January 2014

Structural and Durability Performance of Precast Segmental Tunnel Linings

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A thesis submitted in partial fulfillment of the requirements for the degree in Doctor of Philosophy

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STRUCTURAL AND DURABILITY PERFORMANCE OF PRECAST SEGMENTAL TUNNEL LININGS

(Thesis format: Integrated Articles)

by

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Abstract

Tunnels play a key role in assisting the movement of people, goods, and special services. The functionality of tunnels depends on the structural and durability performance of its lining system. Tunnel lining systems act as lines of defense against large overburden loads and complex geotechnical surrounding conditions. The use of precast concrete tunnel linings (PCTLs) has been escalating due to its efficient and economical installation process compared to that of normal cast in-situ lining practice. Normally, PCTL segments are designed for 100 years of service life. However, tunnel structures often suffer premature degradation primarily due to reinforcement corrosion, which requires costly repair and maintenance. Corrosion induces distress in PCTL leading to micro- and macro-cracking and consequently spalling of the concrete cover, which in turn accelerates damage. Hence, addressing and solving such durability-sustainability dilemma of precast tunnels is a must.

The main aim of this dissertation is to evaluate the structural and durability performance of full-scale conventional reinforced concrete (RC) tunnel lining segments in comparison to steel fibre-reinforced concrete (SFRC). Moreover, the potential for implementing ultra-high performance fibre-reinforced concrete (UHPFRC) in precast tunnel lining systems was also investigated. Tunnel lining segments behaviour was evaluated under simulated field-like conditions including physical and chemical loads encountered during the service life of PCTL segments.

Physical loads consisted of replicating loading conditions induced as a result of surrounding ground stresses, ground settlement, rock expansion behaviour, vehicular accidents inside tunnels and stresses induced by tunnel boring machines during installation of PCTL segments. On the other hand, chemical loads were simulated by exposing PCTL specimens to various chloride ions solutions while monitoring the changes in their visual appearance, cracking patterns and mechanical degradation. Finally, finite element analysis (FEA) was performed in order to verify the experimental findings.

Experimental results showed that the peak load capacity of RC PCTL segments was higher than that of the corresponding SFRC segments; however, SFRC achieved higher cracking resistance and more progressive post-peak behaviour. The tested segments exhibited
comparable behaviour under thrust loading action without cracking and spalling of concrete, indicating the adequate ability of segments to sustain TBM installation loads. The durability assessment showed that the external specimens (extrados and intrados faces) of both the RC and SFRC PCTL segments exhibited lower chloride ion penetration owing to surface treatment using a cement slurry. The chloride ion diffusion coefficient was a function of the exposure period and concentration of the salt solution. SFRC PCTL segments better resisted the ingress of chloride ions than control RC segments. Results indicate that conventional RC PCTL segments are more vulnerable to corrosion damage compared to that of SFRC PCTL segments.

Moreover, the load carrying capacity of UHPFRC tunnel lining segments linearly increased with higher fibre dosage and followed the rule of mixtures, regardless of the fibre length. UHPFRC exhibited higher cracking resistance, leading to better durability properties compared to that of conventional RC and SFRC owing to its very low porosity and denser micro-structure. Furthermore, no deterioration of UHPFRC mechanical properties was observed after various chloride ions exposures.

Based on structural and durability results, it can be concluded that the UHPFRC is a very promising alternative for conventional RC and SFRC tunneling segments. The very high strength and ultra-durable nature of UHPFRC can allow reducing the cross-sectional dimensions of PCTL segments, leading to reduced material cost and more sustainable construction. In addition, UHPFRC PCTL segments can eliminate the laborious and costly manufacturing of curved shape steel cages, which mitigates the corrosion problem, leading to enhanced service life of lining systems at low production cost.

**Keywords:** Precast; concrete; tunnel; linings; chloride; ions; corrosion; structural; durability; fibre; reinforced concrete; ultra-high; performance; flexural, thrust, settlement; finite element.
Co-Authorship Statement

This thesis was prepared according to the integrated-article layout designed by the Faculty of Graduate Studies at Western University, London, Ontario, Canada. All the work stated in this thesis including experimental testing, data analysis, finite element modeling and writing draft manuscripts for publication was carried out by the candidate. The role of the research supervisor and any other co-author (if applicable) was to advise, help in establishing the experimental procedures, proof reading of initial drafts and to help in the development, analysis and submission of the final version of manuscripts. The following publications have been either accepted or submitted to peer-reviewed technical journals and international conferences:


Dedication

To,

My Father: Malik Fakhar Hussain

My Mother: Nasreen Akhtar, “God bless her soul”

My Brothers: Ali, Fuzail and Ammer

My Sisters: Shamsa and Taskeen

My Beloved Wife: Engr. Sbahat
Acknowledgements

First of all, thanks to Almighty Allah for providing me such opportunities and capabilities that led me towards the completion of this dissertation. I would like to state my sincere appreciation and humble thankfulness to my worthy research advisor Prof. Dr. Moncef Nehdi, for his invaluable guidance, supervision, encouragement and support throughout this study.

I would like to acknowledge the contribution of the Canadian precast plant to this research by providing the full-scale PCTL segments and helping in the coring of test specimens at their site. Several companies including BASF and Lafarge are also appreciated for their material donations.

Special thanks to the work-study undergraduate students who helped during the specimens’ preparation for various tests. The role of the University Machine Shop in fabricating the load and reaction frames and segment molds is appreciated. I would also like to thank all technicians and staff members of the Department of Civil and Environmental Engineering at Western University, Canada who helped during this research.

Finally, I would like to thank my father and mother (God bless her soul) who sacrificed a lot in order to provide me with such opportunities that led me towards success. This degree would have not been possible without their sincere encouragement and prayers throughout my life. I would like to express my deep gratitude and appreciation to my brothers and sisters. Furthermore, I would like to acknowledge my wife’s support, encouragement and patience, which played a vital role towards the completion of this study.
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INTRODUCTION

Tunnels are defined as “the covered passage ways with vehicles or subways access that is limited to portals regardless of structure types and construction techniques” according to the American Association of State Highway and Transportation Officials (AASHTO) Technical Committee for Tunnels. Tunnel structures do not include the enclosed highways, subways or railways bridges (Hung et al., 2009). Road tunnel structures are mainly required to overcome the physical obstructions (e.g. mountains or water bodies) in order to facilitate existing highways, subways or railways (Hung et al., 2009). Tunnel structures significantly reduce traffic congestions, leading to improved environmental quality parameters such as noise and air pollution. Moreover, tunnels are viable means of protecting and preserving surface landscapes, cultural heritages and historical buildings, leading towards green and sustainable civil infrastructures (Hung et al., 2009).

1.1. TUNNEL LININGS

An important component of tunnel infrastructure is the tunnel lining systems. The functionality of tunnels significantly depends on the structural and durability performance of its lining system. Tunnel linings act as protective barriers against large overburden loads and complex geotechnical surrounding exposure conditions. The use of precast concrete tunnel lining (PCTL) systems in tunneling projects has been increasing as a result of its efficient and economical application in comparison with the conventional in-situ lining technique (Elliott, 2002). PCTL segments are suitable for both soft and hard ground and can serve both as preliminary and final support against large overburden loads (Hung et al., 2009).

Tunnel linings are normally constructed in a circular shape using tunnel boring machines (TBMs). A number of precast segments are installed at the end of the TBM and assembled in such a way it completes the circle of the tunnel lining (De Waal, 2000). The number of segments required to complete a circle of the tunnel depends on many parameters including the aspect ratio of the segment, diameter of the tunnel and the contractor’s choice. Typical
thickness of segments varies from 200 to 300 mm (8 to 12 in) along with 1000 to 1500 mm (40 to 60 in) width (Hung et al., 2009).

PCTL allows speedy construction along with superior quality due to enhanced control during precast segment fabrication in precast plants. Moreover, the fabrication of PCTL includes repetitive steps of batching and casting of concrete, which ultimately results in wastage reduction compared to traditional in-situ concrete lining (Hariyanto et al., 2005).

Multi-disciplinary skills are required for the designing of PCTL segments in order to meet their structural and durability performance. Thus, a detailed life-cycle analysis is required in order to calculate the total fabrication and installation cost of PCTL systems that satisfy specific design performance criteria (Hung et al., 2009). Generally, the required service life of tunnel linings is considerably higher than that of other structures (e.g. bridges and buildings) (Hung et al., 2009); therefore, special considerations should be given in selecting the PCTL materials to satisfy the structural needs and result in long lasting life with minimum maintenance requirements.

1.2. PROBLEM STATEMENT

Normally, PCTL segments are designed for 100 years of service life (Hung et al., 2009) with conventional steel rebar reinforced concrete (RC). However, cases such as the Koblenz Railway Tunnel, Switzerland; the London Underground Railway Tunnel, UK; and the Michigan Northeast Raw Water Tunnel, USA all suffered premature deterioration before achieving their respective service life. This was mainly attributed to reinforcement corrosion induced by chloride ions penetration (ITA, 1991). Chloride ions from the underground water can attack the extrados faces of PCTL, while de-icing salts carried by vehicular tires can attack the intrados faces. Once these chloride ions reach the embedded reinforcing rebar, it disrupts the passive layer around the rebar and corrosion starts. The formation of corrosion products can induce internal pressures in the concrete surrounding the corroded rebar, thus leading to concrete cracking and spalling of the concrete cover (Uji et al., 1990). Moreover, as the effective cross-section of the rebar decreases, the load carrying capacity of PCTL segments will decrease, which can jeopardize its structural integrity (Broomfield, 2007).
The corrosion of reinforcing steel is the most costly and challenging deterioration mechanism in RC structures. It was the primary reason for several dire structural collapses such as a parking garage in Minnesota and the Berlin Congress Hall (Isecke, 1983; Borgand et al., 1990). In Canada, the annual cost of repairing corrosion induced damage in RC structures was estimated at $3 billion (Davis, 2000). In the United States, the reinforcement corrosion problem costs the economy about $100 billion each year, nearly 1% of the nation’s gross domestic product (Whitmore and Ball, 2004).

From a structural point of view, crack developments in RC PCTL segments during their fabrication, delivery to the job site and installation process using TBM (due for instance to accidental thrust and impact loads) will disturb its normal functioning. In addition, such cracks will facilitate the intrusion of aggressive species, consequently accelerating the corrosion process and leading to decreased structural strength. It was found that the chloride ion diffusion into concrete was directly proportional to the developed crack width (Mangat and Gurusamy, 1987; Tognazzi et al., 1998). Therefore, an alternative higher strength material may be required for more crack resistant and more durable PCTL segments.

1.3. PROPOSED SOLUTION

It is well known that steel fibre-reinforced concrete (SFRC) can better resist crack formation through the crack bridging action of steel fibres. Steel fibres can partially or completely replace traditional reinforcing steel cages in several applications (Plizzari and Tiberti, 2006). It is believed that steel fibres do not allow the onset and propagation of corrosion current due to their discontinuous and dispersed nature. SFRC segmental linings have already been successfully utilized in various tunnelling projects around the world, such as the Line 9 Subway Barcelona, the Madrid Subway, Spain; the Bright Water Sewer System Seattle, USA; the Channel Tunnel Rail Link, UK and the Second Heinenoord Tunnel, the Netherlands (Kooiman et al., 1998; King and Alder, 2001; Woods et al., 2003).

However, the complete replacement of conventional rebar cages with steel fibre reinforcement is not always a feasible option due to higher structural strength requirements. Therefore, an alternative high strength and ultra-durable material is required in order to completely substitute for the conventional steel rebar in PCTL segments without affecting its structural and durability performances. Ultra-high performance fibre-reinforced concrete
(UHPFRC) is an emerging cement-based composite with compressive strength typically higher than 150 MPa (22 ksi) and almost negligible porosity (Graybeal, 2006). Therefore, UHPFRC can be prove to be a more durable and sustainable material for PCTL fabrication. In addition to improving structural and durability properties, complete substitution of conventional steel rebar reinforcement with UHPFRC in tunnel linings can eliminate the laborious and costly manufacturing of curved shape reinforcing rebar cages, which require complicated welding and detailing. Furthermore, the cross-sectional dimensions of UHPFRC lining segments can be reduced owing to its high strength properties, leading to more economical construction.

1.4. RESEARCH NEEDS AND MOTIVATION

Tunneling engineers are increasingly specifying precast concrete tunnel linings (PCTLs) due to its cost effectiveness and enhanced quality. However, the durability of conventional RC PCTL segments is a major challenge facing tunneling stakeholders and transportation authorities. In particular, corrosion of reinforcing steel in PCTL is a prominent deterioration mechanism, requiring costly maintenance and repair. Therefore, a detailed study is required in order to examine the physical (visual) and mechanical degradation of PCTL segments under various corrosive environments.

Generally, the corrosion problem was studied by exposing small beams or prisms in the laboratory under accelerated corrosive environment. However, the corrosion potential in actual field tunneling specimens has so far been largely unexplored. While, the chloride ions penetrability and build-up process in conventional concrete were well studied, to the best of the author knowledge, only scant data on chloride penetrability and build-up process of SFRC and UHPFRC are available in the open literature. Corrosion associated damage issues such as visual appearance and cracking patterns, mass loss, compressive strength loss, tensile strength loss and flexural strength loss on actual field PCTL specimens after various chloride exposures are still lacking in the state-of-the-art knowledge.

Furthermore, previous studies mainly focused on the static flexural resistance of PCTL in order to evaluate its structural properties. However, the performance of full-scale PCTL segments under cyclic loads that simulate their seismic behaviour still needs dedicated research. The settlement and punching behaviour of full-scale PCTL segments that simulate
forces induced as a result of vehicular accidents inside the tunnel and/or settlement due to loose soil underneath the PCTL segments and expansion of rocks or other geotechnical surrounding conditions were ignored in previous studies. Moreover, so far the flexural and thrust load resistance of UHPFRC tunnel lining segments has not been investigated. Therefore, this study was planned in order to fill these knowledge gaps regarding the durability and structural properties of tunnel lining segments.

The findings of this study can motivate the use of UHPFRC PCTL segments, leading to significant reduction in the PCTL production cost through saving the time and effort of fabricating conventional rebar reinforcement cages, while mitigating costly maintenance and repair of corrosion induced damage.

1.5. SPECIFIC RESEARCH OBJECTIVES

In order to accomplish the above-mentioned research needs, the specific research goals are as follows:

1) Explore the flexural and thrust load resistance of full-scale conventional RC and SFRC PCTL segments, along with their cyclic flexural behaviour in order to evaluate their seismic performance.

2) Investigate the settlement and punching behaviour of full-scale RC and SFRC PCTL segments in order to evaluate the effects of real field scenarios such as vehicular accidents, settlement of soil underneath the segments and expansion behaviour of surrounding rock formations.

3) Quantify the chloride ions penetration and corrosion potential of full-scale conventional RC and SFRC PCTL segments to better estimate their durability performance.

4) Establish materials and engineering properties of UHPFRC in order to characterize the effects of the steel fibre length and dosage on the mechanical and durability properties of UHPFRC mixtures suitable for the fabrication of PCTL segments.
5) Evaluate the bending and thrust load resistance of UHPFRC tunnel lining segments in order to establish its structural behaviour and verify experimental results through finite element analysis to develop predictive capability for the engineering properties of UHPFRC PCTL.

1.6. STRUCTURE OF THE THESIS

This dissertation has been prepared according to the integrated-article format predefined by the Faculty of Graduate Studies at Western University, London, Ontario, Canada. It consists of nine chapters covering the scope of this study: structural and durability performance of RC, SFRC and UHPFRC tunnel lining segments. Chapter 1 provides an introductory problem statement, research motivation, objectives and original contributions to research.

Chapter 2 reviews the state-of-the-art knowledge on the corrosion problem and its mechanisms along with case studies on tunnel linings. Furthermore, a detailed literature survey was conducted on the mechanical and durability properties of UHPFRC.

Chapter 3 presents the flexural and thrust load resistance of full-scale RC and SFRC PCTL segments. Displacement at mid-span and cracking patterns for both segment types were analyzed. Moreover, flexural cyclic tests were presented and discussed in this chapter in order to evaluate its seismic behaviour.

Chapter 4 explains the experimental results of settlement and punching behaviour of the full-scale RC and SFRC PCTL segments.

Chapter 5 outlines the chloride ions penetration properties of full-scale RC and SFRC PCTL segments. Chloride diffusion coefficients for various exposure solutions and durations were analyzed in order to evaluate the concrete quality used in fabricating the full-scale tunnel lining segments.

Chapter 6 deals with the corrosion potential of full-scale RC and SFRC PCTL segments. The loss in mechanical properties and physical deterioration due to various chloride exposures of specimens (cores and sawed beams) retrieved from full-scale RC and SFRC lining segments were presented in this chapter.
Chapter 7 presents the experimental evaluation of the UHPFRC material properties. The effects of the steel fibre length and dosage on the mechanical and durability properties of UHPFRC were discussed.

Chapter 8 evaluates the structural behaviour of UHPFRC tunnel lining segments. Furthermore, finite element analysis using commercially available software ABAQUS was conducted in order to verify the experimental behaviour and comparison of the performance of UHPFRC with RC and SFRC tunnel lining segments was presented in this chapter.

Chapter 9 summarizes the research conclusions and future recommendations.

1.7. ORIGINAL CONTRIBUTIONS TO RESEARCH

This research addresses a practical and full-scale infrastructure problem: substantial financial resources are spent worldwide for the maintenance and repair of tunnel linings because of their premature deterioration. In the first phase of this study, actual full-scale precast tunnel lining specimens, manufactured for a subway tunnel in Canada, were investigated for their structural and durability properties in order to realistically simulate field conditions. The second phase of this research program evaluates the potential of UHPFRC for the fabrication of tunnel lining segments. This research should benefit both the design engineer and tunneling contractors, since it provides a more durable and sustainable design solution to mitigate the corrosion and associated damage while yielding enhanced mechanical and structural behaviour, which should result in substantial savings in the initial cost and life cycle maintenance and repair. The key original contributions of this research include:

1) A detailed investigation and comparison of bending and thrust load resistances of full-scale RC and SFRC PCTL segments will provide a benchmark for tunneling engineers in assessing the prototype capacities of segmental linings. Furthermore, the evaluation of the cyclic behaviour of tunnel lining segments adds to the existing knowledge on the structural seismic behaviour of full-scale PCTL segments.

2) The settlement and punching behaviour of full-scale RC and SFRC PCTL segments will assist the design engineers to consider various real field tunneling scenarios such
as vehicular accidents, surrounding ground settlement and expansion behaviour of rocks.

3) The chloride ions penetration and corrosion initiation process in full-scale RC and SFRC PCTL segments were examined by simulating realistic field conditions and environmental exposures. Herein, the developed experimental data should allow engineers to further improve the durability design of PCTL segments.

4) The development of detailed database on the engineering and material properties of UHPFRC mixtures suitable for industrial fabrication of full-scale PCTL segments should represent a quantum leap in the tunneling industry.

5) It was demonstrated that complete replacement of conventional steel rebar reinforcement in tunnel lining segments can be made using UHPFRC owing to its exceptional structural and durability properties. Therefore, this study should provide a source of inspiration and a comfort level to tunneling engineers, pre-casters, contractors and clients in order to explore the development of novel UHPFRC PCTL with unique durability performance.

6) It was observed that the initial cracking load of UHPFRC tunnel lining segments is much higher than that of conventional RC and SFRC segments. Therefore, the durability properties of PCTL can be better ensured by utilizing this promising UHPFRC tunneling system.

7) The excellent engineering properties and durability of UHPFRC tunnel lining segments should allow reducing the lining thickness, thus leading to decreased self-weight. This will facilitate the handling and installation of tunnel lining segments at the job site.

It is expected that the present study could develop into an UHPFRC PCTL initiative in North America in the near future. Furthermore, it is envisioned that the development of such UHPFRC super-durable tunnel lining segments could increase the interest in replacing the conventional steel rebar RC lining segments, leading to more durable and sustainable construction.
1.8. REFERENCES


LITERATURE REVIEW

Concrete is highly alkaline in nature (i.e. pH of 12 - 13) due to presence of sodium, calcium and potassium oxides in its pore structure. These oxides assist in the formation of a passive layer around the steel rebar. The passive layer is basically a thin and impenetrable film of metal oxides and cement minerals, which protects the steel rebar and does not allow the initiation of corrosion activity (Broomfield, 2007; ACI 222, 2001).

2.1. CORROSION MECHANISM

The reinforcing steel rebar corrosion in concrete memebers is similar to an electrochemical reaction where the pore solution in the concrete matrix works as an electrolyte, while the reinforcing steel rebar acts as an electrical conductor (ACI 222, 2001). The corrosion initiates due to the release of electrons during the oxidation process at the anode (Eq. 2.1). These electrons are utilized at some other location in order to maintain electrical neutrality. Therefore, electrons are collected at the cathodic site where hydroxyl ions form upon reaction with oxygen and water (Eq. 2.2) (Broomfield, 2007).

\[ Fe \rightarrow Fe^{+2} + 2e^- \]  \hspace{1cm} \text{Eq. 2.1}

\[ 2e^- + H_2O + \frac{1}{2}O_2 \rightarrow 2OH^- \]  \hspace{1cm} \text{Eq. 2.2}

It should be noted that the anodic and cathodic reactions are the initial steps of forming corrosion or rust. The availability of oxygen and moisture are the main controlling factors that accelerate the corrosion process. Several reactions occur between iron and hydroxyl ions in the presence of oxygen and moisture to form rust. Equations 2.3 to 2.5 show the series of chemical reactions that take place during the corrosion process (Broomfield, 2007).

\[ Fe^{+2} + 2OH^- \rightarrow Fe(OH)_2 \]  \hspace{1cm} \text{Eq. 2.3}

\[ 4Fe(OH)_2 + O_2 + 2H_2O \rightarrow 4Fe(OH)_3 \]  \hspace{1cm} \text{Eq. 2.4}
2Fe(OH)₃ → Fe₂O₃·nH₂O + 2H₂O \hspace{1cm} \textbf{Eq. 2.5}

*Ferric hydroxide → hydrated ferric oxide (rust)*

Where, \( n \) in \textbf{Eq. 2.5} is variable depends on the availability of oxygen and water.

### 2.2. CORROSION EFFECTS IN RC STRUCTURES

The corrosion of steel reinforcement has detrimental effects on the normal functioning of RC structures. A reduction in rebar diameter takes place, which adversely affects the performance of RC structure due to a decrease in flexural capacity. This can significantly reduces the service life of RC structures. Furthermore, the volume of rust can be 6 to 10 times more than the actual volume of steel. Therefore, this excess volume exerts pressure on the concrete in the surrounding area, leading to micro- and macro-cracking and consequently concrete spalling take place when it exceeds the tensile strength of concrete. Initially, some rust stains and hair-line cracks are observed at the surface. These cracks map the rebar locations and provide an easier access for the intrusion of oxygen, moisture and harmful species which further accelerates the damage. The sectional loss due to spalling of concrete decreases the cross-sectional dimensions of the structural elements, leading to reduced load carrying capacity and possibly structural safety risks (Broomfield, 2007).

### 2.3. CAUSES OF CORROSION

The corrosion of reinforcing steel initiates with the disruption of the passive layer around steel rebars. There are two main mechanisms, i.e. carbonation and chloride ions attack that can break the passive layer and disturb the integrity of structural elements (Broomfield, 2007; ACI 222, 2001).

#### 2.3.1. Carbonation

Carbonation is the chemical reaction between the alkaline hydroxides in concrete and carbon dioxide (CO₂) from the surrounding environment (\textbf{Eq. 2.6}). This will form carbonic acid (H₂CO₃) which neutralizes the alkaline phase of concrete. The calcium hydroxides (Ca(OH)₂) present in the pore solution react with H₂CO₃ and lower down the pH value due to the formation of calcium carbonates (CaCO₃) (\textbf{Eq. 2.7}). When the pH value drops below 10, the
passive layer is disturbed, leaving the steel rebar unprotected against corrosion activity (Broomfield, 2007).

\[
CO_2 + H_2O \rightarrow H_2CO_3 \quad \text{Eq. 2.6}
\]

\[
H_2CO_3 + Ca(OH)_2 \rightarrow CaCO_3 + 2 H_2O \quad \text{Eq. 2.7}
\]

The carbonation induced corrosion depends on many factors including concrete quality and porosity, concrete cover and the carbon dioxide percentage in the surrounding environment (Broomfield, 2007). Carbonation can be easily monitored using a pH indicator. The application of a phenolphthalein indicator on the exposed concrete surfaces becomes clear at low pH regions, indicating carbonation activity, and changes color to pink for un-carbonated regions (Broomfield, 2007).

### 2.3.2. Chloride Ions Attack

Chloride induced corrosion is one of the most costly and challenging deterioration mechanisms facing civil infrastructure engineers and other industry stakeholders. Chlorides can be present inside the concrete due to the use of sea water, addition of accelerators or chloride polluted aggregates. Other sources of chlorides include sea salt spray or direct sea water wetting and drying in case of offshore structures and the use of de-icing salts (Broomfield, 2007). It was estimated that around 10 million tonnes of de-icing salt are sprayed on highways/freeways every year in North America (TRB, 1991). Therefore, concrete highways, bridges and tunnels are highly vulnerable to chloride induced corrosion.

Chloride ions attack the embedded reinforcement by disturbing the passive layer. The ferrous ions at the anode react with the chloride ions to form a soluble compound. This soluble compound turns into an insoluble compound iron hydroxide at the cathode, leaving the chloride ions. The free chloride ions are again available at the anode to eliminate iron ions from the steel rebar reinforcement and the process continues (ACI 222, 2001).

The penetration/intrusion of chloride ions into hardened concrete matrices is mainly due to a diffusion process (Bamforth and Chapman-Andrews, 1994; Polder and Larbi, 1995). Hence, Fick’s second law has been widely used to analyze chloride ions diffusion into
Conrete. Considering non-steady state situations (i.e. the concentration changes with time), the law can be expressed as shown in Eq. 2.8 (Crank, 1975):

$$\frac{\partial C}{\partial t} = D \frac{\partial^2 C}{\partial^2 x} \quad \text{Eq. 2.8}$$

Where, $C$ is the chloride ions content; $D$ is the chloride diffusion coefficient; $t$ is the chloride exposure period, and $x$ is a depth variable. This equation can be solved using the following boundary conditions (Crank, 1995; Stanish et al., 1997): (i) the chloride content at the surface remains constant ($x = 0, t > 0 \rightarrow C = C_0$); (ii) initially, the chloride ions content in the concrete is zero ($x > 0, t = 0 \rightarrow C = 0$); and (iii) the chloride ions content is zero at a very far distance from the surface ($x = \infty, t > 0 \rightarrow C = 0$). Applying these boundary conditions and substituting in Eq. 2.8, the Cl$^-$ concentration can be expressed as follows (Crank, 1995; Stanish et al., 1997):

$$C(x, t) = C_0 \left[ 1 - \text{erf} \left( \frac{x}{\sqrt{4Dt}} \right) \right] \quad \text{Eq. 2.9}$$

Where, $C_0$ is the surface chloride content and erf is the error function.

