Ollscoil na hÉireann, Corcaigh

National University of Ireland, Cork

University College Cork, Ireland
Coláiste na hOllscoile Corcaigh

Experimental Behaviour Study of Peat at Small Stress Level

Thesis presented by

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for the degree of

Doctor of Philosophy

University College Cork

School of Engineering and Architecture

Civil, Structural and Environmental Engineering

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2024

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DECLARATION

This is to certify that the work I am submitting is my own and has not been submitted for another degree, either at University College Cork or elsewhere. This dissertation is the result of my own work and includes nothing which is the outcome of work done in collaboration except as specified in the text. All external references and sources are clearly acknowledged and identified in the thesis. All external references and sources are clearly acknowledged and identified within the contents. I have read and understood the regulations of University College Cork concerning plagiarism and intellectual property.

Di Wang

May 2024

ACKNOWLEDGEMENT

I owe my greatest gratitude to my supervisor Dr. Zili Li, for the opportunity for me to come to Ireland, his continuous support and genuine encouragements. I sincerely thank him for his supervision and all the adventures for the years of my PhD research. Without his support, this thesis would never have been done.

I really appreciate the help and support from McCarthy Eileen for her help in collecting the peat samples, Rodrigues de Amorim Romain for his help with the physical tests of peat.

I would like to thank my friends I have made in UCC, Dr. Zhipeng Xiao, Dr. Chao Wang, Qinglai Zhang, Ran Zhang, Jian Liu, Aohui Ouyang, Ye Li, Roberto Pantoja. Thanks for my burger-mate Lara.

To my parents, my elder sisters and my boyfriend Dengyi Wang for their understanding, support and love.

DISSEMINATION OF RESEARCH

1. Wang, D., Xu, Q., McCarthy, E., Rodrigues de Amorim, R., **Li Z**.*, (2022), Experimental investigation of unconsolidated undrained shear behaviour of peat, Bulletin of Engineering Geology and the Environment, 81, 60
2. Wang, D., Li, Z.*, (2022), Shear behaviour of peat at different stress levels, ICE Geotechnical Engineering, https://doi.org/10.1680/jgeen.22.00058
3. Wang, D., Xu, D., McCarthy, E., Li Z.*, (2020), Shear behaviour of peat at different stress levels, Civil Engineering Research in Ireland conference 2020, 665-670
4. Wang, D., McCarthy, E., Rodrigues de Amorim, Romain, Li Z.*, (2018), Experimental investigation of undrained shear behaviour of peat, Civil Engineering Research in Ireland conference 2018, 271-276

ABSTRACT

Peatland is widely distributed in Ireland, Scotland, Northern Eurasia, and North America, as well as other regions across the world. Compared to fine-grained mineral soils, peat usually contains a much higher content of organic matter and water, which in turn results in lower shear strength, high compressibility, and other unfavourable engineering characteristics.

To investigate the shear behaviours of peat, a series of standard classification tests is first conducted to determine the physical properties of peat, followed by 28 unconsolidated undrained triaxial tests. In addition, 14 consolidated undrained triaxial compression tests on samples from 1.65 m depth subjected to relatively small stress levels from 10.4 kPa to 40.5 kPa are conducted in this study.

The results show that the undrained deviatoric shear resistance of peat grows continuously with increasing shear strain, but peat has no obvious peak strength even at a very large strain (i.e. 25%). To compare the stiffnesses of peat specimens, the secant shear modulus G is normalised with the shear modulus $G_{0.1\%}$ at a relatively small strain of 0.1%. A hyperbolic function is then adopted to fit the relationship between the normalised stiffness and the normalised shear strain. Over 78% of the data falls within a ±30 % margin within 5% shear strain, while the predicted stiffness outside the margin is usually underestimated.

For peaty soil, the membrane correction effect on peat shear resistance is strain dependent and becomes significant above 10% shear strain. A critical state line for peat is determined based on the maximum curvature approach. Of the data recorded for peat, 78% falls within the range of 30–60°. This increases to 90.4% when excluding points lower than 10 kPa. Previous test data for very low stress levels (less than 10

kPa) might not be sufficiently reliable due to the limitations of conventional triaxial testing apparatuses, specimen preparation, and other experimental constraints. In addition, organic content plays an important role in the peat shear behaviour. In general, when the organic content exceeds 75%, the deviator stress mimics that of organic soils. Otherwise, the peat behaves more like a mineral soil. In peat samples with organic content greater than 75%, the direct shear box test gives higher estimates of shear strength than the triaxial shear test. However, these values may not be entirely accurate as the mechanism of direct shear acts only at the centre of the specimen. Triaxial testing allows for the development of shear failure throughout the entire specimen, providing a more representative assessment of shear behaviour.

The findings from this study provide additional insight into the mechanical behaviour of peat, particularly at small stress levels. The proposed stiffness degradation curve and critical state line based upon the maximum curvature approach are of value for infrastructure construction and assessments on/across peatland (e.g., wind farms, floating roads) in the light of growing energy and transportation demands.

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LIST OF SYMBOLS

A_0	initial cross-sectional area of the sample (in mm)
A_c	post-consolidation average cross-sectional area of the sample (in mm)
C_c	compression index
D_0	initial sample diameter
D_c	diameter of specimens after the consolidation stage
E_m	young’s modulus for the membrane material
G	shear modulus
G_{max}	maximum shear modulus
$G_{0.1\%}$	shear modulus at 0.1% shear strain
H	sample height
H_0	initial sample height
H_1	the height of specimen at the start of the consolidation stage
H_2	the height of specimen at the end of the consolidation stage
K_{cu}	y-intercept
m_v	coefficient of compressibility
M_{cu}	the slope
OC	organic content
N	ignition loss
OCR	over consolidation ratio
t_m	total thickness of the membrane enclosing the specimen

t_{50}	corresponding to 50% pore pressure dissipation
γ	triaxial shear strain increment
$\sigma_1 - \sigma_3$	deviator stress,
$(\sigma_1 - \sigma_3)_m$	applied axial stress by the axial loading cell;
$(\sigma_1 - \sigma_3)_{mb}$	membrane stiffness correction.
φ'	internal friction angle
V	sample volume
V_0	initial sample volume
V_s	shear wave velocity
λ	slope of the isotropic normal compression line
κ	slope of the isotropic unloading-reloading line
ν	Poisson's ratio
e	void ratio
ε_a	axial strain
ε_r	radial strain
ε_v	volumetric strain
$\Delta\varepsilon_a$	axial strain increment
$\Delta\varepsilon_v$	volumetric strain increment
ω	Water content
s_u	undrained shear strength
ρ	peat density
d	is the maximum drainage path; and
T_v	the Time factor, which relates to the degree of consolidation
c_v	the coefficient of consolidation

p' vertical effective stress

1 INTRODUCTION

1.1 Background information on peat

Peat is defined as the surface organic layer of a soil that consists of partially decomposed organic matter. It is derived mostly from plant material that has accumulated under conditions of waterlogging, oxygen deficiency, high acidity, and nutrient deficiency. It has distinctly different mechanical properties compared to typical mineral soils.

The organic solids in peat, such as shrub rootlets, plant root hairs, and rhizoids, vary in size. These materials range from larger elements like wood fragments and coarse fibres (with stems and roots exceeding 1 mm) to finer components like leaves, stems, and roots measuring less than 1 mm. Additionally, peat contains amorphous matter, as noted by Hobbs (Hobbs, 1986). The arrangement of organic fibres in peat is heavily influenced by factors including the originating plants, the conditions under which the peat formed, and the extent of its decomposition.

Peat's organic content has the potential for further decomposition, which may occur either naturally or due to human activities, as highlighted by Matthiesen (Matthiesen, 2004). This decomposition process leads to the gradual deterioration of the peat structure, a reduction in its organic content, and the generation of biogas. The geotechnical properties of peat gradually evolve over time, and favourable conditions, often unintentionally created by human activities, can accelerate its decomposition rate. Despite the importance of peat decomposition, there has been limited in-depth research on both the decomposition process itself and its effects on the geotechnical characteristics of peat.

In particular, no comprehensive investigation has explored the direct impact of decomposition on peat's compressibility. This research gap may be attributed to the widespread belief among researchers that the natural decomposition of peat progresses at a very slow rate and thus has an insignificant impact on the long-term performance of engineering projects. For example, Hobbs suggested that questions concerning peat decomposition were of academic and scientific interest rather than engineering relevance (Hobbs, 1986). As a result, while there is a wealth of high-quality data available on the geotechnical properties of fibrous peat, the understanding of the microstructure and geotechnical behaviour of amorphous peats remains limited, as discussed by Santagata (Santagata, 2008).

Despite the construction industry's dismissals of this research avenue, there is evidence that peat's decomposition rate could be accelerated prior to the commencement of construction activities if the influencing factors and environmental conditions are favourable (Pichan and O'Kelly, 2013). This could make it possible to positively influence the compression behaviour of peat over the design lifespan of engineering projects.

Peat has many special physical and mechanical properties that differ from those of typical mineral soils. Because of these unique properties, peat presents unique challenges in geotechnical engineering, particularly in regions like Ireland, where its widespread distribution has led to many safety and economic issues. The high water content of peat means that it can undergo significant volume changes; thus, the compressibility of peat is much greater than that of other soil types, such as clay and sand. In addition, the complex structure of peat, characterised by the presence of fibrous organic matter and varying degrees of humification, influences its consolidation behaviour. Peat landslides, such as the landslide caused by excessive rainfall in Galway in 2006, have resulted in property damage of up to tens of millions of euros.

With the continued development and diversification of human production and life, more and more buildings must be built on peat soil. In the 20th century, most structures built on peat soil foundations were embankments and highways. However, in the modern world, increasingly complex buildings have placed new demands on the bearing capacity of peat soil foundations. Consequently, the investigation of peat soil has emerged as a significant research priority. A primary concern is the failure of dams, foundations, and other structures, which can often be attributed to the low shear strength associated with peat. To prevent these failures, it is essential to thoroughly examine the physical and mechanical properties of peat soils. This study compares the mechanical characteristics of peaty soils from various regions to identify the differences and similarities of peat in different areas. This comparative analysis facilitates a deeper understanding of the common traits of peaty soils and enables a more precise assessment of physical and mechanical behaviour.

Peat deformation remains a serious concern regarding infrastructure, especially in regions like Ireland with extensive peatlands. The high compressibility, low shear strength, and anisotropy of peat lead to challenges such as excessive settlement, structural distress, and increased maintenance costs.

Settlement is a key concern, as peat’s high organic content causes large and prolonged volumetric changes under loading. Differential settlement often results in structural damage and costly remediation.

Slope instability and shear failures are also major risks due to peat's low shear strength and anisotropic nature. Shear failures can lead to landslides and structural collapse, thus posing serious safety and environmental hazards.

Economically, peat complicates construction projects, often necessitating more expensive construction methods like ground improvement and lightweight fill as well as long-term maintenance. Environmentally, draining and disturbing peatlands can release stored carbon and harm biodiversity.

Effectively managing peat deformation requires thorough geotechnical investigation, advanced modelling, and careful design. Testing peat for organic content, strength, and stiffness helps engineers more accurately predict its behaviour. With careful planning and monitoring, it is possible to build safely and sustainably on peat soils.

1.2 Objectives and Scope of the Research

Currently, the study of peat is in its foundational phase, with a primary emphasis on understanding peat's fundamental physical attributes and basic mechanical behaviours. This research involves a comprehensive investigation employing both undrained unconfined triaxial tests and undrained confined triaxial tests. The primary objective is to generate valuable empirical data to formulate an effective methodology for determining the reduction coefficient of peat stiffness when peat is subjected to minor shear strains.

The motivation for the research lies in the need to comprehend how the stiffness of peat changes when it undergoes shear deformation. By closely examining the stiffness degradation of peat under shear strain, the research aims to contribute to the development of a constitutive model suitable for peat. This model will serve as a predictive tool, allowing researchers to anticipate the effective shear strength of peat under various conditions and shear strains.

In this study, a series of laboratory tests and analyses were conducted to advance the understanding of how peat behaves under stress, particularly in terms of its stiffness characteristics, and to translate this knowledge into a practical framework that can be applied to engineering and construction projects. Ultimately, this work aims to enhance the predictability and reliability of geotechnical assessments involving peat, thereby contributing to safer and more efficient infrastructure development.

Peatlands cover approximately 3% of the Earth's land surface and are predominantly located in the northern hemisphere across Europe, North America, and Asia (Joosten and Clarke, 2002; Xu, 2018). Europe hosts substantial peat deposits, particularly in Ireland, the United Kingdom, the Baltic states, Germany, Scandinavia, and, notably, extensive regions of western Russia (Montanarella, 2006; Connolly and Holden, 2009). Globally, Canada, Indonesia, and Russia contain the largest peatland areas, which emphasises the significance of peat research on an international scale (Page and Baird, 2016; Yu, 2010).

Peat soils pose considerable geotechnical challenges due to their inherent low strength, high compressibility, and complex hydrological characteristics, which significantly influence infrastructure and construction activities worldwide (Boylan and Long, 2014; Mesri and Ajlouni, 2007). In Europe, and especially in Ireland, the extensive presence of peat soils has caused substantial economic and safety challenges, underscoring the importance of targeted research into peat behaviour and engineering solutions (Long and Jennings, 2006; Dykes and Kirk, 2006).

Given the wide distribution of peat soils across the globe, examining Ireland's peat-related challenges and solutions has significant international relevance and can offer valuable insights and benchmarks applicable to similar geotechnical contexts in other countries.

In summary, the objectives of this study are listed below.

The first objective of this study is to investigate the stiffness degradation of peat under small shear strains. This study determines how the shear modulus of peat changes under small-strain conditions using triaxial tests and develops a reduction coefficient for peat stiffness to improve constitutive modelling for geotechnical applications.

Experimental triaxial tests reveal that peat exhibits unique shear behaviour distinct from mineral soils. In this study, deviator stresses showed continuous growth with increasing strain without observable peak strengths or shear band formation, positive pore pressures developed during shearing, and no conventional failure modes were observed.

The second objective of this study is to assess the influence of organic content on the shear behaviour of peat. It examines how varying organic content affects the shear strength, compressibility, and deformation characteristics of peat and compares results from triaxial and direct shear tests to establish correlations between organic composition and mechanical responses.

The third objective of this study is to investigate the effects of specimen size and anisotropy on the mechanical behaviours of peat. This involves comparing disturbed and undisturbed samples with varying organic content and depth to evaluate scale effects and anisotropic responses under triaxial loading conditions.

The fourth objective of this study is to assess and correct for membrane stiffness effects in triaxial testing. This study aims to apply corrections to improve the accuracy of shear strength measurements, thereby enhancing the reliability of experimental results.

1.3 Research Approach

The research approach involved conducting a series of physical experiments to explore the fundamental physical properties of peat, with a specific focus on contrasting these properties with those of other mineral soils, such as sands and clay. What sets peat apart from other soils is its high water content and considerable organic content. These characteristics significantly influence the behaviour of peat under stress, and particularly its shear behaviour.

First, the investigation compared peat and conventional mineral soils to clarify the distinctive characteristics of peat, such as its high water and organic content, and to understand how these characteristics differ from those of mineral soils like sands. Systematic experimentation yielded insights into how the unique composition of peat impacts its mechanical behaviour, especially its shear behaviour.

To evaluate the mechanical properties of peat, two primary testing methodologies were employed in this study: unconsolidated undrained triaxial tests and consolidated undrained triaxial tests. These tests provided critical data on peat’s responses to various loading conditions and stress levels.

In essence, this research aims to bridge the knowledge gap in the literature regarding the physical and mechanical properties of peat. By conducting precise experiments and data analysis, this study aims not only to advance the understanding of peat but also to provide valuable insights that can improve engineering assessments regarding the predictability and reliability of peat. This knowledge contributes to the development of safer and more effective construction practices in areas where peat is commonly encountered.

1.4 Thesis structure

Following this introduction, Chapter 2 contains the literature review, which introduces peat and its properties with a focus on shear strength. Notably, Section 2.4 has a particular focus on literature relating to the constitutive frameworks of peat.

Chapter 3 introduces the two main research sites that were used in this research: Omagh in Northern Ireland and an area near Millstreet, County Cork, in southern Ireland. It summarises the geological background and basic properties of each site. In addition, the sampling techniques used at the sites are described, and the results of the ground-penetrating radar (GPR) and in situ vane tests are presented. In addition, it describes this study's use of laboratory physical property tests on disturbed samples to investigate the physical properties of peat, as well as the test procedures and results.

Chapter 4 describes the unconsolidated undrained triaxial tests used to test peat specimens under undrained conditions and monitor the stress-strain relationship and shear modulus during shear. This method is briefly described, and studies assessing its applicability to peat are summarised.

Chapter 5 presents the results of the consolidated undrained triaxial testing. The influence of sample disturbance on the strength and compressibility properties of peat and organic soil is also considered. Both the compression and small-strain shearing behaviour of peat is discussed in this chapter, as is the effect of membrane stiffness.

The test data are compared with data from the literature and site investigations carried out for this study.

Following the presentation of the experimental tests in Chapters 3, 4, and 5, Chapter 6 discusses the influence of organic content on the shear behaviour of peat, highlighting the significant effects attributable to this factor. This chapter presents the test results obtained from the various methods used in this study, including triaxial tests and direct shear tests. By analysing these results, Chapter 6 aims to provide a comprehensive understanding of how organic content impacts the mechanical responses of peat under shear loading conditions. The main equations proposed for membrane stiffness correction are also discussed in this chapter. Based on the discussion and the undrained condition of the triaxial tests conducted at shear strains below 20%, an appropriate equation was adopted in this study.

Finally, Chapter 7 presents a summary of the conclusions and recommendations from this study.

2 LITERATURE REVIEW

2.1 Introduction and distribution of peat

Peat is an organic deposit made up of partially decomposed vegetal organic matter. It develops slowly in peatlands where the accumulation of organic matter overrides decomposition due to anoxic conditions caused by water saturation, a lack of oxygen, and high acidity. Under natural conditions, peat is composed of water, gas, and solid matter, as shown in Figure 2-1.

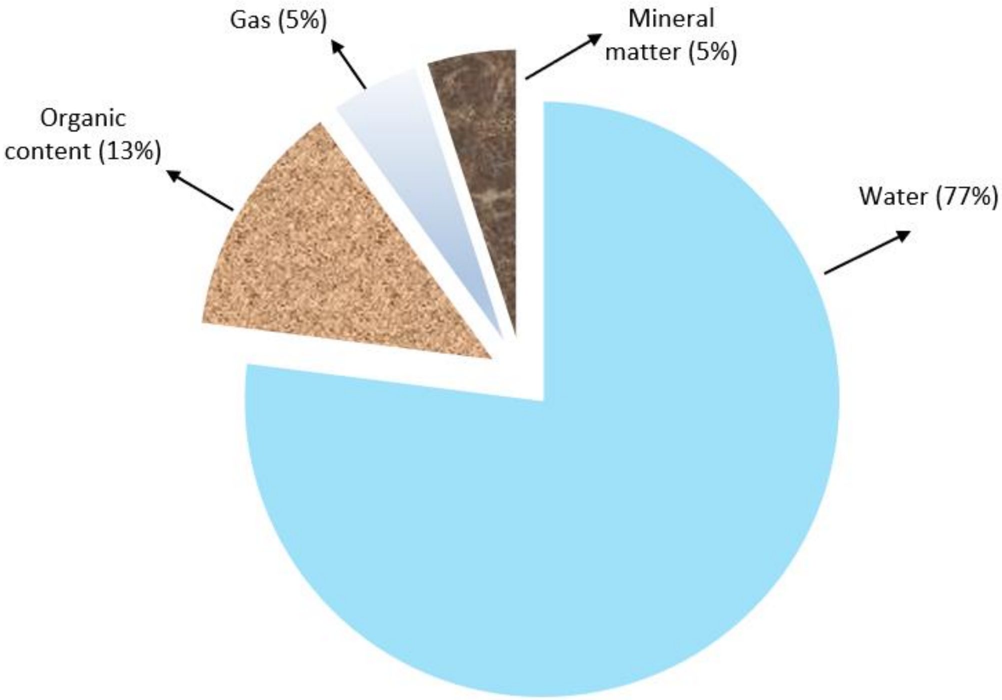


Figure 2-1: The composition of peat

Peat forms in both raised and depressed terrain, dependent on sufficient precipitation levels. In regions such as the British Isles and the west coast of Canada, raised bogs can develop on upland terrain, including hillsides. In contrast, Ireland primarily features peat bogs on the peripheries of poorly drained depressions, largely influenced by high rainfall and the presence of small to large lakes. Peat bog formation occurs through a process known as terrestrialisation. This process begins at the edge of a shallow water body where, under conditions of low pH, anoxia, and specific temperature ranges, the decomposition of plant material is significantly slowed, allowing for its accumulation and the gradual infilling of the aquatic environment (Hobbs, 1986). This leads to the formation of a range of peats depending on the type of plant material accumulated, state of decay, and access to oxygen. As the depth of the peat horizon increases, vertical stresses on the peat also increase, leading to compression of the peat. These high vertical strains cause the plant fibres that make up the bulk of the peat to align

Globally, peat soils are distributed across 59 countries and regions, covering a total area exceeding 415,300 km². Canada holds the distinction of the largest distribution area. Peatland is widely distributed in northern Eurasia and North America, as well as other countries worldwide. In total, it covers an area of 423 million hectares (Xu, 2018), which represents 2.84% of the Earth's total land surface area.

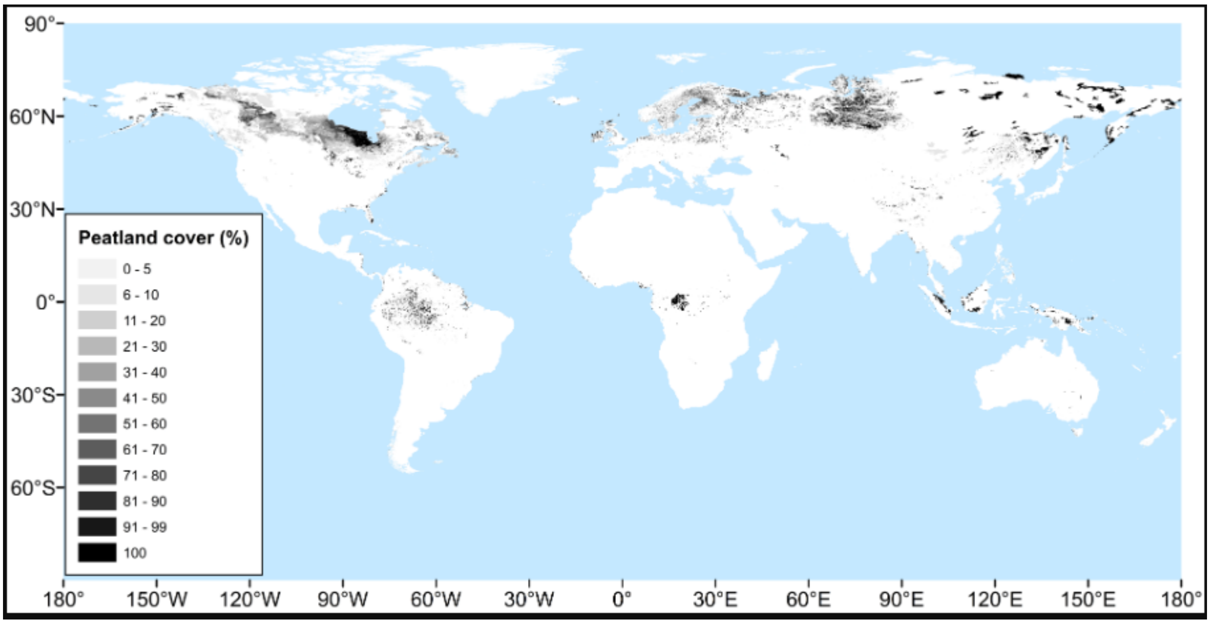


Figure 2-2: Distribution of peatlands expressed as proportion of land surface (Xu, J., 2018)

China has the seventh largest peat distribution globally, with peat soils covering approximately 42,000 km². Within China, significant deposits of peat soils are found in regions such as the Changbai Mountains, the Daxinganling Mountains, the Sanjiang Plain, the mountainous areas of Xinjiang, the Zoige Plateau, the Qinghai-Tibet Plateau, the Yunnan Plateau, and the plains of the middle and lower Yangtze River. The rapid development of infrastructure in China, coupled with the increasing demand for high-grade road construction, necessitates the passage of highways through areas densely populated with peat soil. Concurrently, China’s overseas highway construction projects in regions such as Africa and Southeast Asia have accelerated. In these areas, peat soils are prevalent in coastal alluvial plains, river basins, and low-lying wetlands, and pose significant technical challenges for roadbed construction.

One of Ireland’s most characteristic features is the bog. Bogland covers an area in excess of 12,000 km², approximately 17% of the national land area (Geography of Ireland, 2009). Irish peatlands can be categorised into two primary types: blanket bogs and raised bogs (Abbot, 2008). Blanket bogs are expansive and generally form in wet or upland areas. Raised bogs are smaller and generally form in lowland areas.

Due to the high water content and the complex geological conditions of peatland, more than three peat landslides and other relevant peat-related geological disasters occurred per month worldwide from 1979 to 1985 (Alexander, 1985). In addition, growing energy demand and rapid urbanisation have necessitated an increasing number of onshore wind farms, floating roads, intercity transportation networks, and other infrastructure, some of which inevitably must be built on or across peatland. As such, for future construction projects, it is essential to understand the shear behaviour of peat and its associated failure mechanisms under a variety of different stress levels.

According to the Geological Survey Ireland (GSI; www.gsi.ie), more than 10 peat landslides have occurred over the past 20 years. Peat landslides made up more than half of the landslides in Ireland and have resulted in substantial economic losses.

2.2 Basic physical properties of peat

Peat soil, a collective term encompassing both peat and peat-affected soils, is characterised by its high water content, which can range from 100% to 1,600%. Additionally, peat soils exhibit a significant void ratio of approximately 3.0 to 12.0 and possess considerable compressibility, with a compression coefficient reaching values between 5 and 10 MPa^{-1}. Its bearing capacity is remarkably low, with an allowable bearing capacity typically ranging from 30 to 40 kPa. Furthermore, peat soils are distinguished by high organic matter content, which can constitute 30% to 90% of the total soil composition.

It is necessary to conduct an in-depth study of the engineering characteristics of peat soil to fully comprehend its differences from conventional soft clays. This understanding is essential for developing effective engineering solutions to the unique challenges posed by peat soil in construction projects.

2.2.1 Texture and colour

Peaty soil is described as brown and very soft, with a high moisture content and made up of organic matter. The texture and colour of peat soils change over time according to the level of decomposition, the environmental conditions, and the geology of the location. As peat has an extremely high water content, the texture of peat is very soft, and it is easily cut or broken.

Figure 2-3: Peat block taken from the north of Ireland

As shown in Figure 2-3 above, the colour of peat is lighter brown at the top and becomes darker with increasing depth. The degree of peat decomposition is often reflected in its colour, with more decomposed peat exhibiting a darker appearance.

Figure 2-3 above also shows that the peat at depths of 0.35 m to 0.95 m below ground level is composed of fine, long fibres and displays anisotropy. Below 0.95 m, no obvious fibres or roots can be seen, which reflects a higher degree of decomposition.

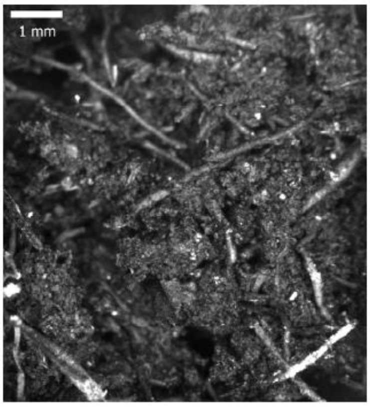
2.2.2 Structure

Peat is formed by the long-term accumulation of plant roots and leaves in the soil. Therefore, peat soil contains criss-crossing plant rhizomes. The structure of peat soil is complicated due to its formation, which is different from that of pure mineral soils like clay, silt, and sand.

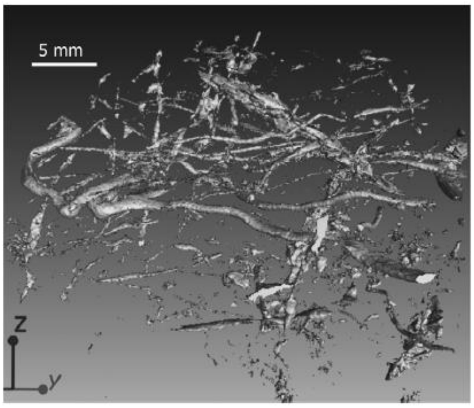
To investigate the microstructural characteristics of peat, researchers have employed advanced microscopic techniques, including scanning electron microscopy (Landva, 2007; Landva and Pheeney, 1980; Wilson, 1972). These studies have shown that the microscopic particles within peat are interconnected and exhibit compressibility, in contrast with the relatively incompressible particles found in mineral soils. In addition, the microstructures and fibres in peat are interwoven in an irregular manner, which significantly contributes to peat's anisotropic behaviour.

Jommi (2021) divided the fibres in peat samples into two groups: small fibres of less than 3 mm and large fibres of over 20 mm. Both groups can be easily recognised and identified, as shown in Figure 2-4 below.

a)



b)



c)

Figure 2-4: (a) A micro CT scan of reconstituted peat; (b) micrographs of natural peat fabric from polarised microscopy; (c) a fibrous network of natural peat reconstructed from a micro CT scan (Jommi, 2021)

Many scholars have shown that the plant fibres in peat have an important impact on the shear stress of the peat soil. Muraro (2019) showed that the presence of medium and long fibre structures (1–3 cm) in peat samples causes inconsistency in transverse and longitudinal strains in triaxial experiments.

2.2.3 Water content

As opposed to organic soil, peat is formed through the decomposition of organic matter under conditions of high moisture and anoxia. Therefore, the water content of environments where peat exists is very high, usually between 800% and 1,600%, which exceeds that of ordinary soil by a factor of ten or more. Landva and Pheeney (1980) reported that the water content of Sphagnum leaves ranged between 900% and 1,100%, compared with an average value of 670% for Sphagnum stems. It is important to note that the water content of peat tends to decrease with increasing sampling depth.

Figure 2-5 below shows the relationship between the water content of peat and the sampling depth, based on data collected from previous studies (O'Kelly and Zhang, 2013; Boulanger, 1998; Hebib and Farrell, 2001; Zwanenburg and Jardine, 2015; Edil and Wang, 2000; Long and Jennings, 2006).

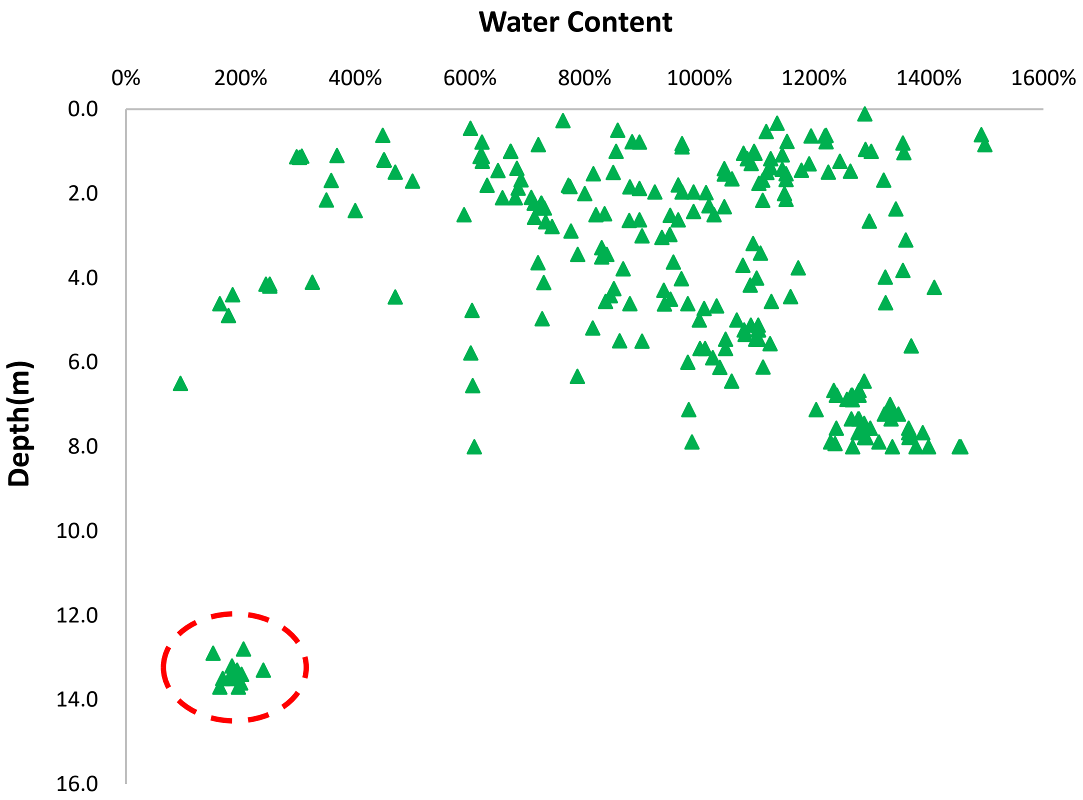


Figure 2-5: Comparison of water content and sampling depths obtained from previous studies

As shown in Figure 2-5 above, the water content of peat typically ranges from 600% to 1,400% across various depths. Notably, an outlying dataset reported by Boulanger and Arulnathan (1998) indicated water content levels of approximately 200% at depths between 12 m and 14 m below ground level. This particular finding suggests a transition to mineral soil characteristics, as it reflects not only low water content but also a distinct reduction in organic matter, which is further discussed in the following Section 2.2.4 below.

Several studies have shown that the water content in peat varies significantly with depth, which in turn affects the peat's undrained shear strength (S_u), stiffness, and compressibility.

2.2.4 Organic content

Peat is formed by the accumulation of plants and roots, and organic matter serves as a critical factor in differentiating peat from organoclay or organosilt. Organic content is usually measured by burning all organics in a furnace and measuring the ignition loss (N). Skempton and Petley (1970) suggested baking organics at 550 °C for three hours. However, when the temperature is higher than 450 °C, clay minerals will lose fixed water, which increases the organic content on the surface. Therefore, the organic content (OC) is calculated using the following formula:

$$\mathrm{OC}\ (\%) = 100 - \mathrm{C}(100 - \mathrm{N}),$$

where C is the correction factor for organic carbon loss (usually 1.04; Skempton and Petley, 1970). To overcome this problem, Arman (1971) suggested calculating N for five hours at 440 ºC. The advantage of this method is that no correction is required, and the organic content can be calculated directly from the fire loss (Landva, 1983).

Peat is generally defined as soil with an ash content of less than 25–50%, indicating that the organic content is greater than 50–75%. While these definitions are often derived from soil science perspectives, for geotechnical engineering purposes, Landva (1983) defined peat as soil with an organic content of over 80%. In the Netherlands, the transition from organic soil to peat occurs at a much lower level: between 15–25% organic content (Nen, 1989). For the purposes of this study, peat is defined as soil with more than 75% organic matter content, whereas soil with a lower organic matter content is referred to as 'organic soil'.

Similar to the water content, the organic matter content of peat is also much higher than that of ordinary soil, ranging from 70% to nearly 100%. The shear strength of peat also has a certain degree of influence on its permeability. Muraro. S. (2019) showed that the tiny fibres in peat do not have a great impact on its mechanical properties, the presence of larger fibres (1–3 cm in length) significantly affects its behaviour. These larger fibres tend to realign under load, promoting longitudinal compression and contributing to anisotropic mechanical responses in the peat.

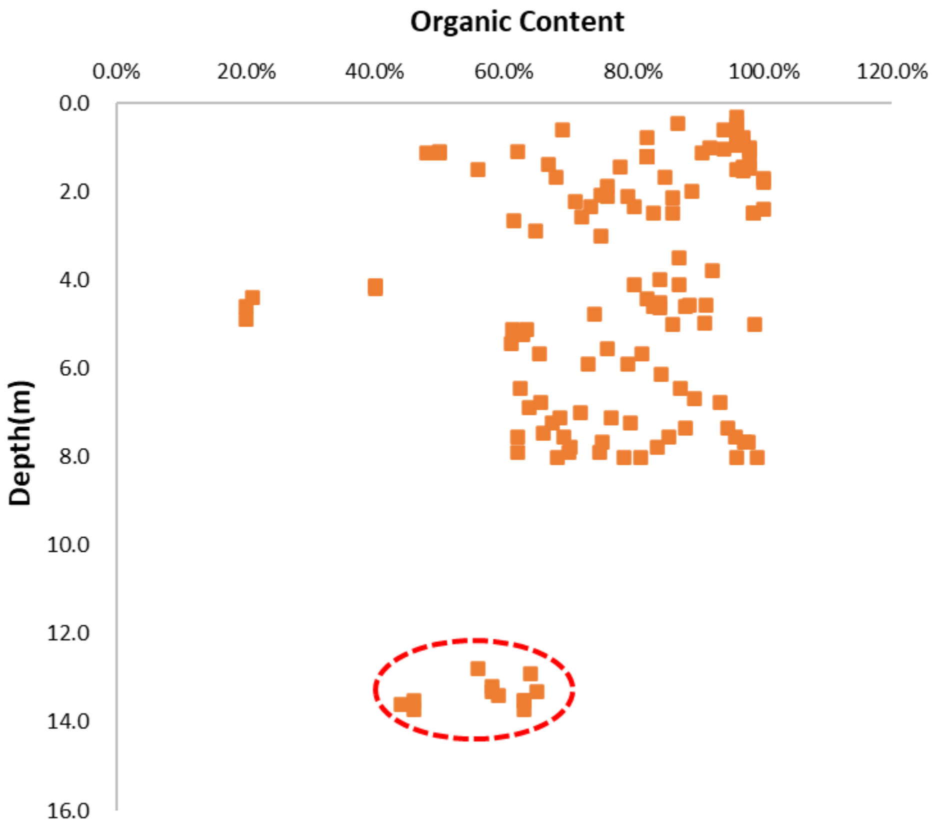


Figure 2-6: Comparison of organic content and sampling depths obtained from previous studies

Since peat is composed primarily of decomposed plant matter, the proportion of organic matter directly influences the density, strength, void ratio, compressibility, and permeability of peat. Several researchers have explored these relationships, and findings have consistently shown that higher organic content leads to softer soil characteristics in peat.

Farrell and Hebib (1998) showed that as the organic content in peat decreases (below 50%), the friction angle begins to converge towards values typical of organic clays or silts (20–30°). Mesri and Ajlouni (2007) found that fibrous peat (with organic contents over 80%) showed a wide range of effective friction angles (ϕ′) ranging from 35° to 60°, but the presence of long fibres (1–3 cm) could enhance apparent cohesion due to fibre interlocking.

The following observations summarise the relationship between organic matter content and the physical properties of peat.

1. **Density and void ratio**: As the organic matter content increases, both the density and soil particle density of peat show a linear decline. Meanwhile, moisture content increases linearly, and the void ratio exhibits an exponential relationship with organic matter content. Zwanenburg and Jardine (2015) reported that peat samples with higher organic contents exhibited exponentially higher void ratios, with a clear correlation between increased organic content and greater porosity.
2. **Compressibility**: Changes in organic matter content significantly impact the engineering properties of peat. Increased organic matter leads to higher compressibility, with maximum compression potentially reaching three-quarters of the specimen thickness. Additionally, the primary consolidation process is greatly influenced by organic content (Mesri and Ajlouni, 2007; O'Kelly and Zhang, 2013).
3. **Cohesion**: The cohesion of peat soil is positively correlated with organic matter content. Cohesion is primarily derived from the colloidal cementation of organic matter and the interaction between clay particles. An increase in organic matter enhances cementation within the soil. Furthermore, the chaotic interweaving of incompletely decomposed plant residues contributes to shear resistance, akin to cohesion.
4. **Permeability**: As the organic matter content increases, the permeability coefficient of peat soil gradually decreases. The directional or semi-directional distribution of undecomposed plant residues results in greater horizontal permeability compared to vertical permeability. Dhowian and Edil (1980) and

Wong et al. (2009) both reported permeability anisotropy in peats, with horizontal permeability exceeding vertical values in fibrous samples.

2.2.5 Degree of decomposition

Decomposition degree refers to the relative content of plant residues that have lost cell structure substances due to decomposition. To a certain extent, it reflects the peatification intensity and affects other physical and mechanical properties of peat soils.

