kruibeke soils, cracking is primarily governed by external boundary restraint. Under constrained conditions, shrinkage is restricted, leading to the accumulation of tensile stresses and the development of a localised deformation zone that evolves into a single dominant crack. In contrast, under unconstrained conditions, partial stress relaxation limits tensile stress build-up, resulting in predominantly uniform shrinkage without visible cracking.

A markedly different behaviour was observed for the Tournai soil, where cracking occurs under both constrained and unconstrained conditions. This response indicates that crack initiation is not primarily controlled by external restraint but rather by internally generated stresses. The formation of multiple cracks and the associated distributed deformation fields suggest that hydro-mechanical heterogeneity, likely induced by moisture gradients within the specimen, governs the cracking process in this material.

Overall, the results highlight that desiccation cracking may be controlled either by externally imposed restraint or by internally generated stress gradients, depending on the intrinsic properties of the soil. This distinction provides important insight into the mechanisms governing crack initiation and propagation in compacted clayey soils subjected to drying.

A consolidated summary of the measured tensile strengths and corresponding crack patterns for all tests is provided in Table 5.3.

Based on the experimental observations, the main conclusions can be summarised as follows:

- Tensile stress increases progressively during drying until reaching a peak value corresponding to crack initiation, followed by a rapid stress drop associated with loss of tensile continuity.
- Higher tensile stresses are generally observed under constrained conditions, particularly for the Soignies and Kruibeke soils, reflecting the key role of external restraint in stress accumulation.
- The combined use of direct force measurements and PIV analysis provides complementary information on global stress evolution and localised deformation mechanisms.
- For Soignies and Kruibeke soils, cracking occurs only under constrained conditions and is

characterised by strain localisation leading to the formation of a single dominant crack.

- Under unconstrained conditions, these soils exhibit predominantly uniform shrinkage, with limited tensile stress development and no visible cracking.
- In contrast, the Tournai soil exhibits distributed cracking under both boundary conditions, indicating that internal stress generation plays a dominant role.
- The proposed experimental methodology provides a reliable framework for investigating tensile strength evolution and crack development along the actual desiccation stress path.

Overall, the findings demonstrate that desiccation cracking mechanisms are strongly soil-dependent and governed by the interaction between mechanical restraint and hydro-mechanical processes. These results provide valuable insight for the design and assessment of compacted clay barriers, liners, and earth structures subjected to drying.

Table 5.3 Summary of measured tensile strengths and observed crack patterns for all desiccation tests

Test Name	Soil	Boundary Condition	Bulk density (g/cm ³)	Tensile Strength (KPa)	Crack Pattern
<i>S1</i>	Soignies	Constrained	2.03	29	Localized Crack
<i>S2</i>	Soignies	Constrained	2.03	34.70	Localized Crack
<i>S3</i>	Soignies	Unconstrained	2.03	23.13	No Crack
<i>S4</i>	Soignies	Unconstrained	2.03	20.12	No Crack
<i>K1</i>	Kruikeke	Constrained	1.84	23.96	Localized Crack
<i>K2</i>	Kruikeke	Constrained	1.84	32.76	Localized Crack
<i>K3</i>	Kruikeke	Unconstrained	1.84	18.46	No Crack
<i>K4</i>	Kruikeke	Unconstrained	1.84	18.24	No Crack
<i>T1</i>	Tournai	Constrained	1.54	10.88	Distributed Crack
<i>T2</i>	Tournai	Constrained	1.54	12.89	Distributed Crack
<i>T3</i>	Tournai	Unconstrained	1.54	10.35	Distributed Crack
<i>T4</i>	Tournai	Unconstrained	1.54	10.95	Distributed Crack

6. HYDRO-MECHANICAL MODELLING OF DESICCATION BEHAVIOUR

6.1 Hydro-Mechanical Framework

The numerical modelling performed in this study is based on a coupled hydro-mechanical formulation for partially saturated soils. The objective is to reproduce the evolution of suction induced by drying and to quantify its effect on the mechanical response of the material. This approach follows the general hydro-mechanical framework proposed for earthen materials subjected to drying conditions (François et al. 2017), in which suction governs both hydraulic transfer and mechanical behaviour.

The hydraulic behaviour is governed by the conservation of water mass within the porous medium. In the present study, a simplified form of the water mass balance equation is adopted by neglecting additional coupling terms such as vapour transfer and density variations. The governing equation is written as

$$\frac{\partial(n \rho_w S_r)}{\partial t} + \nabla \cdot \mathbf{q}_w = 0 \quad (6.1)$$

where n is the porosity, ρ_w is the water density, S_r is the degree of saturation, t is time, and $\nabla \cdot \mathbf{q}_w$ is the divergence of the water flux vector, and $\mathbf{q}_w$ is the liquid water flux vector.

The liquid water flux is described using Darcy's law (Bear 1972).

$$\mathbf{q}_w = -k_{rw} k \nabla p_w \quad (6.2)$$

where k is the intrinsic permeability, k_{rw} is the relative permeability to water depending on the degree of saturation as described by Eq. (6.5), p_w is the pore water pressure, and ∇ denotes the spatial gradient operator. The fluid viscosity is implicitly included in the permeability term.

Suction is defined as the difference between air and water pressures:

$$s = p_a - p_w \quad (6.3)$$

where s is suction, p_a is the air pressure, and p_w is the pore water pressure. In the present work, the air pressure is assumed constant, so that suction is directly controlled by the pore water pressure field obtained from Eqs. (6.1) and (6.2).

The relationship between suction and the hydraulic state of the material is described through a retention law expressed as

$$S_r = \begin{cases} \left(1 + \left(\frac{s}{p_0}\right)^{\frac{1}{1-\alpha_w}}\right)^{-\alpha_w} & \text{for } s > 0 \\ 1 & \text{for } s \leq 0 \end{cases} \quad (6.4)$$

where S_r is the degree of saturation, s is the suction as defined in Eq. (3), p_0 is a material parameter with the dimension of pressure, and α_w is a dimensionless parameter controlling the shape of the curve. This formulation corresponds to the retention law implemented in the numerical model and slightly differs from the classical van Genuchten model, as presented in Equation (3.2).

The reduction of permeability with decreasing saturation is described by the relative permeability coefficient:

$$k_{rw} = \sqrt{S_r} \left[1 - (1 - S_r^{1/n_k})^{n_k}\right]^2 \quad (6.5)$$

where n_k is a dimensionless material parameter controlling the shape of the relative permeability function. This expression allows capturing the strong decrease in hydraulic conductivity during drying.

By solving Eq. (6.1), the spatial and temporal distribution of suction $s(x, y, z, t)$ is obtained. Through Eq. (6.4), this suction field governs the evolution of saturation, while Eq. (6.5) controls the associated variation of permeability. This coupling between suction, saturation, and permeability governs the kinetics of water transfer during drying, as also highlighted in previous studies (François et al. 2017; Gerard et al. 2010).

The mechanical behaviour of the material is directly influenced by suction through its effect on the internal stress state of the soil. In the present study, the effective stress parameter is expressed as:

$\chi = (S_r)^\alpha$ (6.6) where S_r is the degree of saturation and α is a material parameter controlling the evolution of χ , which is calibrated according to the methodology presented in

Chapter 4. As suction increases during drying, the material undergoes volumetric contraction, leading to deformation and stress development, which is consistent with experimental observations of desiccation processes.

Assuming elastic behaviour, the strain increment is related to the effective stress increment through

$$d\varepsilon = D^{-1}d\sigma' \quad (6.7)$$

where ε is the strain tensor, σ' is the effective stress tensor, and D is the elastic stiffness tensor defined by the Young's modulus E and Poisson's ratio ν . In this study, a constant value of $\nu = 0.3$ is adopted. To account for the non-linear evolution of soil stiffness during drying, the Young's modulus is assumed to vary with the mean effective stress according to:

$$E = E_0 \left(\frac{p'}{p'_{ref}} \right)^{n_e} \quad (6.8)$$

where E_0 is the reference Young's modulus, p' is the mean effective stress, p'_{ref} is the reference mean effective stress, and n_e is a material parameter controlling the non-linear evolution of stiffness. Accordingly, the modelling strategy consists of solving the coupled hydro-mechanical problem, in which suction governs both hydraulic transfer and mechanical response. In this way, suction acts as the key variable linking water transfer and deformation in the hydro-mechanical behaviour of the material.

6.2 Geometry and Mesh

The numerical model reproduces the geometry of the experimental specimen presented in Chapter 5. The simulated domain corresponds to a two-dimensional cross-section of the soil sample, obtained by considering a longitudinal section through the desiccation box. In this representation, the length of the specimen is preserved, while the thickness of the soil layer is represented as the vertical dimension of the model.

Accordingly, the model dimensions are defined as 303 mm in the horizontal direction (X-axis) and 30 mm in the vertical direction (Y-axis), corresponding to the actual dimensions of the soil sample within the experimental setup.

A two-dimensional finite element discretization was adopted under plane strain conditions, assuming that deformation perpendicular to the plane is negligible. This assumption is

consistent with the geometry of the specimen and the boundary conditions of the desiccation test.

The mesh is composed of 8-node quadrilateral elements, allowing for a quadratic interpolation of the displacement and pore pressure fields. This choice ensures an accurate representation of both hydraulic gradients and mechanical deformations during the drying process.

