

GEOTECHNICAL BEHAVIOUR OF FROZEN MINE BACKFILLS

By

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A thesis submitted

under the supervision of Dr. Mamadou Fall

**in partial fulfillment of the
requirements for the degree of
Master of Applied Science
in Civil Engineering**

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Dedicated to my family

ABSTRACT

This thesis presents the results of an investigation of factors which influence the geotechnical properties (strength and deformation behaviour) of frozen mine backfill (FMB). FMB, which is either frozen tailings backfill (FTB) or frozen cemented paste backfill (FCPB) used to fill the voids left by ore extracting processes, has extensive application potential for mining in permafrost areas. A better understanding of the geotechnical behaviour of FMB under various conditions will be helpful for engineers who work in mine backfill areas to select the appropriate mix parameters for FMB and to design cost-effective FMB.

Once placed, FMB has to satisfy certain dynamic and static loading requirements to ensure a safe underground working environment for all mining personnel. One of the most important criteria for geotechnical quality in hardened mine backfill is mechanical stability or its mechanical properties at a given time. The uniaxial compressive strength (UCS) of hardened backfill is often used in practice to evaluate mine backfill stability. However, UCS is not the only significant parameter that shows the structural integrity of mine backfill. In the ground support role, the deformation behaviour and stiffness of the FMB are also key design properties of interest.

Most studies conducted to understand the geotechnical properties and behaviour of mine backfills were performed at temperatures above zero. There is a lack of information about the geotechnical behaviour and properties of mine backfills at sub-zero temperature conditions.

In this thesis, uniaxial compressive tests were conducted on a series of FTB and FCPB samples. Information about the geotechnical behaviour (stress-strain behaviour, strength, and modulus of elasticity) of FMB is obtained. The effects of FMB mix components and vertical compression pressure on the strength and deformation behaviour of FMB are discussed and summarized.

An optimum total water content of 25%-35% is found in which the strength and the modulus of elasticity of the FTB are 1.4-3.2 MPa and 35-58 MPa, respectively. It is observed that a small amount (3-6%) of cement can significantly change the stress-strain behaviour and strength of FTB under the same curing and testing conditions. The stress-strain behaviour and geotechnical properties of FTB are compared to those of FCPB.

ACKNOWLEDGMENTS

I would like to thank my supervisor, Professor Mamadou Fall, who encouraged me during my studies and Master's thesis at the University of Ottawa, and provided me with his sound knowledge and expertise on the subject of mining tailings backfill. My special thanks goes to Jean Claude Celestin who provided all of the graduate students with great help in the geotechnical lab and made it possible for this program to be finished in a short period of time. Appreciation is also due to my brother, Fa Bin Han, and his family, for being there during the most difficult moments when I started a new life in Canada. Finally, I gratefully acknowledge my wife, Hai Xia Feng, my daughter, and sons for their support and patience during my study.

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CHAPTER 1:

INTRODUCTION

1.1 Problem statement

Mining activities generate a large amount of solid waste, such as waste rock and tailings.

In Canada, more than 200 mines are currently in production and it is estimated that about 500-650 million tons of waste are generated annually (Pokharel, 2008; Orejarena, 2010).

This issue is also faced by mining industries worldwide because they follow the same mining principle for extracting valuable minerals and disposing waste (Vick, 1983; Robinsky, 1999; Orejarena, 2010).

Mining industries contribute strongly to the economy of Canada and many other countries worldwide (Fall et al., 2009). Figure 1-1 shows the annual contributions of metal mineral productions to the Canadian economy from 1973 to 2008. An apparent increase trend can be observed from Fig. 1-1.

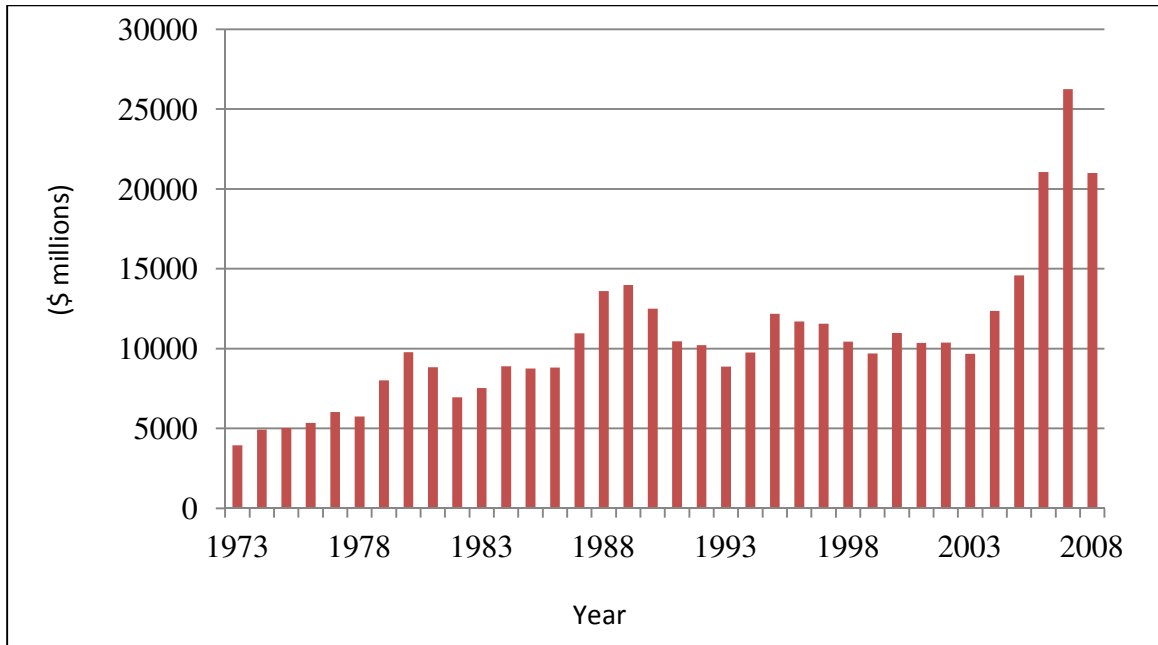


Figure 1.1 Value of the contribution of metal mineral production to the Canadian economy (Data source: Statistics Canada, cat. no. 91-215-X)

Aside from significant economic contributions, mining activities also generate a large amount of waste rock and tailings. In the not so-distant past, surface impoundments were the most widely used method of tailings disposal, and even the returning of tailings to underground mines has long been practiced for the purpose of assisting mining operations (Robinsky, 1999). However, the surface disposal of such waste can create several environmental and geotechnical problems. Public perception and strict government regulations with regards to the disposal of such waste compel the mining industry to develop new strategies which are environmentally sound and cost effective. In this scenario, the recycling of such waste into mining or civil engineering construction materials have become a great challenge for the mining and civil engineering community

(Fall and Samb, 2006). In recent years, there is an increasing trend to make the tailings a useful construction material (e.g., Zou and Sahito 2004; Fall et al. 2009, 2010).

One of these materials is mine backfill. In mine backfilling, the tailings and/or waste rocks are returned to the underground to fill the voids left by ore extracting in the form of mine backfills (hydraulic backfill, rockfill, and cemented paste backfill). Cemented paste backfill (CPB) is the most used method of mine backfilling due to several reasons which will be discussed later.

Once placed, mine backfill has to satisfy certain dynamic and static loading requirements to ensure a safe underground working environment for all mining personnel (Fall and Samb, 2006). Since mine backfill failures not only have considerable financial ramifications, but often result in fatalities or injuries as reported by several mines in Ontario, Canada and worldwide. The reasons for such failures can be complex and involve several factors, such as the quality of the prepared mine backfills (optimized or mix proportioning of the backfill recipes), mining environment (interactions between the backfill and the surrounding or adjacent rock mass, seismic events, rock burst, in situ mine temperature, etc.), and engineering design (Fall et al., 2008).

One of the most important geotechnical quality criteria for hardened mine backfill is mechanical stability or its mechanical properties at a given time. Indeed, the mine backfill structure must remain stable during the extraction of adjacent stopes to ensure the safety of the mine workers and avoid ore dilution. The uniaxial compressive strength

(UCS) of the hardening or hardened backfill is often used in practice to evaluate the backfill stability at a desirable time since the test is relatively inexpensive and can be incorporated into routine quality control programs at the mine (Mitchell, Olsen, and Smith, 1982; Fall and Samb, 2008; Nasir and Fall, 2010). However, UCS is not the only significant parameter that can be used as an indicator for the structural integrity of the backfill. In the ground support role, the deformation behaviour (stress-strain behaviour) and stiffness (modulus of elasticity) of the backfill are also key design properties of interest (Fall, 2008).

Several studies (e.g., Zou and Sahito, 2004; Roux, Bawden and Grabinsky, 2005; Fall and Benzaazoua, 2005; Fall and Samb, 2006; Fall, Benzaazoua, and Samb, 2008; Nasir and Fall, 2010; Fall and Pokharel, 2010) have been conducted in the last decade to understand the geotechnical properties (e.g. strength) and behaviour (e.g. stress-strain behaviour) of mine backfill, especially those of CPB. However, most of these studies were conducted on CPB and at temperatures above zero. The progressive depletion of ore available at shallow depths in a number of underground mines in Canada means that underground mining operations are increasingly being carried out not only at greater depths, but also in permafrost regions, i.e. at sub-zero temperatures. There is a lack of information about the geotechnical behaviour and properties of CPB at sub-zero temperature conditions. Furthermore, many mines located in permafrost regions are very far from the urban places where the cement is produced. This often makes the use of cement in mine backfill in permafrost expensive due to the high transport cost of the cement. To tackle this issue, there is a relatively new trend in which the technology of

artificial ground freezing is used to produce a frozen backfill. The principle of ground freezing is the use of refrigeration to convert in-situ or backfill pore water into ice. The ice (instead of Portland cement) is used as the binding phase in mine backfill to produce frozen backfill. The frozen backfill can be a mixture of waste rock and tailings sprayed with ice cold water (frozen rockfill), or a mixture of tailings sprayed with ice cold water (frozen tailings backfill). This technology is also applied to many geotechnical works (tunnel construction, excavation stability, slope stability, deep underground storage)..

There is a lack of knowledge about the geotechnical properties and behaviour of frozen CPB and mine backfill, especially about:

- the effect of the mix components and loading conditions on the strength of frozen tailings backfill (FTB) and frozen cemented paste backfill (FCPB), and
- the deformation behaviour of FTB and FCPB and the factors which affect it.

1.2 Objectives of this thesis

The main objectives of the project are to:

- study the strength development of FTB and FCPB and the main factors that affect it;
- study the deformation behaviour of FTB and FCPB (compressive stress-strain behaviour, modulus of elasticity); and
- analyze the effect of cement hydration on the strength, stiffness and stress-strain behaviour of FCPB in sub-zero temperature conditions.

To facilitate the reading of the present manuscript, frozen mine backfill will be used to denote FCPB and FTB.

1.3 Scope of the investigation

The main purpose of this research is to conduct a series of uniaxial compressive tests on FTB and FCPB specimens prepared in the laboratory to better understand the strength and deformation behaviour of frozen mine backfill (FMB). In order to achieve this aim, the following approach has been adopted.

The first step of this investigation is to gain sufficient fundamental knowledge that will help to better understand the strength and deformation behaviour of frozen mine backfill (FMB). Therefore, literature on mine tailings backfill, physical and mechanical properties and behaviours of frozen soils, physical and mechanical properties of ice and snow, and cement hydration will be reviewed and related basic knowledge will be acquired.

Then, two series of experimental tests will be designed and performed to investigate the strength and deformation behaviour of FCPB and FTB. The first series of tests (Chapter 3) deals with the effect of factors such as mix components (cement content), binder types, and curing ages on the strength and deformation behaviour of FCPB. The changes in the physical properties of FCPB during the freezing process, such as the changes in void ratio and total water content are also investigated in this study. As for the second series of tests (Chapter 4), the strength and deformation behaviour of FTB and the main factors that

influence its behaviour are studied. The investigated factors include FTB mix components (tailings fineness, water content, snow content) and compression efforts.

Finally, the geotechnical behaviour of FCPB is compared to that of FTB.

1.4 Organization of the thesis

The thesis is organized into five chapters. Chapter 1 presents a general introduction on FMB, the problem statement, objectives of this study, and scope of the investigation to the readers. Chapter 2 provides a theoretical and/or technical background on mine tailings backfill, physical and mechanical properties of ice and snow, physical and mechanical properties of frozen soils, and cement hydration. Chapter 3 presents the results with regards to the strength development and deformation behaviour of FCPB. In this paper, the uniaxial compressive test results on FCPB are provided. The effects of FCPB mix components on the strength and deformation behaviour of FCPB are analyzed and summarized. Chapter 4 provides the results with regards to the strength development and deformation behaviour of FTB. In this paper, the uniaxial compressive test results on the constant strain rate in FTB are provided. The effects of FTB components as well as the compression pressure on the strength and deformation behaviour are analyzed and discussed. In Chapter 5, a comparison of the geotechnical behaviours of FTB and FCTB is provided and the general conclusions of this study are presented. Recommendations for future studies are provided at the end of this chapter.

CHAPTER 2:

THEORETICAL AND TECHNICAL BACKGROUND

2.1 Introduction

As a multi-phase system, FMB has similar components as frozen soils. It is composed of solid particles, ice (or snow), gases of various chemicals, and unfrozen water. The geotechnical behaviour of frozen soil is greatly influenced by internal factors, such as solid particles, ice and unfrozen water content as well as external factors, such as temperature, strain rate, etc. (Tsyrovich, 1973; Andersland and Andersson, 1978; Arenson and Springman, 2007). For a better understanding of the issues addressed in this thesis and the effects of these factors on the geotechnical properties of FMB, some background information on mining tailings backfill, physical and mechanical properties of ice and snow, physical and mechanical properties of frozen soils, and cement hydration will be given in this chapter.

2.2 Background on Mine Tailings Backfill

2.2.1 Mine tailings backfill: composition and application

Safety and environmental performance are the main concerns in mining operations. One of the most popular technologies used in underground mining to ensure the safety of mine

workers, reduce the environmental hazards related to mine waste management and the maximum recovery of ore are mine backfills (Belem and Benzaazoua, 2004; Bandopadhyay and Izaxon, 2004; Fall, 2008). Mine backfills are also an effective means of mine waste management (Fall and Samb, 2006). There are three main types of mine backfills: hydraulic, cemented paste, and rockfill.

- Hydraulic backfill (HB) is a mixture of alluvial sand and mill tailings, and a small amount of cement. Typical HB density is about 60-75% solids by weight and around 10% by weight of particles with a size less than 10 μm to ensure that the design fits the criteria for permeability of the placed backfill (Grice, 1998; Fall, 2008).
- Cemented paste backfill (CPB) is an engineered mixture of mine tailings, water and binders. CPB typically consists of 70-85% solids by weight and 3-7% hydraulic binders by weight and water (Fall and Samb, 2006). The components of CPB are mixed in a plant on the surface and transported (by gravity and/or pumping) to an underground mine (Figure 2-1). CPB is extensively and increasingly used in underground mine operations.
- Rockfill (RF) is usually a mixture of waste rock, tailings, water and cement. RF is often transported by trucks to the openings.

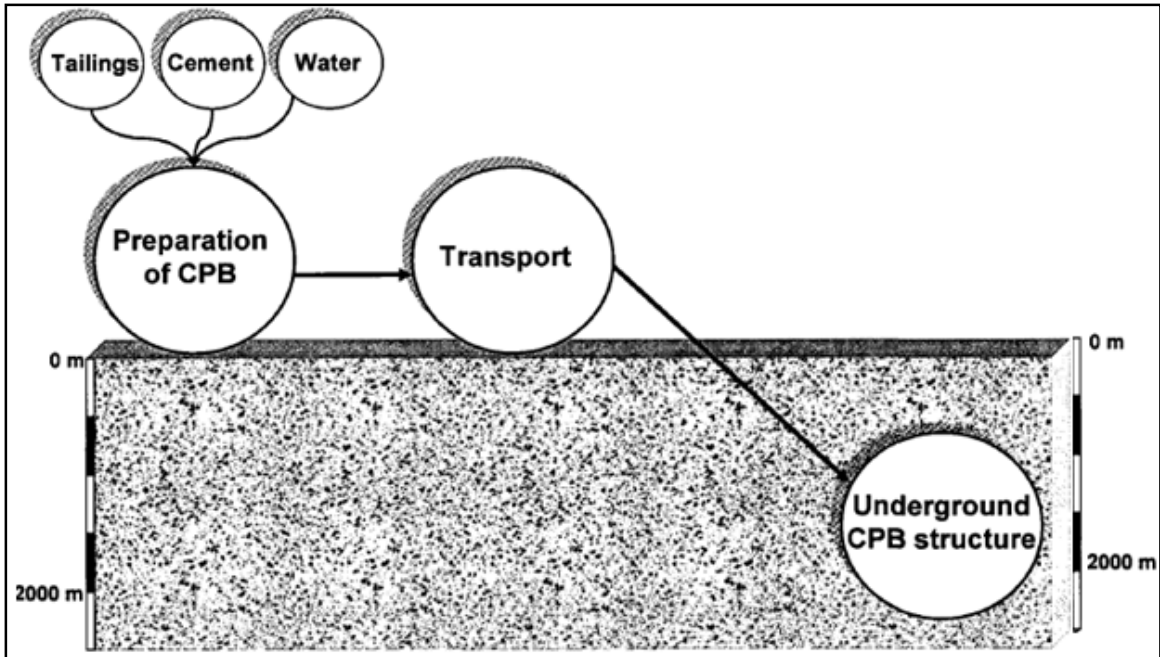


Figure 2.1 Schematic presentation of the different phases of CPB technology: preparation, transport and underground placing of the CPB, and location for building a CPB structure (Fall et al., 2008)

2.2.2 Geotechnical properties and behaviour of mine backfill and factors that affect them

The two geotechnical properties that are of general concern are the target strength and stiffness represented by UCS and modulus of elasticity (E), respectively, for designing mine backfill materials (Scoble and Piciacchia, 1986). The UCS of the hardening or hardened backfill is generally accepted as the target compressive strength parameter for evaluating the stability of mine backfills in a desirable period (Mitchell et al., 1982; Fall and Samb, 2006).

Previous work has shown that factors which influence the mechanical properties and behaviour of mine tailings backfill vary from one type to another. The effects of these factors have been studied and summarized by many authors (e.g., Mitchell et al., 1982; Scoble and Piciacchia, 1986; Farsangi, 1996; Fall and Benzaazoua, 2005.)

The main factors that affect the strength and deformation behaviour of cemented backfill are given below:

- tailings properties (tailings fineness and particle size distribution, tailings type, reactivity of the tailings),
- binder type and content,
- chemical composition of the mixing water (e.g., sulphate content),
- water to cement ratio (w/c ratio),
- curing temperature,
- backfilling rate and overburden stress, and
- drainage conditions.

Due to sub-zero temperature loading conditions, it can be anticipated that the geotechnical properties and behaviours of FTB and FCPB will be different from those of the previously mentioned materials, i.e. HB, RF and CPB. Except for the aforementioned influencing factors, it can be assumed that the ice or snow content and unfrozen water content of FTB and FCPB have significant impact on their geotechnical properties and behaviour. Some background information about the physical and mechanical properties

of ice and snow is given in Section 2.3 for a better understanding of the properties of FTB and FCPB.

2.3 Background on Physical and Mechanical Properties of Pore Ice and Snow

2.3.1 Physical and Mechanical Properties of Pore Ice

2.3.1.1 Ice Crystal Structure

It has been discovered that ice can form in fifteen separate known phases. Figure 2.2 shows most of these ice phases. The predominant ice form found on earth is hexagonal crystalline ice, I_h . Its crystal structure is shown in Figure 2.3. Furthermore, it is generally agreed that pore ice within frozen soil is in the form of I_h (Anderson and Morgenstern, 1973).

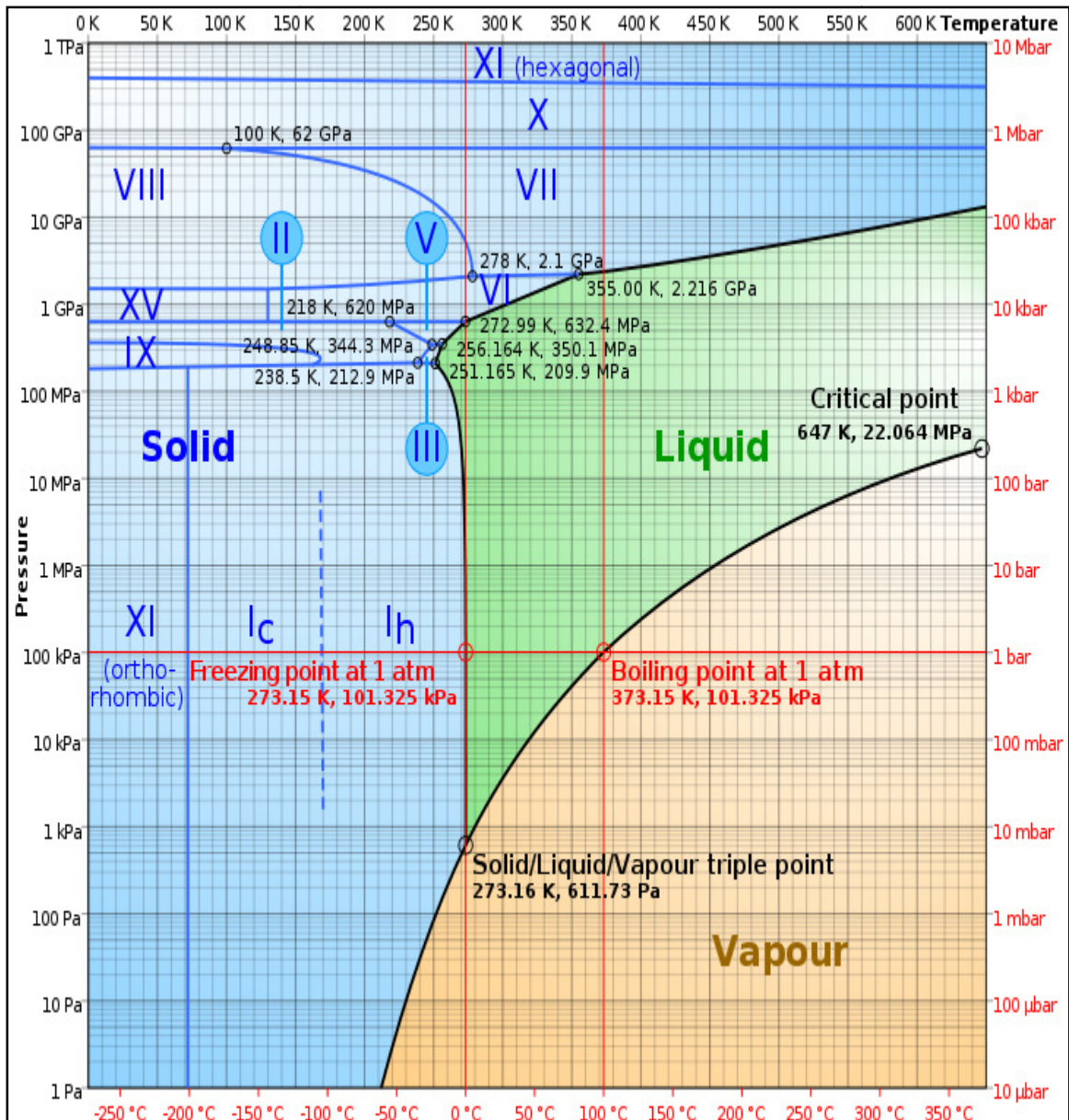


Figure 2.2 Phase diagram of water (Source: <http://en.wikipedia.org/wiki/Ice>)

Pauling proposed the first accepted crystal structure of ordinary ice (I_h) in 1935 (Fletcher, 1970) which is shown in Figure 2.3. The structure of I_h is composed of tessellating hexagons, with an oxygen atom on each vertex. The distance between the oxygen atoms along each bond is about 275 pm and the same between any two bonded oxygen atoms in

the lattice. The angle between the bonds in a crystal lattice is very close to a tetrahedral of 109.5° .

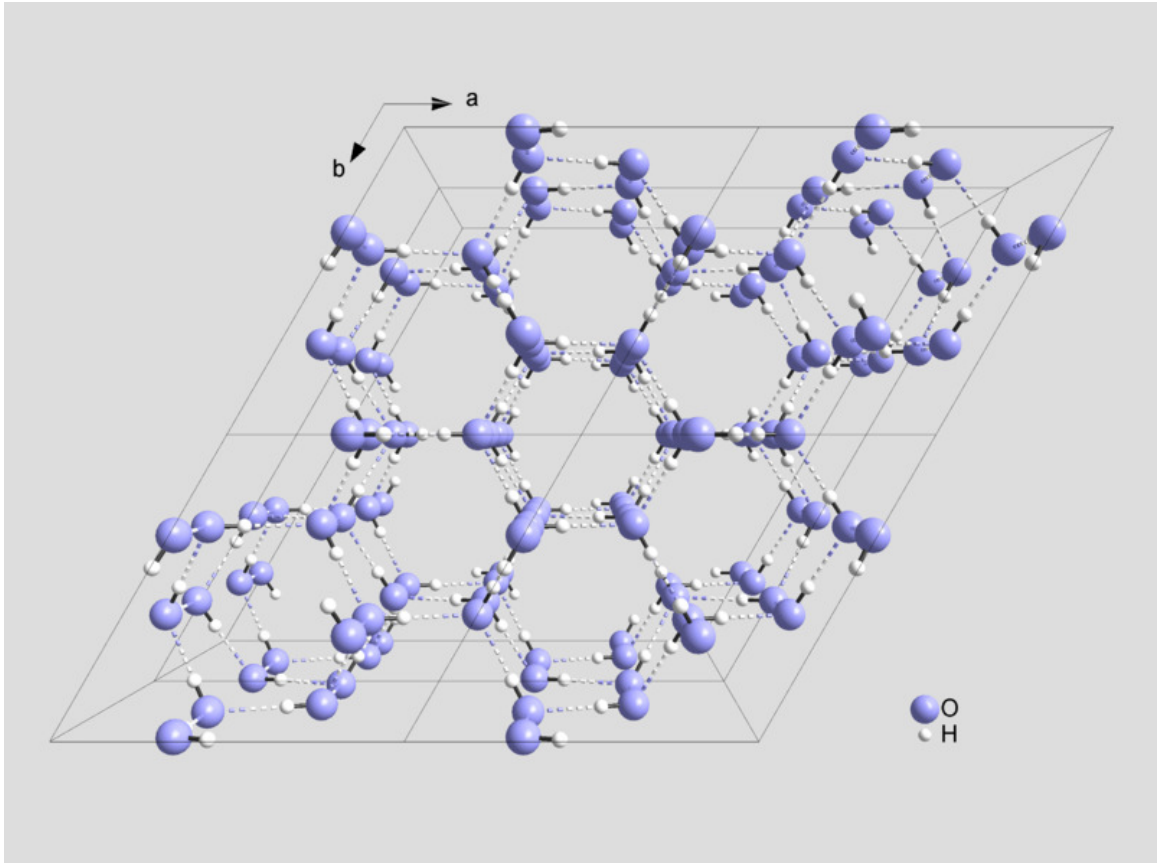


Figure 2.3 Crystal structure of ice I_h (Source: http://en.wikipedia.org/wiki/Ice_Ih)

2.3.1.2 Physical Properties of I_h

I_h has a density of $9.17 \times 10^2 \text{ kg/m}^3$ which is less than that of liquid water. The density of ice increases as temperature decreases. The latent heat of melting is 5987 J/mol (Fletcher, 1970).

2.3.1.3 Tensile and Compressive Strength of I_h

The properties and behaviours of ice have a great impact on those of frozen soils. It is believed that only polycrystalline ice forms in the pores of frozen soil (Anderson and Morgenstern, 1973). So, a strong background about the compressive strength of ice is good for a better understanding of the mechanical properties and behaviours of frozen soils. The strength of ice is generally evaluated by its UCS and the value is approximately 1 MPa (Andersland and Anderson, 1978; Sportt 1983; Hivon, 1991). However, this value is variable because it is influenced by several factors such as mechanical loading rate, temperature.

The failure modes of ice under compression depend on the strain rate applied and are also related to temperature. At high strain rates, the failure mode is brittle. At medium to low strain rates, it is ductile. Finally, a third mode is a combination of brittle and ductile deformation.

Two yield points, as shown in Figure 2.4, can be found in the stress-strain curve of polycrystalline ice while the strain rate is low (less than 10^{-4}s^{-1}).

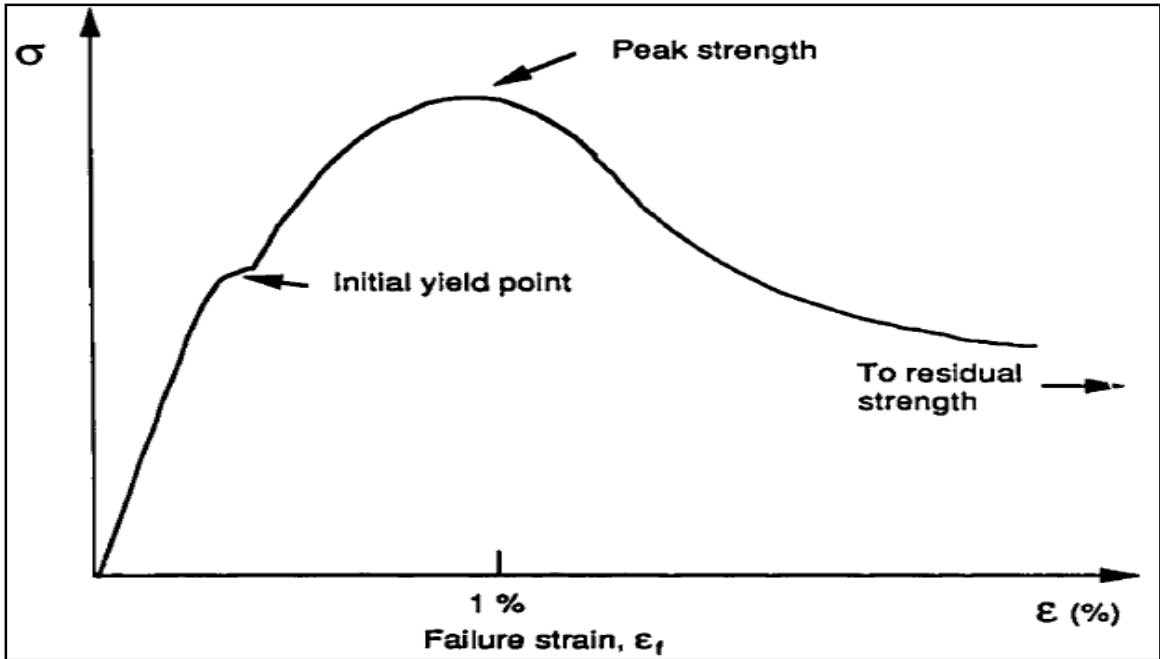


Figure 2.4 Typical ductile stress-strain curve for polycrystalline ice under a constant strain rate (From: Hivon 1991)

2.3.1.4 Deformation properties of I_h

The modulus of elasticity, E of polycrystalline ice can be predicted in the form of Eq. 2.1 (De Souza and Dirige, 2001).

$$E = \delta \times T \dots\dots\dots (2.1)$$

where: E = Modulus of elasticity, GPa

δ = Coefficient, that varies from 0.0569 to 0.0648 depending on the strain rate

T = Temperature difference below freezing point in a positive number (meaning)

The deformation modulus of ice displays various characteristics depending on applied loading rate and temperature. Secant modulus can be a good approximation of the E if the temperature is low and the loading rate is high. However, at relatively high temperatures, but low loading rates, the secant modulus is usually lower than the true value of E because the stress-strain curve is not linear (Hivon, 1991).

The Poisson's ratio of ice increases with temperature and ranges between 0.31 and 0.55. The failure mode of ice under a compressive load can be brittle or viscous depending on the temperature and the applied loading (strain) rate (De Souza and Dirige, 2001).

2.3.1.5 Stress/Strain/Time Relations

Another deformation characteristic of ice is creep under constant stress. Creep deformation behaviour reflects the stress-strain-time relation of ice under constant stress conditions. The creep process is due to recrystallization, grain migration, and crack formations (Hivon, 1991). Four stages are commonly accepted as the response of ice creep under constant stress conditions: 1) instantaneous elastic strain, usually called ϵ_0 ; 2) primary creep stage with a decelerating strain rate; 3) secondary creep stage; and 4) tertiary creep stage with an accelerating strain rate up to failure.

A great number of models, either based on primary or secondary creep, or a combination of both, have been developed to predict ice creep behaviour (Ting and Martin, 1979; Sego and Morgenstern, 1983; Azizi, 1989).

2.3.2 Physical and Mechanical Properties of Snow

2.3.2.1 Introduction

Snow is a granular material which consists of a multitude of snowflakes. Its impact on the mechanical properties and behaviour of frozen tailings backfill is studied in this project. A review of its formation, snowflake type, and physical and mechanical properties is beneficial to the reader for a better understanding of the mechanical properties and behaviours of frozen tailings backfill that contains snow as one of its mix components.