The barrier against Cl$^-$ to reach and attack the embedded steel reinforcement is the concrete cover. Increasing the concrete cover thickness and/or reducing its permeability will delay the corrosion initiation and extend the propagation period, which improves corrosion durability. However, the concrete cover thickness cannot be increased beyond certain limits due to structural, dimensional restrictions and practical reasons (Sibbick, 1993).

2.4. CORROSION IN TUNNEL LININGS

Generally, tunnel structures are subjected to both mechanical and environmental loads due to high in-situ soil stresses and atmospheric pollution (Usman and Galler, 2013). Highways or subways tunnel structures are more vulnerable to chloride induced corrosion due to their underground construction below the water table, leading to reduced load carrying capacity (Zhiqiang and Mansoor, 2013), and consequently life threatening risks. Furthermore, in railway tunnels, the stray currents generated at the traction due to rail movements accelerate the chloride build-up process and further aggravate the situation (ITA, 1991). Chlorides may penetrate due to water infiltration from the water table and disturb the extrados faces of
tunnel linings (FHWA, 2005; Hung et al., 2009). Moreover, the intrados faces of tunnel linings may also get corroded due to de-icing salts carried by vehicular tires. Table 2.1 shows case histories of various tunnels which got corroded mainly due to chloride ions ingress (ITA, 1991).

2.5. ULTRA-HIGH PERFORMANCE CONCRETE

Ultra-high performance concrete (UHPC) is a novel construction material exhibiting enhanced mechanical and durability properties, which can lead to economical construction through reducing the cross-sections of structural members leading to materials savings and lower installation and labor costs (Tang, 2004). The relatively high initial cost of UHPC has restricted its wider use in the construction industry. However, ongoing research and investigations are filling knowledge gaps in order to commence innovative UHPC having reduced initial cost.

Furthermore, the development and wide acceptance of an UHPC design code should encourage stakeholders in the construction industry to implement large scale applications. Examples of UHPC potential applications include the construction of new structures, rehabilitation works, architectural structural and non-structural elements, machine parts and military structures. Table 2.2 shows some of these applications around the world.

2.5.1. UHPC Composition and Mixture Design

The key factor in producing UHPC is to improve the micro and macro properties of its mixture ingredients to ensure mechanical homogeneity, maximum density and dense particle packing (Schmidt and Fehling, 2005; Shah and Weiss, 1998; Vernet, 2004; Wille et al., 2011). The particle size distribution should be selected in such a way that the bigger size particles be surrounded by at least two layers of smaller size particles. For instance, Richard and Cheyrezy (1995) proposed an optimum ratio of 13; the bigger to smaller size particles to achieve dense microstructure (i.e. the cement particle size should be 13 times smaller than the fine aggregate size). Table 2.3 shows the range of UHPC constituents used in various studies for the successful production of UHPC (Fehling et al., 2004; 2008; 2012).
2.5.1.1. Binders

In UHPC, a high proportion of cement is used compared to that of normal strength (NS) and high-performance concrete (HPC) (Schmidt and Fehling, 2005). It was observed that increasing the cement percentage increased the UHPC compressive strength; however, after reaching an optimum value (around 1700 kg/m$^3$ (106 lb/ft$^3$) of cement content), a declining behaviour in compressive strength was observed due to limited participation of aggregates (Talebinejad et al., 2004). Cement with moderate Blaine fineness (4000 cm$^2$/g (281200 in$^2$/lb)) and tri-calcium aluminate ($\text{C}_3\text{A}$) content lower than 6% should be preferred due to its lower water demand (Wille et al., 2011).

Due to the very low water/binder ratio (w/b), only part of the total cement hydrates in UHPC and the un-hydrated cement can be substituted with crushed quartz, fly ash or blast furnace slag. For instance, up to 30%, 36% and 40% by volume of cement in UHPC mixtures can be replaced with crushed quartz, blast furnace slag and fly ash, respectively, without compromising the compressive strength (Ma and Schneider, 2002; Soutsos et al., 2005; Yazici, 2006).

Moreover, the addition of silica fume can improve the workability of UHPC and fill voids between coarser particles due to its finer size and spherical shape, thus enhancing the strength properties through its pozzalonic reactions (Ma and Schneider, 2002; Richard and Cheyrezy, 1995). Various studies (Matte and Moranville, 1999; Ma and Schneider, 2002; Xing et al., 2006; Chan and Chu, 2004) recommended silica fume dosages of 20 to 30% of the total binder material to achieve denser particle packing and pozzalonic reaction, leading to higher strength properties. Wille et al. (2011) recommended 25% by cement weight of low carbon content (< 0.5%) silica fume as an optimum dosage.

2.5.1.2. Water/Binder Ratio

A very low w/b is used in UHPC mixtures. Minimum w/b of 0.08 was reported by Richard and Cheyrezy (1995); however, this ratio did not ensure dense particle packing. An optimum w/b ratio of 0.13 to 0.17 was suggested in previous studies (Richard and Cheyrezy, 1995; de Larrard and Sedran, 1994; Gao et al., 2006; Wen-yu et al., 2004) for maximum relative density and spread flow. However, researchers (Wille et al., 2011; Droll, 2004) achieved
strength higher than 150 MPa (22 ksi) using 0.25 w/b. Therefore, it can be concluded that the w/b is not the sole strength governing parameter of UHPC, but the curing regime, properties of mixture ingredients, mixing procedures and mixer type are also important parameters.

2.5.1.3. Superplasticizer

The reduced workability of UHPC due to its very low w/b can be resolved by adding effective superplasticizers (SP). The required SP dosage significantly depends on the compatibility between the mixture ingredients and the type of SP used. Improved compatibility can lead to lower SP dosage. For example, UHPC incorporating limestone micro-filler is more workable and compatible compared to that incorporating metakaolin at the same SP dosage (Rougeau and Burys, 2004). Furthermore, step wise or delayed addition of SP (rather than at once addition) enhanced the workability of UHPC mixtures due to an improved dispersing effect (Tue et al., 2008). Various studies (Fehling et al., 2004; 2008; 2012) used SP dosage ranging between 1% and 8% by cement weight for enhancing the workability of UHPC mixtures. However, SP dosage of 1.4% to 2.4% by cement weight was recommended (Wille et al., 2011).

2.5.1.4. Aggregates

Generally, failure in conventional concrete is characterized by failure at the interface between the cementitious matrix and aggregates (Jun et al., 2008). Therefore, eliminating coarse aggregates in UHPC mixtures reduces the interfacial weaknesses between the matrix and aggregates. In addition, improving the interfacial properties results in lower porosity in the matrix, leading to enhanced mechanical strength (Mindess et al., 2003). The fine aggregate like quartz sand plays an important role in reducing the maximum paste thickness (MPT), which is also a key factor in the mixture design of UHPC. An optimum ratio for sand to cement was found to be 1.4 for a quartz particle size of 0.8 mm (0.031 in) (Wille et al., 2011).

2.5.1.5. Steel Fibres

Due to its very high strength and homogeneity, UHPC becomes very brittle; yet it can be made ductile by adding steel fibres (Bayard and Ple, 2003; Graybeal, 2006). The most commonly used size of steel fibres is 13 mm (0.5 in) in length and 0.20 mm (0.008 in) in
diameter (Fehling et al., 2004; 2008; 2012). Richard and Cheyrezy (1995) recommended using 2% by mixture volume of steel fibres for an economical and workable UHPC mixture design.

2.5.1.6. **Nano-materials**

The success of mixture design of UHPC is highly dependent on achieving high density and ultra-high consolidation of the concrete matrix. Therefore, the addition of nano-particles produced from silicon dioxide (SiO$_2$), iron oxide (Fe$_2$O$_3$), aluminium oxide (Al$_2$O$_3$), zirconium dioxide (ZrO$_2$) or titanium dioxide (TiO$_2$) can fill the gaps between cementitious materials and fine aggregates, leading to higher packing density. These can also accelerate cement hydration, formation of additional calcium silicate hydrates (C-S-H) through pozzolanic reaction, and reducing calcium leaching and weak zones of calcium hydroxide (Sobolev and Amirjanov, 2004; Bjornstrom et al., 2004; Korpa and Trettin, 2007; Droll, 2004). This can cause significant improvement in mechanical and durability properties of UHPC (Ghafari et al., 2012). Researchers (Ghafari et al., 2012; Shakhmenko et al., 2012) recommended using 1 to 5% by cement weight of nano-particles in mixture design for successful improvement of UHPC material properties.

2.5.2. **Fresh Properties of UHPC**

2.5.2.1. **Fresh Temperature**

The temperature of fresh UHPC mixtures in the literature ranged from 18 to 29 °C (64 to 84 °F) depending on the mixture design, mixing time, admixtures used and ambient conditions (Table 2.4) (Ingo et al., 2004; Kazemi and Lubell, 2012). Furthermore, it was observed that the temperature of fresh UHPC was highly dependent on the placing or pouring method. For instance, it was reported that the temperature of fresh UHPC at the initial storey or ground level was 23 °C (73 °F), but increased to 40 °C (104 °F) at the end of the nozzle when concrete was pumped at higher storey level through long pipes (Ingo et al., 2004). This can be attributed to the increased frictional heat between the UHPC and pipe wall during pumping process.
2.5.2.2. **Air Content**

The reported air content in UHPC mixtures was from 0.3% to 5.4% by mixture volume depending on the mixture design (Wille et al., 2011). Higher w/b and SP dosage increase the air content in UHPC mixtures (Table 2.5). Furthermore, the total air content is highly dependent on the type of concrete mixer used (Ingo et al., 2004). For instance, laboratory mixers with higher mixing speed lead to a sticky consistency of the paste, consequently increasing the air content (approximately 4.3%). On the other hand, ring type mixers usually installed at precast plants apply shear forces that lead to relatively lower air content (approximately 3.2%) for the same mixture composition and proportions (Ingo et al., 2004).

It was reported that an air content below 1% can be achieved using a vacuum accessory with pressure of 50 mbar (Ingo et al., 2004). Also, the placing method significantly affects the air content. For instance, concrete placement into formwork using spiral pump reduced the air content from 2.9% to 1.3% (Ingo et al., 2004). Furthermore, the delayed addition of SP decreased the viscosity of the UHPC mixture and consequently reduced the air content from 2.5% to 1% (Tue et al., 2008). The threshold air content value was considered as less than 2% by mixture volume for improved spread flow and enhanced properties of UHPC mixtures (Wille et al., 2011).

2.5.2.3. **Setting Time**

According to Habel et al. (2006b), the setting time of UHPC is defined as “the time when the mixture attains a stiffness of 1000 MPa (145 ksi) and autogenous shrinkage initiated”. In another study, Graybeal (2006) defined the initial setting time as “a penetration resistance of 3.45 MPa (0.5 ksi) at 15 hours after casting” and the final setting time as “a penetration resistance of 27.60 MPa (4 ksi) at about 18 to 20 hours after casting” based on AASHTO T197 (Standard Method of Test for Time of Setting of Concrete Mixtures by Penetration Resistance).

Generally, the reported setting time of UHPC ranged from 6 to 12 hours depending on the mixture design (Richard and Cheyrezy, 1995; Yoo et al., 2013; Kazemi and Lubell, 2012). However, some studies (Brown, 2006; Morin et al., 2006; Habel, 2004; Graybeal, 2007) showed that the setting time for UHPC can be delayed up to 30 to 40 hours due to the
set retarding effect of high SP dosage. Furthermore, it was observed that the surface covering of freshly mixed UHPC delayed its setting time (Yoo et al., 2013).

2.5.2.4. Workability

The handling of UHPC during casting is a major problem due to its low w/b and reduced flowability. The workability of UHPC is also affected by the addition of steel fibres. Studies showed that UHPC mixtures incorporating fibres with smaller aspect ratio are more workable even at higher fibre dosage compared to that of mixtures with fibres having larger aspect ratio. For instance, 6 mm (0.25 in) long and 0.15 mm (0.006 in) diameter steel fibres can be used up to 10% by mixture volume, while 12 mm (0.5 in) long and 0.15 mm (0.006 in) diameter fibres can be used up to 3% by mixture volume without affecting the mixture workability (Wille et al., 2011; Rossi, 2005).

Wille et al. (2011) recommended adopting 200 to 350 mm (8 to 14 in) limit for the flow diameter spread according to ASTM C230 (Standard Specification for Flow Table for Use in Tests of Hydraulic Cement) for dense UHPC without fibres. Furthermore, spread flow can be increased by utilizing ultra-fine or nano-particle materials. For example, a 16% increase in spread flow was observed with the addition of 1% by cement weight of nano-silica (Shakhmenko et al., 2012).

2.5.3. Mechanical Properties

2.5.3.1. Compressive Strength

Effect of specimen size and shape: It was found that the specimen size has a noteworthy influence on the compressive strength of UHPC. For instance, Skazlic et al. (2008) observed 21% increase in cylinder compressive strength for specimen size of 70×140 mm (2.75×5.50 in) compared to that of 100×200 mm (3.94×7.87 in). This is likely due to the higher probability of encountering larger size flaws in larger specimens (Graybeal, 2006; Ahlborn et al., 2008). Moreover, it was observed that cube specimens exhibited higher strength compared to that of cylindrical specimens (Kazemi and Lubell, 2012; Graybeal and Davis, 2008). Table 2.6 shows proposed conversion factors for various UHPC specimen types and sizes (Skazlic et al., 2008; Graybeal and Davis, 2008; Kazemi and Lubell, 2012). Using 70
mm (2.75 in) cube specimens was recommended considering the machine capacity and cylinder end grinding concerns (Graybeal and Davis, 2008).

Effect of pre-treatment: The rate of hydration in UHPC mixtures can be increased through proper heat treatment. The application of thermal treatment advances pozzolanic reactions, leading to formation of additional calcium silicate hydrates (C-S-H) (Ozyurt et al., 2002; Heinz and Ludwig, 2004). These C-S-H phases fill small pores, leading to denser microstructure and consequently higher mechanical properties (Collepardi et al., 1997; Lee and Chrisholm, 2006; Graybeal, 2006; Cwirzen, 2007; Muller et al., 2008; Lehmann et al., 2009). The cement hydration reactions increase at higher heat treatment temperature. For example, hydration products formation increased from 10% to 55% at eight hours when the temperature was raised from 90 °C to 250 °C (194 °F to 482 °F) (Zanni et al., 1996). Generally, the heat treatment typically applied for UHPC specimens was 90 to 400 °C (194 to 752 °F) for 2 to 6 days (Graybeal, 2006; Richard and Cheyrezy, 1994; Teichmann and Schmidt, 2004; Heinz and Ludwig, 2004). A 40% average increase in compressive strength was observed for 90 °C (194 °F) heat treatment compared to that of untreated control specimens (Bonneau et al., 1997; Soutsos et al., 2005; Xing et al., 2006).

The time of starting the thermal treatment had an insignificant effect on the UHPC compressive strength (Ahlborn et al., 2008). For instance, only 4% difference in compressive strength was observed for UHPC specimens thermally cured for 2 days right after demolding, compared to when the thermal treatment was applied after 10 days from demolding (Ahlborn et al., 2008). This should allow the pre-casters in fabricating various elements at different times and curing them together, leading to energy savings.

Furthermore, during the setting of UHPC, the application of a confining pressure contributes towards increased compactness and denser microstructure, thus leading to higher strength and durability properties (Table 2.7). This can be ascribed to the removal of entrapped air voids and free water (Cwirzen et al., 2008; Richard and Cheyrezy, 1995).

Effect of steel fibres: It was observed that the addition of steel fibres changes the failure mode from complete damage or sudden explosion to a somewhat ductile behaviour where specimens can remain intact without chipping and spalling (El-Deib, 2009).
Various researchers (Reda et al., 1999; Schmidt et al. 2003) reported that the UHPC compressive strength was not influenced by the addition of high dosages of steel fibres. Increased concentration of steel fibres can create fibre bundling, thus leading to weak spots, which can reduce the efficiency of fibres, hence decreasing compressive strength.

A slight increase in UHPC compressive strength due to fibre addition can be observed if proper thermal treatment is applied (Bonneau et al., 1997; Soutsos et al., 2005; Herold and Muller, 2004; Jun et al., 2008). This is mainly a function of type of fibres and their dosage (Bonneau et al., 1997; Herold and Muller, 2004; Soutsos et al., 2005). For example, a 30% increase in UHPC compressive strength was found with the addition of 2.5% by mixture volume of steel fibres when specimens were subjected to a thermal treatment (Graybeal, 2006; Lee and Chisholm, 2006; Bonneau et al., 1997; Soutsos et al., 2005). An increase behaviour in UHPC compressive strength was attributed to the enhanced tolerance of lateral strains owing to steel fibre addition (Orgass and Klug, 2004; Hassan et al., 2012; Kazemi and Lubell, 2012; Magureanu et al., 2012; Ye et al., 2012). Furthermore, it was reported that the addition of fibres resulted in less entrapped air leading to improved density and hence higher compressive strength (Lee and Chisholm, 2005).

*Effect of casting direction:* No appreciable effect of the casting direction on the compressive strength of UHPC was reported in the literature. For instance, Stiel et al., (2004) reported a compressive strength difference of less than 2% for UHPC cube specimens when loaded perpendicular and parallel to the casting direction.

*Effect of loading rate:* Due to its high compressive strength, more time is required to break UHPC specimens at low loading rate. For instance, a 150×300 mm (6×12 in) UHPC cylinder broke after 13 to 16 minutes when a 0.24 MPa/s (35 psi/s) loading rate was applied. Therefore, higher loading rate up to 1.0 MPa/s (150 psi/s) can be applied without significantly affecting the strength properties of UHPC in order to reduce the failure time (Graybeal and Hartmann, 2003; Kazemi and Lubell, 2012). According to AFGC-SETRA (2002) guidelines, a loading rate between 0.24 and 1.7 MPa/s (35 to 250 psi/s) affected the UHPC compressive strength by less than 4%.
2.5.3.2. **Elastic Modulus**

The compressive stress-strain curve of UHPC typically shows a linear elastic portion up to 80 to 90% of the maximum stress value (Cheyrezy, 1999; Graybeal, 2007). It was observed that the addition of fibres in UHPC did not significantly influence its elastic modulus. For example, only 7% increase in the elastic modulus was reported with the addition of 2% by mixture volume of steel fibres compared to that of control UHPC without fibres (Bonneau et al., 1996). Furthermore, the elastic modulus of UHPC is a function of heat treatment (Graybeal, 2006, 2007; Richard and Cheyrezy, 1994). For instance, the elastic modulus increased from 57 GPa to 70 GPa when specimens were subjected to a high temperature of 250 °C (482 °F) for 2 days (Richard and Cheyrezy, 1994). Various models that relate the elastic modulus and compressive strength of UHPC are shown in Table 2.8.

2.5.3.3. **Flexural Strength**

UHPC exhibits high flexural strength properties due to its dense particle packing and steel fibre addition (Kim et al., 2008; Graybeal and Hartmaann, 2003). Researchers (Cheryrezy et al., 1998; Perry and Zakariasen, 2003) reported flexural strength values of up to 48 MPa (7.0 ksi) for UHPC depending on its mixture design and curing regime.

*Effect of sample preparation technique: Concrete casting and pouring direction:* No substantial influence of the casting technique on the initial stiffness of UHPC specimens subjected to bending was observed (Steil et al., 2004). However, it was observed that vertically cast beam specimens showed almost 5 times lower flexural strength compared to horizontally cast beam specimens (Steil et al., 2004). This was attributed to the improved fibre orientation (Steil et al., 2004). Fibres were oriented perpendicular and parallel to the crack surface for horizontally and vertically cast beams, respectively. It was also observed that the failure surface was smoother for vertically cast beams, while horizontally cast specimens showed rougher and wrinkled failure surfaces (Steil et al., 2004).

Moreover, the flexural strength of UHPC was also dependent on the pouring method of concrete into molds (Table 2.9) (Lappa et al., 2004). For instance, the pouring of concrete from the mold end only showed an increased flexural strength by 56% compared to that of the same concrete poured at various locations into the mold (Lappa et al., 2004). This was
attributed to the strong fibre orientation (higher number of fibres crossing at particular sections) parallel to the flow direction (Lappa et al., 2004; Pansuk et al., 2008). Furthermore, Wille and Parra-Montesinos (2012) observed that beam specimens cast only at the middle point exhibited lower peak strength compared to that of similar specimens cast in layers with higher chute speed (0.50 m/s (20 in/s)). A funnel like pattern was observed for beams cast only at the middle point, leading to arranging the fibres along the funnel. However, the beams cast in layers with higher chute speed (0.50 m/s (20 in/s)) formed strong thin layers and desired fibre alignment along the beam axis, thus leading to increased flexural strength (Wille and Parra-Montesinos, 2012). On the other hand, beams cast with lower chute speed (0.13 m/s (5 in/s)) exhibited lower flexural strength compared to that of specimens cast at the middle only. This was attributed to the fact that the slow movement of chute formed thick layers of snake like pattern, leading to vertical orientations of fibres, consequently reducing the flexural strength (Wille and Parra-Montesinos, 2012).

It was observed that the concrete pouring location is also an important factor in achieving higher flexural strength (Table 2.9). For example, beams cast from the mold end exhibited 16% higher flexural capacity compared to that of specimens cast at the middle of the mold (Yang et al., 2010). This was ascribed to the better flow properties of beams cast from the mold end, leading to improved fibre orientation and hence increased flexural strength (Yang et al., 2010).

Effect of fibres: It was observed that fibres significantly affected the UHPC flexural properties (Magureanu et al., 2012; Kazemi and Lubell, 2012). The flexural strength of UHPC increased linearly with increased fibre dosage (Table 2.10) (Kang et al., 2010). An increase in flexural strength by 144% was observed with 2.5% by mixture volume of steel fibres addition compared to that of control beams without fibres (Table 2.10) (Magureanu et al., 2012). The main role of fibres is to prevent the intergrowth of micro-cracks by absorbing tensile stresses and consequently macro-cracks are prevented (Orgass and Klug, 2004). Moreover, beam specimens incorporating fibres showed multiple cracks and exhibited steadier fall in load bearing capacity rather than sudden and rapid drop in load after formation of the first crack (Kazemi and Lubell, 2012). The failure was characterized by a
single vertical macro-crack with multiple micro-cracks for UHPC incorporating steel fibres (Orgass and Klug, 2004).

Furthermore, it was observed that the flexural capacity of UHPC was also dependent on the aspect ratio of fibres. For instance, UHPC mixtures incorporating higher aspect ratio fibres increased the flexural capacity compared to that of those with lower aspect ratio fibres. This was attributed to the fact that mixtures incorporating small diameter fibres (higher aspect ratio) have increased number of fibres per unit volume of concrete, leading to more fibres bridging cracks, and hence increased flexural capacity (Ye et al., 2012).

Furthermore, a hybrid mixture of steel and poly vinyl alcohol (PVA) fibres significantly improved the flexural behaviour of UHPC compared to their individual addition (Bornemann and Faber, 2004). It was also observed that alkali resistant (AR) glass fibres improved the peak load carrying capacity by increasing the energy required for the development of micro-cracks. However, a reduction in ductility using AR-glass fibres was observed compared to steel fibres (Lohaus and Anders, 2004).

**Effect of specimen size:** The flexural strength of UHPC specimens decreased as the specimen size increased (Table 2.10) (Wille and Parra-Montesinos, 2012; Magureanu et al., 2012; Kazemi and Lubell, 2012). For instance, a specimen size of 700×150×150 mm (27.50×5.90×5.90 in) showed a 33% decrease in flexural load capacity compared to that of a similar control specimen with a size of 160×40×40 (6.30×1.57×1.57 in) (Bornemann and Faber, 2004). This was ascribed to the higher wall effect of fibres in smaller specimen (Wille and Parra-Montesinos, 2012; Kooiman, 2000; Magureanu et al., 2012; Kazemi and Lubell, 2012). It was observed that the fibre orientation near the mold surfaces had two dimensional (2-D) patterns which changed to three dimensional (3-D) patterns away from the mold surfaces (Reineck and Greiner, 2007). The 3-D fibre orientation is not favourable for higher flexural strength due to smaller equivalent fibre contents in the direction of flexural stresses. The smaller specimens have more tendency to form favourable 2-D patterns along the cross-section, leading to higher flexural strength (Reineck and Greiner, 2007; Kazemi and Lubell, 2012).
It was also observed that the smaller beam specimens showed higher ductility compared to that of the larger specimens. This was attributed to the improved fibre orientation in smaller specimens (Orgass and Klug, 2004). Furthermore, it was reported that the average number of cracks and their spacing decreased as the specimen size decreased (Nguyen et al., 2013). This was directly related to their increased flexural tensile strain capacity (Nguyen et al., 2013). AFGC-SETRA (2002) recommended a reduction factor of 9% when the specimen height increases from 100 mm (3.97 in) to 150 mm (5.90 in).

**Effect of end support:** It was observed that UHPC beam specimens tested under high frictional support exhibited 30 to 60% higher flexural capacity, depending on the fibre dosage, compared to that of similar beam specimens tested under low frictional support (Wille and Parra-Montesinos, 2012). This was attributed to the increased internal moment due to the additional contribution of the horizontal reaction provided by the frictional force depending on the coefficient of friction (Wille and Parra-Montesinos, 2012).

### 2.5.3.4. Fracture Energy

The fracture energy of UHPC is a complex phenomenon. It is mainly related to the applied loading and crack opening response. Previous studies (Dugat *et al.*, 1996; Gowripalan and Gilbert, 2000; Guvensoy *et al.*, 2004) reported that the fracture energy of fibre-reinforced UHPC can be as high as 30,000 J/m², which is significantly higher than that of NSC (110 J/m²) (Richard and Cheyrezy, 1994). Furthermore, the fracture energy of UHPC is a function of the notch depth in tested specimens (Murthy *et al.*, 2013). It was reported that the fracture energy of UHPC increased with higher dosage of steel fibers (Ioan and Cornelia, 2011; Kreiger, 2012; Noldgen *et al.*, 2013). For instance, approximately 80% increase in fracture energy of UHPC mixture incorporating 2.5% by mixture volume of steel fibers was reported compared to that of a similar UHPC mixture with 1.5% steel fibers (Ioan and Cornelia, 2011). Moreover, fracture energy is highly dependent on the casting direction. For instance, horizontally cast specimens showed approximately 5 times higher fracture energy compared to that of vertically cast specimens (Steil *et al.*, 2004).
2.5.3.5. **Fibre and Rebar Pull-Out (Bond Strength)**

UHPC exhibited high bond strength to rebar and fibres owing to its dense micro- and macro-structures (Holshemacher et al., 2004; Chan and Chu, 2004). The steel fibres pull-out response from UHPC is mainly depend on the fibre orientation and inclination with respect to the loading direction. A 30° inclination with the loading direction resulted in higher pull-out load due to a snubbing effect and concrete matrix spalling (Lee et al., 2010). Moreover, hooked end or twisted fibres exhibited improved bond strength compared to that of straight fibres due to improved mechanical anchorage (Table 2.11) (Wille et al, 2012).