The final mesh consists of 483 finite elements and 1530 nodes. A non-uniform mesh refinement strategy was adopted, with a finer discretization near the upper boundary of the specimen. This refinement was introduced to accurately capture the sharp suction gradients and moisture variations induced by evaporation. The mesh progressively becomes coarser with depth, where hydraulic and mechanical gradients are less pronounced.

In addition, a set of vertical interface elements was introduced. The mechanical properties adopted for the interface elements are summarized in Table 6.1.

It should be noted that the interface friction coefficient was calibrated according to $\mu = \tan(\phi)$, where ϕ is the soil friction angle obtained from the triaxial tests. The cohesion was fixed based on the tensile strength of the soil according to $c = \mu/\tan(\phi)$.

Table 6.1 Mechanical properties of the interface elements

Parameter	Symbol	Unit	Soignies	Kruikebe	Tournai
Normal stiffness	k_N	N/m ³	1.0E+09	1.0E+09	1.0E+09
Shear stiffness	k_τ	N/m ³	1.0E+09	1.0E+09	1.0E+09
Friction coefficient	μ	—	0.45	0.5	0.4
Cohesion	c	kPa	128.7	203.5	305.2

These interface elements are oriented perpendicular to the drying surface and are used to allow crack opening when tensile strength is reached. The contact constraint between the two sides of the interface is enforced by the penalty method (Cerfontaine et al., 2015). The interface element is able to reproduce the contact/loss of contact between two solids as well as shearing/sliding of the interface through an analogy with elastoplasticity. When the interface is in opening mode, the gap may accommodate fluid flow. Here, we consider longitudinal and

transversal permeability of the interface high enough to allow free fluid flow when the interface is open. Consequently, the crack opening creates a preferential path for water evaporation.

The overall mesh configuration is illustrated in Fig. 6.1, which highlights both the geometry of the model and the adopted discretization strategy.

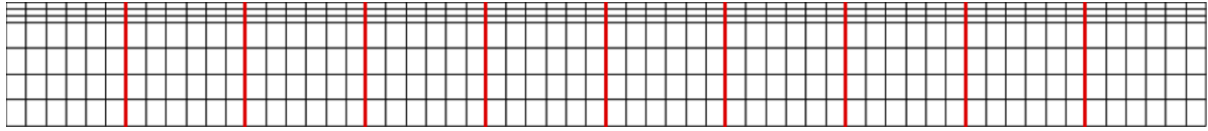


Fig. 6.1 Geometry of the numerical model, finite element mesh and interface elements (red lines) adopted for the drying simulations.

6.3 Material Parameters

The material parameters used in the numerical simulations are derived from the experimental characterization presented in Chapters 3 and 5. To ensure consistency between experimental observations and numerical modelling, all relevant parameters are presented in this section.

The initial state of the soil corresponds to specimens compacted under standard Proctor conditions, at their respective optimum water contents. The associated initial suction and degree of saturation are therefore directly taken from the experimental measurements.

The hydraulic behaviour is governed by the retention and permeability relationships introduced in the theoretical framework. The corresponding parameters are defined for each soil and are consistent with the experimental characterization.

The intrinsic permeability of the soils was experimentally determined, and its evolution with saturation is captured through the adopted relative permeability function. This parameter plays a key role in controlling the kinetics of drying, as lower permeability values lead to slower moisture migration within the sample. The exponent n_k , controlling the evolution of relative permeability with saturation, was assumed equal to the corresponding water retention shape parameter α_w . This assumption ensures consistency between the water retention behaviour and the evolution of relative permeability during drying (Gerard et al., 2010b).

The hydro-mechanical behaviour of the material is described within the framework of generalized effective stress, in which the contribution of suction is incorporated through an effective stress parameter, χ , following a Bishop-type formulation.

In this study, the χ parameter is not assumed a priori but is rigorously calibrated based on the methodology developed in Chapter 4, ensuring consistency between the mechanical response under saturated and unsaturated conditions.

In addition, the elastic behaviour is defined through a Young's modulus evolving with suction, reflecting the dependency of soil stiffness on both suction and effective stress. This formulation enables a more realistic representation of the progressive stiffening of the material during drying.

For clarity and completeness, all material parameters used in the simulations are summarized in Table 6.2. The numerical values are directly taken from the experimental characterization and calibration procedures described in previous chapters. Only the parameters required for the numerical modelling are reported in this table. Additional soil properties are in previous chapters.

Table 6.2 Summary of material parameters adopted for the hydro-mechanical simulations

Parameter	Symbol	Unit	Soignies	Kruikebeke	Tournai
Initial total suction	s_0	MPa	1.291	2.104	1.351
Initial degree of saturation	Sr_0	—	0.87	0.81	0.85
Water retention shape parameter 1	p_0	MPa	1.52	1.62	0.46
Water retention shape parameter 2	α_w	—	0.289	0.253	0.115
Intrinsic permeability	k	m^2	5.79E-17	1.02E-17	2.00E-18
Relative permeability exponent	n_k	—	0.289	0.253	0.115
Effective stress parameter (χ) exponent	α	—	2.20	2.62	3.97
Reference Young's modulus	E_0	MPa	44.99	39.82	93.80
Reference mean effective stress	p'_{ref}	MPa	1	1	1
Non-linear elasticity exponent	n^e	—	2.211	2.260	0.552
Poisson's ratio	ν	—	0.3	0.3	0.3
Porosity	ϕ	—	0.35	0.48	0.59

6.4 Boundary Conditions and Initial State

The boundary conditions of the numerical model are defined to reproduce as closely as possible the experimental drying conditions described in Chapter 5.

From a hydraulic point of view, a time-dependent suction condition is imposed at the top surface, increasing from the initial total suction corresponding to each soil (Table 6.1) to a final value of 110 MPa over a ramp duration of 1000 s, representing the drying conditions applied during the laboratory tests.

As a result, a strong suction gradient progressively develops between the upper drying boundary and the deeper layers of the soil. Initially, the soil is characterized by a relatively low suction corresponding to its as-compacted state under standard Proctor conditions. As drying progresses, suction increases at the surface and propagates downward, inducing water migration within the porous medium. This process is governed by Darcy's law and leads to a progressive decrease in the degree of saturation together with an increase in suction.

The bottom and lateral boundaries are considered impermeable, meaning that no water flux is allowed through these surfaces:

$$q_w \cdot n = 0 \quad (6.9)$$

Consequently, moisture transfer occurs predominantly in the vertical direction, from the interior of the specimen towards the upper drying boundary.

From a mechanical point of view, the boundary conditions are defined to reproduce free or constrained shrinkage configurations, depending on the boundary conditions imposed on the lateral sides.

- The bottom boundary is fixed in the vertical direction ($U_y = 0$) along its entire length.
- To prevent rigid body motion, one single node located at the center of the bottom boundary is fixed in the horizontal direction ($U_x = 0$), while remaining fixed vertically.
- The top surface is fully free to move, allowing the soil to deform vertically during drying.
- The lateral boundaries are free to move in the vertical direction, while the horizontal direction is free to move for the unconstrained case and fixed for the constrained case.

This configuration ensures that no artificial constraints are introduced in the deformation field, while maintaining numerical stability. It also explains the symmetric development of shrinkage-induced stresses, which is consistent with the experimental observations of stress localization.

The initial conditions correspond to the state of the soil immediately after compaction. Each sample is initialized with a uniform suction and degree of saturation, consistent with the experimental values determined under standard Proctor conditions (see Chapter 5). This ensures a direct correspondence between the numerical simulations and the physical tests. The initial stress state is isotropic and corresponds to the value of the effective stress considering initial suction and degree of saturation.

The simulation is performed over a total duration of 259000 seconds (approximately 72 hours). The temporal discretization is defined using progressively increasing time steps, allowing for an accurate description of the early stages of drying and a stable computation over longer durations.

6.5 Numerical Modelling Results

This section presents the numerical results from the desiccation tests conducted on the three investigated soils. The simulations reproduce both the unconstrained and constrained drying conditions presented in Chapter 5. The evolution of the predicted displacement fields is first analysed, followed by a comparison between the numerical predictions and the corresponding experimental observations.