2.3.2.2 Formation of Snow, Snowflake Types and Snow Metamorphosis

Snow formation depends on the weather conditions and the so-called condensation nuclei in the atmosphere. The water vapor in the atmosphere is the source of snow, but condensation nuclei provide a surface for water vapor to condensate on it to form water droplets- the rudiment of snow. When the temperature is below the freezing point, these water droplets can then transform into ice crystals that can stick together in aggregates to form snowflakes (Klesius, 2007; Píšková et al., 2008). Natural snowflakes can be in a variety of shapes and sizes depending on the weather conditions (air pressure, moisture, temperature). The crystal structure of natural snow can be needles, plates, stars, spatially stars, and short scantings, depending mostly on the temperature during formation. Figure 2.5 illustrates the snow morphologies that correspond to the temperature and super saturation conditions (Píšková et al., 2008). It can be easily seen that snowflakes can be

found in various morphologies, such as simple prisms, stellar plates, and sectored plates under various temperatures and super-saturation conditions.

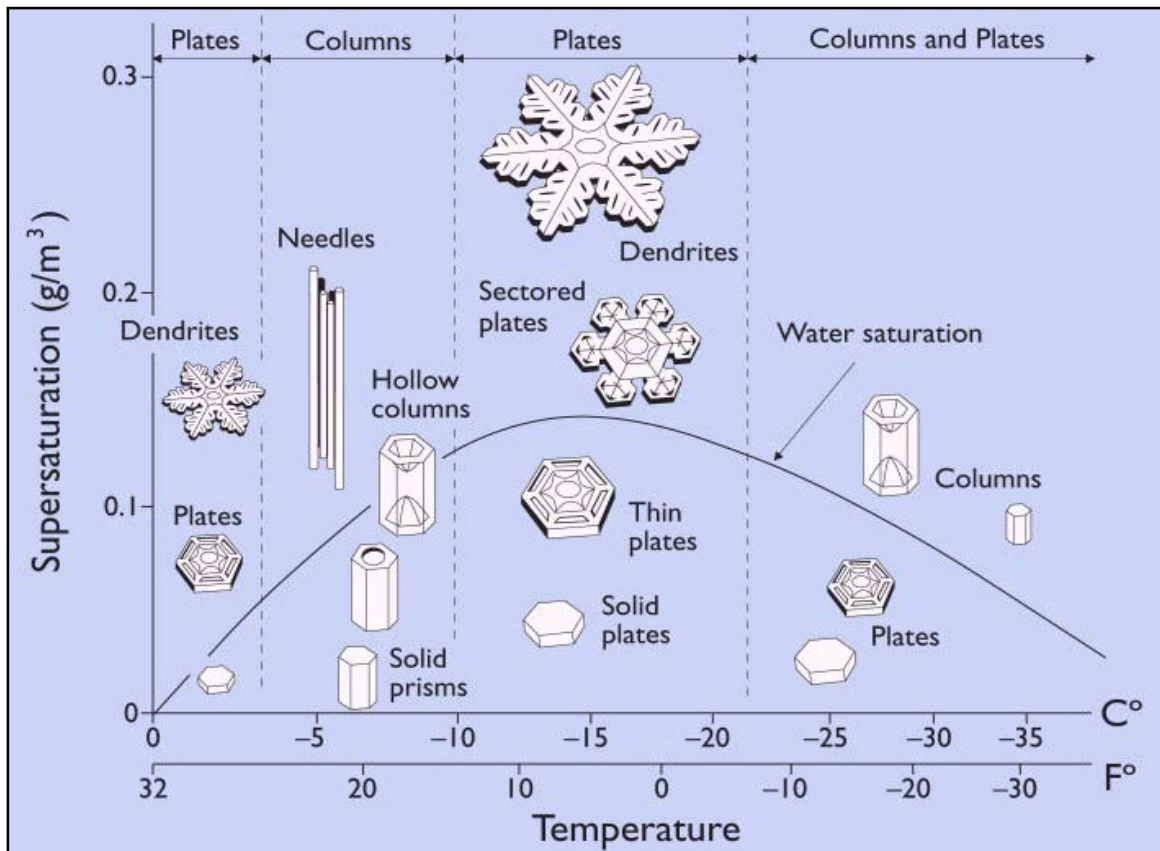


Figure 2.5 Snow morphologies (From: Pířková et al., 2008)

Pířková et al. (2008) summarized the four processes of snow metamorphism: deformation through wind, rounding, faceting and melting/freezing or a combination of these processes.

2.3.2.3 Physical Properties of Snow

The studied physical properties of snow are temperature, density, porosity, and moisture content. The temperature within a snow layer is typically around 0°C because snow is a

good insulator due to high porosity and has the highest specific heat capacity of water. The density of snow widely varies. The range can be 10 kg/m³ to close to the density of water depending on its metamorphosis process (Pířková et al., 2008; California Data Exchange Center, 2007; Glossary of Meteorology, 2009; Glossary of Meteorology, 2009). The porosity of snow is tightly related to its density and can be easily calculated by using Eq. 2.2.

$$i = \frac{\rho_{ice} - \rho_{snow}}{\rho_{ice}} \dots\dots\dots 2.2$$

where i = the porosity of the snow

ρ_{ice} , ρ_{snow} = the density of ice and snow, respectively

The moisture content of snow can be 0 (dry snow) to above 15% (slush) (Pířková et al., 2008).

2.3.2.4 Tensile and Compressive Strength of Snow

Little information is available about the tensile strength of snow. Petrovic (2003) summarized the tensile strength of snow in correspondence to its density, see Figure 2.6. It can be found that the tensile strength of snow is much lower than that of ice.

The compressive strength of snow is illustrated in Figure 2.7 (Mellor, 1974) by collecting the results of uniaxial compressive (i.e. unconfined) tests from several researchers. It is noticed that a common characteristic of the tensile and compressive strength of snow is the large variation up to four orders of magnitude or more.

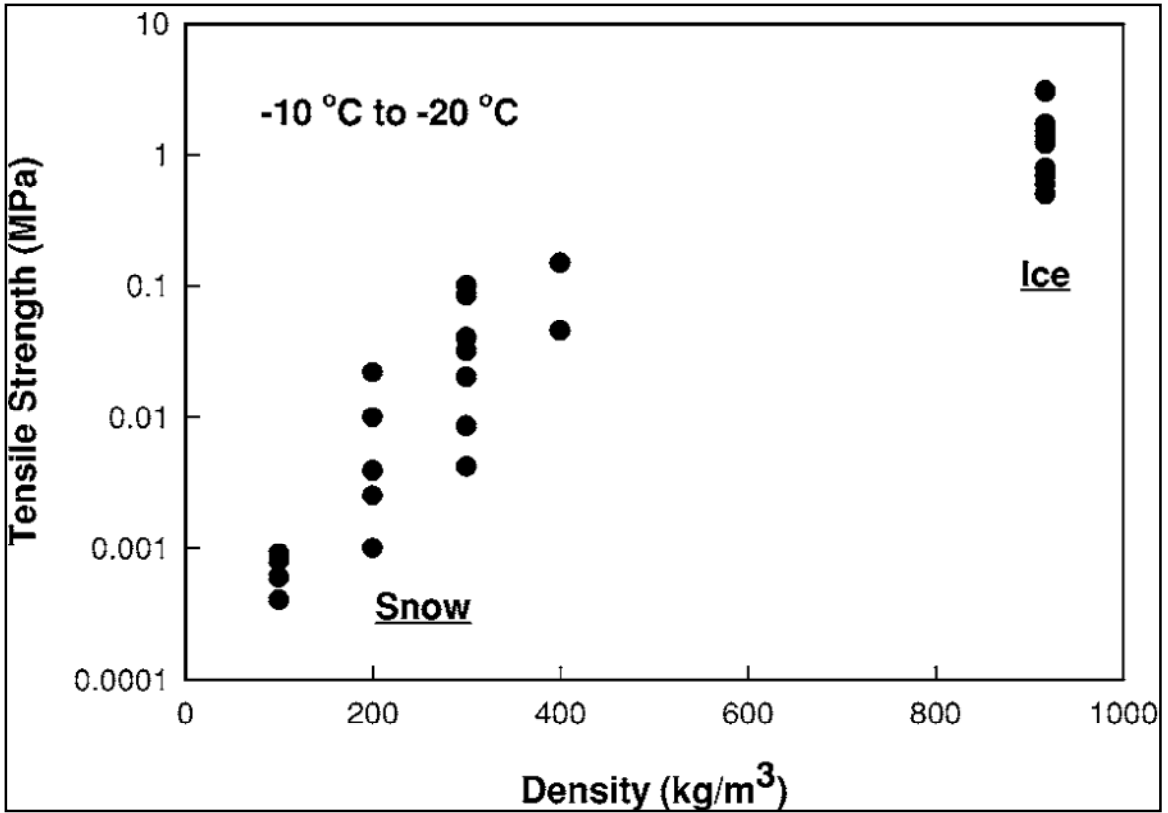


Figure 2.6. Tensile strength of snow as a function of snow density (From: Petrovic, 2003)

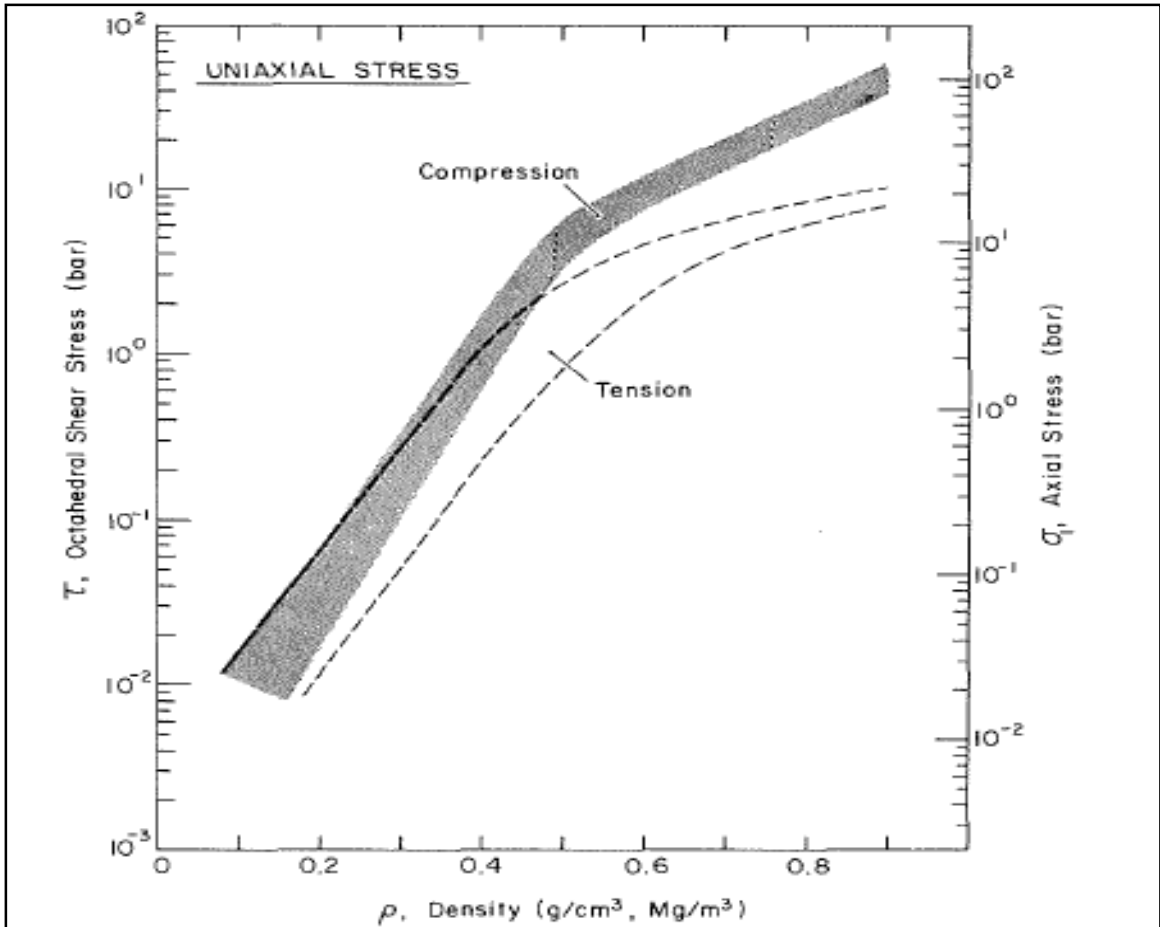


Figure 2.7 Strength of dry, coherent snow under rapid loading in uniaxial stress states

2.3.2.5 Viscoelastic Properties of Snow

Viscoelasticity is the material property that exhibits both viscous and elastic characteristics when undergoing deformation. If snow undergoes pure shear stress, it can be assumed to be linearly elastic. However, if the bulk stress is significant, then a linear elastic model may mislead its deformation behaviour because the density of snow will decrease in response to the applied bulk stress and E is a very strong function of density (Mellor, 1975). Under this consideration, viscoelastic modulus generally obtained by

performing dynamic tests is needed to assess the deformation behaviour of snow. The knowledge required for conducting this testing and calculating of the viscoelastic modulus is beyond the scope of this project.

2.4 Background on Physical and Mechanical Properties of Frozen Soils

Since the components (mineral solid particles, ice, unfrozen water, air) of frozen soils are similar to those of FMB, background information on the physical and mechanical properties of frozen soil is given in this section. This will help to provide a better understanding of the results of the tests performed on FMB.

Frozen soil is defined as soil or rock with temperatures below the freezing point of water (Andersland and Ladanyi, 2004). If a negative temperature state can last more than two years, it is called permafrost. Frozen soils are a natural multi-component system, which includes mineral (and occasionally organic) solid particles, ice, unfrozen water, and gases of various chemical compositions (Razbegin, Vyalov, Maksimyak, and Sadovskii 1996; Maksimyak and Sadovskii 1996). This multi-component multi-phase system of interconnected particles has long been used as surface and underground excavation support systems.

The physical and mechanical properties as well as the research methods, such as sample taking and test conducting, are different from those of unfrozen soils (Arenson and Springman, 2007). The main reason for such differences is that the geotechnical properties of frozen soils are sensitive to temperature which is also influenced by factors such as unfrozen water content, stress state (confined or unconfined), strain rate, grain

size distribution, etc. (Tsytovich, 1973; Andersland and Anderson, 1978; Freitag and McFadden, 1997; Andersland and Ladanyi, 2004 ; Arenson and Springman, 2007).

2.4.1 Physical Properties of Frozen Soils

It is commonly recognized that frozen soil is a four-component system that consists of soil particles, ice, unfrozen water, and voids with gases of various chemicals. Figure 2.8 schematically shows the phase compositions of frozen and unfrozen soils (dried, saturated or unsaturated).

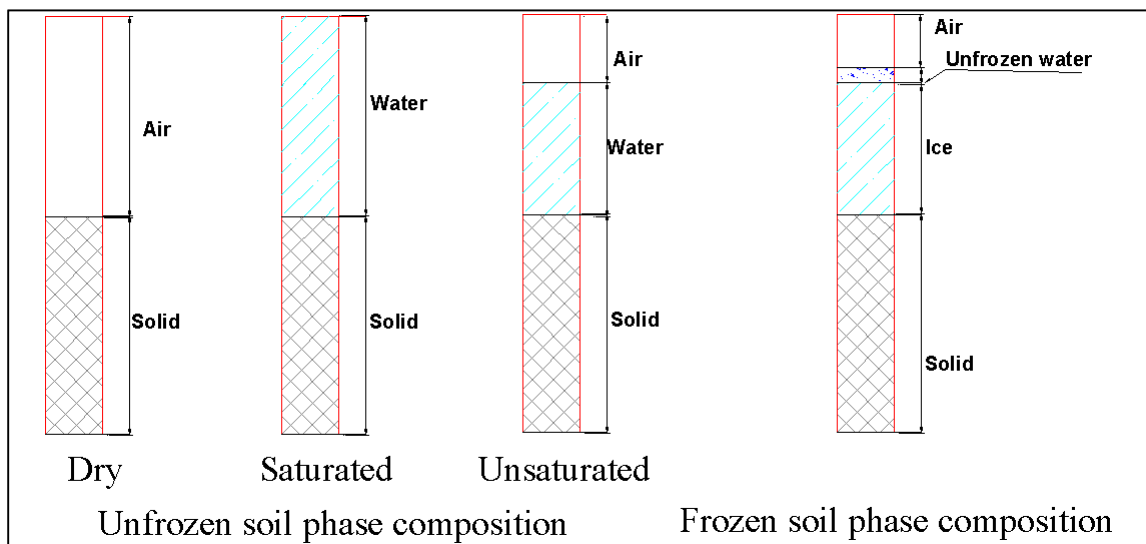


Figure 2.8 Schematic diagrams of the phase compositions of unfrozen and frozen soils

The parameters used to describe the physical properties of frozen soils are: temperature (θ), void ratio (e), porosity (n), total water content (w), unfrozen water content (w_u), ice content (w_i), density (ρ), specific gravity (G_s), etc. More terms can be found in ASTM D7099-04 (reapproved 2010). Currently, ASTM-D4083-89 (reapproved 2007) is used to

describe frozen soils and there is no frozen soil classification system based on physical properties generally used in the USA and Canada.

2.4.2 Unfrozen Water Content in Frozen Soils

Bouyoucos (1921) and Parker (1922) classified soil water into three categories: gravitational, free, and unfree waters. Unfree water can be further classified as capillary-adsorbed and combined waters, and the latter is further divided into water of solid solution or water of hydration. Bouyoucos (1921) found that up to 40% of added water can still be unfrozen even if the temperature is -78°C for many clay or clay loams. However, for quartz sand or sand, all water added readily froze at temperatures slightly below 0°C . Nerseova and Tsytovich (1963) identified three states of soil water: gaseous, bounded (weakly or strongly) and free. They described weakly bounded water as adsorbed water which changes phases when subjected to sub-zero temperatures and strongly bounded water as that which does not go through phase changes, even at sub-zero temperatures. Free water freezes at 0°C in the atmosphere.

It is well known that unfrozen water can significantly affect the physical (e.g., thermal and hydraulic properties) and mechanical properties (e.g. strength, creep behaviour) of frozen soils (Nerseova and Tsytovich, 1963; Williams, 1964). The amount of unfrozen water can be important, even at temperatures lower than 0°C . Unfrozen water content in frozen soil is influenced by factors such as the temperature of the soil, pressure, specific surface area of the soil grains, the mineralogical and chemical compositions of the soil, arrangement of the solid particles, density, and solute concentration and composition of

the pore fluid. These results have been published in the literatures by several researchers (e.g., Nerseova and Tsytovich, 1963; Williams, 1964; Anderson and Tice, 1972; Black, 1995). The reason that water in soil does not freeze at below bulk freezing temperature is mainly due to the melting point depression of water, which is caused by adsorption forces and curvature at the particle surfaces (Watanabe and Mizoguchi 2002). Since Faraday (1859) stated that unfrozen water might be present on the surface of ice, several studies that dealt with the surface melting of ice have been carried out (Dash et al., 1995). It is now known that there is a film of unfrozen water between the ice and the particle surface in frozen porous media, which decreases the free energy of the media. The melting point of water is further depressed if it includes solutes, and this is a function of the solute concentration. These effects must be considered when considering the behavior and amount of unfrozen soils (Watanabe and Mizoguchi 2002).

Unfrozen water content can be directly measured by using dilatometry (Bouyoucos, 1916; Koopmans and Miller, 1966), adiabatic calorimetry (Williams, 1964; Smith and Patterson, 1983), heat capacity (Nereseova and Tsytovich, 1963), nuclear magnetic resonance (Wu, 1964), X-ray diffraction (Anderson and Hoekstra, 1965), differential thermal analysis (Anderson and Tice, 1971) and isothermal calorimetry (Anderson and Tice, 1973). These methods have been reviewed and summarized by Anderson and Morgenstern (1973). Smith and Patterson (1980) reported on the test results when using time domain reflectometry (TDR) and the advantages of the method compared to others. Each of the aforementioned direct measuring methods has made assumptions and thus has some limitations in its application.

As direct measuring method equipment and apparatus are usually complicated and most direct methods are time consuming and costly, many researchers have made various assumptions based on indirect methods or equations to predict the unfrozen water content. Dillon and Andersland (1966) suggested the use of specific surface area, temperature, the activity ratio of the soil (A_c), and the freezing point depression of the pore water as input parameters. The relationship between unfrozen water content and all the aforementioned factors is given in Eq. 2.3 (Dillon and Andersland, 1966).

$$w_u = \frac{ST}{T_0} \cdot \frac{1}{A_c} \cdot l \cdot k \cdot 100 \dots \dots \dots (2.3)$$

where w_u = unfrozen water content, % dry wt.,

S = average specific surface area of the soil particles, $M^2/gm.$,

T = temperature of frozen soil, $^{\circ}K$,

T_0 = temperature of initial freezing of soil pore water, $^{\circ}K$,

$A_c = \frac{I_p}{\% < 2\mu} =$ activity ratio

I_p = plasticity index,

$l = 1$ for non-expandable clays, 2 for expanding clays,

$k =$ a constant $= 2.8 \times 10^{-4}$, gm. of water/ M^2

Anderson and Tice (1972) provided similar methods based either on specific surface area or temperature for their predictions. Liquid limit (LL) had also been used as a parameter for the prediction of unfrozen water content in frozen soil (Tice et al., 1976).

Tice et al. (1976) provided a simple power equation as given in Eq. 2.4 to predict unfrozen water content for field engineers to make quick decisions when there is no

sophisticated equipment available. Equation 2.4 is constructed based on the finding that there is correlation between the unfrozen water content and the LL of the soil.

$$w_u = \alpha \theta^\beta \dots\dots\dots (2.4)$$

where w_u = unfrozen water content, g (H₂O)/g (Soil)

α, β = empirical constants characteristic of a given soil

θ = temperature below 0°C expressed as a positive number, °C

Typical values for α and β for several soils are listed in Table 2.1. The application of this formula has been illustrated by Andersland and Ladanyi (2004).

Other authors also used thermodynamic principles to evaluate the phase composition of frozen soils. Most of them used their own version of the Gibbs free energy, Kelvin or Clayperon equations to model the three phase (water, ice and gas) system (Hivon, 1991). The key parameter for phase composition analysis is freezing temperature. Colbeck (1981) provided an equation in the form of Eq. 2.5 to evaluate the phase composition of a saturated soil system.

$$T_m = \frac{T_0}{L} \left(\frac{1}{\rho_l} - \frac{1}{\rho_s} \right) P \dots\dots\dots (2.5)$$

where T_m =freezing temperature, K

T_0 =freezing temperature of pure water, K

ρ_l = liquid (water) density, kg/m³

ρ_s = solid (ice) density, kg/m³

P = vapor pressure, kPa,

L = latent heat of fusion, kJ/kg

Table 2.1. Empirical constant characteristic - α , values for prediction of unfrozen water content - β (Andersland and Ladanyi, 2004)

Soil	Specific surface area (m ² /g)	A	β	Source
west Lebannon gravel	15	2.10	-0.408	Smith and Tice, 1988
Manchester silt	18	2.50	-0.515	''''
Kaolinite (kGa-1)	23	5.80	-0.864	''''
Chena silt	40	3.20	-0.531	''''
Leda clay	58	10.80	-0.649	''''
Morin clay	60	9.50	-0.479	''''
O'Brien clay	61	10.40	-0.484	''''
Goodrich clay	68	8.64	-0.456	''''
Tuto clay	78	12.80	-0.603	''''
Sweden VFB 478 clay	113	27.10	-0.472	''''
Suffield silty clay	148	11.10	-0.254	''''
Frederick clay	159	14.0	-0.297	''''
Ellsworth clay	184	11.2	-0.293	''''
Regina clay	291	21.1	-0.238	''''
Niagara silt	37	6.60	-0.41	''''
Norway LE-1 clay	52	9.90	-0.523	''''
Kaolinite #7	72	19.8	-0.689	''''
Althena silt loam	83	6.0	-0.301	''''
Sweden CTH 201 clay	106	19.7	-0.492	''''
Hectorite	419	38.4	-0.369	''''
Volcanic ash	474	3.1	-0.097	''''
Fairbank silt	40	4.8	-0.326	Anderson, Tice & McKim, 1973
Hawaiian clay	382	32.42	-0.243	Anderson, Tice & McKim, 1973
Umiat bentonite	800	67.55	-0.343	Anderson, Tice & McKim, 1973
Wyoming bentonite	800	55.99	-0.29	Anderson & Tice, 1972
Basalt	6	3.45	-1.13	Anderson & Tice, 1972
Morin clay	60	13.1	-0.505	Oliphant, Tice, & Nakano, 1983
Caen silt	-	9.5	-0.227	Smith 1984
Calgary silt	-	9.6	-0.364	Patterson & Smith, 1981
Allendale clay	-	15.7	-0.187	Patterson & Smith, 1981
Inuvik clay	-	14.5	-0.254	Smith 1985
Tomokomai clay	54	19.5	-0.305	Kay et al., 1981
Japanese clay (45%)	-	12.8	-0.402	Akagawa 1988

Miller (1980) provided an equation in the form of Eq. 2.6 to predict the freezing temperature of a soil-water-ice-gas system. These equations are useful for estimating the freezing temperature while preparing frozen soil specimens for designed tests in the lab.

$$\left(\frac{L}{273}\right) T = \frac{P_{H_2O}}{\rho_w} - \frac{(P_i)}{\rho_i} \dots\dots\dots (2.6)$$

where L= latent heat of fusion, kJ/kg

T= freezing temperature, K

P_{H_2O} = vapor pressure in free water, kPa

P_i = vapor pressure in ice, kPa

ρ_w = water density, kg/m³

ρ_i = ice density, kg/m³

The above analysis on the estimation of unfrozen water content shows that water content (w) is an important parameter in preparing frozen tailings backfill samples to carry out research on the properties and behaviour of FMB or frozen soil. The added water must be accurately measured and the water should be equally distributed in the tailings or soil.

2.4.3 Deformation Behaviour of Frozen Soils

The special features of the geotechnical properties of frozen soil are dominant creep characteristics under sustained loading and the marked strain rate dependence of the strength. These special features are mainly attributed to the ice and unfrozen water that co-exists in the frozen soil and the various responses of pore ice to loading which can

range from pressure melting to brittle failure, yet another contributing factor (Anderson and Morgenstern, 1973).

The failure characteristics of frozen soil result in both ductile and brittle behaviours when subjected to compressive stresses under different conditions (Vyalov, 1963; Bragg and Andersland, 1981; Zhu and Carbee, 1984; Wijeweera and Joshi, 1990; Andersland and Ladanyi, 2004; Scripta Technica, 1973; Wijeweera, 1990). It is well accepted that the total strain of frozen soil under loading is composed of an instantaneous portion and a time-dependent portion. Furthermore, each of these portions contains a reversible and an irreversible component (Andersland and Landayi, 2004). Many models have been presented to describe the deformation behaviour of frozen soil under various loading conditions. Vyalov (1963) proposed a rheological model to define the total creep strain as given by Eq. 2.7 (Hivon, 1991). Many of later proposed equations are somewhat modifications of this one based on the observations of a peculiar soil or test conditions or both.

$$\varepsilon = \varepsilon_0 + \varepsilon_1(t) + \varepsilon_2(t) + \varepsilon_3(t) \dots \dots \dots (2.7)$$

where ε = total strain

ε_0 = instantaneous elastic strain represented by a spring

ε_1 = visco-elastic strain represented by a spring in parallel with a dashpot

ε_2 = visco-plastic strain represented by a braking element in series with a dashpot

ε_3 = failure strain

Bragg and Andersland (1981) reported a larger initial tangent modulus which corresponds to larger sample diameter even though the length to diameter ratio was constant. They observed that large failure strains occur while the strain rate is slow and attributed this to pressure melting, water migration and refreezing phenomena.

Zhu and Carbee (1984) found that the failure strain of frozen silt (Fairback silt) is not sensitive to temperature and strain rate within some ranges of strain rates, but very sensitive to the dry density of the frozen silt. Moreover, the temperature has less influence on the initial yield strain even if it is deeply influenced by strain rate.

2.4.3.1 The Effect of Soil Type and Concentration of Solid Particles

Lai et al. (2007) studied the stress-strain relationship of frozen sands and presented a nonlinear Mohr strength criterion based on a series of triaxial tests. They reported that an apparent strain softening phenomenon is observed when the applied confining pressure is less than 3.0 MPa. However, strain hardening occurs when the applied confining pressure is higher than 3.0 MPa. The authors stated that the presented nonlinear Mohr strength model can better describe the strain softening behaviour in comparison to the Duncan-Chang model. Moreover, the strain hardening phenomena is better than the general hyperbolic model.

Sayles and Carbee (1981) reported that there is an increase in strength with increasing the volume of ice per unit volume of the soil mass (V_i/V) and provided an equation (Eq.2.8) to calculate the ratio of V_i to V by qualitatively studying the influence of water content and dry unit weight on the strength and deformational characteristics of frozen soils.

$$\frac{V_i}{V} = \frac{\omega - \omega_u}{G_i \gamma_w} \dots\dots\dots (2.8)$$

where:

v_i = the volume of the ice in the frozen soil, m^3

v = the volume of the soil mass, m^3

ω = total water content, (%)

ω_u = unfrozen water content, (%)

G_i = the specific gravity of ice (0.917)

γ_w = the unit weight of water, (kN / m^3)

Many authors (Sayles and Carbee, 1981; Zhu and Carbee, 1984; Wijeweera, 1989) have studied the geotechnical behaviours of frozen silt and frozen clay. The coupled effect of temperature and strain rate has been the most frequently studied.

2.4.4 Strength of Frozen Soils

Factors that control the compressive strength of unfrozen soil have been fully summarized by Poulos (1989) as: i) soil composition (basic soil material), termed mineralogy, grain size and grain size distribution, shape of particles, pore fluid type and content, ions on grain and in pore fluid; ii) state (initial), termed void ratio, stress history; iii) structure, the arrangement of solid particles within the soil; and iv) loading conditions.

Compared to unfrozen soil, the compressive strength of frozen soil is also attributed to the ice formed in the pores and the temperature. The factors which govern the

compressive strength of frozen soils have been classified as internal factors and testing conditions. The internal factors are summarized as shape and grain size distribution of solid particles (soil type), volume concentration of solid particles (dry unit weight of the soil), and total and unfrozen water contents. The testing conditions are differentiated by temperature, strain rate, and stress state.

The strength of frozen soil comprehensively reflects the pore ice strength (cohesion), friction and cohesion of solid particles and soil-ice bonding effect (Hivon, 1991) in contrast to the strength of unfrozen soil, which reflects the cohesion and friction effect among solid particles.

2.4.4.1 The Effect of Temperature and Strain Rate

Bragg et al. (1981) reported that low temperature and high strain rate contribute to high initial yield stress (σ_y), compressive peak strength (σ_m), and initial tangent modulus of frozen sand cylindrical samples. They observed that a prominent region of plastic strain hardening exists with no visible cracking or formation of shear planes that occur in the samples at strains well beyond the compressive strength where the strain rate is less than 10^{-5}s^{-1} . However, at high strain rates (larger than $4 \times 10^{-4}\text{s}^{-1}$) the stress-strain behaves linear until the compressive strength is reached and a brittle failure mode of the sample follows.

Zhu and Carbee (1984) studied the compressive strength of frozen silt with various deformation rates. They found that the peak strength of frozen saturated Fairbanks silt is very sensitive to temperature and strain rate. However, the failure strain is not sensitive to the above parameters within a large strain rate range. Another important finding is a large deformation property, and axial strains up to 30% for most samples of frozen silt without visible fractures were observed.

Zhu et al. (1998) presented a constitutive relation model to simulate the effect of temperature and strain rate on the strength behaviour of frozen saturated silt (Lanzhou loess). The provided model (Zhu et al., 1998) is given in Eq. 2.7, which is constructed based on a regression analysis on the constant strain-rate uniaxial compression test results. The temperatures used for this study are: -2°C, -5°C, -10°C, and -15°C. The strain rates are: $1.1 \times 10^{-6} \text{ s}^{-1}$, $9.3 \times 10^{-6} \text{ s}^{-1}$, and $7.1 \times 10^{-4} \text{ s}^{-1}$.

$$\sigma = 1.123 \dot{\epsilon}^{0.089} \left(\frac{\theta}{\theta_0}\right)^{0.887} \left(\frac{\epsilon}{\epsilon_y}\right)^{0.524} \dot{\epsilon}^{0.055} \left(\frac{\theta}{\theta_0}\right)^{-0.387} \dots\dots\dots (2.7)$$

where σ = the stress, MPa

$\dot{\epsilon}$ = strain rate, s^{-1}

θ = the negative temperature of the frozen soil, °C

θ_0 = a reference temperature, -1°C

ϵ = strain

ϵ_y = yield strain, independent to the strain rate

2.4.4.2 The Effect of Confining Pressure

Frozen soils can be hardened or softened at high confining pressures because it increases the friction between soil particles which is beneficial for the high strength and low deformation behaviour of frozen soils. However, during the same process, high confining pressure also decreases cohesion between the interfaces of ice and solid particles, and increases the unfrozen water content, which has negative effects on the expected mechanical behaviour of frozen soils (He et al. 1999). He et al. (1999) provided a constitutive theory for frozen soil to describe its failure mode and govern its strength properties.

Zhang and Kushwaha (1998) provided a radial function (FBF) model to simulate the strength behaviours of frozen sand by using confining pressure and dry density as inputs. The model could perfectly simulate the strength behaviour of frozen soils with soil densities of 1600 kg/m^3 and 1900 kg/m^3 . However, there was no experimental data given to verify the effectiveness of the model although a “reasonable tendency” is shown in the figures (Zhang and Kushwaha, 1998). The authors also used a similar FBF model to simulate the behaviour of frozen silt and the inputs were strain rate and soil temperature. The domain of the strain rate was 10^{-7} s^{-1} to 1 s^{-1} . The temperature ranged from -0.5°C to -10°C .