Several methods to determine the degree of decomposition (humification) of peat have been reported in the existing research (Farnham and Finney, 1965; Zaccone, 2018). Among those methods, the most well-known and common in situ test used to classify peat soils is the von Post scale (von Post, 1922), which is based on the colour of manually squeezed peat pore water and the remaining solid material.

Another commonly used method was proposed by the United States Department of Agriculture (USDA). The USDA compresses the von Post scale into three levels and divides peat based on the fibre content into fibric (fibrous), hemic (semi-fibrous), and sapric (amorphous) categories (USDA, 1999). The von Post scale and USDA classifications are detailed in Table 2-1.

- **Fibric peat**: Fibric peat is the least humified (H1–H4 on the von Post scale) type of peat and consists of distinctly undecomposed fragments of plant structure. Fibric peat contains more than 66% fibre (American Society for Testing and Materials (ASTM), 2007) and typically exhibits a brown to brownish yellow colouration (Larsson, 1996).

- **Hemic peat**: Hemic peat displays a medium degree of decomposition (H5–H6), retaining a recognisable structure. The fibre content in semi-fibrous peat ranges from 33% to 66%. This peat is usually brown in colour (Larsson, 1996).
- **Sapric peat**: Sapric peat is highly humified (H7–H10) and is characterised by less than 33% of very indistinct or invisible plant fibres (ASTM, 2007; Huat, 2011; Larsson, 1996). Sapric peat typically appears dark brown to black.

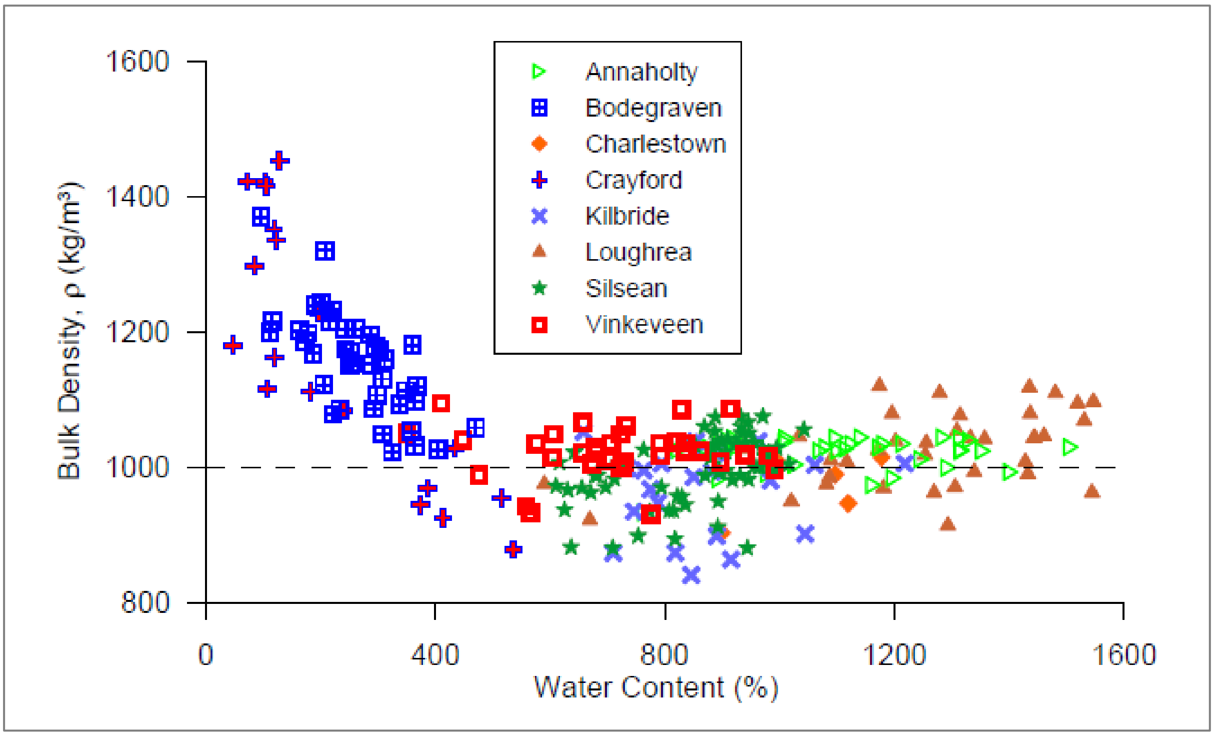
Table 2-1: Systematic outline of von Post classifications

Degree of Humification	Decomposition	USDA classification	Plant structure	Amorphous material content	Colour of released water	Escape of material when squeezing	Nature of residue
H1	None	Fibric peat	Easily identified	None	Clear, colourless water	None	-
H2	Insignificant		Easily identified	None	Clear or yellowish	-	-
H3	Very slight		Still identifiable	-	Muddy brown	-	Not pasty
H4	Slight		Not easily identifiable	Very slight amount	-	None	Somewhat pasty
H5	Moderate	Hemic peat	Recognisable but vague	-	-	Very small amount	-
H6	Moderate strong		Indistinct	-	-	About one-third escapes	-
H7	Strong	Sapric peat	Very faintly recognizable	Lots	Very dark and almost pasty	About one-half escapes	Fibres and roots more resistant to decomposition remain in hand
H8	Very strong		Very indistinct	Large quantity	-	About two-thirds escape	-
H9	Nearly complete		Almost undiscernible	-	-	-	-
H10	Complete		Not discernible	-	-	All the wet peat escapes	-

2.2.6 Bulk density and dry density

The bulk density of peat is usually lower than that of mineral soils due to its high fibre content. Bulk density is often not considered an effective parameter for detecting variations in peat mass.

According to Boylan (2008), when the organic content of peat exceeds 80%, its specific gravity is relatively low (< 1.5), resulting in a bulk density that approaches that of water (approximately 1,000 kg/m³). The relationships between bulk density and water content and dry density and water content are presented below, indicating that for water contents greater than 500% to 600%, there is minimal variation in the bulk density, which remains close to that of water. Similar findings have been reported by Hobbs (1986) for British peats and Den Haan and Kruse (2007) for Dutch peats. In contrast, dry density decreases rapidly with increasing water content.



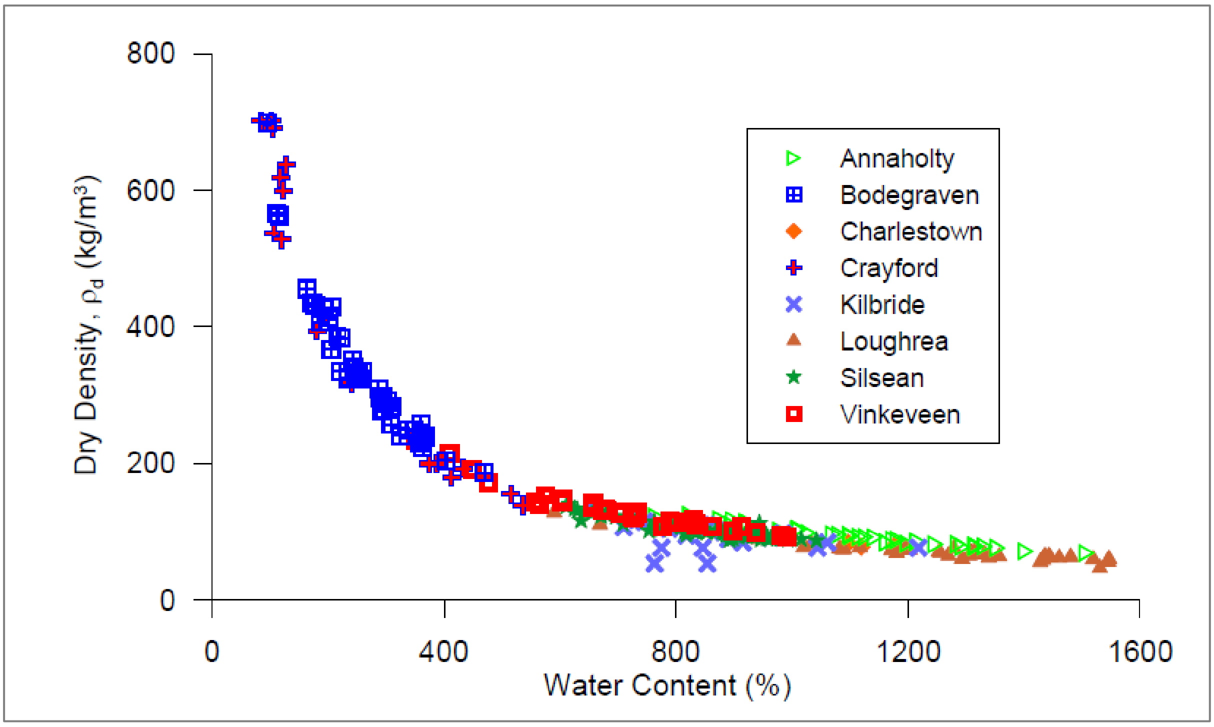


Figure 2-7: Bulk density and dry density versus water content (Boylan, 2008)

Dry density proves to be more useful in reflecting variations in moisture content and the degree of decomposition within peat. Because dry density reflects the structural level within the peat, it serves as a valuable quantity for establishing correlations with compressibility parameters, such as those outlined in the work of Helenelund (1980).

2.2.7 Permeability

The permeability of soils is influenced by several factors, including the void ratio, the size of flow channels perpendicular to the direction of flow, and the shape of flow channels parallel to the direction of flow. Generally, larger void ratios, larger pore sizes, and more linear flow channels result in higher permeability, whereas smaller void ratios, smaller pore sizes, and tortuous flow channels lead to reduced permeability.

Given the anisotropic nature of peat, it is essential to analyse its permeability in both horizontal and vertical directions. Experimental studies have demonstrated that the vertical permeability of peat is approximately twice that of its horizontal permeability.

Dhowian and Edil (1980) reported that permeability anisotropy resulted from fibre orientations, and found horizontal permeabilities about 300 times higher than vertical permeabilities at any given void ratio when using the variable hydraulic head method during consolidated tests. However, much smaller permeability anisotropies have been reported by most other researchers. Mesri (1997) reported anisotropic permeability with the value k_{h0} = k_{vo} = 10 at an in situ void ratio of 12.0 for Middleton peat.

Wong (2009) emphasised that peat's permeability is significantly influenced by its degree of decomposition. This underscores the importance of considering the structural and compositional characteristics of peat when evaluating its hydraulic properties.

2.3 Mechanical properties of peat

2.3.1 Consolidation characteristics

Peat exhibits significant volume changes due to its high water content, which makes it particularly susceptible to consolidation. The voids within peat are predominantly saturated (O'Kelly and Pichan, 2013). Fibric peat, characterised by its high initial void ratio and natural water content, contrasts with sapric and hemic peat in its compressibility. The elevated initial void ratio of fibric peat results in considerably higher compressibility compared to its sapric and hemic counterparts.

The roots and fibres in peat hold water in three distinct forms: intergranular water, intercellular water (held in macropores), and adsorbed or bound water (held in micropores). The distribution of water in these phases determines the way in which pore water is retained and expelled during primary consolidation and secondary compression. The proportion of water stored in each phase depends on the structure and degree of humification of the peat. Volume changes within micropores and macropores dominate the consolidation of peat that is more fibrous and less humic. In highly humified (decomposed) amorphous peat, there are few micropores or macropores, and intergranular water dominates consolidation (Hobbs 1986; Mesri 2007).

The initial phase of peat consolidation involves the drainage of water and the simultaneous rearrangement of peat particles and structures. This continues until the pore pressure dissipates. After the initial consolidation, the volume change continues in a creep-like process as the peat structure continues to rearrange itself and water is almost completely expelled from the micropores of the organic structure. The consolidation of peat does not conform to Terzaghi's theory (1923) of consolidation because water drainage, reorganisation, and a rapid decrease in hydraulic conductivity occur simultaneously during consolidation. Terms used for the consolidation of mineral soils, such as primary consolidation for the initial stage and secondary compression for creep behaviour, should be used with caution in peat. The boundary between these two stages can usually only be distinguished by using pore pressure measurements to determine when the pore water pressure due to applied loads has completely dissipated (Hobbs 1986; Mesri 2007).

Primary consolidation of peat occurs rapidly due to peat's initial high hydraulic conductivity and can be measured over weeks and months. The duration of secondary compression of peat is much longer than that of initial consolidation and is measured in years. Weber (1969) reported significant linear secondary settlement under an embankment constructed over 25 years on peat sediments in San Francisco Bay. The magnitude and rate of secondary compression are high and may eventually account for more than 60% of the total settlement (MacFarlane 1965; Elsayed 2003).

In geotechnical engineering, one-dimensional consolidation tests are predominantly utilised to analyse the consolidation characteristics of peat. The consolidation behaviour of soils is primarily determined through standard oedometer tests, which focus on vertical drainage conditions. In contrast, the evaluation of consolidation properties under horizontal drainage conditions has received limited attention. Consequently, assessments of anisotropy regarding permeability ratios are often based on soil descriptions and engineering judgment. This often results in conservative design predictions for field consolidation rates (Cortellazzo and Simonini, 2001).

To address this issue, studies comparing the responses of duplicate sets of specimens under one-dimensional loading for both vertical and horizontal drainage conditions have been conducted. Mesri and Godlewski (1997) explored the relationship between time, stress, and compressibility concerning effective stress, defining two key indices.

- C_c is the primary consolidation factor, which is calculated by evaluating the change in void ratio from the beginning to the end of consolidation in relation to the applied pressure on a logarithmic scale.
- C_a is the compressibility with respect to time as expressed by the secondary compression index. This represents the difference between the void ratio at the end of the primary and the next log cycle in relation to time.

It was concluded from the research that for any natural soil, a unique relationship exists between the secondary compression index C_a and the primary compression index C_c. In addition, there exists a relationship between C_a and C_c that remains valid over time.

According to Mesri (1997), secondary compression is an important process in organic soils and peat deposits because of the following.

- The lightweight plant materials that constitute peat particles retain substantial amounts of water, leading to the accumulation of peat at high void ratios.
- Peat deposits, characterised by highly deformable organic particles, exhibit the highest values of C_a/C_c. Less deformable particles display low values, typically around C_a/C_c =0.02 ± 0.01 (Mesri 1997).
- Secondary compression is significant in peat because it appears soon after the completion of construction loading.

The concept of compressibility has been successfully applied to interpret laboratory measurements and predict the field secondary compression of peat with and without surcharge. The concept of compressibility states that the secondary compression behaviour at all values of σ_v' in recompression and compression, and throughout the secondary compression stage, is defined by one single value of C_a/C_c together with the end of the primary e versus log σ_v' relationship. The Terzaghi (1996) chart (Table 1) is used to evaluate C_a/C_c ratios.

Due to its low compressibility, the excessive deformation of peat poses a challenge in engineering practice, not only during construction, but also in the long term. Mesri and Ajlouni (2007) reported that the main factors in peat compressibility include the fibre content, natural water content, void ratio, initial permeability, natural arrangement of soil particles, inter-particle bonding, and so on. In addition, the degree of humification of the organic matter in peat soils substantially affects compressibility. Among them, the compressibility of fibric peat is the highest due to the presence of more hollow structures compared to other types of peat. This is followed by hemic and sapric peat (Huat, Prasad, Asadi and Kazemian, 2014).

2.3.2 Anisotropy

Similar to mineral soil, organic peat exhibits anisotropic properties. Landva and LaRochelle (1983) and Farrell and Hebib (1998) conducted triaxial tests and ring shear tests on peat to explore the friction angle (φ') in different directions. Findings indicated that differences in the shear strength of peat are largely attributable to the orientation and composition of peat fibres. Furthermore, O'Kelly and Zhang (2013) conducted a series of consolidated undrained triaxial tests to measure the shear strength of peat. The specimens did not reach a failure state over entire test, and a significant difference in micro-soil structure and fibre content only caused a small difference in shear strength.

The anisotropic behaviour of peat soils is complex and multifaceted. The anisotropic effect can be understood as a comprehensive reflection of the reinforcement effects contributed by the granularity of mineral soil, humic organic colloids, and residual fibres.

For peat soil with a high degree of decomposition, when the organic matter content is low and mineral soil particles dominate, the anisotropy is determined by the directional arrangement of soil particles; when the organic matter content is high, humus plays a dominant role in the anisotropy, and the peat soil tends to be homotropic in various directions. For peat soil with a low degree of decomposition, when the residual fibre content is high, horizontally oriented residual fibres act as reinforcement, and the peat soil shows obvious shear strength anisotropy; however, too much residual fibre content will cause the soil structure to be loose, and the consolidation deformation tends to be isotropic.

Peat soil is a distinct type of soil, exhibiting characteristics that set it apart from typical soft clays. In addition to the common components of soil, such as particles, water, and gas, peat contains a significant proportion of organic matter, which has unique physical, chemical, and mechanical properties. It is insufficient to apply conventional approaches used for inorganic soft soils to peat, as these do not fully capture the complexities introduced by the high organic content. Traditionally, organic matter has often been considered a general solid material within soil mechanics. While this assumption may be valid when the organic content is minimal, as organic particles tend to adhere to mineral soil particles and form a stable soil skeleton, this is not the case for peat.

Peat soils typically contain a high percentage of organic matter, with humus content often exceeding 20–50%. This results in the presence of significant amounts of free humus, which inhibits the formation of a stable mineral particle skeleton. Furthermore, organic matter in peat can deform under load, contrasting sharply with the classical soil mechanics assumption that the solid phase in soil is incompressible. This makes it inappropriate to treat organic matter as a solid phase material. Some researchers have attempted to model peat soil using a four-phase system to better represent its compressibility and deformation characteristics.

In peat with low decomposition, residual plant fibres can provide reinforcement, though contribution is inconsistent due to random orientation, variable length, and relatively weak resistance to pull-out. Thus, assessing the reinforcement effects of these fibres is challenging. Additionally, under certain conditions, these fibres may further decompose into humus or even mineralise, complicating the engineering properties of peat. The current lack of a comprehensive understanding of peat's engineering behaviour is largely due to incomplete knowledge of the role organic matter plays in peat's mechanical performance.

2.3.3 Shear strength

The study of the geotechnical properties of peat began after the Second World War, primarily in response to challenges associated with modern roads and highways built on peat (Landva and La Rochelle, 1983). The earliest publication on peat shear strength seems to be the work of Professor Hanrahan of University College Dublin (Hanrahan, 1952, Hanrahan, 1954). Hanrahan's initial research involved laboratory tests on macerated peat, from which the fibres were removed and the material was reshaped. His conclusions indicated that the shear strength of peat could be considered fully converged ($\phi' = 0$), and the behaviour of the improved material would not accurately reflect that of an undisturbed material due to its structural properties.

However, his later studies (Hanrahan, 1967) came to different conclusions: that the qualitative behaviour of peat in the reshaped and undisturbed states was similar, and the strength of peat is frictional. It is important to realise that this interpretation of peat shear strength is based on the assumption that peat behaves in the same way as other soils and can be described in the same framework. Research on peat has mainly focused on obtaining strength parameters from laboratory and field tests. Few researchers have verified the applicability of these parameters through large-scale field tests. From the beginning, researchers believed that the effective stress principle is the same as the sand and clay principle proposed by Terzaghi (1923), and is also applicable to peat.

The strength and stiffness of peat are functions of the presence of an organic fibre layer and fibre properties controlled by the plant species of origin and the degree of humification. These plant fibres serve to reinforce the structure of the peat.

The measured strength of peat depends on the test method. Triaxial testing of fibrous peat has resulted in effective friction angles (φ') between 48° and 68° (Landva and Rochelle 1983; Farrell and Hebib 1998; Hebib 2001; Long 2005). Direct shear tests and ring shear tests on peat yield lower values, ranging from 20° to 38° (Farrell and Hebib, 1998; Hebib, 2001). The different strength values are attributed to the reinforcing effect of the peat fibres, and the triaxial test provides a measure of strength that includes this reinforcing effect. The results of ring and direct shear tests indicate the friction strength, which indicates the frictional resistance generated between granular and fibre materials (Landva and Rochelle, 1983; Farrell and Hebib, 1998; Hebib, 2001).

The magnitude of the reinforcing effect of peat fibres at yield can be determined by comparing the yield surface defined by a test that includes the reinforcing effect (triaxial test) with the yield surface defined by a test that measures the strength derived from the friction between the particles and the fibres (ring or direct shear test; Landva and Rochelle 1983). The effect of reinforcement is expressed in terms of internal reinforcement stress (σ_{rf}). This acts to reduce the shear stress applied to the peat, thereby increasing the shear stress that the peat can withstand.

The predominantly horizontal orientation of peat fibres also leads to strength anisotropy. In a study presented by Yamaguchi (1985), φ' values of 51° to 55° were measured for vertically oriented samples when the sample axis was perpendicular to the horizontal plane of the fibres. For samples where the sample axis was aligned with the horizontal plane, this angle was reduced to 35°. This enhancement effect is thought to be the result of tension generation in the peat fibres (Den Haan 1997; Yamaguchi 1985). Yamaguchi (1985) showed that strength increased with increasing organic content.

Peat is a low-cohesive, even non-cohesive, soil (Hanrahan, 1952, 1954a and 1954b) due to its fibre content. Although the value of the effective friction angle of peat from triaxial tests has been measured at 40° to 68° (Farrell and Hebib,1998; Mesri and Ajlouni, 2007; Hendry, 2012). Yamaguchi (1985) reported lower friction angles in triaxial extension tests compared to triaxial compression tests and highlighted the difference in friction angles between horizontal (in 35°) and vertical (in 52°) specimens. The friction angles from triaxial compression tests are extremely high, typically between 40° and 60° as reported in the literature, compared to less than 35°, which is typical of clay or silt soil.

Both the volumetric and shear deformation of peat affect its observed strength. The strength of peat depends on the interlocking of its fibres and, therefore, large strains are required to bring out the full strength of the peat (Mesri 2007). Consolidation is a common tool for increasing the strength of peat, as a large reduction in water content usually leads to a significant increase in strength (Den Haan 1997). Alternatively, shearing of peat along the plane of predominant fibre orientation (the horizontal plane) can lead to a reduction in frictional resistance, resulting in post-peak or residual shear strength. The shearing of Middleton peat in a drained direct shear test (along the fibre orientation plane) resulted in a peak effective friction angle of 54° and a peak back angle of 37° (Ajlouni 2000 and Mesri 2007).

One challenging characteristic of peat is its low shear strength in both undrained and drained conditions, which are critical for engineering design and construction on peatland. Munro (2005) and Culloch (2006) reported that the shear strength of peat is affected by its moisture content, rate of decomposition, mineral content, and other physical properties. Peat’s shear strength can be measured by either in situ field tests or laboratory experiments. Landva (1986), however, pointed out that the in situ cone penetration technique is questionable for fibrous peat, whereas the experimental data of a direct simple shear test, on the other hand, tends to estimate lower strengths due to the high fibre content in peat. In addition, the pore-water pressure in a peat specimen can hardly be measured during a direct simple shear test, and the peat failure plane is usually predetermined along the horizontal direction, which cannot represent the peat shear behaviour due to vertical surcharge. As an alternative, triaxial tests may be used to evaluate the shear strength under undrained conditions (Yamaguchi, 1985; Simonetta and Giampaolo, 2005).

In Ireland, O’Kelly and Zhang (2013) carried out a series of consolidated-undrained triaxial tests to measure the shear strength of peat. Results showed that the specimens did not reach failure throughout the tests, and significant differences in the micro soil structure and fibre content only caused small differences in shear resistance. Notably, some past experimental shear test data (O’Kelly, 2015; Boylan, 2008) have indicated that the shear resistance of peat consistently increases with increasing strain, even at large strains up to 30%. Boylan (2008), Garnier (2009), Hendry (2012), Zwanenburg and Jardine (2015), and some other researchers have also conducted consolidated-undrained triaxial tests of peat samples from different countries (e.g., Japan, China, the Netherlands) to investigate shear behaviours. They reported that, regardless of the peat's physical properties, there are no clear peak values throughout the tests for either undisturbed or remoulded peat samples.

In China, peat is mainly distributed in the southwest (e.g., the Kunming province) and northeast regions (e.g., the Liaoning province). Yue (2016), Yan (2008), and Yang

(2011) analysed the characteristics of peat from three different sites in China and conducted direct shear tests to investigate mechanical behaviour. The results showed that peat collected in China usually has relatively lower water contents (150–350%) and organic contents (40–62%) but greater shear resistance than peat collected in Ireland and the Netherlands.

2.3.4 Effective stress

Due to its fibre content, peat is classified as a non-cohesive soil. Although the effective friction angle of peat measured in triaxial tests ranges from 40° to 68° (Farrell and Hebib, 1998; Mesri and Ajlouni, 2007; Hendry, 2012), these values do not accurately reflect the material's true shear strength. Yamaguchi (1985) reported that the friction angle in the triaxial tensile test is lower than that in the triaxial compression test. This article also introduced the difference between the friction angles of horizontal (35°) and vertical (52°) specimens. The friction angle of the triaxial compression test is very high, generally between 40° and 60°, while the friction angle of clay or silt soil is generally below 35°.

Table 2-2 presents data from the literature. According to the effective confining stress proposed by previous research, the slope peat mainly falls in the range of 0–10 kPa, and the peat under a railway or dam mainly falls in the range of 10–50 kPa.

Table 2-2: Shear strengths from previous studies

Peat	Confining stress (kPa)	Effective cohesion, c'(kPa)	Effective friction angle, φ'(°)	Reference
Canadian muskeg	/	0	48	Adams (1965)
Remoulded peat	/	5.5–6.1	36.6–43.5	Hanrahan (1967)

Escuminac peat	/	/	40–50	Landva and LaRochelle (1983)
Ohmiya City peat	/	0	51–55	Yamaguchi (1985)
Reheenmore peat	/	0	55	Farrell and Hebib (1998)
Various peats	/	/	42–66	Edil and Wang (2000)
Middleton peat	/	0	57	Ajlouni (2000)
Correzzola and Adria peat	50	0	49–51	Cola and Cortellazzo (2005)

2.3.5 Stiffness

One significant engineering challenge associated with organic peat is its low stiffness. Research indicates that when the strain level is lower than 10%, peat's stiffness will degrade significantly as the shear strain increases (Zolkefle, 2014; Wehling, 2003a). For fine-grained mineral soils (such as London clay), the hyperbolic function is usually used to estimate the stiffness reduction G/G_{max} (Vardanega and Bolton, 2013), which is largely in agreement with the experimental data. Konstantinou (2017) attempted to apply the same fitting function to both organic soils and peat. However, the reference shear strain γ_{ref} for peat, ranging from 0.8% to 1.65%, was found to be significantly higher than that for clay, which is typically around 0.145%.

The stiffness of peat is quantified using the elastic Young's modulus, which tends to be low and highly variable, with reported values spanning from 400 to 7,000 kPa (Dhowian and Edil, 1978; Elsayed, 2003). The horizontal orientation of fibres in peat contributes to cross-anisotropy in both stiffness and strength (Yamaguchi, 1985).

Elsayed (2003) further noted that the horizontal (cutline) modulus of peat is approximately 50% greater than its vertical modulus.

Similar to shear strength, the stiffness of peat can be increased with a significant decrease in its water content through consolidation (Den Haan 1997). The distribution of shear strength in loose peat strata depends on the presence and condition of fibres, expressed as the degree of humification (Den Haan 1997). This relationship has been demonstrated in leaf shear tests in northern Ontario peat (muskeg strata), in which shear strength typically increased to a depth of about 2 m and then began to decline. This strength distribution correlates with an increase in water content to a depth of about 2.6 m and a subsequent decrease in water content with depth. The higher water content in the upper 2.6 m indicates that the material is less humified; this corresponds to a higher shear strength (MacFarlane and Rutka 1960).

Kishida and Boulanger (2006) conducted a one-dimensional consolidation test indoors, measured the shear wave velocity using reshaped specimens and undisturbed specimens, and studied the influence of organic matter content and stress history on small-strain stiffness. Results showed that the organic content has no significant effect on the stiffness of the specimen, and the maximum stiffness (G_{max}) of the reconstructed specimen under static load conditions was similar to that of the original specimen (Song, 2002). Kishida and Boulanger (2006) showed that the higher the organic matter content, the lower the strength of peat.

Zainorabidin and Mohamad (2015 and 2017) conducted multiple post-cycle triaxial tests to study the behaviour of peat soil at a small strain level of less than 0.1%. They found that when peat is monotonously loaded after cycling, the shear strength and maximum shear stiffness G_{max} under static load conditions are both lower than the initial shear strength and stiffness. G_{max} was almost five times the shear stiffness of G at small strain levels ε = 0.1% (Zainorabidin and Mohamad, 2015 and 2017).

In conclusion, all the evidence suggests that peat fibres contribute significantly to reinforcement within peat. Peat's potential for enhancement, as well as its increase in strength, is related to the prevalence and quality of its fibres. The orientation of peat fibres is predominantly horizontal, which results in peat behaving in an anisotropic manner in terms of strength and stiffness.

2.4 Literature on constitutive frameworks for peat

Modelling the mechanical behaviour of peat presents significant challenges, especially under undrained loading conditions, due to peat's complex and nonlinear stress-strain response, strain rate sensitivity, and long-term creep behaviour. Several constitutive frameworks have been developed or adapted to capture the behaviour of soft soils, and these have had varying degrees of success when applied to peat.

Modified Cam-Clay and extended Cam-Clay models

The Modified Cam-Clay (MCC) model and its variants have been widely applied in the geotechnical modelling of soft clays. These models are based on critical state soil mechanics and assume elasto-plastic behaviour with a unique yield surface and well-defined failure criteria. While the MCC has been employed in some peat studies, it does not capture the unique structure of fibrous peat, nor does it capture the anisotropic or strain-softening behaviour frequently observed in triaxial tests. As a result, its applicability to peat is limited, particularly regarding post-peak response and structure degradation (Mesri and Ajlouni, 2007).

Bounding surface plasticity models

Bounding surface plasticity frameworks, such as those proposed by Dafalias and Popov (1975) and later extended by Gens and Nova (1993), are capable of modelling cyclic behaviour and progressive strain accumulation. These models have been applied to soft clays and organic soils, but require complex calibration and validation. The using for peat remains limited in the literature due to the scarcity of the robust experimental data required to define the bounding surface and hardening rules for fibrous organic soils.

Viscoplastic and creep M = models

Several viscoplastic models have been proposed to account for the time-dependent behaviour of peat and other soft soils, including frameworks based on Perzyna-type overstress theory or rate-dependent plasticity (e.g., Yin and Graham, 1989). These models are useful for capturing long-term settlement and secondary compression in peatlands. However, they often require long-term laboratory or field data, which are not always feasible to obtain. As such, the implementation of these models for routine engineering applications remains limited.

Most modelling effort in the past has been directed towards time-dependent compression behaviour, which has received a lot of attention due to the engineering consequences of unusually high compressibility (Berry and Poskitt, 1972; Edil and Mochtar, 1984; Fox, 1992; Den Haan and Edil, 1994; Den Haan, 1996; Mesri, 1997; Den Haan and Kruse, 2007; Mesri and Ajlouni, 2007; Zhang and O'Kelly, 2013; Madaschi and Gajo, 2015, among others). However, only a few attempts have included wider perspectives on the stress-strain response over general stress paths (e.g., Yamaguchi, 1985; Yang, 2016; Muraro, 2018.).

Despite these developments mentioned above, most constitutive models fail to fully account for the complex stress-strain behaviour of fibrous peat, especially under undrained conditions. The current research aims to bridge this gap by providing new experimental data and proposing a simplified, yet effective, stiffness degradation model based on a normalised shear modulus.

2.5 Summary

Peat is a highly organic soil that forms through the slow accumulation and partial decomposition of plant materials in waterlogged, acidic, and oxygen-deficient environments. It is commonly found in northern latitudes, particularly in areas such as Ireland, Canada, and Northern Europe. Globally, peatlands cover more than 400 million hectares, and in Ireland, they account for roughly 17% of the total land area.

Due to its unique formation and composition, peat exhibits several unique geotechnical properties. It typically contains a very high water content – sometimes exceeding 1,200% – along with low bulk and dry densities, large void ratios, and relatively low shear strength. These characteristics make peat highly compressible and present serious challenges for civil engineering, especially the design and construction of infrastructure such as roads, embankments, railways, and wind farms.

The mechanical behaviour of peat is greatly influenced by its organic content, its degree of decomposition, and the presence of fibrous material. Over time, under its own weight, peat experiences vertical compression, which tends to align plant fibres horizontally. These fibres act as a form of internal reinforcement and contribute to anisotropic behaviour in terms of both strength and stiffness. Laboratory tests have shown that the effective friction angle of peat can vary significantly, typically between 35° and 68°, depending on the fibre content, degree of decomposition, and sample orientation.

With increasing soil depth, peat generally becomes more decomposed and contains less water and organic matter. As a result, deeper layers of peat are usually denser and exhibit slightly improved strength and stiffness characteristics. Peat is also known for its time-dependent behaviour, particularly its secondary compression (creep), which can continue for many years after initial loading and often accounts for more than half of the total settlement. Traditional consolidation theory, such as Terzaghi's model, does not fully explain peat's behaviour because of the simultaneous effects of structure degradation and changes in permeability during compression.

Peat stiffness tends to degrade rapidly with increasing strain. In contrast to mineral soils like clay, which often follow predictable stiffness degradation trends, the behaviour of peat is more complex due to its variable fibrous structure. The reduction in stiffness at small to medium strains is highly dependent on both the fibre network and the strain level applied.

Over the past few decades, various constitutive models have been developed or adapted to simulate peat behaviour. Modified Cam-Clay models have been applied to peat but often fall short in capturing key features such as anisotropy and post-peak softening. More advanced frameworks, such as bounding surface plasticity models, can simulate cyclic loading and accumulated strains but are data-intensive and have seen limited application to peat. Viscoplastic models, which consider time-dependent deformation, offer another approach but require long-term testing data that is not always available.

Thus, despite these developments, the complex mechanical behaviour of fibrous peat under undrained conditions is still not fully understood or adequately captured by existing models. The aim of this research is to address this gap by presenting a simplified yet reliable stiffness degradation model using a normalised shear modulus that has been validated against experimental data obtained from undisturbed peat samples in Ireland.

From an engineering point of view, the most pressing concerns with peat are related to its settlement and stability. Peat undertakes both immediate and long-term settlement, with differential settlement posing a major risk to the integrity of structures. The low undrained shear strength of peat also increases the risk of slope failure and bearing capacity problems, especially in areas with embankments or heavy surface loading.

In addition to these mechanical challenges, peat degradation over time due to drainage or environmental changes (e.g., drying) can further reduce its strength and stiffness. Furthermore, peatlands serve important ecological and hydrological functions. Any disturbance, such as excavation, dewatering, or loading, may have broader environmental impacts, including carbon release and changes to groundwater regimes.

Given these risks, thorough site investigations and geotechnical characterisation are essential before beginning any construction on peat. Appropriate ground improvement techniques – such as preloading, vertical drains, the use of lightweight fill, or deep soil mixing – can be considered to enhance stability.

In conclusion, understanding the mechanical behaviour of peat is important for developing safe and sustainable infrastructure, particularly in Ireland. This study contributes to this understanding by providing new insights into the stiffness and shear strength of fibrous peat and proposing a practical model to support future design and numerical modelling efforts in peatland engineering.

3 SAMPLING LOCATION AND SPECIMEN PREPARATION

3.1 Abstract

Although there have been some specific investigations on peat shear behaviour in the past, the experimental data and associated findings so far, compared to fine-grained mineral soils, are still insufficient for satisfactory engineering reference; in particular, there is a lack of knowledge regarding how to determine peat shear strength and stiffness degradation under increasing strain. This study investigated the peat profile near a wind farm site in Southern Ireland using GPR and the vertical shear wave profiling (VSWP) method. On two wind farm sites (one in southern Ireland, one in Northern Ireland), four blocks of peat were carefully excavated to minimise the peat disturbance and subjected to unconsolidated undrained triaxial tests to investigate shear behaviour.

3.2 Field investigations

The field work involved the collection of GPR profiles to determine the peat thickness, distribution, and internal layering, as well as VSWP to determine the shear wave velocity (V_S) distribution. Detailed von Post logging and water content testing were also conducted at the sample locations.

3.2.1 Sampling locations

Nearly one-sixth of Ireland is covered by peatland. In this study, peat samples were collected at two wind farms in different locations: one near Millstreet, County Cork, in southern Ireland and another one near Omagh in Northern Ireland. These locations are shown in Figure 3-1 below.

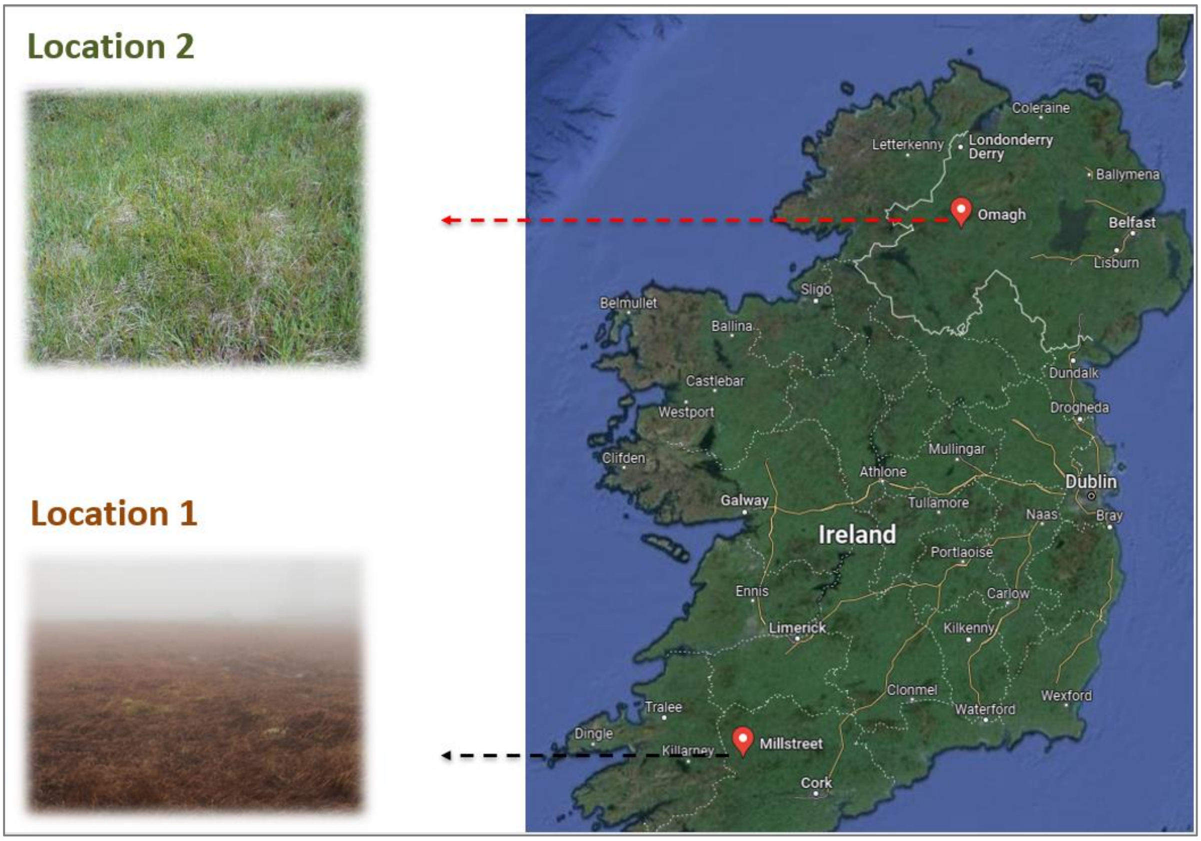


Figure 3-1: Sampling locations

Location 1 was in the Curragh wind farm. This wind farm is located in Ireland, approximately 43.8 km northwest of Cork city. The Curragh wind farm is relatively flat, and the samples were taken 3.5 m below ground level in a relatively deep upland blanket bog with a high cover of Sphagnum moss. The phreatic groundwater level for Location 1 was around 0.3 m below the ground surface.

Location 2 was in the Omagh wind farm. This wind farm is located in Northern Ireland, approximately 2.0 km northwest of Omagh. The Omagh wind farm is also flat, and the samples were taken 3.3 m below ground level in a relatively deep upland blanket bog, which also had a high cover of Sphagnum moss. The phreatic groundwater level for Location 2 was around 0.2 m below the ground surface.