The age of test specimens has minimum effect on the rebar-UHPC initial bond stiffness (Holshemacher et al., 2004). However, a considerable effect of the specimen age was reported on the maximum bond stress depending on the mixture design and proportions (Table 2.11) (Holshemacher et al., 2004). The shape of the bond stress-slip curve is highly dependent on the loading rate. For instance, the smaller the loading rate (0.001 mm/s), the steeper was the ascending slope (higher bond stiffness) and flattens becomes steadier in the post-peak branch of the bond stress-slip plot. However, higher loading rate (i.e. 0.1 mm/s) yielded higher bond stress and corresponding displacement (Holshemacher et al., 2004).

Moreover, it was found that the bond strength of steel rebar embedded in UHPC incorporating steel fibers had less brittle pull-out failure and exhibited larger deformations compared to that of similar rebar embedded in UHPC without steel fibres (Maroliya, 2012).

2.5.3.6. **Reinforcement Cover**

UHPC elements typically require smaller reinforcement cover due to the improved mechanical and durability properties. UHPC incorporating steel fibres exhibits higher tensile properties, thus preventing splitting cracks and concrete spalling (Holshemacher et al., 2004). Furthermore, the decreased permeability due to the very low porosity of UHPC mitigates the intrusion of aggressive species (e.g. chloride ions) into the hardened matrix and the subsequent attack on the steel reinforcement even when the reinforcement cover is relatively thin. Smaller reinforcement covers in UHPC members further reduce the cross-sectional dimensions leading towards economical construction. The recommended UHPC cover is generally 2.5 times the diameter of the reinforcing rebar in order to avoid longitudinal and splitting cracks (Tuchlinski et al., 2006; Schmidt and Fehling, 2005).
2.5.3.7. Shear Resistance

The addition of steel fibres in UHPC can eliminate the requirement of conventional shear reinforcement (Hegger and Bertram, 2008; Fehling and Thiemicke, 2012), leading to more economical and sustainable structures. The steel fibre addition increases the number of micro-cracks and leads to decreased crack spacing and crack width (Fehling and Thiemicke, 2012). An increase in shear strength by 119% and 177% was observed for 0.9% and 2.5% of steel fibres, respectively, compared to that of control specimens without steel fibre addition (Bertram and Hegger, 2012). Schnellenbach-Held and Prager (2012) used micro-reinforcement in the form of thin wires in UHPC in order to further increase the shear capacity of UHPC.

The load-deflection curve of UHPC showed higher stiffness for beams incorporating 1% of steel fibres without stirrups compared to that of similar beam specimens with conventional shear reinforcement stirrups (Fehling and Thiemicke, 2012). Furthermore, it was observed that UHPC beams incorporating 1% steel fibres showed 20% higher shear strength compared to that of UHPC beams incorporating 5 mm (0.20 in) stirrups at 105 mm (4.13 in) spacing (Fehling and Thiemicke, 2012).

Furthermore, UHPC beams cast with pre-stressed reinforcement exhibited higher shear capacity due to the increased arch action compared to that of similar beams cast with conventional reinforced bars without pretressing (Baby et al., 2012; Hegger and Bertram, 2008). It was also reported that the shear capacity of UHPC was highly dependent on the shear span to depth proportion or shear slenderness (a/d) (Cauberg et al., 2012; Hegger and Bertram, 2008; Voo et al., 2010). Higher the a/d, the lower is the shear capacity (Cauberg et al., 2012; Schnellenbach-Held and Prager, 2012; Bertram and Hegger, 2012). The shear stresses decreased to around 70% for 700 mm (27.55 in) deep UHPC specimens compared to that of 400 mm (15.75 in) deep specimens, indicating a significant effect of the beam depth (Bertram and Hegger, 2012).

2.5.3.8. UHPC under Earthquake, Impact or Explosive Loadings

Very limited literature is available on the behaviour of UHPC under vibrant loading such as impact, explosive or earthquake loadings. Rebentrost and Wight (2008) stated that the
application of UHPC reduced earthquake design loads due to decreased overall structural weight, leading to more cost-effective construction.

UHPC has shown excellent performance against impact loading (Bischoff and Perry, 1995). Farnam et al. (2008) tested UHPC panels against an impact load of 8.5 kg and concluded that the member thickness, fibre type, fibre length and fibre dosage were important parameters responsible for impact resistance (Farnam et al., 2008). UHPC has the ability to dissipate higher energy under impact loads than that of NSC due to its high strength and ductility properties (Bindiganavile et al., 2002; Lee et al., 2001).

UHPC application is desirable in military structures where impact resistance due to blast loading is of concern. UHPC structural elements showed a significant improved behaviour against explosive load compared to that of HPC and NSC (Millon et al., 2012). The development of multiple micro-cracks (crack width of about 0.50 mm (0.02 in)) without fragmentation or spalling was observed in UHPC members, leading to decreased global structural damage. The improved cracking behaviour was attributed to the addition of high strength steel fibres, which increased energy absorption capacity and improved ductility (Millon et al., 2012).

2.5.3.9. Fatigue Behaviour

A large scatter exists in fatigue test results of UHPC (Table 2.12). This was attributed to variation in material strength, applied stress level, distribution of fibres, number of fibres at critical cross-sections and test specimen type and size (Table 2.12) (Lappa et al., 2004; Grunberg and Ertel, 2012). UHPC specimens showed no significant sign of failure after $10^6$ load cycles (Bornemann and Faber, 2004). UHPC fatigue results demonstrated that deformations increased rapidly during the initial cycling (up to 5% of the fatigue life) and then grew constantly at the intermediate stage (5% to 95% of the fatigue life). Before the failure point (95% to 100% of the fatigue life), deformation increased rapidly again (Lappa et al., 2004; Grunberg et al., 2008). Under fatigue loading, fibre-reinforced UHPC specimens exhibited large variation in local deformations, indicating the ability the UHPC to redistribute stresses and strains, leading to enhanced fatigue behaviour (Makita and Bruhwiler, 2013). However, an increase in material global strain up to 1.6% (strain hardening stage) decreased the deformation modulus from 39 GPa to 10 GPa (Makita and Bruhwiler,
2013). This was attributed to cracking of the UHPC matrix and fibre pull-out (Makita and Bruhwiler, 2013).

Moreover, Lohaus and Elsmeier (2012), confirmed that UHPC with or without steel fibres exhibited fatigue life considerably higher than prediction of the CEB-FIP Model Code-90. The UHPC fatigue life (number of cycles to failure) decreased with increased load or stress level (Table 2.12) (Lappa et al., 2006). The slopes of the loading and unloading curves for UHPC specimens during cyclic loads were almost identical, indicating an insignificant degradation in stiffness properties (Guvensoy et al., 2004). However, a decreasing trend in residual stresses was observed after achieving the peak stress (Guvensoy et al., 2004). The fatigue fracture surfaces of UHPC incorporating fibres exhibited matrix spalling and fibre abrasion due to snubbing, fretting and grinding effects (Makita and Bruhwiler, 2013).

2.5.4. Durability Properties

2.5.4.1. Porosity and Permeability

UHPC exhibits high durability properties due to a substantial decrease in the number and size of pores (Herold and Muller, 2004; Heinz and Ludwig, 2004). The average pore size in UHPC was found to be less than 5 nm (2×10\(^{-7}\) in) and ranges from 1 to 2% by total volume (Dowd and Dauriac, 1996; Vernet, 2004; Teichmann and Schmidt, 2004; Herold and Muller, 2004; Roux et al., 1996; Heinz and Ludwig, 2004). It was observed that the total porosity is mainly dependent on the heat treatment and w/b (Table 2.13). The total porosity decreased from 8.4% to 1.5% due to heat treatment in UHPC specimens (Cheyrezy et al., 1995; Cwirzen, 2007; Herold and Muller, 2004).

Furthermore, the application of pressure also reduced the overall porosity by removing the entrapped air and additional water (Richard and Cheyrezy, 1995; Bonneau et al. 1997). For instance, Roux et al. (1996) observed 50% reduction in porosity upon the application of pressure during the initial setting time.

UHPC shows very low water absorption capacity, which is approximately 10 and 60 times lesser than that of HPC and NSC, respectively (Roux et al., 1996; Schmidt and Fehling, 2005; Pierard et al., 2009; Ghafari et al., 2012). The water sorptivity coefficient of
UHPC was found to be less than 0.045 kg/m²/h⁰.⁵ (Table 2.14), which is approximately 15 times lower compared to typical HPC.

UHPC exhibits very low permeability to oxygen as compared to that of NSC and HPC. For instance, the oxygen permeability into UHPC is less than 1×10⁻¹⁹ m² (1×10⁻¹⁸ ft²), which is around 10 and 100 times lesser than that of HPC and NSC, respectively (Vernet, 2004). Furthermore, Roux et al. (1996) found zero percent penetration of carbon dioxide (CO₂) into UHPC specimens after 90 days of exposure. However, at later ages, some CO₂ penetration was observed. For instance, a carbonation depth of 0.5 mm (0.020 in) was observed after 6 months of CO₂ exposure (Perry and Zakariasen, 2004; Schmidt et al., 2003). In another study conducted by Schmidt and Fehling (2005), a carbonation depth of 1.50 mm (0.060 in) after 3 years was found for UHPC, which is approximately 2.50 and 4.50 times less compared to that of the HPC and NSC, respectively.

2.5.4.2. Chloride Ions Penetration Properties

The maximum chloride ions content and penetration depth for UHPC specimens reported by various researchers are listed in Tables 2.15 and 2.16. It was observed that the chloride ions penetration is highly dependent on the exposure solution and duration, w/b and curing regimes (Thomas et al., 2012; Scheydt and Muller, 2012). Furthermore, accelerated tests were conducted by applying pressure and voltage on UHPC specimens in order to evaluate the forced chloride penetration depth into UHPC specimens. For instance, Gao et al. (2006) reported penetration depth of 2.7 mm (0.11 in) with hydraulic pressure of 1.6 MPa (230 psi) after 128 hours (Schmidt and Fehling, 2005). The chloride diffusion coefficient (Table 2.17) for UHPC (2×10⁻¹⁴ m²/s) is significantly lower than that of HPC (around 6×10⁻¹³ m²/s) and NSC (around 1×10⁻¹² m²/s) (Roux et al., 1996).

Chloride ions penetration can be measured in terms of the number of coulombs (electric charge) passed through the specimens using the rapid chloride ion penetrability test (ASTM C1202). It was reported that incorporating steel fibres in UHPC did not cause any electrical short circuiting during the rapid chloride ion penetrability test due to their shorter length and randomly discontinuous distribution (Ahlborn et al., 2008; Graybeal, 2006). It was found that the total charges passed through thermally treated UHPC specimens (35 mm (1.4 in) thick) was 22 Coulombs, which is very low compared to that of HPC (215 Coulombs) and NSC
(1735 Coulombs) (Schmidt et al., 2003). It was found that thermal treatment is the key factor controlling charges passed through UHPC specimens (Table 2.18).

### 2.5.4.3. Reinforcement Corrosion

The corrosion rate for reinforcing rebar in UHPC was found to be 0.01µm/year (4×10^{-7} in/year), which is much lower than the limiting value of 1 µm/year (4×10^{-5} in/year), showing no significant potential for the corrosion risk (Roux et al., 1996).

### 2.5.4.4. Freeze-Thaw Damage and Surface Scaling

The reduced permeability and porosity of UHPC enables better resistance to freezing-thawing cycles (Table 2.19) (Bonneau et al., 2000; Graybeal, 2006). For example, no freeze-thaw degradation was observed for UHPC specimens after 800 freeze-thaw cycles, which attributed to lesser number of interconnected pores (Bonneau et al., 1997; Juanhong et al., 2009). The study conducted by Vernet (2004) under marine environment found no appreciable signs of deterioration on UHPC specimens after 500 freeze-thaw cycles (along with wetting and drying). Furthermore, it was reported that the addition of steel fibres in UHPC specimens appeared to lessen the internal material degradation due to freeze-thaw cycling (Cwirzen et al., 2008). No significant length and weight change in UHPC specimens were measured after 300 freeze-thaw cycles (Juanhong et al., 2009; Shaheen and Shrive, 2006).

UHPC showed surface scaling due to de-icing salt of 8 to 60 g/m² (3.03 to 22.78 oz./ft²) after 50 cycles of freeze-thaw (Perry and Zakariasen, 2004; Bonneau et al., 1997). This difference in observed surface scaling values was attributed to the different testing techniques adopted. Studies concluded that the mass loss due to surface scaling in UHPC is well below the limiting values (1000 to 1500 g/m² (380 to 570 oz./ft²)) (Pierard et al., 2012; Cwirzen et al., 2008; Vernet, 2004; Schmidt and Fehling, 2005).

### 2.5.4.5. Alkali-Silica Expansion

Very limited study was conducted on the alkali-silica reaction (ASR) in UHPC. Graybeal (2006) performed ASR tests on small prisms 25×25×280 mm (3.97×3.97×11.02 in) by submerging them in a sodium hydroxide solution for 2 to 4 weeks at 80 °C (176 °F). Average
ASR expansion values of 0.012% and 0.002% at 28 days were observed for untreated and thermally treated specimens, respectively, indicating the significant effect of thermal curing on UHPC ASR expansion. Another study conducted by Moser et al. (2009) showed a maximum ASR expansion of 0.02% for both undamaged and pre-damaged UHPC specimens after 600 days, which is lower than the threshold limit of 0.04% (Moser et al., 2009).

2.5.4.6. **UHPC under Fire**

UHPC structures are more vulnerable to fire and elevated temperatures due to its reduced porosity, which hinders the release of vapour pressure, leading to physical damage (Way and Wille, 2012). However, the use of polypropylene (PP) fibres can mitigate this issue. Various studies (e.g. Schmidt et al., 2003; Heinz et al., 2004) reported that the addition of 0.6% by mixture volume of PP fibres improved the fire resistant properties (prevented spalling) of UHPC since melting of the PP fibres at high temperature creates space to release the build-up of pressure. However, cracks (approximately 0.30 to 0.50 mm (0.012 to 0.020 in) wide) were observed at the specimen’s surface. Furthermore, disintegration of UHPC mechanical properties was observed due to dehydration of calcium silicate hydrate products, chemical decomposition of UHPC materials and thermal expansive damage (Way and Wille, 2012; Pimienta et al., 2012). Table 2.20 shows the mechanical degradation of UHPC properties with increased temperature. It was observed that UHPC with 0.6% by volume mixture of PP fibres showed weight loss less than 9% at 1000 °C (1832 °F) (Heinz et al., 2004). Experimental results showed that UHPC exhibited improved mechanical behaviour at elevated temperature compared to that of NSC and HPC (Hosser et al., 2012).

2.5.5. **Cost Estimation and Sustainability of UHPC**

Generally, the initial material cost of UHPC is higher than that of NSC due to the very high cement content and steel fibre addition in UHPC. Table 2.21 compares the cost of UHPC per unit volume in Europe and North America. In North America, the cost of UHPC is higher than that in Europe due to its limited use.

The application of UHPC results in more sustainable construction due to its better economical, social and environmental impacts (Schmidt and Teichmann, 2007). The overall cost of structures is directly linked with the cross-sectional dimensions of structural elements.
The use of UHPC structural members assists in reducing the cross-sectional dimensions (Hajek and Fiala, 2008). Therefore, the material cost can be reduced compared to that of NSC, even though the cement content required in UHPC is higher. Moreover, the quantity of fine aggregates can be reduced to 30%, while no coarse aggregate is used in UHPC (Walraven, 2002). Racky (2004) concluded that approximately 56% reduction in materials costs can be achieved by utilizing UHPC rather than NSC. The high strength properties of UHPC allow the design of slender structures, leading to reduction in self-weight of the structure due to less use of materials. This can result in decreasing the demolition waste, leading to reduced transportation demand and consequently, lesser effects on the environment (Hajek and Fiala, 2008).

Furthermore, the utilization of by-products such as fly ash instead of cement makes UHPC more sustainable material (Aitcin, 2000). UHPC members require less maintenance cost due to their improved durability characteristics and hence their life-cycle cost can be reduced while longer service life can be achieved (Blais and Couture, 1999; Racky, 2004). Likewise, neighboring communities would not be disturbed by the routines of maintenance or/replacement of the facility, leading to positive social effect. Clearly, not the entire cement content available in UHPC mixtures is hydrated. Thus, recycling of UHPC can be used more effectively because the unhydrated cement content is available for further reactions (Aitcin, 2000).

The favourable environmental impacts of UHPC include less effect on increasing the ozone layer, less potential to harm the environmental quality parameters and less greenhouse gas emission (Schmidt and Teichmann, 2007; Aitcin, 2000; Hajek and Fiala, 2008). A study reported by Schmidt and Teichmann (2007) concluded that the utilization of UHPC results in 50% energy reduction compared to that of NSC. In short, UHPC can be a sustainable material due to its improved durability, ecological factors, economical benefits and its recyclability in various applications.

2.5.6. Current Challenges for Implementation of UHPC

Although UHPC is being utilized in several applications around the world, it still has some challenges for wider implementation, especially in North America. The benefits for this innovative material are still not well known. The following are the major challenges that
need to be addressed in order to provide comfort to stakeholders, designers, contractors and manufacturers for implementing UHPC successfully in the field:

1) A detailed and accurate method for the optimization of UHPC constituents and mixture design is required (rather than relying on trial mixes) for its successful development and implementation in the field.

2) Due to the very low w/b of UHPC, a high energy mixer is required for mixing its constituents. Furthermore, special modifications in precast site mixers are required for the successful production of UHPC precast elements.

3) The flexural properties of UHPC are mainly influenced by the orientation of fibres. Therefore, a method for more effective distribution of fibres in its matrix with desired orientation is required, especially for casting slender elements.

4) Shrinkage strains in UHPC mixtures are higher than that in normal concrete. Therefore, special admixtures or preventive measures need to be developed and implemented for reducing its impact on dimensional stability for full-scale structures.

5) UHPC mixtures are difficult to pour due to its low workability. Therefore, more energy is required to handle the mixture, especially in the case of fibre-reinforced UHPC. Hence, special pouring equipment may need to be developed.

6) The high strength and durability properties of UHPC are highly dependent on thermal treatment. Therefore, special arrangements for thermal curing at on-site construction and at precast facilities are required.

7) Globally accepted design provisions need to be developed in order to provide confidence to the design engineer in utilizing the high strength and other properties of UHPC.

2.6. IN-SITU AND PRECAST CONCRETE TUNNEL LININGS

In-situ concrete tunnel linings can be used as the initial lining in order to support ground stresses or as the final lining system in two pass linings. In-situ linings are suitable for any type of excavation technique and ground irregular shapes developed during excavation.
Normally, a water proofing cover is placed before casting the in-situ linings (Hung et al., 2009). However, there are certain limitations involved in in-situ concrete linings. For example, the proper consolidation of concrete, especially around the reinforcing rebar, is a problem due to limited access for vibrators in tunnel lining construction. This can result in weaker concrete with higher porosity. The increased porosity favourably allows aggressive materials (e.g. chlorides) to enter into the hardened concrete matrix and attack the embedded steel rebar reinforcement. Therefore, before achieving their service life, signs of premature degradation have been observed due to reinforcement corrosion, leading to cracking and spalling of concrete (Hung et al., 2009).

Precast concrete tunnel linings (PCTLs) are normally constructed in circular shapes using tunnel boring machines. Precast segments are fabricated in industrial precast plants and joined together to form a ring of tunnel lining. PCTL segments can be designed for 100 years of service life with minimum maintenance. PCTL provides a complete secure support during excavation and for on-going construction work without any primary ground support. PCTL removes all ancillary works such as formwork and curing, which are required for cast in-situ linings. Moreover, the transportation of PCTL segments inside the tunnel is much simpler in comparison with the transportation of raw materials needed for in-situ casting. PCTL segments are manufactured in factories, so better quality control can be achieved, leading to durable construction with low maintenance costs (Dean et al., 2006; Plizzari and Tiberti, 2006; Burgers et al., 2006; Hung et al., 2009; Moccihino et al., 2010;).

However, PCTL segments are manufactured with a very strict tolerance, thus it may result in large wastage if it is not within the tolerance limits. PCTL segments must be delivered to the site with much care since mishandling may damage segments, leading to edge cracking and spalling. Furthermore, a large storage place is required at the fabrication plant as well as at the construction site. PCTL segments should be stored in stacked form; each stack should contain the segments of one ring. The gasketed lining segments should be installed with great care so that it can serve the design purpose (Hung et al., 2009).

2.7. PREVIOUS STUDIES ON TUNNEL LININGS

Table 2.22 summarizes previous studies conducted on tunnel linings. Most of these studies have evaluated the flexural resistance of tunnel lining segments under static load.
Furthermore, limited studies have been performed on the durability properties (corrosion performance) of full-scale PCTL segments.

Therefore, this research was planned to address the following knowledge gaps: flexural resistance of full-scale PCTL segment under cyclic load to simulate seismic behaviour, performance of PCTL segments under such loads that induced due to vehicular accidents, ground settlement and rock expansion. The behaviour of tunnel lining segments under various corrosive environments simulating the real field scenarios was investigated. Furthermore, the potential of UHPC for the manufacturing of tunnel lining segments was explored and compared to that of conventional RC and SFRC.
2.8. REFERENCES


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Chapter 2

Symposium on UHPC and Nanotechnology for High Performance Construction Materials, Kassel, Germany, pp. 411-418.


### Table 2.1 – Damaged/Corroded tunnels due to chloride ingress

<table>
<thead>
<tr>
<th>Tunnels*</th>
<th>Location</th>
<th>Tunnel type</th>
<th>Diameter</th>
<th>Completion year</th>
</tr>
</thead>
<tbody>
<tr>
<td>Basel/Olten Hauenstein</td>
<td>Switzerland</td>
<td>Railway</td>
<td>-</td>
<td>1916</td>
</tr>
<tr>
<td>Northern Line Old Street to</td>
<td>U.K.</td>
<td>Metro</td>
<td>3.5 m</td>
<td>1924</td>
</tr>
<tr>
<td>Moorgate</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Shimonoseki/Moji Kanmon</td>
<td>Japan</td>
<td>Railway</td>
<td>-</td>
<td>1944</td>
</tr>
<tr>
<td>Mikuni National Route 17</td>
<td>Japan</td>
<td>Highway</td>
<td>7.6 m</td>
<td>1959</td>
</tr>
<tr>
<td>Uebonmachi-Nipponbashi</td>
<td>Japan</td>
<td>Railway</td>
<td>10.0 m</td>
<td>1970</td>
</tr>
<tr>
<td>Dubai</td>
<td>U.A.E.</td>
<td>Road</td>
<td>3.6 m</td>
<td>1975</td>
</tr>
<tr>
<td>Tokyo Underground</td>
<td>Japan</td>
<td>Road</td>
<td>-</td>
<td>1976</td>
</tr>
<tr>
<td>Berlin Tunnel Airport</td>
<td>Germany</td>
<td>Road</td>
<td>23.1 m</td>
<td>1978</td>
</tr>
<tr>
<td>Second Dartford</td>
<td>U.K.</td>
<td>Road</td>
<td>9.6 m</td>
<td>1980</td>
</tr>
<tr>
<td>Mass Transit Railway</td>
<td>Hong Kong</td>
<td>Metro</td>
<td>5.6 m</td>
<td>1980</td>
</tr>
<tr>
<td>Ahmed Hamdi</td>
<td>Egypt</td>
<td>Road</td>
<td>10.4 m</td>
<td>1980</td>
</tr>
<tr>
<td>Stockholm Underground</td>
<td>Sweden</td>
<td>Metro</td>
<td>-</td>
<td>1988</td>
</tr>
</tbody>
</table>

* Data collected from ITA, 1991

### Table 2.2 – Example applications of UHPC around the world

<table>
<thead>
<tr>
<th>Structures/Applications*</th>
<th>Location</th>
<th>Completion/production year</th>
<th>Compressive strength (MPa)</th>
<th>Flexural strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sherbrooke footbridge</td>
<td>Sherbrooke, Canada</td>
<td>1997</td>
<td>200</td>
<td>40</td>
</tr>
<tr>
<td>Joppa clinker silo</td>
<td>Illinois, USA</td>
<td>2001</td>
<td>220</td>
<td>50</td>
</tr>
<tr>
<td>Seonyu footbridge</td>
<td>Seoul, Korea</td>
<td>2002</td>
<td>180</td>
<td>32</td>
</tr>
<tr>
<td>Sakata Mirai footbridge</td>
<td>Sakata, Japan</td>
<td>2002</td>
<td>238</td>
<td>40</td>
</tr>
<tr>
<td>Millau Viaduct toll gate</td>
<td>A75 Motorway, France</td>
<td>2004</td>
<td>165</td>
<td>30</td>
</tr>
<tr>
<td>Shepherds creek bridge</td>
<td>Sydney, Australia</td>
<td>2005</td>
<td>180</td>
<td>-</td>
</tr>
<tr>
<td>Blast resisting panels</td>
<td>Melbourne, Australia</td>
<td>2005</td>
<td>160</td>
<td>30</td>
</tr>
<tr>
<td>Papatoetoe footbridge</td>
<td>Auckland, Newzealand</td>
<td>2006</td>
<td>160</td>
<td>30</td>
</tr>
<tr>
<td>Glenmore/Legsby bridge</td>
<td>Calgary, Canada</td>
<td>2007</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Gaertnerplatz bridge</td>
<td>Kassel, Germany</td>
<td>2007</td>
<td>150</td>
<td>35</td>
</tr>
<tr>
<td>UHPC girder bridge</td>
<td>Iowa, USA</td>
<td>2008</td>
<td>150</td>
<td>-</td>
</tr>
<tr>
<td>Wind turbine foundations</td>
<td>Denmark</td>
<td>2008</td>
<td>210</td>
<td>24</td>
</tr>
<tr>
<td>Haneda Airport slabs</td>
<td>Tokyo, Japan</td>
<td>2010</td>
<td>210</td>
<td>45</td>
</tr>
<tr>
<td>Whiteman Creek bridge</td>
<td>Brantford, Canada</td>
<td>2011</td>
<td>140</td>
<td>30</td>
</tr>
<tr>
<td>Sewer pipes</td>
<td>Germany</td>
<td>2012</td>
<td>151</td>
<td>-</td>
</tr>
<tr>
<td>Spun concrete columns</td>
<td>Germany</td>
<td>2012</td>
<td>179</td>
<td>-</td>
</tr>
<tr>
<td>UHPC truss footbridge</td>
<td>Spain</td>
<td>2012</td>
<td>150</td>
<td>-</td>
</tr>
</tbody>
</table>

* Data taken from Fehling et al., 2004; 2008; 2012 and Toutlemonde and Resplendino, 2011
Table 2.3 – Typical composition of UHPC

<table>
<thead>
<tr>
<th>UHPC constituents</th>
<th>Range (% by weight)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>27 - 40</td>
</tr>
<tr>
<td>Silica fume</td>
<td>6 - 12</td>
</tr>
<tr>
<td>Quartz powder</td>
<td>7 - 14</td>
</tr>
<tr>
<td>Sand</td>
<td>35 - 45</td>
</tr>
<tr>
<td>Superplasticizer</td>
<td>0.5 - 3</td>
</tr>
<tr>
<td>Water</td>
<td>4 - 10</td>
</tr>
<tr>
<td>Steel fibre</td>
<td>0 - 8</td>
</tr>
</tbody>
</table>