2.5 Background on Cement Hydration

One of the components of FCPB is cement. It is known that cement hydration significantly affects the mechanical behaviour of any cementitious porous medium (e.g., concrete, cemented soils, shotcrete). Hence, it is desirable to provide some background

information on cement and cement hydration for readers to better understand the mechanical behaviour of the FCPB studied in this thesis.

2.5.1 Cement- definition and compositions

In the most general sense of the word, cement is a binder, a substance that sets and hardens independently, and can bind other materials together (Wikipedia, web). Currently, the most frequently used cement is Portland cement, which has four primary types of minerals in the cement grain, including tricalcium silicate (Ca_3SiO_5), dicalcium silicate (Ca_2SiO_4), tricalcium aluminate ($\text{Ca}_3\text{Al}_2\text{O}_5$), and calcium aluminogerrite. An abbreviation nomenclature, shown in Table 2.2, based on the oxides of various elements, is created to indicate the chemical formulas of the main compounds (Taylor 1997; Barron 2010). Gypsum is also added to the clinker of Portland cement before grinding to control the rate of hydration of aluminates which helps to avoid flash set.

Table 2.2 Chemical formulas and cement nomenclatures for major constituents of Portland cement

Mineral	Chemical formula	Oxide composition	Abbreviation
Tricalcium silicate (alite)	Ca_3SiO_5	$3\text{CaO}\cdot\text{SiO}_2$	C_3S
Dicalcium silicate (belite)	Ca_2SiO_4	$2\text{CaO}\cdot\text{SiO}_2$	C_2S
Tricalcium aluminate	$\text{Ca}_3\text{Al}_2\text{O}_4$	$3\text{CaO}\cdot\text{Al}_2\text{O}_3$	C_3A
Tetracalcium aluminoferrite	$\text{Ca}_4\text{Al}_n\text{Fe}_{2-n}\text{O}_7$	$4\text{CaO}\cdot\text{Al}_n\text{Fe}_{2-n}\text{O}_3$	C_4AF
Abbreviation notation: C=CaO, S=SiO ₂ , A=Al ₂ O ₃ , F=Fe ₂ O ₃			

(Source: <http://cnx.org./content/m16445/latest>)

The compositions of cement vary from plant to plant, usually depending on the location of the plant and the mother materials (mainly limestone and clay). Taylor (1997) showed that a typical clinker (Portland cement) has a composition of 67% CaO, 22% SiO₂, 5% Al₂O₃, 3% Fe₂O₃, and 3% other components. A cross-section view is shown in Fig. 2.9 which schematically illustrates the composition of a cement grain.

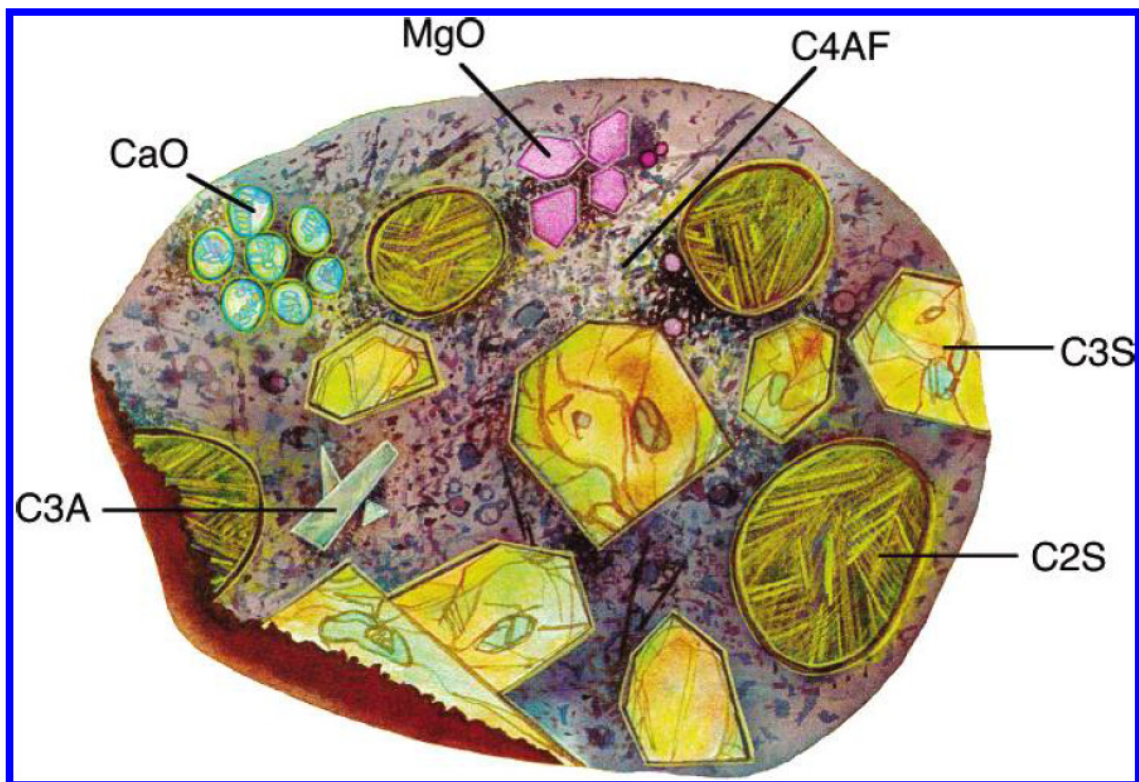


Figure 2.9 A pictorial representation of the cross-section of a cement grain. (Source: <http://cnx.org/content/m16445/latest>)

2.5.2 Cement hydration

The strength of cement or cemented materials (e.g., concrete, mortar, CPB) is attributed to the products of cement hydration. Generally speaking, cement hydration is defined as a

chemical combination of cement with water. “ In cement chemistry, the term ‘hydration’ denotes the totality of the changes that occur when an anhydrous cement, or one of its constituent phases, is mixed with water” (Taylor, 1997). Even though the definition is not difficult to understand, it is well accepted that cement hydration is a complex chemical reaction process and there is no single chemical reaction equation used to represent the whole process because it is not completely understood (Odler, 1998). In the presence of water, the aforementioned components of Portland cement react to form hydration products (e.g., calcium silicates hydrates, C-H-S; calcium hydroxides, CH; tricalcium aluminate hydrate, C_3AH_6 , and calcium sulphoaluminate (ettringite)) which form a firm and hard mass with time.

The short term strength of cement or cemented materials is mainly due to the hydration products of C_3S . Moreover, the hydration products of C_2S dominant the long term strength of cement or cemented materials (Taylor, 1997; Ylmen et al., 2009). Typical curves of compressive strength development versus the phase hydration process in pure cement pastes are given in Figure 2.10.

The differentiation of short and long term strength for cement or cemented materials is mainly due to the partition of the hydration periods of cement which is usually in three periods: early, middle and late periods of hydration which are related to cement hydration rates (Taylor, 1997; Ylmen et al., 2009). The hydration rates can be affected by many factors such as the curing temperature and additives (Odler, 1998). Recent monitoring results on the initial and final setting times are reported to be 180 and 240 minutes, respectively (Ylmen et al., 2009).

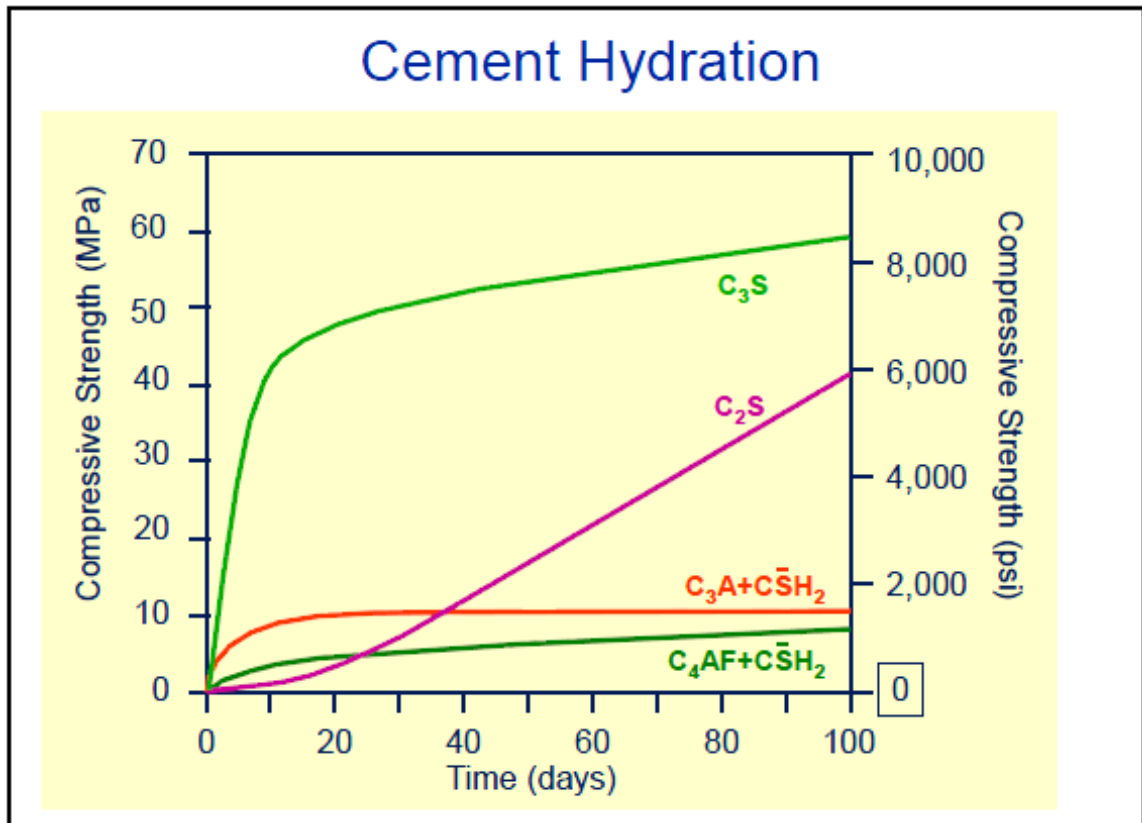


Figure 2.10 Compressive strength development in pastes of pure cement compounds

(Source: Mindess et al., 2003; Kurtis, 2007)

2.6 Conclusion

Mine tailings backfill (FTB and/or FCPB) used for filling voids left by mining processes can be a cost-effective underground support system and an environmentally beneficial material. The mechanical properties (UCS and E) and mechanical behaviour (stress-strain curves) of FTB could be related to those of frozen soils. Furthermore, the mechanical properties and behaviours are strongly affected by those of ice and snow. The theories, test methods, and test equipment/apparatus for frozen soils are important tools for doing

research on FTB. As for studies on FCPB, basic knowledge about cement and cement hydration is also useful for better understanding its strength formation, geotechnical properties and behaviour.

The mechanical properties of frozen soil are governed by internal factors and can be affected by testing conditions, such as confining pressure, loading rate, length to height ratio, sample size, etc. A special phase, in which unfrozen water exists in frozen soil, is an important factor on the geotechnical behaviour of frozen soil. The failure mode behaviours of frozen soil can be plastic or brittle, mainly depending on the temperature and the loading rate applied.

The uniaxial strength and deformation behaviour of cemented backfill is affected by factors such as cement types and content, w/c, tailings fineness, sulphate content, temperature and curing time, the testing conditions such as loading rate, confining pressure. The ice content and properties could have great impacts on its strength and deformation behaviours. Moreover, these behaviours are very sensitive to the fluctuation of temperature.

The mechanical properties (i.e., strength and stiffness parameters) of frozen soil are complicated because frozen soil has complex components and is sensitive to temperature, water content and pressure. The stress-strain behaviours of frozen soils can be dilatants, ductile, or brittle depending on internal factors, such as volumetric ice content and

external conditions, such as loading rate of the testing (Razbegin, Vyalov, Maksimyak, and Sadovskii, 1996; Arenson et al., 2007).

In contrast to the deformation and strength behaviours of unfrozen soils which are dependent on interparticle friction, particle interlocking and cohesion, ice bonding could be a dominant strength factor for frozen soils or porous medium as stated in ASTM D 7300-06. In terms of fine-grained soils (clay and silts), the mechanical behaviours of frozen soil under loading are similar to those of ice when solid concentrations are less than 50%. However, for low ice content frozen soils or porous media, interparticle friction will contribute to the strength, and unfrozen water films that exist in frozen soils will be an important factor.

Cement is a binder, a substance that independently sets and hardens, and can bind other materials together to form different materials (concrete, mortar). Cement hydration products are responsible for the strength development of cement or cemented materials. In the early age, C_3S (alite) hydration is the main contributor for cement strength formation, but long term strength development is mainly due to the hydration of C_2S . The initial setting time for cement is reported to be as short as 180 minutes, but the strength development progress can last 150 days or more.

From the literature review presented above, it can be concluded that a significant amount of technical data are available with regards to the geotechnical properties or behaviours of frozen soils and cemented backfill. However, there is a paucity of data with regards to the strength and deformation behaviours of FCPB and FTB. Therefore, the following

chapters deal with the experimental investigations of the strength and deformation behaviour of frozen mine backfill (FCPB, FTB).

CHAPTER 3:

STRENGTH AND DEFORMATION BEHAVIOUR OF

FROZEN CEMENTED PASTE BACKFILL

3.1 Introduction

CPB, first used in Germany in the 1970s as mine construction materials, has gained extensive application in the past decade because of its economical and environmental benefits (Jeremic and Prudhomme, 1984; Archibald and James 1986; Fall and Samb, 2006; Célestin and Fall, 2008). Fresh CPB material exhibits a slump that is generally between 150mm and 250mm and solids concentration about 70-85% (Fall and Samb, 2006).

Once placed, CPB must satisfy some technical requirements for ensuring safer mining conditions and economical requirements. Generally, the technical design requirements in question are sufficient compressive strength (commonly 0.7-2.0 MPa), acceptable technical consistency, and high solids concentration (70-85%) (Archibald and Nantel, 1986; Bandopadhyay and Izaxon, 2004; Fall and Samb, 2008). However, in the underground support role, compressive strength is not the only concern. The stiffness and

stress-strain behaviour are also important parameters for optimal CPB designs (Fall, 2008).

Several laboratory studies have been performed in the past few decades to understand the strength development and deformation behaviour of CPB (Fall and Samb, 2006; Fall et al., 2007; Fall and Pokharel, 2008). Most of those studies were performed on CPB cured at room temperature or temperatures higher than 20°C. This means that our understanding of the strength development and deformation behaviour of CPB cured in sub-zero temperatures is limited. There is a crucial need to acquire sufficient technical data on the aforementioned properties of CPB because there is increasing mining activities in permafrost regions.

By performing laboratory tests, the objectives of this chapter are to:

- study the strength development of FCPB;
- investigate the deformation behaviour (stress-strain behaviour, E) of FCPB; and
- assess the effect of the main mix components on the strength and deformation of FCPB.

A series of FCPB specimens with various combinations of compositions (binder type, cement amount, and curing age) are prepared and tested in the laboratory under sub-zero conditions. Valuable data have been obtained. This information will be beneficial to mine backfill designers, the mining industries as well as the public.

3.2 Experimental Program

3.2.1 Materials used

The materials used for this study are artificial tailings, distilled water and binders.

3.2.1.1 Tailings

Commercially available artificial tailings (silica tailings, SI), known as SILT-CO-SIL, were used to prepare the samples. These artificial tailings are made from ground silica which contains 99.8 % of SiO₂ (quartz). The solid particle of the artificial tailings is very similar to the physical characteristics of the natural tailings available at the mining sites in eastern Canada (Fall and Samb, 2006). The grain size distribution of the tailings is given in Fig. 3.1. The average grain size distribution of natural tailings from 9 Canadian mines of eastern Canada is also shown in the same figure for comparison. The main objective of using artificial tailings is for an accurate control of the mineralogical and chemical compositions of the tailings. This means that there will be no chemical changes within the specimens during the entire study process from material preparation to specimen testing. Furthermore, natural tailings may contain varying amounts of sulfate and other undesirable minerals which can bring about uncertainties in the results obtained and their interpretation. The physical properties and main chemical elements of the used tailings are given in Tables 3.1 and 3.2. It can be noted from Fig. 3.1 that silica tailings has about 45% fines. The term fine is defined as the size of the solid particles that is less

than 20 μm . Moreover, Fall et al. (2009) tested SI tailings for index properties (e.g., liquid and plastic limits) by following ASTM standards. From the experimental work, the used tailings (SI) were non-plastic and could be classified as sandy silts of low plasticity; ML in the Unified Soil Classification System (UCCS). ML is characteristic for tailings from hard rock mines as also measured by Vick (1990). The specific surface areas of these tailings were determined to be equal to $1.45 \text{ m}^2/\text{g}$ (Fall et al., 2009).

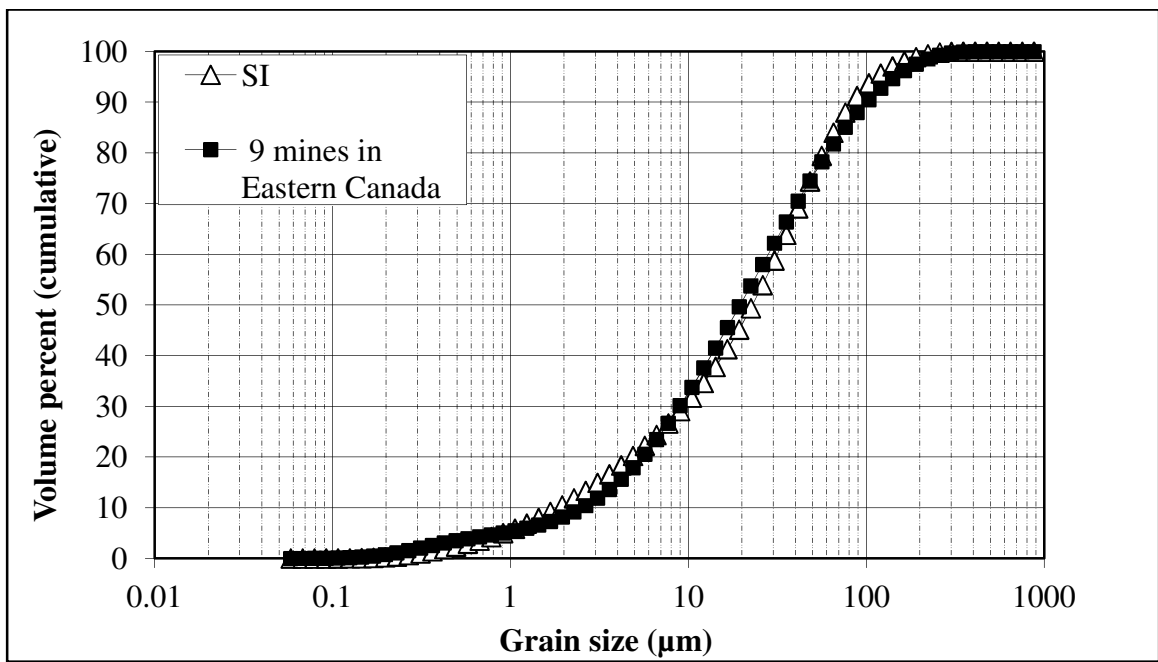


Figure 3.1 Grain size distribution curves for tailings used and that of the average of natural tailings from 9 mines in eastern Canada (Fall and Samb, 2006).

Table 3.1 Physical properties of the tailings (SI)

Element Unit	Gs	D ₁₀	D ₃₀	D ₅₀	D ₆₀	D ₉₀	C _u	C _c
	-	μm	μm	μm	μm	μm	-	-
	2.7	1.9	9.0	22.5	31.5	88.9	16.2	1.3

Table 3.2 Chemical properties of the tailings by percentage (Source: US Silica Company)

	SiO ₂	Fe ₂ O ₃	Al ₂ O ₃	TiO ₂	CaO	MgO	Na ₂ O	K ₂ O	LOI	pH
ST	99.8	0.035	0.05	0.02	0.01	<0.01	<0.01	0.02	0.1	7

3.2.1.2 Binders

Three types of binders were used to prepare the FCPB materials. These binders are Portland cement type I (PCI), PCI/FA (50/50)-mix of Portland cement (PCI) and fly ash (FA) with a mass to mass ratio of 50 to 50 (PFA), and PCI/Slag (50/50) - mix of Portland cement (PCI) and Slag with a mass to mass ratio of 50 to 50 (PSL). The main components and properties of the PCI and Slag binders are given in Table 3.3.

Table 3.3 Chemical and physical properties of the binders PCI and Slag used

Binder	MgO	CaO	SiO ₂	Al ₂ O ₃	Fe ₂ O ₃	SO ₃	Relative Density	Specific Surface (m ² /g)
PCI	2.65	62.82	18.03	4.53	2.7	3.82	3.1	1.3
Slag	10.98	41.14	34.23	9.54	-----	3.87	3.3	2.2

3.2.1.3 Mixing water

The mixing water used was distilled water. The purpose of using distilled water is that tap water may contain undesirable chemicals that can affect the behavior of FCPB. This potential existing small amount of chemicals may affect cement hydration and thus increase the uncertainty in the test results and their interpretation.

3.2.2 Preparation of the specimens

3.2.2.1 Design parameters and mixing of FCPB

The cement contents used for this study are 3.0%, 4.5%, and 6.0%, and the corresponding w/c is 11.1, 7.6 and 5.9. Table 3.4 lists the design details on the paste used to prepare the FCPB specimens. The slumps for these pastes are 18 cm, which is the most common slump of CPB in the practice.

The first step was to mix the binder and the tailings. This assures a uniform mix of binder and tailings without introducing interference factors which will influence the properties of the mix. As it is well known that the setting time of the cement is limited, if the water is first mixed with tailings or cement, possible impacts on cement hydration during the mixing process are unavoidable. If a blended binder is used (PCI blended with FA or Slag), the blending mix should be prepared first.

Table 3.4 Mix design for the investigation of the effects of binder, cement content and curing age on the strength and deformation behaviour of FCPB

Sample Name	Cement proportion	Binder Name	Blending ratio	W/C ratio	Curing age (days)
RPC-1	3%	PCI	100	w/c=11.1	7
RPC-2	4.50%	PCI	100	w/c=7.6	7
RPC-3	6%	PCI	100	w/c=5.9	7
RS-1	3%	PCI/Slag	50/50	w/c=11.1	7
RS-2	4.50%	PCI/Slag	50/50	w/c=7.6	7
RS-3	6%	PCI/Slag	50/50	w/c=5.9	7
RFA-1	3%	PCI/Fly Ash	50/50	w/c=11.1	7
RFA-2	4.50%	PCI/Fly Ash	50/50	w/c=7.6	7
RFA-3	6%	PCI/Fly Ash	50/50	w/c=5.9	7
DPC-1	3%	PCI	100	w/c=11.1	28
DPC-2	4.50%	PCI	(PCI)	w/c=7.6	28
DPC-3	6%	PCI	100	w/c=5.9	28
DS-1	3%	PCI/Slag	50/50	w/c=11.1	28
DS-2	4.50%	PCI/Slag	50/50	w/c=7.6	28
DS-3	6%	PCI/Slag	50/50	w/c=5.9	28
DFA-1	3%	PCI/Fly Ash	50/50	w/c=11.1	28
DFA-2	4.50%	PCI/Fly Ash	50/50	w/c=7.6	28
DFA-3	6%	PCI/Fly Ash	50/50	w/c=5.9	28
SPC-1	3%	PCI	100	w/c=11.1	90
SPC-2	4.50%	PCI	100	w/c=7.6	90
SPC-3	6%	PCI	100	w/c=5.9	90
SS-1	3%	PCI/Slag	50/50	w/c=11.1	90
SS-2	4.50%	PCI/Slag	50/50	w/c=7.6	90
SS-3	6%	PCI/Slag	50/50	w/c=5.9	90
SFA-1	3%	PCI/Fly Ash	50/50	w/c=11.1	90
SFA-2	4.50%	PCI/Fly Ash	50/50	w/c=7.6	90
SFA-3	6%	PCI/Fly Ash	50/50	w/c=5.9	90

The second step was to add a measured amount of water into the dry prepared material and manually mix in a container to achieve the desired consistency of paste. The mixing time was 7 minutes. The paste was then poured into curing cylinders with a diameter of

50 mm and height of 100 mm to obtain cylindrical specimens. The cylinders were shaken several times during the filling process to ensure that there were no trapped air bubbles. Generally, this step should be finished in not more than 7 minutes (Fall, 2008; Célestin, 2008). Then, the cylinders together with the samples were sealed and cured in a temperature and humidity controlled environment for designated periods of time. This will be further described in Section 3.2.2.2.

3.2.2.2 Curing samples

Figure 3.2 shows the freezer, and the temperature and humidity controlling and monitoring system. The temperature and the humidity in the freezer were -6°C and 67%, respectively. The freezing periods were 7, 28, and 90 days. The cylinder was cut off 3 days before the curing age expired and the test specimen obtained. The entire operation of cutting off the cylinder was finished in a cold room with sub-zero temperature. The specimen obtained was then completely covered with plastic film and cured in the same freezer for 3 more days. Plastic film used to cover the specimen can avoid moisture variation (evaporation) during the curing period. A specimen was ready for performing the UCS test after a 3 day curing period.



Figure 3.2 Sample curing equipment and control system

3.2.3 Mechanical tests

UCS testing is carried out according to ASTM D7300-06 in this study to evaluate the strength and deformation behaviour of the prepared cylindrical FCPB specimens. Each test was repeated at least twice (up to five times for some samples) to ensure the repeatability of the results.

An FCPB specimen was placed into a cold chamber which was then mounted onto the platen of the testing machine. A 0.15mm/min speed for head in compression was applied to the specimen at a temperature of -6°C for the duration of the testing. The applied load and the deformation of the specimen were automatically recorded by LabVIEW VIs. These data together with the initial size of the specimen were used to calculate the stress, strain, and draw the stress versus strain curves. The mechanical properties (strength and E) as well as the stress-strain behaviour of FCPB were obtained by analyzing the stress-

strain curves. The stress that corresponds to a strain of 20% was selected as the (peak) strength of the FCPB. A detailed description on the testing procedures and the data extraction and interpolation are given in Appendix A.

3.3 Results and discussions

A large amount of experimental data or results with regards to the strength and deformation behaviour of FCPB have been obtained. Typical examples of the results will be presented and discussed below.

3.3.1 Strength Development and Deformation Behaviour of FCPB

Figure 3.3 shows typical results of the UCS evolution with time of the FCPB made from 4.5% PCI, CPB made of 4.5% PCI and cured at room temperature (20°C) as well as normalized (with UCS of CPB cured at 20°C) strength development of FCPB. From this figure, it can be noticed that the strength of the CPBs cured at sub-zero temperature (-6°C) is much higher than that of CPB cured at room temperature regardless of the curing time (7, 28, 90 days). The strength of FCPB samples made from PCI (PCI-FCPB) is 10.3, 9.9 and 6.9 times that of CPB cured at 20°C after 7, 28 and 90 days, respectively. This large difference in strength between the FCPB and CPB samples can be explained by the strengthening effect of the ice formed within the FCPB. This strengthening effect results from the ice strength (cohesion) and the solid particles (tailings, hydrated cement products)-ice binding effect. Indeed, it is accepted that the strength of frozen porous geological media or soil is a combination of ice strength in the pores, matrix strength and

bonding at the interface between solid mineral particles and ice (Goughnour et al., 1968, Ting et al., 1983).

Furthermore, an analysis of Figure 3.3 indicates that the strength of the FCPB increases up to 28 days and then slightly decreases. The 7 day UCS of FCPB is approximately 25% and 20% lower than that of 28 day and 90 day-FCPB, respectively. This increase in the UCS from 7 to 28 days can be attributed to the fact that, besides the strengthening effect of the ice, the Portland cement hydration products (e.g., C-S-H, CH) which are formed within the FCPB provide additional strength to the FCPB. However, it should be assumed that the amount of hydration products formed after 28 days of curing will be low. This is because it is well known that cement hydration is inhibited by low temperatures (McIntosh 1956, Fall et al. 2009). The observed slight decrease in strength from 28 to 90 days can be explained by the negative impact of heat (causes partial melting of ice) produced by cement hydration on the strengthening effect of the ice. Indeed, because of the freezing curing conditions, cement hydration starts very slowly and thus produces a low amount of heat at the early ages of curing (up to 28 days) (McIntosh 1956, Fall et al. 2009). As the curing time advances (beyond 28 days), the amount of heat generated by cement hydration increases and becomes significant, thereby, resulting in the partial melting of the ice. Future quantitative mineralogical analyses of cement hydration products and determination of heat development within FCPB will help to support the aforementioned arguments.

From the results presented above, it can be concluded that at early ages (up to 28 days), the strength of the FCPB is mainly controlled by the strengthening effect of the ice. However, beyond 28 days of curing time, the heat produced by cement hydration starts to have a weakening effect on the strength of FCPB. These findings may have relevant practical implications with regards to the stability and thus the design of field FCPB structures in cold or permafrost environments. Indeed, since the amount of heat produced within a cemented backfill increases as its size increases (e.g., Nasir and Fall 2009), it can be concluded that the aforementioned weakening effect will be more significant and start at a younger CPB age as the size of the CPB increases. Furthermore, considering the relatively low thermal conductivity of CPB (Celestin and Fall, 2009), it should be assumed that a large part (especially the middle part) of the field CPB structure will remain unfrozen, thereby showing lower strength (lower mechanical stability) than the frozen CPB.

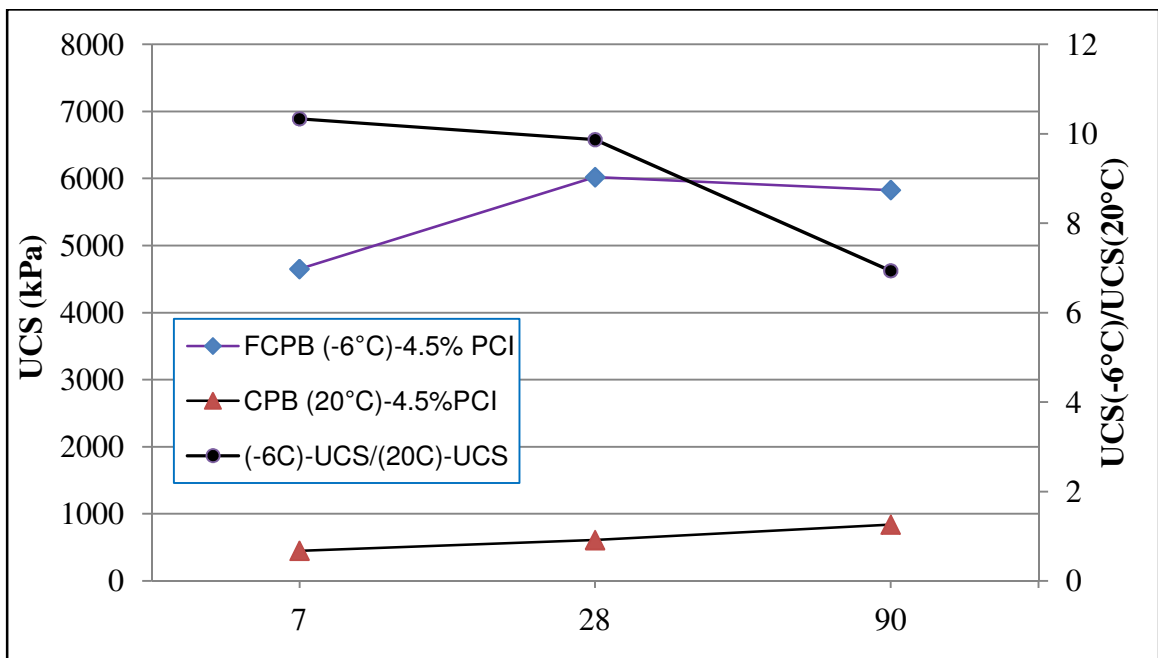


Figure 3.3 Strength development of FCPB and CPB as well as relative strength of FCPB to CPB cured at 20°C (source of data for the CPB: Fall et al. 2009; CPB Mix: Tailings: SI, 4.5% PCI, w/c=7.6)

To illustrate the influence of sub-zero temperatures on the deformation behaviour of CPB, typical stress-strain curves of PCI-FCPB with 4.5% cement after 7 and 90 days of curing are given in Figure 3.4, whereas the 4.5% PCI-FCPB modulus of elasticity is plotted against curing time (7 to 90 days) (Figure 3.5). In addition, the stress-strain curves and the modulus of elasticity of CPB made of 4.5% PCI and cured at room temperatures are shown in Figures 3.4 and 3.5, respectively, for comparison purposes. From Figure 3.4, it is clear that, independent of the curing age, the stress-strain curve of FCPB is significantly different from that of CPB. The stress-strain behaviour of FCPB samples is characterized by ductile behaviour with strain hardening, whereas the CPB shows strain softening behaviour. The CPB shows a well defined peak stress, whereas there is no apparent peak stress for the FCPB. The peak stress (strength) of FCPB is taken as the stress at 20% strain in this study. The main difference in the deformation behaviour of FCPB and CPB is believed to be due to the ice in the FCPB (Arenson and Springman, 2007). From Figure 3.4, it can be seen that the initial yield strain for the FCPB samples is approximately 2%, independent of the ages. It should be emphasized that in a typical stress-strain curve, the yield point is considered as the point at which the initial slope of the curve starts to noticeably decrease. This value of 2% is higher than the yield strains (1%) for most frozen soils. The yield point of frozen soil is commonly viewed as the consequence of the failure or fracture of ice matrix within frozen soil (e.g., Mellorm and

Cole 1982, Wijeweera and Joshi 1989). Consequently, the higher value of yield strength observed for FCPB could be attributed to the cementing effect of the cement hydration products.