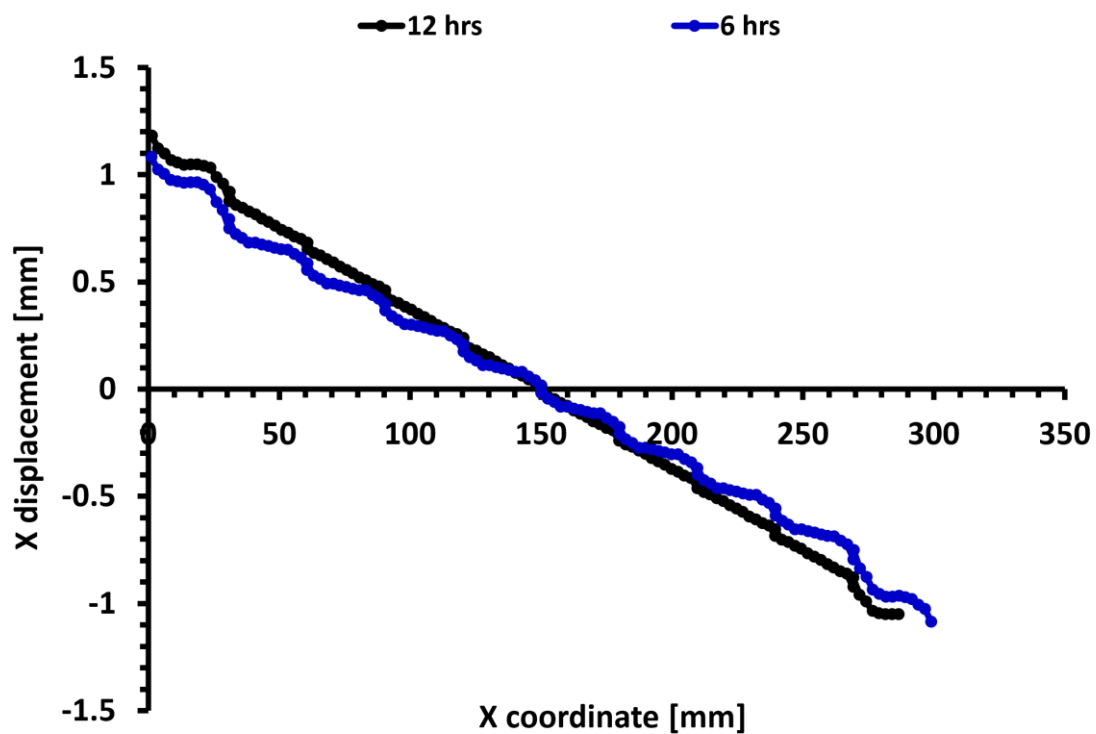
6.5.1 Displacement fields under unconstrained conditions

The horizontal displacement profiles obtained after 6 h and 12 h of unconstrained drying for Soignies, Kruibeke, and Tournai are presented in Fig. 6.2. For the three soils, the horizontal displacement evolves progressively along the specimen length, changing from positive values at one end of the specimen to negative values at the opposite end. In all cases, the zero-displacement point is at the middle of the specimen where one point is fixed to prevent rigid body motion.

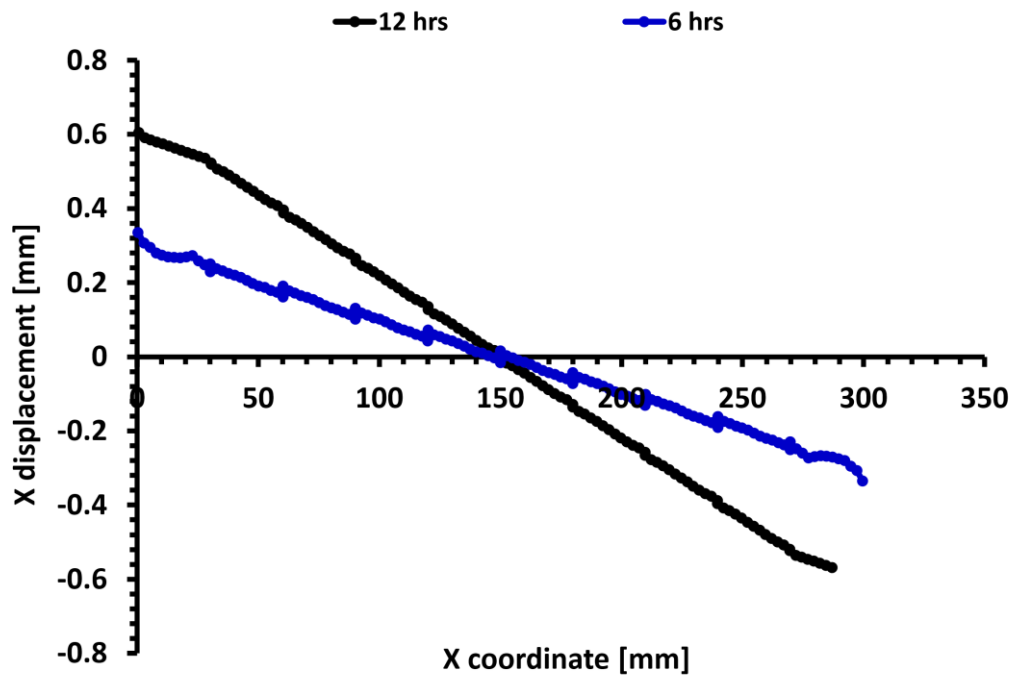
An increase in the magnitude of horizontal displacement is observed between 6 h and 12 h for all three soils, while the overall shape of the displacement profiles remains essentially unchanged. For Soignies, the maximum horizontal displacement increases from 1.10 mm after

6 h to 1.20 mm after 12 h. A similar evolution is obtained for Tournai, where the maximum displacement increases from 0.95 mm to 1.35 mm over the same period. Kruibeke exhibits smaller horizontal displacements than the other two soils, with the maximum displacement increasing from 0.35 mm after 6 h to 0.60 mm after 12 h. These results indicate that the effect of drying is primarily reflected by an increase in the magnitude of horizontal shrinkage, whereas the overall deformation pattern remains practically unchanged throughout the analysed drying period.

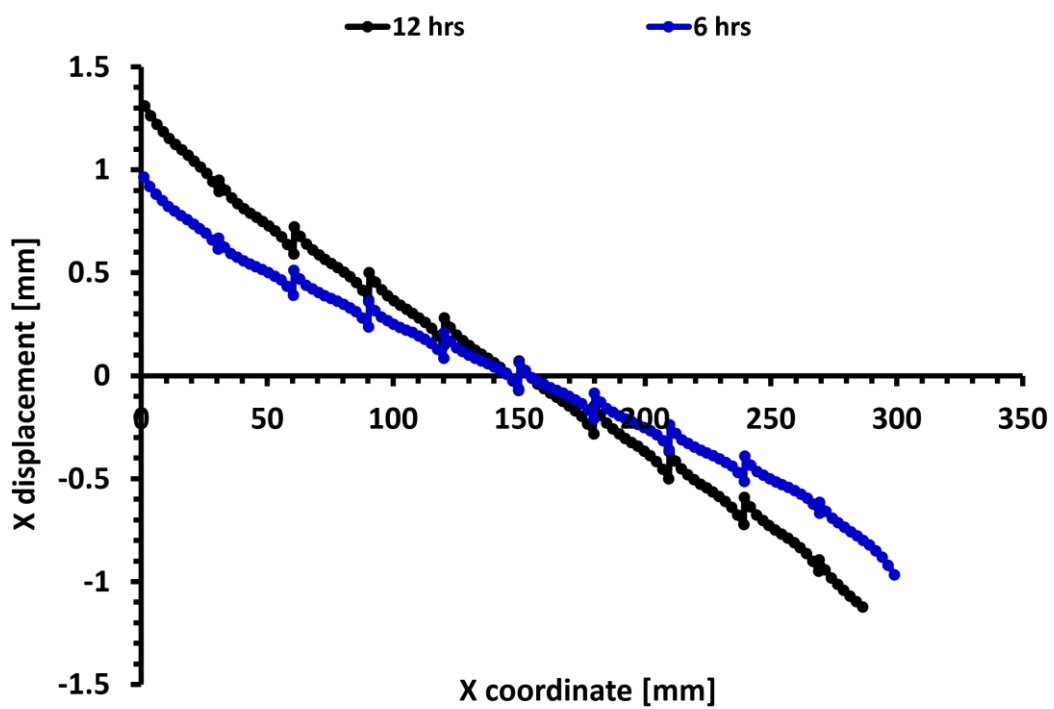
Overall, the numerical simulations reproduce the progressive development of horizontal shrinkage under unconstrained drying conditions for the three investigated soils. Although differences in displacement magnitude are observed between the soils, the numerical model consistently captures the evolution of the horizontal displacement profiles with increasing drying time.



(a)



(b)



(c)