* Data collected from Fehling et al., 2004; 2008; 2012; Toutlemonde and Resplendino, 2011

Table 2.4 – Effect of mixing time on fresh temperature of UHPC

<table>
<thead>
<tr>
<th>References</th>
<th>Mixing time</th>
<th>Fresh temperature</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ingo et al., 2004</td>
<td>8 minutes</td>
<td>20 to 23 °C</td>
</tr>
<tr>
<td>Kazemi and Lubell, 2012</td>
<td>20 minutes</td>
<td>26.5 to 28.5 °C</td>
</tr>
</tbody>
</table>

1 °C = 33.8 °F

Table 2.5 – Effect of w/b and superplasticizer on air content of UHPC

<table>
<thead>
<tr>
<th>References</th>
<th>w/b</th>
<th>Superplasticizer</th>
<th>Air content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ingo et al., 2004</td>
<td>0.25</td>
<td>-</td>
<td>4.3</td>
</tr>
<tr>
<td>Maeder et al., 2004</td>
<td>0.18</td>
<td>45 kg/m³</td>
<td>3.5</td>
</tr>
<tr>
<td>Kamen et al., 2009</td>
<td>0.13</td>
<td>46 kg/m³</td>
<td>1.8</td>
</tr>
<tr>
<td>Pierard and Cauberg, 2009</td>
<td>0.17</td>
<td>20 kg/m³</td>
<td>1.0</td>
</tr>
<tr>
<td>Pierard et al., 2012</td>
<td>0.11</td>
<td>15 kg/m³</td>
<td>2.5</td>
</tr>
<tr>
<td>Magureanu et al., 2012</td>
<td>0.13</td>
<td>52 kg/m³</td>
<td>4.6</td>
</tr>
</tbody>
</table>

1 kg/m³ = 0.0624 lb/ft³
Table 2.6 – Conversion factors for various type and size of UHPC specimens for compressive strength

<table>
<thead>
<tr>
<th>References</th>
<th>Specimen type and size</th>
<th>Conversion factor</th>
</tr>
</thead>
<tbody>
<tr>
<td>Skazlic et al., 2008</td>
<td>Cylinder, (70×140/100×200)</td>
<td>1.05-1.15</td>
</tr>
<tr>
<td></td>
<td>Cylinder, (150×300/100×200)</td>
<td>0.85-0.95</td>
</tr>
<tr>
<td>Graybeal and Davis, 2008</td>
<td>Cylinder 76/cube 100</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>Cylinder 76/cube 71</td>
<td>0.94</td>
</tr>
<tr>
<td></td>
<td>Cylinder 76/cube 51</td>
<td>0.96</td>
</tr>
<tr>
<td></td>
<td>Cylinder 76/cylinder 102</td>
<td>1.01</td>
</tr>
<tr>
<td>Kazemi and Lubell, 2012</td>
<td>Cube, (50/100)</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td>Cube 50/cylinder 100</td>
<td>1.14</td>
</tr>
<tr>
<td></td>
<td>Cube 50/cylinder 50</td>
<td>1.09</td>
</tr>
</tbody>
</table>

Table 2.7 – Effect of pressure application on UHPC compressive strength

<table>
<thead>
<tr>
<th>References</th>
<th>Heat treatment</th>
<th>Pressure application</th>
<th>Compressive strength</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roy et al., 1972</td>
<td>250 ºC</td>
<td>50 MPa</td>
<td>510 MPa</td>
</tr>
<tr>
<td>Richard and Cheyrezy, 1995</td>
<td>400 ºC</td>
<td>50 MPa</td>
<td>800 MPa</td>
</tr>
<tr>
<td>Roux et al., 1996</td>
<td>20 ºC</td>
<td>60 MPa</td>
<td>230 MPa</td>
</tr>
<tr>
<td>Shaheen and shrive, 2006</td>
<td>300 ºC</td>
<td>26 MPa</td>
<td>280 MPa</td>
</tr>
</tbody>
</table>

Table 2.8 – Relationship between elastic modulus and compressive strength of UHPC

<table>
<thead>
<tr>
<th>References</th>
<th>Models</th>
</tr>
</thead>
<tbody>
<tr>
<td>ACI 363R-92</td>
<td>$E = 3300 \cdot \sqrt{f'_c} + 6.9$</td>
</tr>
<tr>
<td>Ma and Schneider, 2002</td>
<td>$E = 16364 \cdot \ln(f'_c) - 34828$</td>
</tr>
<tr>
<td>Sritharan et al., 2003</td>
<td>$E = 4150 \cdot \sqrt{f'_c}$</td>
</tr>
<tr>
<td>Ma et al., 2004</td>
<td>$E = 19000 \cdot \frac{3}{10} \sqrt{f'_c}$</td>
</tr>
<tr>
<td>Graybeal, 2007</td>
<td>$E = 3840 \cdot \sqrt{f'_c}$</td>
</tr>
</tbody>
</table>
### Table 2.9 – Effect of casting method on flexural capacity of UHPC

<table>
<thead>
<tr>
<th>References</th>
<th>Steel fibre</th>
<th>Curing conditions</th>
<th>Casting direction/pouring method</th>
<th>Flexural strength (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Steil et al., 2004</td>
<td>6/- + 13/-</td>
<td>90 °C in water tank for 7 days</td>
<td>Horizontal</td>
<td>49.6</td>
</tr>
<tr>
<td></td>
<td>5%+1%</td>
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<td>Vertical</td>
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<td>Concrete pouring at one end only</td>
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<td>90 °C for 3 days and 20 °C wet curing thereafter</td>
<td>End casting of beams</td>
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### Table 2.10 – Effect of steel fibre dosage and beam size on flexural capacity of UHPC

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Table 2.11 – Bond strength of steel fibre and rebar in UHPC

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<td></td>
<td>0.85</td>
<td>10000000</td>
<td></td>
<td></td>
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<tr>
<td>Makita and Bruhwiler, 2013</td>
<td>13/0.16</td>
<td>3.0</td>
<td>-</td>
<td>Prism 150×40×750</td>
<td>0.80</td>
</tr>
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<td></td>
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<td></td>
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<td>0.85</td>
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Table 2.13 – Effect of w/b and curing regime on UHPC porosity

<table>
<thead>
<tr>
<th>References</th>
<th>UHPC type</th>
<th>w/b</th>
<th>Curing regimes</th>
<th>Total porosity (%)</th>
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</thead>
<tbody>
<tr>
<td>Heinz and Ludwig, 2004</td>
<td>Fibre cocktail</td>
<td>0.22 w/c</td>
<td>20 °C and 93% RH</td>
<td>8.2</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>65 °C and 93% RH</td>
<td>5.3</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 °C and 93% RH</td>
<td>4.2</td>
</tr>
<tr>
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<td></td>
<td></td>
<td>105 °C and 93% RH</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>120 °C and 93% RH</td>
<td>3.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>180 °C and 93% RH</td>
<td>2.9</td>
</tr>
<tr>
<td>Herold and Muller, 2004</td>
<td>With steel fibre</td>
<td>0.16</td>
<td>20 °C</td>
<td>10.5</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 °C for 2 days</td>
<td>6.4</td>
</tr>
<tr>
<td>Cwirzen, 2007</td>
<td>No steel fibres</td>
<td>0.17</td>
<td>Storage at 95% RH</td>
<td>5.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 °C for 4 days</td>
<td>1.1</td>
</tr>
<tr>
<td>Scheydt and Muller, 2012</td>
<td>With steel fibres</td>
<td>0.21</td>
<td>Water cured at 28 °C</td>
<td>8.9</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>90 °C for 3 days</td>
<td>5.4</td>
</tr>
<tr>
<td></td>
<td>No steel fibres</td>
<td></td>
<td>Water cured at 28 °C</td>
<td>10.9</td>
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</tbody>
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Table 2.14 – UHPC water sorptivity coefficient

<table>
<thead>
<tr>
<th>References</th>
<th>w/b</th>
<th>Curing conditions</th>
<th>Fibre</th>
<th>Sorptivity coefficient (kg/m²/h⁰.⁵)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roux et al., 1996</td>
<td>0.14</td>
<td>20 °C water cured</td>
<td>2% (13/0.175)</td>
<td>0.0100</td>
</tr>
<tr>
<td></td>
<td></td>
<td>20 °C water cured</td>
<td></td>
<td>0.0052</td>
</tr>
<tr>
<td>Franke et al., 2008</td>
<td>0.17</td>
<td>90 °C for 2 days</td>
<td>No fibre</td>
<td>0.0330</td>
</tr>
<tr>
<td></td>
<td>0.19</td>
<td></td>
<td></td>
<td>0.0440</td>
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</table>
### Table 2.15 – Maximum chloride ions penetration into UHPC

<table>
<thead>
<tr>
<th>References</th>
<th>Curing regimes</th>
<th>w/b</th>
<th>Exposure solution</th>
<th>Exposure period</th>
<th>Maximum chloride contents</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roux et al., 1996</td>
<td>20 °C water cured (60 Mpa pressure applied during setting time)</td>
<td>0.14</td>
<td>0.5 M NaCl</td>
<td>-</td>
<td>0.03% of concrete mass</td>
</tr>
<tr>
<td>Scheydt and Muller, 2012</td>
<td>water cured</td>
<td>0.21</td>
<td>3% NaCl</td>
<td>16 months</td>
<td>1.4 % by mass of binder</td>
</tr>
<tr>
<td>Thomas et al., 2012</td>
<td>20 °C for 2 days and 90 °C for another 2 days</td>
<td>0.12</td>
<td>Marine environment at Treat Island</td>
<td>5 years</td>
<td>0.21 % of concrete mass</td>
</tr>
</tbody>
</table>

### Table 2.16 – Chloride ions penetration depth into UHPC specimens for various salt exposures

<table>
<thead>
<tr>
<th>References</th>
<th>Curing regimes</th>
<th>w/b</th>
<th>Exposure solution</th>
<th>Exposure period</th>
<th>Chloride penetration depth</th>
</tr>
</thead>
<tbody>
<tr>
<td>Graybeal, 2006</td>
<td>Laboratory environment</td>
<td>0.12</td>
<td>3% NaCl</td>
<td>3 months</td>
<td>4 to 6 mm</td>
</tr>
<tr>
<td>Scheydt et al., 2008</td>
<td>90 °C for 3 days</td>
<td>0.21</td>
<td>3% NaCl</td>
<td>4 months</td>
<td>2 to 3 mm</td>
</tr>
<tr>
<td>Pierard and Cauberg, 2009</td>
<td>20 °C and 95% RH</td>
<td>0.18</td>
<td>16% NaCl</td>
<td>2 months</td>
<td>3 to 4 mm</td>
</tr>
<tr>
<td>Thomas et al., 2012</td>
<td>90 °C for 2 days</td>
<td>0.12</td>
<td>Marine environment at Treat Island</td>
<td>5 years</td>
<td>6 to 10 mm</td>
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<tr>
<td>Scheydt and Muller, 2012</td>
<td>90 °C for 3 days</td>
<td>0.21</td>
<td>3% NaCl</td>
<td>16 months</td>
<td>4 to 6 mm</td>
</tr>
<tr>
<td>Pierard et al., 2012</td>
<td>20 °C for 90 days</td>
<td>0.21</td>
<td>16% NaCl</td>
<td>3 months</td>
<td>2 to 3 mm</td>
</tr>
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</table>
Table 2.17 – UHPC chloride ions diffusion coefficient for various curing exposures and w/b

<table>
<thead>
<tr>
<th>References</th>
<th>Curing regimes</th>
<th>w/b</th>
<th>Exposure solution</th>
<th>Exposure period</th>
<th>Diffusion coefficient (×10⁻¹³)</th>
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<tbody>
<tr>
<td>Roux et al., 1996</td>
<td>20 °C water cured and pressure application</td>
<td>0.14</td>
<td>0.5 M NaCl</td>
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<td>0.20</td>
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<tr>
<td>Pierard and Cauberg, 2009</td>
<td>20 °C and 95% RH</td>
<td>0.18</td>
<td>16% NaCl</td>
<td>56 days</td>
<td>4.00</td>
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<tr>
<td>Juanhong et al., 2009</td>
<td>90 °C steam for 1 day</td>
<td>0.15</td>
<td>10% NaCl</td>
<td>-</td>
<td>4.00</td>
</tr>
<tr>
<td>Scheydt and Muller, 2012</td>
<td>90 °C for 3 days</td>
<td>0.21</td>
<td>16% NaCl</td>
<td>63 days</td>
<td>1.30</td>
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<tr>
<td>Pierard et al., 2012</td>
<td>20 °C for 90 days</td>
<td>0.21</td>
<td>16% NaCl</td>
<td>90 days</td>
<td>2.30</td>
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<tr>
<td>Thomas et al., 2012</td>
<td>90 °C for 2 days</td>
<td>0.12</td>
<td>Marine environment at Treat Island</td>
<td>5 years</td>
<td>1.30</td>
</tr>
<tr>
<td>Tao et al., 2012</td>
<td>90 °C and 90% RH</td>
<td>0.19</td>
<td>3% NaCl</td>
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<td>2.68</td>
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Table 2.18 – Number of coulombs passed through UHPC specimens

<table>
<thead>
<tr>
<th>References</th>
<th>Curing regimes</th>
<th>w/b or w/c*</th>
<th>Coulombs</th>
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<tbody>
<tr>
<td>Bonneau et al., 1997</td>
<td>-</td>
<td>-</td>
<td>10</td>
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<tr>
<td>Graybeal, 2006</td>
<td>90 °C for 2 days Laboratory environment</td>
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<td></td>
<td></td>
<td>360</td>
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<tr>
<td>Ahlborn et al., 2008</td>
<td>Air cured 90 °C and 100% RH for 2 days</td>
<td>0.20*</td>
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<td></td>
<td></td>
<td></td>
<td>15</td>
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<td>Chang et al., 2009</td>
<td>Water curing of 25 °C 85 °C and 95% RH</td>
<td>0.17*</td>
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<td></td>
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<td>307</td>
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<tr>
<td>Tao et al., 2012</td>
<td>90 °C and 90% RH</td>
<td>0.19</td>
<td></td>
</tr>
<tr>
<td>Scheydt and Muller, 2012</td>
<td>90 °C for 3 days</td>
<td>0.21</td>
<td></td>
</tr>
<tr>
<td>References</td>
<td>w/b or w/c*</td>
<td>Steel fibres</td>
<td>curing regimes</td>
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<td>--------------</td>
<td>----------------------------------------</td>
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<td>Shaheen and shrive, 2006</td>
<td>0.13*</td>
<td>0%</td>
<td>150 °C</td>
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<td></td>
<td></td>
<td>1.50%</td>
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<tr>
<td>Ahlborn et al., 2008</td>
<td>0.20*</td>
<td>6%</td>
<td>Air</td>
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<td>90 °C and 100% RH for 2 days</td>
</tr>
<tr>
<td>Juanhong et al., 2009</td>
<td>0.21*</td>
<td>2%</td>
<td>90 °C steam for 1 day</td>
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<tr>
<td>Magureanu et al., 2012</td>
<td>0.12</td>
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<td>0%</td>
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<td>90 °C and 80 to 90% RH for 5 days</td>
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### Table 2.20 – UHPC under elevated temperature

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<tr>
<th>References</th>
<th>w/b</th>
<th>Polypropylene fibre (%)</th>
<th>Temperature (°C)</th>
<th>Relative compressive strength (%)</th>
<th>Relative modulus of elasticity (%)</th>
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<tr>
<td>Pimienta et al., 2012</td>
<td>0.19</td>
<td>3.0</td>
<td>300</td>
<td>92</td>
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<td>500</td>
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<td>600</td>
<td>50</td>
<td>20</td>
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<tr>
<td>Way and Wille, 2012</td>
<td>0.14</td>
<td>4.2</td>
<td>300</td>
<td>78</td>
<td>79</td>
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<td></td>
<td></td>
<td></td>
<td>500</td>
<td>78</td>
<td>62</td>
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<td></td>
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<td>600</td>
<td>67</td>
<td>52</td>
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<td>Hosser et al., 2012</td>
<td>0.31</td>
<td>6.3</td>
<td>300</td>
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<td>500</td>
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<td>600</td>
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### Table 2.21 – UHPC cost estimation

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<th>Regions</th>
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<tr>
<td></td>
<td>$ / m³</td>
<td>$ / yd³</td>
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<tr>
<td>Bonneau et al., 1996</td>
<td>1400</td>
<td>1070</td>
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<td>Blais and Couture, 1999</td>
<td>750</td>
<td>520</td>
</tr>
<tr>
<td>Aitcin, 2000</td>
<td>1000</td>
<td>760</td>
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<td>Voort et al., 2008</td>
<td>2620</td>
<td>2000</td>
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Table 2.22 – Various studies conducted on tunnel lining segments

<table>
<thead>
<tr>
<th>References</th>
<th>Purpose of test</th>
<th>Specimen details</th>
<th>Reinforcement type</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Nakamura et al. (1998)</td>
<td>Effect of lining thickness and width on load carrying capacity</td>
<td>Lining segment L = 3450 mm W = 1220 mm T = 250 mm</td>
<td>Conventional steel rebar</td>
<td>Segments subjected to two points loading.</td>
</tr>
<tr>
<td>Mashimo et al. (2002)</td>
<td>Effect of various load conditions on SFRC lining segments</td>
<td>Semi-circular full scale tunnel lining</td>
<td>Steel fibre</td>
<td>SFRC linings exhibited higher peak load carrying capacity compared to that of unreinforced concrete.</td>
</tr>
<tr>
<td>Nishikawa (2003)</td>
<td>Flexural performance of pre-stressed lining segment joint</td>
<td>Lining segment L = 1500 mm W = 1000 mm T = 150 mm</td>
<td>Pre-stressed reinforcement</td>
<td>Pre-stressed reinforcement can be useful in lining segments for large diameter tunnels. Bolt joints can be eliminated using pre-stressing at joints.</td>
</tr>
<tr>
<td>He and Wu (2005)</td>
<td>Structural performance of tunnel lining</td>
<td>Small scale tunnel lining</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Lu et al. (2006)</td>
<td>Measurement of earth pressure and associated deformation on complete ring</td>
<td>Complete ring consisted of 10 wedge type lining segments</td>
<td>Conventional steel rebar</td>
<td>-</td>
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<tr>
<td>Yan and Zhu (2007)</td>
<td>Effect of fire on mechanical properties of lining segments</td>
<td>-</td>
<td>RC, SFRC, PFRC</td>
<td>-</td>
</tr>
<tr>
<td>Poh et al. (2009)</td>
<td>Flexural behaviour of plain concrete and SFRC lining segments</td>
<td>Lining segment L = 1400 mm W = 2359 mm T = 350 mm</td>
<td>Conventional concrete and SFRC</td>
<td>SFRC segment exhibited higher peak and cracking loading than plain concrete.</td>
</tr>
<tr>
<td>Ahn (2011)</td>
<td>Effect of thin spray on performance of lining segment</td>
<td>Lining segment L = 1800 mm W = 610 mm T = 150 mm</td>
<td>Conventional steel rebar</td>
<td>Thin spray effectively reduced crack propagation and concrete spalling besides improving the peak load carrying capacity of lining segments.</td>
</tr>
<tr>
<td>Caratelli et al. (2011)</td>
<td>Bending resistance of RC and SFRC segments under static loading.</td>
<td>Lining segment L = 3640 mm W = 1500 mm T = 200 mm</td>
<td>Conventional steel rebar in RC segment and SFRC in SFRC segments</td>
<td>SFRC segment can be installed where cracking is the governing criteria.</td>
</tr>
<tr>
<td>Zhiqiang and Monsoor (2013)</td>
<td>Corrosion performance</td>
<td>Beam specimen L = 1290 mm W = 290 mm H = 360 mm</td>
<td>Convention steel rebar</td>
<td>Bond strength, cracking and mechanical performance should be considered for the corroded lining segments.</td>
</tr>
</tbody>
</table>

H = height; L = length; T = thickness; W = width
FLEXURAL AND THRUST LOAD RESISTANCE OF FULL-SCALE RC AND SFRC PRECAST TUNNEL LINING SEGMENTS*

3.1. INTRODUCTION

In this chapter, the structural behaviour of full-scale precast concrete tunnel lining (PCTL) segments from a subway extension tunnel project in Canada was investigated. This extension is 8.6 km (5.34 mile) long tunnel and consists of six segments forming a 5.7 m (18.70 ft) diameter ring. Flexural monotonic and cyclic load tests were performed on full-scale conventional reinforced concrete (RC) and steel fibre-reinforced concrete (SFRC) PCTL segments to evaluate their bending resistance. Moreover, a thrust load test was conducted to simulate the thrust loading action during tunnel boring machine operation.

The tested full-scale RC and SFRC lining segments were fabricated at an industrial precast plant in Canada. Concrete was poured into the segment molds using an overhead crane (Figure B.1). During concrete placement, the segment molds were continuously vibrated using a vibration system installed on each segment mold. This ensured adequate compaction of the concrete without segregation or honey combing. After pouring concrete, the upper plates of molds were lowered while keeping about 100 mm (4 in) clear space at the fresh concrete surface. The entire mold was then covered with a plastic tarp that fits tight to the floor in order to eliminate steam and moisture loss (Figure B.2). Steam (at 45 ± 3 °C (113 ± 6 °F)) was applied (for a maximum of 5 hours) to the bottom side of the molds. Steam circulated around the mold under the plastic tarp cover. This provided sufficient moisture during the curing period. After steam curing, the upper plates of the mold were opened (Figure B.3). Any small cavities on the entire extrados faces were then uniformly filled with cement slurry (Figure B.4). The cement slurry had the same composition as the original

* A version of this chapter was accepted in the ACI Materials Journal (2013). Part of this chapter was published in the Tunneling Association of Canada Conference, Montreal, Canada (2012).
CHAPTER 3

concrete mixture without aggregates and steel fibres. Segments were then de-molded and turned 180° to have their intrados faces pointing upward. Afterwards, accessories including gasket rubber, guide rods and packer boards were installed on the segment and a thin coating of cement slurry was uniformly applied to the entire faces to avoid uneven surfaces or cavities. The lining segments were further cured inside moist curing chambers (relative humidity of more than 95%) for about 5 days. Thereafter, the segments were stacked outdoors in the storage yard (Figure B.5).

3.2. RESEARCH SIGNIFICANCE

An attempt can be made to eliminate corrosion of the conventional steel rebar reinforcement by utilizing steel fibre reinforcement. Besides improving durability, the substitution of fibre reinforcement for conventional steel reinforcement cages in tunnel linings eliminates the laborious and costly manufacturing of curved shape cages, which require complicated welding and detailing. Limited studies were examined the structural behaviour of full-scale lining segments. Therefore, full-scale precast RC and SFRC field segments are investigated herein to overcome the gaps in prototype testing. Moreover, since very limited research has studied the cyclic behaviour of PCTL segments, this study should contribute knowledge on the seismic behaviour of PCTL segments.

3.3. EXPERIMENTAL PROCEDURE

3.3.1. Segment Description and Mixture Design

The length and width of RC and SFRC segments are 3180 mm (125.20 in) and 1500 mm (59.05 in), respectively, while the thickness is 235 mm (9.25 in). Segments are skewed at their ends rather than straight edges. Figure 3.1 shows the geometrical and reinforcement details of the RC segments.

The concrete mixture compositions for RC and SFRC PCTL segments are listed in Table 3.1. Cold drawn hooked end steel fibres (60 mm (2.36 in) length and 0.75 mm (0.030 in) diameter) having ultimate tensile strength greater than 1050 MPa (152.30 ksi) were added (1.5% by mixture volume). Moreover, micro-polypropylene (PP) fibers (18 µm nominal diameter and 6 mm (0.24 in) in length) were added at a dosage of 1.0 kg/m³ (0.0624 lb/ft³) in both the RC and SFRC segments. However, the focus of this study was on the difference
between the conventional steel rebar and the steel fiber reinforcement. The fresh and hardened properties of concrete used in fabricating full-scale RC and SFRC segments are shown in Table 3.2.

Initially, four 150×150×500 mm (6×6×20 in) beams were tested in order to determine the flexural beam bending performance of steel fibre-reinforced concrete using ASTM C1609 (Standard Test Method for Flexural Performance of Fibre-Reinforced Concrete (Using Beam with Third-Point Loading)). The average beam bending test results of SFRC beams are shown in Table 3.3.

3.3.2. Core Sampling

In order to assess the quality of the cast concrete (compressive and tensile strengths), cylindrical cores were taken from both the RC and SFRC PCTL segments (Figure C.1). The coring process was conducted at site according to ACI 214.4R (Guide for Obtaining Cores and Interpreting Compressive Strength Results). All the core drilling was performed using diamond drill rig equipment firmly placed at the segments intrados faces through a vacuum suction base. The diamond drill bit attached to the core rig was continuously lubricated with water for lowering the diamond bit temperature. The locations for drilling cores on the segments were selected randomly to capture any variation along the segment’s cross-section. However, in the case of RC segments, the locations of rebar were mapped using a rebar locator in order to avoid the extraction of reinforcing steel from the retrieved cores. Cores with 100 mm (3.94 in) diameter were cut through the full depth of 235 mm (9.25 in) of the segments. After drilling and sufficient drying, cores were sealed in plastic bags and placed in a wooden case. During transportation to the laboratory, core specimens were handled with special care in order to avoid micro-cracking and end chipping. The length of cores was adjusted to 200 mm (7.87 in) using a laboratory concrete saw in order to satisfy the recommendations of ASTM C42 (Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams of Concrete).

3.3.3. Flexural Testing

Two 1800 mm (70.86 in) long stiffened I-beams (W10x49) welded together and fixed with bolts to the ground frame were used as a reaction frame for the flexural tests. Each PCTL
segment tested was simply supported on the reaction frame with a span of 3000 mm (118.11 in). Figures 3.2 and B.6 show the experimental set up for flexural testing. A waffle tree loading frame (Figure 3.2 (b)) was used to apply a uniformly distributed load in agreement with previous studies (Nishikawa, 2003; Moccihino et al., 2010; Caratelli et al., 2011). A rubber bearing pad was placed underneath the loading frame to ensure smooth contact surface. An incremental load (2 kN/min (0.45 kip/min)) was applied on the segments using a load cell actuator with an applied load capability of 1500 kN (337 kip). Three linear variable displacement transducers (LVDTs) were positioned at the segment mid-span in order to measure vertical displacements. All load and displacement readings were collected simultaneously using a high speed data acquisition system. The crack widths at various loading steps were measured using a crack width ruler.

Quasi-static compressive loading was applied in terms of percentage of maximum displacement ($\Delta_{\text{max}}$) achieved during the monotonic tests at the failure point. Two cycles of loading and unloading were conducted for 1.25%, 2.5%, 5%, and 10% of $\Delta_{\text{max}}$. Subsequently, three cycles were applied for 20%, 40%, 60%, 80%, 100% of $\Delta_{\text{max}}$. The loading cycles were continuously applied until the tested segment failed into two pieces (Figure B.7).