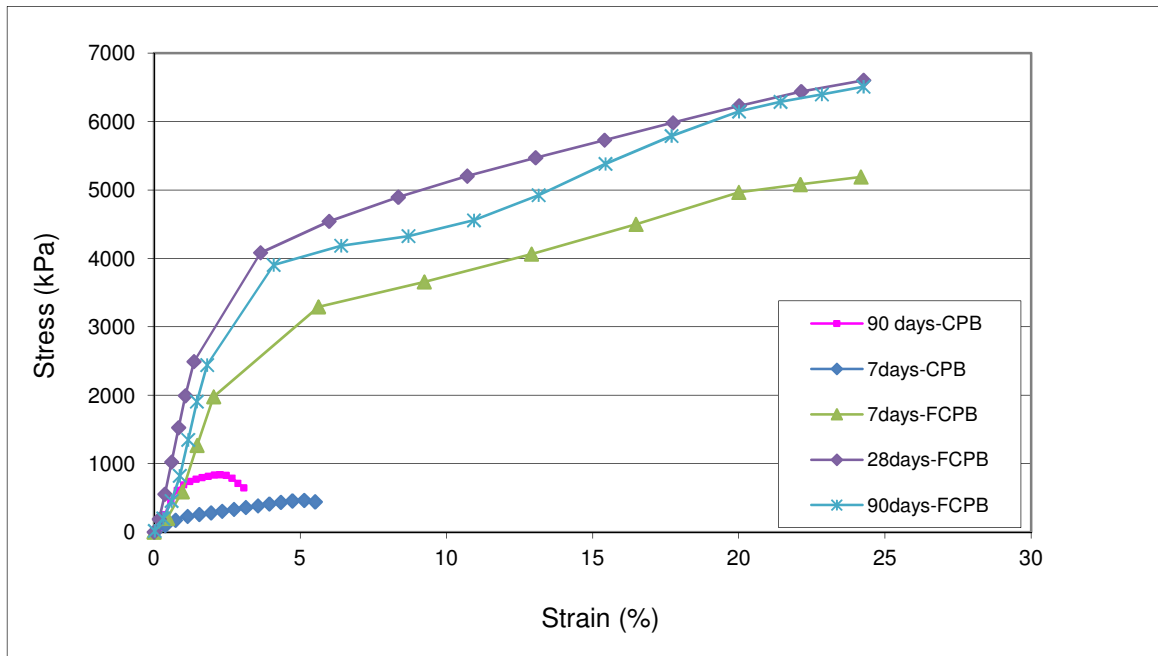


Figure 3.4 Typical stress-strain curves of FCPB and CPB (source of data for the CPB: Fall et al. 2009; CPB Mix: Tailings: SI, 4.5% PCI, w/c=7.6)

Typical results of the development of the modulus of elasticity of FCPB and CPB made of 4.5 PCI are given in Figure 3.5. The evolution of the modulus of elasticity is relatively similar to that of the strength described earlier. There is a significant difference between the modulus of elasticity of FCPB and CPB, and the modulus of elasticity of FCPB increases first up to 28 days of curing, and then decreases. The mechanisms responsible for this behaviour have already been explained above.

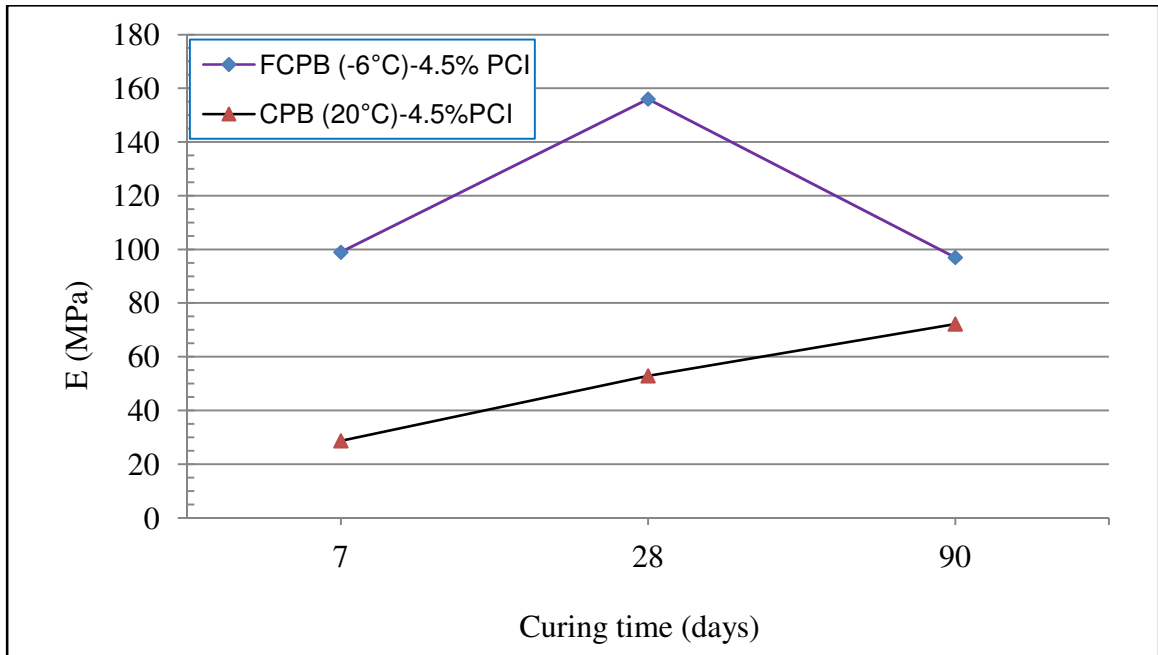


Figure 3.5 Modulus of elasticity of FCPB and CPB (source of data for the CPB: Fall et al. 2009; CPB Mix: Tailings: SI, 4.5% PCI, w/c=7.6)

3.3.2 Effect of binder content on the strength development and deformation behaviour of FCPB

Cement content has a significant effect on the strength development and cost of CPB (e.g., Grice 2001, Fall et al., 2008; Nasir and Fall, 2010). If a CPB with a small amount of binder can satisfy the mechanical requirements (strength, stiffness, durability), thus a significant cost saving can be achieved in the design of the CPB structure. This is because the cost of the binder can represent up to 75% of that of the CPB (Grice 2001). Thus, there is the need to study the relationship between binder content and mechanical behaviour (strength development as well as deformation behaviour) of FCPB. In order to assess the effect of cement content on the strength and deformation behaviour of FCPB,

FCPB specimens with binder content of 3.0%, 4.5% and 6.0% are prepared and tested in the laboratory. Selected results are presented in Figures 3.6 to 3.8 and discussed below.

In Figure 3.6, the strength of FCPB made of PCI is plotted against the curing time (7, 28 and 90 days) for different cement content (3, 4.5, and 6%), while Figure 3.7 illustrates the development of modulus elasticity of the FCPB with various binder content. An analysis of Figures 3.6 and 3.7 indicates that the cement content has no significant effect on the general behaviour of the development curve for strength and elasticity modulus. This means, for any binder content, the strength and modulus elasticity increase up to 28 days and then decrease. The reasons have been given earlier. However, an interesting finding in Figure 3.6 is that the 28 day and 90 day-strength of the 3%-FCPB samples, despite lower cement content, are around 8% higher those of the 4.5%- and 6.0%-FCPB. This can be explained by the fact that as the cement content increases, the amount of heat produced by the cement hydration increases, thereby resulting in a more negative effect of the strengthening effect of ice due to the melting of a larger portion of ice in the FCPB. From a practical point of view, these results suggest that in the case of freezing curing temperatures, the use of a lower amount of cement will provide higher strength to the CPB than higher cement content. This is obviously associated with a significant reduction of the binder consumption, i.e. the cost of the CPB.

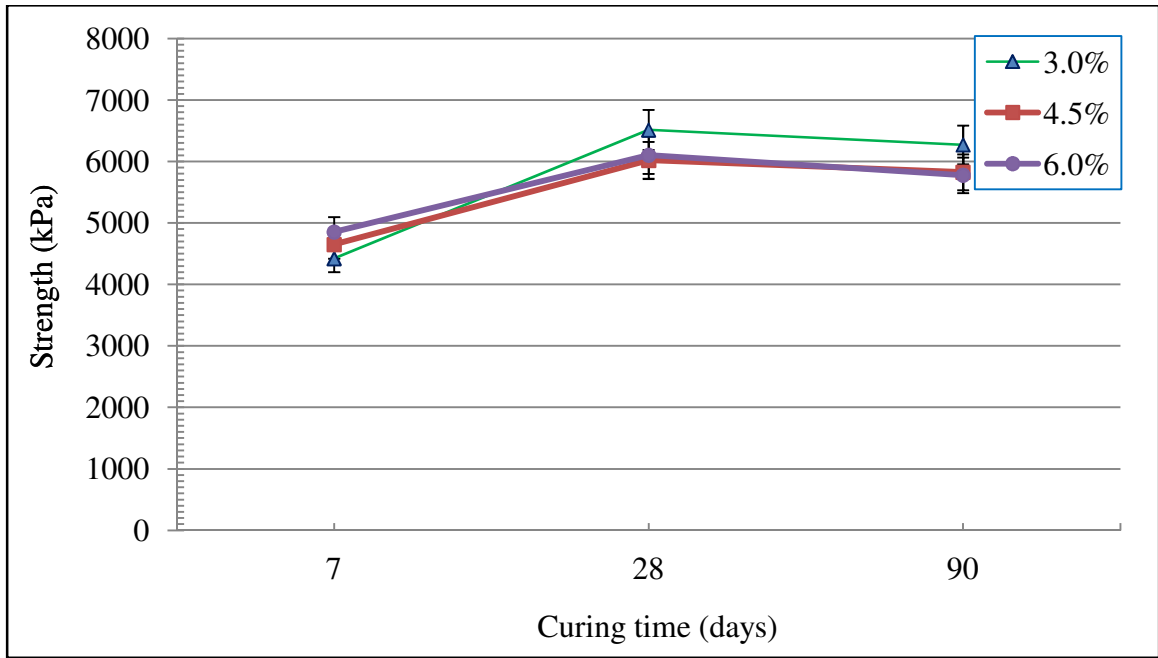


Figure 3.6 Effect of cement content on the strength development of PCI-FCPB

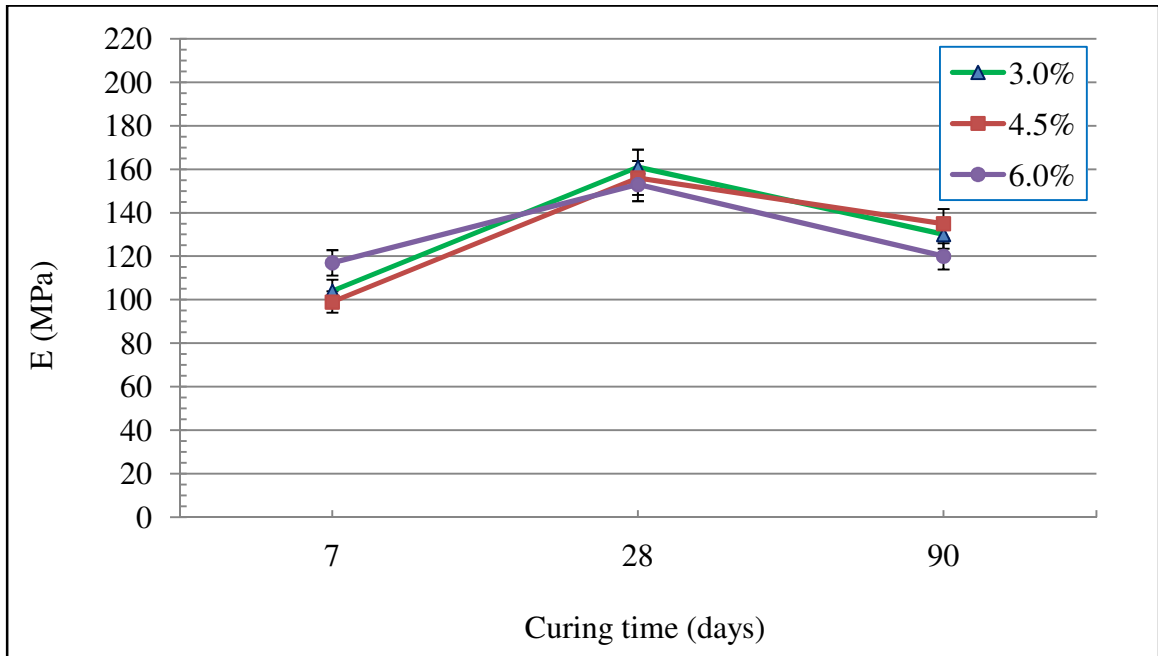
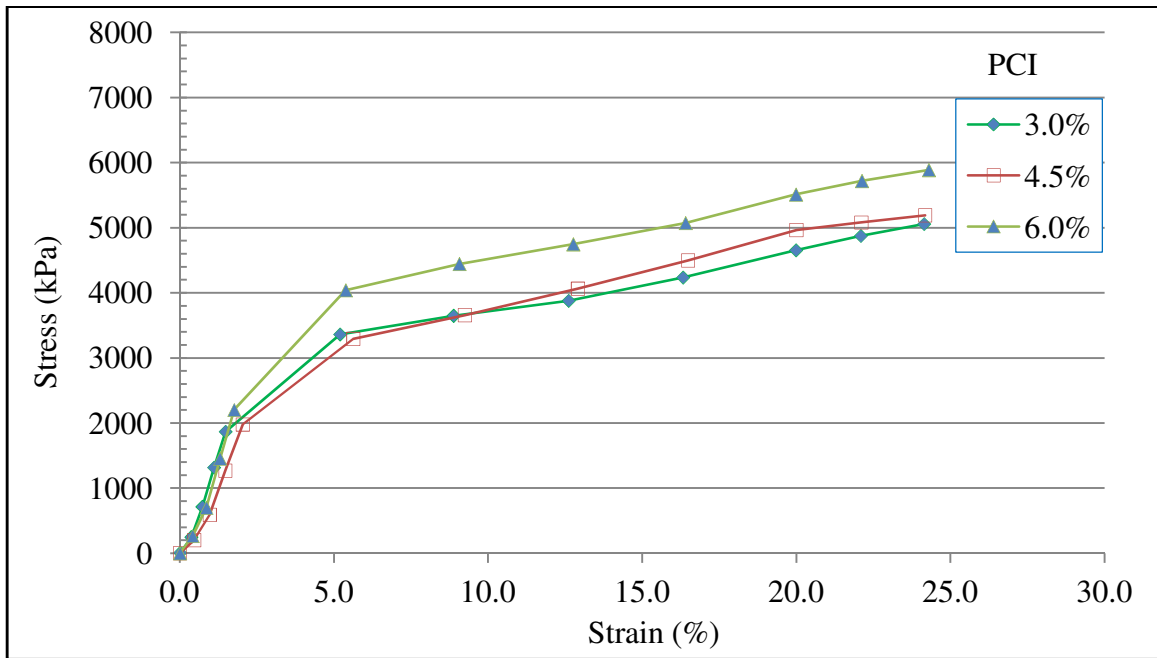


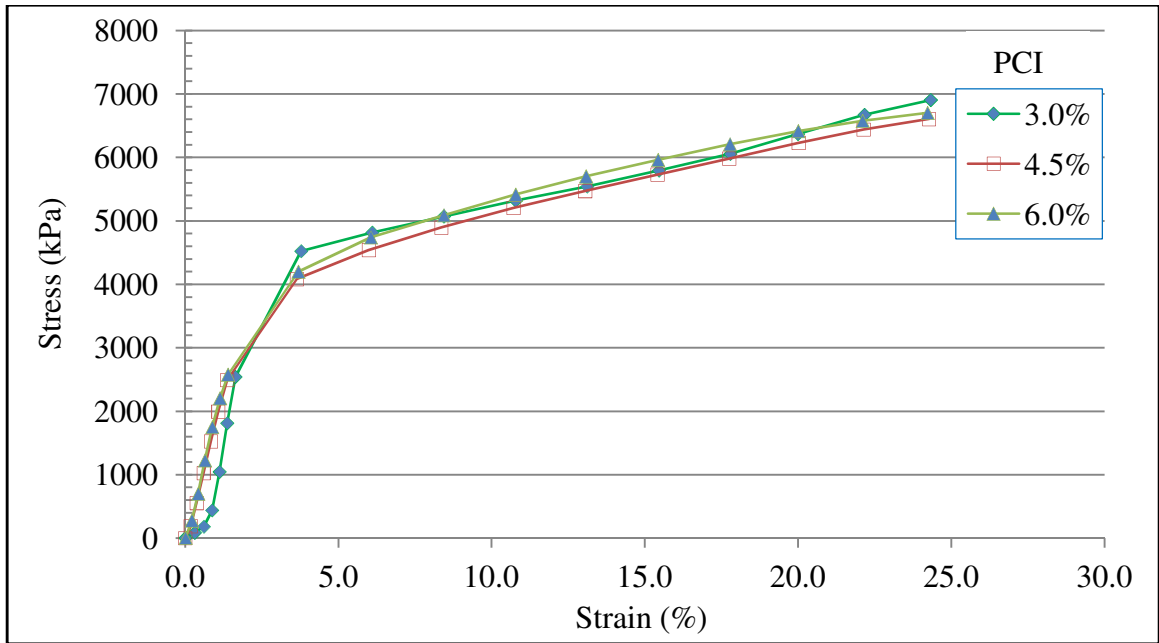
Figure 3.7 Effect of cement content on the modulus of elasticity of PCI-FCPB

Figures 3.8 illustrates the stress-strain behaviour of FCPB made of various binder content and cured for 7 (Figure 3.8a), 28 (Figure 3.8b) and 90 days (Figure 3.8c). From

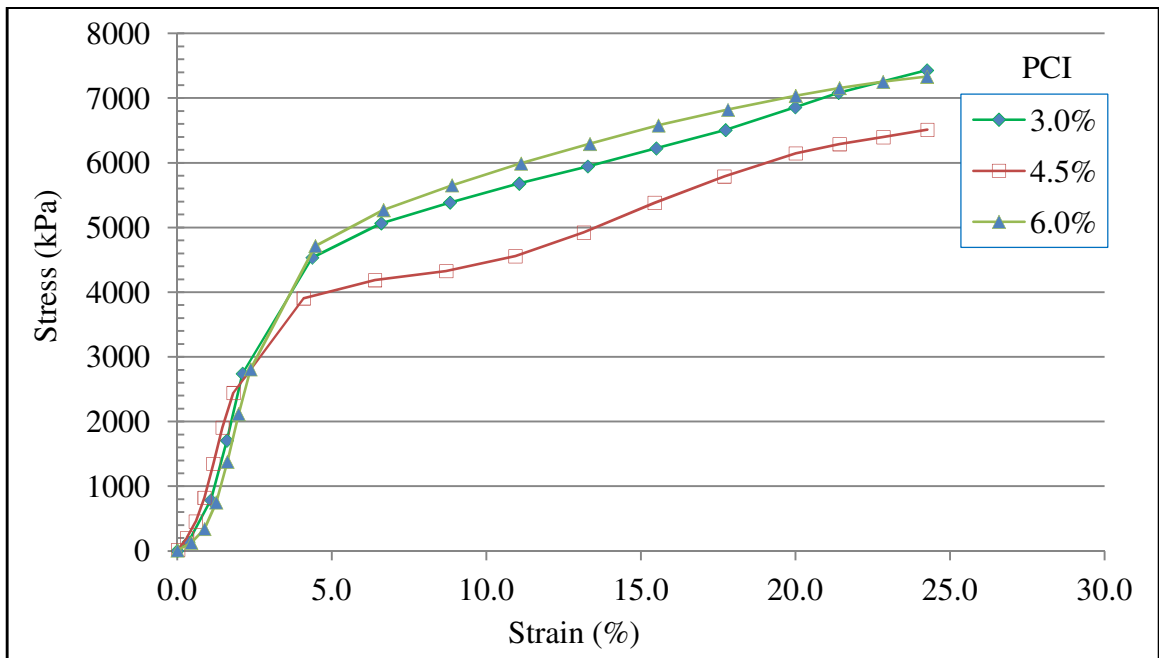
this figure, it can be observed that the cement content does not significantly influence the shape of the stress-strain curve and the initial yield strain. The shape of the curve and the yield strain have been discussed in the previous section.



a) curing time : 7 days



b) curing time : 28 days



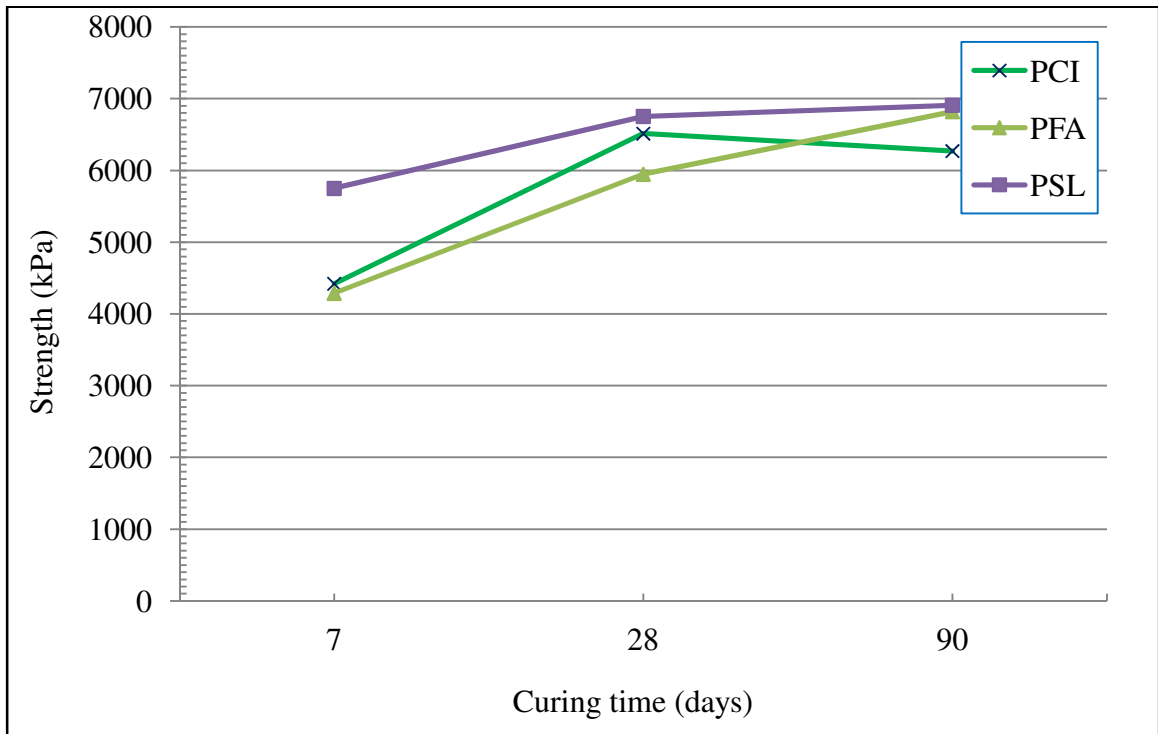
c) curing time: 90 days

Figure 3.8 Effect of cement content on the stress-strain behaviour of PCI-FCPB at different curing times: a) 7 days; b) 28 days; c) 90 days.

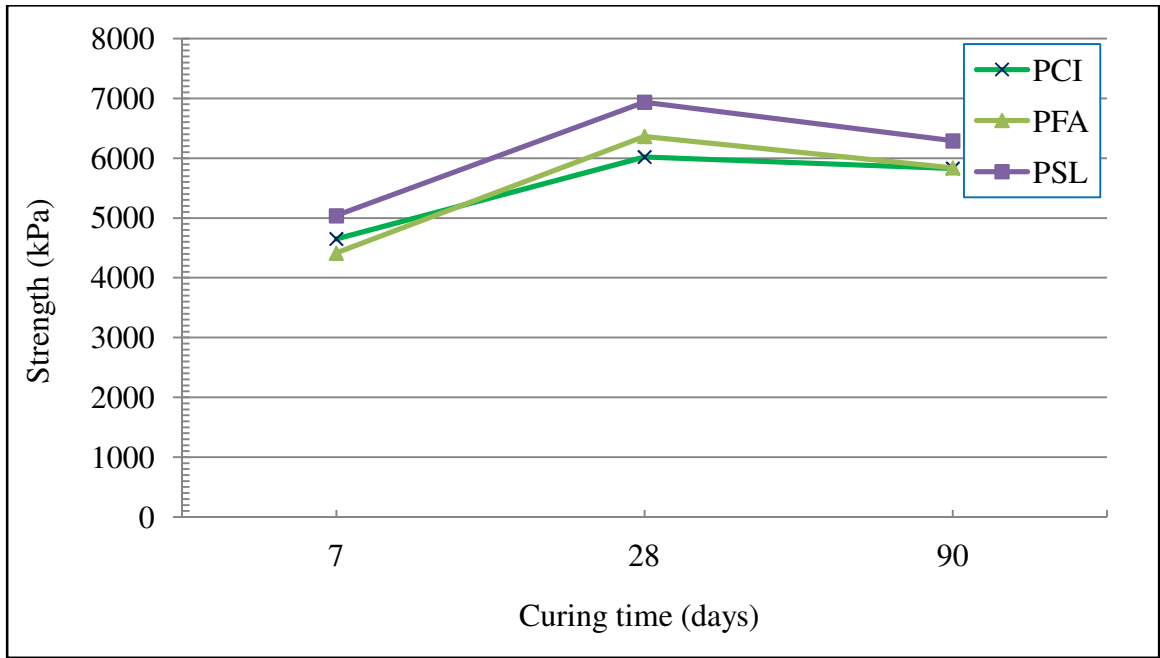
3.3.3 Effect of binder type on the strength development and deformation behaviour of FCPB

In recent years, blended cements (PCI blended with mineral admixtures such as Slag and FA) have been increasingly used as binder in CPB. Advantages that accompany the use of blended cements include savings in Portland cement consumption (cost reduction of CPB) and improvement in the durability of the CPB. Hence, additional laboratory tests were performed to study the mechanical response (UCS, deformation behaviour) of FCPB samples made of PCI, PCI and Slag as well as PCI and FA. The main results are presented in Figures 3.9 to 3.11. Figure 3.9 illustrates typical results of the effect of binder type on the strength development of FCPB for different binder contents. Figure 3.10 highlights the effect of binder type on the strength and modulus of elasticity of 4.5%-FCPB for different curing times, 7 days (Figure 3.10a), 28 days (Figure 3.10b), and 90 days (Figure 3.10c). From these figures, it can be noted that except for the 3.0%-FCPB cured at 90 days, the strength and stiffness of Slag-FCPB and FA-FCPB follow the same trend as PCI-FCPB. This means that there is an increase in their values until 28 days of curing, and then followed by a decrease. This is due to the same mechanisms explained earlier. However, Figure 3.9 shows that Slag-FCPB and FA-FCPB specimens with 3% binder do not experience any decrease in strength and modulus of elasticity at 90 days of curing time contrary to the PCI-FCPB specimens. This can be attributed to the fact that Slag and FA, due to their lower clinker content, produce less heat during cement hydration (Taylor, 1990), thereby, resulting in inhibition or less negative impacts on the strengthening effect of the ice. These results suggest that in the case of freezing curing

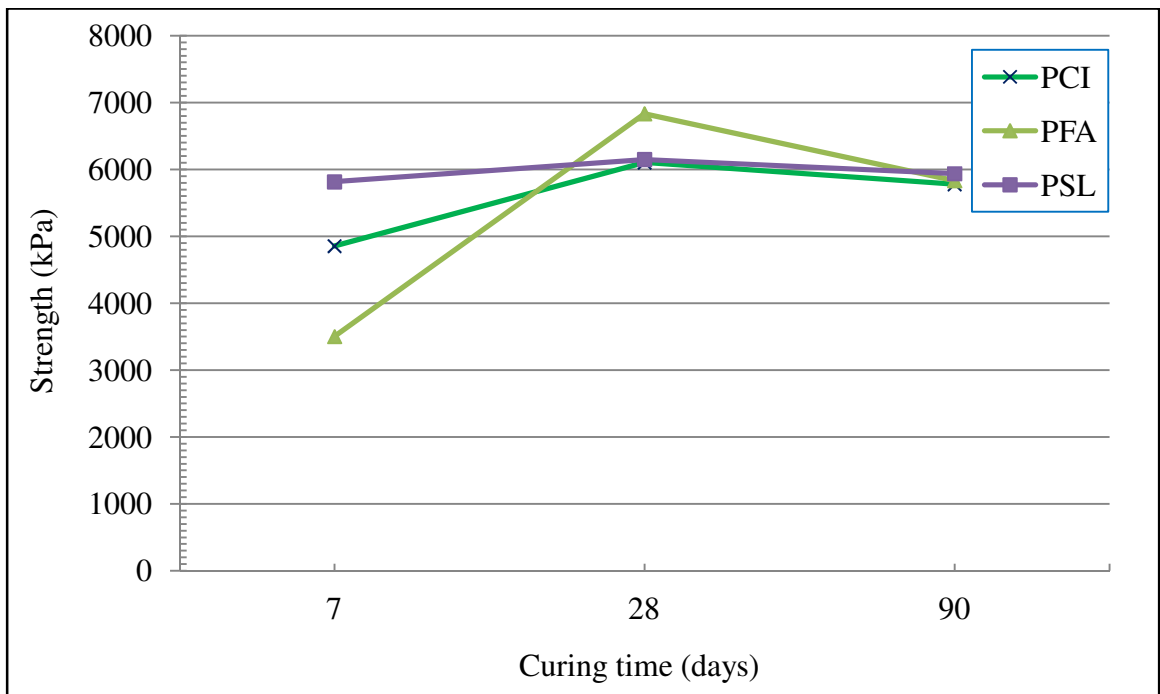
temperatures and CPB with binder content lower than 3%, the partial replacement of ordinary Portland cement by Slag or FA enhances the mechanical properties (compressive strength, modulus of elasticity) of FCPB.