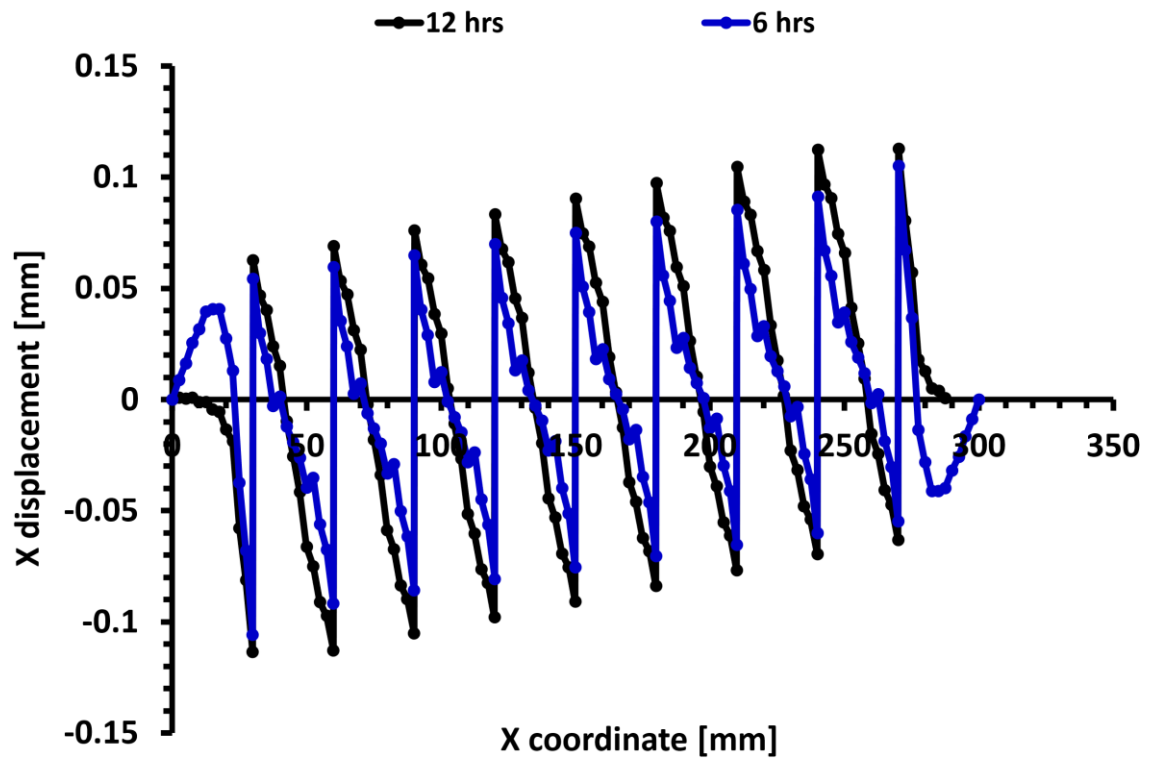
Fig. 6.2 Horizontal displacement profiles (mm) under unconstrained drying conditions at 6 h and 12 h for (a) Soignies, (b) Kruibeke, and (c) Tournai.

6.5.2 Displacement fields under constrained conditions

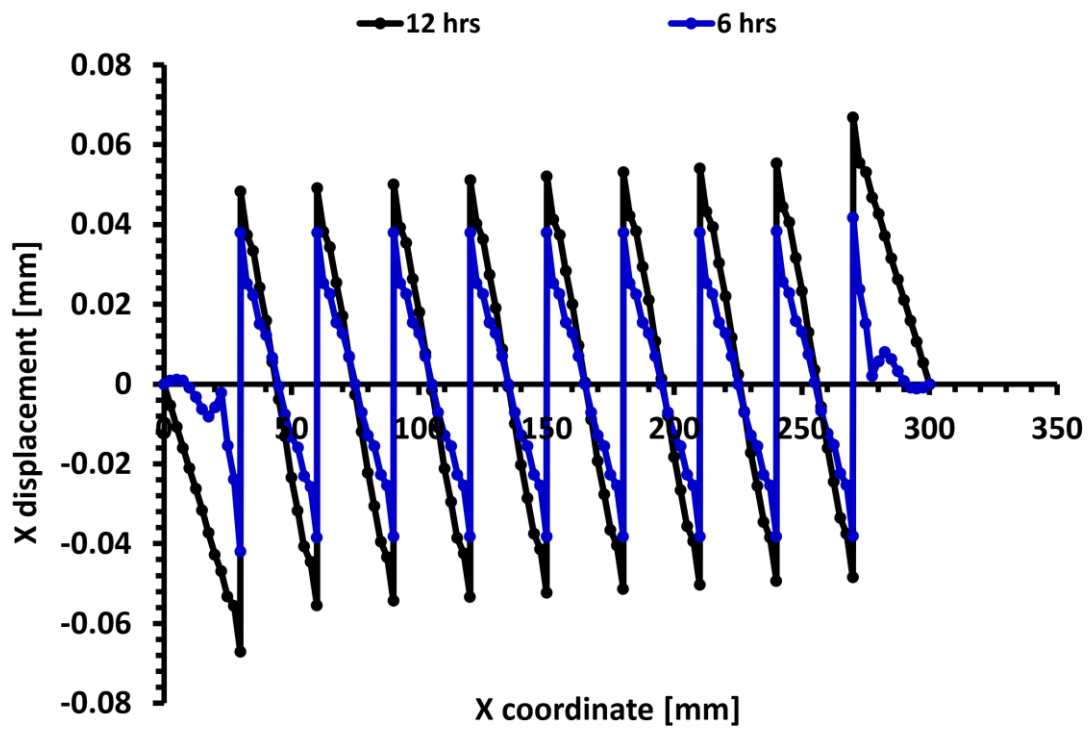
The horizontal displacement profiles obtained after 6 h and 12 h of constrained drying for Soignies, Kruibeke, and Tournai are presented in Fig. 6.3. Unlike the unconstrained condition, the displacement profiles exhibit a repetitive saw-tooth pattern along the specimen length for the three investigated soils. This characteristic profile results from the opening of the interface elements induced by horizontal tensile stress across the interface. The soil between the interfaces is under contraction while the interfaces are in opening mode, leading to this saw-tooth pattern.

For all three soils, the displacement amplitude increases between 6 h and 12 h while the characteristic saw-tooth profile remains essentially unchanged. The largest displacement amplitude is obtained for Tournai, reaching 0.21 mm after 12 h of drying, followed by Soignies with a maximum displacement of 0.11 mm. Kruibeke exhibits the smallest deformation throughout the simulation, with a displacement of 0.07 mm after 12 h. Although the displacement amplitudes differ, the numerical simulations consistently predict the same deformation pattern under constrained drying conditions.

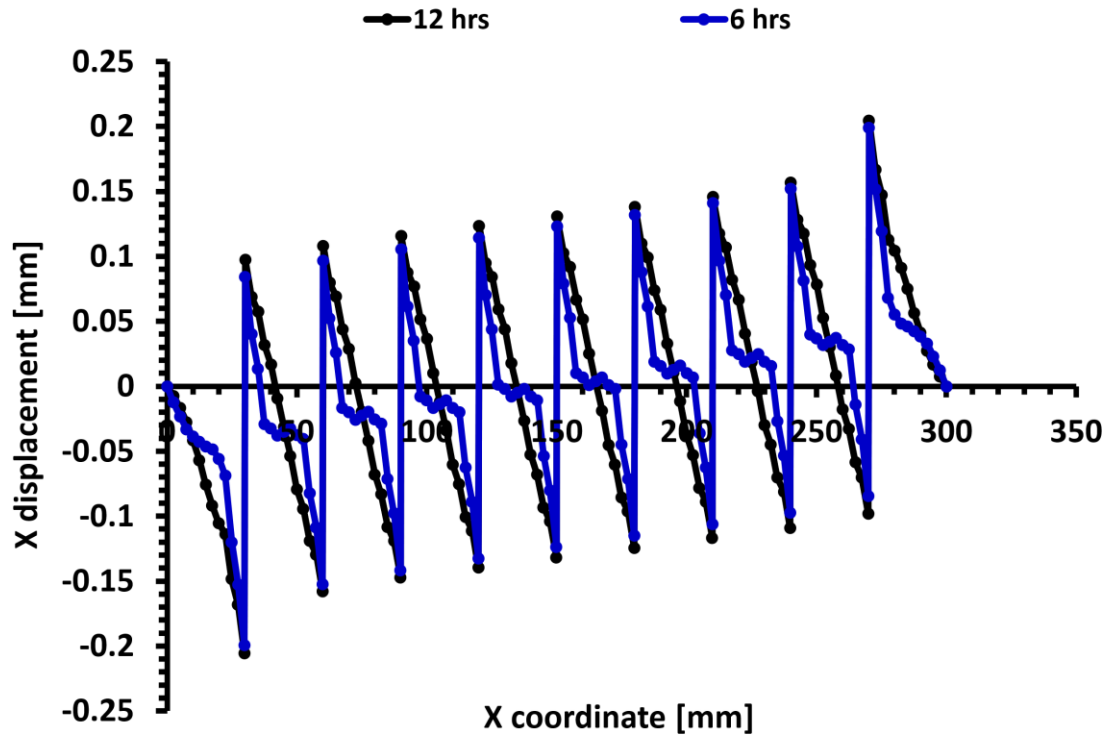
Overall, the numerical simulations reproduce the characteristic deformation pattern observed under constrained drying conditions. While the imposed boundary conditions considerably reduce the magnitude of horizontal displacement compared with the unconstrained configuration, the increase in displacement with drying time is consistently captured for the three investigated soils.



(a)



(b)



(c)

Fig. 6.3 Horizontal displacement profiles (mm) under constrained drying conditions at 6 h and 12 h for (a) Soignies, (b) Kruibeke, and (c) Tournai.