The vegetation in both sampling locations was generally dominated by *Tricophorum cespitosum*, *Calluna vulgaris*, *Racomitrium lanuginosum*, and *Sphagnum capillifolium* (Conaghan, 2009). Plants such as mosses can retain 16–26 times as much water as dry weight while alive. The mosses and other plants in the habitats accumulate and degrade slowly in an anoxic environment and experience diagenic changes with depth.

A half-cylinder Russian corer was used to sample peat profiles, as shown in Figure 3-2. When the sampler reached the target depth, the 'T' handle was turned 180° to fill the half cylinder with peat and then closed again to collect a sample every 0.5 m along the depth. Maximum peat depths of 3.5 m and 3.3 m were reached at locations 1 and 2, respectively, under which a silty sand layer was discovered. The collected field samples were used for von Post (1926) Humification Scale classification, and the water and organic contents were measured in the laboratory.

Figure 3-2: Field peat logging and classification using a Russian corer

At Location 1, after peat profile characterisation along the depth range, a hand-dug pit was excavated, and a block of peat located at a depth of 0.6 m below the ground surface was carefully cut and removed from the trial pit, causing minimum disturbance. The size of this undisturbed block was 47 cm in length, 36 cm in width, and 27 cm in height, as shown in Figure 3-3. At Location 2, large peat blocks were dug out at a depth of 0.35 m to 1.60 m below ground level using an excavator, then carefully cut into two smaller blocks using a knife. The size of each block used for the laboratory tests was about 50 cm in length, 40 cm in width, and 25 cm in height.

Figure 3-3: Undisturbed peat block sample excavated from a depth of 0.6–0.87 m below ground level in Location 1

Besides soil sampling, geophysical methods were also employed to determine the peat profile in Location 1, including both electromagnetic wave techniques and mechanical wave measurements: GPR was used to determine the peat thickness, distribution, and internal layering, whilst VSWP was conducted to determine the in situ shear wave velocity (V_s) profile and shear modulus along the depth.

3.2.2 Ground-penetrating radar

GPR has become a well-established technique for use in detailed peat site investigation, enabling the mapping of continuous peat profiles along a depth range based upon high-frequency electromagnetic wave measurements (Venkateswarlu and Tewari, 2014). Using GPR, it is possible to generate continuous cross sections of the peat thickness and determine the internal structure (layering) of the peat according to the different properties encountered, as reported by Saarenketo (1992). In Ireland, GPR was adopted by Trafford and Long (2016) to profile 3D site images for landslide risk assessment.

In this site investigation, GPR was used to produce a continuous profile by transmitting high-frequency waves down through the ground. Thus, it was possible to map the peat thickness and sub-peat topography across the site. Figure 3-4 shows the topographically corrected GPR profile for a cross section of 220 m in length and 8 m in depth. In this figure, GPR detected the presence of a gravel road and reinforced concrete pad foundation. The block of undisturbed peat sample was taken near the line 'VSWP2', 60 m away from the gravel road. As identified in the GPR data, the base of the peat was approximately 3.5 m deep. This was consistent with the physical sampling of the peat described above.

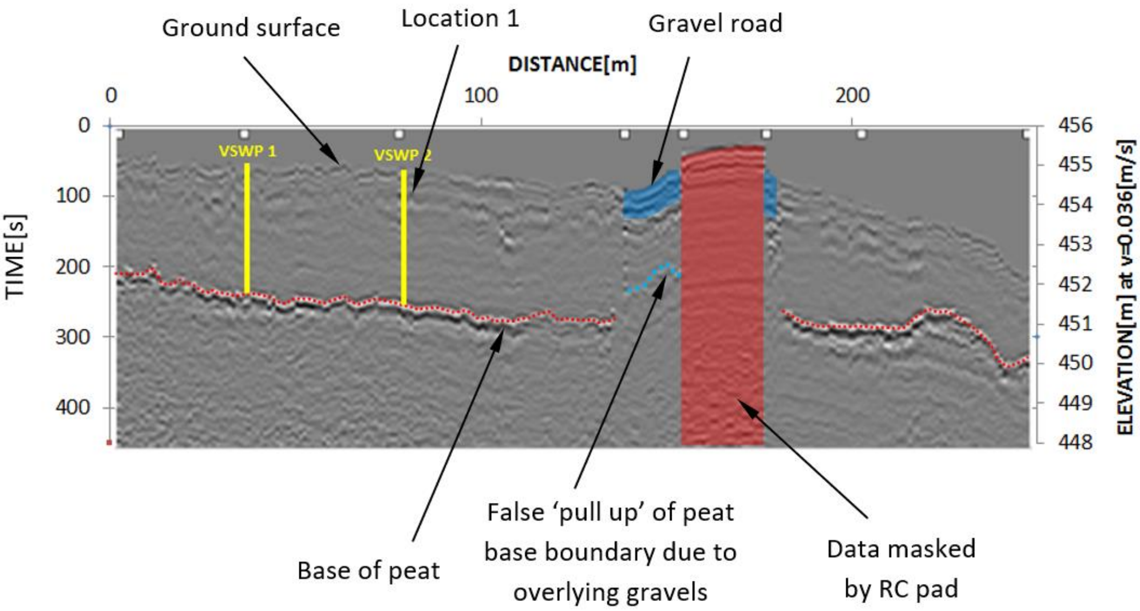
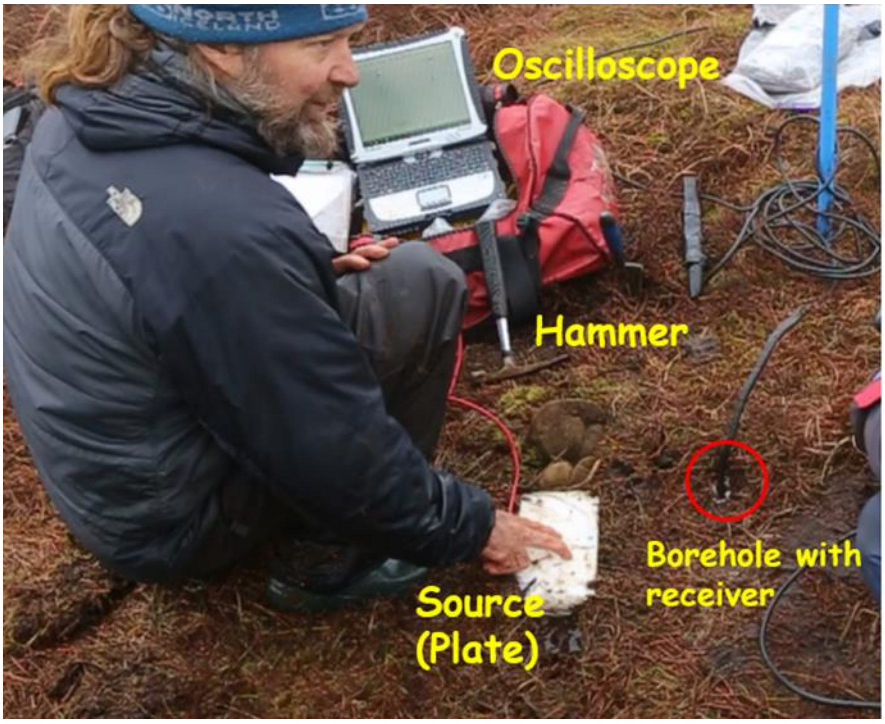


Figure 3-4: Ground-penetrating radar profile at Location 1

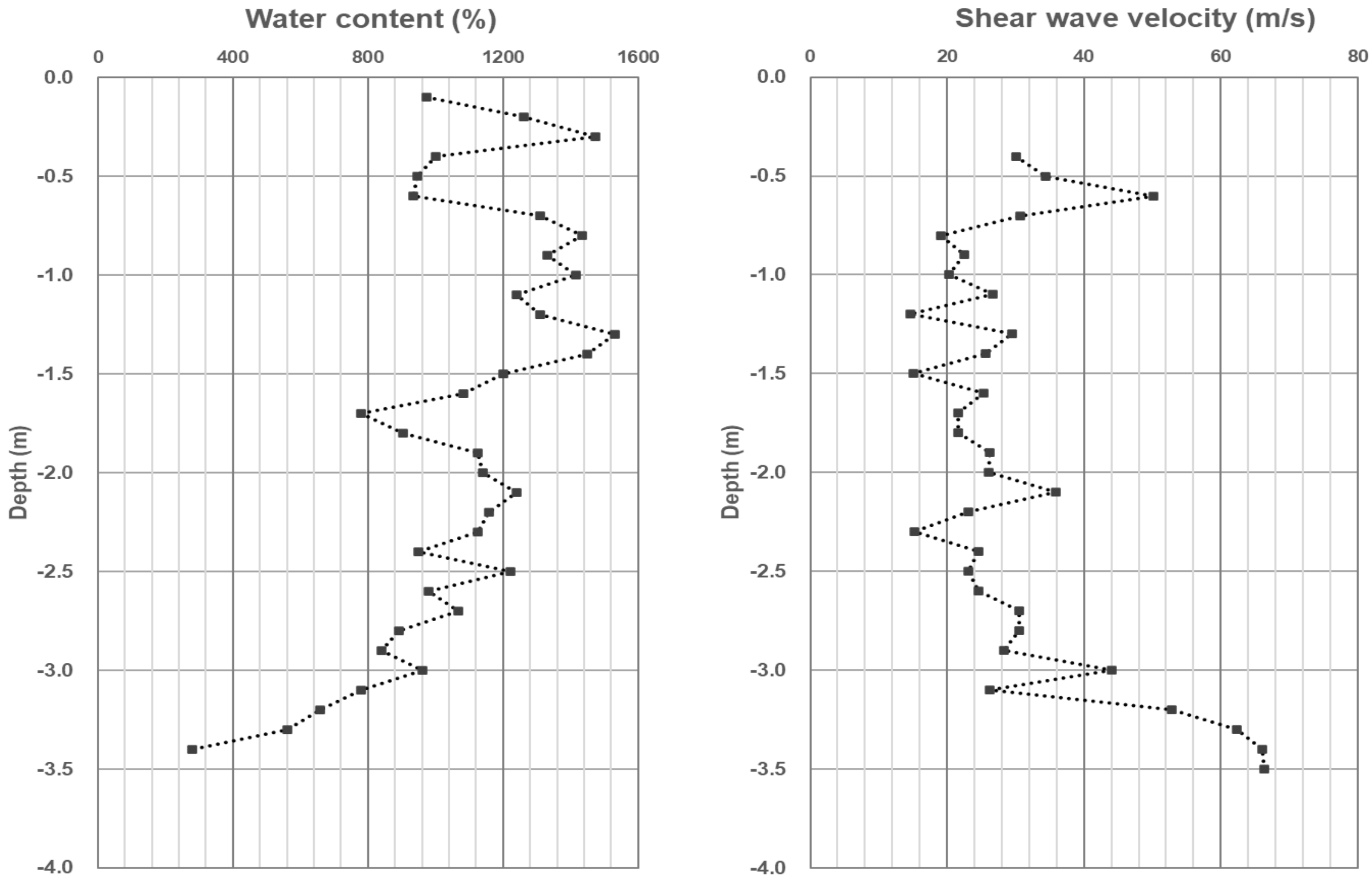
3.2.3 Vertical shear wave profiling

VSWP generates continuous shear wave data throughout the full peat column. By collecting samples for water content analysis at the VSWP locations, it is possible to estimate the undrained shear strength of the peat from the interrelationship between shear wave velocity (V_s), water content (w), and the undrained shear strength (S_u) of the peat.

Figure 3-5a shows the on-site downhole shear wave test. In this test, a borehole was drilled using a Russian corer and velocity transducers (geophone receivers) were placed to detect the vertical shear wave generated by a hammer hitting a wave source plate.



(a) On-site downhole shear wave test



(b) Laboratory-derived water content (c) Shear wave velocity variation along the peat depth

Figure 3-5: Shear wave velocity profiles along the peat depth in Location 1

Figure 3-5c shows the variation of the shear wave velocity measurement V_s from 10 m/s to 67 m/s along the peat depth. In general, the higher the water content of peat, the lower the shear wave velocity is likely to be, as water in peat has little shear resistance to shear wave propagation. The shear wave velocity measurement V_s can then be used together with water content w to derive the undrained peat shear strength S_u along the depth gradient as shown in Figure 3-6, based upon the equation below by Trafford and Long (2016):

$$S_u = C(V_s\ /w)^n, \qquad \textit{Equation 3-1}$$

where C and n are constants (55.8 and 0.683, respectively, for saturated fibrous peat).

The undrained peat shear strength ranges from 2.5 kPa to 8 kPa from the ground surface to 3.1 m below ground, and increases rapidly to 20 kPa at a depth of 3.3 m. At the end of the downhole, the soil was not considered peat.

Meanwhile, the maximum shear modulus of peat G_{max} can be derived based on the measured shear wave velocity V_s and the peat density ρ using the equation from Aki and Richards (2009) below.

$$G_{max} = \rho * {V_s}^2 \qquad \textit{Equation 3-2}$$

According to Equation 3-2, G_{max} = 4,049.37 kPa when the depth is 0.6 m, which is where the peat block samples were excavated for subsequent laboratory triaxial tests.

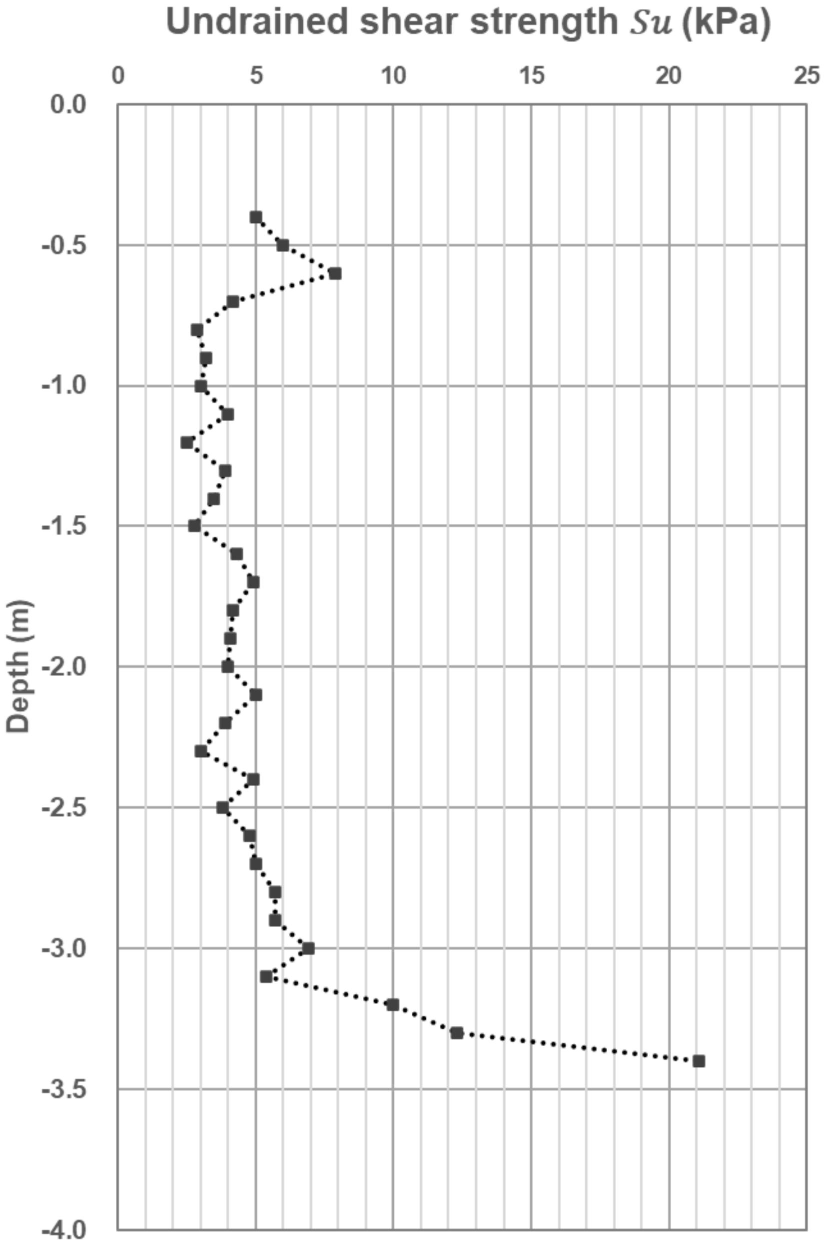


Figure 3-6: The undrained shear strength (S_u) estimated based on the shear wave velocity

3.3 Physical properties of peat

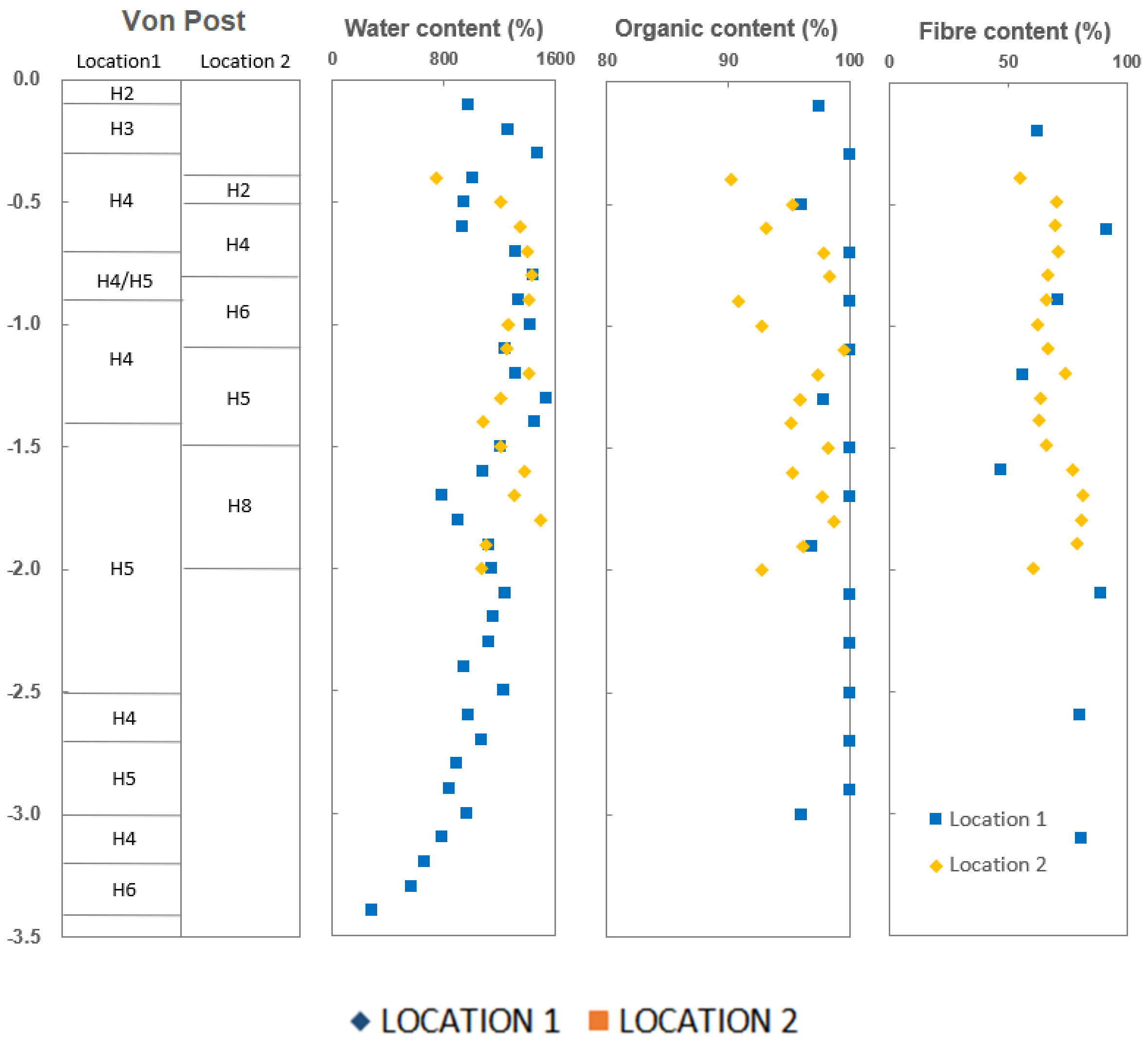
After conducting site investigations in the two locations, the disturbed peat samples collected by the Russian corer were used to determine physical properties, including water content, organic content, and fibre content, along the depth gradients.

Meanwhile, the undisturbed peat blocks were cut and trimmed to the designated sizes for unconsolidated undrained and consolidated undrained triaxial tests.

Figure 3-7a shows the physical properties (water content, organic content, and fibre content) and von Post scale of peat along the depth profiles from the two locations. In these tests, the water content of the peat was measured every 0.1 m along the depth gradients using the oven-drying method at a temperature of 105 ℃ (BS1377 2, 1990) for over 24 hours (O'Kelly 2004). The loss on ignition in the peat samples was determined every 0.5 m along the depth using a muffle furnace at 440 ℃ for five hours, following BS1377 2 (1990) and BS1377 3 (1990), to measure the organic content. The peat fibre content was determined every 0.1 m along the depth gradients according to the ASTM D1997-91 (1997) procedures. Each peat sample was first immersed in 5% sodium hexametaphosphate for over 15 hours and then placed in a 150 μm sieve tray. The peat samples in the sieve trays were washed thoroughly with water, and the fibrous material left on the sieves was oven-dried until the weight did not reduce any more (after five hours).

The fibre orientation results are shown in Figure 3-7b(i), which shows that the fibres in peat are largely horizontally aligned and include few roots. The fibre length in the peat samples ranged from 45 mm to 115 mm and was in the same order of magnitude as the specimen dimensions of 50 mm to 100 mm in Figure 3-7b(ii).

The specimens of 100 mm could contain most of the fibres, and the fibres at the edge of the specimens were cut to create minimal disturbance. Regardless of the fibre length, the fibre density within each specimen can still represent in situ peat conditions at a large scale. In terms of fibre strength under large displacement but not strain, it is difficult to conduct peat laboratory triaxial or shear box tests subject to large displacements.



(a) Physical properties of the peat soil in Locations 1 and 2

(b.i) Fibre orientation

(b.ii) Fibre length

(b) Fibres in the peat soil

Figure 3-7: Physical properties and fibres in the peat soil

In Location 1, the top layer of peat within 0.3 m of the ground surface was almost entirely undecomposed peat (H2) or very slightly decomposed peat (H3), according to the classification system of von Post. Below 0.3 m, the von Post classifications of the peat were H5 (moderately decomposed peat), H4 (slightly decomposed peat), and H6 (moderately highly decomposed peat with a very indistinct plan structure) until 3.2 m, where the mineral silty sand layer was reached.

The water content of the peat near the ground surface was around 900–1,500% – significantly greater than the peat water content of 800% at 0.5 m. Since the peat sample in Location 1 was collected on a rainy day, the water content of the top layer of peat was probably affected by the wet weather conditions. The water content then increased with depth down to the phreatic groundwater level, below which the water content varied between 800% and 1,520% until a depth of 3 m, after which the water content gradually decreased from 1,000% to 25% approaching the silty sand layer at

3.5 m. In this sample, the organic content held fairly constant at approximately 96% along the depth gradient, whereas the fibre content varied between 47% and 90%. Liquid limit tests were also conducted in Location 1 following the procedure described in BS1377-2 (1990), and the test data are displayed in Figure 3-8 below. The results indicate that the water content from the liquid limit test is around 896%, which is less than the 933% in situ water content. That is, in situ peat with a high water content is more likely to be in a liquid state than a plastic state.

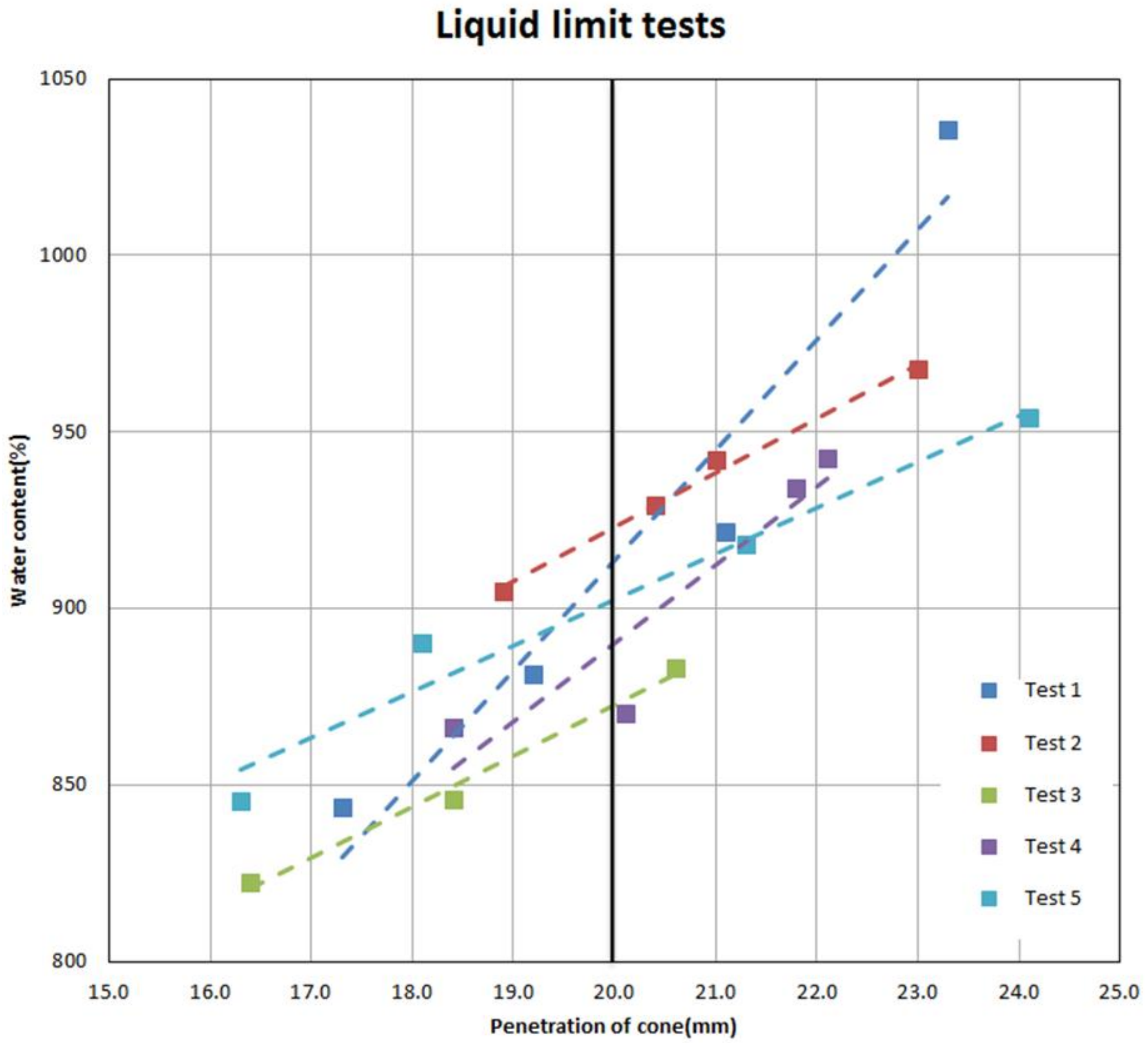


Figure 3-8: Liquid limit tests of the peat soil in Location 1

For Location 2, the peat was investigated using an excavator. Only one location was vertically sampled, and three large blocks were extracted: from 0.35 m to 0.65 m, from

1 m to 1.3 m, and from 1.6 m to 1.85 m. On the von Post scale, the three blocks were, respectively, hardly decomposed (H2), decomposed (H6), and strongly decomposed (H8). At the time of sampling, the phreatic groundwater level was measured at 0.15 m below the surface. The soil layer that was 0 to 0.35 m below ground level was removed because it was too disturbed by plant roots. The field investigation revealed that the bottom of the peat layer was around 3 m under the surface. The von Post H scales generally increased with greater depth. The physical properties along the depth gradient at Location 2 were determined using samples collected from 0.4 m to 2 m below ground level and are marked by the solid orange squares in Figure 3-7a. Between 0.4 m and 0.7 m, the water content increased from over 800% to 1,600%. From 0.7 m to 1.8 m, the water content fluctuated around 1,400%, and then decreased to 800% below 1.9 m. Along the depth gradient, the organic content remained high at around 90–97%, while the fibre content showed a slight fluctuation with an average magnitude of 68%.

4 UNCONSOLIDATED UNDRAINED TRIAXIAL TEST

4.1 Abstract

This chapter investigates the undrained shear behaviour and stiffness degradation characteristics of natural peat through a series of unconsolidated undrained triaxial tests. A total of 28 undisturbed specimens were prepared from the collected block samples. The specimens encompassed both vertical and horizontal orientations to evaluate anisotropy, and two different specimen sizes (50 mm and 100 mm diameter) were used to assess scale effects. The results showed that deviator stress increased with shear strain up to 25% without a clear peak strength, and that higher water content significantly reduced shear strength. Deformation modes were classified into three groups: bulging, cracked, and upright, depending on specimen size and structure. The stiffness of peat was found to degrade considerably with increasing strain. A predictive model was proposed to estimate shear modulus degradation based on a normalised modulus at 0.1% strain. The model achieved over 95% accuracy within a ±30% margin for strains up to 2.5%. A comparison with 82 additional peat test datasets from the literature further validated the proposed model. These findings provide practical insight for geotechnical engineering applications involving peat and contribute to advancing constitutive modelling of fibrous organic soils.

4.2 Test procedure

Past experimental investigations of peat have included the consolidated drained test, the consolidated undrained test, the unconsolidated undrained test (Huat, 2011). Since peat in Ireland is very soft with a high compressibility, a chamber/cell consolidation pressure greater than 10 kPa is likely to generate significant deformation and distortion of soil specimens and, therefore, create some inconvenience in the subsequent axial loading step. On the other hand, it is difficult for an advanced pressure/volume controller (for example, controllers from Global Digital Systems [GDS] Ltd.) to apply precise chamber stress lower than 10 kPa for consolidation, as the standard resolution is limited to a minimum of 1 kPa. A slight pressure inaccuracy may lead to significant deviation in consolidation effective stress and, in turn, jeopardise the credibility of the experimental data. In the light of these experimental difficulties, an unconsolidated undrained test was conducted in this study to reveal the undrained shear behaviour of peat at low stress levels in situ. This may provide some guidance on peat deformation and landslides due to cutting and construction.

4.2.1 Specimen preparation for triaxial testing

After excavation from the ground, the peat blocks were immediately packaged using plastic film to maintain the moisture content. Following extraction, the blocks were carefully placed into a suitcase to facilitate secure transportation. And the blocks were packed with soft plastic materials to minimise disturbances during the transportation to the laboratory. Upon delivery to the soil laboratory, the block samples were stored in a fridge at a constant temperature of 15 ℃ following Zhang (2017)'s procedure.

Later, peat specimens were cut from the block samples in the designated diameters for triaxial tests and in both vertical and horizontal directions to factor in mechanical anisotropy behaviour. When trimming the specimens, a pair of scissors was used to cut the fibres in the soil rather than a knife in order to minimise soil disturbance. In addition, two sets of specimens of different sizes were compared in the triaxial tests to consider scale effects: (a) small specimens of 50 mm in diameter x 85 mm long and (b) large specimens of 100 mm in diameter x 170 mm long. A height-to-diameter ratio of 1.7 was used for triaxial specimen dimensions, as suggested by Berre (1982) and Boylan (2008). For the Location 1 sample, 12 specimens were prepared from the block sample taken from a depth of 0.6 m: six of the specimens were small, and the other half were large ones. For each set of six specimens (small and large), three were vertically cut and three were horizontally cut.

The specimens from Location 2 were obtained at three different depths (i.e., 0.35 m, 1 m, and 1.65 m). For each depth, eight specimens were prepared in different sizes and cut directions. The specimens were trimmed using a soil lathe and a sharp blade, and then labelled according to size, direction, and test sequence. For example, '0.6m-50V1' represents the specimen for the first test measuring 50 mm in diameter and vertically cut from 0.6 m below the ground surface. In total, 28 peat specimens were prepared for the unconsolidated undrained tests in this study. These specimens are summarised in Table 4-1 below.

4.3 Triaxial testing

A series of unconsolidated undrained triaxial tests was performed on the peat specimens using a GDS triaxial tester. The accuracies of the load cell and pressure controller were both less than 0.1%, as provided by GDS Instruments Ltd. Figure 4-1 shows the specimen setup for the triaxial tests. At the base, each specimen was placed above five layers consisting of, from top to bottom, an impermeable membrane disc, an impermeable plastic disc, a filter paper disc, porous stone, and a base pedestal. In particular, the surface of the plastic disc below the membrane disc was lubricated to minimise the end restraint, following Boylan (2008)'s suggestion. A filter paper disc, a porous stone, and a top cap were placed on top of the specimens.

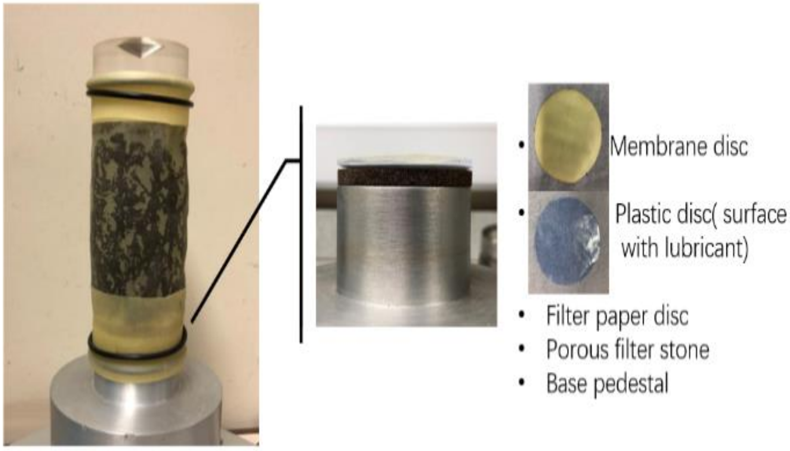


Figure 4-1: Specimen setup to minimise end restraint at the base pedestal

In each unconsolidated undrained triaxial test, a constant cell pressure of 3–4 kPa was first applied to the specimen to mimic the water pressure in the initial natural ground condition.

After one hour, the specimen was stabilised. It was then loaded at a rate of 20% axial strain per day until 25% of axial strain was reached. Considering that peat is extremely soft, its deformation is significant, and thus, the loading rate may affect the results.

In this study, a strain rate of 20% axial strain per day (approximately 0.014% per minute) was adopted during the triaxial testing of peat specimens. Axel strain was applied to a maximum of 25%. This slow loading rate was selected to allow adequate time for the specimens to respond to applied loads, enabling the development of internal pore pressure and strain without inducing rapid failure. Such a strain rate is essential for capturing the mechanical behaviour of peat, including creep, delayed consolidation, and structural rearrangement, which are commonly observed in organic soils.

Yamaguchi (1987) confirmed that the strain rate significantly influences the undrained behaviour of peat, and care must be taken in selecting strain rates during laboratory testing to ensure a realistic representation of in situ conditions. Yamaguchi supported the adoption of slow strain rates in triaxial tests for peat to observe more representative mechanical behaviour. Mesri and Ajlouni (2007) also discussed the significance of applying slow strain rates in organic soils to better assess secondary compression and prevent overestimation of strength.

4.4 Experimental data analysis

In an unconsolidated undrained test, the soil volume remains unchanged while the axial load and vertical displacement are recorded to analyse the development of deviatoric stress with increasing shear strain.

In the test, the deviator stress $(\sigma_1 - \sigma_3)$ in kPa was derived following BS1377-8 (1990):

$$\sigma_1 - \sigma_3 = (\sigma_1 - \sigma_3)_m - \Delta(\sigma_1 - \sigma_3)_{mb}, \qquad \textit{Equation 4-1}$$

where

- $(\sigma_1 - \sigma_3)$ is the deviator stress in kPa,
- $(\sigma_1 - \sigma_3)_m$ is the axial stress (in kPa) applied by the load cell, and
- $\Delta(\sigma_1 - \sigma_3)_{mb}$ is the deviator stress (in kPa) caused by membrane stiffness.

In terms of the membrane stiffness, the equation below from ASTM D 2850-95 (1999) was adopted:

$$\Delta(\sigma_1 - \sigma_3)_{mb} = \frac{4E_m t_m \Upsilon}{D_c}, \qquad \textit{Equation 4-2}$$

where

- Υ is triaxial shear strain increment (in %);
- D_c is the diameter of the specimen (in mm);
- t_m is the total thickness of the membrane enclosing the specimen, which is usually 0.2 mm; and
- E_m is the Young's modulus for the membrane material. For a typical latex membrane, E_m is 1,400 kPa.

The triaxial shear strain increment (γ), following Wood (1990), was calculated as

$$\gamma = \frac{2(\Delta\varepsilon_a - \Delta\varepsilon_r)}{3}, \qquad \textit{Equation 4-3}$$

where in the undrained test (Powrie, 1997), $\varepsilon_{vol} = \varepsilon_a + 2\varepsilon_r = 0$, and hence, $\varepsilon_r = -\frac{1}{2}\varepsilon_a$. Thus, the shear strain γ is equal to ε_a.

The membrane stiffness increases with the developing shear strain and has a significant effect on the peat specimen stiffness after 10% shear strain.

4.4.1 Stress-strain relationship

Figure 4-2 shows the stress-strain relationship of peat shear behaviour for all the unconsolidated undrained tests conducted in this study. For almost all the specimens, the deviator stress begins to increase substantially with the shear strain at a small strain, typically up to approximately 10%. At larger strain levels, the stress continues to increase slightly and ends with no obvious peak value even at up to 25% shear strain. The unique exception is the curve for specimen 1m-50H2, which continued to increase substantially even at a large strain. This could have been due to test error or to undecomposed wood pieces remaining in the specimen.

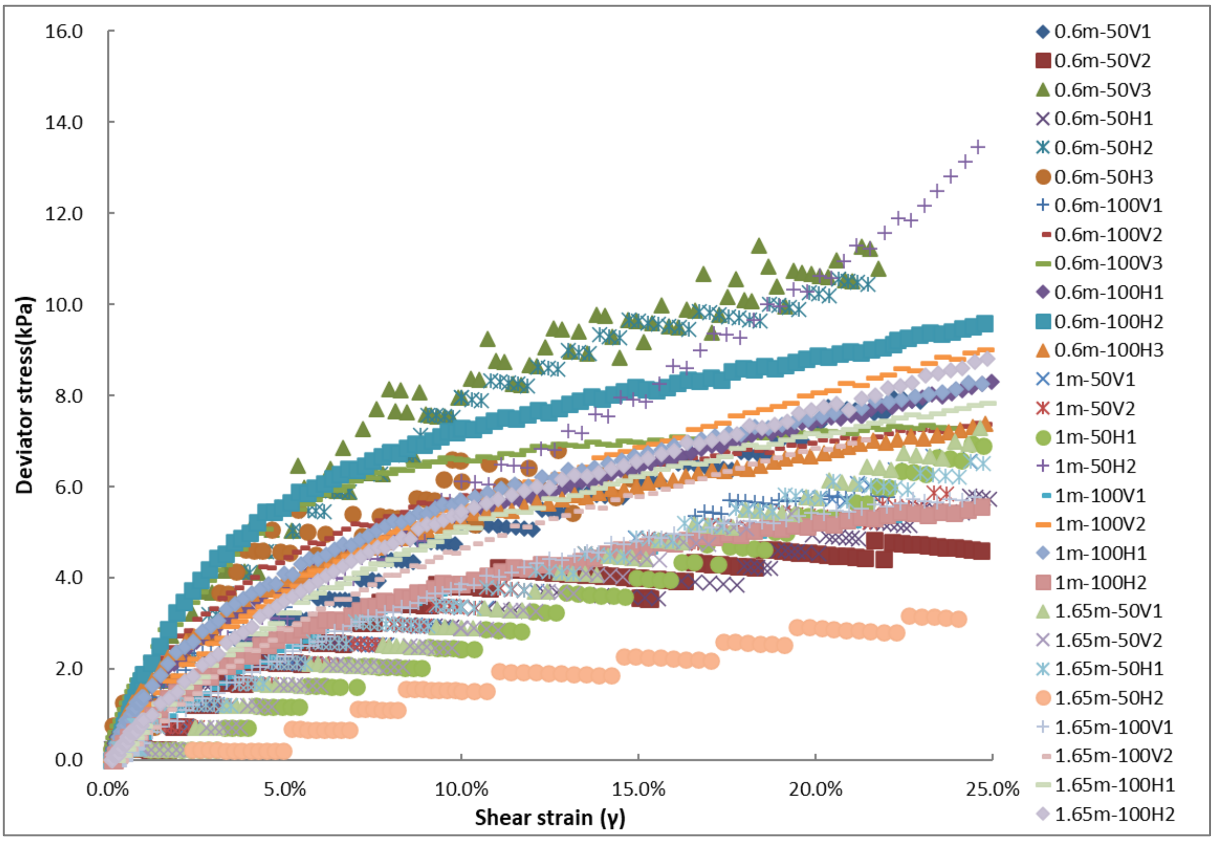


Figure 4-2: The development of deviator stress with increasing strain for peat from Locations 1 and 2

For clarity, the deviator stresses at some representative strains from 0.1% to 20% are listed in Table 4-1. The data for specimen 0.6m-50H3 from Location 1 was lost after 15% shear strain due to test errors. For the laboratory tests, the specimens from Location 1 were taken from 0.6 m below the ground surface, and deviator stresses at 20% strain were between 5.71 kPa and 11.09 kPa. This experimental result shows a high degree of agreement with the range of S_u from 5 to 10 kPa according to the in situ shear wave velocity measurements shown in Figure 3-6 above in Section 3.2.