a) 3.0%

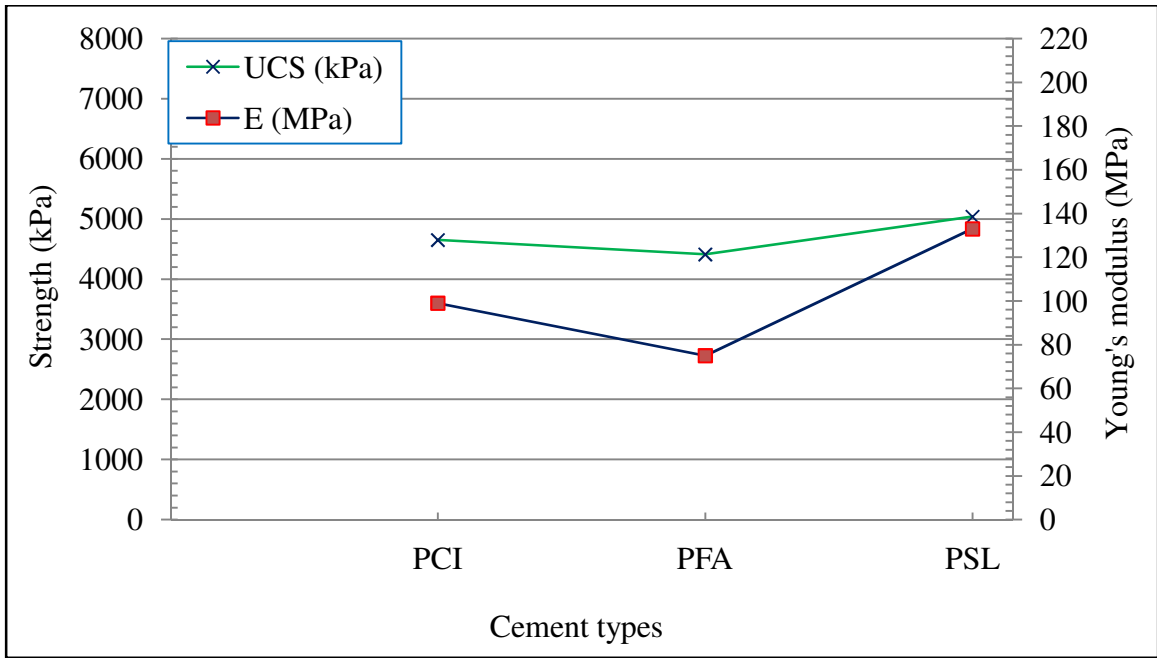


b) 4.5%

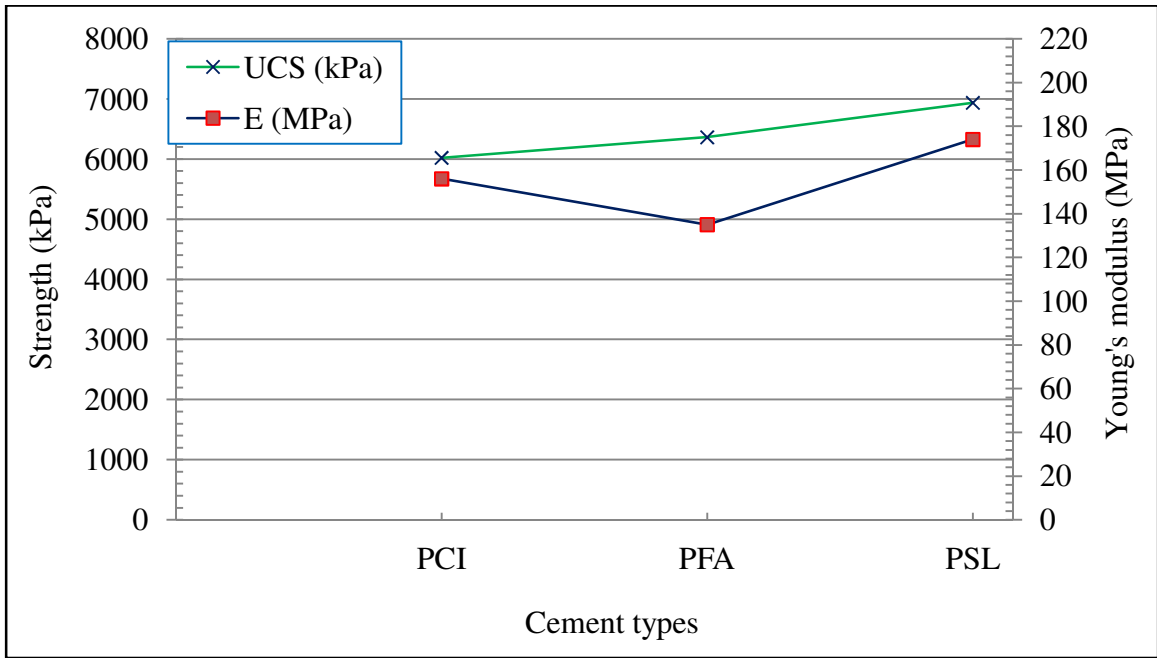


c) 6.0%

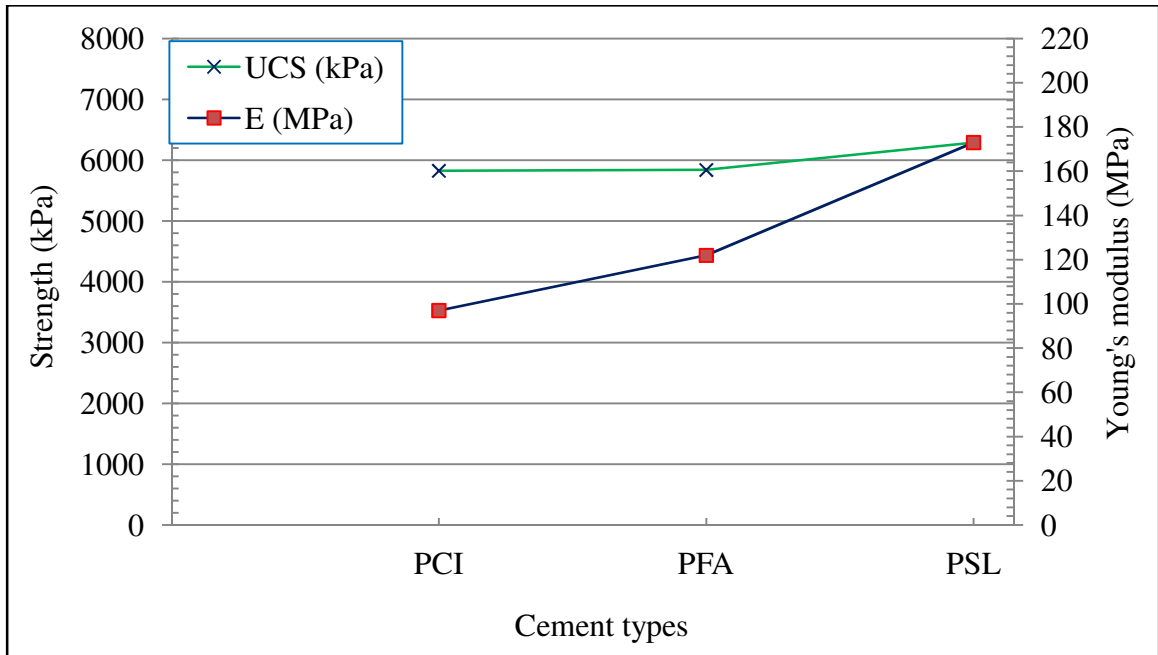
Figure 3.9. Effect of binder type on the UCS development of FCPB for different binder content: a) 3.0%, b) 4.5%, and c) 6.0%



a) 7 days



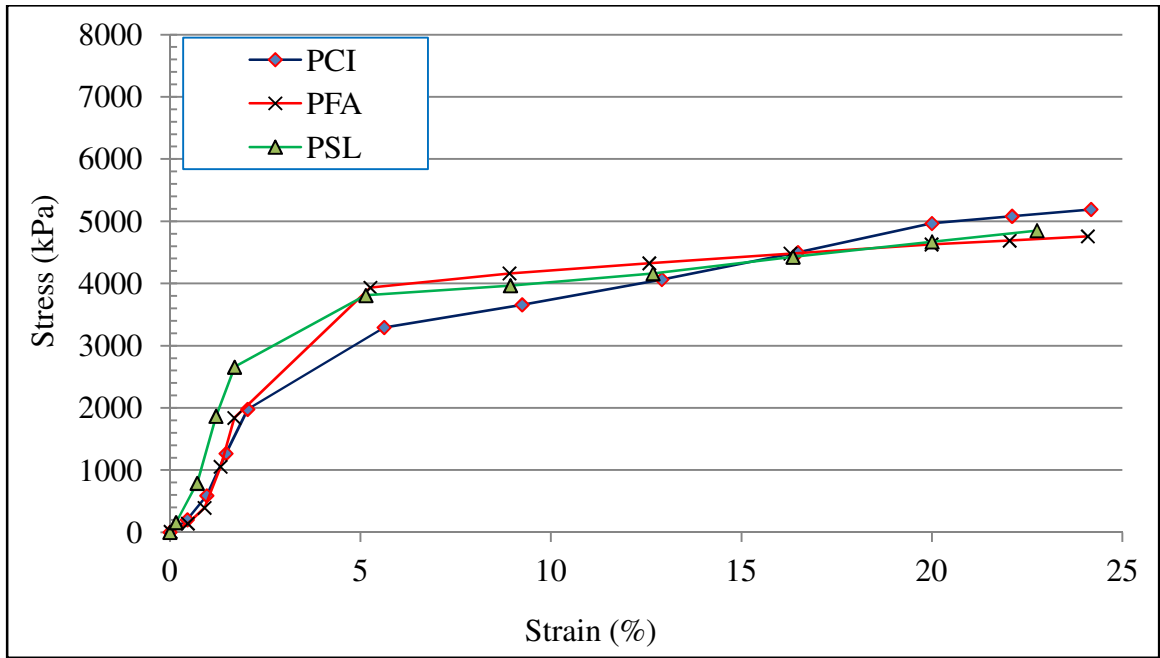
b) 28 days



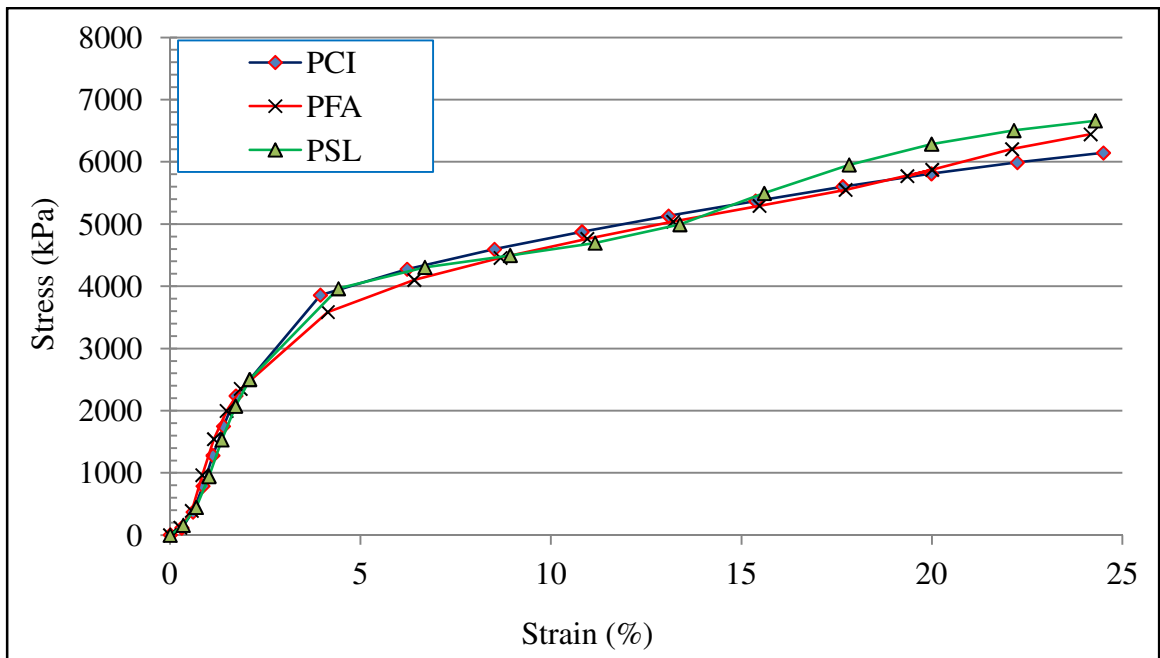
c) 90 days

Figure 3.10 Effect of binder type on the strength and modulus of elasticity of FCPB (cement content, 4.5%), curing age: a) 7days; b) 28 days; c) 90 days

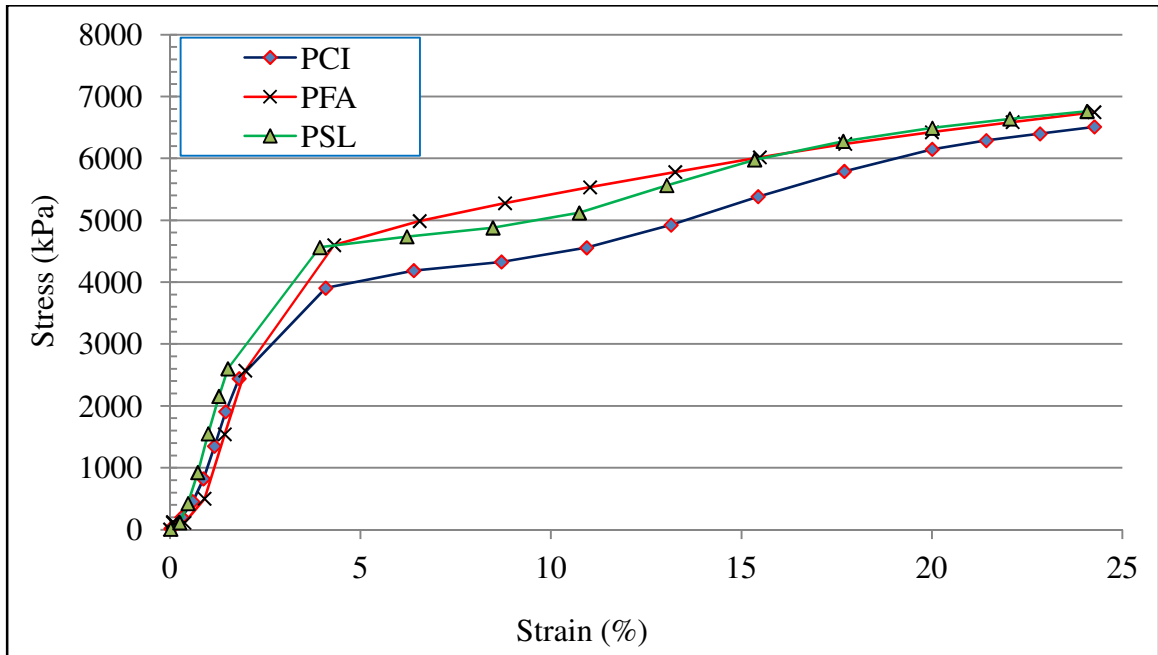
Figure 3.11 shows selected examples of the effect of binder type on the stress-strain curve of FCPB with a cement content of 4.5%. As described for Figures 3.12a to 3.12c, it can be found that the strain that corresponds to the initial yield point does not significantly vary with binder type; it remains almost constant, i.e. equal to 2%. It is interesting to notice that the behaviours of FCPB with the three binders in a cement content of 4.5% are almost similar for the three curing ages.



a) 7 days



b) 28 days



c) 90 days

Figure 3.11. Effect of binder type on the stress-strain behaviour of FCPB (cement content, 4.5%)

3.4 Summary and conclusions

The effects of cement content, curing age and cement type on the deformation behaviour and strength of FCPB have been investigated in this chapter.

The peculiar characteristic of the deformation behaviour of FCPB is that there is no remarkable failure surface that occurs within the test specimen even for strains as large as 25%. Strain hardening phenomena are commonly seen after the initial yield point for all tested FCPB specimens. Another common characteristic is that the strain which corresponds to the initial yield point is about 2%. There is no real peak stress for the

FCPB regardless of the curing time, and binder content and type. Thus, the stress which corresponds to a strain of 20% is used as the nominal peak strength.

The mechanical behaviour and properties of FCPB are found to be closer to those of frozen soils rather than those of concrete under the testing conditions given in this study. It is found that the strength development of FCPB as well as the modulus of elasticity are significantly influenced by the coupled effects of ice strengthening and binder hydration processes. The evolution of the curing time can have two opposite effects on the strength development of FCPB. On the one hand, the coupled effects of ice and cement hydration lead to an increase in FCPB strength at early ages (up to 28 days) due the dominating influence of the ice strengthening effect and contribution to the cohesion of the cement hydration products. However, on the other hand, beyond 28 days (except for 3% Slag-FCPB and FA-FCPB) the strength decreasing factors counteract the beneficial effect of the strength increasing factors and cause a decrease in strength if their effects outweigh those of the increasing factors (ice strengthening effect, cement hydration products, cement hydration consumes water, thus contributes to decreasing unfrozen water content). The identified strength decreasing factor is the heat produced by binder hydration. This heat leads to the partial melting of the ice, and thus decreases its strengthening effect.

It is also found that FCPB made of lower cement content (3%) has higher strength than that with higher cement content (4.5%, 6.0%). This finding could have a significant impact on the reduction of the cost of CPB. It also raises the following question: what

would be the strength and deformation behaviour of tailings backfill without cement cured in sub-zero temperatures? This question is answered in the next chapter.

CHAPTER 4:

STRENGTH AND DEFORMATION

BEHAVIOUR OF FROZEN TAILINGS BACKFILL

4.1 Introduction

The increased interest of the mining industry in exploiting mineral resources located in permafrost areas has left the geotechnical and mining community with some challenging problems related to the design of cost-effective mine backfill structures suitable for these climatic conditions. Furthermore, most of the mine sites in permafrost regions are located in remote areas which make the use of cement to prepare mine backfill often uneconomical. One of the most promising mine backfill technologies in permafrost regions is the FTB. Realizing the paucity of technical data on the strength and deformation behaviour of the FTB, comprehensive testing has been undertaken to better understand the strength and deformation of the FTB and the main factors that can affect them. The objectives of this chapter are to:

- study the strength and deformation behaviour of FTB by conducting laboratory experimental tests;
- investigate the effect of mix components (snow, tailings, water) and compression pressure on the strength and deformation behaviour of FTB; and

- develop a better understanding of the mechanical properties and the mix proportioning of FTB.

The experimental program conducted and the main results obtained are presented and discussed in the following sections.

4.2 Experimental Program

The experimental program was designed to perform UCS testing on specimens with various compositions of solid, water and dry snow, and subjected to various compression pressures and a sub-zero curing temperature of -6°C . By analyzing the experimental data, valuable information about the mechanical properties and behaviour of FTB is obtained.

4.2.1 Materials used

The materials used for this study are artificial tailings (SI), distilled water, and dry snow.

4.2.1.1 Tailings

A detailed description of the used tailings is given in Section 3.2.1.1 of Chapter 3 in this thesis. A small amount of another type of tailings with 20% fines (particle with size $<20\ \mu\text{m}$) and tailings with 0% fines are also used to prepare FTB specimens with different fineness to study the effect of tailings fineness on the mechanical properties and behaviour of FTB.

4.2.1.2 Snow

The snow used to prepare the samples was collected from outside snowflakes. The snow had not experienced the melting process because it was stored in a cold room where the temperature is always negative (-5.0°C on average). This measure assures that the snow is kept in a dry state.

4.2.1.3 Mixing water

Distilled water was used to prepare all specimens.

4.2.2 Preparation of the specimens

The preparation process of the specimens includes preparing the materials, and taking and curing the specimens.

First, a moisture uniform water-tailings system was prepared by mixing the solid mass (tailings) and distilled water based on designed FTB compositions. This process can be carried out in ambient temperatures. Then, dry snow was added and quickly mixed to obtain a uniform water-snow-solid particle mixture. This key step is undertaken in a cold room. The water-tailings system should be pre-cooled before dry snow is added because if the temperature difference between the dry snow and water-tailings mix is too large, the snow will thaw during the adding process and thus change the water content of the sample. A good measure to avoid snow melting during the process of mixing is to leave

the water-tailings mix in the cold room for a period of time to decrease its temperature to a value close to 0°C. The “ideal” situation is achieved when there are no phase changes (snow melting and/or water freezing) during the mixing process, even though heat transfer is happening between the snow and water. This ideal state can only be achieved if the released latent heat from water can be exactly absorbed by the added snow.

Figure 4.1 shows an example of one of the typical soil-water-snow system compositions used for this study. Table 4.1 highlights the degrees of water saturation and total water saturation of the mixes presented in Figure 4.1

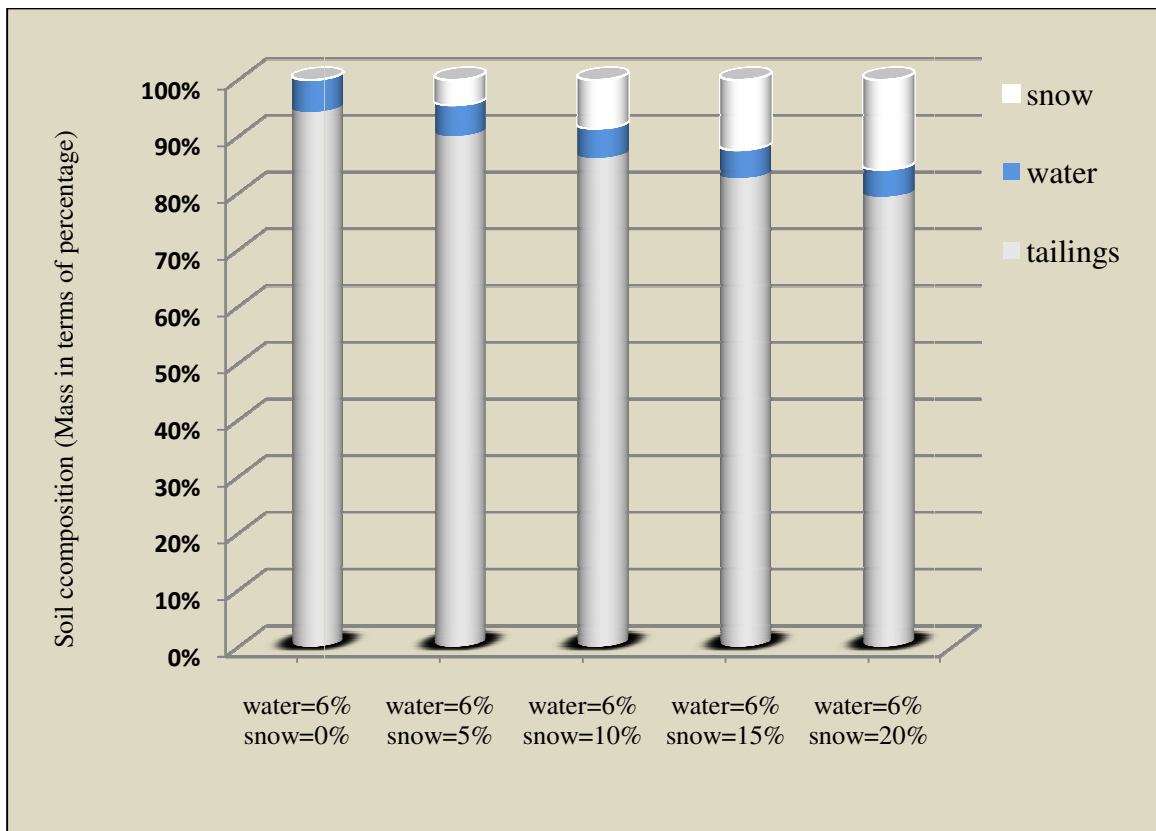


Figure 4.1 Typical tailings-water-snow compositions

Table 4.1: *Degrees of water saturation and total water saturation of the mixes given in Figure 4.1

Snow content (%)	Water saturation degree (%)	Total water saturation degree (%)
0	9.5	9.5
5	9.3	17.0
10	8.5	22.7
15	7.3	25.4
20	6.6	28.7

*Water saturation degree: only the water liquid is considered; total water saturation degree: in addition to the water liquid, the snow is taken into account too.

The water-snow-solid mix is compressed into a plastic cylinder to form cylindrical specimens with a height of 100 mm and diameter of 50 mm. The samples are compressed within the cylinders in order to simulate the compression effect of a backfilled stope of different heights. Before the mix is compressed into the cylinder, a pre-test is performed to estimate the needed amount of water-snow-solid to fill the cylinder. Then, the total weight is divided into three parts.

The compression pressures used to assess the effect of compression or the overburden pressure of the FTB (by ignoring the arching effect) are 25 kPa, 50 kPa, 100 kPa, 200 kPa, and 400 kPa. These compression pressures are measured by using calibrated rings as shown in Figure A1.3. The dial (gauge) mounted on a ring helps to judge the pressure applied. The procedures to compress the tailings are summarized in Appendices A.1.1.1.

The freezing equipment is a temperature and humidity automatically controlled freezer. Figure A1.4 shows the freezer and the temperature and humidity controlling and monitoring system. For this study, the temperature and the relative humidity in the

freezer are set to be -6°C and 87%, respectively. The cylinder that contains the specimen is covered with polymer films and stored in this freezer for 4 days of freezing. Then, the cylinder is cut off to obtain the testing specimen. The whole operation of cutting off the cylinder is undertaken in the cold room. The specimen is then completely covered with polyester film and cured in the freezer for 3 days. Polyester film is used to cover the specimen to avoid moisture variation (evaporation) during the curing period.

4. 2.3 Compressive Tests

The experimental procedures are already described earlier in Section 3.2.3 of this thesis.

4.3 Results and discussions

4.3.1 Effect of FTB mix components on the strength and deformation behaviour of FTB

The investigated mix component factors are tailings fineness, snow content, and water content. The total water content is defined as the sum of the initial snow content and the initial water content.

4.3.1.1 Effect of tailings fineness

The effect of solid grain size on the geotechnical behaviour of FTB has not been studied until now. An understanding of the effect of tailings fineness on the strength and deformation behaviour of FTB has significant relevance for mine backfill design

engineers. This is because the fineness of tailings is not always the same from one mine to another as well as during the lifetime of the mine.

Tailings with different proportions of fine (20 μm) particles (0%, 20%, 35%, 45%) were used to prepare the FTB samples to evaluate the effect of tailings fineness on the strength and deformation behaviour of FTBs. The specific surface areas of these tailings were determined to be equal to 0.6 m^2/g , 1.0 m^2/g , 1.25 m^2/g , and 1.45 m^2/g , respectively (Fall et al. 2009). With these known specific surface values and the curing temperature as well as considering that the tailings is non-plastic (see Chapter 3), the unfrozen water content of FTB is predicted by using Eq. 2.1 and the results are showed in Table 4.2. It can be noticed that the unfrozen water content is only slightly influenced by the tailings fineness for the studied range of fineness.

Table 4.2 Predicted unfrozen water content within FTB specimens

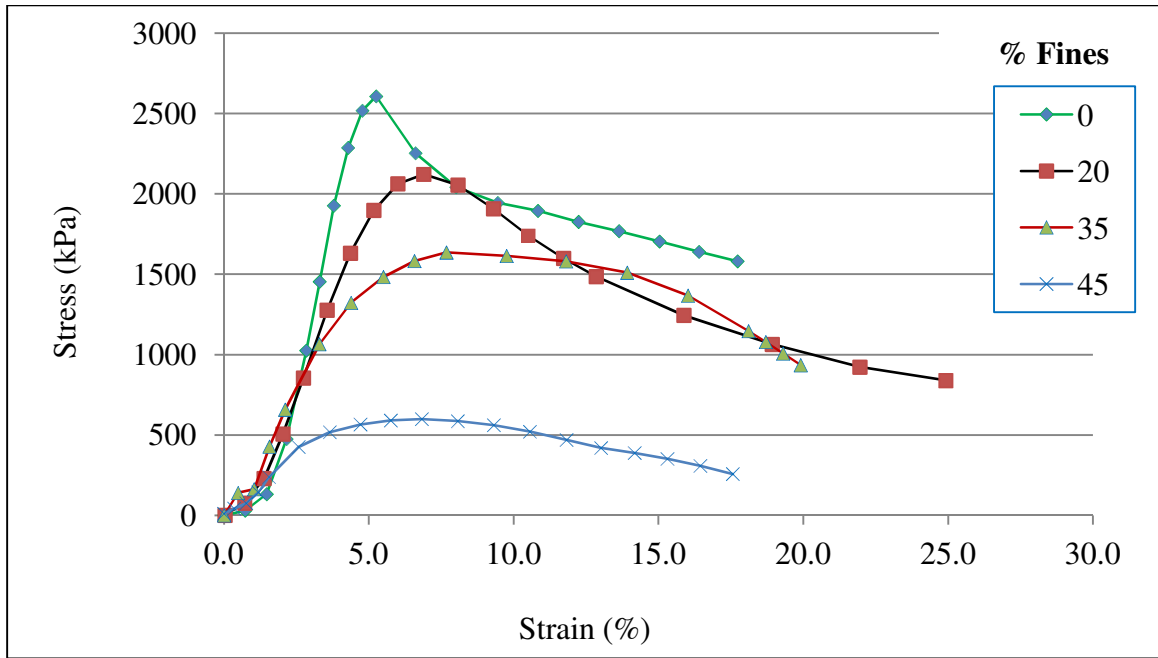
% fine	Specific surface area (m^2/g)	Unfrozen water content (%)
0	0.6	1.6
20	1.0	2.7
35	1.25	3.4
45	1.45	4.0

The typical stress-strain behaviours of FTB made of tailings with different fineness for various water and snow contents are given in Figure 4.2 (a, b). The effects of tailings fineness on the strength and stiffness properties are showed in Figure 4.3 (a,b). It can be observed from Figures 4.2 and 4.3 that the deformation behaviour and strength of FTB are strongly influenced by the fineness of the tailings. It is observed that the failure mode of the specimens is significantly affected by the tailings fineness. In terms of the FTB

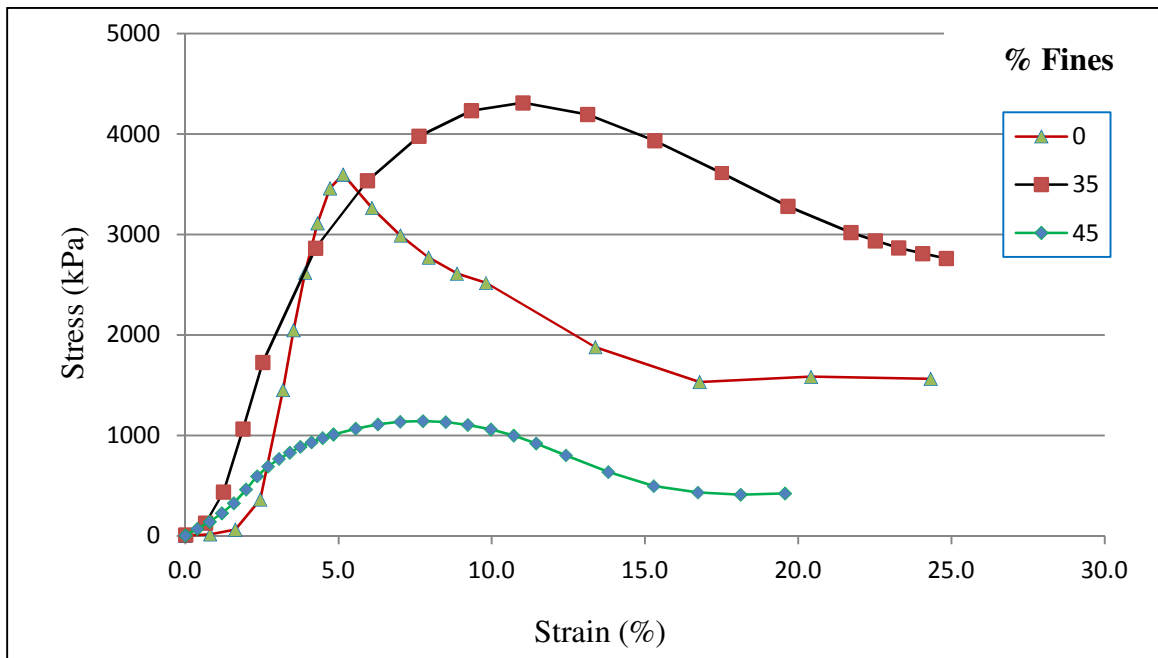
made of coarse tailings (0% and 20% fine), the failure mode is relatively brittle failure; whereas the FTB made of finer tailings (medium tailings) display some degree of plasticity (Figure 4.2). Figure 4.3 shows that the UCS and the modulus of elasticity decrease as the tailings fineness increases. This means that coarse tailings provide higher strength and modulus of elasticity to FTB for the studied water and snow contents. Similar observations were made on frozen soils by several researchers, such as Neuber and Wolters (1970), Tsyotovich and Sumgin (1937), Sayles and Haines (1974). Neuber and Wolters (1970) experimentally studied the compressive strength of six different types of frozen soils with various proportions of fine particles and concluded that the compressive strength of the soils decreases as the fine content increases. Sayles and Haines (1974) experimentally observed that the strength of frozen sand is four times higher than that of frozen clay. The aforementioned effect of tailings fineness on the mechanical behaviour and properties of FTB can be attributed to the following mechanisms or factors:

- (i) an increase in the coarse grain fraction in the FTB results in increased contribution to frictional resistance of the FTB, thereby leading to higher strength, and
- (ii) a higher proportion of fine tailings is associated with a higher specific surface area of the tailings as discussed in Section 4.3.1.1. It is well known that higher specific surface area values in frozen soils result in lower compressive strength (e.g., Wijeweera and Joshi 1990). Indeed, Dillon and Andersland (1996) determined a higher unfrozen water content in frozen fine-grained soils with larger specific surface area values. This is also in good agreement with

the prediction results of unfrozen water content of FTB made of tailings with different fineness as presented in Table 4.2. From this table, it can be seen that finer tailings means higher unfrozen water content. It is well accepted that unfrozen water content has a strong impact on the strength and deformation behaviour of frozen soils (Nerseova and Tsytovich, 1963; Williams, 1964). The higher unfrozen water content corresponds to the lower strength and modulus of elasticity of frozen soils. However, it should be emphasized that for the studied tailings fineness, the contribution of the specific surface area of the tailings or unfrozen water content to strength variation of the FTB should be assumed to be much lower than the contribution due the aforementioned frictional resistance. This is because there is only a slight variation in the specific surface area values and unfrozen water content for the range of tailings fineness considered in this study. Therefore, further studies are necessary to investigate the effect of tailings with 60% to 90% fine on the deformation and strength behaviour of FTB.