**3.3.4. Thrust Load Testing**

Figure 3.3 illustrates the laboratory setup for the thrust load test. The main aim of thrust load testing is to evaluate the tensile splitting stresses and thrust induced by Tunnel Boring Machines (TBM) during the construction phase (Burgers et al., 2007). The segment was directly placed on the ground and an incremental load (10 kN/min (2.25 kip/min)) was applied to its edge on the long dimension. Two LVDTs were fastened to the top concrete surface and one LVDT was placed on the loading steel plate for measuring the vertical displacement of the segment. Another two LVDTs were attached at the segment intrados and extrados faces at the mid span in order to capture splitting in the horizontal direction (Figure 3.3).

**3.4. RESULTS AND DISCUSSION**

The compressive and splitting tensile strengths of concrete mixtures used in casting the RC and SFRC PCTL segments were each evaluated based on the average of five core samples.
The cracking load, ultimate capacities, cracking deflection, ultimate deflection, stiffness change and crack widths were the main studied parameters for the tested RC and SFRC segments. Moreover, the cyclic hysteretic envelopes, energy dissipation, displacement ductility and stiffness degradation were also evaluated for both the RC and SFRC PCTL segments.

3.4.1. Compressive and Tensile Strengths

The average core compressive strengths for concrete mixtures of RC and SFRC PCTL segments were 60.0 MPa (8700 psi) and 61.4 MPa (8900 psi), respectively. The addition of steel fibres did not influence the compressive strength significantly, in agreement with previous research (Ayan et al., 2011). The initial splitting tensile strength for RC and SFRC was 7.5 MPa (1008 psi) and 9.0 MPa (1030 psi), respectively. This indicates that the addition of 1.5% of steel fibres enhanced the splitting tensile strength by 20% with respect to that of the concrete without steel fibres. This can be attributed due to the crack bridging and arresting property of steel fibres (King and Alder, 2001).

3.4.2. Flexural Monotonic Behaviour

3.4.2.1. Load-Displacement Curve

Figure 3.4 shows the typical monotonic flexural load-mid span displacement response for RC and SFRC PCTL segments. The mid-span displacement was monitored by three LVDTs. All three LVDTs showed nearly identical displacement readings, indicating the absence of torsion effects (Moccihino et al., 2010). The self-weight of the loading frame waffle tree was added to the measured load carrying capacity of the segments in consistent with previous work (Caratelli et al., 2011).

The load-mid span displacement response of the RC segment can be divided into three parts (Figure 3.4). The first part, a linear behaviour was observed up to the crack initiation (Point A in Figure 3.4) at a load of 45 kN (10.12 kip). Subsequently, the slope of the curve became less steep due to the development of cracks and reached yielding (Point B in Figure 3.4) at a load of 210 kN (47.20 kip). Finally, further increase in load continued up to the peak load (Point C in Figure 3.4) at 244 kN (54.85 kip). Thereafter, a brittle failure occurred along with an abrupt reduction in the load carrying capacity, without exhibiting any significant post
ductility or a descending branch behaviour. The failure of the RC segment was characterized by splitting off a concrete cover followed by rupture of steel rebar (Figure 3.5 (a)).

The load-mid span displacement curve of the SFRC segment can be divided into ascending, yielding and descending branches (Figure 3.4). First, a linear ascending portion was observed up to crack initiation (Point a in Figure 3.4) at a load of 71 kN (15.96 kip), which is higher than that of the RC segment. At this point, more cracks formed, leading to higher stresses in the segment cross-section. The load was then transferred to steel fibres at the crack location until yielding of the fibres (Point b in Figure 3.4) at a load of 113 kN (25.40 kip). The stresses were further increased in the segment and reached the peak load (Point c in Figure 3.4) at 119 kN (26.75 kip). The descending branch began thereafter and its slope changed as fibres were pulled-out or fractured until complete failure of the segment. This descending branch exhibits a somewhat ductile nature of the post-peak behaviour of the SFRC segment. Examining the failure surface indicates that fibre de-bonding and pull-out was the dominant behaviour rather than fibre fracture (Figures 3.5 (b) and 6). Moreover, yielding of fibres before pull-out from concrete had increased the capability to deform under load, which increases energy absorption capacity (Hameed et al., 2009).

Table 3.4 summarizes the flexural test results for both RC and SFRC PCTL segments. The first crack load is the load where initial linear elastic slope of the load-displacement plot ends while, peak/ultimate load describe the maximum load of the load-mid span displacement curve. The load corresponding to first crack initiation for the SFRC segment was about 58% higher than that of the RC segment. Moreover, the SFRC segment exhibited around 11% lower displacement than that of the RC at the cracking load as a result of anchorage and crack arrestment mechanisms provided by the steel fibres (Moccihino et al., 2010). In the case of SFRC segment, initial cracks less than 0.10 mm (0.004 in) average crack width were observed when the slope of the initial SFRC load-mid displacement plot changes. However, for RC segment, crack width increased more rapidly and 0.20 mm (0.008 in) average crack width was measured at the end of linear slope of RC load-mid span displacement plot.
3.4.2.2. Cracking Pattern

Generally, RC and SFRC segments failed in a flexural mode and no shear cracks were observed in either type of segments. Figure 3.7 illustrates the monitored cracking pattern in RC and SFRC segments. Generally, fewer cracks were observed in the SFRC segment compared to the RC segment. For the RC segment, at a load value of 48 kN (10.79 kip) (Point A in Figure 3.7 (a)), one straight discontinuous crack at mid-span was observed at the segment intrados face. Few small cracks of around 100 mm (3.93 in) length were also observed around the first crack. These small cracks propagated further in addition to new parallel cracks that developed at a load of 125 kN (28.10 kip) (Point B in Figure 3.7 (a)). The crack spacing was around 180 mm (7.08 in). These cracks were mapping the location of reinforcing rebars. At a load of 210 kN (47.20 kip) (Point C in Figure 3.7 (a)), a wide spread of continuous and discontinuous straight cracks were formed. A significant increase in crack width was also observed close to the failure point (Point D in Figure 3.7 (a)).

On the other hand, a single discontinuous small crack occurred on the intrados face of SFRC segment at a load of 71 kN (15.96 kip) (Point a in Figure 3.7 (b)). This crack propagated further along with formation of new hairline cracks at a load of around 107 kN (24.05 kip) (Point b in Figure 3.7 (b)). At the ultimate capacity (Point c in Figure 3.7 (b)), cracks parallel to the previously developed ones increased in number. Moreover, widespread small discontinuous cracks were observed before complete failure of the segment (Point d in Figure 3.7 (b)).

The crack width in the SFRC segment was found to be less than 0.25 mm (0.0098 in) at ultimate load, which satisfies the accepted limit of 0.30 mm (0.012 in) crack width for serviceability conditions (ACI 224, 2001; Caratelli et al., 2011). Since cracking occurred at a higher load for SFRC compared to RC (71 kN (15.96 kip) versus 45 kN (10.11 kip)), its behaviour can be perceived as an advantage from a serviceability perspective. For instance, the penetration of aggressive species to concrete can be reduced in SFRC, leading to an enhanced SFRC PCTL segments durability. Figure 3.8 shows the cracked intrados faces of both the tested RC and SFRC segments.

Although the peak load for the RC segment and its energy absorption are higher than that of the SFRC segment, some concerns need to be considered. First, during the fabrication
of PCTL segments, delivery to the site and installation process using TBM, accidental thrust and impact loads may result in segment cracking, consequently jeopardizing its structural integrity. Therefore, the initial cracking load of the PCTL segment is considered as the ultimate design criteria (Moccihino et al., 2010). The structural design of PCTL segments also needs to satisfy the serviceability state of crack control. Based on these two criteria, it can be argued that SFRC has a high potential for application in tunnel linings. Moreover, with increased fibre dosage in SFRC segments, the difference in the peak loads between conventional RC and SFRC segments can be decreased (Rivaz, 2007).

3.4.3. Flexural Cyclic Behaviour

3.4.3.1. Hysteresis Curve

The elasto-plastic behaviour of RC and SFRC PCTL segments under cyclic load is illustrated through the load-mid span displacement hysteresis curves (Figure 3.9). Initially, a linear relationship between load and displacement was found before concrete cracking. At this stage, there was no evidence of stiffness degradation and residual deformations were very small, indicating that the segments were in the elastic range. As cracking initiated (i.e. at 48 kN (10.79 kip) and 75 kN (16.86 kip) for RC and SFRC segments, respectively), the slope of the hysteresis curves changed as the load and displacement increased. A parabolic unloading response was noticed after the linear elastic portion of unloading. This parabolic unloading pattern was primarily due to the permanent deformations (percentage of the maximum deformation evidenced in each cycle) that occurred in RC and SFRC segments. This agrees with other study (Kesner et al., 2003). It was observed that the deformations in both segment types were larger during the initial cycling and then decreased to a constant growth at intermediate cycling. These deformations increased again more rapidly nearing failure point. This is in agreement with previous study (Karsan and Jirsa, 1969). For each loading amplitude and for both the RC and SFRC segments, little reduction in maximum load was observed in the subsequent cycles compared to that of the first cycle. Similar observation was reported by Xue et al. (2008) in their cyclic experiments on fibre-reinforced concrete beams.

The slope of the hysteresis of RC segment changed rather abruptly compared to that of SFRC segments due to more internal material damage in RC segments. The spalling of the concrete cover was observed in RC segments subjected to cyclic loads (Figure B.8). On the


other hand, SFRC segments did not exhibit severe spalling compared to that of the RC segments. This can be attributed to better holding of the concrete matrix through the bridging action of steel fibres (Figure B.9) until pull-out or fracture of fibres. This reduction of concrete spalling in SFRC segments under cyclic loads could lead to lower rehabilitation costs after earthquake events (Hameed et al., 2009). Moreover, the bridging action of steel fibres tends to reduce the crack width in SFRC PCTL segments compared to that of cracks in RC segments. Similar finding was reported by other study (Moccihino et al., 2010). The RC segment showed a sudden drop in its load carrying capacity after the peak/ultimate loading cycle and the segment failed into two pieces.

Prior to cracking of the SFRC segment, its compressive stresses are in the elastic range, leading to elastic unloading behaviour. After cracking, the loading and unloading cycles were no longer linear. At that point stresses were transferred to the steel fibres and the crack opening initiated. These cracks further opened along with the formation of new cracks as the tensile strain increased beyond the previous loading cycle. After reaching the peak load, a softening cyclic response is initiated. As damage progressed in the segment, the load carrying capacity dropped during each additional cyclic amplitude. This damage is a function of the interaction between the matrix and the steel fibres (Kesner et al., 2003). The incorporation of steel fibres in SFRC segments reduces the strain magnitude thus restricting the propagation of micro and macro-cracks and leading to lower internal damage compared to that of the RC segment (Daniel and Loukili, 2002; Holschemacher and Muller, 2007). Therefore, as fibre pull-out and de-bonding increased, the stresses carried by steel fibres increased with higher number of loading cycles until failure of the segment (Li and Leung, 1992).

3.4.3.2. Skeleton Curve and Energy Dissipation

Envelopes of hysteresis curves (i.e. skeleton curves) for RC and SFRC segments are shown in Figure 3.10. These curves summarize the maximum average value of load and corresponding displacement for the number of cycles applied at each amplitude. Initial elastic, yielding and ultimate phases of RC and SFRC segments were clearly captured by the skeleton curves (Figure 3.10). Initial cracking of segments led to decrease the slope of the skeleton curve after the elastic phase and up to the yielding point. For the RC segment, at the yield point, loads were carried by steel rebars. After yielding, the slope of the skeleton curve
decreased due to the internal damage of concrete. At ultimate load, the slope became constant and thereafter a sudden decrease in load carrying capacity was observed with brittle failure of the RC segment. On the other hand, the yield and ultimate points for the SFRC segment occurred at a comparable load level followed by a descending branch of the skeleton curve. This post ductility behaviour after the ultimate phase can be ascribed to the contribution of steel fibres in bridging and restricting crack opening.

The energy dissipation capacity of a structural member is related to its ability to resist cyclic inelastic stresses (Sinha and Naraine, 1991). The total area enclosed within the skeleton of hysteresis curves describes the energy dissipation capacity (Figure 3.10). Areas within the RC and SFRC skeleton curves were calculated using the trapezoidal rule (Xue et al., 2008). It was observed that the RC segment exhibited 3 times higher value of energy dissipation compared to that of the SFRC segment. This is due to its substantially higher steel cage reinforcement effect compared to that of the dispersed steel fibres. However, if the design criteria is the initial cracking (as explained previously), the SFRC segment showed 1.5 times higher energy dissipation value than that of the RC segment. This can be ascribed to the higher cracking load for SFRC, which may be beneficial for tunnel lining applications from a serviceability perspective. Initial cracking in PCTL is important as it is most likely to develop during the handling and installation processes of PCTL segments. Therefore, high initial cracking energy dissipation capacity of SFRC segments will be advantageous for the installation processes of segments through TBM.

3.4.3.3. Displacement Ductility

Displacement ductility is an important factor in assessing the behaviour of structures under seismic load. The ratio of ultimate displacement ($\Delta_u$) to the yield displacement ($\Delta_y$) defines the displacement ductility coefficient (Xue et al., 2008). The RC segment exhibited approximately 2.95 times higher displacement ductility coefficient than the SFRC segment. The reason this is that the yielding plateau of the RC segment was much higher, while the SFRC ultimate capacity and yield point were at comparable level. The higher displacement ductility coefficient of RC segments indicates a more ductile behaviour than that of the SFRC segment due to heavily reinforced conventional steel cage. However, both segments (RC and SFRC) meet the seismic design criteria (described later on).
3.4.3.4. Stiffness Degradation

Figure 3.11 shows the stiffness degradation curve for both the RC and SFRC segments. At smaller displacement increments, the stiffness of the RC segment degraded more rapidly due to the early crack opening, while the SFRC segment exhibited a steadier drop in stiffness. This reduction in stiffness can be attributed to the internal material disturbances (Spadea and Bencardino, 1997). The improved stiffness degradation behaviour of the SFRC segment can be ascribed to the contribution of steel fibres to arresting cracks during the loading and unloading cycles (Guvensoy et al., 2004). Furthermore, the stiffness of the SFRC segment decreased more as the fibres were pulled-out from the concrete matrix. The stiffness for both the RC and SFRC PCTL segments became constant near the failure point (Figure 3.11).

3.4.4. Thrust Load Test

The thrust load test was conducted on both the RC and SFRC segments to simulate field TBM loading conditions and possible concrete splitting in the horizontal direction. Figure 3.12 shows the load-displacement curves of the thrust load test for the RC and SFRC segments. It can be observed that the RC and SFRC segments exhibited comparable behaviour. Moreover, no appreciable cracking pattern was found, indicating the ability of the RC and SFRC segments to carry typical thrust loads induced by TBM. These results are in agreement with previous findings in the literature (Moccihino et al., 2010; Caratelli et al., 2011).

3.5. DESIGN CONSIDERATIONS

The basic design criteria for PCTL segments are to bear the ground forces induced by imposed and construction loads. These forces differ from site to site as they are dependent on the soil characteristics. Moreover, the thrust loads induced by TBM require special attention, otherwise it can result in severe segmental damage and damaged segments may need to be replaced. Other induced loads that segments should be able to resist include those produced during handling, storage, lifting and erecting processes (Hung et al., 2009). Therefore, Einstein and Schwartz (1979) proposed the following equations (Eqs. 3.1 and 3.2) to evaluate the design thrust load \( T_s \) and bending moment \( M_s \) for static loads based on relative stiffness:
Where, $\sigma_v$ and $\sigma_h$ are the normal stresses; $R_l$ is the radius of tunnel and $C_1$, $C_2$ and $C_3$ are constants depending on the loading, surrounding soil properties and tunnel lining characteristics.

Under earthquake events, tunnel structures behave better than other structures as the surrounding ground restrains the lining system and does not allow excitations independent of ground like other structures (i.e. bridges and buildings) (Hung et al., 2009). The main criterion for seismic design of tunnel lining is its ability to resist the “ovaling” effect of the tunnel cross-section induced by ground deformations during an earthquake (Park et al., 2006). Therefore, Wang (1993) proposed the following equations (Eqs. 3.3 and 3.4) to evaluate the maximum design thrust ($T_e$) and bending moment ($M_e$) for circular tunnels under earthquake loading:

$$T_e = \frac{K_2 E_s R_l \gamma_{max}}{2(1+\nu_s)} \quad \text{Eq. 3.3}$$

$$M_e = \frac{K_1 E_s R_l^2 \gamma_{max}}{6(1+\nu_s)} \quad \text{Eq. 3.4}$$

Where, $K_1$ and $K_2$ are the response coefficient; $\gamma_{max}$ is the free field shear strain; $E_s$ and $\nu_s$ are the modulus of elasticity and Poisson’s ratio of the surrounding soil, respectively.

Using information provided for a subway tunnel in Canada (Table A. 1), the design thrust and bending moment values under static and cyclic loads were calculated. Comparing the design and experimental values showed that both the RC and SFRC PCTL segments meet the design loads (Table 3.5). Therefore, in addition to its relatively higher cracking load and low reinforcement corrosion potential, SFRC showed an adequate structural performance, thus making it a viable alternative to conventional RC tunnel linings.
3.6. SUMMARY

This study explored the structural performance of RC and SFRC PCTL segments for a subway tunnel in Canada. The key finding of this research is the high potential of using SFRC tunnel lining segments in applications where the primary concern is serviceability cracking limitations. The cracking load for the RC and SFRC segments was 45 kN (10.11 kip) and 71 kN (15.96 kip), respectively. Hence, SFRC segments should behave better during fabrication in precast plants, handling, delivery and installation processes at the site using TBM due to its higher cracking load. Moreover, the SFRC segments exhibited lower crack width compared to that of the RC segments and satisfied serviceability limitations. The flexural monotonic and cyclic test results showed that the RC segment failed abruptly after achieving its ultimate load without exhibiting significant residual strength. On the other hand, the SFRC segment, though it had significantly lower peak load, showed an enhanced post-peak behaviour, exhibiting a steady drop in load carrying capacity due to the crack bridging action of steel fibres. No appreciable spalling of concrete was found in the SFRC segments compared to that of the RC segments.

It was observed that RC segments exhibited higher energy dissipation capacities and displacement ductility compared to that of the SFRC. Furthermore, analysis of experimental results indicates that although the ultimate load carrying capacity of the conventional steel rebars RC PCTL segments was much higher than that of the SFRC segments, the SFRC segments also met the design criteria for monotonic and cyclic loads and could perform better under normal serviceability loads.

An optimization of the SFRC mix design needs to be further performed in order to attain more desirable mechanical properties for PCTL segments. An effort has been made herein to gain confidence in the structural performance of SFRC for PCTL segmental application. Furthermore, the structural behaviour of conventional RC segments with secondary steel fibre reinforcement needs to be evaluated in future investigation.
3.7. REFERENCES


### Table 3.1 – Concrete mixture composition for RC and SFRC segments

<table>
<thead>
<tr>
<th>Materials</th>
<th>RC (mass/cement mass)</th>
<th>SFRC (mass/cement mass)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement (SG = 3.15)</td>
<td>1.00</td>
<td>1.00</td>
</tr>
<tr>
<td>Hydraulic slag (SG = 2.90)</td>
<td>0.49</td>
<td>0.49</td>
</tr>
<tr>
<td>Fly ash (SG = 2.20)</td>
<td>0.43</td>
<td>0.43</td>
</tr>
<tr>
<td>Silica fume (SG = 2.12)</td>
<td>0.11</td>
<td>0.11</td>
</tr>
<tr>
<td>Coarse aggregate (SG = 2.74)</td>
<td>2.82</td>
<td>2.76</td>
</tr>
<tr>
<td>Fine aggregate (SG = 2.73)</td>
<td>1.76</td>
<td>1.76</td>
</tr>
<tr>
<td>Steel fibres (SG = 7.80)</td>
<td>0.00</td>
<td>0.11</td>
</tr>
<tr>
<td>Polypropylene fibres (SG = 0.94)</td>
<td>0.003</td>
<td>0.003</td>
</tr>
</tbody>
</table>

**SG** = specific gravity

Water/binder ratio = 0.29

Air entrainment = 25 ml/m³ (0.024 fl oz/ft³) was added in both the RC and SFRC segments

Polycarboxylate superplasticizer = 480 ml/m³ (0.46 fl oz/ft³) was added in both the RC and SFRC segments

### Table 3.2 – Fresh and hardened properties of concrete mixture composition of full-scale RC and SFRC segments

<table>
<thead>
<tr>
<th>Properties</th>
<th>RC segment</th>
<th>SFRC segment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fresh concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Slump, mm (in)</td>
<td>170 (6.70)</td>
<td>150 (5.90)</td>
</tr>
<tr>
<td>Air content (%)</td>
<td>6.9</td>
<td>4.5</td>
</tr>
<tr>
<td>Temperature, °C (°F)</td>
<td>25 (77)</td>
<td>22 (72)</td>
</tr>
<tr>
<td>Hardened concrete</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Compressive strength, MPa (psi)</td>
<td>60.0 (8700)</td>
<td>61.4 (8900)</td>
</tr>
<tr>
<td>Splitting tensile strength, MPa (psi)</td>
<td>7.5 (1008)</td>
<td>9.0 (1300)</td>
</tr>
<tr>
<td>Density, kg/m³ (lb/ft³)</td>
<td>2380 (149)</td>
<td>2456 (153)</td>
</tr>
<tr>
<td>Water permeability coefficient, m/s (ft/s)</td>
<td>43×10⁻¹⁵</td>
<td>22×10⁻¹⁵</td>
</tr>
<tr>
<td></td>
<td>(141×10⁻¹⁵)</td>
<td>(72×10⁻¹⁵)</td>
</tr>
</tbody>
</table>
### Table 3.3 – Bending properties of SFRC

<table>
<thead>
<tr>
<th>$\delta_1$ (mm)</th>
<th>$\delta_p$ (mm)</th>
<th>$f_1$ (MPa)</th>
<th>$f_p$ (MPa)</th>
<th>$f_{600}$ (MPa)</th>
<th>$f_{300}$ (MPa)</th>
<th>$f_{150}$ (MPa)</th>
<th>$T_{150}$ (MPa)</th>
<th>$R_{150}$ (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.068</td>
<td>0.345</td>
<td>7.62</td>
<td>8.13</td>
<td>5.68</td>
<td>3.86</td>
<td>2.70</td>
<td>93.82</td>
<td>52.40</td>
</tr>
</tbody>
</table>

$\delta_1$ = deflection at first peak load; $\delta_p$ = deflection at peak load; $f_1$ = first peak strength; $f_p$ = peak strength; $f_{600}$, $f_{300}$, $f_{150}$ are the residual strength at net deflection of L/600, L/300 and L/150, respectively; $T_{150}$ = area under the load-deflection curve 0 to L/150 and $R_{150}$ is the equivalent flexural strength ratio. (1 in = 25.4 mm; 1 ksi = 6.90 MPa)

### Table 3.4 – Flexural test results for RC and SFRC PCTL segments

<table>
<thead>
<tr>
<th>Parameters</th>
<th>RC</th>
<th>SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td>First crack load, kN (kip)</td>
<td>45 (10.11)</td>
<td>71 (15.96)</td>
</tr>
<tr>
<td>Displacement at first crack, mm (in)</td>
<td>1.80 (0.070)</td>
<td>1.60 (0.62)</td>
</tr>
<tr>
<td>Average crack width at first crack, mm (in)</td>
<td>0.20 (0.008)</td>
<td>&lt; 0.10 (0.004)</td>
</tr>
<tr>
<td>Peak load, kN (kip)</td>
<td>244 (54.85)</td>
<td>119 (26.75)</td>
</tr>
<tr>
<td>Peak load displacement, mm (in)</td>
<td>50.26 (1.97)</td>
<td>5.06 (0.20)</td>
</tr>
<tr>
<td>Average crack width at peak load, mm (in)</td>
<td>8.20 (0.32)</td>
<td>0.25 (0.0098)</td>
</tr>
</tbody>
</table>

### Table 3.5 – Design results comparison for RC and SFRC PCTL segments

<table>
<thead>
<tr>
<th>Loading</th>
<th>Parameters</th>
<th>RC</th>
<th>SFRC</th>
</tr>
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<tbody>
<tr>
<td></td>
<td></td>
<td>Design</td>
<td>Experimental</td>
</tr>
<tr>
<td><strong>Static</strong></td>
<td>Moment, kN-m (kip-in)</td>
<td>25 (221)</td>
<td>183 (1673)</td>
</tr>
<tr>
<td></td>
<td>Thrust, kN (kip)</td>
<td>795 (179)</td>
<td>1425 (320)</td>
</tr>
<tr>
<td><strong>Cyclic</strong></td>
<td>Moment, kN-m (kip-in)</td>
<td>77 (681)</td>
<td>202 (1788)</td>
</tr>
<tr>
<td></td>
<td>Thrust, kN (kip)</td>
<td>238 (53)</td>
<td>N/A</td>
</tr>
</tbody>
</table>
1 in = 25.4 mm

**MD90.3** = metric deformed (MD) rebar having area 90.3 mm$^2$

**Figure 3.1** – RC segment dimensions and detailing.
Figure 3.2 – Flexural testing of PCTL segments, a) instrumentation test setup, b) waffle tree loading frame, and c) schematic of flexural test.
Figure 3.3 – Thrust load testing of segments, a) Experimental setup, b) Close view of loading area, and c) Schematic top view.
Figure 3.4 – Load-mid span displacement curves for RC and SFRC PCTL segments.

Figure 3.5 – Break down of lining segments.
Figure 3.6 – Failure surfaces of SFRC segments.

Figure 3.7 – Representation of crack patterns for a) RC and b) SFRC segments under static load.
a) RC segment  
b) SFRC segment  

**Figure 3.8** – Cracks in RC and SFRC lining segments.

a) RC segment  
b) SFRC segment  

**Figure 3.9** – Hysteresis curves for a) RC and b) SFRC PCTL segments under cyclic load.
Figure 3.10 – Skeleton and energy dissipation for a) RC and b) SFRC segments under cyclic load.

Figure 3.11 – Stiffness degradation for RC and SFRC segments under cyclic load.
Figure 3.12 – Thrust load test results for RC and SFRC segments.
SETTLEMENT AND PUNCHING BEHAVIOUR OF FULL-SCALE RC AND SFRC PRECAST TUNNEL LINING SEGMENTS*

The overall stability of a segmental tunnel lining ring is highly dependent on the structural integrity of its individual precast segments, which provide protection against surrounding conditions. In this chapter, the settlement and punching behaviour of full-scale conventional reinforced concrete (RC) and steel fibre-reinforced concrete (SFRC) precast tunnel lining segments fabricated for a subway tunnel in Canada were investigated. Precast concrete tunnel lining (PCTL) segment specimens were subjected to mid-span load on their intrados faces. Such loading can be induced as a result of vehicular accidents inside the tunnel and/or settlement underneath the PCTL segments. Moreover, a punching test was conducted to replicate the expansion of rocks or other geotechnical surrounding conditions above or underneath the tunnel segments. The geotechnical properties of job site where full-scale tested PCTL segments were installed are shown in Table A.1.