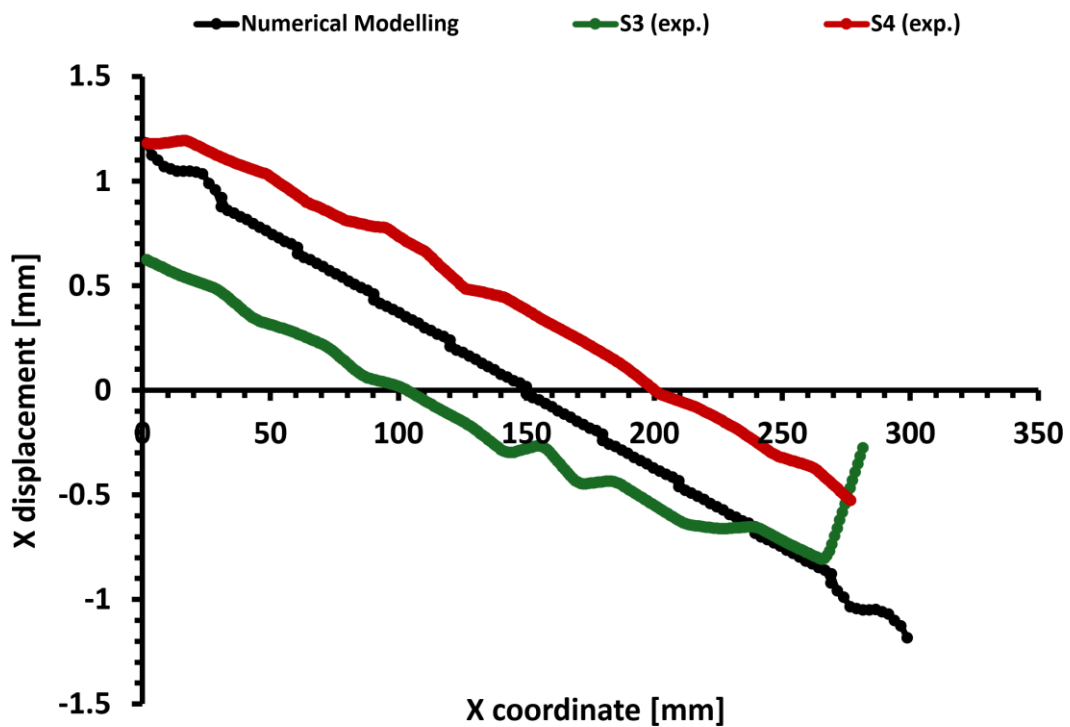
6.5.3 Comparison of experimental and numerical results under unconstrained conditions

Figure 6.4 compares the numerical and experimental horizontal displacement profiles obtained after 12 h of unconstrained drying for the three investigated soils. The experimental horizontal displacement was extracted at the mid-section of the specimen from the PIV analysis. Overall, the numerical simulations reproduce the general distribution of horizontal displacement along the specimen length, with positive displacements developing at one side of the specimen and negative displacements at the opposite side. However, the degree of agreement between the numerical predictions and the experimental measurements differs depending on the investigated soil.

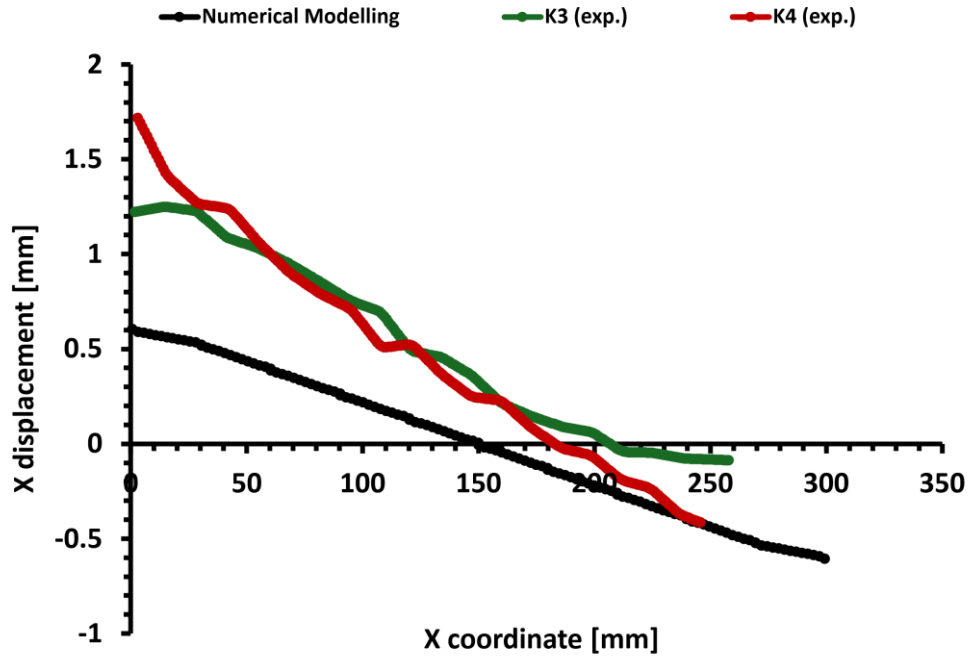
For Soignies, the numerical profile shows a good agreement with the experimental measurements, reproducing both the displacement magnitude and the overall slope of the experimental profiles. The numerical results remain between the two experimental measurements over most of the specimen length, indicating that the overall deformation

response is well captured. For Kruibeke, although the numerical model reproduces the general trend of the displacement profile, it underestimates the displacement magnitude and does not fully reproduce the slope observed experimentally. For Tournai, the numerical model is not able to capture the main pattern of displacement. The numerical model predicts a uniform displacement field with very limited opening of the interfaces, while the experiments show a more discontinuous pattern of displacement revealing numerous crack openings. Experimentally, the occurrence of cracks, even under unconstrained conditions, was attributed to the strong suction gradient through the thickness of the soil, leading to internal constraints that induce tensile stress at the specimen surface. Those conditions were not obtained by the numerical model.

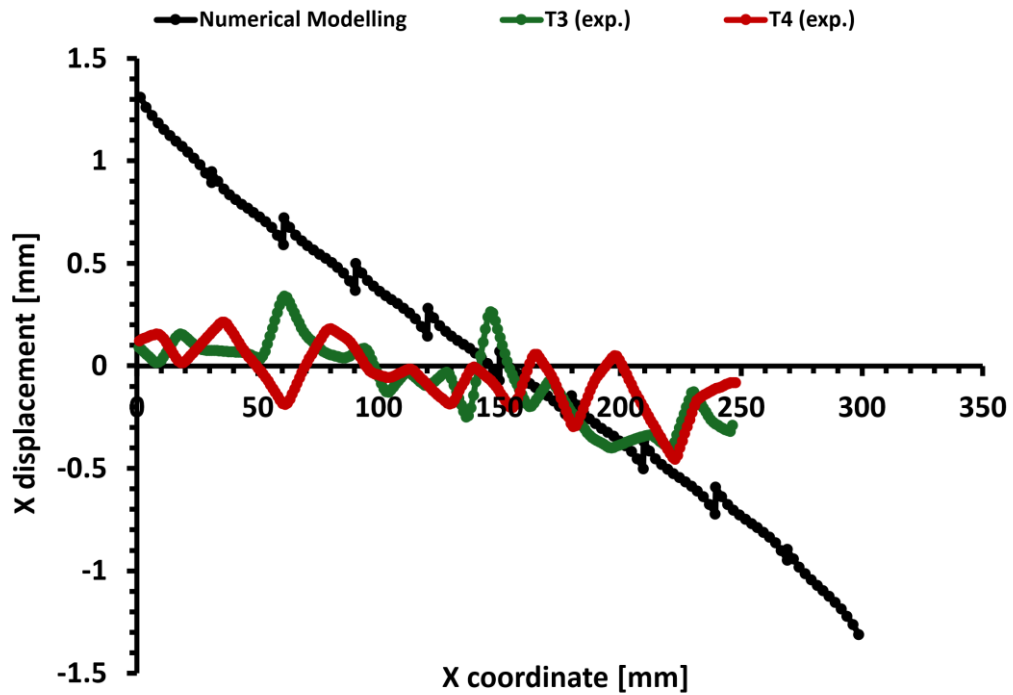
It should be noted that, unlike the numerical simulations, where the zero-displacement point is consistently located near the specimen mid-length, the experimental measurements exhibit different zero-displacement locations for the repeated tests.



(a)



(b)



(c)

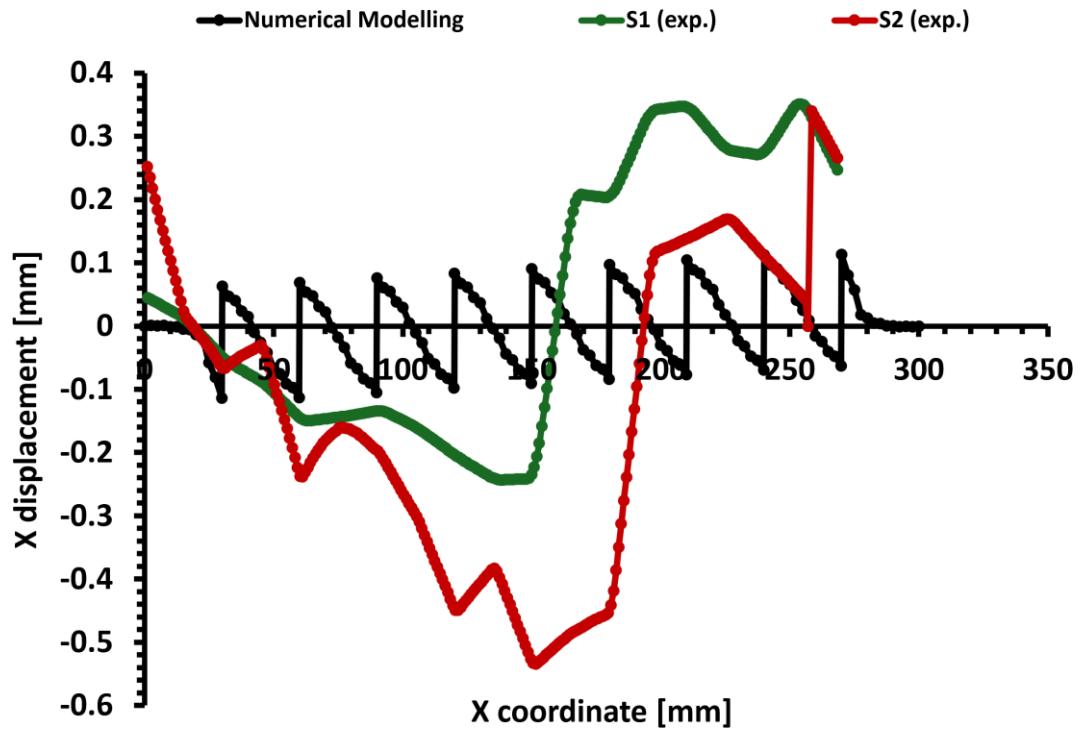
Fig. 6.4 Comparison of two experimental tests and numerical displacement profiles at 12 h under unconstrained conditions, for (a) Soignies, (b) Kruibeke, and (c) Tournai.