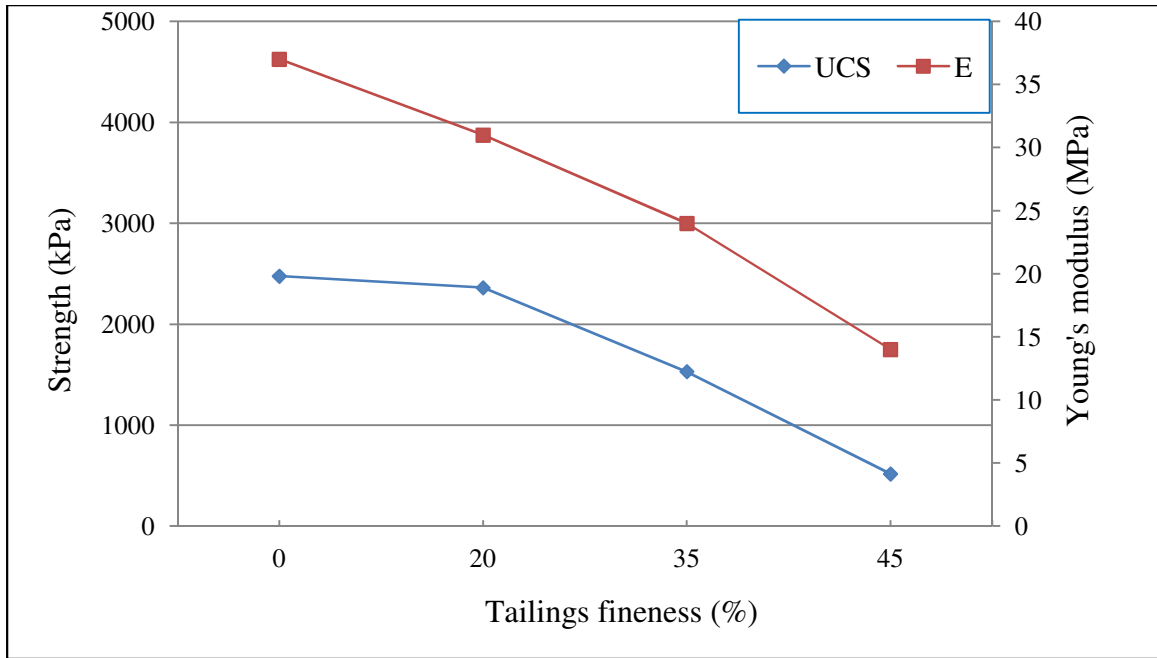


a) w=13%, snow=10%

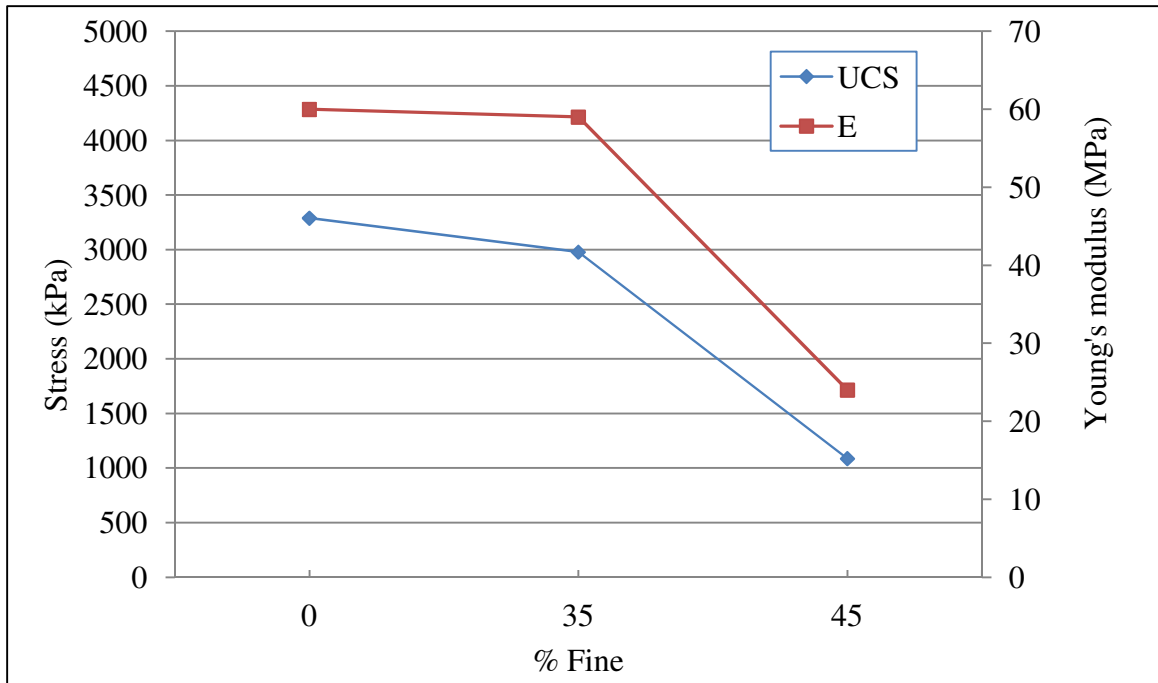


b) w=20%, snow=0

Figure 4.2 Effect of tailings fineness on stress-strain behaviour of FTB, a) w=13%, snow=10%; b) w=20%, snow=0



a) $w=13\%$, $snow=10\%$



b) $w=20\%$, $snow=0$

Figure 4.3 Effect of tailings fineness on the strength and modulus of elasticity of FTB, a)

$w=13\%$, $snow=10\%$; b) $w=20\%$, $snow=0$

4.3.1.2 Effect of snow content

Specimens with four different water contents and five different snow contents were prepared in a laboratory environment to assess the effect of snow content on the deformation behaviour and strength of FTB. Table 4.3 gives the initial water and snow contents for the mixes of FTB prepared to study the effect of snow content as well as that of water content. The initial water content is defined as the ratio of the mass of added water to the mass of the solid. The snow content is defined as the ratio of the mass of the added snow to the mass of the solid.

Table 4.3 The water and snow contents for the mixtures of FTB

Water content (%)	Snow content (%)				
	0	5	10	15	20
6	0	5	10	15	20
13	0	5	10	15	20
20	0	5	10	15	20
25	0	5	10	15	20

Note: 1. The solid is silica tailings and the calculated percentage is based on the mass. 2. Water content= (mass of water/mass of tailings) ×100%; 3. Snow content=mass of snow/mass of tailings×100%; 4. Freezing temperature is T=-6°C; 5. Compaction pressure P=25 kPa.

Typical examples of the effect of snow content on the deformation behaviour of FTB are shown in Figures 4.4 to 4.7, whereas the effects of snow content on the strength and modulus of elasticity of the FTB are given in Figures 4.8 and 4.9, respectively. From these figures, it can be noticed that the snow content significantly affects the stress-strain behaviour, UCS and modulus of elasticity of FTB. However the nature and magnitude of this effect is a function of the initial water content of the FTB, i.e. on the total water

(liquid water + snow) content. An analysis of the results presented in Figures 4.4 to 4.9 reveals that three main trends or behaviours, which depend on the initial water content or total water content, can be noted with regards to the mechanical response of the FTB to various amounts of initial snow content:

- for a water content of 6% and 13%, it can be observed (See Figures 4.4, 4.5, 4.8 and 4.9) that the FTB which contains snow has higher strength and modulus of elasticity. Furthermore, the strain at failure remains approximately equal to 5-6% regardless of the snow content. The FTB shows a clear peak strength. For example, Figure 4.4 shows the typical test results for the FTB with an initial water content of 6%. It can be noticed that the snow has a positive impact (enhancing) on the mechanical behaviour of the FTB. The UCS value increases from less than 20 kPa which corresponds to a 0% snow content to about 180 kPa which corresponds to 10% snow content. The snow content has little impact on the failure strain, which is about 5% regardless. Similar observations can be made in Figure 4.5 which illustrates the typical results of the influence of snow content on the stress-strain behaviour of FTB with 13% initial water content. However, the FTB with 13% water content shows higher strength regardless of the snow content and the maximum compressive strength is reached for a snow content of 20%, in contrast to the 10% observed for the FTB with 6% water content. This observed enhancing impact of snow on the mechanical properties of FTB with 6% and 13% water content, which adds to its strength, can be attributed to the contribution of ice generated by the added snow. Furthermore, the higher strength and modulus of elasticity of the FTB with 13% water content can be explained by

the fact that higher initial water content results in higher volumetric ice content, thereby leading to higher strength (Wijeweera and Joshi, 1990);

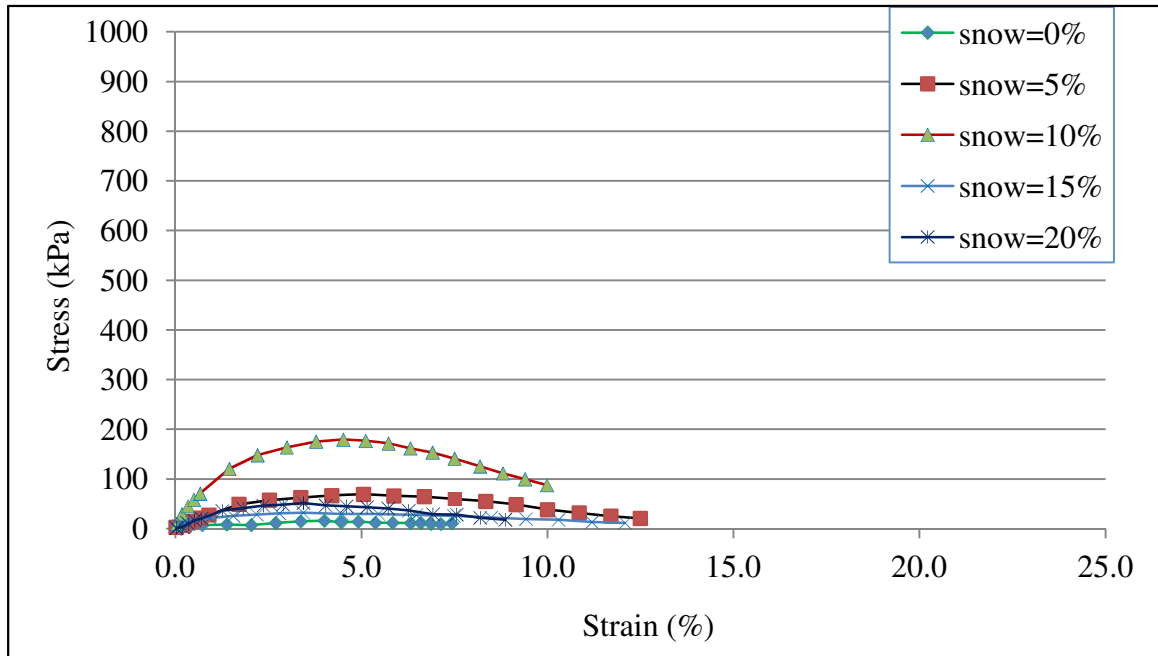


Figure 4.4 Effect of snow content on the stress-strain behaviour of FTB (w=6%)

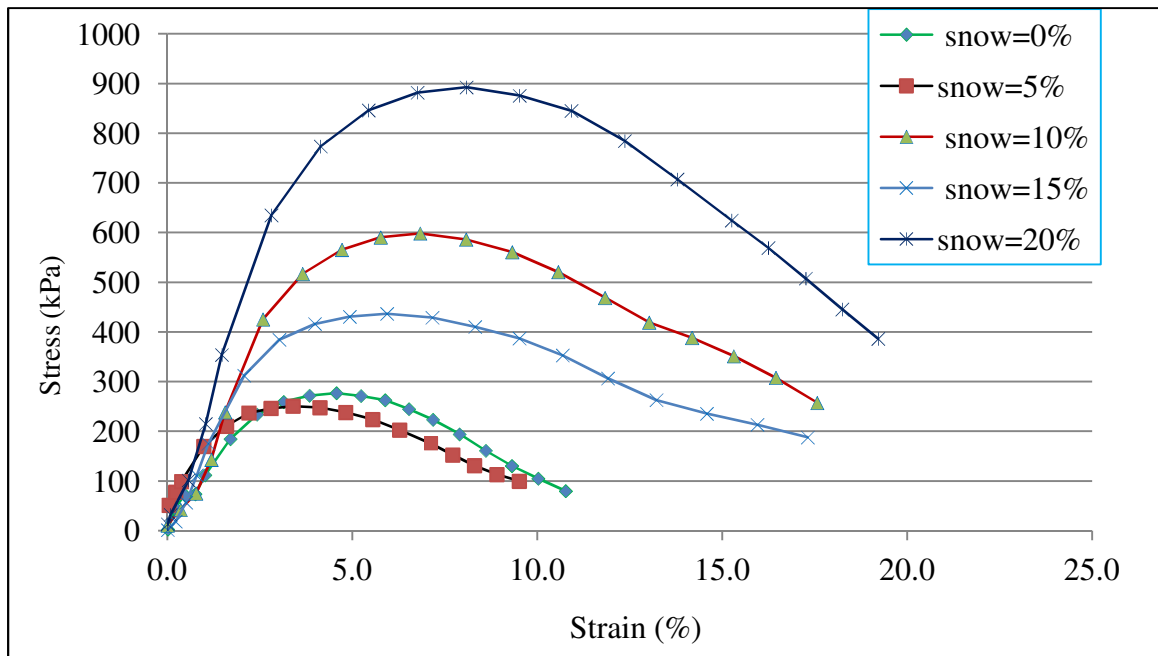


Figure 4.5 Effect of snow content on the stress-strain behaviour of FTB (w=13%)

- for a water content of 20%, it can be seen from Figures 4.6 and 4.8 that the snow has a negative effect (weakening) on the strength of FTB. The FTB with the highest snow content (20%) shows the lowest strength, whereas the FTB without snow has the highest strength value. Again, snow content has a negligible influence on the failure strain of the FTB, which is about 6-7%. The highest strength observed for the FTB without snow can be considered as a result of the considerable contribution of frictional resistance of the tailings particles. An increase in the snow content up to 5-10% increases the ice content within the FTB, and thus decreases the contribution of the frictional resistance of the tailings grains. This reduction can be considered as the cause of the drop in strength (compared to an FTB with 0% snow) experienced by the FTB samples with 5% and 10% snow content. The large difference in strength between the FTB with 5% and 10% snow and those with 15% snow can be explained by the significant contribution of the strengthening effect of ice in the latter due to their much higher initial snow content (15%). This strengthening effect results from the ice strength (cohesion) and the tailings particles-ice binding effect. The lowest strength of the FTB with 20% snow can be explained by the weakening effect of dispersely distributed tailings in the ice matrix. Similar observations were made on frozen soils by Arenson and Springman (2005);

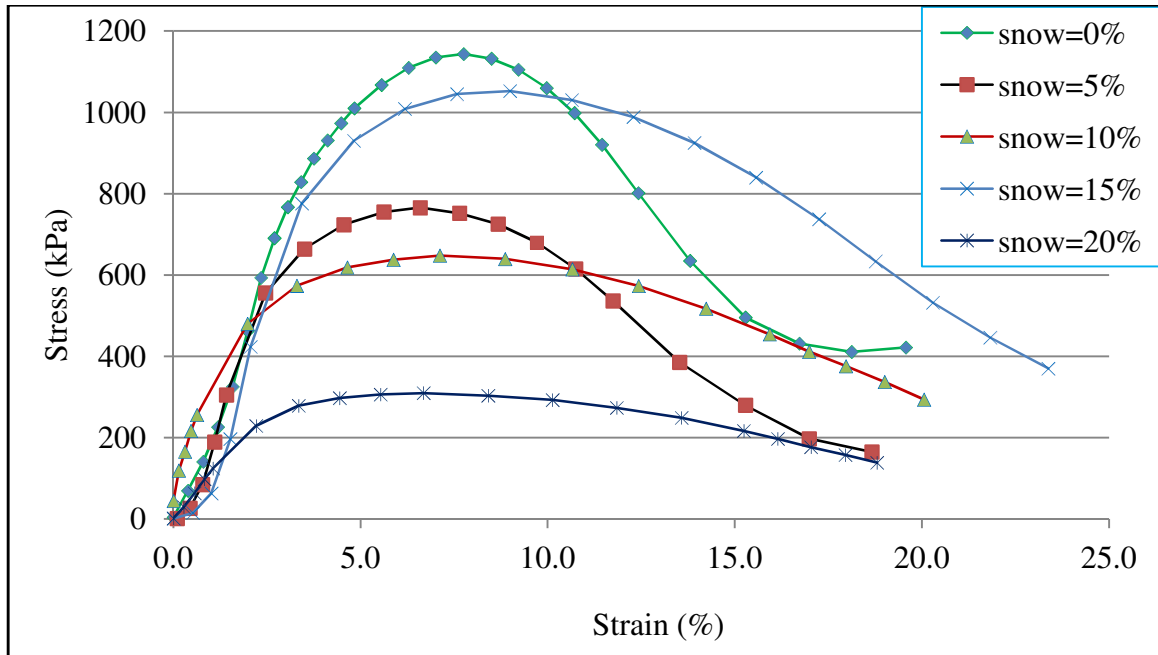


Figure 4.6 Effect of snow content on the stress-strain behaviour of FTB (w=20%)

- for a water content of 25%, it can be deduced from Figures 4.7 and 4.8 that two distinctive mechanisms operate with respect to the compressive strength behaviour; one at a snow content lower or equal to 10% (low snow content) and the other at a snow content higher than 10% (high snow content). It can be observed that as the snow content increases from 0% to 10%, the strength increases. This can be attributed to the increased contribution of the ice-strengthening effect to the strength of the FTB. However, at high snow content, the strength of the FTB is lower than the strength of FTB with low snow content. This can be attributed to the mechanisms explained earlier, i.e. the weakening effect of dispersely distributed tailings in the ice matrix of the FTB with high snow content.

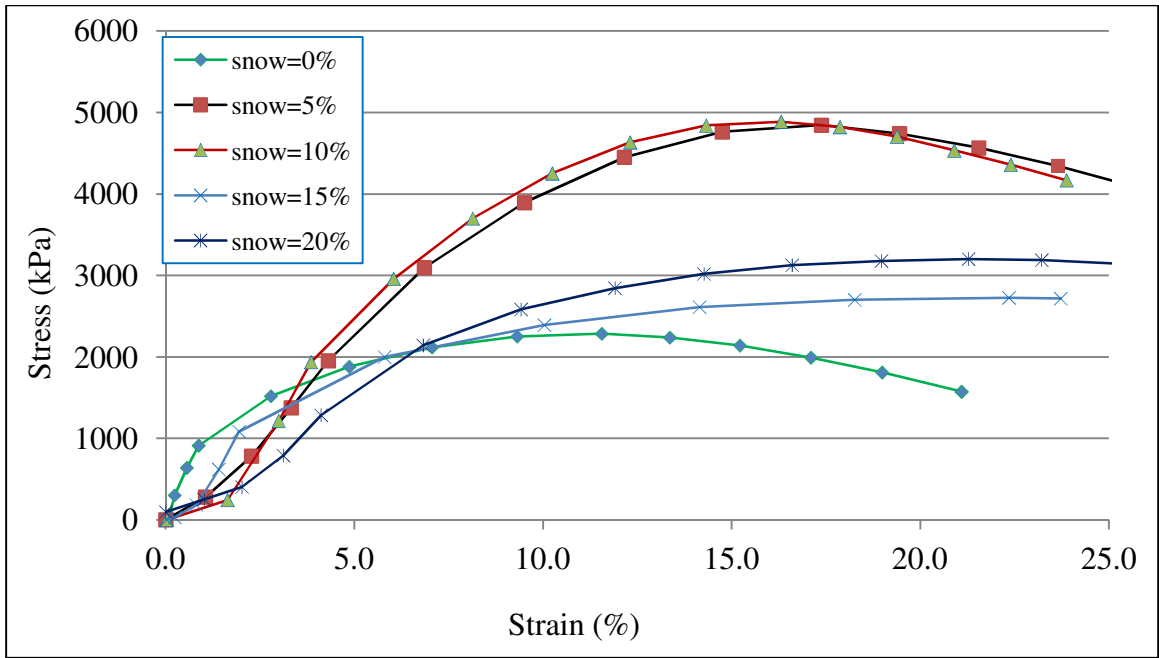


Figure 4.7 Effect of snow content on the stress-strain behaviour of FTB (w=25%)

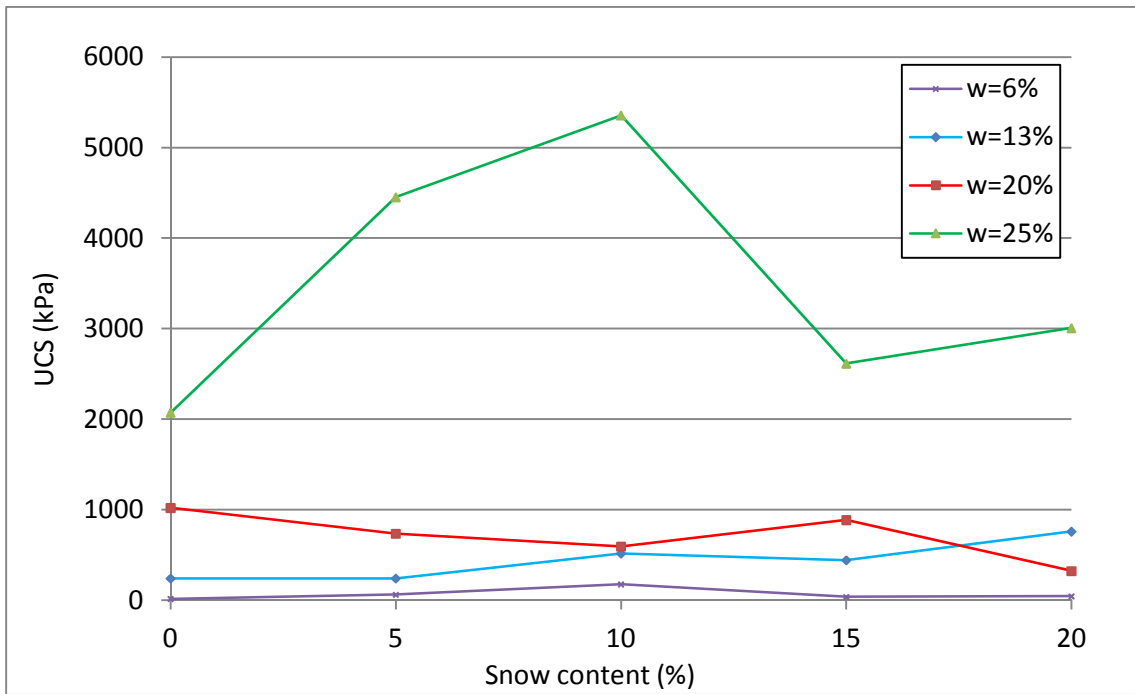


Figure 4.8 Effect of snow content on the strength of FTB

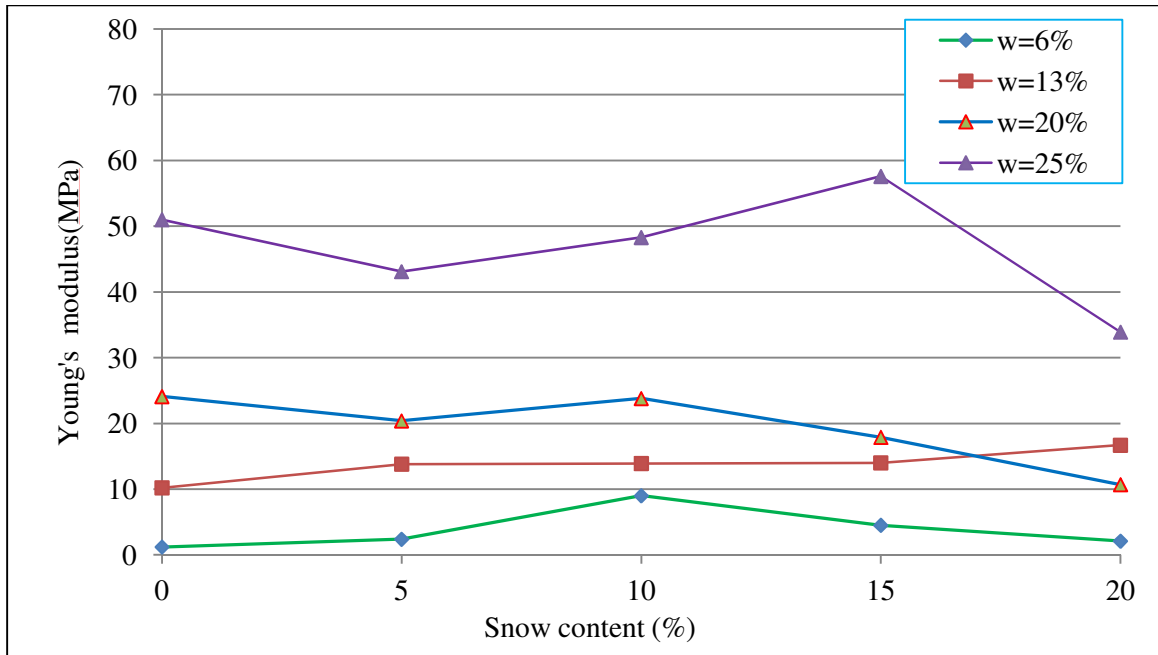


Figure 4.9 Effect of snow content on the E of FTB

4.3.1.3 Effect of water content

The discussions in Section 2.3.2 of Chapter 2 disclose that initial water content has a significant influence on the mechanical properties and behaviour of frozen soils. This is because the initial water content not only affects the ice formation and distribution within the pores of the soils, but can also to some degree influence the unfrozen water content within frozen soils. In order to quantitatively assess the influence of initial water content on the mechanical behaviour and properties of FTB, FTB specimens with four different amounts of water content and five different amounts of snow content were prepared and tested in the laboratory.

The mechanical test results of these specimens are rearranged and displayed in Figures 4.10 to 4.16 to show the impact of the initial water content on the stress-strain behaviour, strength and modulus of elasticity of FTB for different amounts of initial snow content. From the figures, it is obvious that the strength and modulus of elasticity of the FTB increase as the water content increases for any given snow content. It is also easy to observe that higher water content corresponds to a larger failure strain. These observations are consistent with the results of many other investigators (e.g., Shusherina and Bobkov, 1969; Wijeweera and Joshi, 1990; Hivon 1991) who showed that the strength properties of frozen soils are a function of the total moisture content. This increase in strength and modulus of elasticity as the water content increases can be explained by the fact that as the water content increases, so does the ice content in the FTB, thereby resulting in an increased contribution of the ice strengthening effect to the resistance of the FTB. This means that the mechanical property of FTB is greatly affected by the ice content.

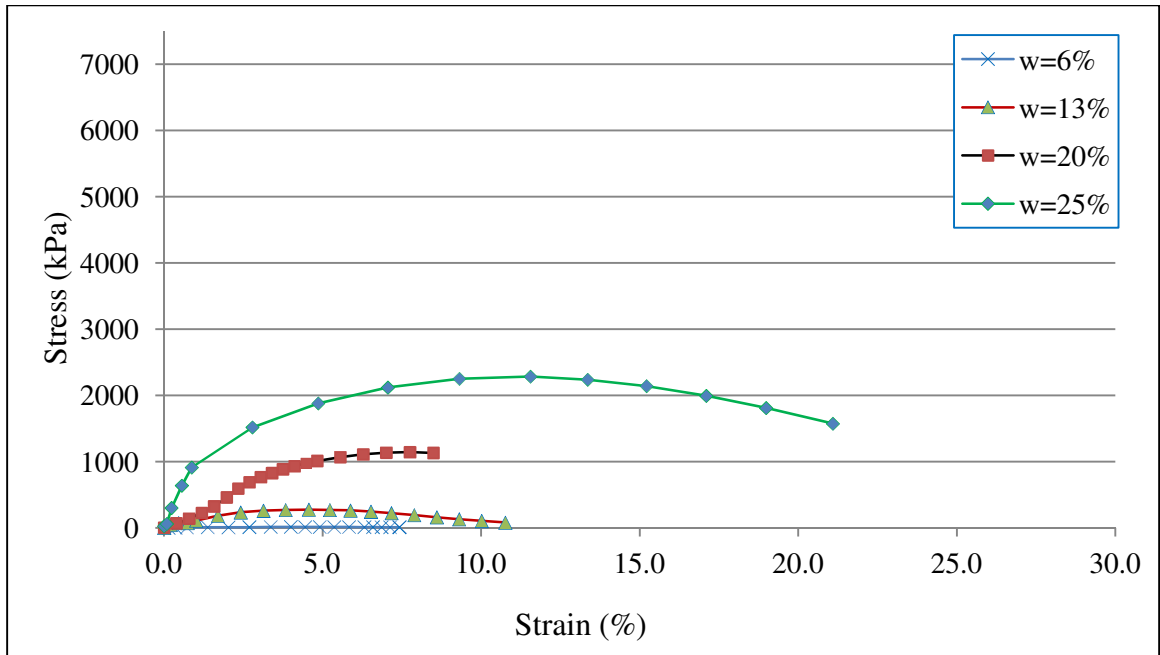


Figure 4.10 Effect of water content on the stress-strain behaviour of FTB (snow=0)

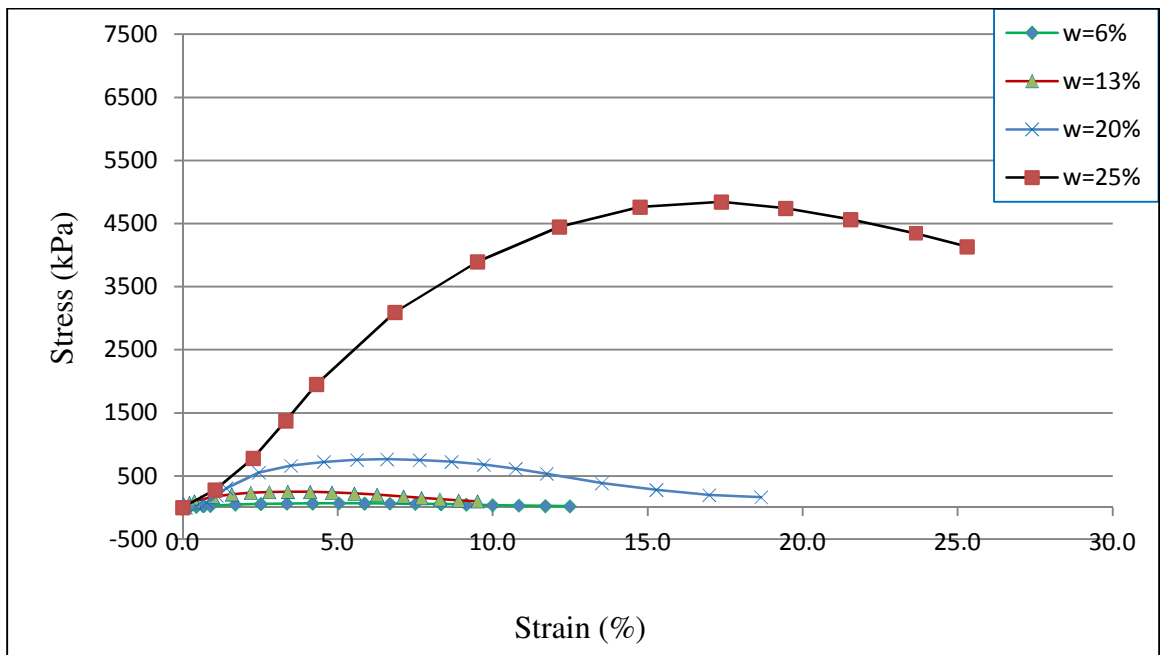


Figure 4.11 Effect of water content on the stress-strain behaviour of FTB (snow=5%)

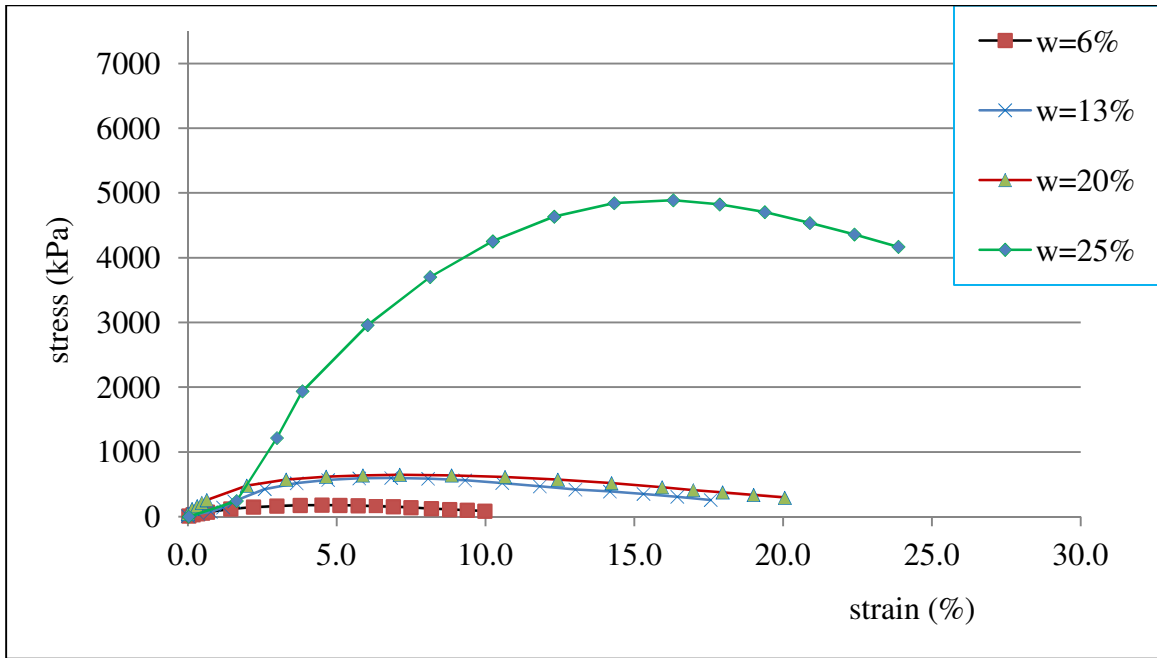


Figure 4.12 Effect of water content on the stress-strain behaviour of FTB (snow=10%)

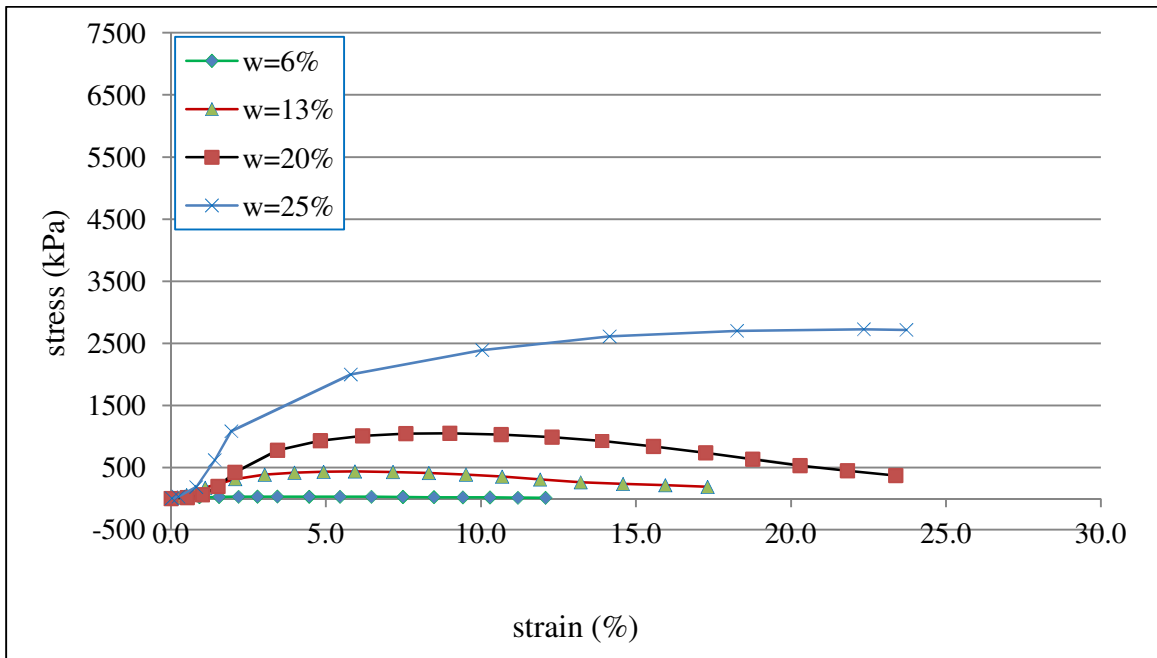


Figure 4.13 Effect of water content on the stress-strain behaviour of FTB (snow=15%)