4.1. RESEARCH SIGNIFICANCE

Lining segments are exposed to external loads either from inside the tunnel (intrados faces) and/or outside the tunnel (extrados faces). External loads on the tunnel lining from inside the tunnels include vehicular load while, expansion of rocks or other concentrated loads from outside the tunnel are acting at the extrados faces of tunnel linings. Prior studies (Nishikawa, 2003; Moccihino et al., 2010; Caratelli et al., 2011) were conducted by applying a uniformly distributed load on the extrados faces of segment’s mid-span. However, the effect of concentrated load or punching load on the extrados face of tunnel linings is missing. Furthermore, the behaviour of PCTL segments under loads on intrados faces is largely unexplored. Therefore, this research investigates the settlement and punching behaviour of full-scale RC and SFRC PCTL segments in order to overcome this knowledge gaps.

* A version of this chapter was submitted to the Engineering Structures Journal (2013).
4.2. TUNNEL LINNING SEGMENT DESCRIPTION

The length, width and thickness of the tested full-scale RC and SFRC segments were 2120 mm (83.50 in), 1500 mm (59.00 in) and 235 mm (9.25 in), respectively. The edges of segments were skewed rather than straight ends (Figure 4.1). The concrete mixture composition used for the fabrication of conventional RC and SFRC PCTL segments was mentioned in earlier chapter (Table 3.1).

4.3. EXPERIMENTAL METHODOLOGY AND SETUP

4.3.1. Settlement Response Testing

A reaction frame consisting of two 1800 mm (70.86 in) long stiffened W10x49 beams welded together and assembled with bolts to the ground frame on both sides was used for testing the PCTL segments. Figure 4.2 shows the experimental setup for settlement testing of the PCTL segments. Each segment with its intrados face upward was simply supported on the reaction frame with a span of 1900 mm (74.80 in). A waffle tree loading frame was used to apply a uniformly distributed load, in agreement with previous studies (Moccihino et al., 2010; Caratelli et al., 2011). A rubber bearing pad was placed underneath the loading frame to ensure smooth contact surface. A controlled displacement load was applied at a rate of 0.5 mm/min (0.02 in/min) using a load cell actuator with an applied load capacity of maximum 1500 kN (337 kip). The load was continuously applied until the segment broke into two pieces. Four linear variable displacement transducers (LVDTs) were placed at the segment’s mid-span in order to measure the vertical displacement. All load and displacement readings were collected simultaneously through a high speed data acquisition system. Crack widths at various loading steps were measured using an elcometer crack width ruler.

4.3.2. Punching Test

Figure 4.3 illustrates the laboratory setup for the punching load test. The main goal of this test is to replicate the punching and load concentration induced due to the expansion behaviour of rocks or other geotechnical conditions surrounding PCTLs. Segments were placed on the reaction frame with their extrados facing up and incremental load was applied at their mid-span. The loading rate was 5 kN/min (1.12 kip/in). A loading plate 25 mm (1.0 in) thick was placed in between the loading actuator and the segments. Four LVDTs were
fastened at each side of the loading plate (Figure 4.3) for measuring the vertical displacement of the segments. Segments were white painted in order to inspect the cracking patterns. Load was continuously applied until the loading plate was completely sunk into the segments. This point was considered as the failure point for the tested segments.

4.4. RESULTS AND DISCUSSION

4.4.1. Settlement Behaviour

4.4.1.1. Load-Displacement Response

The settlement load-displacement response for RC and SFRC PCTL segments is shown in Figure 4.4. All the four LVDTs installed at the segment’s mid-span showed approximately similar displacement readings, indicating the absence of torsion effects (Mocchihi et al., 2010). Therefore, the average displacement values were reported. The load shown in Figure 4.4 represents the actual load carrying capacity of the tested PCTL segments plus the self-weight of the waffle tree loading frame. This is in agreement with previous study (Caratelli et al., 2011). The load-displacement curve (Figure 4.4) shows that the initial stiffness of the SFRC segment was higher than that of the RC segment. This can be ascribed to the role of steel fibres in the SFRC segment, which restrict the strain magnitude and formation of micro-cracks, thus leading to reduced internal material damage (King and Alder, 2001; Plizzari and Tiberti, 2006).

The load-displacement response of the RC segment can be categorized into three phases (Figure 4.4). First, a linear response was observed up to crack initiation (Point A in Figure 4.4) at a load level of 97 kN (21.80 kip). Afterwards, due to the development of more cracks, the slope of the curve decreased until reaching the yielding (Point B in Figure 4.4) at a load of 548 kN (123.20 kip). Finally, further increase in load continued up to the peak load (Point C in Figure 4.4) of 592 kN (133.10 kip). Subsequently, a brittle failure occurred along with an abrupt decrease in load carrying capacity. The failure of the RC segment was characterized by rupture of steel rebar, leading to segment breakage into two pieces.

On the other hand, the load-displacement curve of the SFRC segment can be divided into two phases: ascending and descending branches (Figure 4.4). First, a linear ascending portion was observed up to crack initiation (Point a in Figure 4.4) at a load of 192 kN (43.20
kip), which is almost double the initial crack load of the RC segment. At this point, more cracks started to form leading to higher stresses in the segment’s cross-section. As cracks developed, the load was progressively transferred to the steel fibres across cracks until reaching the peak load (Point c in Figure 4.4) of 203 kN (45.63 kip). Afterwards, the descending branch began and its slope changed as fibres were pulled-out or fractured, until complete failure of the segment. This descending branch indicates the ability of steel fibre reinforcement to provide adequate post-cracking behaviour of PCTL segments beyond the peak load. The failure surface demonstrated that fibres pull-out was more dominant than the fibres fracture (Figure 4.5). Moreover, yielding of fibres before pulled-out from the concrete matrix was observed, which is known to improve the energy absorption capacity of the PCTL segments.

Table 4.1 shows the results of settlement tests for the RC and SFRC PCTL segments. The load corresponding to first initial crack for the SFRC segment was almost double that of the RC. The peak/ultimate load carrying capacity of the tested RC PCTL segment was higher than that of the SFRC segment. However, during the fabrication of PCTL segments, delivery to the job site and installation process through TBM, accidental thrust and impact loads are dominant and may result in segment cracking, possibly compromising its structural integrity. Therefore, the initial cracking load of PCTL segments is considered as the ultimate design criteria (Mocchino et al., 2010; Caratelli et al., 2011). It should also be noted that the added amount of steel fibres for the tested SFRC segments can be increased, which would lead to increasing the peak load carrying capacity of SFRC PCTL (Rivaz, 2008).

4.4.1.2. Cracking Pattern

Schematic representation of the observed cracking patterns in the RC and SFRC PCTL segments is shown in Figure 4.6. In the case of the RC-PCTL segment, at a load level of 100 kN (22.50 kip) (Point A in Figure 4.6 (a)), one straight discontinuous crack at the mid-span was observed. Few small cracks of around 90 to 120 mm (3.54 to 4.72 in) in length were also observed around that discontinuous first crack. These smaller cracks spread further on both sides of the initially developed crack, in addition to the development of new parallel cracks at a load of 250 kN (56.20 kip) (Point B in Figure 4.6 (a)). The spacing between these parallel cracks was around 185 mm (7.30 in), mapping the location of rebars. A wide spread of
continuous and discontinuous straight cracks were observed at a load of 520 kN (117 kip) (Point C in Figure 4.6 (a)). These cracks became continuous near the failure point and a significant increase in crack width was also observed (Point D in Figure 4.6 (a)).

On the other hand, a single discontinuous small crack occurred at mid-span of the SFRC segment at a load of 194 kN (43.61 kip) (Point a in Figure 4.6 (b)) which is almost double that of the RC segment. This crack propagated further along with the formation of new hair-line cracks at the ultimate load (Point b in Figure 4.6 (b)) of 203 kN (45.63 kip). After reaching the ultimate capacity of the segment, a descending branch began and cracks parallel to the previously developed cracks increased in number (Point c in Figure 4.6 (b)). Moreover, a widespread of small discontinuous cracks was noticed before the complete failure of the segment (Point d in Figure 4.6 (b)).

It can be concluded that the SFRC segment exhibited a smaller number of cracks compared to that of the RC segment (Figures 4.6 and 4.7). Moreover, it was observed that in SFRC segments, stresses and deformations were localized at a single point, leading to the development of a single macro-crack after reaching the peak load carrying capacity (Figure 4.7).

Table 4.2 shows the crack widths at the first crack load and peak load of the PCTL segments. The crack width in the SFRC segment was found to be smaller than that of the RC segment due to the crack bridging action of steel fibres. Furthermore, it was observed that the crack width at peak load in the SFRC segment was less than 0.30 mm (0.012 in), which satisfies the accepted crack width limit for serviceability conditions (ACI 224, 2001). It can be argued that SFRC can limit the initial crack width of PCTL segments. Thus, the penetration of aggressive species to concrete can be reduced, possibly leading to enhanced durability.

According to the International Tunneling Association (ITA, 2007), concrete cracking is the major cause of damage, which can be categorized as “architectural, functional and structural” damage depending on developed crack width. Based on the monitored crack width data for the tested PCTL segments and criteria mentioned in the ITA (Table 4.3), the SFRC segments showed very slight damage, while the RC segments demonstrated moderate
damage. Moreover, the crack width can be correlated with the concrete tensile strain due to
settlement loads (Burland and Wroth, 1975). Table 4.3 shows that the SFRC segment
exhibited critical concrete extension of less than 0.10%, which satisfies the tensile strain
criteria for concrete tunnel linings with respect to serviceability and ultimate limit states
(ITA, 2007). Therefore, the SFRC segments can be considered as a potential alternative for
precast tunnel linings.

4.4.2. Punching Behaviour

4.4.2.1. Load-Displacement Response

Figure 4.8 shows the load-displacement curve under punching load for the RC and SFRC
PCTL segments. Under punching load, SFRC segments exhibited higher initial stiffness
compared to that of the RC segments due to reduced internal material damage owing to the
effect of steel fibres bridging micro-cracks (King and Alder, 2001). The mid-span
displacement was monitored using four LVDTs installed on each side of the loading plate.
All four LVDTs showed approximately identical displacement. Therefore, their mean value
was represented in Figure 4.8. The RC segment failed in a very abrupt way and its concrete
cover was detached before the failure point (Figure 4.9). Conversely, the SFRC PCTL
segment exhibited more progressive behaviour as steel fibres effectively bridge the cracks.
This is in agreement with previous study (Nguyen-Minh et al., 2011).

The load-displacement curve shows that the crack formation started (Point A in Figure
4.8) on the extrados face of the RC segment at a load of 105 kN (23.60 kip). Afterwards, the
slope of the curve (stiffness) decreased due to the formation of additional cracks. These
cracks significantly widened at the peak load (Point B in Figure 4.8) of 523 kN (117.60 kip).
Thereafter, a sudden drop in load carrying capacity of RC-PCTL segment took place with a
very brittle failure. Conversely, cracks were initiated (Point a in Figure 4.8) at a load of 235
kN (52.83 kip) for the SFRC segment, which is twice the load of the RC segment. Then, the
slope of the curve decreased up to the peak load (Point b in Figure 4.8) of 253 kN (56.87
kip). After reaching the ultimate load, a descending branch of the curve was recorded. The
slope of the descending branch decreased as fibres were pulled-out or fractured. A steadier
drop in load carrying capacity of the SFRC segment was attributed to the role of steel fibres,
which stabilized the post cracking behaviour.
While the punching shear capacity of the RC PCTL segments was much higher than that of the SFRC segments, the SFRC segments exhibited higher initial shear cracking load and more stable post-peak behaviour. This improved cracking behaviour of the SFRC segments can be beneficial for sustaining the initial handling and installation processes of PCTL segments at job sites. Furthermore, the higher cracking strength of SFRC segments can be favourable in resisting the penetration of chloride ions or other corrosive media that can jeopardize PCTL durability.

4.4.2.2. Cracking Pattern

As the applied load increased, the loading plate had sunk into the segments (Figure 4.9 (a)) in agreement to previous study (Hamada et al., 2008). The failure mode for both the RC and SFRC segments was punching shear. No uplift at the segment’s corner was observed due to punching action at the mid-span. Diagonal shear cracks at an angle of around $45^\circ$ were observed on the extrados faces of both the RC and SFRC segments (Figure 4.10). The SFRC segment exhibited a fewer number cracks in comparison with the RC segment.

In case of the RC segment, at a load of 150 kN (33.72 kip), a diagonal crack greater than 0.25 mm (0.01 in) in width initiated from the edge of the loading plate at the segment’s mid-span and propagated towards the corner of the segment (Point A in Figure 4.10 (a)). More diagonal shear cracks continuously developed and propagated with increased width as the loading increased (Point B in Figure 4.10 (a)). Near to the failure load (Point C in Figure 4.10 (a)), continuous and discontinuous cracks were observed around the loading plate, which propagated towards the corner of the RC segment along with further increase in crack width. Conversely, the SFRC segment at a load of 235 kN (52.83 kip) showed some discontinuous shear cracks that propagated diagonally towards the corner of the segment with a relatively smaller crack width (< 0.15 mm (0.006 in)) (Point a in Figure 4.10 (b)) than that in the RC segment. At the peak load of 253 kN (56.87 kip), some new discontinuous diagonal cracks developed along with diagonal propagation of the previously developed cracks (Point b in Figure 4.10 (b)). After reaching the peak load, the crack width increased in the descending branch as the fibres were pulled-out from the matrix or fractured and more diagonal discontinuous cracks propagated towards the corner of the SFRC segment (Point c in Figure 4.10 (b)).
The concrete cover on both the extrados and intrados faces was detached before failure of the RC segment (Figures 4.9 and 4.11). Moreover, few shear cracks at an angle of 20° were also observed along the thickness of the RC segment (Figure 4.9 (b)). Conversely, the SFRC segment showed more stable and uniform cracks due to the crack bridging effect of steel fibres (Figures 10 (b) and 11 (b)). No concrete spalling was observed neither at the intrados nor the extrados surfaces of the SFRC segment. The improved punching crack behaviour of the SFRC-PCTL segment can be attributed to the steel fibres effect in resisting crack initiation and propagation by transferring stresses across cracks until fibres were pulled-out or fractured (Nguyen-Minh et al., 2011).

4.5. DESIGN CONSIDERATIONS

Generally, the tunnel lining settlement depends on the surrounding ground features, loading conditions and tunnel lining properties (ITA, 2007). Therefore, the maximum settlement \( s_{\text{max}} \) can be estimated using the following formula (ITA, 2007).

\[
s_{\text{max}} = K \frac{YR^2}{E}
\]

Eq. 4.1

Where, \( K \) is dependent on ground stresses, \( \lambda \) is the stress release coefficient, \( Y \) and \( E \) are the unit weight and modulus of elasticity of the surrounding soil, respectively and \( R \) is the radius of the tunnel.

The punching shear capacity of the RC and SFRC lining segments can be calculated using the following equations (Eqs. 4.2 and 4.3) proposed by Hamada et al. (2008) and Higashiyama et al. (2011).

\[
V = 1.35 \sqrt{f_c} \sqrt[4]{\rho u_p k_c h}
\]

RC segment  Eq. 4.2

\[
V = k_c (0.24f_{sp} + 0.41\tau F)u_p d
\]

SFRC segment  Eq. 4.3

Where, \( V \) is the punching shear strength, \( f_c' \) is the compressive strength of concrete for RC and SFRC PCTL segments, \( u_p \) is the critical perimeter, \( \rho \) is the tension reinforcement ratio, \( k_c \) is a factor depending on the thickness \( h \) and concrete cover, \( f_{sp} \) is the splitting tensile
strength, \( d \) is the effective depth, \( \tau \) is the interfacial bond stress between fibre and matrix, and \( F \) is a fibre factor depending on fibre geometrical properties.

The design settlement and punching shear values were calculated using the information provided for the precast tunnel lining segments tested in the present study (Table A.1). Table 4.4 shows a comparison between the design predicted and experimental values, indicating that both the RC and SFRC PCTL segments meet the design criteria. Therefore, it appears that SFRC can be considered as a valuable alternative for conventional RC lining segments due to its improved cracking behaviour, low corrosion potential and cost efficiency, which warrants further investigation of PCTL segments with various fibre dosages and concrete mix design to further enhance SFRC PCTL mechanical performance.

### 4.6. SUMMARY

This chapter explored the settlement and punching responses of full-scale precast RC and SFRC tunnel lining segments. The settlement and punching test results showed that the ultimate load carrying capacity of RC PCTL segments was higher than that of similar SFRC segments. However, the SFRC segments showed enhanced cracking behaviour and exhibited a steady drop in load carrying capacity due to the crack bridging action of steel fibres. The RC segments showed more abrupt decrease in load carrying capacity after reaching their peak/ultimate load and failed in a more brittle manner. It was observed that the crack widths in SFRC-PCTL segments were smaller than that of the RC segments. Moreover, both the RC and SFRC segments satisfied the design criteria for settlement and punching loads. Therefore, SFRC can be considered as an attractive alternative for manufacturing PCTL segments compared to conventional RC owing to eliminating laborious and costly rebar cages. Steel fibres restrict the initial formation of cracks during the fabrication, handling, delivery and installation processes at the precast plant and construction site, possibly leading to more durable tunnels compared to conventional RC PCTL. The optimization of the SFRC mixture composition for PCTL applications is important for achieving desired structural and durability properties of the overall tunnel systems, which warrants further investigation.
4.7. REFERENCES


Table 4.1 – Settlement and punching test results for RC and SFRC PCTL segments

<table>
<thead>
<tr>
<th>Test type</th>
<th>Segment Type</th>
<th>Cracking load kN, (kip)</th>
<th>Cracking displacement mm, (in)</th>
<th>Ultimate load kN, (kip)</th>
<th>Ultimate displacement* mm, (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement</td>
<td>RC</td>
<td>97 (21.80)</td>
<td>1.9 (0.075)</td>
<td>592 (133.08)</td>
<td>20.1 (0.80)</td>
</tr>
<tr>
<td></td>
<td>SFRC</td>
<td>192 (43.16)</td>
<td>3.0 (0.12)</td>
<td>203 (45.63)</td>
<td>3.8 (0.15)</td>
</tr>
<tr>
<td>Punching</td>
<td>RC</td>
<td>105 (23.60)</td>
<td>1.6 (0.063)</td>
<td>523.2 (117.62)</td>
<td>16.0 (0.63)</td>
</tr>
<tr>
<td></td>
<td>SFRC</td>
<td>235 (52.83)</td>
<td>2.9 (0.11)</td>
<td>253.2 (56.92)</td>
<td>3.6 (0.14)</td>
</tr>
</tbody>
</table>

* Ultimate displacement is the displacement at peak load

Table 4.2 – Monitored crack widths of PCTL segments

<table>
<thead>
<tr>
<th>Test type</th>
<th>Segment type</th>
<th>Maximum crack width, mm (in)</th>
<th>Crack load</th>
<th>Peak load</th>
</tr>
</thead>
<tbody>
<tr>
<td>Settlement</td>
<td>RC</td>
<td>0.20 (0.008)</td>
<td>12.50 (0.50)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SFRC</td>
<td>&lt; 0.15 (0.006)</td>
<td>0.25 (0.010)</td>
<td></td>
</tr>
<tr>
<td>Punching</td>
<td>RC</td>
<td>0.25 (0.010)</td>
<td>15.00 (0.60)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>SFRC</td>
<td>&lt; 0.15 (0.006)</td>
<td>0.20 (0.008)</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.3 – Damage criteria of tested PCTL segments

<table>
<thead>
<tr>
<th>Segment type</th>
<th>Damage type</th>
<th>Damage Description</th>
<th>Crack width (mm)*</th>
<th>critical extension, εc (%)*</th>
</tr>
</thead>
<tbody>
<tr>
<td>SFRC</td>
<td>1</td>
<td>very slight damage</td>
<td>&lt; 1 mm</td>
<td>0.050 &lt; εc ≤ 0.075</td>
</tr>
<tr>
<td>RC</td>
<td>3</td>
<td>moderate damage</td>
<td>5 to 15 mm</td>
<td>0.15 &lt; εc ≤ 0.30</td>
</tr>
</tbody>
</table>

1 in = 25.4 mm

* Values taken from ITA (2007)
Table 4.4 – Comparison of experimentally predicted versus design values of settlement and punching load tests

<table>
<thead>
<tr>
<th>Test type</th>
<th>Punching shear values</th>
<th>Segment type</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>RC</td>
<td>SFRC</td>
<td></td>
</tr>
<tr>
<td>Settlement</td>
<td>Experimental, mm (in)</td>
<td>20.80 (0.82)</td>
<td>3.70 (0.14)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Design, mm (in)</td>
<td>1.70 (0.067)</td>
<td>1.70 (0.067)</td>
<td></td>
</tr>
<tr>
<td>Punching</td>
<td>Experimental, kN (kip)</td>
<td>523.20 (117.62)</td>
<td>253.40 (57.00)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>Design, kN (kip)</td>
<td>193.46 (43.50)</td>
<td>237.00 (53.27)</td>
<td></td>
</tr>
</tbody>
</table>
1 in = 25.4 mm

MD154 = Metric deformed rebar having cross-sectional area 154 mm$^2$

**Figure 4.1 – RC segment dimension and detailing.**
Figure 4.2 – Settlement testing setup for PCTL segments, a) Experimental laboratory setup, and b) Schematic representation of laboratory setup.
Figure 4.3 – Punching test setup, a) Experimental laboratory setup, and b) Schematic representation of laboratory setup.
Figure 4.4 – RC and SFRC load-displacement curves for settlement test.

Figure 4.5 – Failure surfaces of SFRC segment during settlement test.
a) RC segment  
b) SFRC segment

Figure 4.6 – Schematic representation of cracking pattern in RC and SFRC segments during settlement test.

a) RC segment  
b) SFRC segment

Figure 4.7 – Cracking pattern in RC and SFRC segments during settlement test.
Figure 4.8 – Load-displacement curve for punching test for RC and SFRC segments.

Figure 4.9 – Concrete flaking in RC segment during punching test.
a) RC segment  
b) SFRC segment

Figure 4.10 – Cracking pattern of RC and SFRC segments during punching test.

a) RC segment  
b) SFRC segment

Figure 4.11 – Punching failure pattern for RC and SFRC segments.
Chapter 5

CHLORIDE IONS PENETRATION IN RC AND SFRC
PRECAST TUNNEL LINING SEGMENTS*

Steel corrosion induces distress in concrete structures, resulting in the cracking and spalling of the concrete cover, which accelerates further damage. Chloride ions are the primary causes of steel corrosion in concrete. This chapter investigates the effects of chloride ions in precast concrete tunnel lining (PCTL) segments through conducting various tests on cylindrical cores extracted from full-scale conventional reinforced concrete (RC) and steel fibre-reinforced concrete (SFRC) segments.

5.1. RESEARCH SIGNIFICANCE

Conventional reinforced concrete precast tunnel lining segments often have high life-cycle maintenance costs due to premature deterioration induced by chloride ions (Cl\textsuperscript{-}) penetration leading to reinforcement corrosion. Conversely, steel fibre reinforcement has little potential for corrosion due to its discontinuous and dispersed nature, in addition to its ability to reduce Cl\textsuperscript{-} ingress (Roque et al., 2009; Rivaz, 2008; 2010). While significant research has studied Cl\textsuperscript{-} diffusion in conventional RC, very limited research has explored that behaviour in SFRC. Therefore, the performance of SFRC under various chloride exposure conditions was investigated in this chapter and compared to that of conventional RC. Furthermore, a full \(2^3\) factorial analysis was conducted on the experimental data to evaluate the contribution of individual factors and their interaction. The potential chloride profile results thus obtained in this study may form the basis for service life prediction models of RC and SFRC PCTL.

5.2. SPECIMEN PREPARATION

Cores were retrieved from full-scale RC and SFRC tunnel lining segments (Figures 5.1 and C.1). The details of coring process were mentioned earlier (Section 3.3.2). Figure C.2 shows the retrieved cores. The full depth core was divided into three specimens. The upper and bottom specimens represented the external (extrados or intrados faces of segments), while the

*A version of this chapter was submitted to the ACI Materials Journal (2013). Part of this chapter was published in the Tunneling Association of Canada Conference, Montreal, Canada (2012).
middle part of the core represented the internal core of the segment (Figure 5.2). The number of tested specimens for different tests carried out is listed in Table 5.1.

5.3. CHLORIDE IONS PENETRATION ASSESSMENT

5.3.1. Sorptivity Test

The sorptivity test was conducted according to the ASTM C1585 specifications (Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic Cement Concretes). External and internal 50×100 mm (2×4 in) specimens were tested to measure water sorptivity through the RC and SFRC. Each result reported represents the average value obtained on three identical specimens with an average coefficient of variation less than 6%, which satisfies the ASTM C1585 recommendations. Specimens were placed in a walk-in environmental chamber at 50 °C (122 °F) and 80% relative humidity for 3 days. The specimens were then stored in a sealable container at room temperature (25 °C (77 °F)) for 15 days. Before testing, the circumference of the specimens was sealed using epoxy and duct tape and the initial weight of the specimens was measured. Afterwards, the surface of specimens was immersed in 3 to 5 mm (0.1 to 0.2 in) deep water (Figure C.3). The specimens were removed and weighed frequently according to the time duration defined by ASTM C1585. The sorptivity coefficient (S) was evaluated as the slope of the best fit line between absorption (I) and under root of time (t), as given by Eq. 5.1.

$$S = \frac{I}{\sqrt{t}} = \frac{m/(a*d)}{\sqrt{t}}$$  \hspace{1cm} \text{Eq. 5.1}

Where, \(m\) is the variation in specimen weight (g), \(a\) is the exposed area of the tested specimen (mm\(^2\)), \(t\) is the time and \(d\) is the water density (g/mm\(^3\)).

5.3.2. Permeable Pore Space Test

Three 100×200 mm (4×8 in) RC and SFRC cored specimens were tested according to ASTM C642 (Standard Test Method for Density, Absorption, and Voids in Hardened Concrete). The volume of permeable voids can be evaluated using Eq. 5.2:

$$p = \frac{(g_2-g_1)}{g_2} \times 100$$  \hspace{1cm} \text{Eq. 5.2}
Where, \( p \) is the permeable pores (%), \( g_1 \) and \( g_2 \) are the dry bulk and apparent densities, respectively. Each result reported represents the average value of three identical specimens with an average variation of less than 5%, which satisfies the ASTM C642 requirements.