6.5.4 Comparison of experimental and numerical results under constrained conditions

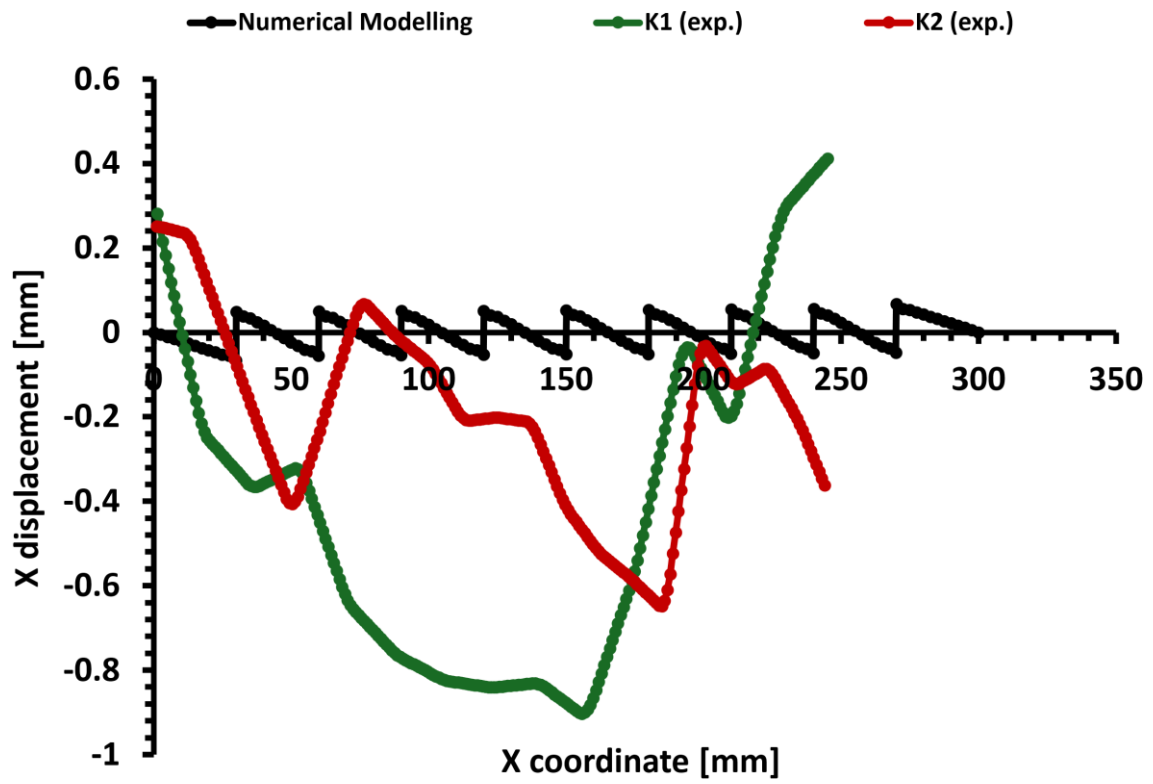
Figure 6.5 compares the numerical and experimental horizontal displacement profiles obtained at 12 h of constrained drying for the soils. In contrast to the unconstrained configuration, the experimental displacement profiles exhibit pronounced local variations along the specimen length, and noticeable differences are observed between the two repeated tests performed for each soil. Those discontinuities in the displacement field are characteristics of crack occurrence. Cracks may appear at random location, depending on the slight heterogeneity of the soil specimen. Consequently, the deformation pattern under constrained drying is less reproducible and the two experimental tests show different displacement profiles.

The numerical model predicts a regular saw-tooth displacement profile resulting from the displacement discontinuities introduced by the interface elements. For the Soignies soil, although differences in displacement magnitude are observed, the numerical profile exhibits local slopes that are comparable to those measured experimentally over several portions of the specimen. However, experimentally, it is observed that the deformation is localized in one main crack while some secondary cracks also develop. On the contrary, numerically, every implemented interface is in opening mode, leading to a saw-tooth displacement profile, not really observed experimentally. A similar observation can be made for Kruibeke, where the displacement magnitude differs from the experimental measurements, while the local evolution of the numerical profile remains comparable in some regions. Among the three soils, the best overall agreement is obtained for Tournai, where the numerical simulation reproduces not only the general trend of the experimental displacement profiles but also a comparable displacement magnitude and the characteristic spacing of the displacement discontinuities.

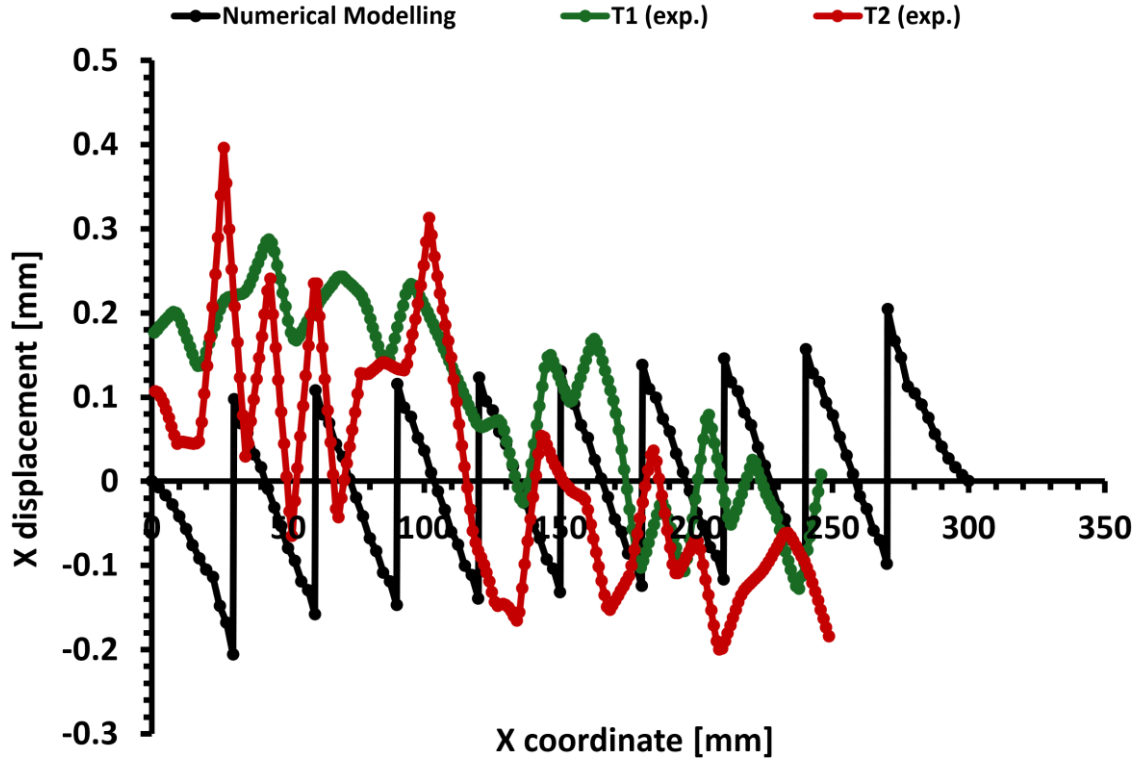
As discussed in Chapter 5, the irregular shape of the experimental displacement profiles is associated with the initiation and propagation of desiccation cracks during drying. Since the present numerical model is based on a continuous finite element formulation, with a localized interface allowing local discontinuities, the location of the crack is imposed by the model and cannot initially form freely everywhere.



(a)



(b)



(c)

Fig. 6.5 Comparison of two experimental tests and numerical displacement profiles at 12 h under constrained conditions, for (a) Soignies, (b) Kruibeke, and (c) Tournai.

6.6 Discussion

6.6.1 Comparison between experimental and numerical results

The comparison between the numerical and experimental displacement profiles demonstrates that the proposed hydro-mechanical model reproduces the main deformation characteristics observed during drying. Under unconstrained conditions, the numerical simulations capture the overall evolution of the horizontal displacement profiles, particularly for the Soignies soil, where both the displacement magnitude and the overall profile are in good agreement with the experimental measurements. For the Kruibeke and Tournai soils, the numerical model reproduces the global displacement trend, although differences in displacement magnitude and profile shape remain.