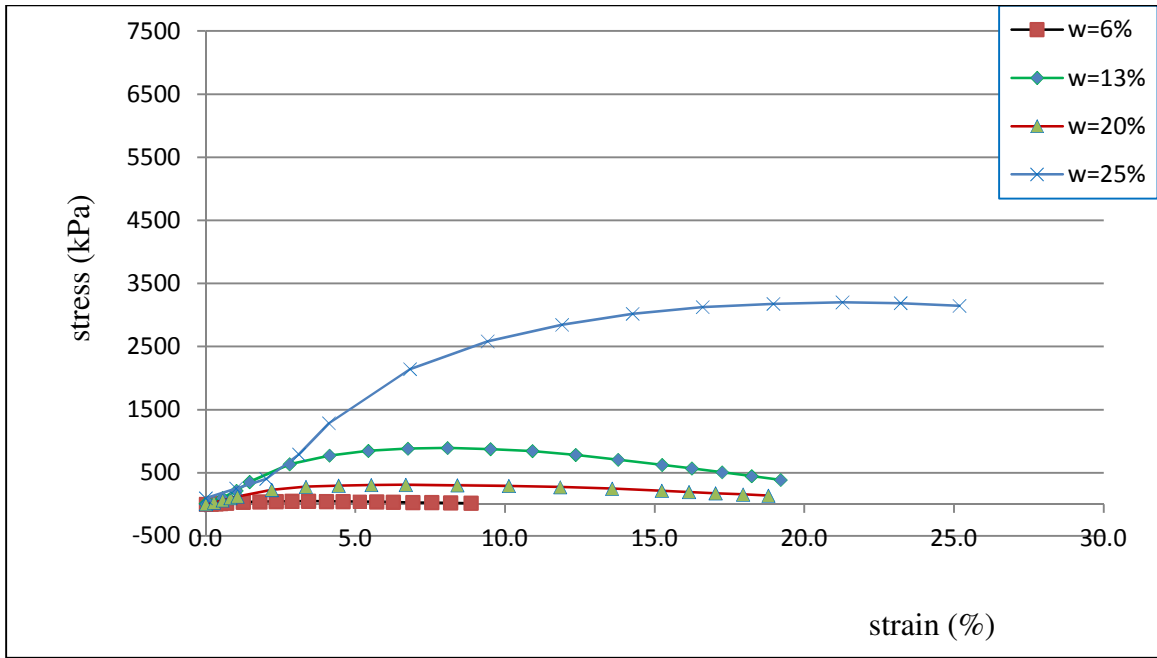


Figure 4.14 Effect of water content on the stress-strain behaviour of FTB (snow=20%)

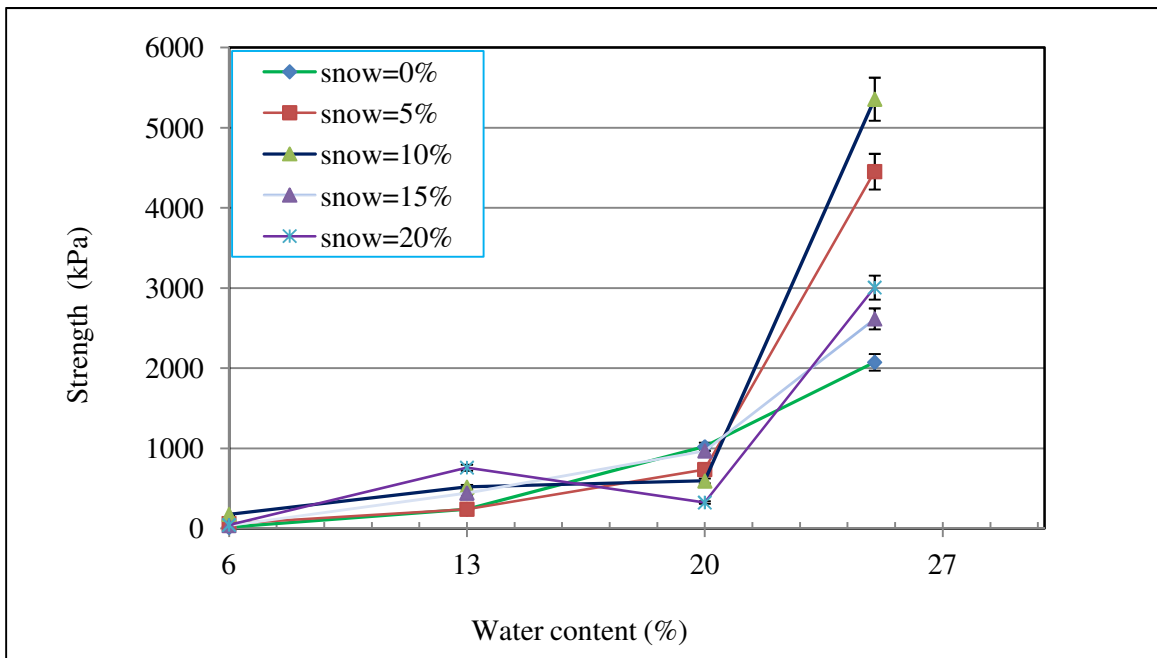


Figure 4.15 Effect of water content on the strength of FTB

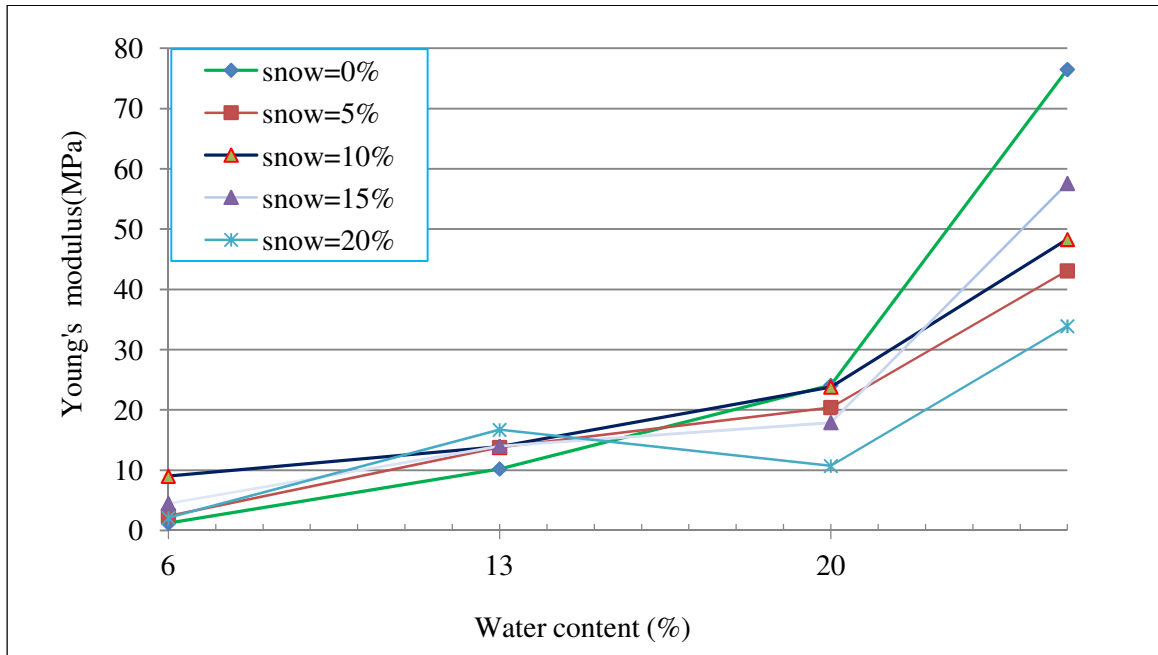


Figure 4.16 Effect of water content on the modulus of elasticity of FTB

4.3.2 Effect of vertical pressure on the strength and deformation behaviour of FTB

In order to assess the compression effect, i.e. the compression effect induced by the backfill overburden pressure on the FTB, FTB specimens with a water content of 20%, a snow content of 10%, and a water content of 25%, are prepared and tested. The static compression efforts measured by the applied vertical pressure are 25 kPa, 50 kPa, 100 kPa, 200 kPa, and 400 kPa. The curing temperature and the loading rate are the same as those for assessing the effect of water content.

Figure 4.17 shows the stress-strain behaviour of the FTB with a water content of 20% and snow content of 10%. It can be noted that the failure strain is not significantly sensitive to

the applied pressure, but the strength increases from 1500 kPa to more than 4500 kPa which corresponds to the vertical pressure of 25 kPa and 400 kPa, respectively. Even though the applied vertical pressure can change the pore size and pore water distribution, it cannot change the adsorption state of water to the solid particles. However, the failure strain is controlled by the connection of ice to the solid particles, so vertical pressure has little impact on the failure strain of FTB. It is known that the strength of frozen soil is affected by factors such as pore size, pore water distribution, etc. Furthermore, applied vertical pressure has an influence on almost all of these factors. So, it has a significant impact on the peak strength of an FTB.

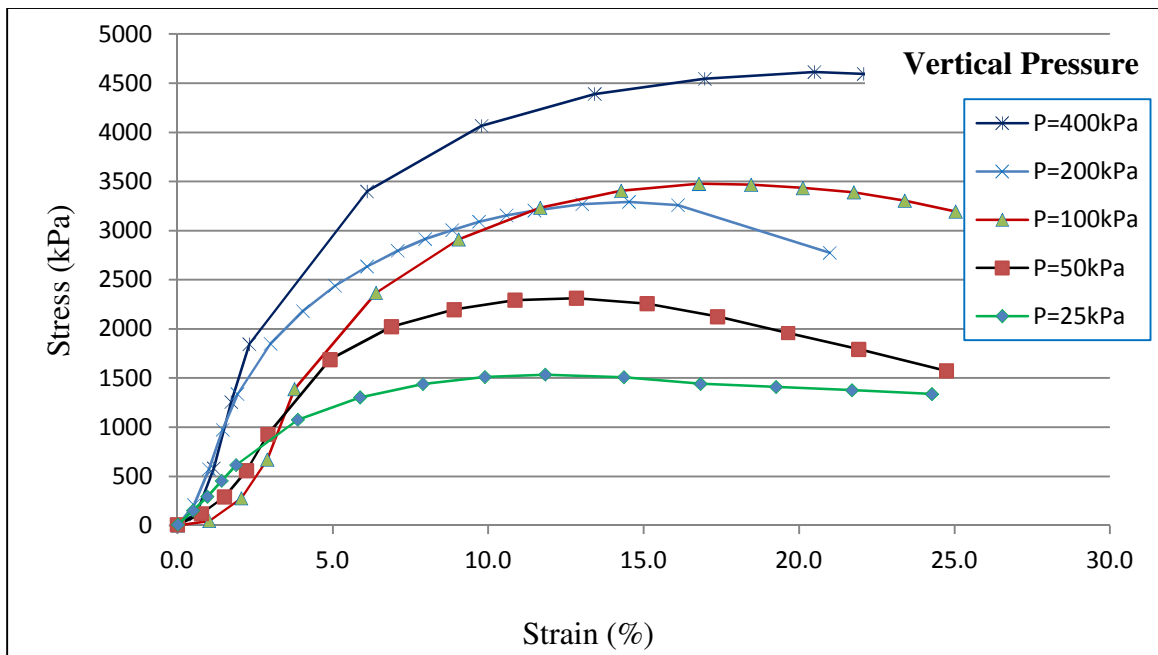


Figure 4.17 Effect of vertical pressure (compaction efforts) on the stress-strain behaviour of FTB ($w=20\%$, $\text{snow}=10\%$)

The strength and modulus of elasticity of FTB are given in Fig. 4.18. The curves clearly show an increase in strength and modulus of elasticity with an increase in applied vertical pressure. This correlation indicates that vertical pressure has an effect on changing the porous medium structure and pore water/pore ice distribution within the FTB system. Moreover, an increase in applied vertical pressure results in a greater change in these factors, which enhances increases in strength and elasticity.

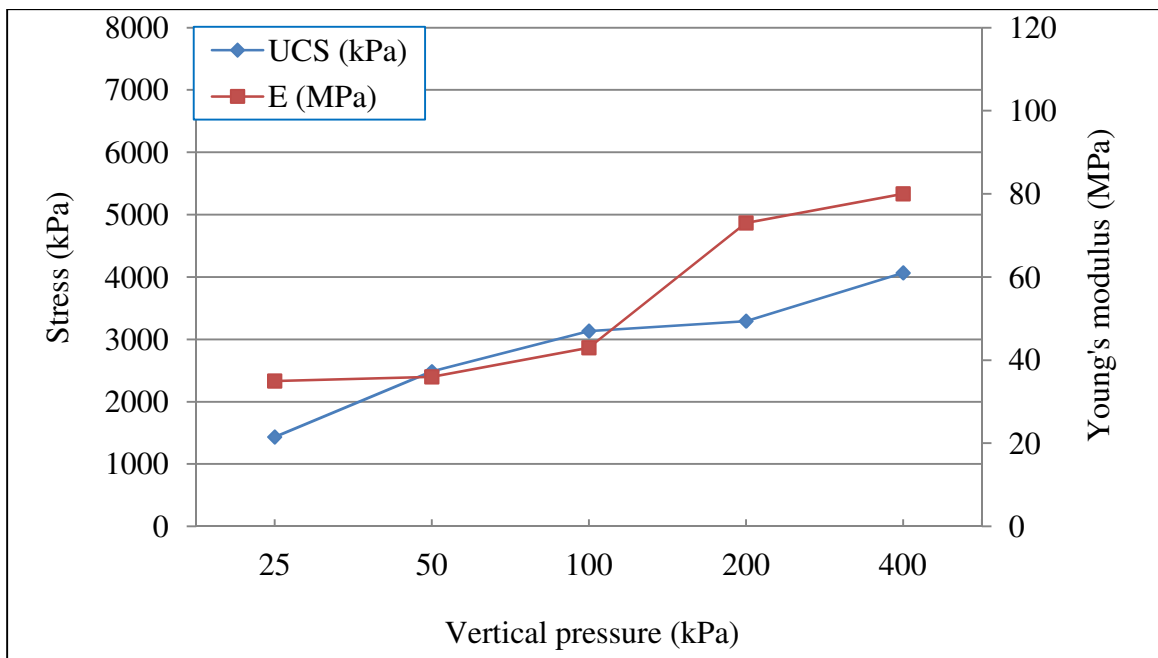


Figure 4.18 Effect of vertical compression pressure on the stress-strain behaviour of FTB (w=20%, snow=10%)

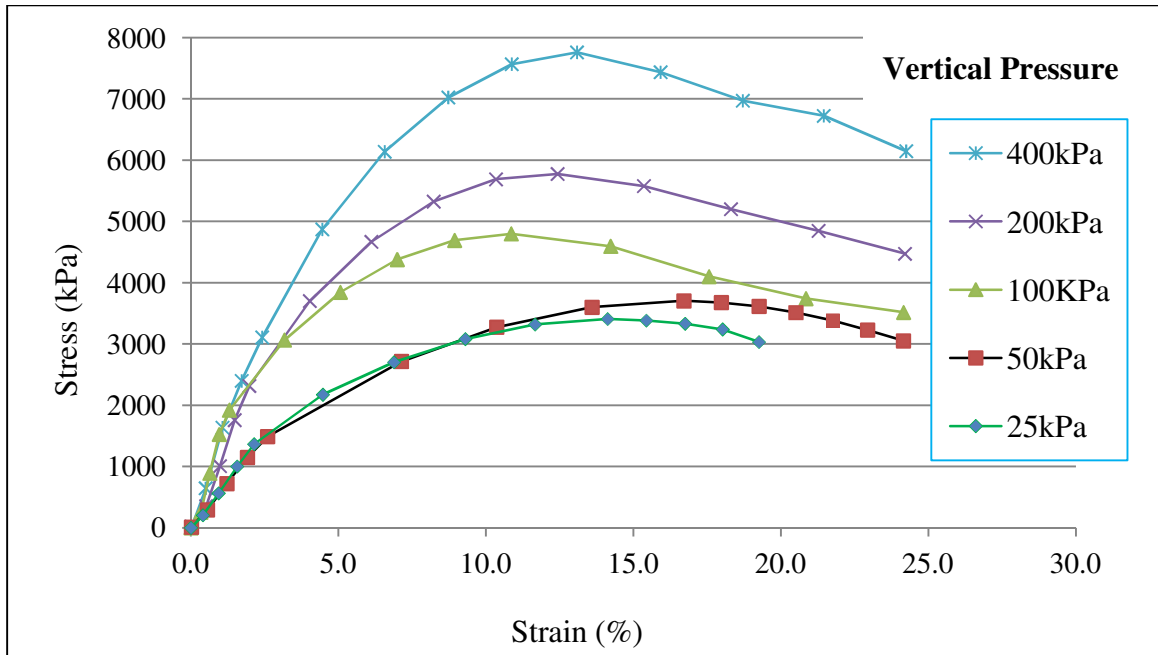


Figure 4.19 Effect of vertical pressure (compaction efforts) on the stress-strain behaviour of FTB ($w=25\%$, $snow=0\%$)

Figure 4.20 shows the effect of vertical pressure on the strength and modulus of elasticity of FTB with 25% water content and no snow. It is easy to notice that higher vertical pressure corresponds to higher strength. These test results can be explained by the fact that vertical pressure densifies the FTB specimens, and changes their saturation degree. Moreover, a higher vertical pressure corresponds to higher density and higher degree of saturation. Higher density makes the resistance of the soil skeleton higher. A higher degree of saturation leads to ice bonding within the FTB to be much stronger than that in a lower degree of saturation with the same water content but not saturated (Sayles and Carbee, 1981).

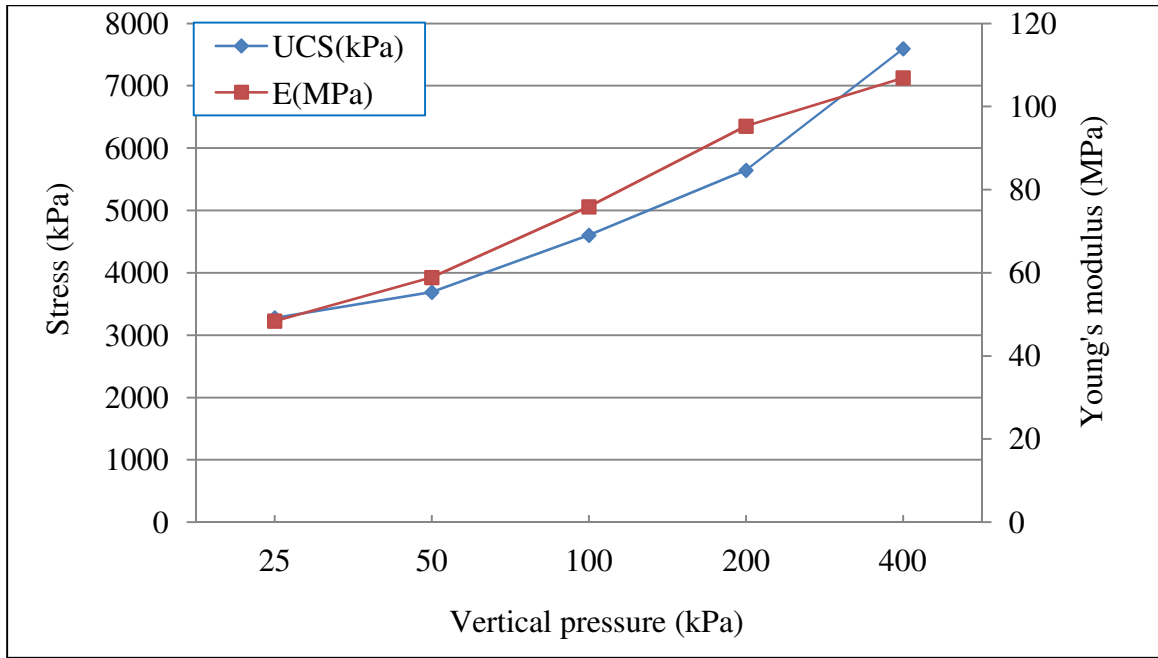


Figure 4.20 Effect of vertical pressure (compaction efforts) on the mechanical properties of FTB ($w=25\%$, $snow=0$)

4.4 Summary and conclusions

In this chapter, the implementation of UCS tests to investigate the strength and deformation behaviour of FTB, and the effect of several other factors on these mechanical properties (strength and stiffness) and behaviour are described. The investigated factors are tailings fines, water content, snow content, and vertical compaction pressure.

Tailings fineness has a strong impact on the mechanical properties of FTB; finer tailings means lower strength and E within a given fineness range from 0% to 45%. It is noticed that the strength of FTB with 35% fines is four times that of FTB with 45% fines under the same testing conditions. The impact of tailings fineness on the failure strain is not as remarkable in comparison to the strength for FTB.

The impact of dry snow on the deformation behaviour and properties of FTB is dependent on the initial water and dry snow content. It is found that when the initial water content is less than 20%, the added snow has nearly no impact on the failure strain which is about 6-7% for FTB with initial water contents of 6%, 13% and 20%. However, for the FTB with an initial water content of 25%, the failure strain is drastically influenced by the amount of added snow. The failure strain ranges from 10% to about 20% and it is observed that high snow content corresponds to a large failure strain. This is due to the viscoelastic property of the added snow. The porosity of a specimen with high snow content is larger than that of a low one. So, the former displays a long dense stage before failure occurs compared to the latter. The optimum total water content to

obtain high strength and modulus of elasticity is found to be 25%-35%. Variation depends on the snow content.

The compression pressure (vertical pressure) has a drastic impact on the mechanical behaviour and properties of FTB with an initial water content of 25%. It is found that higher applied vertical pressures result in higher UCS and E values. The UCS values range from 3200 kPa to 7800 kPa and the E values range from 55 MPa to about 140 MPa which correspond to a vertical pressure from 25 kPa to 400 kPa. It is also observed that the failure strain decreases with an increase in the applied vertical pressure from about 15% to about 10% which corresponds to the applied vertical pressures of 400 kPa to 25 kPa. Still another apparent characteristic is the residual strength of FTB which is also increased with an increase in the applied vertical pressure.

CHAPTER 5:

SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

FOR FUTURE RESEARCH

5.1 Comparison between FTB and FCPB behaviour

5.1.1 Stress- Strain Behaviour

The FTB specimens display similar mechanical behaviours as frozen soils. If the freezing effect is removed, the strength of the specimen will approach zero. The mechanical behaviour and properties of FCPB are governed by the coupled effects of ice-strengthening and cement bonding. For the designed compositions (i.e. cement content and binder types) and curing conditions (i.e. -6°C , relative humidity 67%), it can be deduced that the ice-strengthening effect is a dominant factor. This is because the strength of the FCPB is higher than that of CPB (Figure 5.1). It can be inferred that if the FCPB specimen experiences a melting process, it can still hold some strength.

Figure 5.1 shows the typical stress-strain curves for FTB and FCPB specimens subjected to the same mechanical testing conditions (e.g., same constant strain rate) and cured at a sub-zero temperature (-6°C). It can be found that the FTB specimen displays strain

softening, but the FCPB specimen displays strain hardening behaviour. The former can be explained as the “pressure melting” phenomenon (Andersland and Anderson, 1978; Zhu and Carbee, 1984). However, the latter is due to the interlocking effect enhanced by the hydrates of large molecules (Liu et al., 2011).

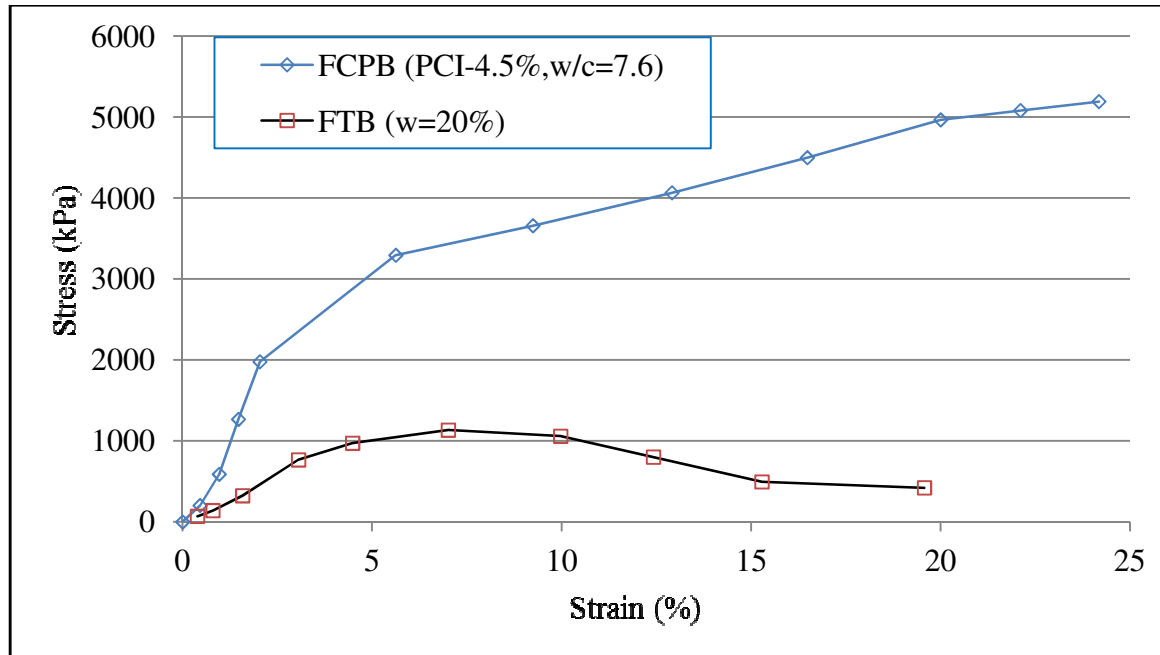


Figure 5.1 A sample of a comparison of the typical stress-strain curves of FTB and FCPB

5.1.2 Strength behaviour

The strength development of FTB and FCPB is very different. The (peak) strength of FTB is reached in a relatively short period (low strain), but the strength of FCPB is considered as that which corresponds to a 20% strain. It can be noted from Figure 5-1 that there is no “peak stress” for FCPB even if the strain is increased to 25%. However, FTB reaches its peak stress at a strain of about 6-7%.

Figures 5.2 to 5.4 illustrate the significant difference between the strength of FCPB and that of FTB with various initial water contents, 25% (FTB#1), 20% (FTB#2) and 13% (FTB#3).

It can be easily observed that a common characteristic of Figures 5.2 to 5.4 is that the UCS values of FCPB are much higher than those of FTB regardless of the binder content and type, curing time and the mix components of the FTB. This finding can have significant practical application in mine backfill operations in permafrost areas. This means, an additional small amount of binder to the mine backfill could allow the development of a high strength CPB.

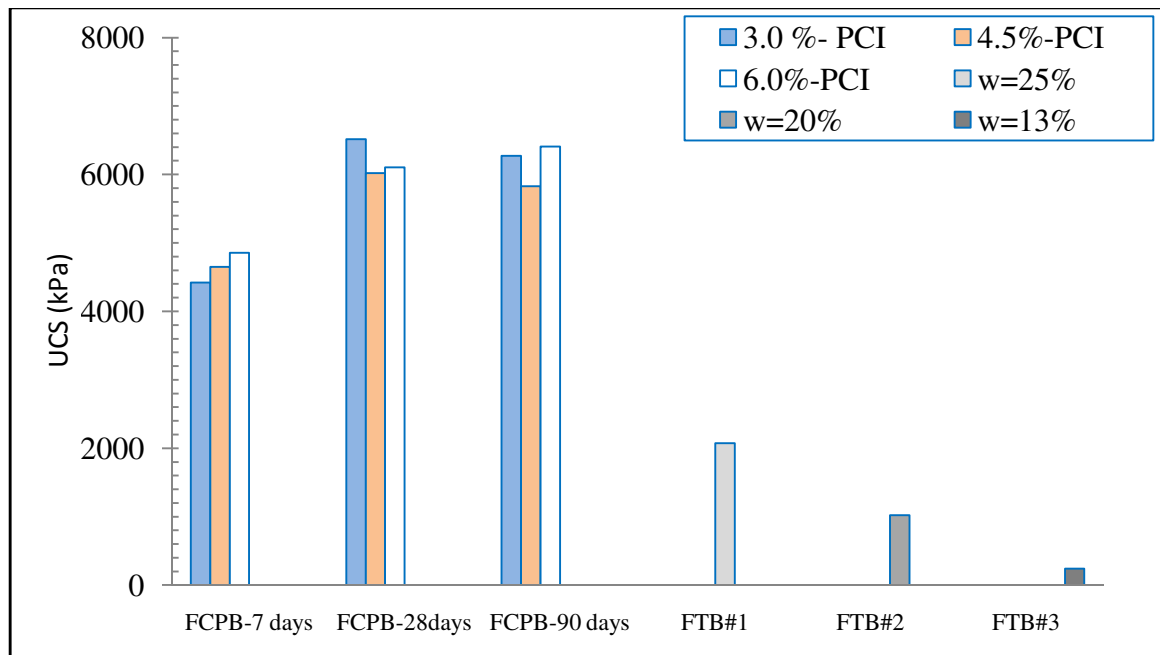


Figure 5.2 Comparison of the strengths of PCI-FCPB and FTB

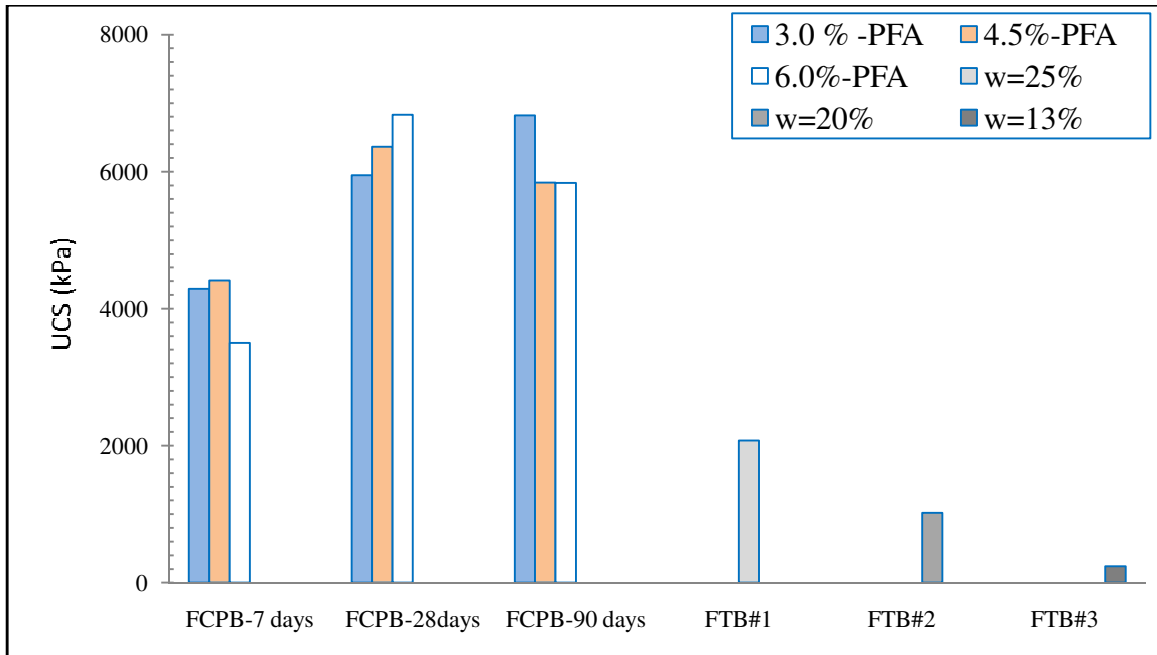


Figure 5.3 Comparison of the strengths of FA-FCPB and FTB

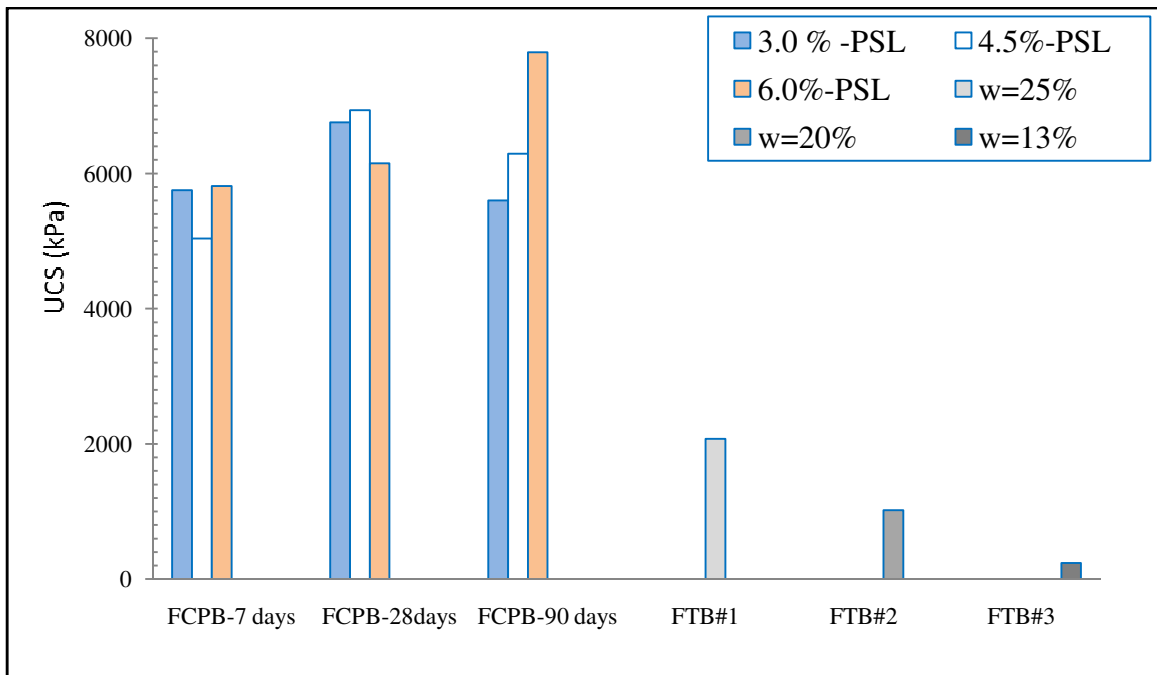


Figure 5.4 Comparison of the strengths of FSL-FCPB and FTB

5.2 General summary and conclusions

The mechanical behaviours and properties of FTB and FCPB are studied. It is found they are very different with regards to strength development and deformation behaviour.