Figure 4-2 also indicates that samples from greater depths exhibited lower deviator stress compared to those from shallower depths. This trend may be attributed to higher degrees of humification, weaker soil fabric, and, consequently, reduced shear strength in peat at deeper depths.

Table 4-1: Deviator stress ($\sigma_1 - \sigma_3$) for unconsolidated undrained triaxial test

Location	Specimen	Depth (m)	Strain				
			γ 0.1% (kPa)	γ 0.5% (kPa)	γ 5% (kPa)	γ 10% (kPa)	γ 20% (kPa)
1	0.6m-50V1	0.6	-0.25	0.381	2.99	4.94	7.30
	0.6m-50V2	0.6	-0.76	-0.253	3.58	5.13	5.73
	0.6m-50V3	0.6	-0.25	0.381	5.67	8.86	11.09
	0.6m-50H1	0.6	0.25	0.253	2.57	3.57	5.71
	0.6m-50H2	0.6	-0.25	0.254	4.78	7.80	10.27
	0.6m-50H3*	0.6	0.258	1.104	4.09	5.65	/
	0.6m-100V1	0.6	0.129	0.761	3.44	4.85	5.71
	0.6m-100V2	0.6	0.256	1.141	4.49	5.82	6.99
	0.6m-100V3	0.6	0.256	1.141	5.41	6.61	7.19
	0.6m-100H1	0.6	0.129	0.76	3.82	5.34	7.39
	0.6m-100H2	0.6	0.0017	0.887	5.64	7.25	8.85
	0.6m-100H3	0.6	0.129	0.951	3.97	5.28	6.73
2	1m-50V1	1.0	-0.76	-0.253	2.14	3.36	5.75

1m-50V2	1.0	-0.25	0.254	2.14	3.36	5.34
1m-50H1	1.0	-0.25	-0.253	1.17	2.44	5.34
1m-50H2	1.0	-0.25	0.592	3.59	5.65	10.65
1m-100V1	1.0	-0.125	0.222	2.52	3.85	5.16
1m-100V2	1.0	0.001	0.592	3.73	5.57	7.97
1m-100H1	1.0	0.129	0.824	4.09	5.69	7.56
1m-100H2	1.0	-0.125	0.38	2.76	3.97	5.22
1.65m-50V1	1.65	-0.25	0.254	1.65	2.90	5.75
1.65m-50V2	1.65	-0.76	-0.253	1.65	2.90	5.40
1.65m-50H1	1.65	-0.25	0.254	2.14	3.36	5.76
1.65m-50H2	1.65	-0.25	-0.253	0.69	1.53	2.90
1.65m-100V1	1.65	-0.253	0.042	2.28	3.85	5.42
1.65m-100V2	1.65	-0.253	0.253	3.00	4.77	6.84
1.65m-100H1	1.65	-0.125	0.253	3.25	5.11	7.15
1.65m-100H2	1.65	0.002	0.38	3.61	5.46	7.67

*The data for 0.6m-50H3 in Location 1 was lost after 15% shear strain due to test errors.

In Figure 4-2 and Table 4-1, no clear difference appears between horizontally cut and vertically cut specimens. However, some discrepancy is noted among specimens of the same size and cut directions from the same depth, for example, 0.6m-100V1, 0.6m-100V2, and 0.6m-100V3. Table 4-2 shows the water content of the specimens

100 vertical 1, 2, and 3, all taken from the same peat block from Location 1. The water content of specimen vertical 1 is greater than that of vertical 2 and 3. This in turn leads to smaller deviator stresses in vertical 1 compared to vertical 2 and 3, as shown in Figure 4- and Table 4-2. A higher water content results in lower deviator stress for peat. A similar correlation was observed for the 50 mm diameter specimens, suggesting that water content plays an important role in the mechanical behaviour of peat. That in situ water content varies significantly even along 0.1 m of depth can be observed in Figure 3-7a in Section 3.3. This will lead to differences in stress behaviours between depths.

Table 4-2: Water content of samples 0.6-100V1, 2, and 3 from Location 1

Specimen	0.6m-100V1	0.6m-100V2	0.6m-100V3
Water content (%)	1398	1078	1064

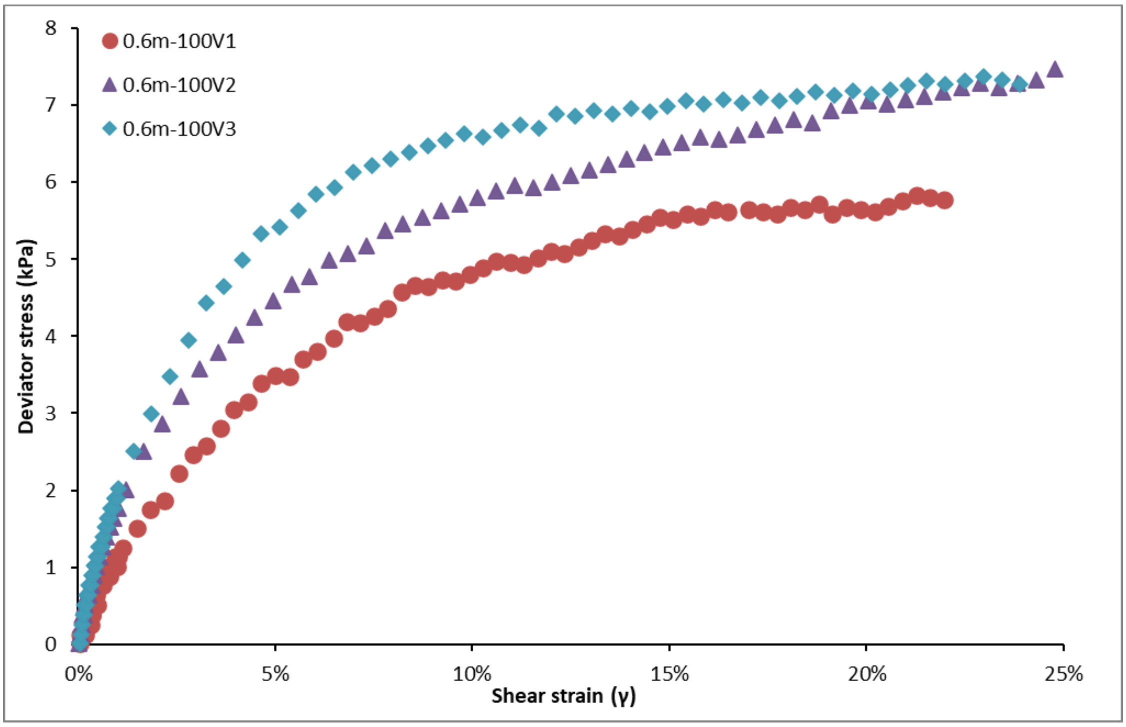


Figure 4-3: Deviator stress given different water contents

4.4.2 Disturbed samples versus undisturbed samples

At the Omagh wind farm, a peat block was collected from the same 0.35 to 0.65 m as the disturbed samples. Similarly, eight peat specimens of different diameters and orientations were collected at a depth of 0.35 m below ground level. The water content for each specimen is listed in the legend of Figure 4-4 below.

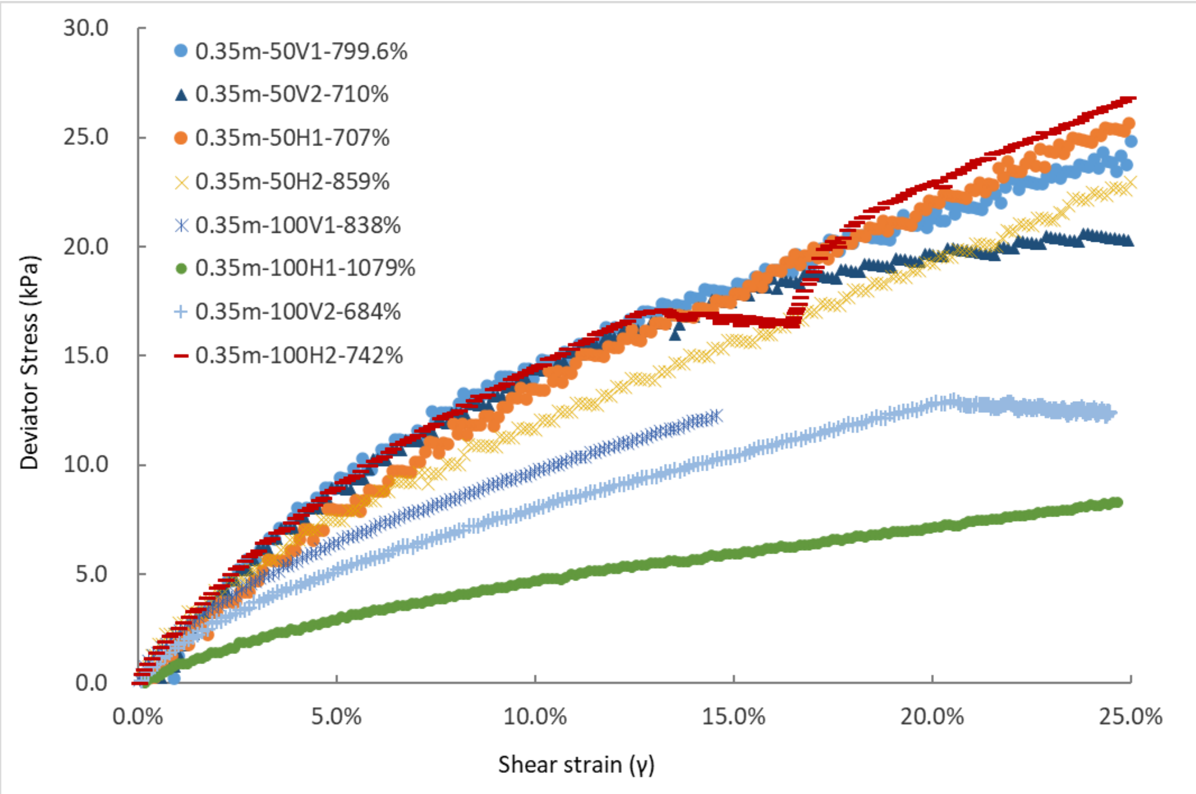


Figure 4-4: The development of deviator stress with increasing strain for disturbed samples

The deviator stress exhibited a consistent and proportional increase across the range of tested shear strains. This increase followed a linear uptrend, indicating a direct correlation between shear strain and deviator stress. The deviator stress values ranged from 7 kPa to 27 kPa at a shear strain of up to 25%.

For the undisturbed specimens, the deviator stress also increased until the end of the test. However, after 5% of shear strain, the rate of increase for deviator stress gradually slowed down, indicating a transition towards strain hardening behaviour, as shown in Figure 4-2 and Figure 4-3 above. The disturbed specimens maintained a constant increase rate until the end of the tests.

The water content of the peat specimens played a significant role in influencing the shear strength of the peat. The specimen labelled 0.35m-100H1-1079%, which had the highest water content of 1,079%, exhibited the lowest deviator stress among the eight tested specimens. This observation indicated the detrimental effect of a high water content on shear strength in peat soils.

Conversely, specimens with water content levels ranging from 700% to 800% exhibited relatively similar levels of deviator stress. This suggests a threshold effect, wherein, within this range of water content, the impact on shear strength remained relatively consistent.

The sensitivity of deviator stress to variations in water content highlights the critical role of water content control in engineering applications. Excessive water content can negatively affect soil stability and shear strength, and requires careful consideration and management in construction and geotechnical projects involving peat.

4.4.3 Shear deformation mode of peat

When a fine-grained mineral soil specimen is loaded to failure, shear bands or cracks usually develop across the specimen with increasing shear load (Nishimura, 2007). An example is shown in Figure 4-2a. Unlike this typical shear failure mode of mineral soil, a peat specimen usually holds straight, and no major shear band of failure evidence is observed during shear tests, as shown in Figure 4-2b.

To ensure the shear deformation mode of peat, an additional peat specimen was tested under significant shear strain of up to 48%, as shown in Figure 4-3c. Even though this specimen shrank and inclined significantly, there were still no major shear cracks observed in the specimen. During the undrained triaxial tests, shear load caused a soil volume contraction tendency and, therefore, generated positive excess pore water pressure inside the specimen. Once a specimen membrane was opened after a test, large amounts of brown and black water burst out due to the excess positive pore water pressure generated during shearing.

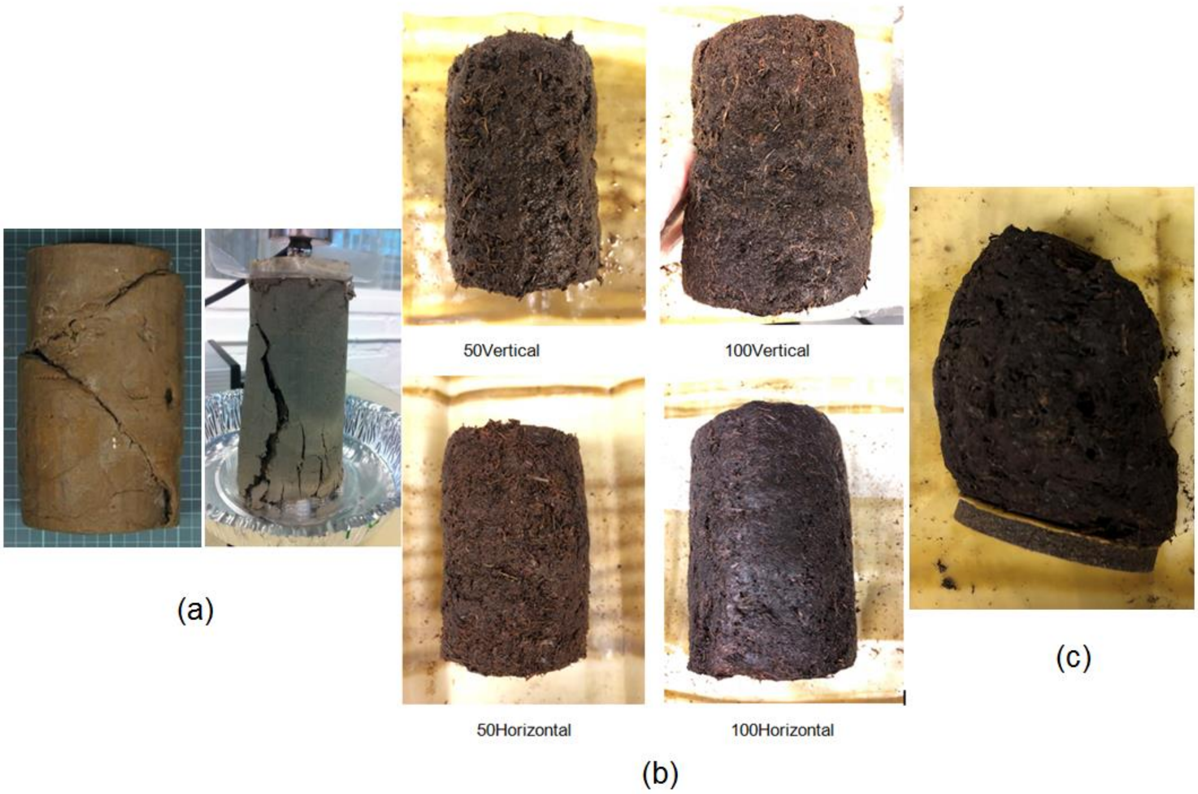


Figure 4-2: Comparison of peat and mineral soil specimens after triaxial tests, featuring (a) two typical shear failure modes of clay, (b) peat specimens under shear strains of up to 25%, and (c) peat specimen under a shear strain of up to 48%

As shown in Figure 4-2b, the horizontally cut specimens tended to develop slightly greater inclinations than the vertically cut ones after shearing, possibly due to the soil fabric anisotropy and fibre orientation. Peatland consists mainly of organic matter derived from the remains of decaying plant materials buried layer by layer over recent centuries and millennia. As such, peat behaviour in a vertical direction may differ from that in a horizontal direction. Muraro (2019) reported that large fibres of 1–3 cm in length can influence axial compression and radial expansion as they stretch and begin to diverge, which results in the peat anisotropy. More laboratory tests are needed to reveal the fundamentals of peat anisotropy. Regarding the scale effect, the 100-mm diameter specimens tilted significantly after the tests, whereas the 50-mm diameter specimens remained upright throughout, as smaller triaxial specimens are usually less susceptible to inclination or eccentric loading. This difference is also noted in the stress-strain relationship, which will be discussed explicitly in the following sections.

4.4.4 Peat deformation at large shear strain

After testing, the membranes were opened to view the deformed peat specimens. According to the observed deformation mode, after tests at large shear strain, specimens can be classified into three deformation groups. For each group, three representative specimen photos are provided along with stress-strain curves. These are shown in Figure 4-3 and described below.

(a) Bulged specimens (Figure 4-3a)

Specimens in the bulged group (0.6m-100V1, 0.6m-100H1, and 0.6m-100H2) exhibited a classical bulging deformation, in which the middle section expands uniformly during loading. These samples generally had more interlocking between particles and more homogeneous structures. The deviator stress increased smoothly even at large shear strain, suggesting elasticity behaviour. The gradual increase in deviator stress implies that the internal structure of the peat remained relatively intact, and energy dissipation occurred mainly through uniform shearing rather than localised failure.

Bulged behaviour indicates that the peat can accommodate large deformations with minimal risk of sudden failure.

(b) Specimens with visible cracks (Figure 4-3b)

Visible cracks were observed at the bottom of some specimens (0.6m-50H3, 0.6m-50V2, and 1.65m-100H1). In this deformation mode, the deviator stresses of most specimens initially rise but then gradually level off or decrease after crack formation.

Cracking has a critical influence on the deviator stress. The formation of microcracks reduces the stiffness of peat. As these cracks merge into larger fractures, the material behaviour transitions from strain hardening to strain softening.

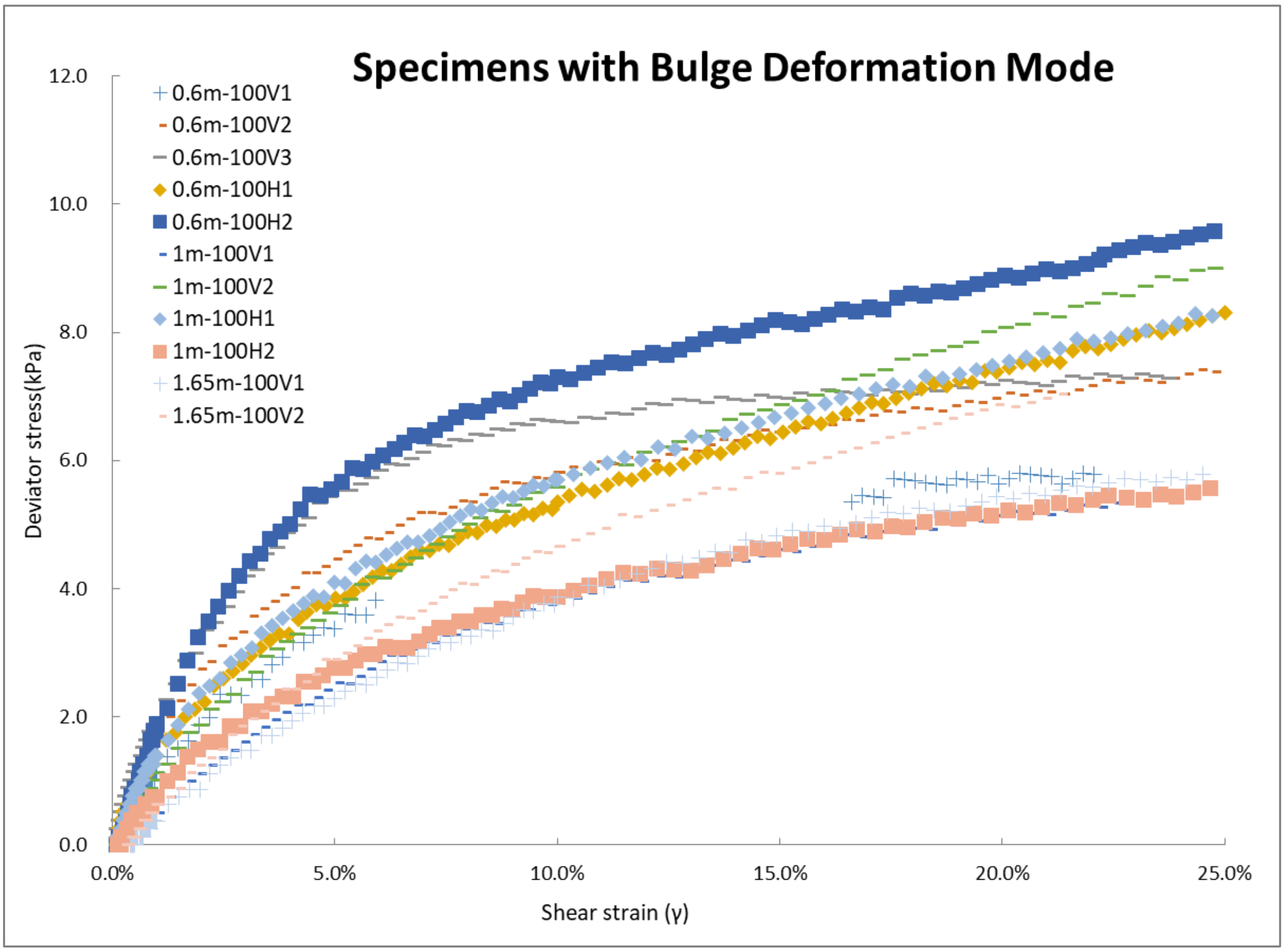
(c) Upright specimens (Figure 4-3c)

Smaller specimens of 50 mm diameter, such as 0.6m-50H1, 0.6m-50H2, and 0.6m-50V1, remained upright and stable without visible cracks or bulging, even at large strains. Deviator stress continued to increase steadily, with no visible peaks or failure points. This response may be attributed to the specimen size effect – smaller specimens have fewer structural discontinuities and less opportunity for shear localisation.

0.6m-100V1

0.6m-100H1

0.6m-100H2

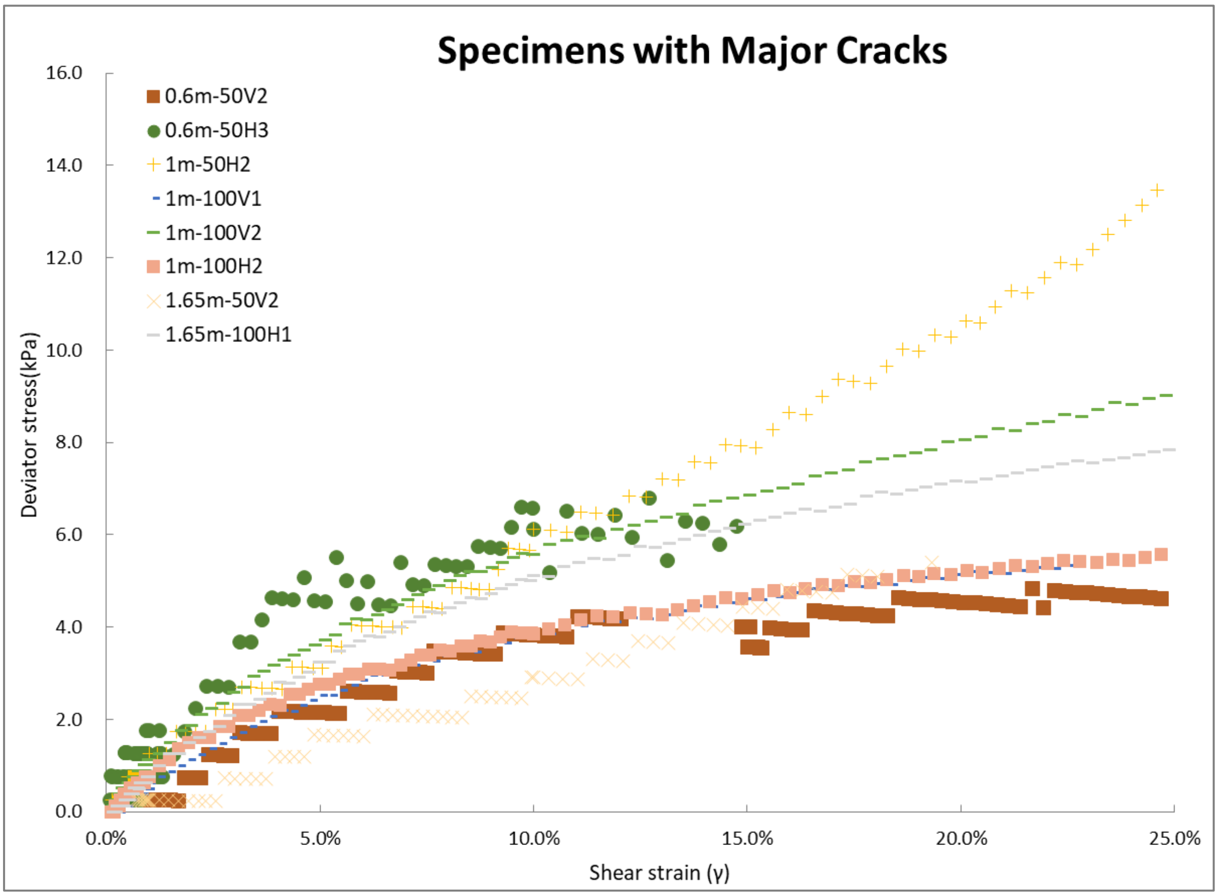


(a) Bulging deformation mode

0.6m-50H3

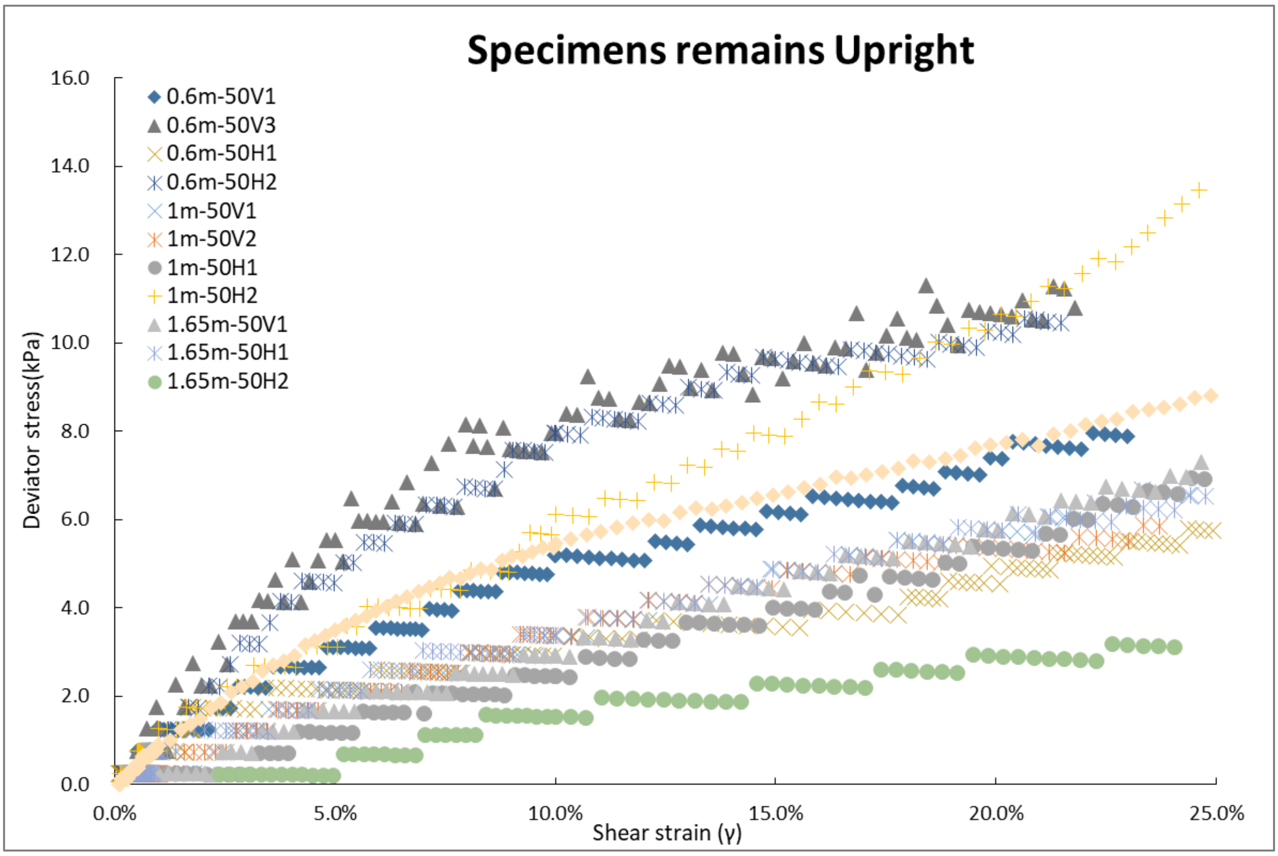
0.6m-50V2

1.65m-100H1



(b) Specimens with major cracks after the test

0.6m-50H1 0.6m-50H2 0.6m-50V1



(c) Specimens remaining upright after the tests

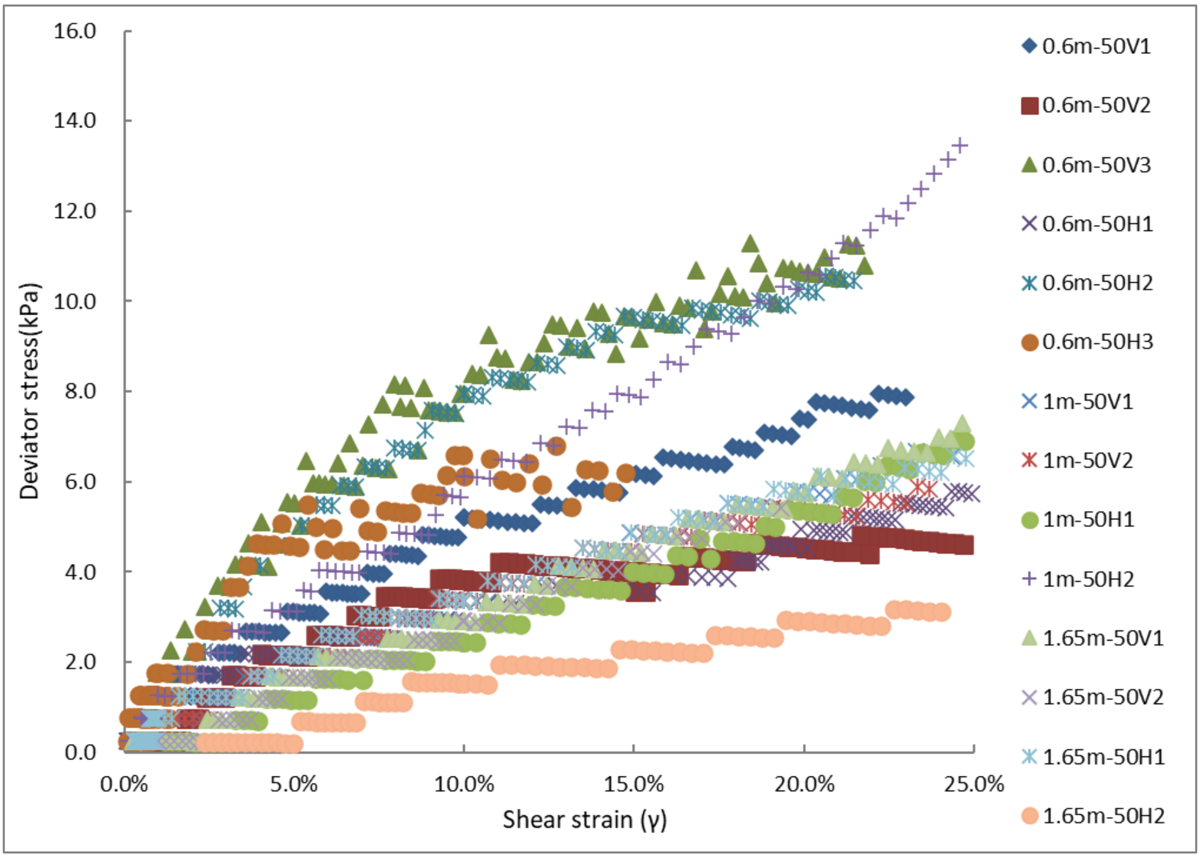
Figure 4-3: Three deformation modes of specimens after tests at large shear strain

The observed deformation modes reflect the complex, heterogeneous, and strain-dependent behaviour of peat. The transition from bulging to cracking represents a shift in failure mechanisms, which are strongly influenced by the degree of decomposition, anisotropy, and boundary effects.

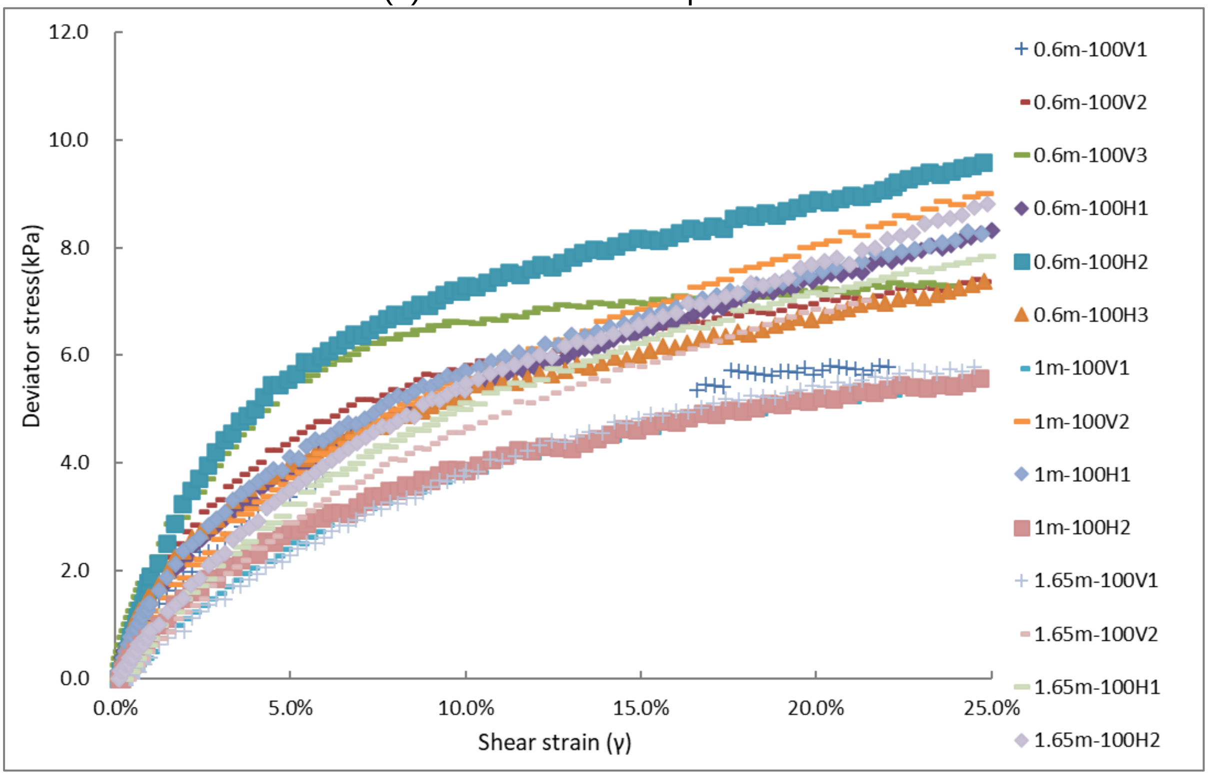
In design practice, this highlights the danger of relying solely on peak strength or assuming uniform deformation. Peat can behave deceptively well in the lab under certain conditions, but may fail rapidly under field stresses that trigger cracking or localisation. This reinforces the need to incorporate strain-softening, anisotropy, and scale effects in both numerical modelling and empirical design frameworks.

4.4.5 Scale effect

Figure 4-4 compares the deviator stress to the shear strain behaviour of the unconsolidated undrained triaxial tests between different specimen sizes (50 mm diameter vs. 100 mm diameter) to investigate the scale effect on peat shear behaviour. For the 50 mm diameter specimens in Figure 4-4a, the deviator stresses of some curves increase with shear strain in a stepwise manner (e.g., specimen 1.65m-50H2), as opposed to the consistent build-up observed in 100 mm diameter specimens shown in Figure 4-4b. Due to limitations of the experimental data recording system, the resolution of the load cell system is 0.1 kN. Hence, the shear strain of the 50 mm diameter specimens exhibited less continuity than that of the larger specimens.



(a) 50 mm diameter specimens



(b) 100 mm diameter specimens

Figure 4-4: Scale effect of peat

Omar and Sadrekarimi (2015) reported that specimen size significantly affects the stress-strain characteristics of sand samples regarding shear band development and specimen boundary conditions: smaller specimens usually exhibit greater shear strength and friction angles than larger samples. Similar findings have been noted regarding the scale effects of clay specimens (Ang and Loehr, 2003; Mountohar, 2011). In terms of peat, Muraro (2019) also investigated the scale effect using specimens with height to diameter ratios ranging from 1.5 to 3 and reported that the normalised deviator stress decreased with an increasing ratio of height to diameter. Likewise, the specimen dimension also influences the shear behaviour of peat, as seen in the difference between Figure 4-4a and b. Given the same shear strain, the 50 mm diameter specimens generally showed greater deviator stress and shear resistance than the 100 mm diameter ones.

4.4.6 Shear modulus

In a triaxial test, the shear modulus of a specimen can be derived from deviatoric stress q and deviatoric strain ε_s. The equation below shows the derivation procedure. More details can be found in Potts and Zdravkovi'c (1999) and other soil mechanics textbooks.

$$q = \sigma_a - \sigma_r$$

$$\varepsilon_v = \varepsilon_a + 2\varepsilon_r$$

$$\varepsilon_s = \frac{2}{3}(\varepsilon_a - \varepsilon_r)$$

For the undrained triaxial test condition,

$$\Delta\sigma_r = 0$$

$$\Delta q = \Delta\sigma_a$$

$$\Delta\varepsilon_v = \Delta\varepsilon_a + 2\Delta\varepsilon_r = 0$$

$$\Delta\varepsilon_r = -\frac{1}{2}\Delta\varepsilon_a$$

$$\Delta\varepsilon_s = \frac{2}{3}(\Delta\varepsilon_a - \Delta\varepsilon_r) = \Delta\varepsilon_a$$

$$\begin{bmatrix} \Delta p' \\ \Delta q \end{bmatrix} = \begin{bmatrix} K' & 0 \\ 0 & 3G' \end{bmatrix} \begin{bmatrix} \Delta\varepsilon_V \\ \Delta\varepsilon_S \end{bmatrix}$$

$$G' = G = \frac{\Delta q}{3\Delta\varepsilon_s} = \frac{\Delta q}{3\Delta\varepsilon_a}$$

To make different triaxial test data comparable, the secant shear modulus of peat in each individual test is normalised against shear moduli at the same reference shear strains. Ideally, G_{max} at very small strain levels (for example, 10^{-6}%) should be adopted as the reference shear strain value. In practice, however, it is enormously challenging to obtain high-quality, intact soft peat samples on site. After peat block excavation, sample disturbance may be further exacerbated during transportation and specimen preparation. Hence, 0.1% is selected as a reference strain for peat in this study. Strains smaller than that are usually of less concern for construction in soft peatland. Figure 4-5 and Figure 4-6 plot the normalised shear modulus with respect to the shear modulus at 0.1% shear strain against the shear strain γ from 0.1–1% and 1–10% on a logarithmic scale, respectively.