Under constrained conditions, the numerical model reproduces the characteristic displacement pattern induced by the imposed boundary conditions. The experimental displacement profiles exhibit significant local variations and differences between repeated tests, whereas the numerical simulations predict more regular displacement profiles. As discussed in Chapter 5, these local variations are closely related to the initiation and propagation of desiccation cracks during drying.

6.6.2 Limitations of the numerical model

Although the introduction of interface elements improves the numerical representation of constrained drying conditions, the model remains based on a continuum finite-element formulation. Consequently, the displacement field remains governed by the continuity assumptions adopted in the constitutive formulation, with some possible jumps in the displacement field where interface elements are implemented. This explicit simulation of cracks improves the prediction of the displacement field under constrained conditions but is not able to tackle the random character of crack location.

As a result, the numerical model reproduces the global evolution of the displacement profiles but does not capture the pronounced local fluctuations observed experimentally. These differences become more evident under constrained drying conditions, where desiccation cracking plays a dominant role in the deformation process. Therefore, the remaining discrepancies between the numerical and experimental results mainly reflect the inability of the present modelling approach to explicitly represent discontinuous deformation associated with crack development.

6.7 Conclusions

This chapter presented a coupled hydro-mechanical numerical model for simulating the desiccation process in compacted clayey soils.

The numerical simulations reproduced the overall hydro-mechanical response observed during the experiments. In particular, the model captured the evolution of suction, saturation, stress development, and shrinkage deformation during drying. The comparison with the experimental results showed that the model can reproduce the overall trends of the measured displacement profiles for the three studied soils.

Although the proposed model reproduced the global behaviour of the specimens, the explicit consideration of interface elements in predefined locations allow to account for strain localization but cannot reproduce the spontaneous occurrence that can occur at random locations. Consequently, local variations observed in the experimental displacement fields are not fully reproduced.

Overall, the proposed hydro-mechanical model provides a consistent framework for simulating the desiccation behaviour of compacted clayey soils.

7. CONCLUSIONS AND PERSPECTIVES

7.1 Conclusions

This thesis aimed to improve the understanding of desiccation mechanisms in compacted clayey soils by developing an integrated experimental, constitutive, and numerical framework.

From an experimental perspective, a dedicated setup was developed to investigate the mechanical response of soils subjected to desiccation under controlled boundary conditions. The device enabled direct measurement of tensile forces generated by restrained shrinkage, providing access to a key variable that is rarely quantified in desiccation studies. The reliability of the measurements was ensured through calibration and correction of the mechanical system. In addition, sequential imaging of the soil surface combined with Particle Image Velocimetry (PIV) enabled the characterization of full-field displacement and strain localisation during drying. This coupled approach made it possible to relate the development of deformation patterns to the progressive build-up of tensile stresses.

The experimental results provided a detailed insight into the mechanisms governing desiccation. In particular, tensile stresses were shown to develop progressively during drying, reflecting a time-dependent evolution of the mechanical response and ultimately leading to crack initiation. The combined analysis of force measurements and displacement fields highlighted the importance of coupling mechanical and kinematic observations. The results also confirmed the dominant role of boundary conditions in controlling stress build-up and crack initiation.

From a constitutive perspective, a methodology was proposed to calibrate the effective stress parameter within the framework of generalized effective stress. The relationship between the effective stress parameter and the degree of saturation was established based on shear strength considerations. This approach provides a practical framework to account for suction effects in the mechanical behaviour of unsaturated soils.

From a numerical modelling perspective, a coupled hydro-mechanical approach was implemented to reproduce the behaviour observed during desiccation. The integration of experimentally derived parameters and constitutive relationships allowed the model to consistently capture the evolution of suction, deformation, and stress. The comparison between experimental observations and numerical simulations showed that the main features of the desiccation process, including stress development and deformation patterns, can be satisfactorily reproduced within the proposed framework.

Overall, the main contribution of this work lies in the development of a consistent link between experimental observations, constitutive interpretation, and numerical modelling. This integrated approach provides a improved understanding of desiccation mechanisms, particularly regarding the evolution of tensile stresses and their role in crack initiation.

7.2 Perspectives

The results obtained in this thesis open several directions for further research.

From an experimental point of view, the proposed setup could be extended to investigate a wider range of testing conditions, including different levels of mechanical restraint and controlled drying rates. The influence of initial state variables such as density, water content, and soil type could also be explored in a more systematic manner to generalize the observed behaviour.

From a constitutive perspective, the proposed calibration of the effective stress parameter could be further refined by considering additional experimental data and extended to more complex loading conditions. In addition, further investigation of the link between the evolution of χ and the soil microstructure would provide deeper insight into the physical mechanisms governing the mechanical behaviour during desiccation.

From a numerical modelling point of view, the current approach could be extended to explicitly account for crack initiation and propagation. The use of advanced numerical techniques, such as enriched finite element methods or discrete approaches, would allow a more detailed representation of discontinuities. In addition, the application of the proposed framework to boundary value problems at a larger scale would provide valuable insight into the practical implications of desiccation in geotechnical and geo-environmental structures.

More generally, the integrated methodology developed in this work could be applied to other coupled hydro-mechanical problems in unsaturated soils, contributing to a better understanding of complex multi-physical processes in geotechnical engineering.

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List of Figures

Fig. 2.1 Comprehensive justification of the power law based on microstructural features, as proposed by (Alonso et al. 2010). Full lines and dashed line correspond to Eqs. 2.4 and 2.5, respectively.	14
Fig. 2.2 Typical soil-water characteristic curve of soil (Fredlund and Xing 1994).	16
Fig. 2.3 Soil-water characteristic curves for a sandy soil, a silty soil, and a clayey soil (Fredlund and Xing 1994).	16
Fig. 2.4 Schematic illustration of shear strength envelopes at different matric suction levels based on the extended Mohr–Coulomb failure criterion (Fredlund et al. 1978).	19
Fig. 2.5 Schematic drawings of soil total and matric suction measurements (Bulut and Leong 2008)	24
Fig. 2.6 Schematic use of the osmotic technique (Cui and Delage 1996).	27
Fig. 2.7 Schematic representation of the vapour equilibrium technique used for suction control through a saturated salt solution (Tang and Cui 2005).	28
Fig. 2.8 Schematic representation of a Dynamic Vapour Sorption (DVS) apparatus (Hill et al. 2010).	29
Fig. 2.9 Experimental apparatus proposed by (Varsei et al. 2016) for direct measurement of tensile forces generated during restrained desiccation of compacted soils: (a) desiccation box equipped with load cells; (b) complete experimental setup including the imaging system, weighing scale, and data acquisition units.	36
Fig. 3.1 Particle size distribution curve for the three tested soils	41
Fig. 3.2 Plasticity chart (ASTM D2487 2025)	42
Fig. 3.3 (a) molded and (b) unmolded soil sample for standard proctor test.	43
Fig. 3.4 Standard Proctor Test Results for three tested soils (a) Soignies, (b) Kruibeke, (c) Tournai	45
Fig. 3.5 Experimental setup of the saline-solution suction control technique.	47
Fig. 3.6 Monitoring evolution of relative humidity and temperature inside the chambers during the saline-solution suction control test.	48
Fig. 3.7 Evolution of relative humidity during the saline-solution suction control test.	48
Fig. 3.8 Evolution of mass vs. time of specimen tested with the saline-solution technique (a) Soignies, (b) Kruibeke, (c) Tournai soil.	50
Fig. 3.9 Evolution of Degree of Saturation vs. time of specimen tested with the saline-solution technique (a) Soignies, (b) Kruibeke, (c) Tournai Soil.	52
Fig. 3.10 Dynamic vapor sorption (DVS) device (www.carpor.uliege.be).	53
Fig. 3.11 Water retention curve in terms of (a) water content, and (b) degree of saturation ...	56
Fig. 3.12 Volumetric deformation (ε_v) versus suction under UCS conditions.	57
Fig. 3.13 Uniaxial compression tests at various suction levels: a $s = 4.2$ MPa, b $s = 12.55$ MPa, c $s = 20.8$ MPa, d $s = 37.8$ MPa, e $s = 59.4$ MPa, f $s = 110.1$ MPa	59