The mechanical behaviour and properties of FTB are strongly influenced by water content, snow content, tailings fineness, and compaction pressure, whereas those of FCPB are significantly affected by cement content, curing age, and binder type. Other possible factors, such as curing temperature and testing conditions, need further studies to determine their effects.

It is found that FTB materials display strain softening, but FCPB materials display strain hardening behaviours. After the initial yield point, there is a long strain softening process for FTB materials. However, there exists a long strain hardening process for FCPB. It is noticed that the FCPB material does not reach peak strength even when the strain is up to 25%.

The failure mode of FTB can be summarized as relatively brittle because there is visible surface failure which occurred in most tested FTB specimens. However, FCPB materials display ductile property. There is no visible surface failure observed for FCPB materials even if the strain is up to 25%. Figure B.1 given in the appendices shows the typical external appearances of FTB and FCPB specimens after UCS testing has been conducted.

The cement hydration process can either contribute positively (UCS increase) or negatively (UCS decrease) to the strength of FCPB. On the one hand, the cement hydration process sucks unfrozen water, and hydration products bond the solid particles of the FCPB system. On the other hand, hydration heat results in pore ice melting thus increasing the unfrozen water content. The former is considered to contribute to strength, but the latter is the opposite. The final effect depends on which process dominates.

5.3 Recommendations for future research

Even though several of the most possible influencing factors have been investigated and a great deal of lab tests have been carried out on FTB and FCPB materials in this study, there are still many factors that need to be quantitatively determined. For FTB, the factors can be: freezing temperatures, testing conditions (i.e. confining pressure, loading rate), the effect of mine process waters, and reactive tailings. For FCPB, the testing conditions and tailings fineness are considered to require further studies.

The micro-structure and mineralogical evolution of FTB and FCPB need to be studied for a better understanding of the mechanisms of strength development for these materials.

Simulating the filling rate of FTB and FCPB in underground supporting systems and performing related tests might be valuable for a better understanding of their mechanical behaviors and properties.

Better knowledge of the thermal properties of FTB and FCPB materials might be useful for constructing a coupled thermal-chemical-mechanical model to describe the geotechnical behaviour of these materials.

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Appendices

A. Introduction of Uniaxial Compression Strength (UCS) Tests

A.1.1 Uniaxial compression tests

Uniaxial compressive testing is quick and easy to perform, and a cost effective method to obtain unconfined compressive strength (UCS) and stiffness (E) as well as the mechanical behaviour of frozen soils. The apparatus used for performing UCS tests are: i) loading machine, and ii) data acquisition system. The equipment and the installation are shown in Figure A1.1.

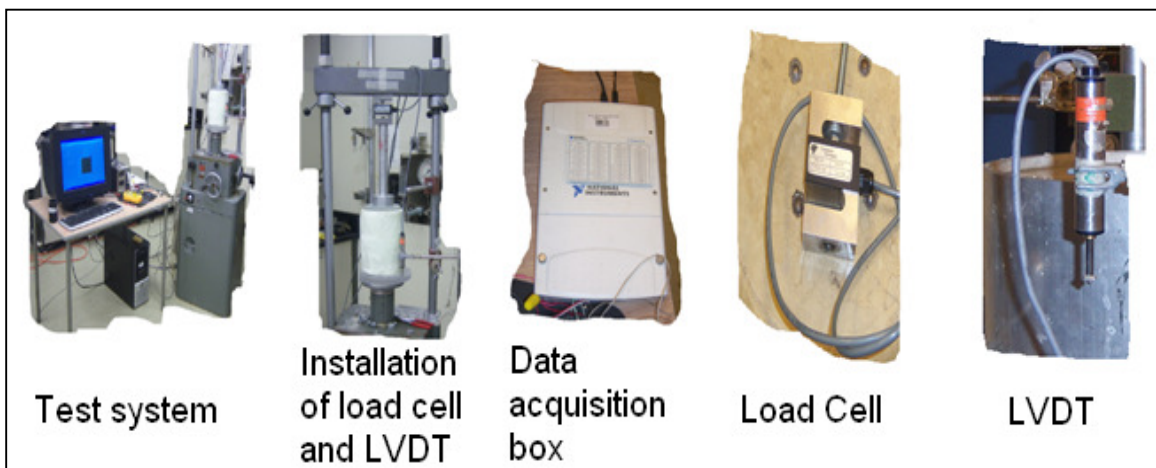


Figure A1.1 Test equipment, apparatus and the installation

i) Loading machine: A motor with a constant head speed machine and a manually controlled speed system is used to perform uniaxial compression tests. The loading rate is varied from 0.000024 in/min to 0.300 in/min. The selected loading rate can be obtained

by changing the pair of wheels as well as switching the gears in the transmission box. A constant loading rate of 0.06 in/min is used for this study.

ii). Data acquisition system:

A LabVIEW VIs driven data acquisition system is used to adapt and store the transient (one second) load and deformation information, while the test is being conducted. The transient load and deformation information, as well as the initial size (diameter and height) of the specimen are then used to calculate the transient stress and strain. A stress-strain curve automatically displays on the screen to monitor the progress of the testing.

A load cell with the capacity of 5000 lbs (0.03% accuracy) is used to measure the applied load and a linear variable differential transformer (LVDT) transducer with a 25 mm travel range was used to measure the deformation of the specimen. The deformations are measured between the loading platens and the initial deformation between the specimen ends. The loading surface of the platens is assumed negligible.

A.1.1.1 Test procedures

Before performing the UCS tests, the specimens need to be set into a cold chamber. This installing process should be finished in the cold room to avoid any heat impact from the indoor air (temperature about 22°C). If this is not carried out, the impact on the mechanical behaviours of the frozen specimen will be unacceptable. The outer layer around the cylindrical specimen and the two ends of a specimen will to some degree, melt during the operating process, if the procedure is carried out indoors.

The procedures for carrying out the UCS testing are as follows:

- i) starting the LabVIEW VIs and setting the constant compression head speed;
- ii) opening the cold chamber (which is stored in the cold room);
- iii) setting the cold specimen into the cold chamber;
- iv) covering the specimen with snow to a height that is 1/2-2/3 of the specimen;
- v) closing the cold chamber;
- vi) mounting the cold chamber onto the loading machine;
- vii) setting the measuring apparatus (load cell, LVDT); and
- viii) starting the testing.

A.1.1.2 Data extracting and interpolating methods

This method is based on ASTM D7012-10: Standard test method for compressive strength and elastic moduli of intact rock core specimens under varying states of stress and temperatures. It is used to calculate the mechanical properties and find representative stress to strain points to construct stress-strain plots which reflect the geotechnical behaviour of FCPB.

- i) The stress which corresponds to a 20% strain is selected as the failure strength (peak strength) because there is no maximum stress achieved within the maximum strain. The initial stress-strain point is found as the first point to calculate the E;

- ii) 40% of the strength is calculated and the corresponding strain is determined, and this point is used as the second point to calculate the E;
- iii) the final stress-strain point of the test is determined;
- iv) the points between the initial point, the 40% strength point, the strength point and the final point are interpolated, and the stress-strain plot based on these points and the interpolated points are constructed; and
- v) the method (c) that is shown in Fig. A.1.1 (ASTM D7012-10) is used to calculate the E. The strength is fixed at 40% for this calculation.

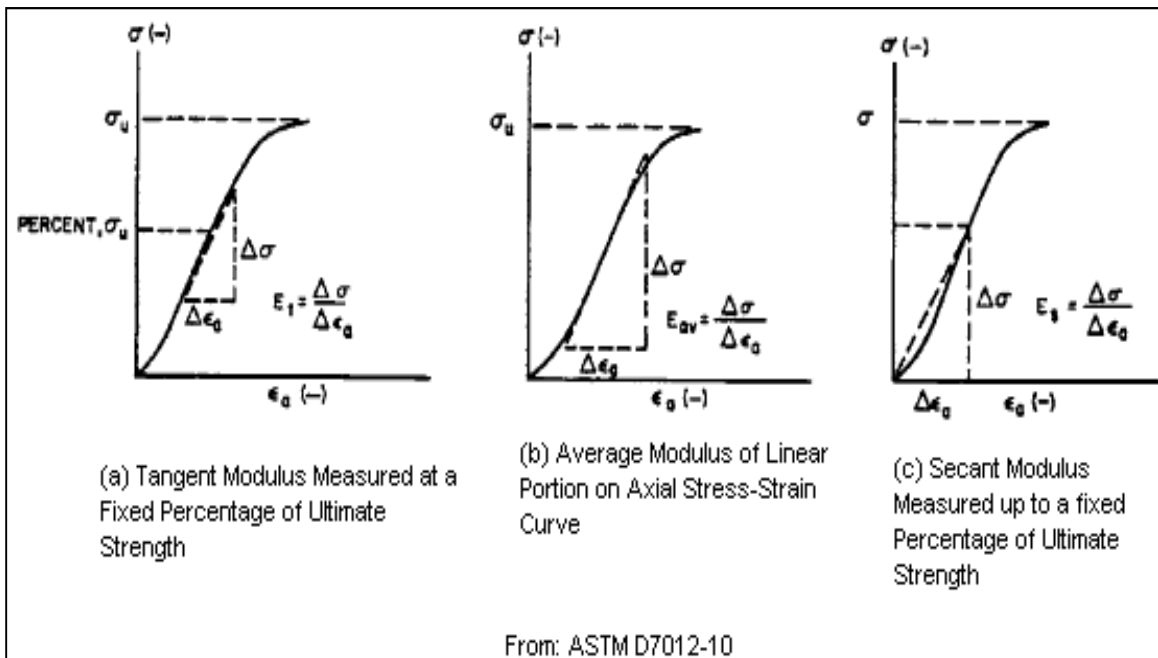


Figure A1.2 Schematic which illustrates the method for data extraction and interpolation

A.1.2 Procedures for compressing the tailings into the cylinders

- i) Vertical pressure is quickly applied onto the ring (Fig.A.1.2) with determined values;
- ii) the determined pressure is used for 10 seconds, then released;
- iii) Steps 1 and 2 are repeated one more time for each layer;
- and iv) another layer is compacted into the cylinder.

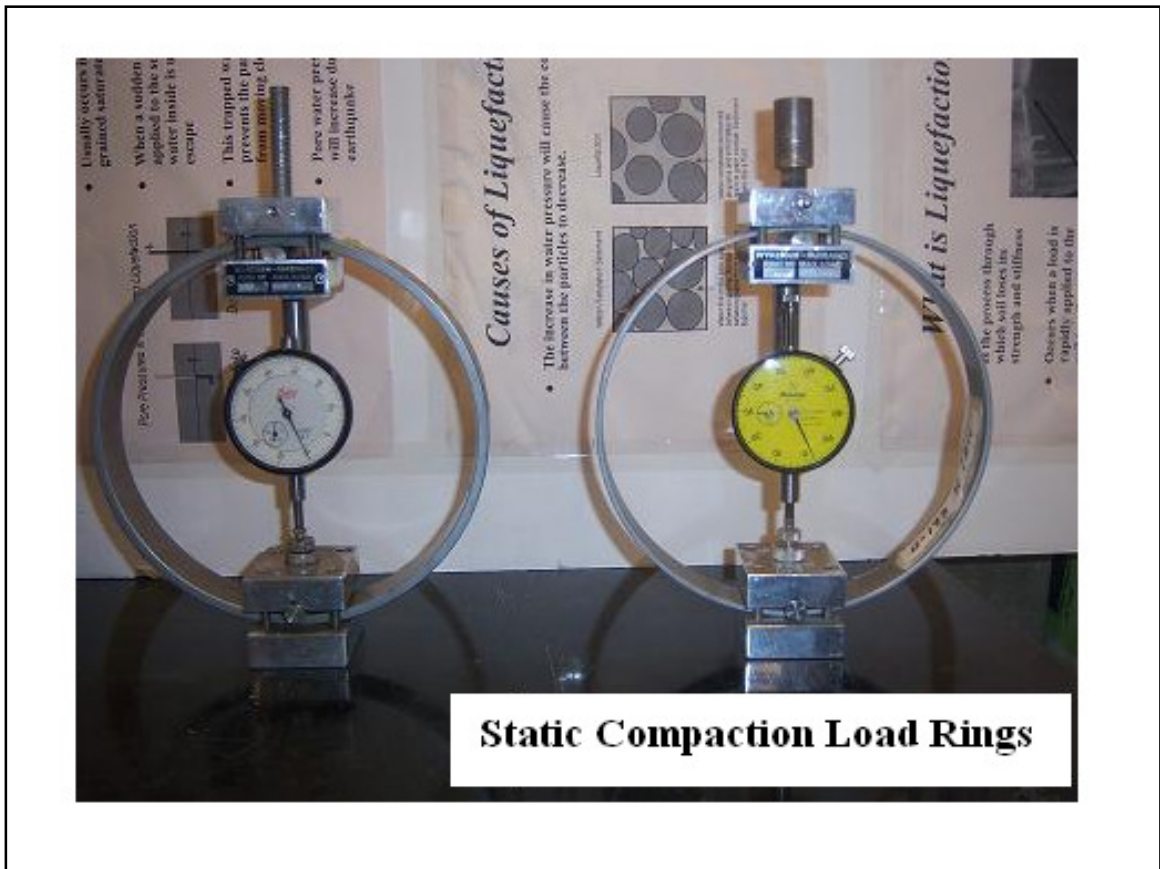


Figure A1.3 Rings used to measure the compacting pressure in this study



Figure A1.4 Sample curing equipment and control system

B. Contrast in the surface of FTB and FCPB after UCS testing

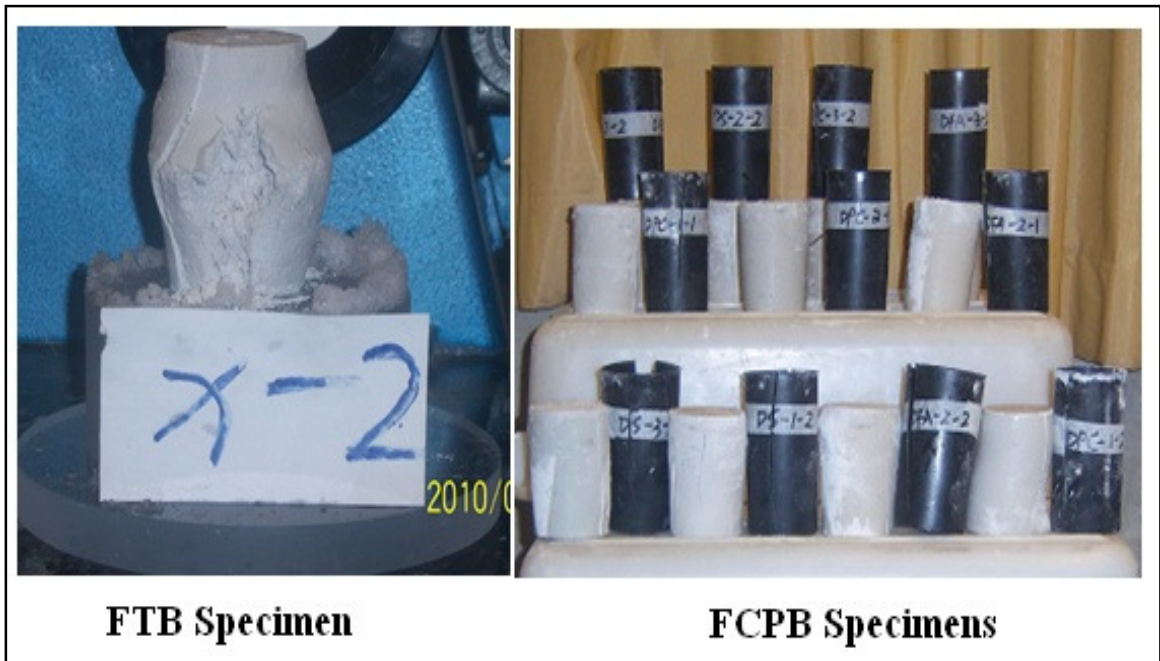


Figure B.1 Typical appearance of FTB and FCPB specimens after UCS testing

C. UCS Test Results for FCPB

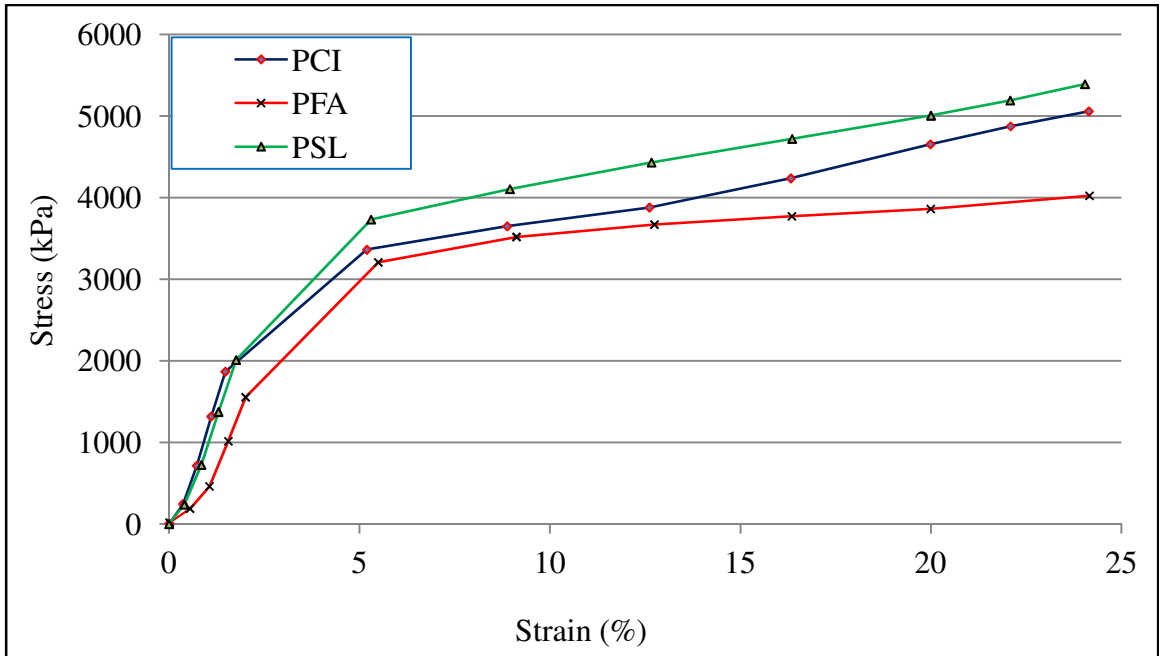


Figure C.1 Effect of binder type on the stress-strain behaviour of FCPB (cement content, 3%; curing age, 7 days)

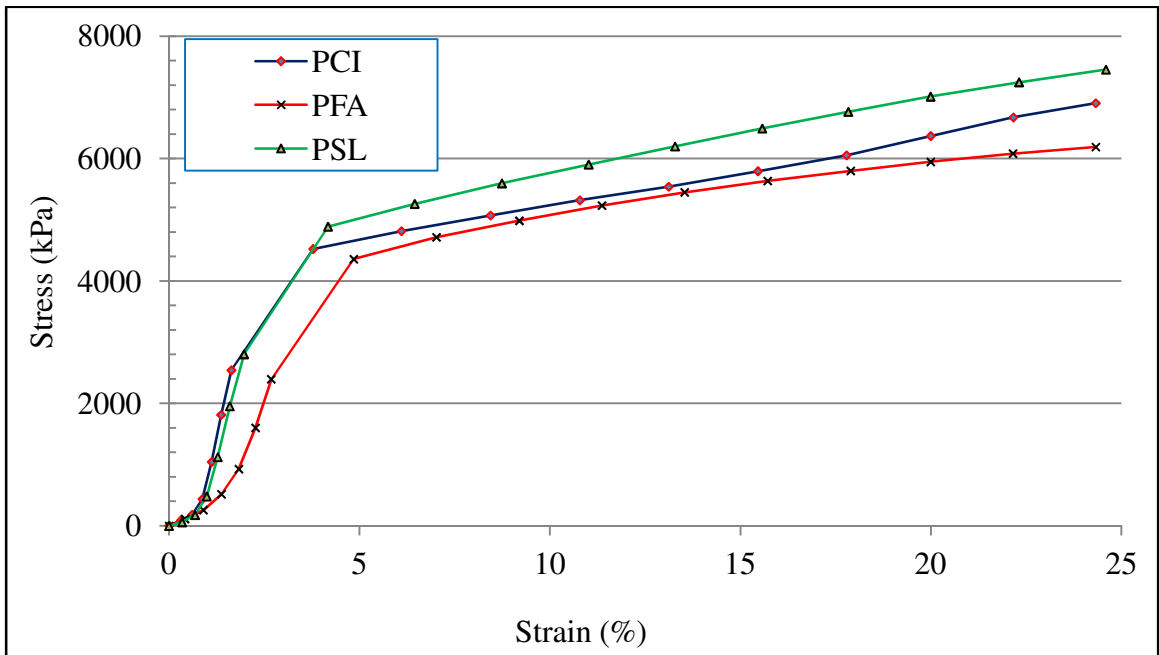


Figure C.2 Effect of binder type on the stress-strain behaviour of FCPB (cement content, 3%; curing age, 28 days)

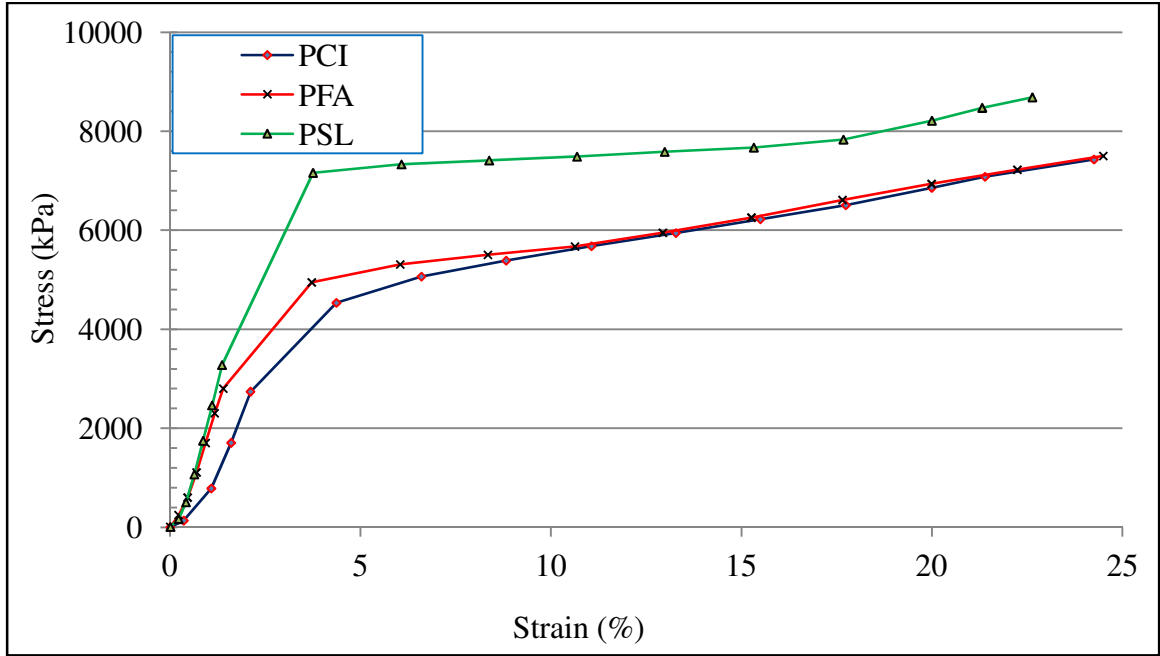


Figure C.3 Effect of binder type on the stress-strain behaviour of FCPB (cement content, 3.0%; curing age, 90 days)

The effect of the binder type on the stress-strain behaviour, as shown in Figs. C.4 to C.6, varies with the curing time.

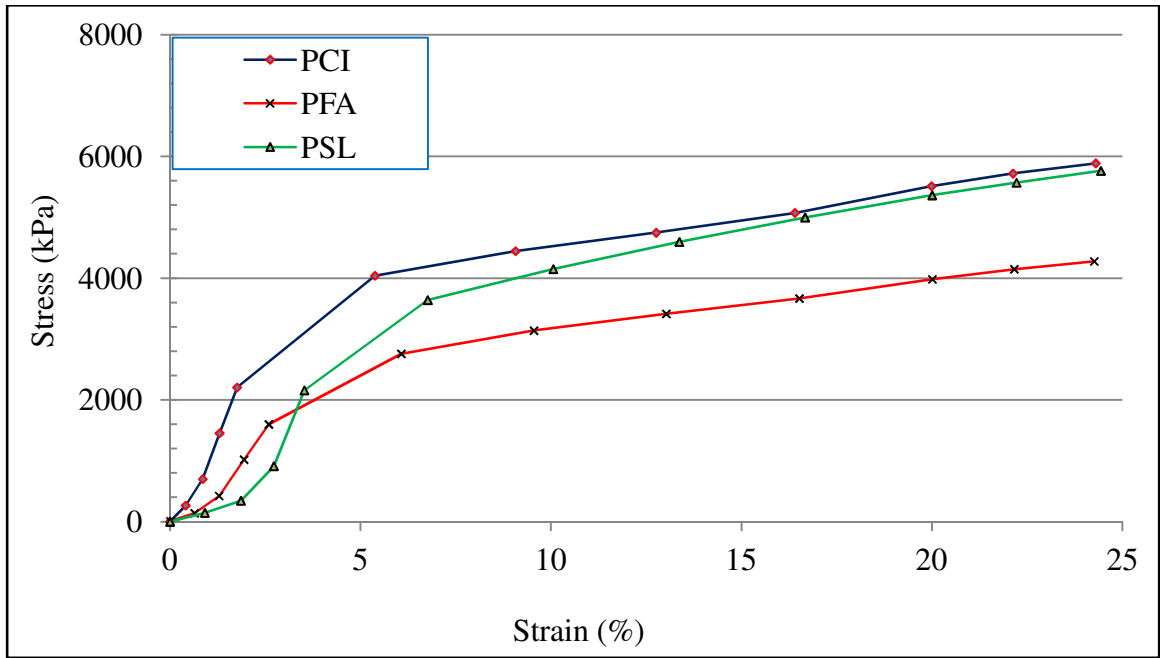


Figure C.4 Effect of binder type on the stress-strain behaviour of FCPB (cement content, 6.0%; curing age, 7 days)

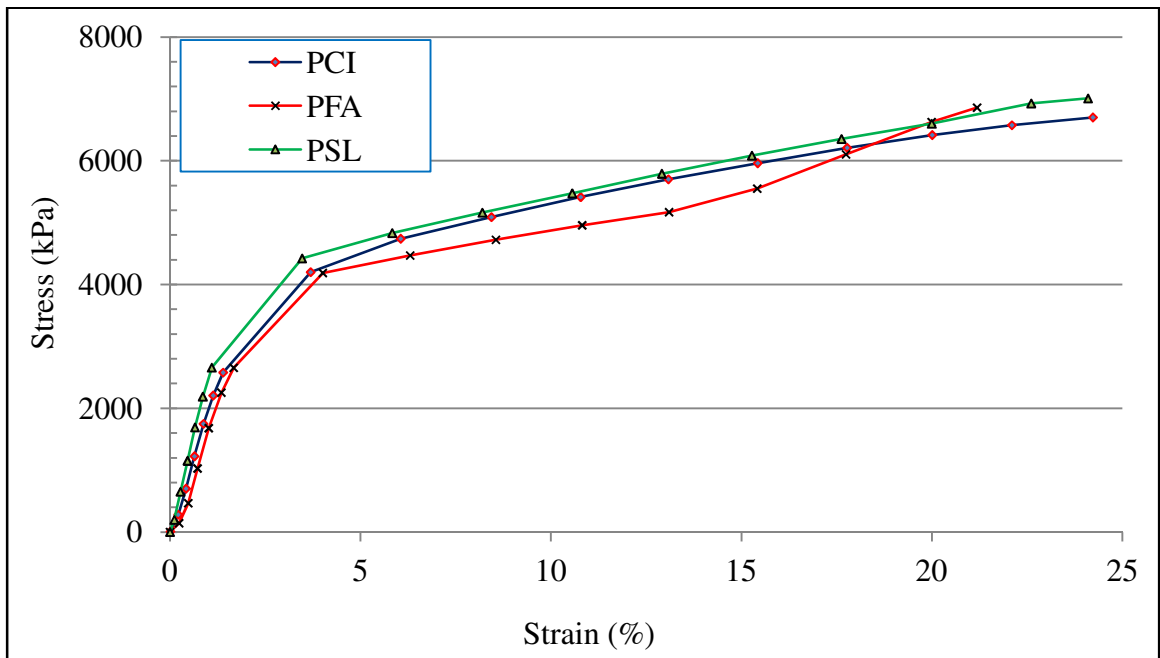


Figure C.5 Effect of binder type on the stress-strain behaviour of FCPB (cement content, 6.0%; curing age, 28 days)

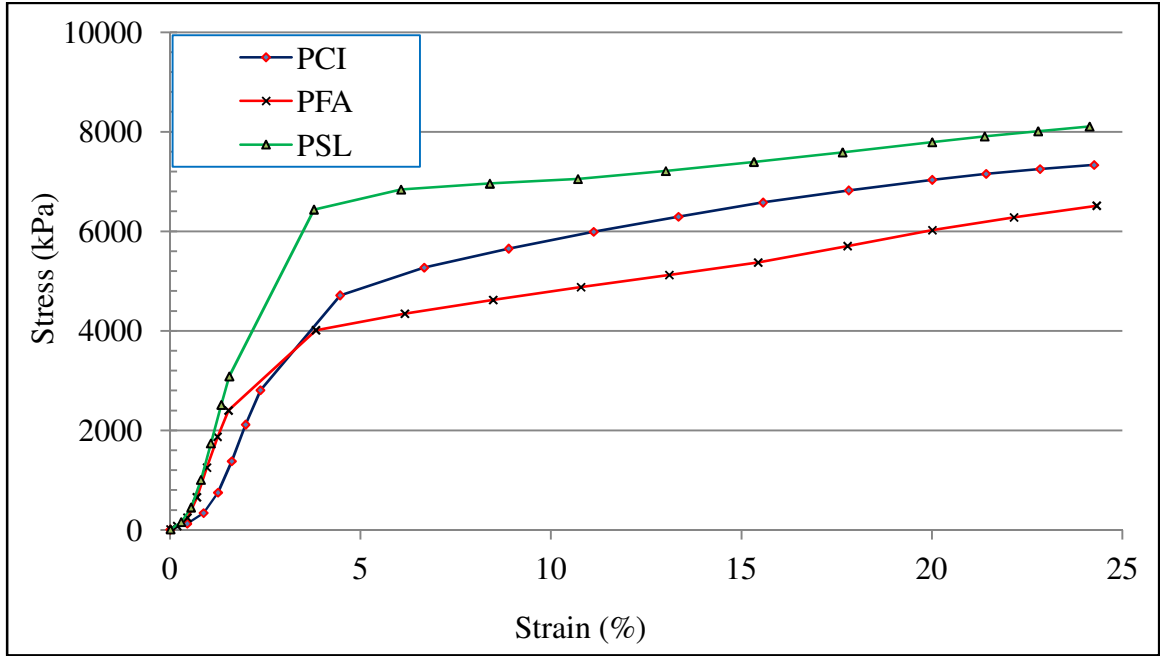


Figure C.6 Effect of binder type on the stress-strain behaviour of FCPB (cement content, 6.0%; curing age, 90 days)