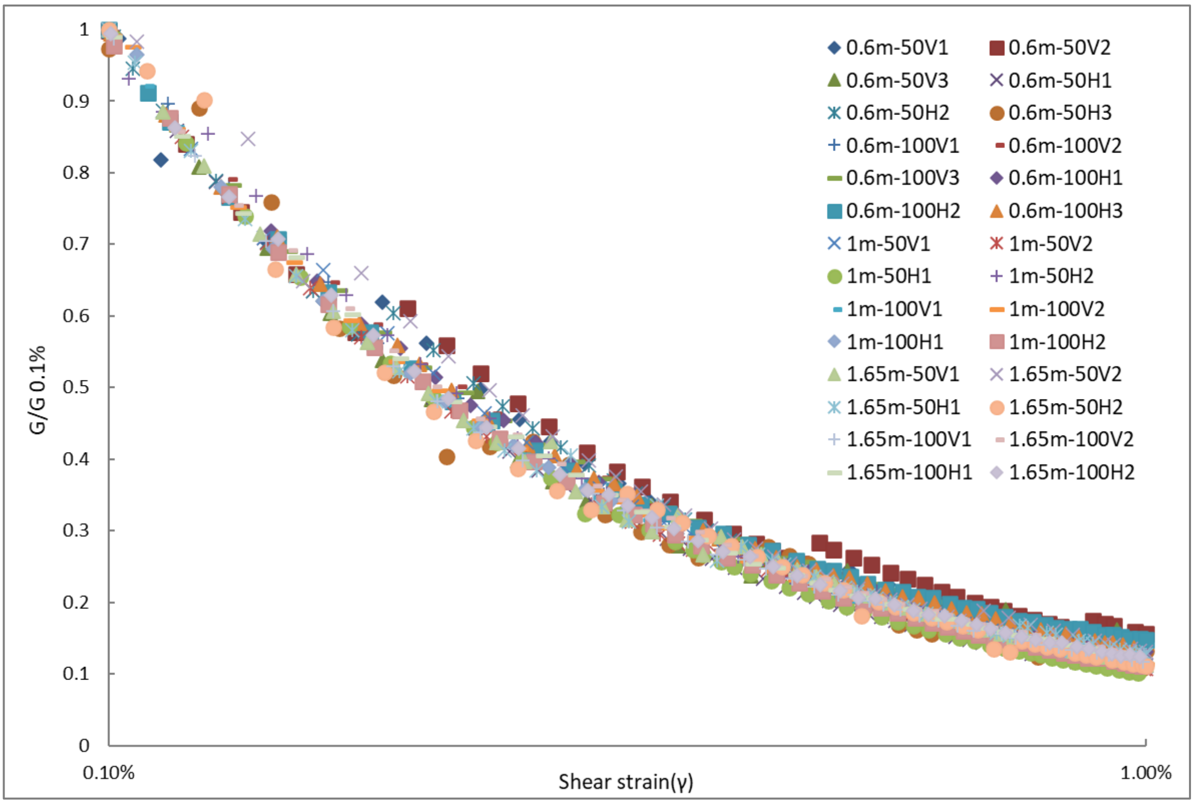


Figure 4-5: The shear stiffness degradation with increasing strain (0.1–1% in log scale)

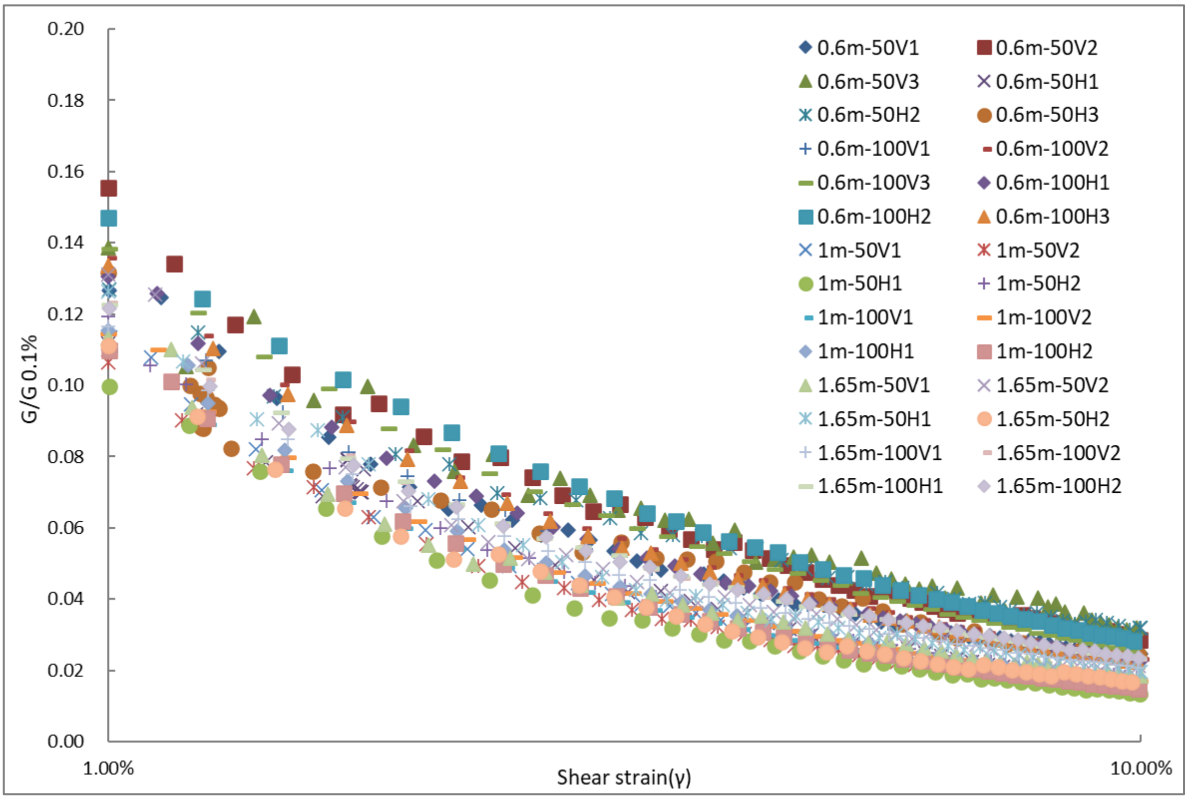


Figure 4-6: The shear stiffness degradation with increasing strain (1–10% in log scale)

Figure 4-5 and Figure 4-6 showed the normalised shear modulus reduction with increasing shear strain on a logarithmic scale. All the normalised data points of $G/G_{0.1\%}$ versus log γ generally fall within a narrow band. In Figure 4-5, the normalised shear stiffness $G/G_{0.1\%}$ decreases significantly with increasing shear strain, down to a range between 0.09 and 0.18 at 1% shear strain. Figure 4-6 shows that the normalised shear modulus $G/G_{0.1\%}$ continues to degrade after 1% shear strain and eventually reaches a range of 0.013 – 0.032 as shear strain reaches 10%.

4.5 Stiffness degradation

During the undrained triaxial tests, excess pore water pressure inside the specimens developed significantly and, therefore, reduced the radial effective stresses to nearly zero at a relatively large strain. Due to zero-lateral effective stress, there were likely to be some gaps between the specimens and the membranes, allowing for partial specimen drainage. After the tests, once the specimen membranes were opened, a large amount of brown and black water burst out, likely due to the drained water collecting between the specimen and the membrane.

Accordingly, the transition point of the stress-strain curve in each undrained triaxial test is considered an estimate of the specimen strength, in accordance with the method proposed by O'Kelly (2017).

Table 4-3 summarises the estimated shear strength for the 100 mm diameter specimens based on the transformation point method and the corresponding shear strain. It is noted that most of the estimated shear strength was developed at large strains, with an average strain of 8.599%.

Although no obvious shear failure modes were observed during the undrained triaxial tests, the shear stiffness of peat degrades distinctly with increasing shear strain. In addition to the unconsolidated undrained triaxial tests performed in this study, 16 extra sets of unconsolidated undrained triaxial test data (see Table 4-3) and 66 sets of consolidated undrained test data (see Table 4-4) on peat were collected from literature, including 13 tests on peat in Ireland, 26 tests on peat in the Netherlands by Boylan (2008), eight tests on peat in China by Xiong, (2006), three tests from France (Garnier, 2007), 22 tests from the Netherlands (Zwanenburg and Jardine, 2015), and seven tests from Canada (Hendry, 2012). The normalised stiffness degradation curves of these tests from the literature are shown in Figure 4-10. In general, the physical properties (i.e., von Post, water and organic contents) of samples from Ireland were similar to those tested in this study. In contrast, samples 2, 3, 5, and 6 from the Netherlands were buried over four metres below the ground and had water contents as low as 164% with relatively small organic contents of 20%.

Table 4-3: The estimated shear strengths for the 100 mm diameter specimens

Specimen no.	Shear strain	Estimated shear strength (kPa)
0.6m-100V1	8.988	4.48
0.6m-100V2	9.70%	5.71
0.6m-100V3	9.79%	6.629
0.6m-100H1	8.31%	4.876
0.6m-100H2	7.02%	6.37
0.6m-100H3	9.95%	5.24
1m-100V1	8.53%	3.58
1m-100V2	8.23%	5.227
1m-100H1	7.12%	4.82
1m-100H2	8.05%	3.48

1.65m-100V1	9.01%	3.56
1.65m-100V2	8.99%	4.37
1.65m-100H1	5.30%	4.63
1.65m-100H2	11.40%	5.827

Table 4-4: Physical properties of peat specimens from unconsolidated undrained tests available in the literature

Author / Year	Samples	Location	Size of specimens (H/D) mm	Depth (m)	Von Post	Water Content (%)	Organic Content (%)	Confining Stress (kPa)
Boylan / 2008	1	Ireland	120/70	0.63	H4	1,195	96	1.3
	2			0.63	H4	1,219	96	1.3
	3			0.62	H4	1,221	96	1.2
	4			0.33	H4	1,136	96	/
	5			0.6	H4/H5	1,492	94	1
	6			1.03	H4/H5	1,357	94	5
	1	Netherlands	120/70	2.1	H6/H7	657	76	16
	2			4.77	H5/H6	604	74	42.7
	3			4.62	H5/H6	939	84	41.2
	4			1.11	H5/H6	307	50	6.1
	5			4.19	H6/H7	251	40	36.9
	6			4.61	/	164	20	41.1
Xiong / 2006	1	China		/	/	/	/	50
	2			/	/	/	/	100

3	N/A	/	/	/	/	200
4		/	/	/	/	400

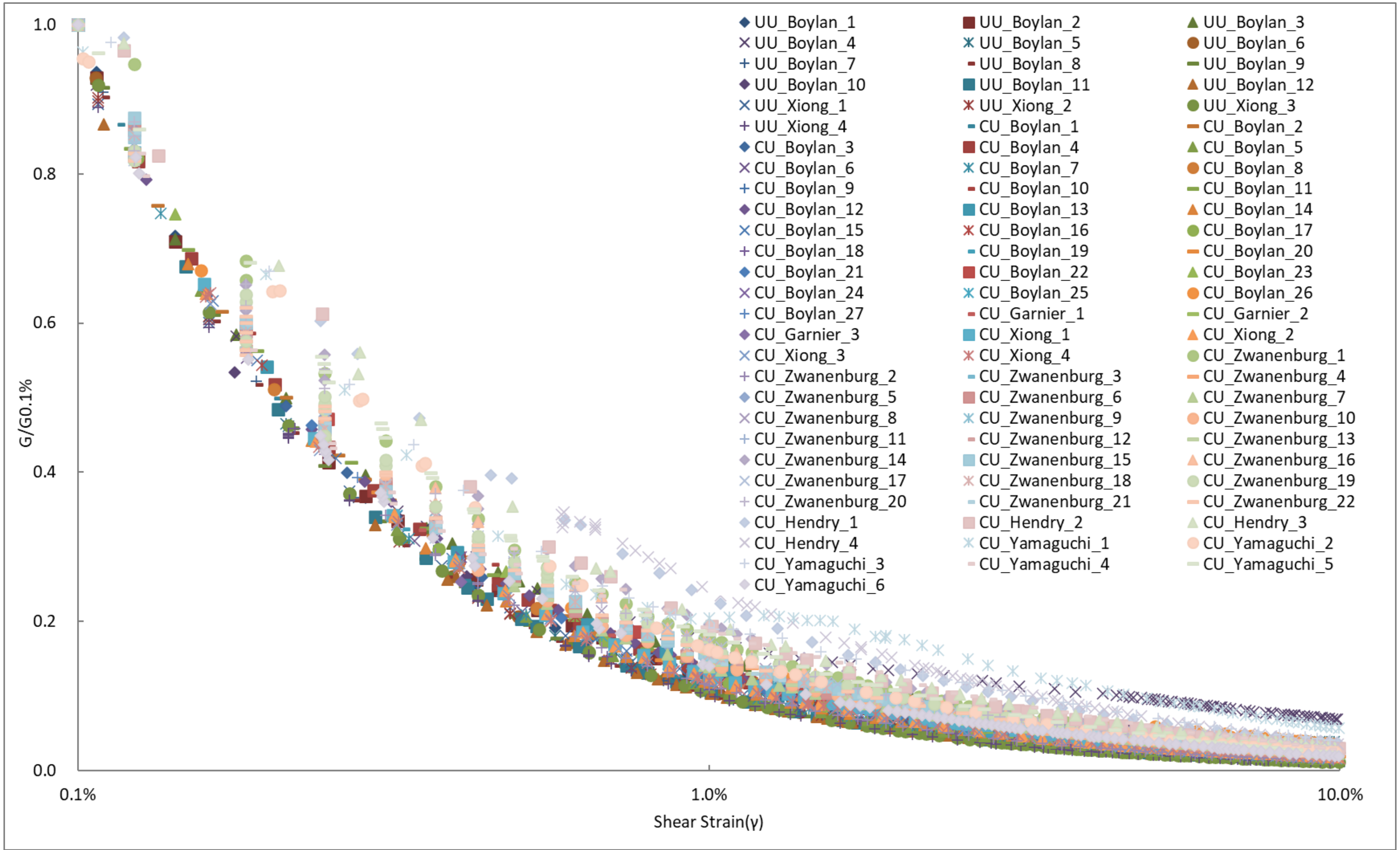


Figure 4-7: The normalised stiffness degradation of 82 peat shear lab tests from the literature

The normalised stiffness data from the current study aligns well with the existing literature from Figure 4-10, confirming that peat follows the general degradation pattern of soft soils. However, there is notable scatter between 0.3% and 2% of shear strain. Some data show more rapid reductions in stiffness, while others degrade more slowly. The variation observed is due to differences in organic content, sample structure, and confining stresses.

4.5.1 Determination of $G_{0.1\%}$

The methods for the small-strain measurement (for example, G_{max}) were investigated, and an attempt was made to attach Hall effect sensors to the cylindrical soil specimen (Figure 4-8 below). One Hall effect sensor was for the local radial strain, while another two were installed to measure the local axial strains in the middle section of the specimen. In theory, the two local axial strain measurements should be consistent and very similar. However, the results were unsatisfactory.

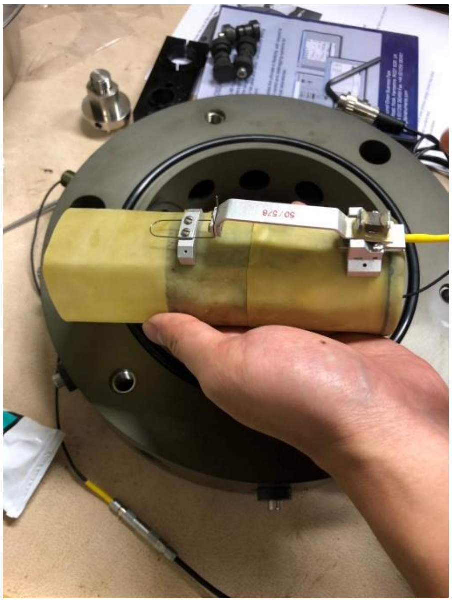


Figure 4-8: A specimen with a Hall effect sensor

As shown in Figure 4-9, the developments of the two local axial displacements/strains are distinctly different, even from the beginning of the triaxial test. This contradicts the assumption regarding Hall effect sensors for local small strains, suggesting that specimens might lean significantly during the tests due to high water content and soft soil. In addition, there might be a risk of water leakage around the Hall effect sensors. In summary, this attempt demonstrated that installing Hall effect sensors on peat for small-strain measurement seems unsuitable.

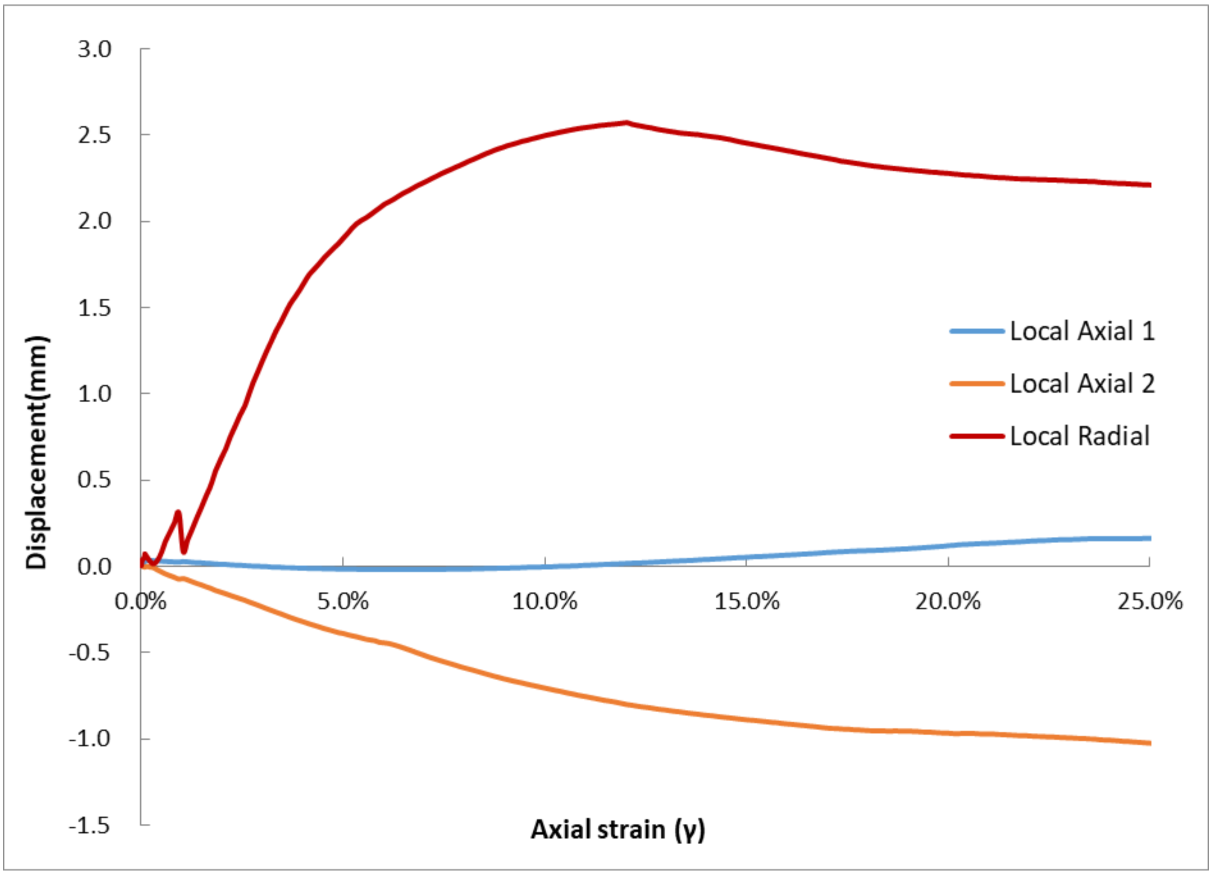


Figure 4-9: Local strain measurements from Hall effect sensors in a triaxial peat test

Likewise, in a bender element test, on-sample transducers must be installed carefully with a rubber sleeve to avoid leaks through the membrane. Such delicate instrumentation may also end up causing water leakage through the membrane under conditions of relatively large deformation in a peat test.

On the other hand, in a construction project, peat usually deforms significantly with relatively large strain. Therefore, it does not appear practically necessary to place excessive emphasis on small strain behaviour in the case of peat.

Considering that peat is extremely soft and highly susceptible to disturbance, G at 0.1% shear strain was adopted as the reference value instead of G_{max} at very small strain levels (for example, 10^{-6} %) for all the test data, both in this experiment and in the literature, as explained earlier. For mineral clay soil, some functions have been proposed to determine G_{max}, for example, that suggested by Vardanega and Bolton (2013). However, little information is available in the literature regarding the determination of G_{max} or $G_{0.1\%}$ for peat. Figure 4-10 shows the variation of $G_{0.1\%}$ for peat specimens subject to different confining pressures. For the vast majority of test data, shear stress increases linearly with greater confining pressure, and as such, a linear regression function for $G_{0.1\%}$ is given as follows:

$$G_{0.1\%}(kPa) = 0.0003\sigma'(kPa) + 0.6586, \qquad \textit{Equation 4-4}$$

where σ' is the confining stress, which ranges from 0.0 to 100 kPa.

In particular, six data outliers are plotted away from the linear regression line, as marked in Figure 4-10. These data outliers have much lower measured stiffness than that predicted by Equation 4-4. This discrepancy is probably due to high von Post Humification Scale rating of H8 compared against the von Post ratings of H4–H6 for other peat samples. In summary, the proposed linear regression formula (Equation 4-4) is mainly suitable for peat in the von Post range of H4–H6. More experimental data is required to derive the correlation between the von Post Humification Scale and the peat reference shear modulus $G_{0.1\%}$.

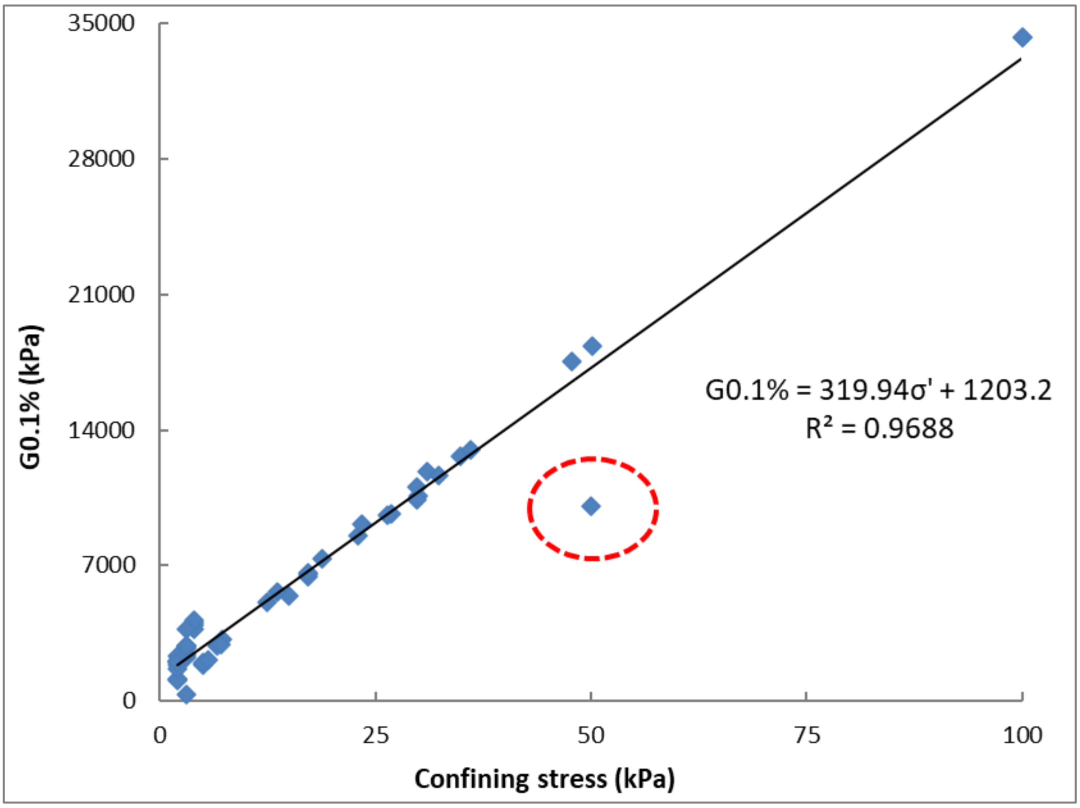


Figure 4-10: The method of determining $G_{0.1\%}$by confining stress

4.5.2 Fitting a model for shear modulus reduction with strain

For clay, Hardin and Drnevich (1972) suggested fitting the curve of stiffness degradation versus strain using a hyperbolic function as shown below:

$$\frac{G}{G_{max}} = \frac{1}{1+\beta\times(\frac{\gamma}{\gamma_{ref}})}, \qquad \textit{Equation 4-5}$$

where

- *G* is the shear modulus at the corresponding shear strain *γ*,
- G_{max} is the initial maximum shear modulus under small strain (10^{-6} %),
- β is the fit parameter, and
- γ_{ref} is the reference shear strain. Stokoe and Rosenblad (1999) suggested determining γ_{ref} as the shear strain for which G/G_{max} = 0.5.

In attempting to measure G_{max} in soft peat specimens, three triaxial tests using Hall effect sensors on the specimens were conducted, but the test results were not reliable. During the shearing stage, the specimens bulged unevenly around the Hall effect sensors, which failed to record the actual peat deformation.

Since accurate G_{max} values cannot be obtained for soft peat in lab tests, as discussed earlier, the reference strain $G_{0.1\%}$ was adopted to replace G_{max} in Equation 4-6. Correspondingly, $\gamma_{ref,0.1\%}$ represents the strain level for which $G/G_{0.1\%}$ = 0.5. After applying $G_{0.1\%}$ and $\gamma_{ref,0.1\%}$ to Equation 4-7, a modified equation was derived as follows:

$$\frac{G}{G_{0.1\%}} = \frac{G}{G_{max}} \times \frac{G_{max}}{G_{0.1\%}} = \frac{1}{1+\beta(\frac{\gamma}{\gamma_{ref,0.1\%}})} \times \frac{G_{max}}{G_{0.1\%}}, \qquad \textit{Equation 4-6}$$

where $\frac{G_{max}}{G_{0.1\%}}$ may be assumed as a constant value a when $\gamma = \gamma_{ref,0.1\%}$, as

$$\frac{G}{G_{0.1\%}} = 0.5 = \frac{1}{1+\beta} \times a. \qquad \textit{Equation 4-7}$$

Hence, β = 2a - 1. Replacing β in Equation 8 gives

$$\frac{G}{G_{0.1\%}} = \frac{a}{1+(2a-1)*(\frac{\gamma}{\gamma_{ref,0.1\%}})}. \qquad \textit{Equation 4-8}$$

For simplicity, $\gamma_{ref,0.1\%}$ was taken as 0.2188%, which is the average value for all the test data. Later, Equation 8 was adopted to determine a = $G_{max}/G_{0.1\%}$ = 5.358 by fitting all the data with maximum coefficients of determination, R^2 = 0.9913.

According to the VSWP described in Section 3.2.3, G_{max} was 4,049.37 kPa in the in situ test and $G_{0.1\%}$ was 1,140.38 kPa in the triaxial test. Hence, $G_{max}/G_{0.1\%}$ = 3.55. Kishida and Boulanger (2006) compared the in situ peat V_s against the V_s predicted by a function and found that in situ V_s tends to be 7% greater than predicted V_s, which may be due to various factors of peat diversity, including fabric, peat age, and density. The magnitude of curve-fitting a = $G_{max}/G_{0.1\%}$ = 5.358 is generally in line with $G_{max}/G_{0.1\%}$ = 3.55 from in situ and triaxial tests. In contrast, the discrepancy is likely due to peat diversity and disturbance during sampling, testing, and so on.

4.5.3 Accuracy of the prediction model

Figure 4-11 compares the normalised shear modulus $\mathrm{G}/G_{0.1\%}$ measured in the triaxial tests against the $\mathrm{G}/G_{0.1\%}$ predicted by Equation 4-9 for shear strain up to 5%. In this figure, the solid thick red line that runs diagonally across the graph represents the relationship between the average measured $\mathrm{G}/G_{0.1\%}$ and the average predicted $\mathrm{G}/G_{0.1\%}$ as below:

$$\frac{G}{G_{0.1\%}} = 0.9704\left(\frac{G}{G_{0.1\%,predicted}}\right). \qquad \textit{Equation 4-9}$$

Although not all the test data follow the equation above, 78.5% fall within a $\pm 30\%$ margin, as marked by the broken lines in the figure. Most of the measured *G* outside of the margin was conservatively underestimated by the prediction method. When shear strains up to 2.5% are considered for comparison, 87.9% of all the data fall within a $\pm 30\%$ margin, as shown in Figure 4-12. At large strains over 5% or 10%, the measured shear stiffness may not be as reliable as that at smaller strains due to specimen inclination under eccentric loading, as described earlier.

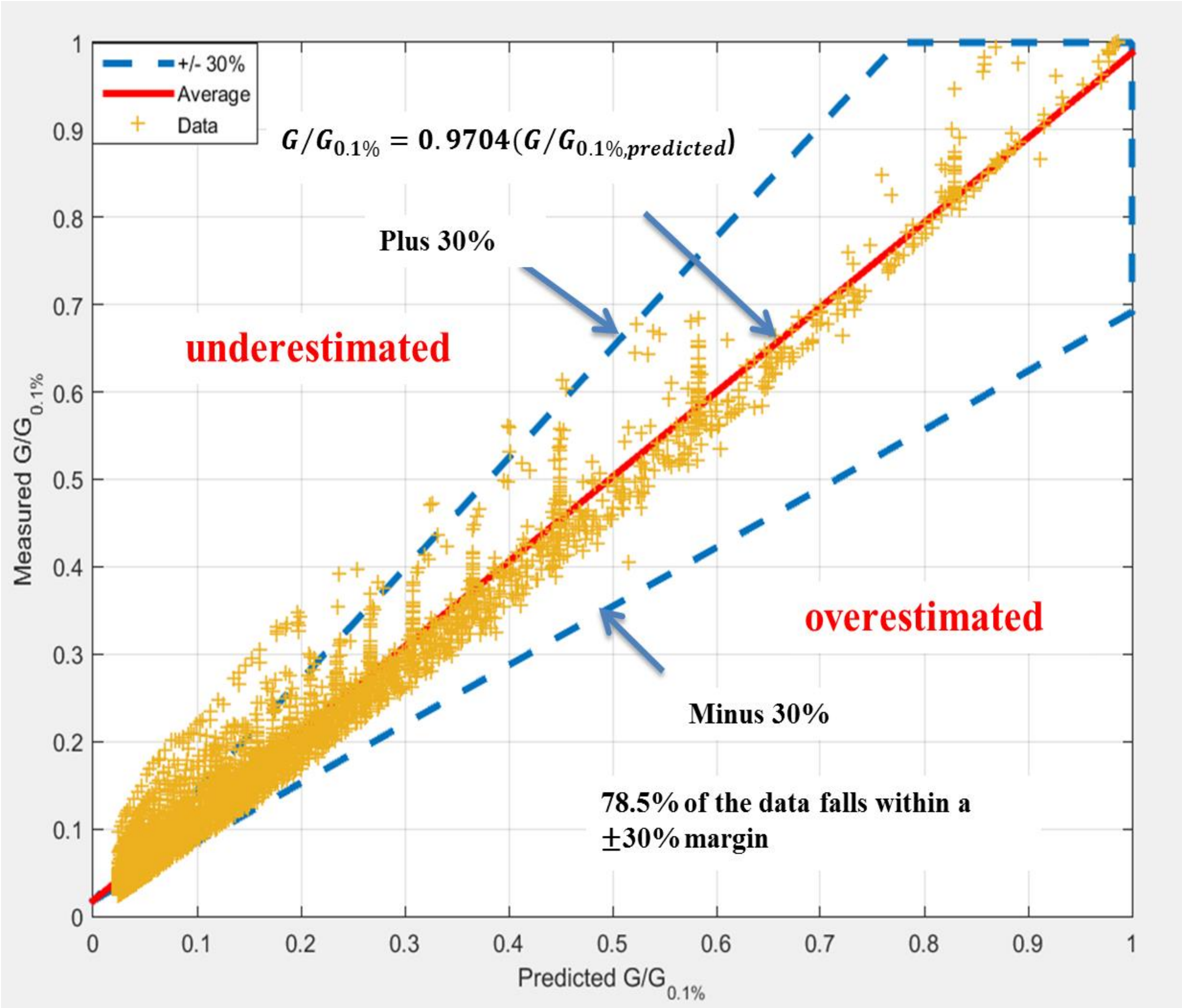


Figure 4-11: Accuracy of the prediction model for shear strains up to 5%

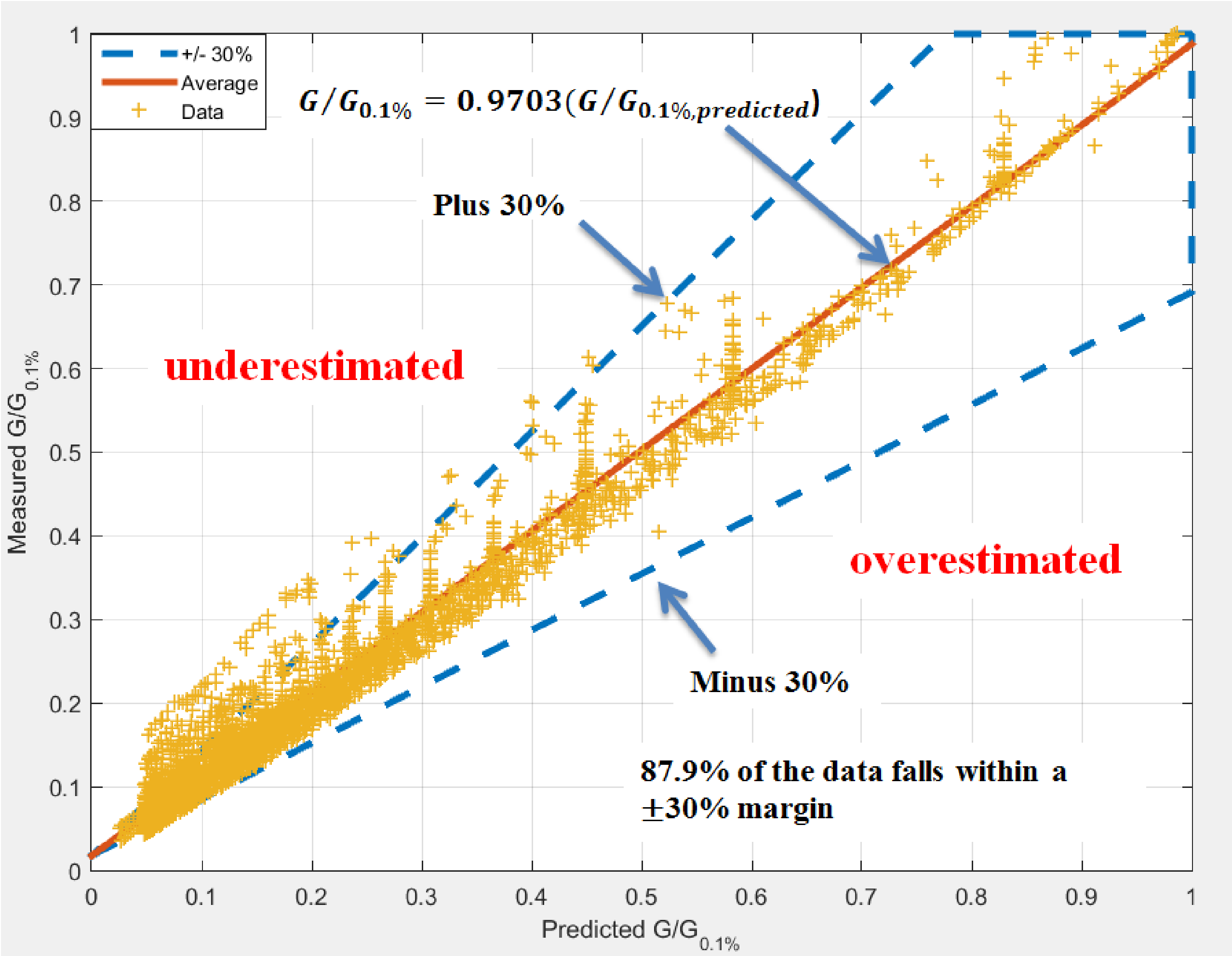
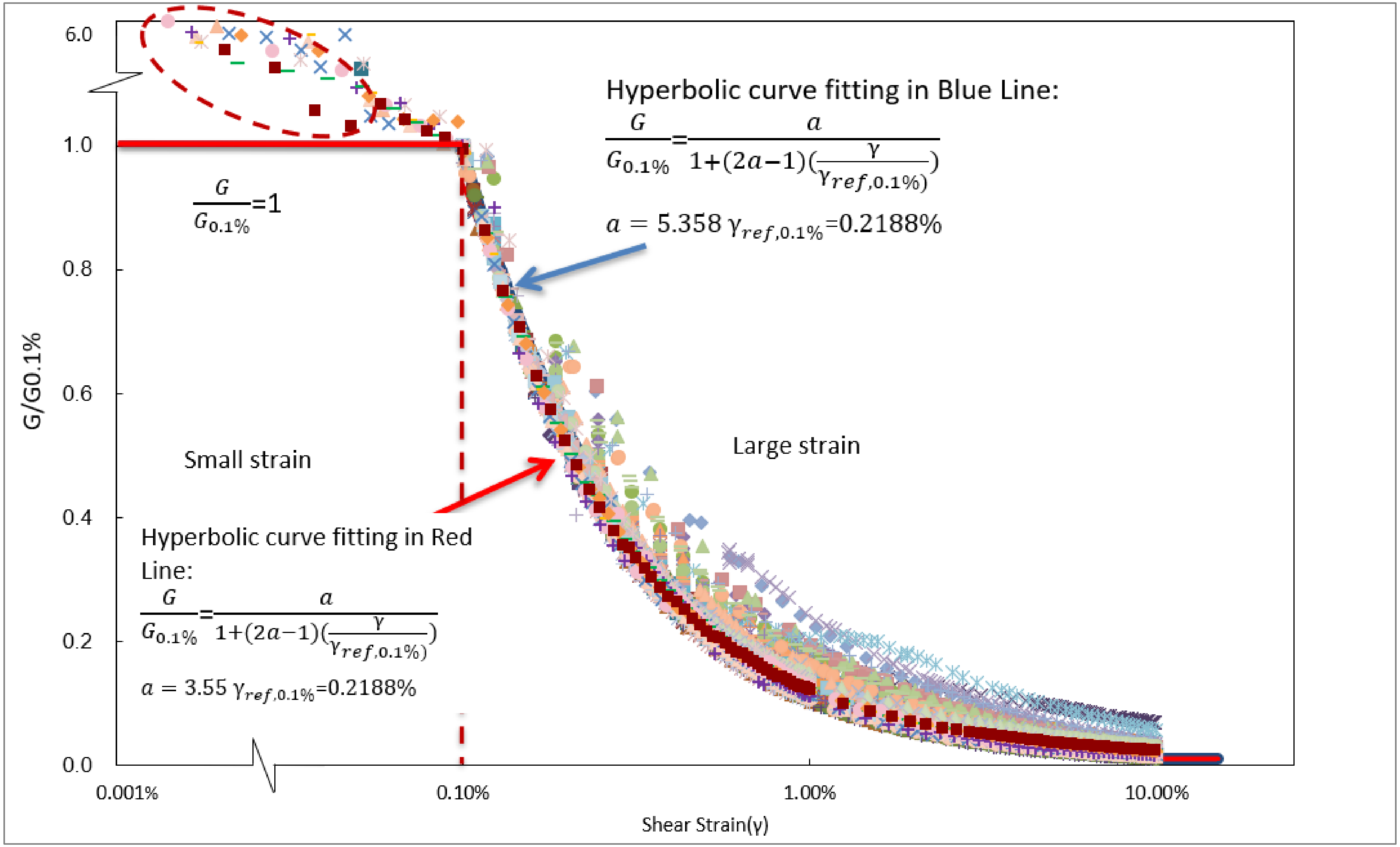