Fig. 3.14 Evolution of (a) uniaxial compressive strength and (b) Young's modulus, as a function of suction.....	59
Fig. 3.15 Stress-strain and pore pressure responses under varying confining pressures of Soignies soil: a Deviatoric stress vs. axial strain; b Pore water pressure vs. axial strain; c Stress paths in p' - q space; d Shear stress vs. effective normal stress.	61
Fig. 3.16 Stress-strain and pore pressure responses under varying confining pressures of Kruibeke soil: a Deviatoric stress vs. axial strain; b Pore water pressure vs. axial strain; c Stress paths in p' - q space; d Shear stress vs. effective normal stress.	61
Fig. 3.17 Stress-strain and pore pressure responses under varying confining pressures of Tournai soil: a Deviatoric stress vs. axial strain; b Pore water pressure vs. axial strain; c Stress paths in p' - q space; d Shear stress vs. effective normal stress.	62
Fig. 3.18 Experimental setup for the Brazilian tensile test using a universal testing machine (Testometric Micro 350).	64
Fig. 3.19 Tensile strength vs suction.	64
Fig. 4.1 Mohr circles experienced during the uniaxial compression test expressed in terms of total stress and generalized effective stress (Gerard et al. 2015)	70
Fig. 4.2 Sensitivity of χ calibration to $\pm 10\%$ variation in UCS for the tested soils: (a) Soignies, (b) Kruibeke, and (c) Tournai. Solid, dashed, and dotted lines represent the measured UCS, +10% UCS, and -10% UCS cases, respectively.	73
Fig. 4.3 Mohr circles at failure expressed in terms of generalized effective stress for uniaxial compression tests performed at different initial suctions: (a) Soignies soil, (b) Kruibeke soil, (c) Tournai soil	75
Fig. 4.4 Pore size distribution (PSD) of the three soils obtained from mercury intrusion porosimetry (MIP); Differentiation between micro and macro voids is delimited by the lowest point of the PSD at intermediate pore diameter; (a) Soignies soil, (b) Kruibeke soil, (c) Tournai soil	77
Fig. 4.5 Evolution of total void ratio (e), macro void ratio (e_M) and micro-void ratio (e_m) during drying, assuming that the micro-void ratio (intra-aggregate) remains constant; (a) Soignies soil, (b) Kruibeke soil, (c) Tournai soil	79
Fig. 4.6 χ parameter as a function of the degree of saturation of the three soils: (a) Soignies, (b) Kruibeke, (c) Tournai according to four different approaches: (i) From uniaxial compression strength at various suction; (ii) Fitting through a power law of the point obtained by approach (i); (iii) from Eq. (2) assuming the void ratio remains constant upon drying; (iv) from Eq. (2) with varying macro void ratio upon drying.	81
Fig. 5.1 Constrained desiccation box showing the end-wall screws used to anchor the soil.	86
Fig. 5.2 Schematic illustration of the split desiccation box showing the position of the inline load cells and end-wall screws.....	87
Fig. 5.3 Calibration of the loading system: (a) relationship between the applied load and the measured force recorded by the load cells; (b) relationship between the applied load and the corresponding friction force used for force correction.	89
Fig. 5.4 Schematic illustration of the constrained desiccation box showing the split aluminium mould, external dimensions, end-wall screws used to anchor the soil, sliding rail system, and inline load cells.	91
Fig. 5.5 Static compaction of the soil specimen inside the aluminium desiccation box using a hydraulic jack and a rigid steel plate to achieve the target dry density.	92

Fig. 5.6 Compacted soil specimen inside the aluminium desiccation box after preparation and surface levelling before testing.	93
Fig. 5.7 Desiccation test setup showing the installed specimen, aluminum box, camera, and lighting system used for continuous monitoring and data acquisition.	94
Fig. 5.8 Measurements of particle tracking of 2D unsteady flow around a cylinder by using cinematography (Nayler and Frazer 1917)	97
Fig. 5.9 Principles of PIV analysis (White and Take 2002).....	100
Fig. 5.10 Evolution of the forces measured by the two load cells (F1 and F2) and the corresponding total tensile force during a desiccation test of <i>S2</i>	103
Fig. 5.11 Relationship between tensile stress evolution and crack development during a representative constrained desiccation test. (a) Tensile stress–time curve obtained from the load cell measurements. The numbered points indicate the times at which the surface photographs were captured. (b–g) Sequential images of the soil surface corresponding to the numbered points on the stress curve, showing the progressive development of cracking during drying.....	105
Fig. 5.12 Evolution of the Tensile Stress during the desiccation test for three tested soils (a) Soignies, (b) Kruibeke, (c) Tournai, for two boundary conditions; Blue and red graphs represent the constrained and unconstrained conditions, respectively.....	107
Fig. 5.13 Crack pattern obtained from (a) Soignies Constrained condition (b) Kruibeke Constrained Condition (c) Tournai Constrained Condition (d) Tournai Unconstrained Condition.....	110
Fig. 5.14 Desiccation test results for <i>S1</i> under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	112
Fig. 5.15 Desiccation test results for <i>S2</i> under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	113
Fig. 5.16 Desiccation test results for <i>S3</i> under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	113
Fig. 5.17 Desiccation test results for <i>S4</i> under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	114
Fig. 5.18 Desiccation test results for <i>K1</i> under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	114

Fig. 5.19 Desiccation test results for <i>K2</i> under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	115
Fig. 5.20 Desiccation test results for <i>K3</i> under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	116
Fig. 5.21 Desiccation test results for <i>K4</i> under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	116
Fig. 5.22 Desiccation test results for <i>T1</i> under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	117
Fig. 5.23 Desiccation test results for <i>T2</i> under constrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	117
Fig. 5.24 Desiccation test results for <i>T3</i> under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	118
Fig. 5.25 Desiccation test results for <i>T4</i> under unconstrained conditions: (a) specimen surface before crack initiation, (b) surface after cracking, (c) PIV-derived horizontal displacement field, and (d) PIV-derived horizontal linear strain field. Blue indicates extension and red indicates compression. Displacement and strain magnitudes are reported in calibrated physical units.	119
Fig. 5.26 Horizontal displacement profiles along the specimen midline for test <i>S1</i> at 3 h, 6 h, 12 h, and time at peak stress.	121
Fig. 5.27 Horizontal displacement profiles along the specimen midline for test <i>S2</i> at 3 h, 6 h, 12 h, and time at peak stress.	121
Fig. 5.28 Horizontal displacement profiles along the specimen midline for test <i>S3</i> at 3 h, 6 h, 12 h, and time at peak stress.	122
Fig. 5.29 Horizontal displacement profiles along the specimen midline for test <i>S4</i> at 3 h, 6 h, 12 h, and time at peak stress.	122
Fig. 5.30 Horizontal displacement profiles along the specimen midline for test <i>K1</i> at 3 h, 6 h, 12 h, and time at peak stress.	123

Fig. 5.31 Horizontal displacement profiles along the specimen midline for test <i>K2</i> at 3 h, 6 h, 12 h, and time at peak stress.	124
Fig. 5.32 Horizontal displacement profiles along the specimen midline for test <i>K3</i> at 3 h, 6 h, 12 h, and time at peak stress.	124
Fig. 5.33 Horizontal displacement profiles along the specimen midline for test <i>K4</i> at 3 h, 6 h, 12 h, and time at peak stress.	125
Fig. 5.34 Horizontal displacement profiles along the specimen midline for test <i>T1</i> at 3 h, 6 h, 12 h, and time at peak stress.	126
Fig. 5.35 Horizontal displacement profiles along the specimen midline for test <i>T2</i> at 3 h, 6 h, 12 h, and time at peak stress.	126
Fig. 5.36 Horizontal displacement profiles along the specimen midline for test <i>T3</i> at 3 h, 6 h, 12 h, and at failure time.	127
Fig. 5.37 Horizontal displacement profiles along the specimen midline for test <i>T4</i> at 3 h, 6 h, 12 h, and time at peak stress.	127
Fig. 5.38 Schematic illustration of internal moisture gradient and associated stress distribution for (a) and (b) localized cracking patterns.....	129
Fig. 6.1 Geometry of the numerical model, finite element mesh and interface elements (red lines) adopted for the drying simulations.....	136
Fig. 6.2 Horizontal displacement profiles (mm) under unconstrained drying conditions at 6 h and 12 h for (a) Soignies, (b) Kruibeke, and (c) Tournai.	141
Fig. 6.3 Horizontal displacement profiles (mm) under constrained drying conditions at 6 h and 12 h for (a) Soignies, (b) Kruibeke, and (c) Tournai.	144
Fig. 6.4 Comparison of two experimental tests and numerical displacement profiles at 12 h under unconstrained conditions, for (a) Soignies, (b) Kruibeke, and (c) Tournai.	146
Fig. 6.5 Comparison of two experimental tests and numerical displacement profiles at 12 h under constrained conditions, for (a) Soignies, (b) Kruibeke, and (c) Tournai.	149

List of Tables

Table 2.1 Various expressions of χ parameter as a function of the degree of saturation or suction, and some micro-structural features	11
Table 3.1 Mineralogical composition of the tested soils, focusing on clay minerals (Schmitz et al. 2004)	41
Table 3.2 Atterberg limit of the studied materials	42
Table 3.3 Specific gravity of the studied materials	43
Table 3.4 Standard Proctor Test Results	45
Table 3.5 Dimensions of the samples for each test (WRC= Water retention curve, UCS= Uniaxial compression test, Triax CU = Consolidated undrained triaxial test)	46
Table 3.6 Initial matric and degree of saturation of the soils under as-compacted conditions	46
Table 3.7 Imposed relative humidity and equivalent suction for the saline-solution method	49
Table 3.8 Calibration Parameters (P_r and n) Derived from the van Genuchten Model	55
Table 3.9 Hydraulic Conductivity (k) of studied soils.	65
Table 3.10 Physical parameters of the three tested soils.	67
Table 4.1 Microstructural parameters of the three soils as deduced from mercury intrusion porosimetry	77
Table 5.1 Geometrical characteristics of the desiccation box	86
Table 5.2 Summary of desiccation test programme	101
Table 5.3 Summary of measured tensile strengths and observed crack patterns for all desiccation tests	130
Table 6.1 Mechanical properties of the interface elements	135
Table 6.2 Summary of material parameters adopted for the hydro-mechanical simulations	137