### 5.3.3. Salt Ponding Test

In order to determine Cl\(^-\) penetration into the cored RC and SFRC specimens, the salt ponding test was conducted in accordance with AASHTO T259 (Standard Method of Test for Resistance of Concrete to Chloride Ion Penetration), modified for different temperature, duration and solution concentration as described below. External and internal core specimens of 100 mm (4 in) in diameter and 75 mm (3 in) in thickness from both RC and SFRC were tested. The circumference of specimens was coated with epoxy and duct tape (Figure C.4). The height of the salt ponding solution from the specimen’s surface was 12 to 15 mm (0.5 to 0.6 in). After the 90 days of salt ponding prescribed by AASHTO T259, the concentration and penetration of Cl\(^-\) inside the concrete were determined. In addition to the 3% sodium chloride (NaCl) solution recommended by AASHTO T259, 3.5% and 10% NaCl solutions were also investigated to simulate the salinity level in underground water and to accelerate deterioration, respectively (NT BUILD 492, 1999, Granju and Balouch, 2005; Matsumura et al., 2008; Balouch et al., 2010; Guoping et al., 2011). Specimens were further investigated after 180 days in order to examine the effects of longer term chloride ion exposure. All specimens were stored at 40 °C (104 °F) during testing in order to accelerate chloride penetration (McGrath and Hooton, 1999).

At the end of the testing period, approximately 10 g (0.35 oz.) of concrete powder samples were collected from the tested specimens at different depths from the top surface at intervals of 3 mm (0.12 in) for chemical analysis. Three powder samples at each depth were analyzed for percentage chloride contents and the average value was reported. The coefficient of variation for percentage chloride contents at each depth ranged from 2% to 7% for all specimens.

### 5.3.4. Chemical Analysis of Chlorides

The chloride content versus penetration depth was assessed according to ASTM C114 (Standard Test Methods for Chemical Analysis of Hydraulic Cement, Section 19-Chlorides)
and FHWA-RD-72-12 (Sampling and Testing of Chloride Ion in Concrete). The percentage chloride content (by weight of concrete) was calculated using the following formula (Eq. 5.3).

\[
\% \ Cl = \left( \frac{3.5453}{w_c} \right) E_a N \tag{Eq. 5.3}
\]

Where, \(E_a\) actual end point (ml) determined by performing the titration using Metrohm Auto titrator; \(N\) normality of the AgNO\(_3\) solution and \(w_c\) concrete sample weight (g).

5.3.5. Rapid Chloride Ion Penetrability Test

The rapid chloride ion penetrability test (RCPT) was conducted according to ASTM C1202 (Standard Test Method for Electrical Indication of Concrete's Ability to Resist Chloride Ion Penetration) (Figure C.5). Five specimens having 100 mm (4 in) in diameter and 50 mm (2 in) in thickness were tested from both external and internal core specimens for both the RC and SFRC PCTL segments. The circumferential surface of these specimens was coated with epoxy and duct tape (Figures C.4 and C.5). After sufficient drying, specimens were placed in a vacuum dessicator for 3 hours in such a way that both of their ends were exposed. The specimens were subsequently soaked in water for 18 hours before testing. Various salt concentration solutions (3%, 3.5% and 10%) were investigated for RCPT to capture the effect of variation in the chloride exposure. The amount of coulombs passed was recorded every 30 minutes for up to 6 hours.

5.3.6. Mercury Intrusion Porosimetry

Fragments were taken from the external and internal zones of cored RC and SFRC specimens. The pore size distribution was examined using a Micromeritics AutoPore IV 9500 Series porosimeter for pressure range upto 414 MPa (60000 psi). Specimen’s preparation and testing procedure can be found elsewhere (Soliman and Nehdi, 2011).

5.4. RESULTS AND DISCUSSION

5.4.1. Sorptivity Coefficient

The initial (i.e. measured during the first 6 hours of the test) and secondary (i.e. measured after the first 6 hours and up to 9 days) sorptivity coefficients were determined on external
and internal specimens of RC and SFRC PCTL segments (Table 5.2). Generally, SFRC exhibited lower sorptivity coefficient values than that of the normal concrete. SFRC exhibited around 5% lower initial sorptivity coefficient for both the external and internal specimens compared to that of the RC specimens. This difference was more manifest for the secondary sorptivity coefficient, indicating the significant effect of exposure duration on water absorption capacities (Figure 5.3). The SFRC had around 13% and 25% lower secondary sorptivity coefficient than that of the RC specimens for the external and internal specimens, respectively. This indicates that the addition of steel fibres resulted in relatively less connected porosity and micro-cracks. Similar findings were reported in another study (Roque et al., 2009).

Regardless of the concrete type, the initial sorptivity values for the external and internal specimens were comparable (i.e. difference was less than 1%). Conversely, the secondary sorptivity results for the external specimens showed 35% and 24% lower water absorption capacities than that of the internal specimens for the RC and SFRC, respectively. This can be ascribed to the filling of small cavities at the extrados and intrados faces of PCTL segments with cement slurry after the demolding process. Figures B.3 and B.4 show the demolding process of PCTL segments and the filling of cavities at the precast plant. This filling process sealed the pores at the external faces (i.e. extrados and intrados faces) of the PCTL segments. This was confirmed by MIP test results shown in Figure 5.4. It was found that the external specimens had 22% and 14% lower porosity values compared with that in internal specimens for the RC and SFRC specimens, respectively. Moreover, SFRC specimens exhibited 18% and 26% lower total porosity than that of RC for external and internal specimens, respectively. This points to possibly better durability performance of SFRC and lower chloride penetration than that of RC specimens. Furthermore, it can be concluded that the external surfaces of RC and SFRC PCTL segments exhibited denser pore structure compared with that of the internal core, which can possibly lead to improved durability properties if cracking is also prevented.

5.4.2. Percentage of Permeable Voids

The volume of permeable voids for RC and SFRC specimens was 9.74% and 9.06%, respectively. This percentage of permeable voids in the RC and SFRC PCTL segments (less
than 14%) indicates potentially high durability according to the VicRoads classification for concrete durability (CCAA, 2009). Apparently, the addition of steel fibres contributed to reducing the volume of permeable voids from 9.74% to 9.06% which is in agreement with previous findings (Roque et al., 2009). The enhanced behaviour of SFRC can be attributed to the crack bridging effect induced by steel fibres as it interferes with water movement within the concrete matrix (Roque et al., 2009). This slightly lower permeable void of SFRC can probably increase its resistance to Cl⁻ penetration compared to that in RC.

5.4.3. Chloride Profiles Parameters

5.4.3.1. Penetration of Chlorides

Chloride profiles (i.e. chloride content expressed as a percentage weight of concrete versus depth from the ponding surface) were plotted in order to assess the quality of RC and SFRC PCTL segments. Figures 5.5, 5.6 and 5.7 show typical potential chloride profiles for RC and SFRC specimens. Chloride penetration decreased with increasing depth for both the RC and SFRC specimens at different rates for various exposure solutions (i.e. 3%, 3.5% and 10% NaCl). The chloride contents were assumed to approach the baseline or initial chloride content in the lower flatter portion of the chloride profiles. External and internal specimens for RC and SFRC showed similar profile of chloride content versus depth. However, external specimens exhibited lower chloride contents at each depth compared to that of the internal specimens for both RC and SFRC specimens (Figure 5.5). This is likely due to the application of a cement slurry as discussed earlier.

The chloride penetration at each depth in SFRC specimens was lower than that in RC specimens for all exposure solutions (3%, 3.5% and 10% NaCl) at the various test ages (90 and 180 days), indicating better durability behaviour of SFRC. This can be attributed to the role of steel fibres in reducing chloride penetration inside the concrete as mentioned earlier. Furthermore, the improved durability properties of SFRC were confirmed through computer tomography (CT) analysis of RC and SFRC specimens (Figures 5.8 and 5.9). The pores or voids percentage was determined on CT scanned RC and SFRC specimens images by generating the histogram profiles using MicroView analysis software. CT scan results revealed that the RC and SFRC specimens exhibited 3.8% and 1.5% voids or empty spaces,
This demonstrates potentially enhanced durability of SFRC segments through reduced chloride penetration owing to decreased porosity.

After 90 days of exposure and at depth of 3 mm (0.12 in) from the surface ponding solution (3% NaCl), SFRC showed approximately 30% and 24% lower Cl\(^-\) contents than that in RC for the internal and external cored specimens, respectively. A similar trend was found for the 3.5% NaCl ponding solution. For the 10% ponding solution, after 90 days of exposure, SFRC specimens exhibited around 11% and 18% lower Cl\(^-\) contents compared to that of the RC specimens for the internal and external specimens, respectively. This difference was slightly higher at 180 days of exposure. For instance at 180 days, compared to that of RC, SFRC exhibited 34% and 27% lower chloride contents for exposure to the 3% and 3.5% ponding solutions, and 16% and 19% for exposure to the 10% ponding solution for the internal and external specimens, respectively.

Moreover, a few corrosion spots were visible at the surface steel fibres for all exposure solutions after 2 weeks from starting the ponding test (Figure 5.10 (a)) in agreement with previous studies (Balouch et al., 2010; Mangat and Gurusamy, 1988). The formation of corrosion product at the surface fibres results in closing small pores (Figure 5.11 (a)); this was confirmed through scanning electron microscopy (SEM). This restricts the further penetration of chlorides into the hardened concrete. Conversely, micro-cracks were observed at the surface of RC specimens (Figure 5.11 (b)). This was attributed to the formation of expansive chloro-aluminate cubical crystals due to reaction between Cl\(^-\) and cementitious materials, leading to internal distress and micro-cracking (Ben-Yair, 1974).

Embedded steel fibres were visually examined for signs of corrosion by cutting slices from the cylindrical SFRC specimens. No evidence of corrosion of steel fibres was found beyond a depth of 5 mm (0.19 in) from the ponding surface (Figure 5.10 (b)). Although Cl\(^-\) had penetrated deeper, its content was below the corrosion threshold value (i.e. 0.2%). Furthermore, oxygen necessary for the onset of corrosion was not available in sufficient amount. Thus, only surface external fibres were corroded. This agrees with previous studies (Balouch et al., 2010; Granju and Balouch, 2005; Balouch and Granju, 1999; Mangat and Gurusamy, 1987).
The difference in Cl\(^{-}\) penetration for the 3% and 3.5% chloride exposure conditions was insignificant, which is expected. However, increasing the chloride concentration to 10% induced a worst scenario, highlighting the significant effect of very severe chloride exposure conditions (Figure 5.6). Beyond 25 mm (1 in) from the ponding surface of RC and SFRC specimens, no signs of chloride penetration were detected after 90 days of exposure (Figures 5.5 and 5.6), which is in agreement with previous research (Mangat and Gurusamy, 1987; Costa and Appleton, 1999). At 180 days of exposure, the RC specimens showed chloride penetration up to 35 mm (1.38 in) depth from the ponding surface, while Cl\(^{-}\) penetrated to only 30 mm (1.18 in) in SFRC specimens. Moreover, the Cl\(^{-}\) content at all depths was higher at later ages (Figure 5.7), indicating the significant effect of the exposure period.

### 5.4.3.2. Diffusion Coefficient and Surface Chlorides

Equation 2.9 was used in order to determine the chloride diffusion coefficient \((D)\) and surface chloride content \((C_0)\). Table 5.3 shows the chloride diffusion coefficient results for the RC and SFRC specimens extracted from their respective prototype PCTL segments. The diffusion coefficient varied between 3.55×10\(^{-12}\) to 7.76×10\(^{-12}\) m\(^2\)/sec (38.21×10\(^{-12}\) to 83.53×10\(^{-12}\) ft\(^2\)/sec) and 2.30×10\(^{-12}\) to 5.09×10\(^{-12}\) m\(^2\)/sec (24.76×10\(^{-12}\) to 54.79×10\(^{-12}\) ft\(^2\)/sec) for the RC and SFRC specimens, respectively. This indicates significant variability in the results caused by differences in the specimen’s surface conditions, exposure solutions and duration of exposure.

Generally, SFRC specimens showed lower diffusion coefficient values than that of the RC specimens under all exposure conditions. For instance, at age of 90 days, SFRC external and internal specimens exhibited about 41% and 34% lower diffusion coefficient values for the 3% ponding solution, and 24% and 19% lower diffusion coefficient values for the 10% ponding solution, respectively, compared to that of similar RC specimens. This confirms that the type of concrete can influence Cl\(^{-}\) ingress. The lower diffusion rate in SFRC specimens can be attributed to the barrier provided by steel fibres against Cl\(^{-}\) ingress. This barrier effect restricts the formation of micro- and macro-cracks and hence, blocks the Cl\(^{-}\) passage within the hardened concrete matrix. Similar findings were reported in previous work (Roque et al., 2009).
The external specimens of both the RC and SFRC PCTL segments showed lower Cl\textsuperscript{-} penetration compared with that of corresponding internal specimens, which is beneficial for tunnel linings exposed to aggressive environments. For instance, at 90 days of exposure, the RC external specimens exhibited around 17% and 9% lower diffusion coefficient than that of the internal specimens for the 3% and 10% solutions, respectively. The improved performance of the external specimens was due to the cement slurry surface treatment, which limited the accessible pores, and thus restrained Cl\textsuperscript{-} penetration. This was confirmed by MIP results shown earlier.

As expected, analysis of test results showed that the 10% NaCl ponding solution was the most severe exposure condition and exhibited higher diffusion coefficient than that of the 3% and 3.5% NaCl solutions. About 69% increase in the diffusion coefficient was observed as the concentration of the ponding solution increased from 3% to 10% for both the RC and SFRC specimens indicating that the exposure solution significantly affects the Cl\textsuperscript{-} penetration, regardless of the concrete type.

It was observed that the chloride diffusion coefficient decreased as the exposure age/period increased. For instance, the RC specimens exhibited around 18% lower diffusion coefficient at 180 days of exposure compared to that at 90 days. Similar reduction in the diffusion coefficient was observed for SFRC specimens. This reduction was also reported in previous studies (Bamforth and Chapman-Andrews, 1994; Polder and Larbi, 1995). It can be attributed to the following reasons: (1) the early formation of a surface protective layer of brucite and aragonite restricted the further penetration of chlorides inside the concrete (Figure 5.12) (Buenfeld and Newman, 1984; Haynes, 1980; Wegen et al., 1993); (2) the concrete porosity decreased as time passed due to advancing hydration of cement (Mangat and Gurusamy, 1987); and (3) the chemical reaction between cementitious hydration products and Cl\textsuperscript{-} resulted in the formation of hexagonal chloro-aluminate plates and consequently the porosity was reduced (Midgely and Illston, 1984; Kayyali, 1989; Hornain et al., 1995).

Table 5.3 shows the results of surface chloride concentration based on Eq. 2.9. Analysis shows that the surface chloride content was influenced by the exposure condition. For instance, the surface chloride concentration increased by about 50% as the concentration of
NaCl ponding increased from 3% to 10%. SFRC specimens exhibited about 15% and 22% lower surface chloride concentration than that of similar RC specimens under the 3% NaCl exposure for the external and internal specimens, respectively. At higher concentration of salt ponding solution, a relatively lower reduction was observed in the surface chloride concentration of SFRC specimens compared to that of the RC specimens (i.e. 13% and 18% for the external and internal specimens, respectively, under 10% NaCl ponding). Moreover, the surface Cl\(^-\) concentration increased over time under all exposure conditions for both the RC and SFRC specimens. This increase in the surface Cl\(^-\) concentration can be attributed to continuous surface deposition of chlorides with time (Figure 5.12) (Mangat and Gurusamy, 1987; Costa and Appleton, 1999; Swamy, 1994; Jaegermann, 1990; Lin, 1990). Therefore, it can be argued that the surface Cl\(^-\) concentration is a function of the exposure solution, exposure period and concrete quality.

5.4.4. Electric Resistivity and Chloride Ions Penetration

No significant increase in temperature due to electrical heating was observed during RCPT tests. For SFRC, no short circuiting was observed likely due to the relatively low percentage of steel fibres by volume of mixture and the fibre dispersed nature in agreement with previous work (Vaishali and Rao, 2012). Electric resistivity measurements of chloride ion penetrability for core specimens extracted from the RC and SFRC PCTL segments are summarized in Table 5.4. The coefficient of variation ranged from 3% to 9% for all specimens, which satisfies limits of the ASTM C1202 (i.e. 12.3%). A similar observation for the coefficient of variation was reported in previous works for normal concrete and SFRC specimens (Vaishali and Rao, 2012; Gergely et al., 2006). SFRC specimens at 3% Cl\(^-\) exposure showed an average 84 coulombs and 111 coulombs lower values for external and internal specimens respectively, compared to that of the RC specimens. A similar trend was observed for the 3.5% NaCl exposure. At 10% NaCl exposure, higher coulombs values were observed for both the RC and SFRC specimens compared to that of the 3% NaCl exposure. This trend was also depicted through the salt ponding test as described earlier. Analysis of results confirmed potentially better durability performance of SFRC PCTL segments compared with conventional RC segments. This is attributed to the addition of steel fibres, which tend to restrict the formation and growth of plastic and drying shrinkage cracks, resulting in decreased penetrability (Gergely et al., 2006). Moreover, it was observed that the
external specimens of PCTL segments exhibited lower coulomb values than that of the internal specimens due to difference in porosity, which was confirmed through MIP test results. For instance, the RC external surfaces exhibited 178 lower coulomb values for the 10% NaCl solution than that of the internal specimens, respectively.

Analysis of test results for specimens from both the RC and SFRC PCTL full-scale segments showed coulomb values lower than 1000, indicating low chloride penetrability according to ASTM C1202 specifications. This reflects the beneficial effects of the quaternary cement/slag/fly ash/silica fume binder used and the relatively low w/cm ratio.

5.5. STATISTICAL ANALYSIS USING FACTORIAL DESIGN

Three factors (segment type, surface type and % chloride exposure) at two levels (level 1 and 2), as shown in Table 5.5, require eight treatment combinations to run full factorial analysis (Montgomery, 2009). Table 5.6 shows the percentage contribution of each factor and their interactions on the response variables (chloride diffusion coefficient and number of coulombs passed). Analysis of results shows that the contribution of the Cl− exposure (3% and 10%) was higher than that of the segment type and surface type (external and internal). Therefore, the concentration of the Cl− exposure solution has a dominant effect on the Cl− diffusion coefficient and the number of coulombs passed. No significant interaction between factors was observed. For example, the percentage contribution of two factors interaction (i.e. segment type and surface type (TS) or segment type and % chloride (TC) or surface type and % chloride (SC)) and three factors interaction (i.e. segment type, surface type and % chloride (TSC)) was less than 2%, which is insignificant compared to that of individual factors. Therefore, it can be concluded that the individual factors are more dominant than the combined effect of two or more factors. The negative values in Table 5.6 indicate that the change in the levels of factors from level 1 to 2 resulted in a decrease in the response variables and vice versa (Montgomery, 2009). For instance, the negative sign for the effect of the segment type in Table 5.6 indicates that the change in the segment type from RC to SFRC reduced the chloride diffusion coefficient and the number of coulombs passed, which confirms the trend in experimental results.
5.6. SUMMARY

Based on the experimental results, it can be concluded that the SFRC PCTL segments achieved better resistance to Cl\(^-\) penetration than that of RC segments. This was attributed to the effect of steel fibres, which can restrict micro-cracking and interfere with the passage of Cl\(^-\) ions. The following specific conclusions can also be drawn from this study:

1) SFRC specimens exhibited lower initial and secondary sorptivity coefficient for both the external and internal specimens, compared to that of the RC specimens. The addition of steel fibres in SFRC segments appeared to have reduced the volume of permeable voids from 9.7% to 9.1%.

2) The Cl\(^-\) ion diffusion coefficient and number of coulombs passed for SFRC specimens was lower than that of RC specimens at all exposure conditions.

3) Factorial analysis confirmed that the effect of the concentration of Cl\(^-\) exposure condition dominated that of the segment type and surface conditions.

Based on limited experimental evaluation, it was observed that the diffusion coefficient increased with higher concentration of Cl\(^-\) in the ponding solution, and decreased as the exposure period increased. The relationship between the diffusion coefficient and concentration of Cl\(^-\) in the surrounding environment provides an interesting direction for future investigation.
5.7. REFERENCES


### Table 5.1 – Number and size of specimens for various tests

<table>
<thead>
<tr>
<th>Test</th>
<th>Chloride exposure</th>
<th>Specimen type</th>
<th>Number of specimens</th>
<th>Size of specimen (width × diameter)</th>
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</thead>
<tbody>
<tr>
<td>Sorptivity</td>
<td>-</td>
<td>External</td>
<td>3</td>
<td>50×100 mm (2×4 in)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td>3</td>
<td>50×100 mm (2×4 in)</td>
</tr>
<tr>
<td>Volume of permeable void</td>
<td>-</td>
<td>External</td>
<td>3</td>
<td>200×100 mm (8×4 in)</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Internal</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Salt ponding</td>
<td>3.0%</td>
<td>External</td>
<td>3</td>
<td>75×100 mm (3×4 in)</td>
</tr>
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<td></td>
<td></td>
<td>Internal</td>
<td>3</td>
<td>75×100 mm (3×4 in)</td>
</tr>
<tr>
<td></td>
<td>3.5%</td>
<td>External</td>
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</tr>
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<td>3</td>
<td>75×100 mm (3×4 in)</td>
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<td></td>
<td>10.0%</td>
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<td>75×100 mm (3×4 in)</td>
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<td></td>
<td>Internal</td>
<td>3</td>
<td>75×100 mm (3×4 in)</td>
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<td>50×100 mm (2×4 in)</td>
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<td>50×100 mm (2×4 in)</td>
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### Table 5.2 – Sorptivity coefficient for RC and SFRC specimens

<table>
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<th>Sorptivity</th>
<th>Surface</th>
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<th>SFRC</th>
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<tr>
<td>Initial</td>
<td>External</td>
<td>4.12×10⁻³</td>
<td>3.90×10⁻³</td>
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<td></td>
<td>Internal</td>
<td>4.15×10⁻³</td>
<td>3.92×10⁻³</td>
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<tr>
<td>Secondary</td>
<td>External</td>
<td>7.05×10⁻⁴</td>
<td>6.12×10⁻⁴</td>
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<tr>
<td></td>
<td>Internal</td>
<td>10.90×10⁻⁴</td>
<td>8.08×10⁻⁴</td>
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Table 5.3 – Diffusion coefficient and surface chloride for RC and SFRC specimens

<table>
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<th>Concrete Type</th>
<th>Surface</th>
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<th>3% NaCl</th>
<th>3.5% NaCl</th>
<th>10% NaCl</th>
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<tr>
<td></td>
<td></td>
<td></td>
<td>90 days</td>
<td>180 days</td>
<td>90 days</td>
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<td>RC</td>
<td>External</td>
<td>Diffusion</td>
<td>4.06</td>
<td>3.55</td>
<td>4.38</td>
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<tr>
<td></td>
<td></td>
<td>(×10^{-12} \text{ m}^2/\text{sec})</td>
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<tr>
<td></td>
<td></td>
<td>Surface Chloride (%)</td>
<td>0.23</td>
<td>0.32</td>
<td>0.24</td>
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<td>Internal</td>
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<td>3.88</td>
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<td></td>
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<td>SFRC</td>
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<td>Surface Chloride (%)</td>
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1 m^2/sec = 10.764 ft^2/sec

Table 5.4 – RCPT results for RC and SFRC specimens

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<th>Concrete Type</th>
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<th>Specimens</th>
<th>Average Coulombs</th>
<th>Coefficient of Variation</th>
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<td></td>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
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<tr>
<td>RC</td>
<td>3%</td>
<td>External</td>
<td>356</td>
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<td>Internal</td>
<td>401</td>
<td>422</td>
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<td></td>
<td>3.5%</td>
<td>External</td>
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<td>370</td>
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<td>501</td>
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<td></td>
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<td>245</td>
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<tr>
<td></td>
<td></td>
<td>Internal</td>
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<td>300</td>
<td>277</td>
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<tr>
<td></td>
<td>3.5%</td>
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<td>Internal</td>
<td>391</td>
<td>339</td>
<td>374</td>
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<td></td>
<td>10%</td>
<td>External</td>
<td>587</td>
<td>612</td>
<td>617</td>
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<td></td>
<td></td>
<td>Internal</td>
<td>779</td>
<td>732</td>
<td>698</td>
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Table 5.5 – Factors and their levels for factorial analysis

<table>
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<th>Factors</th>
<th>Level</th>
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<th>level 2</th>
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</thead>
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<tr>
<td>Segment type (T)</td>
<td></td>
<td>RC</td>
<td>SFRC</td>
</tr>
<tr>
<td>Surface type (S)</td>
<td></td>
<td>External</td>
<td>Internal</td>
</tr>
<tr>
<td>% chloride (C)</td>
<td></td>
<td>3</td>
<td>10</td>
</tr>
</tbody>
</table>

Table 5.6 – Percentage contribution of factors

<table>
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<tr>
<th>Treatment combination /Factors</th>
<th>Effects</th>
<th>Sum of squares</th>
<th>Percentage contribution</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Diffusion coefficient</td>
<td>RCPT</td>
</tr>
<tr>
<td></td>
<td></td>
<td>90 days</td>
<td>180 days</td>
</tr>
<tr>
<td>Segment type (T)</td>
<td></td>
<td>-2.15</td>
<td>-1.23</td>
</tr>
<tr>
<td>Surface type (S)</td>
<td></td>
<td>0.76</td>
<td>0.40</td>
</tr>
<tr>
<td>% Chloride (C)</td>
<td></td>
<td>2.45</td>
<td>2.12</td>
</tr>
<tr>
<td>TS</td>
<td></td>
<td>-0.02</td>
<td>0.06</td>
</tr>
<tr>
<td>TC</td>
<td></td>
<td>-0.48</td>
<td>-0.015</td>
</tr>
<tr>
<td>SC</td>
<td></td>
<td>-0.08</td>
<td>0.030</td>
</tr>
<tr>
<td>TSC</td>
<td></td>
<td>-0.02</td>
<td>0.020</td>
</tr>
</tbody>
</table>

‘TS’ represents the combined/interaction effect of the segment type and surface type.

‘TSC’ represents the combined/interaction effect of the segment type, surface type and % chloride.
Figure 5.1 – Coring process from full-scale SFRC PCTL segments.

Figure 5.2 – Designation of internal and external specimens retrieved from full-scale PCTL segments.
Figure 5.3 – Sorptivity test results for RC specimens.

Figure 5.4 – Measured porosity in RC and SFRC specimens using MIP test.
Figure 5.5 – Chloride profiles for 3% NaCl ponding at 90 days for internal and external specimens of RC and SFRC PCTL.

Figure 5.6 – Chloride profiles for 3.5% and 10% NaCl at 90 days for internal specimens.
Figure 5.7 – Chloride profiles for 3.5% NaCl for external specimens at 90 and 180 days.
Figure 5.8 – Computer tomography (CT) analysis of internal RC and SFRC specimens, 
a) CT scan 3-D view of RC specimen, and b) Fibre distribution in SFRC specimen.