Figure 4-12: Accuracy of the prediction model for shear strains up to 2.5%

4.5.4 Simplified stress-strain model

The shear modulus degradation curve below 0.1% shear strain may be approximately estimated using Equation 4-10. However, this cannot be well-validated against reliable unconsolidated undrained test data from the literature. On the conservative side, Figure 4-16 shows a simplified shear stiffness degradation curve for the estimation of peat stiffness change due to civil construction on peatland, calculated as follows.

$$\begin{cases} \dfrac{G}{G_{0.1\%}(\gamma)} = 1 & \gamma \leq 0.1\% \\ \dfrac{G}{G_{0.1\%}} = \dfrac{5.358}{1+9.716*\left(\dfrac{\gamma}{\gamma_{ref,0.1\%}}\right)} & 0.1\% < \gamma < 10\% \\ \dfrac{G}{G_{0.1\%}(\gamma)} = 0.012 & \gamma \geq 10\% \end{cases} \qquad \textit{Equation 4-10}$$


6.0
1.0
0.8
0.6
0.4
0.2
0.0
G/G0.1%
0.001%
0.10%
1.00%
10.00%
Shear Strain(γ)
Hyperbolic curve fitting in Blue Line:
$\frac{G}{G_{0.1\%}}=\frac{a}{1+(2a-1)(\frac{\gamma}{\gamma_{ref,0.1\%}})}$
a = 5.358 $\gamma_{ref,0.1\%}$=0.2188%
$\frac{G}{G_{0.1\%}}$=1
Small strain
Large strain
Hyperbolic curve fitting in Red Line:
$\frac{G}{G_{0.1\%}}=\frac{a}{1+(2a-1)(\frac{\gamma}{\gamma_{ref,0.1\%}})}$
a = 3.55 $\gamma_{ref,0.1\%}$=0.2188%

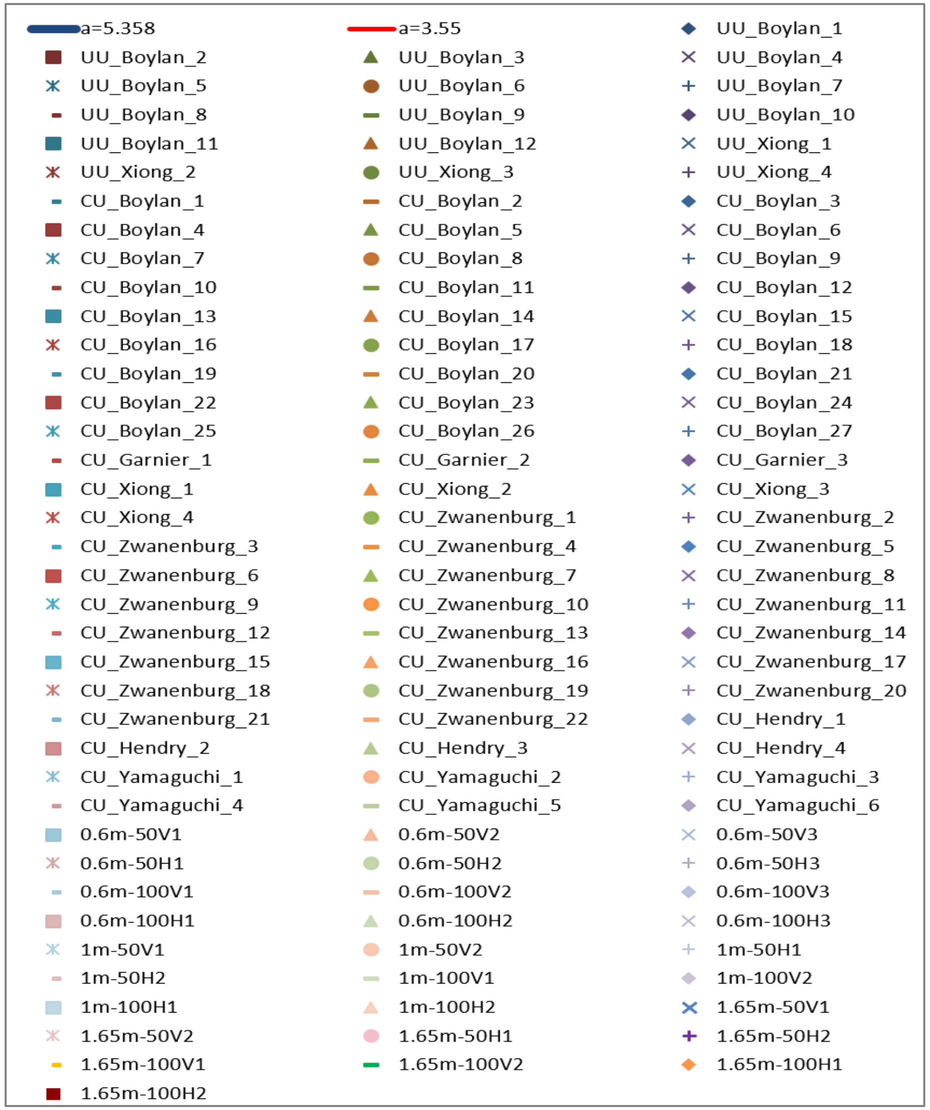


Figure 4-13: Simplified function for peat stiffness degradation with figure legend

For shear strains smaller than 0.1%, $G/G_{0.1\%}(\gamma)$ may be assumed to be 1, which conservatively underestimates peat shear stiffness at small strain levels. As shear strain increases from 0.1% to 10%, the stiffness degradation function in Equation 8 is adopted. At large shear strains above 10%, the value of $G/G_{0.1\%}(\gamma)$ may be assumed as a constant 0.012 for simplicity. At very large shear strain levels, substantial or even intolerable ground displacement may have already developed on peatland. In these cases, the magnitude of peat deformation cannot be accurately predicted based on the stiffness degradation method, as the fundamental assumption of small deformation for continuum mechanics may not apply.

4.6 Summary

In this chapter, a series of unconsolidated undrained triaxial tests were performed on peat specimens of two different sizes in two different orientations collected from two locations in Ireland. The mechanical behaviour of peat was found to be significantly different from that of fine-grained mineral soil. The main conclusions are summarised as follows.

The in situ water content of peat was found to be greater than the liquid limit. The water content plays an important role in peat mechanical behaviour and can vary significantly within a meter, both vertically and horizontally. The von Post Humification scale can also affect the behaviour of peat, mainly depending on the fibres remaining in the peat.

In all the triaxial tests, the deviator stresses increased continuously with shear strain and no obvious peak strength was found, even at large strains over 25%. Meanwhile, positive excess pore water pressure developed in the specimens due to shearing.

Unlike the typical shear failures of mineral soil, no clear evidence of shear failure modes or shear bands was observed in any peat specimen. Based upon the images from the triaxial tests, three peat deformation modes at large strain levels were identified: bulged specimens, specimens with visible cracks, and upright specimens. All of these groups show different stress-strain relationships.

The deviatoric stresses at large shear strains in the triaxial tests show high degrees of agreement with the undrained peat shear strength S_u predicted by the in situ shear wave velocity measurement. It is demonstrated that the shear wave velocity measurement V_s can be used together with the water content ω to derive the undrained peat shear strength S_u along a depth gradient based on Trafford and Long (2016)'s method.

In the triaxial tests, the horizontally cut specimens tended to exhibit slightly steeper failure plane inclinations than the vertically cut specimens after shearing. This difference could possibly be due to the soil fabric anisotropy and fibre orientation that develops during peatland formation layer by layer over centuries and millennia. In addition, the specimen size influences the stress-strain relationship of peat. Given the same shear strain, smaller specimens generally show greater shear resistance strength than larger specimens.

The shear stiffness of peat degrades significantly with the increase of shear strain. A normalised shear modulus was plotted against shear strain on a logarithmic scale. The normalised shear stiffness $G/G_{0.1\%}$ decreased substantially to 0.09–0.18 at 1% shear strain and continued to degrade to 0.013–0.032 at a shear strain of 10%. Two methods were proposed to estimate the magnitude of shear modulus G at 0.1% shear strain based upon the peat depth and confining pressure.

Shear modulus reductions with strain may be expressed using a hyperbolic function. All the data points of $G/G_{0.1\%}$ versus the shear strain γ were plotted within a narrow band. When the shear strain γ is smaller than 5%, 78.5% of the $G/G_{0.1\%}$ falls within a margin of ±30%. Data falling outside the predicted margin is usually underestimated. A conservative, simplified shear stiffness degradation curve is proposed for the estimation of peat stiffness changes due to civil construction on peatland under short-term undrained conditions.

5 CONSOLIDATED UNDRAINED TRIAXIAL TEST

5.1 Abstract

In this chapter, a series of consolidated undrained triaxial compression tests were conducted to investigate peat shear behaviour on samples collected from 1.65 m depths and subjected to stress levels ranging from 10.4 kPa to 40.5 kPa. At the consolidation stage, the triaxial test specifically investigated peat isotropic compressibility at low stress levels, the results of which show agreement with oedometer test data available in the literature. The subsequent triaxial shearing stage results show that most of the test data fail to reach the tension cut-off line (q/p' = 3), which indicates that the deviator stress may represent more of an interparticle connection than the tension of fibres and wood in peaty soils.

For soft peat, the membrane correction effect on peat shear resistance is strain dependent; generally, it is small below 10% shear strain, but it becomes significant above 10% shear strain. A critical state line for peat was determined based on the maximum curvature approach for cases in which the Mohr-Coulomb model has difficulty determining the friction angle for soft peat. Of the data recorded for peat, 78% fell within the range of 30° to 60°. This increased to 90.4% when ignoring points lower than 10 kPa. Previous test data for very low stress levels (less than 10 kPa) might not

be sufficiently reliable due to the limitations of conventional triaxial testing apparatuses, specimen preparation procedures, and related factors. In addition, organic content also plays an important role in peat shear behaviour. In general, when the organic content exceeds 75%, the deviator stress behaves like that of organic soils, otherwise, the peat behaves more like a mineral soil. In peat samples with organic contents higher than 75%, the direct shear box test gives higher estimates of shear strength than the triaxial shear test, however, these estimates may not accurately reflect the true strength characteristics, the mechanism of direct shear acts only at the centre of a specimen, while triaxial shear can shear throughout a specimen.

5.2 Test procedure

In Ireland, there are over 12,000 km^2 of bogland (Geography of Ireland, 2009), growing energy demand and rapid urbanisation necessitate an increasing number of onshore wind farms, floating roads, intercity transportation networks, and other infrastructure, some of which will inevitably be built on or across peatland. As such, it is essential to understand peat shear behaviour and failure mechanisms for a variety of ground construction projects subject to different stress levels (Figure 5-1).

In general, in situ peat has a very low shear strength, typically less than 10 kPa. The determination of shear strength is challenging due to peat's high water content, high organic content, variation in degrees of humification, and inevitable sample disturbance.

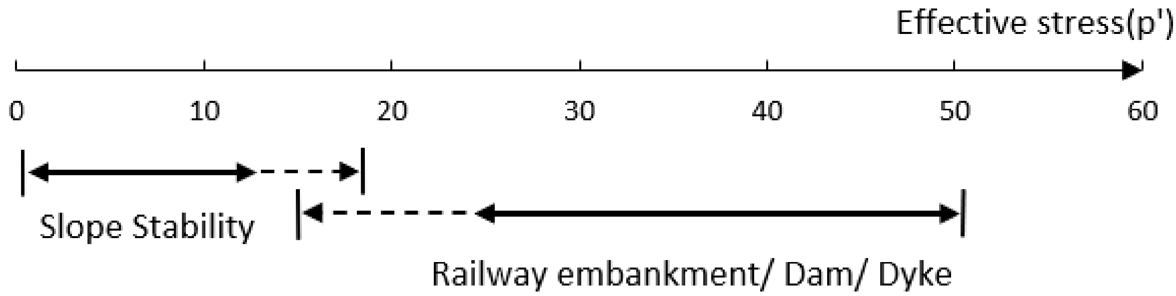


Figure 5-1: Effective stress levels for different ground construction projects

Figure 5-1 shows data from previous investigations of triaxial tests on peat subject to different stress levels: 0–10 kPa was mainly considered for peat slope stability analysis, while 10–50 kPa was applied to specimens to determine peat responses under railways or dams.

In this study, 14 consolidated undrained triaxial tests were conducted following the BS1377-8 (1990) to investigate the shear behaviour of peat samples subjected to different stress levels. The experimental data were then compared against previous studies' oedometer and triaxial test results. The results of this work have important implications for the determination of peat's compressibility and shear properties and, in particular, for examining the effect of membranes on deviatoric shear stress.

The peat specimens used in this study were obtained from a wind farm located in Omagh, Northern Ireland. The samples were taken from a depth of 1.65 m below the ground surface and subjected to different stress levels ranging from 10.4 kPa to 40.5 kPa. For each stress level, there were 3–4 repeated triaxial tests of peat samples (14 in total) from the same depth to minimise experimental errors. The blocks were excavated carefully, and the centres were cut out with a flat shovel to minimise sample disturbance.

In previous peat triaxial tests, a variety of specimen diameter ratios have been adopted for different consolidated undrained tests, for example, 1.7 by Boylan (2008), 1.3 by Garnier (2007), and 2.0 by Hendry (2012). The majority of studies have adopted a 1.7 specimen diameter ratio (Boylan, 2008). In line with previous unconsolidated undrained peat tests (Wang, 2021), a height-to-diameter ratio of 1.7 was used for the triaxial specimen dimensions for the peat unconsolidated undrained tests in this study. This ratio was suggested by Berre (1982) and Boylan (2008).

Table 5-1 lists the physical properties of the 14 peat specimens subject to different confining stresses. On average, the water content for the tested peat was around 1,100%. At the beginning of each consolidated undrained triaxial test, the specimen was fully saturated, reaching a B-value (indicating the degree of saturation) greater than 0.95. After the saturation stage, the specimen was consolidated subject to different confining stress values ranging from 10.4 kPa to 40.5 kPa. The higher the confining stress, the smaller the diameter and height of the specimens after consolidation. In the final shearing stage, the specimens were slowly sheared up to a 40% shear strain at a rate of 0.0118 mm/min in an undrained condition.

Table 5-1: Physical properties of the triaxial test specimens

No.	Water content	Humification of samples	e_0	$\Delta\varepsilon$	Confining stress (kPa)	After consolidation	
						Diameter (mm)	Height (mm)
1	1,209%	H8	22.05	10.90%	10.4	46.33	79.57
2	1,225%	H8	20.772	16.70%	10.7	45.8	77.86
3	1,188%	H8	21.762	19.10%	12.3	45.28	76.98
4	1,154%	H8	21.96	21.70%	12.5	48.2	81.9
5	1,209%	H8	21.762	25.90%	17.8	44.35	75.39
6	1,173%	H8	22.644	22.20%	18.4	46.78	79.53
7	1,124%	H8	21.42	24.40%	21.4	44.86	72.67
8	1,258%	H8	21.636	25.80%	29	49.03	83.35
9	1,202%	H8	21.168	40.20%	31.7	41.04	69.77
10	1,176%	H8	21.114	38.20%	32.6	42.59	72.4
11	1,190%	H8	21.204	37.80%	36.5	40.76	69.29
12	1,234%	H8	21.618	35.50%	36.5	41.81	71.077
13	1,159%	H8	20.862	31.90%	37.5	45.19	70.067
14	1,194%	H8	21.384	35.40%	40.5	41.547	70.6299

A test normally consists of four stages and involves the use of GDG triaxial equipment. The first and second stages are the saturation stage and the B-check. First, the cell pressure and back pressure are both increased to 100 kPa. Then, the changing back pressure is divided by the changing cell pressure; once $B = \Delta u/\Delta x \geq 0.95$, the specimen is regarded as fully saturated. After that, the test will go to the consolidation stage. Specimens will be consolidated under different confining stresses. The consolidation time for each group will depend on the variety of confining stresses used and will take longer with a higher stress level. The time required ranges from 10 hours to 48 hours. Once the volume change within 5 min is smaller than 5 cm^3, the consolidation stage ends, and the shearing stage begins. All specimens will shear at a rate of 0.0118 mm/min until a shear strain of 40% is reached.

Figure 5-2 shows the procedure for the consolidated undrained triaxial test.

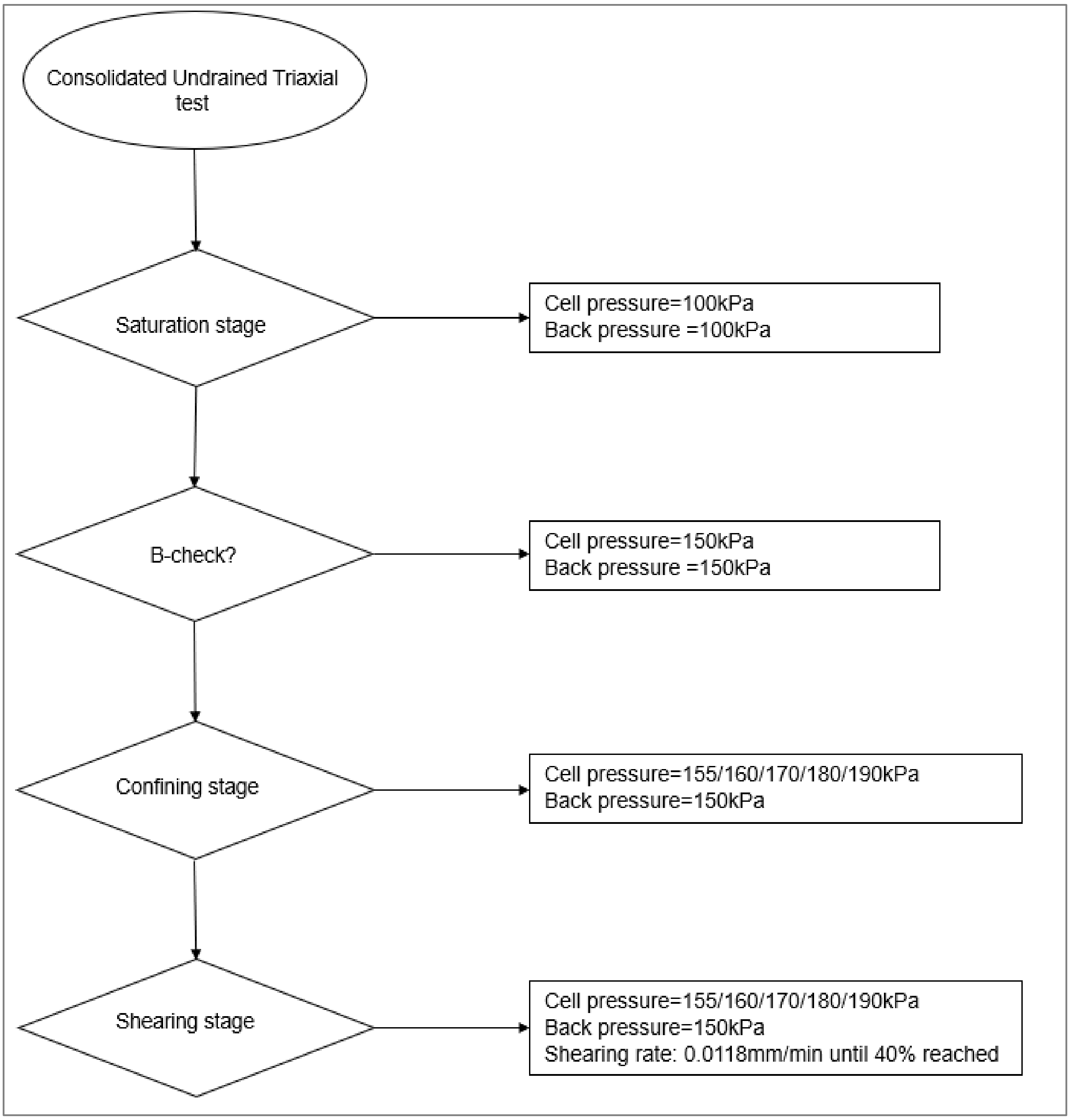


Figure 5-3: Consolidated undrained triaxial test procedure

5.2.1 Consolidation stage

5.2.1.1 Details of the specimens

After the consolidation stage, both the diameter and height of the specimens had changed in correspondence with the confining stress. Table 5-2 below records the sample diameters and heights at the end of the consolidation stage.

Table 5-2: Parameters after the consolidation stage

Test No.	Confining stress (kPa)	Diameter (mm)	Height (mm)
1	5.5	49.515	84.175
2	4.6	50.17	85.298
3	4.4	50.792	86.3464
4	10.7	48.2	81.9
5	12.5	45.28	76.98
6	10.4	46.33	79.57
7	12.3	45.80	77.86
8	21.5	45.31	77.03
9	17.8	44.35	75.39
10	18.4	46.78	79.53
11	29	49.03	83.35
12	31.7	41.04	69.77
13	32	42.59	72.40
14	36.5	40.76	69.29
15	36.5	41.81	71.077
16	37.5	45.19	70.067
17	40.5	41.547	70.63

The table shows that, as the confining stress increases, both the diameter and height of the specimen after consolidation tend to decrease.

Notably, the specimens with 5 kPa of confining stress were larger than they were before consolidation. This was due to the parent peat blocks being taken from 1.65 m below the ground surface. Once these blocks were extracted, then the effective stress inside the specimens became negative, around 8 kPa. Since this is larger than 5 kPa, when the confining stress was 5 kPa, the outside pressure was less than the internal pressure. This led water to flow into specimens, and the volume increased.

5.2.1.2 Coefficient of consolidation (C_V)

Based on the BS 1377-6:1990, the definition of the coefficient of consolidation is

$$C_V = \frac{0.196\overline{H}^2}{t_{50}},$$ *Equation 5-1*

where

- $\overline{H}$ = 1/2$(H_1 + H_2)$,
- H_1 is the height of the specimen at the start of the consolidation stage (in mm),
- H_2 is the height of the specimen at the end of the consolidation stage (in mm), and
- t_{50} corresponds to 50% pore pressure dissipation.

5.2.1.3 Coefficient of volume compressibility (m_v)

$$m_v = \frac{\Delta V_2 - \Delta V_1}{V_0 - \Delta V_1} X \frac{1000}{P_2 - P_1}$$ *Equation 5-2*

For triaxial tests, the coefficient of volume compressibility changes with the changing sample volume, where

- ΔV_1 is the cumulative change in volume of the specimen, from the initial volume to the volume at the end of the previous consolidation stage (in cm^3);
- ΔV_2 is the cumulative change in volume of the specimen, from the initial volume to the volume at the end of the consolidation stage being considered (in cm^3);
- V_0 is the initial volume of the specimen (in cm^3);
- P_1 is the effective pressure applied to the specimen in the previous consolidation stage (in kPa); and
- P_2 is the effective pressure applied to the specimen in the consolidation stage being considered (in kPa).

5.2.1.4 Settlement time

The equation for determining the time required to reach a particular percentage of consolidation is as follows:

$$t = \frac{T_v . d^2}{c_v}, \qquad \textit{Equation 5-3}$$

where

- T is the settlement time (in s);
- T_v is the time factor, which relates to the degree of consolidation;
- d is the maximum drainage path (in m); and
- c_v is the coefficient of consolidation (in m^2/s).

The time required for 90% consolidation to occur was estimated based on either an open or closed drainage layer where required. The reported settlements were based on a six-month construction period.

5.2.2 Shearing stage

In the test, the deviator stress $(\sigma_1 - \sigma_3)$, measured in kPa, was derived following BS1377-8 (1990):

$$\sigma_1 - \sigma_3 = (\sigma_1 - \sigma_3)_m - \Delta(\sigma_1 - \sigma_3)_{mb}, \qquad \textit{Equation 5-4}$$

where

- $(\sigma_1 - \sigma_3)_m$ is the axial stress in kPa applied by the axial loading cell and
- $(\sigma_1 - \sigma_3)_{mb}$ is the membrane stiffness correction in kPa.

The triaxial shear strain increment (γ, in %) is given by Wood (1990):

$$\gamma = \frac{2(\Delta\varepsilon_a - \Delta\varepsilon_r)}{3}. \qquad \textit{Equation 5-5}$$

In the undrained test, $\varepsilon_{vol} = \varepsilon_a + 2\varepsilon_r = 0$ (Powrie, 1997). Hence, $\varepsilon_r = -\frac{1}{2}\varepsilon_a$, and thus, the shear strain is given by $\gamma = \varepsilon_a$.

5.3 Laboratory test results

Shear strength is considered one of the most important parameters in engineering design. Peat has very low shear strength. The determination of peat's shear strength is challenging due to its high water content, high organic content, and variations in its degree of humification. In addition, sample disturbances can affect the shear strength of peat.

The friction angles obtained from triaxial compression tests on peat are extremely high, typically ranging from 40° to 60°, in contrast to the values generally observed for clay or silt soils, which are usually less than 35°

5.3.1 Consolidation Indexes

Table 5-1 above lists the physical properties of the 14 peat specimens that were subjected to different confining stresses. On average, the water content of the tested peat samples was around 1,100%. At the beginning of each consolidated undrained triaxial test, the specimen was fully saturated to reach a B-value greater than 0.97. After the saturation stage, the specimen was consolidated subject to different confining stresses ranging from 10.4 kPa to 40.5 kPa. The higher the confining stress is, the smaller the diameter and height of the specimens after consolidation. During the final shearing stage, the specimens were slowly sheared at a rate of 0.0118 mm/min in an undrained condition until they reached 40% shear strain.

The consolidation stage usually takes 24 to 48 hours, as this is how long it takes for a peat sample to complete the excess pore water pressure dissipation process due to peat's high compressibility and low permeability.

As shown in Table 5-1 above, the initial void ratios (e_0) ranged from 11.64 to 13.03 – more than 20 times that of classic clay. After the consolidation stage, the diameters of the samples decreased by 10.9% on average, and heights decreased by 11.7%. Those decreases led to significant volume changes occurring during the consolidation stage, ranging from 10% to 35%, with an average of 27.55%. This range displays the wide variety of moisture contents in peat. The moisture content strongly affects shear behaviours such as the friction angle and cohesion.

Figure 5-3 compares the mean effective stress to the volume changes of the samples during the consolidation stage. The compression index (C_c) was determined to be 0.3691, and the coefficient of compressibility m_v was 0.051–0.2 m^2/MN for peat at stress levels ranging from 10.4 kPa to 40.5 kPa. The measured peat compressibility properties in the triaxial tests in this study were similar to oedometer test results from past investigations, which reported that C_c ranged from 0.28 to 6 and m_v ranged from 0.0014 to 25.61 m^2/MN for confining stresses ranging from 5 to 320 kPa (O'Kelly,2005; Johari and Bakar, 2015; Gofar, 2006).

Previous studies have mainly conducted oedometer tests to investigate the compressibility of peat. O'Kelly (2005) tested fine and coarse peat separately and found compression index (C_c) values of 5.8 for fine peat and 4.2–4.6 for coarse peat. Furthermore, O'Kelly (2006) reported C_c values that ranged from 0.28 to 6 in fine to coarse peat samples. Johari and Bakar (2015) tested peat soils with particle sizes ranging from 0.3 mm to 3.35 mm; results showed that the values of C_c ranged from

1.7 to 2.364, and the coefficient of volume compressibility (m_v, m^2/MN) values ranged from 0.012 to 25.613. Gofar (2006) also conducted one-dimensional consolidation tests in both horizontal and vertical directions on peat specimens. Results showed a value of C_c is 3.128 in the vertical direction and 2.879 in the horizontal direction, while the value range of m_v was 0.0014 to 0.00331. However, there is a paucity of triaxial consolidation testing data on peat compressibility to compare to the results of one-dimensional oedometer tests. The compressibility of peat in 3D spaces at different stress levels thus remains to be investigated. Understanding this aspect of peat behaviour is crucial to a wide range of engineering applications.

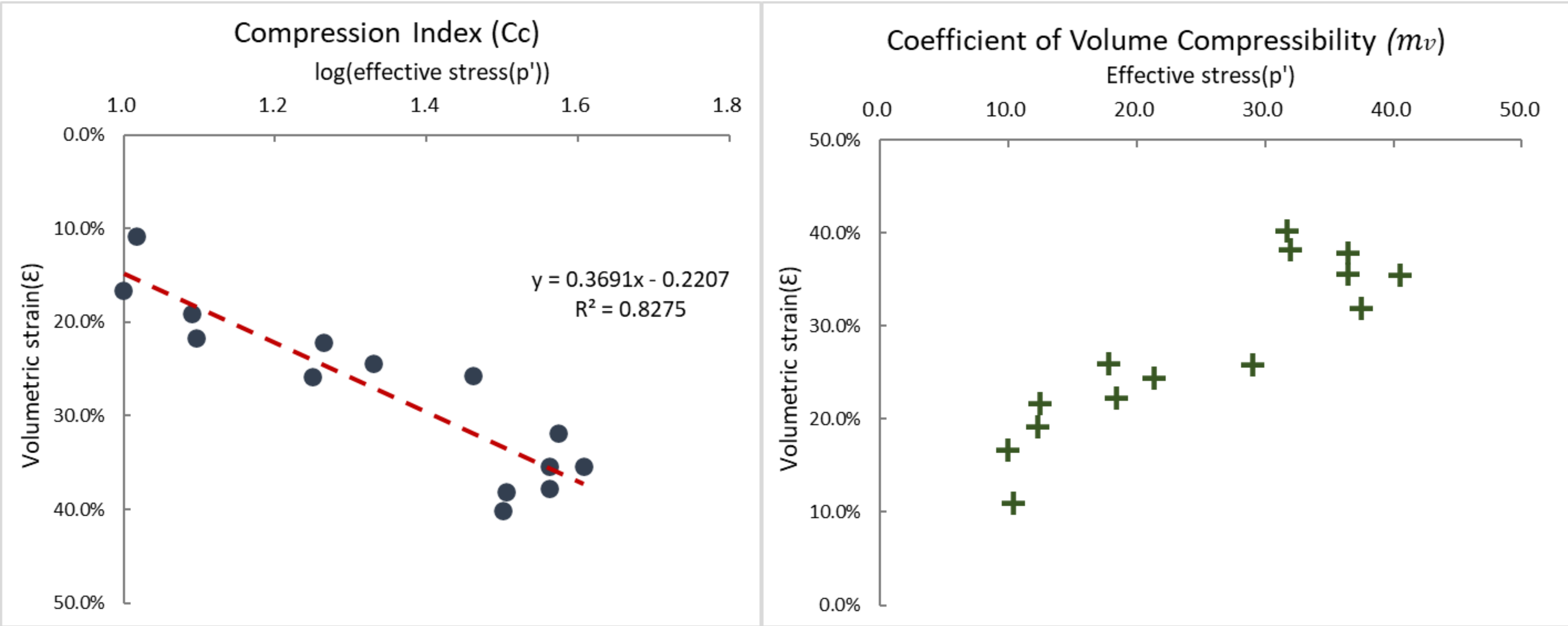


Figure 5-4: Compression index (c_c) and coefficient of volume compressibility (m_v) values

Table 5-3: Details of oedometer tests from previous studies

Compression index C_c	Coefficient of volume compressibility, $\boldsymbol{m_v}$ (m^2/MN)	Confining stress level (kPa)	Test type	Reference
5.8 4.6 4.2	N/A	12.5 to 200	Oedometer test	O'Kelly (2005)
2.9 to 4.7	N/A	25 to 400	Oedometer test	Dhowian and Edil (1980)
0.28 to 6	N/A	12.5 to 200	Oedometer test	O'Kelly (2006)
1.7 to 2.36	25.61 to 0.012	5 to 320	Oedometer test	Johari and Bakar (2015)
2.879 to 3.128	0.0014 to 0.0033	25 to 200	One-dimensional consolidation	Gofar (2006)
0.59 to 6	N/A	12.5 to 200	Oedometer test	O'Kelly (2006)

The coefficient of volume compressibility curves (ε vs. vertical effective stress, p' plots) for the peat samples tested using the triaxial tester in this study are shown in Table 5-3. m_v values ranged from 0.051 to 0.2 m^2/MN and were within the lower range of C_c for peat in the literature. The compressibility curves (ε vs. log of vertical effective stress, p'plots) for the peat samples tested using the triaxial tester in this study are shown in Figure 5-3. The data are presented with a line of best fit. The compression index, C_c, represents the slope of the ε versus $log\ p'$ plot for the incremental triaxial loading test ($C_C = \Delta\varepsilon/\Delta log p'$). In this case, the C_c value for the undisturbed tests was 0.3691, which lies in the lower range of C_c values for peat in the published literature.

It can be seen from the above figure that the value of the compression index (C_c) generally increases linearly with increased effective stress, and the values of C_c are concentrated near the prediction curve without major fluctuations. It can be predicted that the log of effective stress will appear under certain conditions shown in the figure.

For typical mineral soils, the compression index (C_c) is considered constant within the consolidation stage, while in some organic soils, like peat, C_c can increase with increasing effective stress.

Some previous studies have also highlighted this finding. Mesri and Ajlouni (2007) provided detailed insight into the compressibility of peats and noted the increase in C_c with stress due to the collapse of fibrous structures and organic matter decomposition. Huat, Prasad, Asadi, and Kazemian (2014) discussed how peat shows stress-dependent compression behaviour and explained the variation of C_c with increasing σ' as occurring due to peat's fibrous and organic makeup. Landva and LaRochelle (1983) described the unusual compressibility behaviour of organic soils, stating that Cc can increase as stress causes structural collapse. Edil (2003) highlighted the non-linear behaviour of compression curves and the increases in C_c with load, especially in high-organic-content peats.

Although both the C_c and m_v values obtained in this study were plotted within the expected range from the literature presented above, they nonetheless had much lower values than the majority of previously published results. Many of the studies described above used one-dimensional compression tests in which the vertical effective stress typically increased from 10 kPa to 400 kPa, while the vertical effective stress values in this study were only in the range of 10 kPa to 40 kPa. This significant difference in vertical effective stress ranges may explain the lower values of C_c and m_v recorded in this study. O'Kelly (2006) also presented an increase in the C_c value from 0.59 to 6 corresponding with a vertical effective stress increase from 12.5 kPa to 200 kPa, indicating that increasing the vertical effective stress may cause the compression index value to increase.

5.3.2 Deviator stress and shear strain

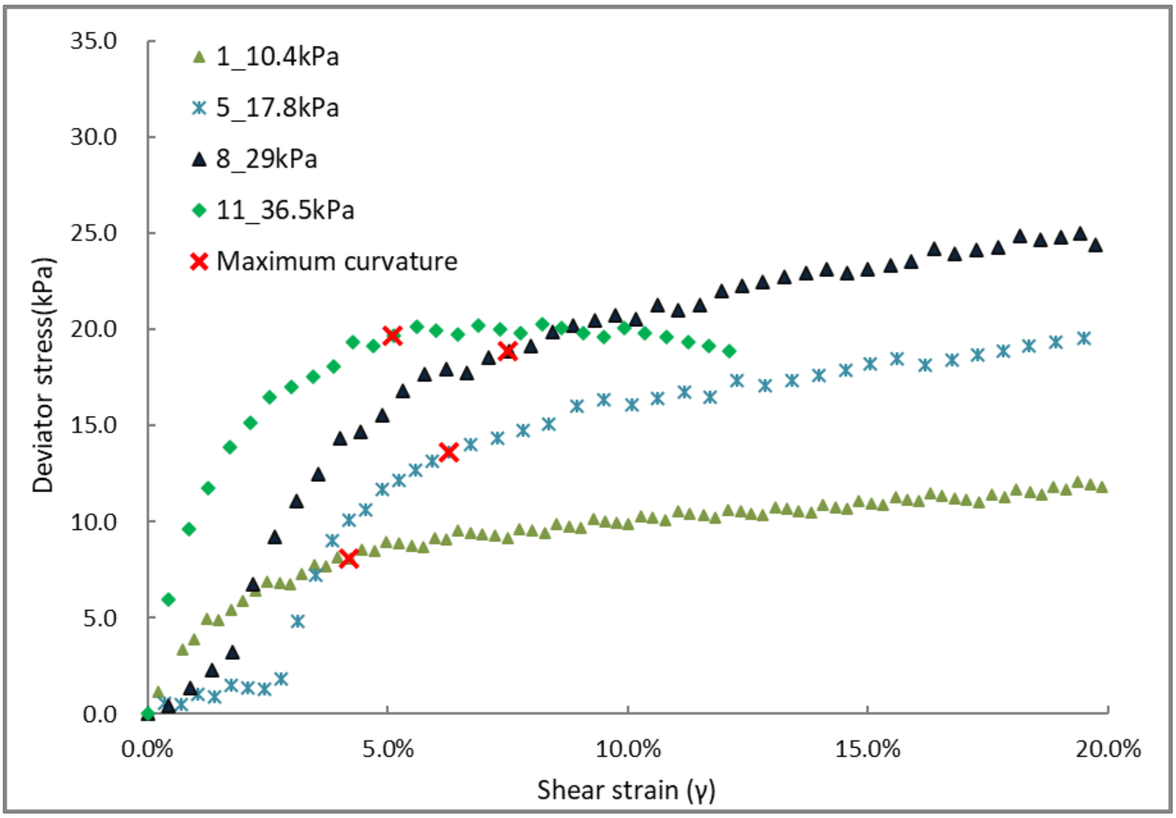
After testing, the membrane of each specimen was opened. The shear behaviour of the peat specimens can be observed in Figure 5-5. Unlike the typical shear failure modes of mineral soils, for instance, shear bands or cracks across the specimens, a peat specimen usually retains its overall morphology without showing evidence of any major shear band failures during testing, as shown in Figure 5-5a. The specimens remained upright with no failure modes even at large shear strains, despite constantly increasing deviator stresses. However, some specimens did show heavy cracks (Figure 5-5b), which were associated with deviator stresses gradually levelling off at high strain values (e.g., specimens 2, 10, and 11; Figure 5-6). This observation was likely due to high moisture content or large fibres in the peat causing inhomogeneous consolidation of the specimens.

(a) Specimen that experienced increasing deviator stress to the end of the test

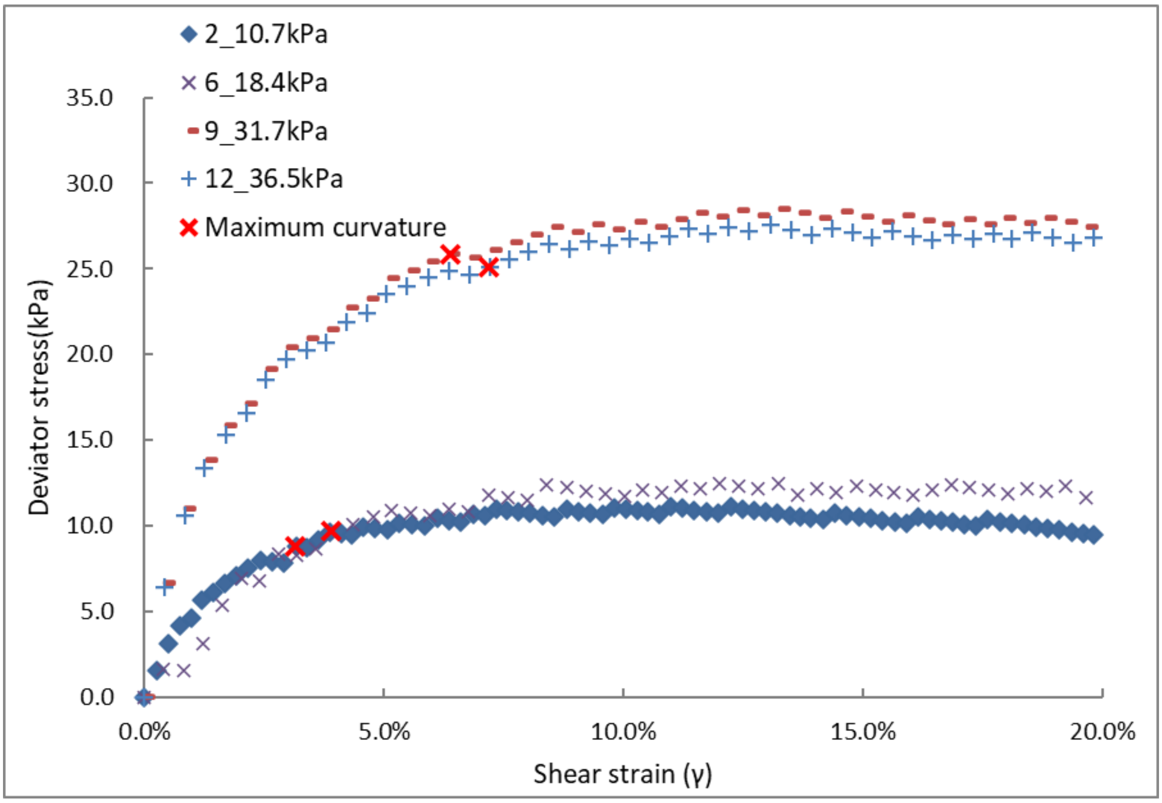
(b) Specimen that experienced deviator stress exhibiting a peak value at 40% shear strain

Figure 5-5: Photos of specimens after tests showing different shear failure modes

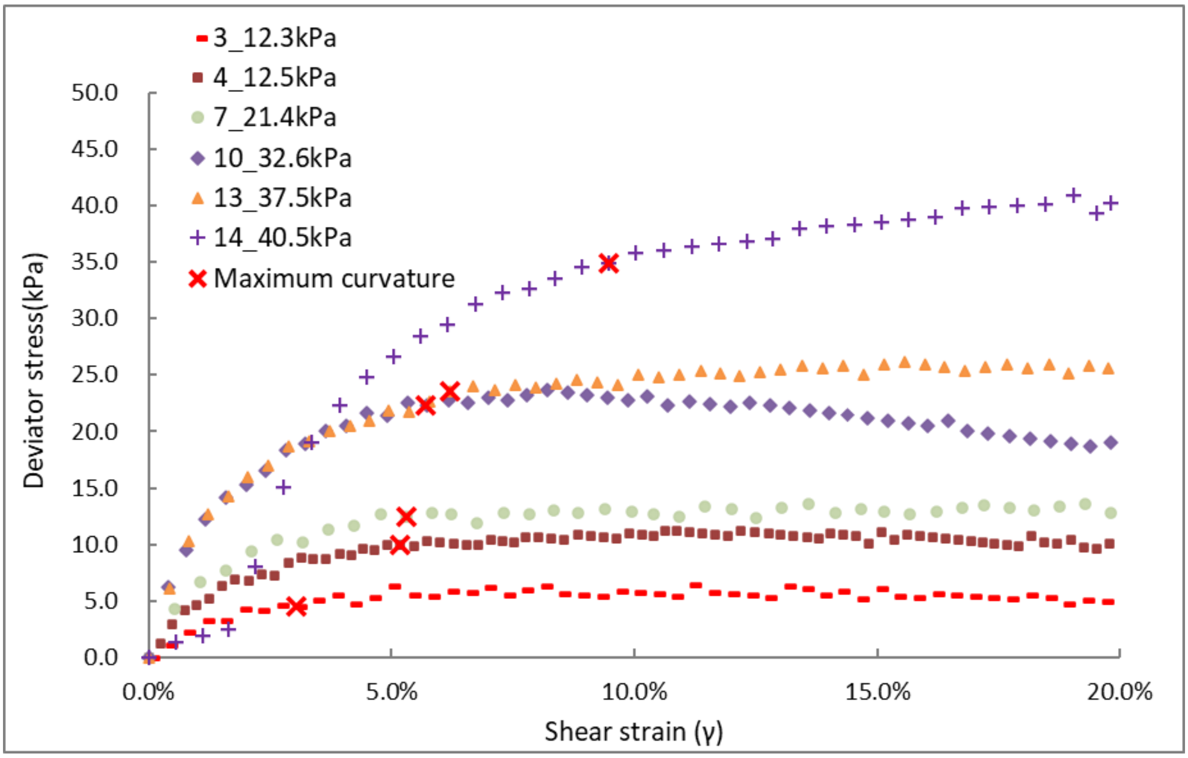
Figure 5-6 shows the development of corrected deviator stress with increasing shear strain subject to different confining stresses. For better clarity, the stress-strain relationship curves from the 14 triaxial tests are shown in three subfigures. In most of the triaxial tests, the deviator stress first built up rapidly with increasing shear strain and then gradually levelled off or changed slightly after 10% shear strain. In the tests of specimens 1, 5, and 14, the deviator stresses keep increasing at high shear strains over 20%, probably due to the existence of some large fibres inside the three specimens.



(a) Group 1



(b) Group 2



(c) Group 3

Figure 5-6*:* The deviator stresses as corrected by Equation 4-2

All of the curves in Figure 5-5 show strain-hardening behaviour, with the stress gradually increasing with strain. Specimens under higher confining stress reached higher deviator stresses, and specimens under lower confining stress showed earlier peaking or plateauing, consistent with weaker behaviour at lower confining pressures. The data can be fit to a Mohr-Coulomb failure envelope using the maximum curvature points from different confining pressures to determine the effective friction angle (φ') and cohesion (c').

Peat shows significant differences from most other inorganic soils, which typically lack internal microstructure and fibre. Laboratory testing methods used to determine the strength properties of peat are generally the same as those used for mineral soil; however, the validity of determining strength parameters such as c' and φ' (cohesion and effective angle of shearing resistance) using these approaches is questionable. The actual failure of peat in triaxial tests is also different from that of other soil types under large strains due to peat's high degree of deformation. Thus, shear strain values not in excess of 20% are recommended to determine the failure of peat.

A triaxial tester can provide accurate control over the stress and pore water pressure. This allows other strength parameters to be determined using the following method. For each triaxial test, the maximum curvature point of the stress-strain curve is chosen as the first point at which the specimen shows a linear strain-hardening response (Silva and Lima, 2017). In general, the greater the confining stress, the larger the shear strain at which the maximum curvature point appears. For example, the maximum curvature point of test 12, which had a large confining stress value of 36.5 kPa, occurred at a shear strain of 7.2%, whereas the shear strain of the maximum curvature point in test 2, which had a low confining stress value of 10.7 kPa, was 3.16%. Thus, higher confining stress on a peat specimen will enhance the material's elasticity under greater yield stress and strain.

In previous studies, the deviator stress in most tests showed fairly slow changes after 10% shear strain; in some cases, a peak value was reached, while in others the deviator stress continued increasing until the test ended. As noted above, in this study, the first point showing a linear strain-hardening response was taken as the maximum curvature point. The maximum curvature points from Figure 5-6 are plotted in terms of the mean effective stress versus deviator stress in Figure 5-7. Figure 5-7 shows the change in the deviatoric stress q with the mean effective stress p' for all of the triaxial tests.

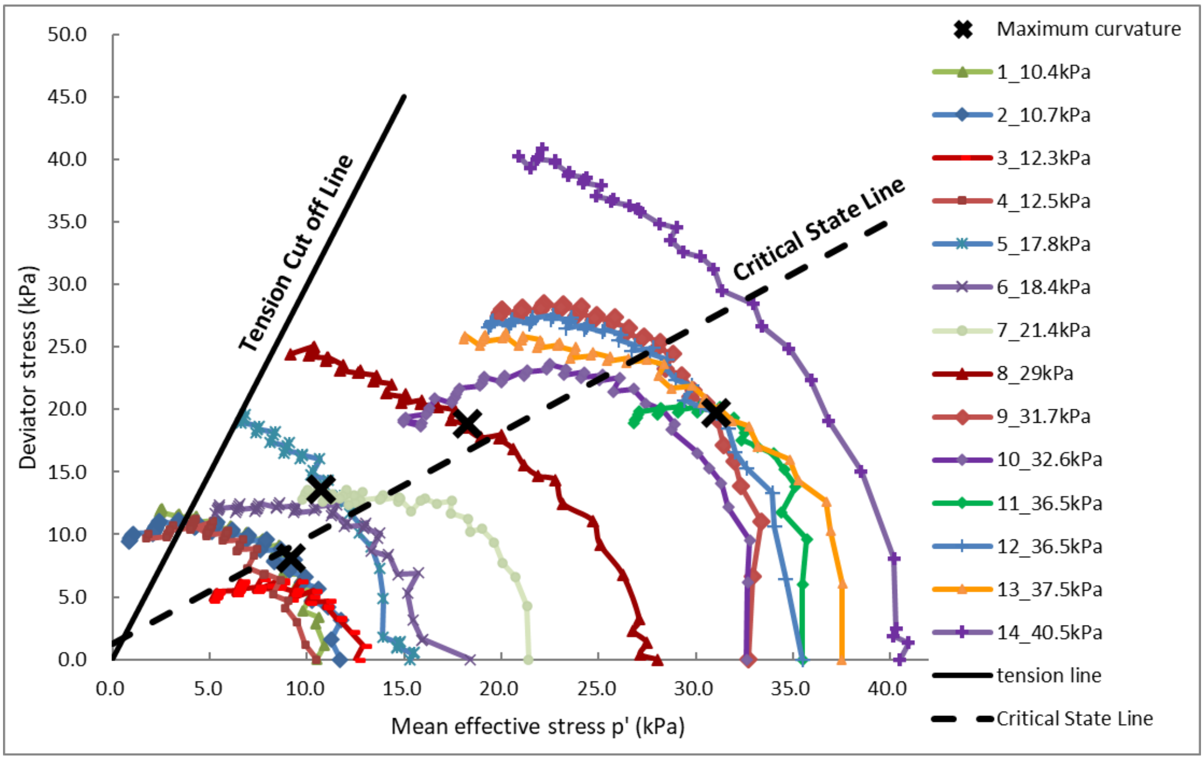


Figure 5-7: Deviator stress q versus mean effective stress p'

In the stress path plots (p'-q), the lower effective stress tests of around 10–40 kPa exhibit contractive behaviour with a gradual decrease in p' to the end of shearing. Most of the samples (10 out of 14) failed without reaching the tension cut-off line (q/p' = 3).

The tension cut-off line in the stress path plot (p'-q) denotes the condition under which the cell pressure becomes zero due to the increase in the excess pore water pressure of the sample. The stress path plots for the samples all fall below the tension cut-off line at an axial strain of 20%. The stress path lines that do not reach the tension cut-off line may indicate that the deviator stress is more strongly related to interparticle connections in these samples, rather than the tension of fibres and wood in the peaty soils.

The critical state (or critical void ratio) line is the locus of void ratio-effective stress conditions achieved after shearing a soil to large displacement and after all net void ratio changes and effective stress changes are complete (Sadrekarimi and Olson, 2009).

The tension cut-off, a phenomenon that occurs when the pore water pressure equals the cell pressure, reduces the effective stress of the peat sample to zero (Hendry, 2012).

Most previous triaxial tests found that peat does not show shear failure modes but more often fails due to the tension cut-off, where the pore water pressure equals the cell pressure while the effective stress of the peat sample is reduced to zero (Hendry, 2012). Zwanenburg (2015) summarised the deviator stress(q) to axial strain (ε) relationship based on data from Yamaguchi (1985), Den Haan and Kruse (2007), and Zwanenburg (2012), and noted that shear strength develops up to the tension cut-off line, which is also the σ'_3 = 0 plane. Landva and LaRochelle (1983) suggested that the tension cut-off line in the triaxial tests was due to the horizontal resistance induced by fibres, which have often been assumed to be mainly horizontally aligned.

The determined critical state of peat is represented by the critical state line in the p'-q plot (Figure 5-7). The critical state line is the linear best-fit line obtained from the values of deviatoric stress at the maximum curvature point and the corresponding mean effective stress for each triaxial test, although the deviatoric stress-shear strain relationship did not result in failure until the end of the tests. In particular, the maximum curvature point in each test, as determined from Figure 5-6, is marked with a cross in this figure.

In Figure 5-6, the stress path plots pass the critical state line and continue along the trends instead of turning to follow the critical state line. This was likely due to the fibre tensile strength in peat. Hendry (2011) stated that horizontal orientation of the fibres in peat provides additional shearing resistance and an elastic stiffness that is cross-anisotropic (Yamaguchi 1985; Mesri and Ajlouni 2007). Landva and Pheeney (1980) described the cross-anisotropic fibrous fabric of peat as one that readily comes apart in a vertical direction but offers considerable resistance to tension in the horizontal direction. The reinforcing effect of these fibres is due to tension generated in the fibres during expansive (tensile) strains in the horizontal plane. Thus, normal compression of peat in the vertical direction results in the fibres experiencing tension and providing reinforcement. No such mobilisation of tension occurs during the compression of peat in the horizontal direction. Consequently, the load–deformation response of peat is a function of the orientation of the principal stresses with respect to the horizontal orientation of fibres, with the stiffest response observed for cases in which the major principal stress is perpendicular to the predominant orientation of fibres. The results of the laboratory testing of peat presented by Yamaguchi (1985) confirmed such anisotropic load–deformation behaviour. For vertically oriented peat specimens (that is, those with a horizontal orientation of fibres), Yamaguchi (1985) reported a friction angle ranging between 51° and 55° for vertically oriented specimens, whereas a significantly lower value of approximately 35° was reported for horizontally oriented specimens. In contrast, the work of Muraro (2019) and Zwanenburg and Jardine (2015) demonstrated that when fibrous peat is tested, the deviatoric stress exhibits transitional behaviour, reflecting the combined response of the soil matrix and the reinforcing effect of the embedded fibres. This effect appears in the form of a convex inflexion towards the critical state line, which causes the stress path to exhibit a 'hook' trajectory as the material perceives higher resistance with increasing load.

Farrell and Hebib (1988) investigated the mechanical behaviour of fibric peat and found that specimens in drained triaxial compression tests deformed almost one-dimensionally under loading without achieving failure, as defined by peak deviatoric stress by 35% axial strain. In undrained triaxial compression tests, the pore water pressure may rapidly build up, almost reaching the applied cell pressure. This produces an effective lateral stress that is approximately equal to zero on account of the low Poisson's ratio of fibric peat.

The friction angle and cohesion are two important parameters for determining a soil's strength. When determining these values, M_{cu} = 0.707 and K_{cu} = 2 kPa are the slope and y-intercept, respectively. Comparing this study's results with the data presented by Hendry (2012), the M_{cu} values of the specimens cut from the block in this study were 1.5 times smaller than the remoulded specimens, while the values of $K_{cu}$for the specimens cut from the block in this study were higher than those of the remoulded specimens. Based on the slope M_{cu} and Equation 5-7 below, the friction angle can be determined as 18.43°.

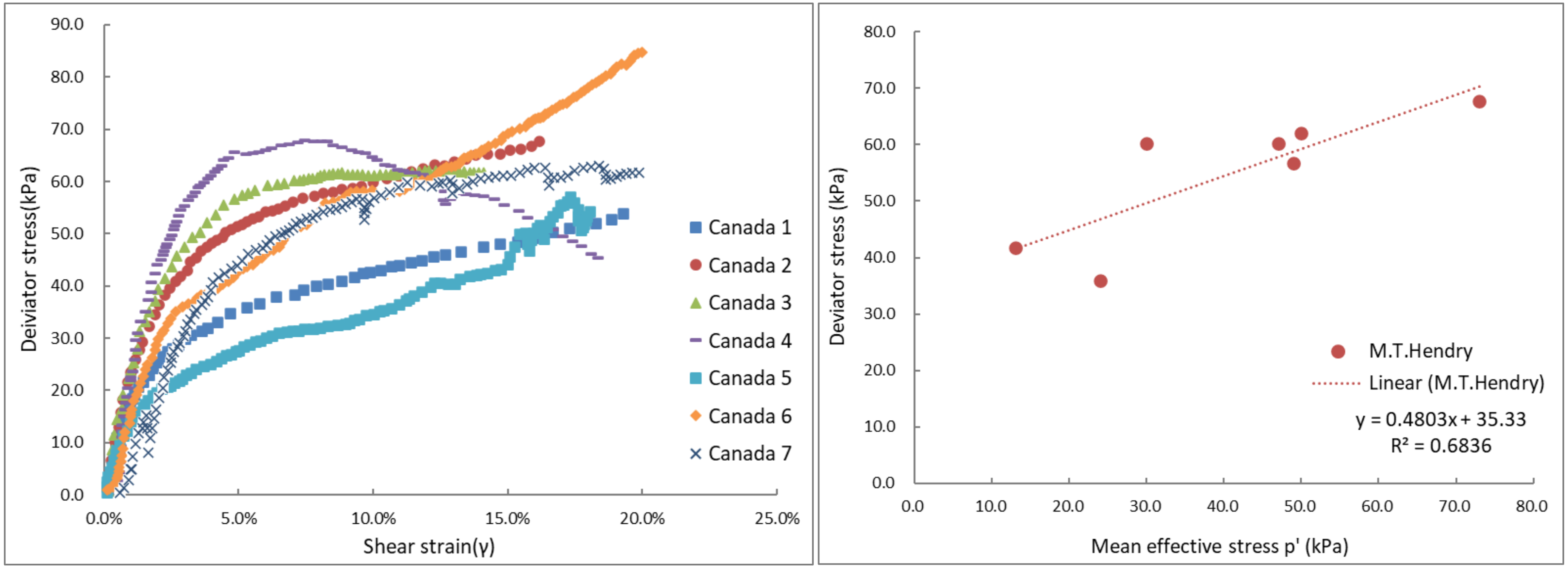
$$M_{CU} = \frac{6sin\varphi}{3-sin\varphi} \qquad \textit{Equation 5-7}$$

The peat from this study's tests had a relatively low friction angle compared with other natural peat. This difference is probably caused by the different percentages of fibres in peat and its high compressibility.

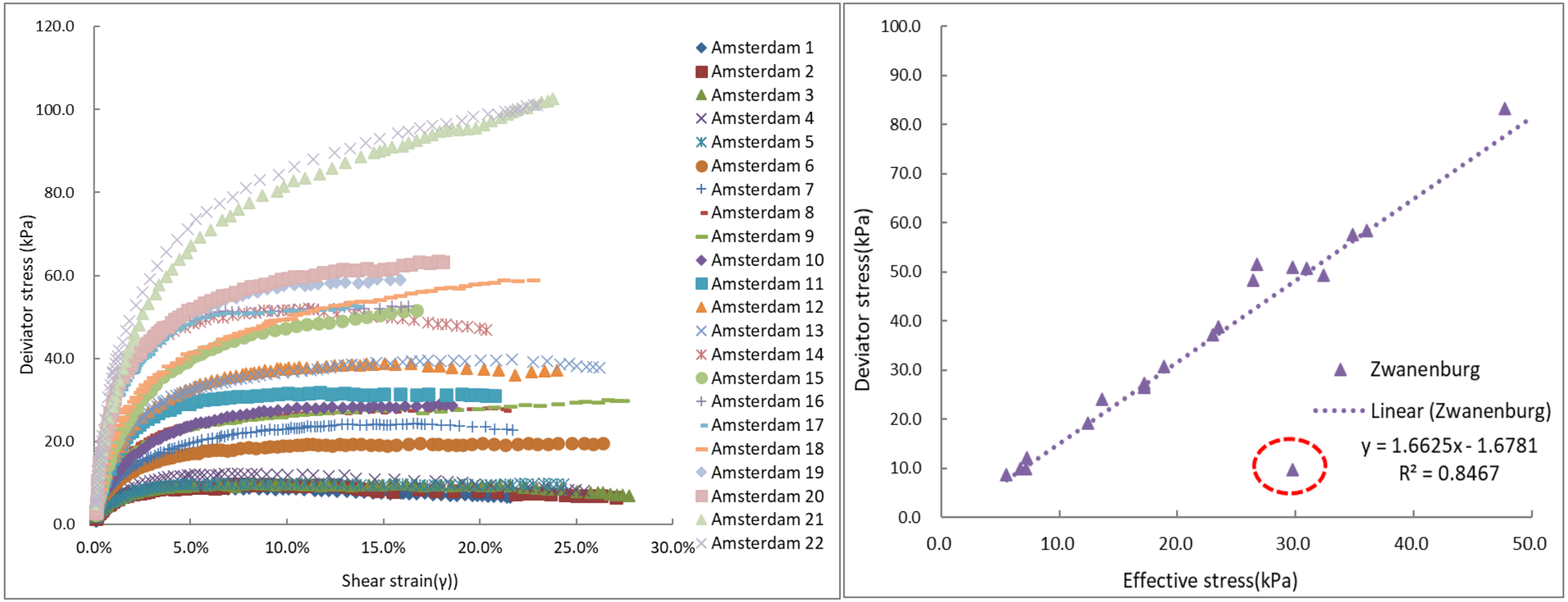
Previous investigations have reported that peat has a high friction angle (ϕ') and the cohesion tends to be zero in the triaxial tests conducted in undrained conditions (Hollingshead and Raymond, 1972; Marachi, 1983; O'Kelly and Zhang, 2013).

5.3.3 Triaxial test data from the existing literature

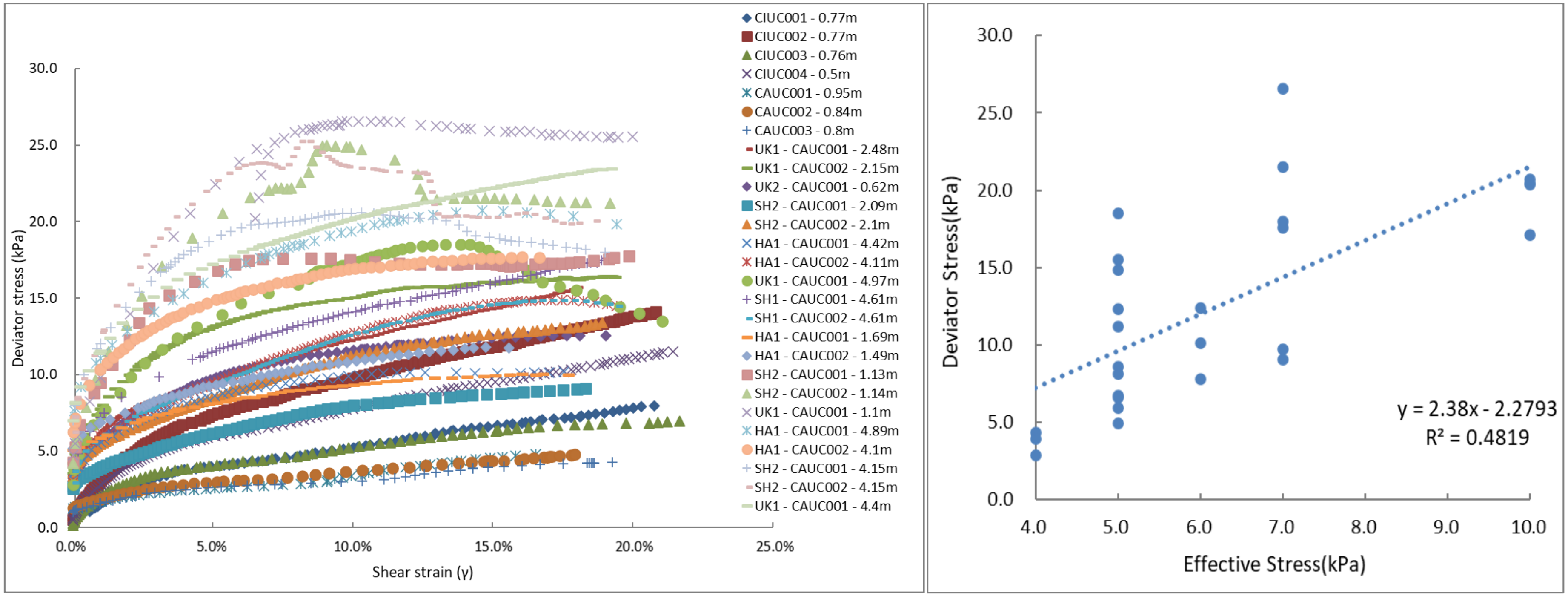
Although several authors have performed similar triaxial tests to investigate the shear behaviour of peat, most did not consider the effect of membrane stiffness on peat behaviour in triaxial tests. Figure 5-8 summarises the effective stress and deviator stress collected from previous studies with Equation 4-2 applied to the deviator stress to minimise the potential effect of membrane stiffness.



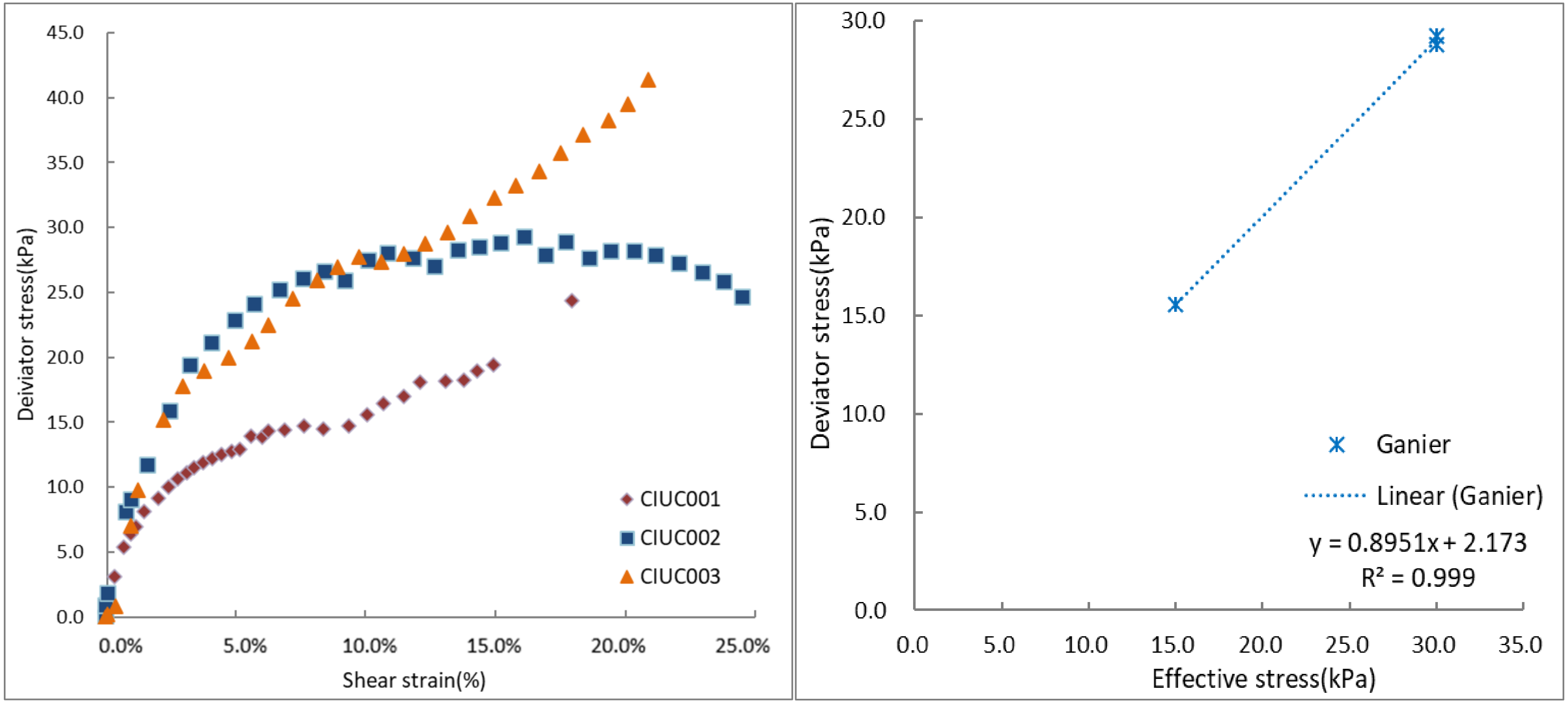
(a) Data collected from Hendry (2012)



(b) Data collected from Zwanenburg (2015)



(c) Data collected from Boylan (2008)*



(d) Data collected from Ganier (2008)

Figure 5-8: The deviator stress q versus the mean effective stress p' from previous studies

*Note: The test data reported by Boylan (2008) includes corrections for membrane effects applied by the author of the present study.

Figure 5-8 summarises the results of consolidated-undrained triaxial tests on peat specimens from previous studies. The results are presented as the stress paths for deviatoric stress (q) and effective stress (p′). From this figure, even though a few curves continue increasing, most curves level off after the correction of membrane stiffness, which means that membrane stiffness plays a significant role when analysing the deviator stress and will also affect the subsequent calculations.

Hendry (2012; Figure 5-8a) showed a linear strain-hardening response below 3% shear strain for reconstituted peat samples (fibrous peat of H2 classification) obtained from a field site in Alberta, Canada, with no evidence of approaching failure, even at higher shear strain values (20%) and higher effective stresses (80 kPa). The results of Zwanenburg (2015; Figure 5-8b) showed a similar linear strain response below 3% for peat samples collected near a dyke along Lake Markermeer in the Netherlands. With the membrane stiffness correction applied, most of the deviator stresses levelled off after 5% of shear strain. An unusual result, highlighted by the red dashed circle in Figures 5-8b, was likely an underestimate. Zwanenburg (2015) did not speculate as to the cause of this unique point, however, it was most likely due to a test error. Boylan (2008) tested peat samples at low stresses (effective stress around 5 kPa) from different locations in Ireland and Northern Ireland. Large fluctuations in these values can be observed in Figure 5-8c, with the deviator stress showing a decreasing trend above 8% shear strain until the end of the tests. Garnier (2007; Figure 5-8d) conducted three triaxial tests on peat taken under a railway in France. Even at higher shear strains (up to 25%), the deviator stress still showed an upward trend.

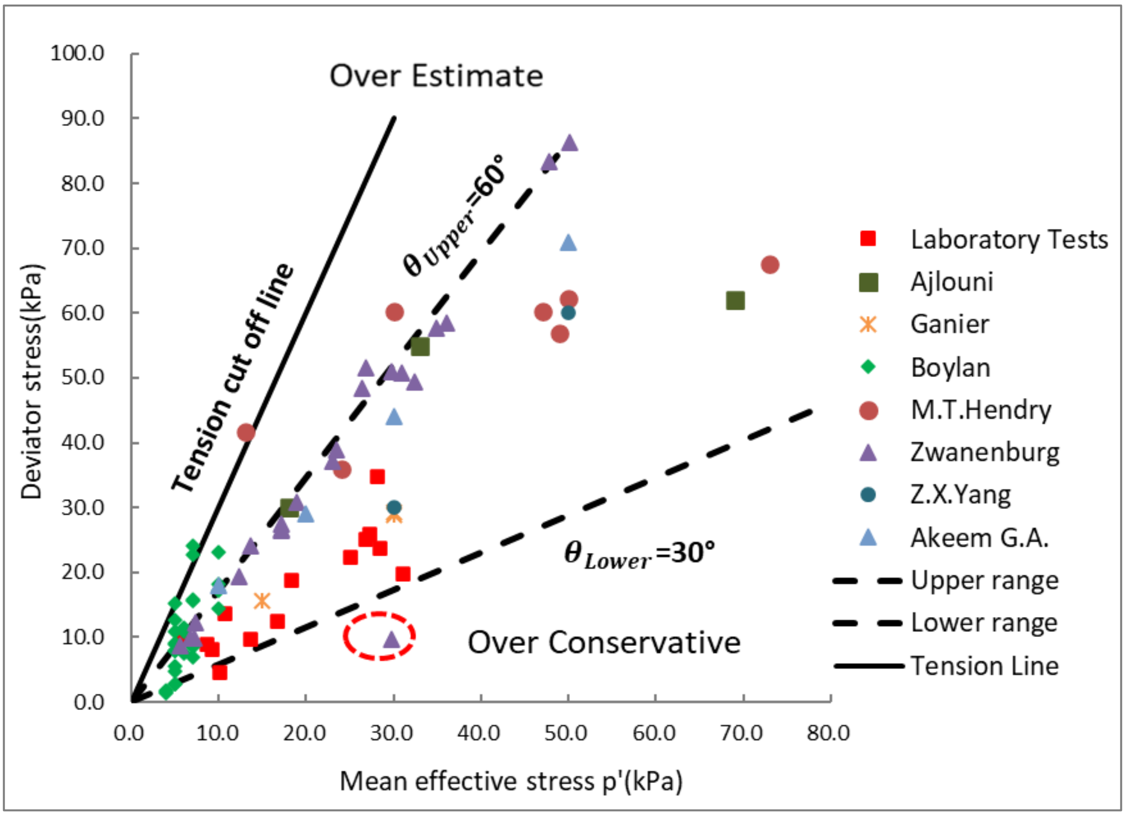
The data from Hendry (2012) showed a high shear strength value (35.33 kPa) corresponding to a low friction angle (12.84°). The data from Garnier (2007) had a similar low friction angle (22.92°), albeit with a low cohesion (2.173 kPa). The other two studies (Figures 5-7c and 5-7d) showed relatively high friction angles (40° to 63°) with low cohesion.

For peat, based on the figures presented above, with increasing effective stress, the effect of cohesion can be ignored as the friction angle plays a relatively more important role in the shear resistance of peaty soil. Due to the high degree of decomposition and high levels of organic content in peat, cohesion is not only related to soil particles but also fibre and wood particles, which are difficult to constrain. As such, it is challenging to represent shear strength through cohesion. The frictional behaviours of peaty soils are complicated and show such variability that the Mohr-Coulomb model is most likely not suitable for assessing strength. Only considering the relationship between shear strength and effective stress would potentially be a better way to evaluate peaty soils.

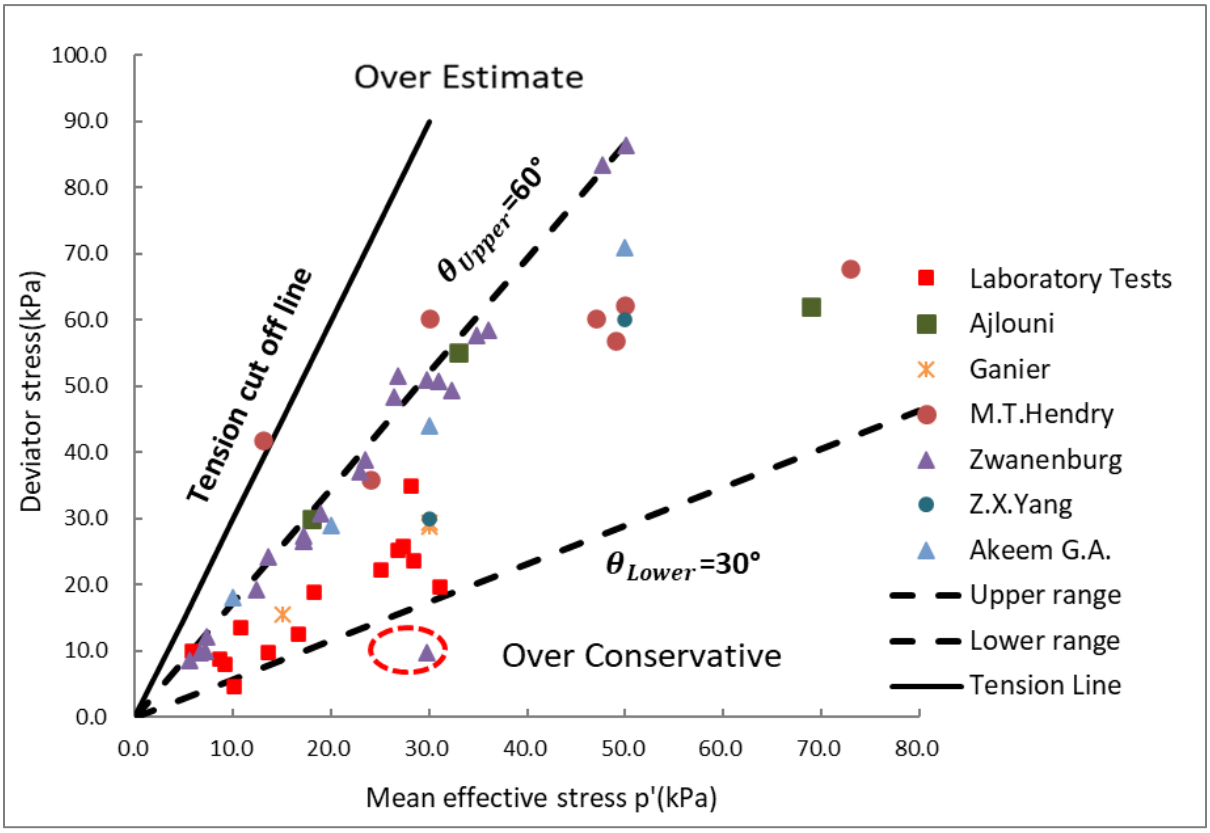
Some previous studies (e.g., Zwanenburg, 2015) did not present the von Post classification of the samples under study, or corresponding depths and moisture contents. Thus, with the currently available information, it is challenging to interpret the common shear behaviours of peat. Further investigation is required in this area.

The determined critical state of peat is also indicated by the critical state line in the p'-q stress space, as shown in Figure 5-9a. The results of the various studies summarised above appear very different in terms of the stress-strain relationships, and the resulting friction angles are also very different from the M_{cu} values. However, when all the data presented in Figure 5-8, in addition to the laboratory test data from this study, are summarised into one graph (Figure 5-9a), it is found that, with the exception of a few abnormal data fluctuations, most of the data fall within a similar range. Regarding the data points for the 10 to 80 kPa effective stress tests in the p'-q stress space, nearly all of the samples failed to reach the tension cut-off line (q/p' = 3).

Specifically, 78% (69 of 88) of the samples fell within the range of 30° to 60°. The points outside this range, with overestimates of deviator stress, were mainly those at lower stress levels (0–10 kPa) from Boylan (2008). Considering the observed high fluctuations in deviator stress at low peat strengths, this percentage increased to 90.1% when points lower than 10 kPa were disregarded (Figure 5-9b). In addition, the reliability of previous test data at such low stress levels (0–10 kPa) seems questionable due to limitations of conventional triaxial testing apparatuses, specimen preparation, and so on. In Figure 5-8, most of the stress path lines that do not reach the tension cut-off line can also be considered proof that the deviator stress may be more representative of interparticle connections than the tension of fibres and wood in peaty soils.



(a) Summary of test data from previous studies with low stress levels



(b) Summary of test data from previous studies without low stress levels

Figure 5-9: Summary of test data from previous studies

5.4 Summary

In this chapter, a series of consolidated undrained triaxial tests were conducted to evaluate the deviator stress values in peat under different stress levels. The main conclusions derived from the numerical results are as follows.

1. The findings from previous studies have generally yielded a wide range of values at stress levels from 5 to 400 kPa in oedometer tests, with values of the compression index C_c ranging from 0.28 to 6 and values of the coefficient of compressibility m_v ranging from 0.0014 to 25.61 m^2/MN. The consolidation data from triaxial tests in this study determined C_c to be 0.3691 and $m_v$to be 0.0072 m^2/MN for peat at low stress levels from 10 kPa

to 40 kPa, which shows a high degree of agreement with the value ranges from the previous studies quoted above that specifically investigated peat compressibility at low stress levels.

2. For soft peat, the membrane correction effect on peat shear resistance is strain-dependent. With increasing shear strain values over 10%, the difference between the uncorrected data and the corrected deviator stress can be up to 10 kPa. Meanwhile, the difference may be small when the shear strain is under 10%. The lower the confining stress, the higher the reduction percentage appears to be. This result clearly demonstrates the significance of the membrane stiffness effect and, thus, cannot be ignored, especially at lower stress levels.
3. Most laboratory test data (10 of 14 tests) in this study and previous test data failed to reach the tension cut-off line (q/p' = 3), which indicated that the deviator stress may more represent interparticle connections than the tension of fibres and wood in peaty soils, especially at relatively high stress levels greater than about 15 kPa. A critical state line for peat was determined based on the values of the deviatoric stress at the maximum curvature and corresponding mean effective stress for each triaxial test.
4. Both this study and previous peat test data show that 78% of data for peat fall within the range of 30° to 60° in the p'-q stress space; this percentage may increase to as much as 90.1% when disregarding points measured at values lower than 10 kPa. In the literature, few past studies (e.g., Boylan (2008)) reported laboratory test data for peat shear behaviour at very low stress levels (0–10 kPa). In addition, the reliability of previous test data at such low stress levels seems questionable due to the limitations of

conventional triaxial testing apparatuses, specimen preparation, and so on. To increase test data reliability at very low stress levels, future studies need to specifically investigate peat shear behaviour using dedicated laboratory devices with sophisticated experimental capabilities.

The triaxial tests conducted in this study offer several novel insights compared to previous investigations presented in the literature. While earlier studies (Boylan, 2008; Hendry, 2012; Zwanenburg, 2015; Silva and Lima, 2017) have explored the strength behaviour of peat using standard triaxial procedures, this study incorporates a more refined and systematic approach, using tests that account for anisotropy, scale effects, and stiffness degradation. Moreover, advanced corrections for membrane penetration and stiffness, which are often overlooked or only partially addressed in prior research, have been applied to improve data reliability and accuracy. The triaxial testing in this study also includes a comprehensive analysis of shear strength normalised by effective confining stress and strain-dependent stiffness behaviour ($G\ /G_{0.1\%}$), which is rarely reported in detail for peat soils.

The major contribution to knowledge arising from this study lies in its integrated investigation of peat's mechanical behaviour under varied stress paths using both unconsolidated undrained and consolidated undrained triaxial tests. This includes the development of a framework for interpreting stiffness degradation and strength anisotropy in relation to organic content and fabric. Unlike many prior studies that treat peat as a highly variable and unpredictable material, the findings of this study offer a more predictive and quantifiable basis for interpreting peat behaviour, which has significant implications for geotechnical design in peat-rich regions. These insights help bridge the existing gap between empirical observations and the constitutive modelling of peat, thereby contributing to safer and more reliable engineering practices.

6 INFLUENCES ON PEAT SHEAR STRENGTH

6.1 Abstract

Following the laboratory tests presented in Chapters 4 and 5, this chapter examines the influence of organic content on the strength behaviour of peat through both laboratory triaxial test results and a wide review of the existing literature. Organic matter, which is primarily fibrous in nature, plays a dominant role in determining the mechanical characteristics of peat due to its porous, compressible structure and cementation effects within the soil matrix. Through a comparison of test data with different levels of organic content, the study identifies trends in stress-strain behaviour and highlights the significant influence of organic matter on shear strength development. Triaxial test data, especially under undrained conditions, reveal that specimens with organic contents above 75% usually fail to exhibit peak deviator stresses. Thus, alternative analysis methods such as the maximum curvature approach for evaluating friction angles are required. Furthermore, the chapter provides a critical discussion on the effect of membrane stiffness on measured shear strength to enhance the reliability of deviator stress data. Overall, the chapter contributes to the understanding of how organic content affects the mechanical behaviours of peat and discusses the limitations of conventional models like the Mohr-Coulomb in highly organic soils.

6.2 The organic content effect on shear behaviours

The organic content can be regarded as the fibre content in peat soil. The leading role of the porous sponge structure of organic matter and its cementation in the aggregates of peat soil play significantly controlling roles in both the relative physical and mechanical indexes and mechanical effects of peat.

Li (2020) conducted a series of tests on the shear strength of reconstituted peat specimens with organic contents ranging from 30% to 80%. Based on the strain-stress curves presented in this study, across different confining stress levels and with increasing deviator stress, the relationship between strain and stress shows a clear strain-hardening trend. At low shear strains (< 8%), the stress exhibits nonlinear growth. As the strain keeps increasing, the deviatoric stress increases linearly. As the organic matter content increases, the deviatoric stress also increases correspondingly.

Table 6-1 presents data from previous studies on peat obtained through triaxial testing. Three distinct cases are considered: (1) specimens in which the shear strength exhibited a peak value during testing, (2) specimens that did not reach a peak value or level off, and (3) tests where insufficient information was available.

The three different cases under investigation are those in which 1) the shear strength exhibited a peak value during the testing stage, 2) no peak value or level off was reached, and 3) there was insufficient information.

Based on Table 6-1 below, the organic content of peat can affect its shear strength behaviour. From previous studies, the deviator stress increased when the organic content was below 75%, with peak values reached at shear strains from 5% to 25%. In these cases, the shear strength behaved more like that of a mineral soil, and friction angles could be directly assessed from triaxial tests. However, in these scenarios, it was difficult to predict where the peak shear strain value would be reached. When the organic content exceeded 75%, the deviator stress continued increasing and reached large shear strain values at the end of the tests. When the shear strength of the peat did not fail during tests, the Mohr-Coulomb model had difficulty in determining the friction angle. In this situation, the maximum curvature approach proposed above can be used (Silva and Lima, 2017). One particular point can be determined when a curve reaches its maximum curvature. However, as the curves for peat usually continue to increase, it is unlikely that a point of zero gradient will be reached.

For the tests where peak values were not reached, the friction angles (φ') were in the range of 12° to 33°. The high friction angle of around 63° from the study by Boylan (2008) was likely due to the very low effective stress. Similar results have been obtained by Coutinho and Lacerda (1989) on organic Brazilian soils, who found φ' values increasing up to 57° in soil with organic contents up to 60%. Mitachi and Fujiwara (1987) experimented on mixes of clay and organic matter and also found that higher organic content (up to 57% was investigated) correlated with higher φ' values (up to 52°).

Table 6-1: Details of triaxial tests from previous studies

Friction angle, φ' (°)	Organic content (%)	Reached peak value?	Shear strain of maximum curvature or peak value determined	Method to determine the friction angle	Reference
Peat with over 75% organic content					
25.69	90–96	No	8–22%	Maximum curvature method	Ajlouni (2000)
22.92	76	No	8%	Maximum curvature method	Garnier (2007)
62.99 Note 1	96	No	8%	Maximum curvature method	Boylan (2008)
12.84	82	No	5–10%	Maximum curvature method	Hendry (2012)
40.61 Note 2	75–92	No	5–10%	Maximum curvature method	Zwanenburg (2015)
33.3	92-96	No	12–15%	Maximum curvature method	Amuda and Hasan (2018)
14.3	96%	No	5–10%	Maximum curvature method	Current study
Peat with below 75% organic content					
51–55	71–73	Yes	8%	Level off	Yamaguchi (1985): Undrained shear characteristics of normally consolidated peat under triaxial compression and extension conditions

35–60	60	Yes	5%	Peak value achieved	den Hann (1997): An overview of the mechanical behaviour of Peats and organic soils and some appropriate construction techniques
49–51	68–75	Yes	13%	Level off	Cola and Cortellazzo (2005): The Shear Strength Behaviour of Two Peaty Soils
47.3	32.4[Note3]	Yes	20%	Level off	Yang and Zhao (2016): Modelling the engineering behaviour of fibrous peat formed due to rapid anthropogenic terrestrialization in Hangzhou, China
43	43	Yes	25%	At 25% of shear strain	Muraro and Jommi (2019): Experimental determination of the shear strength of peat from standard undrained triaxial tests: correcting for the effects of end restraint
Peat triaxial tests with insufficient experimental data					
36.6–43.5	N/A	N/A	N/A	N/A	Hanrahan (1967): Shear strength of peat. In: Proceedings of Geotechnical Conference Oslo
40-50	N/A	N/A	N/A	N/A	Landva and LaRochelle (1983): Compressibility and Shear Characteristics of Radforth Peats
55	80	N/A	N/A	N/A	Farrell and Hebib (1998): The determination of the geotechnical parameters of organic soils
42–66	83–95	N/A	N/A	N/A	Edil and Wang (2000): Shear strength and Ko of peats and organic soils

Note 1: The extremely high friction angle from Boylan (2008) is probably due to the very low effective stress (4–10 kPa).

Note 2: The higher friction angle is probably due to relatively few of the q-Ɛ peaks developing before the end of tests.

Note 3: Organic content was calculated from fibre content = 35%, OC = 100 - C (100 - N) where C = 1.04 (Skempton and Petley, 1970).

6.3 Triaxial test versus direct shear tests

A comparison is carried out below to compare the friction angles φ’ collected from direct shear tests, simple shear tests, and triaxial tests in previous literature. Farrell and Hebib (1998) presented the results of laboratory experiments investigating the shear strength of Irish peat from the Raheenmore bog through direct shear tests, ring shear tests, and triaxial tests. The friction angle of shearing resistance, as measured in undrained triaxial compression tests, was about φ’ = 55°. An effective friction angle of φ’ = 38° was measured in both the direct shear box and the ring shear test, while the direct simple shear test showed a lower value of φ’ = 31°. Further testing was carried out by Hebib (2001) on undisturbed peat samples from Ballydermot bog. The peat was between 94% and 98% organic, with moisture content values between 750% and 950%. The peat samples were tested in both ring shear tests and triaxial tests. Angles of shearing resistance of φ' = 21° and 38° were obtained from the ring shear test. Similar behaviour to that of the Raheenmore peat was observed under undrained triaxial conditions, yielding friction angle results between 55° and 68°.

Cola and Cortellazzo (2005) presented experimental research concerning the shear behaviour of two types of Italian peat through undrained triaxial tests. Results showed friction angles between 49° and 51°. Zainorabidin and Mansor (2016) conducted direct simple shear tests and direct shear box tests to determine the shear strength of peat. The values of c and φ’ for the direct shear box test (around 37°) were higher than those achieved from direct simple shear (around 21°). These results show that direct simple shear testing is more suitable for determining the shear strength of peat. A series of consolidation and direct shear tests were conducted on samples of peat by

Badv and Sayadian (2011), who reported friction angles ranging from 32° to 45° with vertical effective stress values from 8.98 kPa to 26.94 kPa. Two sets of direct shear tests were conducted on peat with 30% and 70% organic content, which showed that with the higher organic content of 70%, peat had a higher friction angle of 45°. Yamaguchi (1985b) showed that the effect of fibre is negligible in peat studies. High friction angles in the range of 50° to 85° have been reported in previous studies on undrained triaxial tests of peat (Den-Haan and Feddema, 2012; Farrell and Hebib, 1998; Hebib, 2001; Yamaguchi, 1985b).

Some papers quote very high friction angles from triaxial tests with no peak values reached; these tests are not suitable for comparison with friction angles determined from the direct shear test method due to the difficulty of determining the friction angle. As noted above, organic content will significantly affect the friction angles. As most triaxial test data do not show peak values or level off, it is recommended to compare the friction angles determined by the maximum curvature approach with those from direct shear tests. Table 6-2 presents friction angles obtained from peat tests in previous studies where no peak value or levelling off in the stress-strain curves were observed.

In general, the direct shear box test gives higher estimates of shear strength than the triaxial shear test (in Table 6-1), as a result of either the shearing mechanism or specimen size, when comparing Table 6-1 and Table 6-2. For direct shear box tests, the fibre in the middle of a sample is likely to affect the entire area of the specimen. Even though the direct shear box approach yields higher values of c and φ’, it is not necessarily accurate – the mechanism of direct shear acts only at the centre of a specimen, while triaxial shear can shear throughout a specimen.

Table 6-2: Details of direct shear tests and direct simple shear tests from previous studies

Friction angle, φ' (°)	Organic content (%)	Reached peak value?	Shear strain of maximum curvature or peak value determined	Method to determine the friction angle	Reference
Peat with over 75% organic content					
30.4	92.5	Level off	22%	Mohr-Coulomb	Den Haan and Grognet (2014)
17.8–39.8	82–89	Peak value reached	20%	Mohr-Coulomb	Saberian and Rahgozar (2016)
46.2 44.9	87.9[Note4] 82.4 [Note4]	Critical state Peak value reached	N/A	Mohr-Coulomb	Lengkeek (2014)
Peat with below 75% organic content					
γ = 10%, φ' = 20 γ = 40%, φ' = 20–30	73	Level off	15%	Mohr-Coulomb	Grognet (2011)
30	42–60	Level off	15%	Mohr-Coulomb	Sarkar and Sadrekarimi (2020)
Peat direct shear tests or direct simple shear tests with insufficient experimental data					

31	N/A	N/A	N/A	N/A	Farrell and Hebib (1998)
34	N/A	N/A	N/A	N/A	Farrell, Jonker, Knibbeler and Brinkgreve (1999)
3–20	79–98	N/A	N/A	N/A	Huat (2004)
33	N/A	N/A	N/A	N/A	McInerney, O'Kelly and Johnston (2006)
33.5 44	30 70	N/A	N/A	N/A	Badv and Sayadian (2011)
38	94–99	N/A	N/A	N/A	Hebib (2011)
22 37	>75	N/A	N/A	N/A	Zainorabidin and Mansor (2016)

Note 4: Organic content was calculated from fibre content = 35%, OC = 100 - C(100 - N) where C = 1.04 (Skempton and Petley, 1970)

6.4 The effect of membrane stiffness correction

Henkel and Gilbert (1952) stated that membrane stiffness E_m may cause increases in deviator stress. Raghunandan (2014) pointed out that the rubber membrane used in tests has a significant influence on both measured deviatoric stress and volume change data in laboratory triaxial tests. Hence, measured test data must be corrected by using a suitable equation to minimise the membrane effect.

Regarding membrane stiffness in triaxial tests, researchers have proposed several equations to correct the effects of the membrane on measurements of deviator stress. Henkel and Gilbert (1952) proposed membrane correction formulas based on two theories: the compression shell and hoop stress theories.

6.4.1 Compression shell theory

Compression shell theory was originally proposed by Henkel and Gilbert (1952) and has been widely applied in the American standard ASTM D 4767-95 *Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.*

Henkel and Gilbert (1952) first investigated the confinement of membranes in bulging type failures by conducting a series of tests on remoulded London clay with three different membrane thicknesses. The strength contributed was proportional to the stiffness of the membrane observed, and not subjected to the cell pressure. However, when only using a membrane to confine samples (i.e., under zero cell pressure), the load caused by the membrane was significantly lower. Based on these observations, Henkel and Gilbert (1952) proposed compression shell theory and hoop stress theory to correct for membrane stiffness.

Compression shell theory assumes that, in undrained or constant volume tests, a sample deforms as a right cylinder under compression stresses. During the testing process, when the cell pressure is high enough, the rubber membrane is firmly pushed against the specimen. Little buckling of the rubber membrane is likely, especially under small strains, and the membrane may be assumed to act as a reinforcing compression shell outside the sample. This is considered to contribute to the shear strength of specimens.

Given this membrane stiffness effect, Equation 6-1 below was adopted to correct the deviator stress as follows:

$$\Delta q_m = \Delta(\sigma_1 - \sigma_3)_{mb} = \frac{4E_m t_m \Upsilon}{D_c}, \qquad \textit{Equation 6-1}$$

where

- D_c is the diameter of specimens after the consolidation stage, and $D_C = \sqrt{4A_C\pi}$ (in mm);
- A_C is the post-consolidation average cross-sectional area of the sample (in mm^2);
- t_m is the total thickness of the membrane enclosing the specimen (in mm); and
- E_m is Young's modulus for the membrane material (in kPa).

6.4.2 Hoop stress theory

In opposition to compression shell theory, Henkel and Gilbert (1952) argued that if the rubber membrane is not held firmly against the specimen, buckling could be caused by the hoop tension associated with the circumferential strains on the specimen during deformation. It is recognised that the actual mode of operation of the rubber involves a complex combination of these two effects, and that the specimen does not deform like an upright cylinder. Nevertheless, an approximate theoretical argument can be made. In the case of triaxial tests, the rubber membrane may be assumed to act as a reinforcing shell, and in 'rubber only' tests, the result may be assumed to be affected by hoop tension.

Regarding hoop tension, according to Henkel and Gilbert (1952), the principal stress difference correction for the restraining effect of the membrane is given by the following equation:

$$\Delta q_m = \Delta(\sigma_1 - \sigma_3)_m = \frac{2E_m t_m}{D_0} . \frac{(1-\sqrt{1-\varepsilon_a})}{1-\varepsilon_a} \approx \frac{E_m t_m \, \varepsilon_a}{D_0}, \quad \textit{Equation 6-2}$$

where D_0 is the initial specimen diameter.

6.4.3 Modified compression shell theory

In order to measure the relatively low stress caused by the membrane in triaxial tests, gelatine specimens were prepared by Greeuw (2001) to stimulate soft samples of low stiffness. Greeuw (2001) stated that about 7 kPa deviator stress is needed for 20% compression of the gelatine itself, and about 1 kPa can be attributed to the membrane.

Greeuw (2001) proposed a bilinear formulation to correct the membrane stiffness. This is shown in the equation below:

$$\Delta q_m = \alpha \frac{4Et}{D} \varepsilon_a \qquad \varepsilon_a < 0.07$$

$$\Delta q_m = \alpha \frac{0.28Et}{D} + \beta \frac{4Et}{D} (\varepsilon_a - 0.07) \qquad \varepsilon_a < 0.07, \qquad \textit{Equation 6-3}$$

where

Δq_m is the membrane correction to be subtracted from measured deviator stress (in kPa), ε_a is the axial strain (in %), and α and β are coefficients. The curve fitting parameters are $\alpha \approx 0.38 \pm 0.19$ and $\beta \approx 0.19 \pm 0.08$.

Following Greeuw (2001), Boylan (2008) also conducted three gelatine specimen triaxial tests with three different types of membranes, as well as two dummy specimens without any membranes to verify whether the method of Greeuw (2001) is appropriate for correcting for membrane resistance and providing guidance on the selection of curve fitting parameters.

In the research conducted by Greeuw (2001), only two test samples were utilised to investigate the membrane effects. This limited number of experimental data points may suggest a narrow scope of study or a specific focus on particular conditions or materials.

Additionally, regarding the equations proposed by Greeuw (2001), there appears to be a lack of clear explanation or justification for selection or derivation of the curve fitting parameters. Specifically, the use of a value of 0.07 for axial strain in a bi-linear formulation raises questions about why this specific value was chosen and how the coefficients were obtained. This lack of transparency regarding the methodology and rationale behind equation parameters may undermine the robustness and interpretability of the findings.

6.4.4 Modified hoop stress theory

Konstadinou and Zwanenburg (2020) conducted a series of triaxial tests on clay specimens. They proposed that the membrane stiffness correction given by compression shell theory be adopted when $\varepsilon_a \leq 4\%$, and at larger strains of $4\% \leq \varepsilon_a \leq 20\%$,

$$\Delta(\sigma_1 - \sigma_3)_m = Eq(6-1)_{\varepsilon_a=0.04} + \frac{E_m t_m(\varepsilon_a - 0.04)}{D_0}. \qquad \textit{Equation 6-4}$$

At $\varepsilon_a \geq 20\%$, the membrane stiffness contribution is negligible and remains constant with the strain.

6.4.5 Comparison

The thickness and type of membrane material determine the value of E_m. Thus, it is essential to determine the thickness and stiffness of the membrane used before starting any correction procedure.

A thickness t_m of 0.2 mm and a Young's modulus E_m of 1,400 kPa was taken from the membrane manufacturer's manual to compare the membrane correction theories in this study. The initial diameter of the peat specimens was 50 mm.

Figure 6-1 below indicates that both the compression shell theory and hoop stress theory corrections increased at a constant rate with shear strain. In comparison, the equations proposed by Greeuw (2001) and Konstatinou and Zwanenburg (2020) increased slowly after 4% and 7% shear strain, respectively. The membrane correction values within 10% of shear strain did not change significantly.

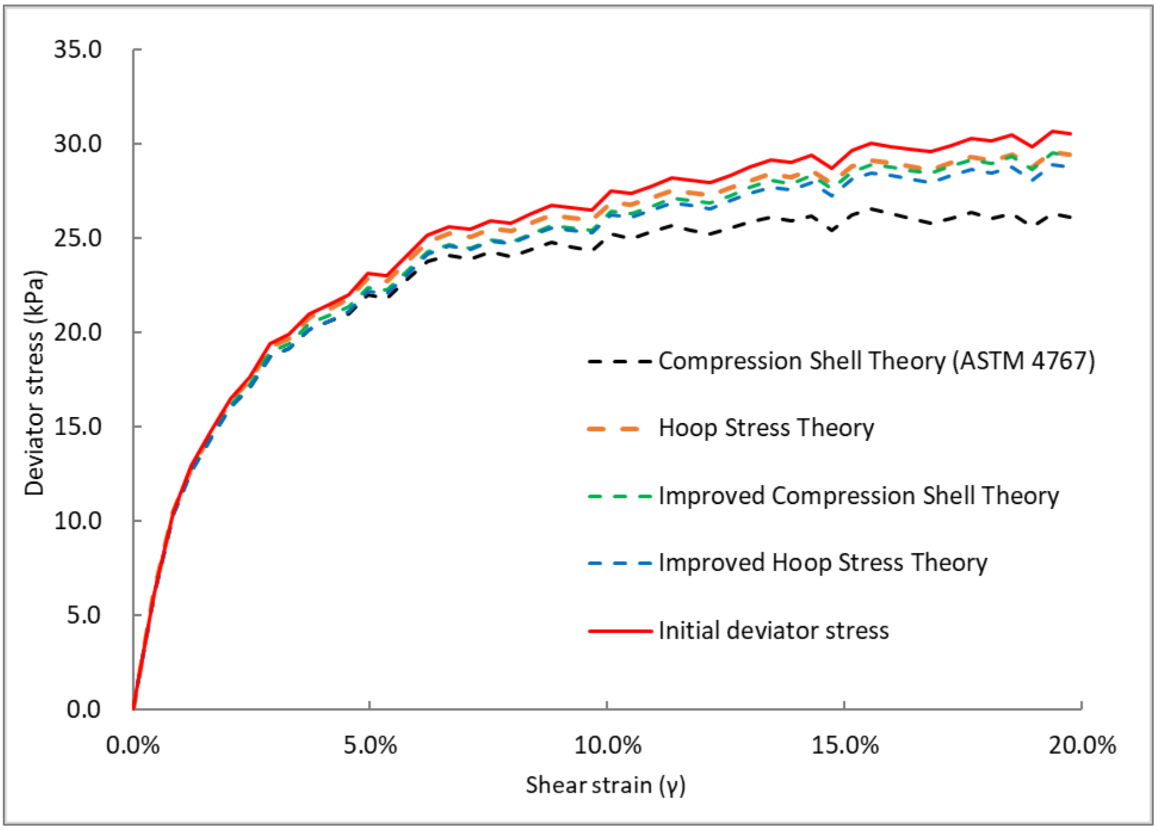


Figure 6-1: Comparison of the proposed methods for membrane correction

Both Greeuw (2002), Boylan (2008) and Konstatinou and Zwanenburg (2020) stated that using compression shell theory can lead to significant overcorrection, especially for soft soils. Given the situation in this study, compression shell theory was adopted for the following reasons.

1. As discussed in Chapter 4.4.3 regarding the shear deformation model of peat, in the undrained condition, peat specimens usually display bulging failure. No failures along a shear plane occurred in this research. For soft samples with low effective stress, no buckling is likely to occur, so the membrane is assumed to act as a compression shell reinforcing the specimen. In this case, the compression shell theory should be adopted.
2. Konstatinou and Zwanenburg (2020) stated that for specimens exhibiting a bulging mode of failure and at axial strains of less than 20%, compression shell

theory can closely predict the actual deviator stress caused by the rubber membrane. The membrane stiffness correction from ASTM, which was used in this study, is mainly intended for axial strains ε_a smaller than 10%.

3. Regarding the equations proposed by Greeuw (2002) and Konstatinou and Zwanenburg (2020), further investigation needs to be conducted to verify the accuracy.

It should be highlighted that compression shell theory was chosen based on undrained conditions. It is assumed that the buckling tendency of the membrane would be counteracted by its confining force in this study. Future work could investigate how to choose the proper equation to make membrane stiffness corrections based on specific situations.

Using Equation 6-1 above, the measured deviatoric stress was corrected at each shear strain value. However, the correction can be ignored when the shear strain is less than 5%.

Figure 6-2 below compares the original uncorrected deviator stress values against the ones corrected by Equation 6-1, subject to different confining stress levels. In the legend of this figure, '*' indicates the corrected deviator stress, whereas the curves without '*' are original, uncorrected data. In the figure, there is little difference between corrected and uncorrected deviator stresses when the shear strain is below 10%. With increasing shear strain over 10%, the uncorrected data increases gradually, whereas the corrected deviator stress reaches a peak value at 10% of shear strain before slightly decreasing or levelling off. This indicates that the membrane stiffness contribution becomes significant after 10% of shear strain in triaxial compression tests.

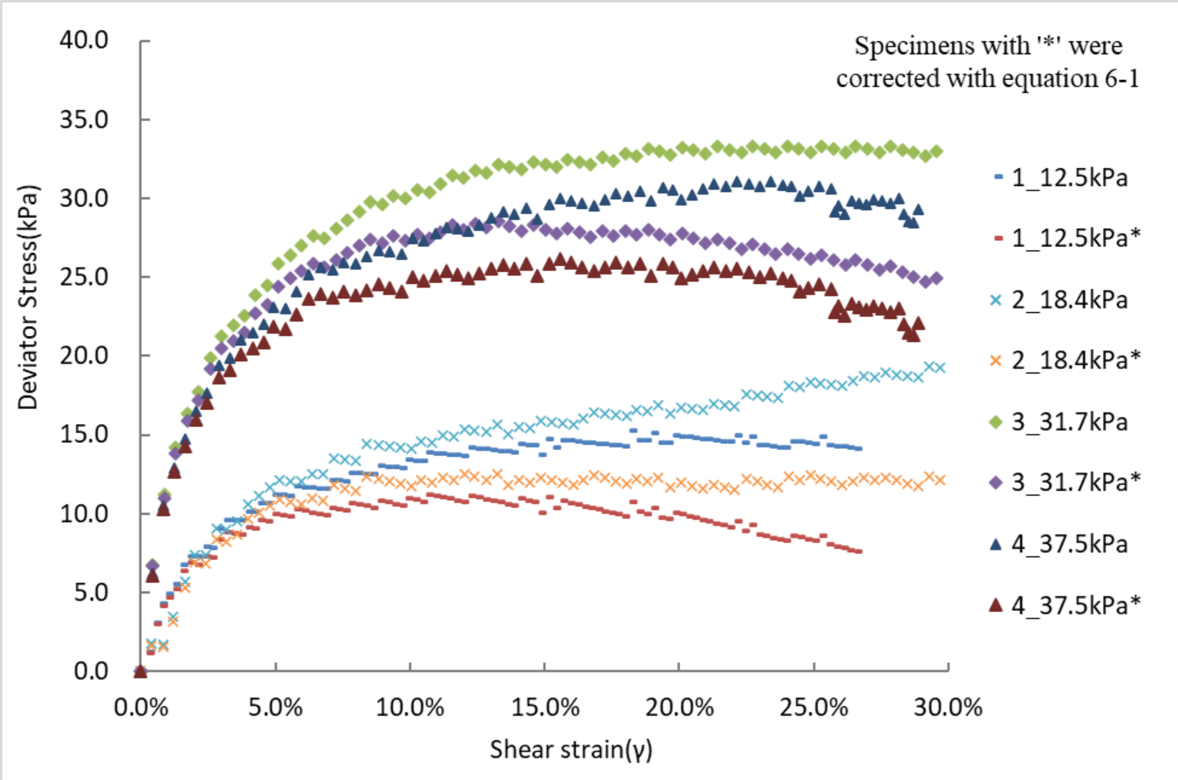


Figure 6-2: Comparison between the original uncorrected deviator stress values and the deviator stress values corrected using Equation 6-1

To better explain how membrane stiffness affects the strength of peat samples, Equation 6-1 was also applied to data collected from the literature and laboratory tests, as presented in Table 6-3 below. Table 6-3 shows the reduction of each value collected from the literature and laboratory tests after its correction with Equation 6-1. At 5% shear strain, the membrane effect ranges from 0.36 kPa to 1.43 kPa, with an average decrease of 1.17 kPa. For 10% shear strain, correcting for the membrane effect can result in a decrease of up to 2.52 kPa. Stress levels below 10 kPa showed much higher relative reduction percentages than those above 10 kPa – the lower the confining stress, the greater the apparent percentage reduction. These results demonstrate that the influence of membrane stiffness cannot be disregarded, especially at lower stress levels.

Table 6-3: Comparison of deviator stress values – initial uncorrected data and data after membrane correction

No	Stress level (kPa)	Deviator stress (kPa): 5% of shear strain			Deviator stress (kPa): 10% of shear strain			Reference
		Initial data	Data after correction	Percentage reduction	Initial data	Data after correction	Percentage reduction	
Triaxial test data in this study								
2_10.7kPa	10.7	10.93	9.75	11%	13.32	10.98	18%	Laboratory tests in this study
4_12.5kPa	12.5	11.19	9.91	11%	13.40	10.89	19%	
13_37.5kPa	37.5	23.03	22.67	2%	27.48	24.98	9%	
14_40.5kPa	40.5	28.03	26.67	5%	38.05	35.80	6%	
Triaxial test data in the literature								
CIUC001	15	14.19	12.91	9%	18.12	15.60	14%	Garnier (2007)
CIUC002	30	25.52	24.09	6%	30.04	27.52	8%	
Canada 1	24	37.06	35.87	3%	45.29	42.99	5%	Hendry (2012)
Canada 2	49	52.98	51.83	2%	62.86	60.51	4%	
Amsterdam 1	5.52	9.85	8.72	11%	10.77	8.51	21%	Zwanenburg (2015)
Amsterdam 2	6.68	10.15	8.98	12%	11.69	9.35	20%	

Table 6-4: Comparison of friction angles (φ') before and after membrane correction

Reference	Friction angle φ' (°) before membrane correction	Friction angel φ' (°) after membrane correction	Reduction (%)
Hendry (2012)	18.27	12.85	29.68
Zwanenburg (2015)	48.05	40.61	15.50
Ganier (2008)	27.4	22.93	16.36
Current study	19.8	14.3	27.78

Membrane correction significantly reduced the estimated effective friction angles (φ') across all the studies in Table 6-4. This effect was especially pronounced in peats and soft organic soils where the confining pressure was low and the membrane stiffness contributed appreciably to the measured resistance. In the current study, φ' decreased from 19.8° to 14.3°. This was a reduction of approximately 28%, which highlights the critical need for membrane correction when evaluating shear strength parameters. Similar results were observed for the data from previous studies, with reductions ranging from 16% to 30%. These findings reinforce the importance of correcting for boundary effects to obtain reliable strength parameters for peat.

The membrane stiffness effect offers a probable explanation for to why the deviator stress did not exhibit a peak value during the shear stage. These results support the findings of Raghunandan (2014) and indicate that there is a 10% reduction when applying Equation 6-1. Similarly, Table 6-3 also shows that the membrane stiffness effect can be identified when using Equation 6-3 (Yamaguchi, 1985; Garnier, 2007; Zwanenburg, 2015).

6.5 Summary

This chapter discussed the relationship between deviator stress and organic content for peat with an organic content of over 75% and the shear strength from triaxial tests and direct shear tests. The membrane stiffness correction was also investigated.

1. When peat's organic content is below 75%, the deviator stress increases to a peak value following the Mohr-Coulomb model. When the organic content is higher than 75%, the deviator stress continues increasing until the end of testing. The maximum curvature approach can be used in scenarios in which the Mohr-Coulomb model encounters difficulty in determining the friction angle.
2. For peat with an organic content over 75%, direct shear box tests tend to give higher estimates of shear strength than triaxial shear tests based on the shearing mechanism. However, the shear strength given by direct shear box tests is not necessarily accurate – the mechanism of direct shear acts only at the centre of a specimen, while triaxial shear can shear throughout specimens.
3. In this chapter, a thorough discussion and analysis were provided regarding the proposed equations concerning membrane stiffness corrections. It was determined that compression shell theory is suitable for use in this study.

7 CONCLUSIONS AND RECOMMENDATIONS

The overall goal of this research project was to investigate the undrained shear strength and stiffness of peat soils. Basic physical properties were investigated in situ, and tests in the laboratory were conducted at appropriate low effective stresses. This study was designed to enhance the understanding of peat's strength and develop a model to assess this strength in situ, following the research objectives set out in Chapter 1.

1. Peat samples from various depths were subjected to low effective stresses to visually assess deformation and shear behaviours during shearing. The objective was to propose an appropriate constitutive model for stiffness degradation in relation to shear strain for peat. This model was intended to serve as a predictive tool, enabling the anticipation of peat's effective shear strength across various conditions and shear strains.

2. The experimental results confirm that peat exhibits significant stiffness reduction even at small shear strains (below 20%). A reduction coefficient was successfully derived, enabling better prediction of the shear modulus degradation of peat in engineering applications.

3. The experimental results reveal that most peat specimens (10 out of 14 tests) fail to reach the theoretical tension cut-off (q/p' = 3), suggesting that deviator stress

primarily reflects interparticle connections rather than fibre tension. This study establishes a critical state line using maximum curvature points from triaxial tests, providing a framework for peat strength analysis.

4. Peat exhibits distinct shear behaviour depending on its organic content. Below 75% organic content, peat follows Mohr-Coulomb failure criteria with clear peak strengths. Meanwhile, above 75%, it shows continuous stress increases without peak values. Direct shear tests overestimate peat's strength compared to triaxial tests for high-organic-content peat (> 75%) due to localised shearing mechanisms versus triaxial shearing, highlighting the importance of appropriate testing methods for accurate strength characterisation.

7.1 Summary of Key Findings

7.1.1 In situ tests

GPR and VSWP were carried out at the sample collection locations. Peat samples were collected for laboratory physical tests and von Post classification.

1. The deviatoric stresses in the triaxial tests at large shear strains showed a high degree of agreement with the undrained peat shear strengths S_u predicted by the in situ shear wave velocity measurements. It was demonstrated that the shear wave velocity measurement $\mathrm{V_s}$ can be used together with the water content ω to derive the undrained peat shear strength S_u along a depth gradient, based on Trafford and Long (2016)'s method.
2. The in situ water content of peat was found to be greater than water contents recorded in the laboratory due to water loss during sample transportation. A

higher water content results in a lower shear strength. Water leakage during transportation to a laboratory must be carefully managed, as this is of considerable significance in the shear behaviours of peat. Peat's water content and organic content both exhibit significant variability, fluctuating within a meter along both the depth and width of a peat block.

3. The mechanical characteristics of peat are also influenced by its degree of decomposition, as measured by the von Post classification scale. The fibre content within the peat matrix and its fibrous composition can significantly affect the mechanical responses of peat.

7.1.2 Laboratory tests

A series of undrained unconsolidated and undrained consolidated triaxial tests were conducted on peat specimens from different depths taken from wind farms. The main findings in relation to the laboratory tests are as follows.

1. Peat soil is very soft. During the sample removal and preparation process, disturbance is inevitable. Therefore, it is prohibitively difficult to obtain undisturbed peat soil, which affects the subsequent evaluation of the maximum stiffness value of peat. In response to this problem, a relatively more reliable stiffness value at a strain of 0.1% was chosen as a reference value to replace the maximum stiffness value at the beginning of the experiment and consider stiffness reductions.
2. In triaxial tests, differences in specimen orientation and size can affect the behaviour of peat samples. Horizontally cut specimens may exhibit slightly greater inclinations after shearing compared to vertically cut ones. This could

be attributed to the anisotropy of the soil fabric and the orientation of fibres during the formation of peat layers over the years.

Furthermore, the size of the specimen can impact the stress-strain relationship in a peat sample. Generally, smaller specimens tend to display higher shear resistance strength compared to larger ones when subjected to the same shear strain. This phenomenon suggests that specimen size plays a role in the mechanical properties of peat during triaxial testing.

3. For soft peat, the influence of the test membrane on shear resistance is contingent upon the applied strain. When shear strain surpasses 10%, disparities between uncorrected and corrected deviator stresses may reach up to 10 kPa. The variance tends to be minor for strains within 10%. After comparing the proposed equations correcting for membrane effects, compression shell theory was chosen for use in this study.
4. In all the triaxial tests conducted, deviator stresses exhibited continuous increases with shear strain. These persisted without distinct peak strengths even at large strains exceeding 25%. Similarly, positive excess pore water pressure arose in specimens due to shearing. Unlike the typical shear failure modes observed in mineral soils, there was a notable absence of clear evidence regarding shear failure modes or shear bands in any peat specimen.
5. The undrained shear behaviour of peat can be described using critical state theory. In most samples, the stress path failed to reach the tension cut-off line prior to the occurrence of critical state failure. This was primarily due to peat’s fibre content. In addition, the peat samples had lower critical state friction angles of approximately 20°, which is consistent with previous literature.

6. The experimental results and literature data allow for the proposal of a simple elastic-plastic modelling framework for the deviatoric behaviour of fully saturated undisturbed peat. This model represents a step forward, but it is not yet a complete predictive modelling tool for the deviatoric behaviour of peat. Induced anisotropy is not accounted for in the constitutive formulation.

   An equation was proposed for the normalised shear modulus plotted against shear strain on a logarithmic scale. As mentioned earlier, the maximum modulus might not be accurate due to unavoidable sample disturbance. Thus, a normalised shear stiffness $G/G_{0.1\%}$ was used instead of G/G_{max} in this study. The $G/G_{0.1\%}$ decreased to 0.013–0.032 at a shear strain of 10%. Meanwhile, two methods were proposed to estimate the magnitude of the shear modulus *G* at 0.1% shear strain based upon the peat depth and confining pressure, respectively.

The influence of the organic content of peat on the strength of peat soil was investigated at two of the research sites and through data collected from previous investigations. The main findings of this part of the study are as follows.

1. When the organic content of peat falls below 75%, the deviator stress exhibits a characteristic increase, reaching a peak value in accordance with the Mohr-Coulomb model. Conversely, when the organic content exceeds 75%, the deviator stress continuously rises throughout the duration of the test. In cases where the Mohr-Coulomb model encounters difficulty in determining the friction angle, the maximum curvature approach can be used as a viable alternative.
2. For peat samples with organic contents over 75%, higher estimates of shear strength are given by direct shear tests than triaxial shear tests due to the shearing mechanism. However, it is essential to note that while direct shear box

tests may offer higher shear strength estimates, their accuracy is not assured. This discrepancy arises because the mechanism of direct shear primarily acts at the centre of a specimen, whereas triaxial shear entails shearing throughout the entirety of a specimen.

This research offers critical insights into the complex behaviour of peat, particularly under undrained shear loading conditions. The combination of advanced laboratory testing, membrane corrections, and strain-dependent stiffness analysis provides a more accurate understanding of peat's mechanical properties. Achieving a better understanding of peat's mechanical properties is an essential step in improving engineering design and risk assessment in peat-rich environments. The findings of this study are especially relevant in countries like Ireland, where soft organic soils are widespread and often encountered in infrastructure projects.

The study confirms that peat exhibits highly strain-dependent, anisotropic, and structurally variable behaviour that cannot be adequately captured through traditional strength and stiffness parameters alone. The implications of this study extend beyond laboratory understanding – the results can be used to influence how engineers assess settlement, stability, and the long-term performance of foundations, embankments, and transportation infrastructure built on or within peat layers.

The findings of this study carry significant implications for real-world geotechnical engineering applications in peatlands. By establishing the 75% organic content threshold that differentiates conventional Mohr-Coulomb behaviour from continuous stress-strain responses, this research provides guidance for foundation design and ground improvement strategies. The demonstrated advantages of triaxial testing over direct shear methods for high-organic-content peat (> 75%) call for revisions in site investigation standards, particularly for critical infrastructure projects in peat-rich regions. The research also highlights the pressing need for specialised low-stress testing equipment (< 10 kPa) to accurately characterise shallow foundation behaviour. These advancements collectively move peatland engineering from empirical rules to mechanics-based design, offering both environmental advantages and economic benefits through reduced remediation costs.

7.2 Recommendations for future research

Further research efforts should prioritise the development of a constitutive soil model specifically for peat and designed for implementation within a finite element program. Additionally, attention should be directed towards investigating the influence of the fibre and organic contents of peat specimens and exploring the long-term deformation characteristics of peat samples.

1. Previous experimental findings in the literature indicate that both water and organic contents significantly impact measurements of peat strength. Consequently, it is essential to consider the influence of water and organic contents on experiments. Accurate measurement of the moisture and organic content of each peat specimen is necessary to effectively analyse the impacts of these factors on experimental results.
2. During specimen preparation, the careful trimming of both fibres and soil is essential to minimise sample disturbance. It is recommended to use scissors rather than a knife to cut fibres in a peat sample.
3. The unconsolidated and undrained experiments in this study may not adequately verify the anisotropic characteristics of peat soil, and the influence of the size

effect has not been thoroughly investigated. Therefore, additional experimental data are needed to validate anisotropy and size effects in peat.

4. Further investigation into remoulded peat samples with different organic and fibre contents is essential, given the effect of organic content. Based on the equations for membrane stiffness correction, clarifications of and experimental tests on membrane correction are essential for advancing research and modelling efforts in the future.
5. The development of a numerical model modified specifically for peat soil based on existing constitutive models will enable the full exploration of stress, fibre, and pore pressure responses resulting from the anisotropic stiffness of peat. This development will facilitate the simulation and exploration of the peak stress locations and potential failure models of peat.

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