Figure 5.9 – Void analysis using CT of internal RC and SFRC specimens, a) Voids in 
RC specimen, and b) Voids in SFRC specimen.
a) Surface steel fibres corrosion spots  
b) No corrosion at 5 mm from surface

Figure 5.10 – Surface corrosion of steel fibres at the surface.

a) Closing of micro-pores with corrosion products in SFRC specimen  
b) Micro-cracking in RC specimen

Figure 5.11 – SEM images of RC and SFRC specimens exposed to chloride solutions.
Figure 5.12 – Deposition of chlorides at the surface.
CORROSION PERFORMANCE OF RC AND SFRC PRECAST TUNNEL LINING SEGMENTS*

The applications of precast concrete tunnel lining (PCTL) segments are escalating due to its efficient and economical installation process compared to that of the normal in-situ lining practice. The proper functioning of these PCTL segments is highly affected by its durability performance. Reinforcement corrosion significantly compromises the PCTL performance as it leads to cracking and spalling of concrete, thus disturbing the overall structural integrity. In this chapter, the corrosion potential of specimens extracted from full-scale conventional reinforced concrete (RC) and steel fibre-reinforced concrete (SFRC) lining segments fabricated for a subway tunnel in Canada were evaluated.

6.1. RESEARCH SIGNIFICANCE

The durability of tunnel linings is a major problem for tunneling stakeholders. The penetration of chloride ions into the hardened concrete disturbs the integrity of lining systems due to corrosion initiation. Furthermore, the situation becomes more aggressive due to the development of micro- and macro-cracks during the manufacturing and TBM installation of lining segments, which significantly increases the chloride ions penetrability. In this chapter, an alternative material, SFRC, was evaluated regarding its corrosion potential and mechanical degradation under various chloride exposures. Moreover, all the tested specimens were retrieved from full-scale RC and SFRC lining segments. This simulates realistic field conditions rather than casting small-scale specimens in the laboratory. This study aims to compare the corrosion performance of full-scale RC and SFRC PCTL segments and analyze their service life under various chloride exposures. The findings can stimulate the implementation of more durable and sustainable precast concrete tunnel lining systems.

* A version of this chapter was submitted to the Construction and Building Materials Journal (2013).
6.2. DESCRIPTION OF FULL-SCALE TUNNEL LINING SEGMENTS AND MIXTURE DESIGN

The description of full-scale tunnel lining segments and their manufacturing process was mentioned in detail in Chapter 3 (Section 3.1). Table 3.1 shows the concrete mixture design for both the RC and SFRC segments.

6.3. SPECIMENS PREPARATION FOR CORROSION ASSESSMENT

6.3.1. Core Sampling

The coring process from full-scale conventional RC and SFRC lining segments was explained earlier (Section 3.3.2).

6.3.2. Saw Cut Beams

Small beams (Figure C.6) were cut from both the RC and SFRC PCTL segments in accordance with ASTM C42 (Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams for Concrete) (Figures 6.1 and C.7). Sawed faces of beams were further grinded to create plane, parallel and groove free surfaces. The RC beams were sawed in such a way that each specimen had identical reinforcing steel configuration. Furthermore, a concrete cover of 40 mm (1.57 in) was maintained in order to simulate the cover of full-scale PCTL segments. Both the RC and SFRC beams were cracked at three different locations according to recommendations of previous study (Granju and Balouch, 2005), in order to simulate realistic field scenarios and accelerate the corrosion process.

6.3.3. Steel Rebars

Short pieces of rebar were extracted from the reinforcement cages used in the full-scale RC PCTL segments. These steel rebar specimens were immersed in various chloride solutions (e.g. 3.5% and 10.0%) in order to simulate the field condition at which the concrete cover has been spalled and the steel rebar became directly exposed to the corrosive environment.

6.3.4. Bond Pull-out Specimens

Using the same concrete mixture and steel rebars used in the full-scale fabrication of RC PCTL segments, bond pull-out specimens were cast (Figure C.8). Concrete was poured into
100×200 mm (4×8 in) cylinders with a steel rebar having 15 mm (0.60 in) diameter at their centers. After concreting, the pull-out specimens were cured using the same curing method of the full-scale PCTL segments.

6.4. CORROSION PERFORMANCE ASSESSMENT PROGRAM

6.4.1. Salt Immersion Test Description

The salt immersion test was conducted according to the ASTM D870 (Standard Practice for Testing Water Resistance of Coatings Using Water Immersion) and ASTM G31-72 (Standard Practice for Laboratory Corrosion Testing of Metals) to simulate field conditions of tunnel linings exposed to corrosive environments (Figures C.9 and C.10). All extracted specimens were oven dried at 105 °C (221 °F) for 24 hours and subsequently cooled at room temperature (20 °C (68 °F)) for 24 hours before immersing in the salt solution. Two different chloride solutions were investigated: one representing the salinity level of underground water (or sea water) (i.e. 3.5% Cl–), and the other to create an accelerated effect (i.e. 10% Cl–) in agreement with previous studies (Guoping et al., 2011; Matsumura et al., 2008; NT BUILD 492, 1999). The cylindrical concrete cores, beams, bare steel rebars and specimens for concrete mass loss and bond testing were immersed in the chloride solutions and stored at 40 ± 1 °C (104 ± 2 °F) inside a walk-in environmental chamber (Figure C.11). The chloride solutions were changed monthly or whenever it got cloudy or colored. Weekly wetting and drying cycles for specimens were maintained to accelerate the corrosion process, similar to previous studies (Granju and Balouch, 2005; Hong, 1998; Graeff et al., 2009).

6.4.2. Testing and Measurements

The compressive and splitting tensile strengths were each measured on four 100×200 mm (4×8 in) cylindrical core specimens according to ASTM C39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens) and ASTM C496 (Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens), respectively. The flexural strength was measured on two 100×100×600 mm (4×4×24 in) beams according to ASTM C1609 (Standard Test Method for Flexural Performance of Fibre-Reinforced Concrete (Using Beam with Third-Point Loading)). Concrete mass loss was measured on five small discs of 10 mm (0.40 in) thickness and 100 mm (4 in) diameter, while rebar mass loss...
was measured using eight 15 mm (0.60 in) diameter and 300 mm (11.80 in) long rebar. The rebar mass loss values were further used to evaluate the rebar’s diameter loss using Eq. 6.1 (Francois et al., 2013).

\[
d = 100 \left[ 1 - \frac{\Delta M}{M} \right]
\]

Eq. 6.1

Where, \(d\) is the rebar’s diameter loss (%); \(M\) is the initial mass of the tested rebar (g) and \(\Delta M\) is the mass loss of steel rebar (g).

The average bond strength \((u)\) was evaluated using the following formula (Eq. 6.2).

\[
u = \frac{P}{\pi DL}
\]

Eq. 6.2

Where, \(P\) is the maximum pull-out load, \(D\) is the rebar diameter and \(L\) is the embedment length.

All specimens were taken out from the plastic containers after every 4 months of chloride exposure. Loose materials on the specimen’s surface were gently removed, cleaned and dried with paper towel. After one hour of drying, compressive strength, splitting tensile strength, flexural strength, bond strength and mass loss were measured. In addition, visual inspection including surface scaling, chipping and crack patterns for all specimens was conducted. Crack widths of corroded RC beam specimens were measured using an elcometer crack width ruler. Physical appearance was assessed based on the visual rating of the specimen’s surface conditions including edge conditions and scaling deterioration (Table 6.1) similar to previous study (Wang et al., 2006). The selected specimens were also examined for chloride contents according to FHWA-RD-72-12 (Sampling and Testing of Chloride Ion in Concrete) in order to determine the chloride diffusion coefficient. Furthermore, micro-structural analysis and energy dispersive X-ray analysis (EDX) were conducted on small fragments and thin polished sections from selected RC and SFRC specimens using a Hitachi S-4500 scanning electron microscope (SEM) (Figure C.12).

6.5. RESULTS AND DISCUSSION

After 8 months, RC specimens subjected to the low Cl\(^-\) concentration (i.e. 3.5%) only showed some surface distress and chipping, which is in agreement with previous studies (Wang et al.,
2006; Darwin et al., 2007). Conversely, at the high Cl− concentration (i.e. 10%), specimens exhibited severe surface degradation (Figure 6.2) along with damage of the embedded steel reinforcement. This deterioration can be ascribed to the formation of friedls salt (Figure 6.3) leading to expansion, cracking and spalling of concrete. Furthermore, the excessive formation of calcium chloride (CaCl₂) can increase the permeability of concrete since it leaches out calcium ions (Islam et al., 2005). This can be explained according the following chemical reactions (Eqs. 6.3 and 6.4) that take place during chloride attack on concrete specimens (Islam et al., 2005; Ben-Yair, 1974).

\[
Ca(OH)_2 + 2NaCl \rightarrow CaCl_2 + 2NaOH
\]  
\[CaCl_2 + 3CaO.Al_2O_3 + 10H_2O \rightarrow 3CaO.Al_2O_3.CaCl_2.10H_2O\]

The formation of chloro-aluminates (3CaO.Al₂O₃.CaCl₂.10H₂O) consumes chloride ions, mitigating its further penetration and attack of the embedded steel rebar reinforcement. Therefore, the stable growth of chloro-aluminates (Figure 6.4) tends to reduce the corrosion risk (Sanjuan, 1998). However, at high temperature (i.e. 40 °C (104 °F)), the phase transformation of chloro-aluminate from hexagonal plates (Figure 6.4) to cubical crystals (Figure 6.5) leads to internal distress and micro-cracking due to the associated expansive property (Sanjuan, 1997; 1998). Hence, the increased porosity and/or mass loss due to leaching-out of calcium ions and development of micro-cracks can provide easier access for chlorides to further penetrate to the steel reinforcement and accelerate the corrosion initiation process.

### 6.5.1. Visual Inspection and Crack Patterns

A deposition of white precipitation (salt material) (Figures 6.6 (a) and C.13) was observed on both the surfaces of RC and SFRC specimens after two weeks of wetting and drying cycles. More deposition material was observed for the 10% Cl− exposure condition compared to that of the 3.5% Cl− exposure due to increased chloride content. SFRC specimens showed various corrosion spots at the surface after one month of chloride exposure (Figure 6.6 (b)). However, no signs of corrosion and reduction in fibre diameter were observed for the embedded fibres (Figure 6.7). A substantial amount of micro-pores (Figure 6.2 (b)) was
observed at the surface of both the RC and SFRC specimens, leading to mass loss and other detrimental effects.

After flexure testing, a thorough visual inspection for each beam was conducted after desired periods of chloride exposure. Each face of the predefined crack locations was broken carefully and investigated with the help of a magnifying glass. After 4 months of exposure, the RC specimens showed no visible signs of corrosion at the surface of the steel reinforcement, except at the surface exposed rebar diameter (Figure 6.6 (a)) and the predefined crack regions for both the 3.5% and 10% Cl⁻ solutions. After 8 months of exposure, the steel reinforcement around the predefined crack locations was severely corroded, especially at the higher Cl⁻ concentration (i.e. 10%).

The corrosion of steel reinforcement in the RC specimens (Figure C.14) increased with the passage of time around the predefined crack region, leading to the development of internal stresses. Once these stresses exceeded the tensile capacity of concrete, splitting cracks initiated and mapped the location of rebars (Figures 6.8 and C.15). For instance, after 11 months of 10% Cl⁻ exposure, some discontinuous cracks less than 0.20 mm (0.0078 in) in width were observed along with surface deterioration. A significant increase in crack width was observed with increased exposure duration as shown in Table 6.2.

The development of cracks and progress in crack width were relatively slower for specimens exposed to the 3.5% Cl⁻ solution. For instance, cracks with an average width of around 0.30 mm (0.012 in) were observed mapping the rebar locations for the RC beam specimens after 16 months of 3.5% Cl⁻ exposure. Such crack formation is critical for accelerating the corrosion mechanism since it assists the penetration of oxygen, moisture and chloride ions to attack the reinforcing steel rebar, consequently jeopardizing the overall integrity of the structural system (Bertolini et al., 2004).

In the case of SFRC beams, some deposits of rust were observed on the predefined crack regions. All fibres intersecting the predefined cracks were severely corroded but no spalling or bursting occurred. Concentrated fibre corrosion spots were observed after 16 months of exposure at the specimen surface and on the predefined crack regions. The 10% Cl⁻ solution caused more corroded areas than that of the 3.5% Cl⁻ solution, which can be attributed to the
higher concentration of chloride ions. Furthermore, the external surfaces of SFRC specimens showed a brownish color after excessive formation of corrosion spots (Figure C.16), which can be visually unpleasant in large-scale applications. Figure 6.9 shows the average visual rating of RC and SFRC specimens based on Table 6.1 under various Cl$^-$ exposures. It can be observed that the 10% Cl$^-$ solution represented the worst exposure scenario and exhibited more physical degradation, especially for the RC specimens.

Furthermore, after testing each pull-out specimen, it was visually examined at various exposure ages. No signs of corrosion on the surface of the reinforcing steel rebar were observed after 4 months of exposure. For the 10% Cl$^-$ exposure, signs of corrosion were visible near the start of the protruded rebar length after 8 months. After 16 months, corrosion at the surface of rebars was observed for both exposure solutions. Specimens exposed to the 10% Cl$^-$ solution exhibited more significant rebar corrosion, especially at various rib areas after 16 months of exposure, compared to that of specimens exposed to the 3.5% Cl$^-$ solution.

6.5.2. Concrete Specimens Mass Change

Figure 6.10 shows the percentage mass loss over the exposure duration for both the RC and SFRC specimens subjected to the 3.5% and 10% Cl$^-$ exposures. Initially, an increase in mass was observed after 4 months of Cl$^-$ exposure for both the RC and SFRC specimens. For instance, 0.84% and 0.35% increase in mass was observed after 4 months of 3.5% Cl$^-$ exposure for the RC and SFRC specimens, respectively. This can be attributed to water adsorption and the filling of pores with salt crystals (Darwin et al., 2007; Santagata and Collepardi, 2000). Exposure to the 10% Cl$^-$ solution caused higher initial mass increase compared to that of the 3.5% exposure; likely due to increased crystallization into the pores resulting from the higher concentration of the Cl$^-$ solution.

A declining trend in mass was observed at later ages for both the RC and SFRC specimens. For example, 1.48% and 1.27% mass loss was observed for RC and SFRC specimens at 12 months of 10% Cl$^-$ exposure, respectively. Mass loss was attributed to the leaching-out of calcium ions due to excessive formation of calcium chloride (explained earlier) and formation of cubical crystals of chloro-aluminate, leading to internal distress, micro-cracking and higher porosity (Sanjuan, 1997; Santagata and Collepardi, 2000; Islam et al., 2005). Results indicated that SFRC specimens showed relatively smaller increase or
decrease in mass compared to that of the conventional RC specimens. This was attributed to the fact that steel fibres restrict the expansion of chloro-aluminate crystals leading to more stable behaviour. SFRC specimens exhibited almost constant mass after 12 months of exposure to both the 3.5% and 10% Cl⁻ solutions.

### 6.5.3. Rebar Specimens Mass and Diameter Loss

The mass loss of bare steel rebar increased as the percentage of chloride ions in the exposure solution and exposure period increased. Mass loss of 0.42%, 0.92%, 2.25% and 2.75% was observed after 4, 8, 12 and 16 months of 3.5% Cl⁻ exposure, respectively (Figure 6.11 (a)). A 10% Cl⁻ exposure led to around 24% higher mass loss than that of the 3.5% Cl⁻ exposure at all ages. Similar reduction trend in the rebar diameter was observed as shown in Figure 6.11 (b). Rebar mass and diameter loss due to corrosion decrease its yield and ultimate strengths, consequently reducing the service life of RC structures (Du et al., 2005; Apostolopoulos and Papadakis, 2008). Therefore, steel rebar should be protected against corrosion to achieve durable and sustainable RC PCTL.

### 6.5.4. Compressive Strength

Table 6.3 shows the compressive strength results versus chloride exposure periods for both the RC and SFRC specimens. Initially, an increase in compressive strength was observed for RC specimens under 3.5% and 10% Cl⁻ exposure. For instance, the compressive strength increased by 9% and 12% after 4 months of 3.5% and 10% Cl⁻ exposure, respectively. This can be attributed to the reduction in total porosity due to the filling effect of salt crystals into micro-pores, in agreement with previous studies (Minin, 1979; Salam et al., 2012).

A decreasing trend in compressive strength was observed at later ages depending on the Cl⁻ exposure. For example, about 19% and 28% reduction in compressive strength was observed after 12 months of 3.5% and 10% Cl⁻ exposure, respectively, compared to that of the control specimen. This can be attributed to the formation of expansive (chloro-aluminate crystals) and leachable (calcium ions) products. The chloride ions solution reacts with the hydration products of cement and supplementary cementitious materials, forming an expansive product according to the chemical reactions explained earlier. The formation of micro-cracks due to the stresses exerted by these expansive products results in weaker bond
between aggregates and the cementitious matrix, thus disturbing and rupturing the internal micro-structure (Minin, 1979). Hence, the internal and external degradation of concrete contributes to mass loss due to leaching-out of calcium ions and formation of cubical chloro-aluminate crystals, consequently reducing the mechanical strength (Mather, 1964; Islam et al., 2005). Exposure to the 10% Cl\(^-\) solution led to higher compressive strength reduction than that to the 3.5% solution due to higher formation of leachable and expansive materials, leading to increased mass loss, internal distress and porosity. Fan et al. (2006) reported similar findings and concluded that the higher the concentration of chlorides, the more will be the reduction in compressive strength of the concrete.

SFRC cylindrical core specimens showed a progressive increase in compression strength for both the 3.5% and 10% Cl\(^-\) exposure. For instance, the compressive strength increased by 22% and 33% after 12 months of 3.5% and 10% Cl\(^-\) exposure, respectively, compared to that of the control specimen. This was attributed to the self-healing of micro-pores with continuous hydration and rust products at the fibre surface (Figure 6.7). Similar increase in compressive strength for SFRC specimens under Cl\(^-\) exposure was reported in other studies (Graeff et al., 2009; Mangat and Gurusmany, 1987). In addition, the formation of expansive product and internal distress due to chemical reactions between the hydration products and Cl\(^-\) can be restricted by steel fibres, leading to decreased micro-cracking and porosity.

### 6.5.5. Splitting Tensile Strength

As the exposure period increased, the splitting tensile failure surfaces of RC specimens changed from matrix or aggregate fracture to an interfacial failure between the matrix and aggregates. This can be attributed to a reduction in the bond between the matrix and aggregates (Fan et al., 2012). Conversely, cylindrical SFRC specimens did not separate after failure due to fibre bridging. Table 6.3 shows the splitting tensile strength versus Cl\(^-\) exposure duration. For RC specimens, a slight increase in splitting tensile strength at early-age was observed for specimens exposed to the 3.5% and 10% Cl\(^-\) solutions. For example, 3% and 6% increase in tensile strength was observed after 4 months of exposure to the 3.5% and 10% solutions, respectively. This was ascribed to the continuous hydration of cementitious materials, leading to increased tensile strength (Minin, 1979; Salam et al., 2012).
The tensile strength of RC specimens showed a progressive decrease at longer periods of Cl\(^-\) exposure. For instance, the tensile strength decreased by 34% and 43% after 16 months of 3.5% and 10% Cl\(^-\) exposure, respectively, compared to that of the control specimen. The reduction in splitting tensile strength was attributed to the formation of expansive products, leading to increased stresses and micro-cracks.

On the other hand, no reduction in splitting tensile strength was observed for SFRC specimens under Cl\(^-\) exposure. Rather, a constant growth in tensile strength was observed for both the 3.5% and 10% Cl\(^-\) exposures. For example, an increase in tensile strength of up to 20% and 29% was observed after 16 months of 3.5% and 10% Cl\(^-\) exposure, respectively. Steel fibres provide a barrier against the penetration of Cl\(^-\) through micro-cracks deeper into the SFRC specimen and restrict the formation of expansive products. Therefore, the splitting tensile strength increase was ascribed to the filling of micro-pores with continuous cementitious hydration products, leading to strengthening of the fibre-matrix bond.

### 6.5.6. Flexural Strength

RC beam specimens showed a reduction in flexural capacity along with increasing Cl\(^-\) exposure duration and concentration. For example, about 55% and 70% reduction in flexural strength was observed after 16 months of exposure to the 3.5% and 10% Cl\(^-\) solutions compared to that of the control specimen, respectively. This can be ascribed to the formation of loose corrosion/rust products around the steel reinforcement, reducing the rebar-matrix bond strength and consequently decreasing the flexural strength (Salam et al., 2012).

Conversely, SFRC beams showed an increase in flexural capacity under Cl\(^-\) exposure. For instance, an increase in flexural strength of 11% and 38% was observed after 8 months of exposure to the 3.5% and 10% Cl\(^-\) solutions compared to that of the control beam specimen, respectively. This flexural strength increase can be attributed to self-healing at the predefined crack locations resulting from filling of corrosion products into the micro-pores (Figure 6.7) (Mangat and Gurusmany, 1985; 1987). Moreover, at the predefined crack regions, slight corrosion of fibres increased their bond strength due to increased friction between the fibres and the cementitious matrix, leading to higher flexural strength (Figure 6.12). Similar findings were reported elsewhere (Granju and Balouch, 2005). It can be concluded that the chloride environment does not affect the crack bridging phenomena of steel fibres.
6.5.7. Bond Strength

Figure 6.13 shows the correlation between the steel rebar bond strength and chloride exposure duration. A slight increase in bond strength of 2% and 4% was observed after 4 months of exposure to the 3.5% and 10% Cl⁻ solutions compared to that of the control specimens, respectively. This was attributed to the increase in concrete strength due to the filling of micro-pores with hydration products and salt crystals. However, no signs of corrosion were observed on the rebar surface, while the Cl⁻ content (by weight of concrete) was below the corrosion threshold value of 0.2% (Angst et al., 2009). Further, an increase in rebar bond strength by 6% and 12% was observed after 8 months of exposure to the 3.5% and 10% Cl⁻ solutions, respectively. This was attributed to the formation of initial corrosion products at the rebar surface (Saether, 2011), which provides a rougher surface, thus leading to increased friction and enhanced bond properties (Figure 6.14 (a)).

However, a decline in bond strength was observed at later ages of chloride exposure. For instance, 32% and 50% decrease in bond strength was observed after 16 months of exposure to the 3.5% and 10% Cl⁻ solutions, respectively. This reduction in bond strength was attributed to increased rebar corrosion, leading to reduced contact between the reinforcing steel rebar and the concrete due to the formation of loose rust products. Furthermore, the loss of rebar rib area and formation of small rust pits at various locations due to corrosion can contribute towards bond loss (Figure 6.14 (b)) (Saether, 2011).

6.6. SERVICE LIFE PREDICTION FOR TUNNEL LINING SEGMENTS

The service life of structures (or structural members) exposed to chloride corrosive environment is the sum of corrosion initiation and propagation periods (Browne, 1980; Tuutti, 1982). The initiation period ($t_i$) and propagation period ($t_p$) can be estimated using the following formulas (Eqs. 6.5 and 6.6) (Polder, 1996).

$$t_i = \frac{L^2}{A \cdot D}$$  \hspace{1cm}  \text{Eq. 6.5}

$$t_p = 12.5 \times 10^{-3} f_t \rho_{\text{concrete}}$$  \hspace{1cm}  \text{Eq. 6.6}

Where, $L$ is the concrete cover (m); $A$ is a constant depending on the surface and critical values of chloride contents; $D$ is the diffusion coefficient (m$^2$/sec) determined using Crank’s
solution of Fick’s second law (Crank, 1975); $\rho_{\text{concrete}}$ is the concrete resistivity (ohm-m); $f_t$ is the tensile strength of concrete (N/mm²).

The predictions of service life for the RC and SFRC PCTL segments based on the measured chloride diffusion coefficients and concrete resistivity are shown in Table 6.4. It was found that SFRC PCTL segments can exhibit longer service life than that of conventional RC segments. Indeed, SFRC segments showed 25 years more potential service life than that of the RC segments when subjected to the 3.5% Cl⁻ exposure. Furthermore, exposure to higher concentration of Cl⁻ reduces the service life of the PCTL segments. For example, SFRC segments exhibited 33 years shorter service life when subjected to the 10% Cl⁻ exposure compared to that of the 3.5% Cl⁻ exposure.

It should be noted that the service life calculation does not account for the fact that damage mechanics due to corrosion are quite different in RC and SFRC. Service life was also predicted using the Life365 software by considering the same concrete mixture design used for fabricating the tested lining segments and assuming an urban structure in the Toronto region environmental exposure. The Life365 software predicted approximately 48 years of service life, which is comparable to that of the tested RC lining segment exposed to the 3.5% Cl⁻ solution (Table 6.4).

6.7. SUMMARY

A comparative study of the corrosion resistance of specimens extracted from full-scale RC and SFRC PCTL segments was conducted. It was observed that the mechanical properties of RC specimens were adversely affected by long-term Cl⁻ exposure. However, SFRC specimens did not show any reduction in mechanical properties, yet their visual appearance was affected due to the surface corrosion of steel fibres. The following specific conclusions can also be drawn from this study:

1) An increase in concrete mass by 0.84% and 0.35% was observed after 4 months of 3.5% Cl⁻ exposure for RC and SFRC specimens, respectively, due to water adsorption and filling of micro-pores with salt crystals. However, after 8 months of Cl⁻ exposure, a decline in mass was observed for both the RC and SFRC specimens due to
increased porosity and mass loss as a result of leaching-out of calcium ions and formation of cubical chloro-aluminate crystals.

2) The bare steel rebar results show that the rebar mass loss increased as the concentration of chloride ions and exposure period increased.

3) Initially, an increase in compressive, tensile, flexural and rebar bond strengths was observed for RC specimens for both the 3.5% and 10% Cl\textsuperscript{−} exposures due to the advancing hydration of cementitious materials and filling of micro-pores with salt crystals. However, after 8 months of Cl\textsuperscript{−} exposure, a reduction in mechanical strength properties was observed for RC specimens due to internal and external concrete degradation as a result of leachable and expansive products formation. Conversely, SFRC specimens showed a progressive increase in mechanical strength properties with increased Cl\textsuperscript{−} concentration and exposure duration.

4) The service life of RC and SFRC PCTL segments was estimated based on the chloride diffusion coefficient and concrete resistivity properties. The predicted service life values were comparable to that predicted by the Life 365 software. It was found that SFRC segments can exhibit higher service life when subjected to corrosive environments compared to that of similar RC segments. Moreover, exposure to higher Cl\textsuperscript{−} concentration was found to cause more drastic effects on the service life of PCTL segments.
6.8. REFERENCES


### Table 6.1 – Specimen’s visual rating after chloride exposure

<table>
<thead>
<tr>
<th>Visual rating</th>
<th>RC specimens</th>
<th>SFRC specimens</th>
</tr>
</thead>
<tbody>
<tr>
<td>0</td>
<td>No scaling or chipping</td>
<td>No visible corrosion spots</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>1</td>
<td>Minor scaling, small chipping ≤ 50 mm$^2$ (0.08 in$^2$), noticeable on specimen’s surface</td>
<td>Some corrosion spots are visible on surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2</td>
<td>Minor to modest scaling, flakes &gt; 50 mm$^2$ (0.08 in$^2$) and specimen’s edge damage, no cracks were developed.</td>
<td>Corrosion spots are visible over the entire surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3</td>
<td>Modest/intermediate scaling, scaling on edges, some discontinuous cracks &lt; 0.10 mm (0.004 in) were noticed around the rebar locations.</td>
<td>Entire longitudinal oriented fibres are corroded on the specimen’s surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>4</td>
<td>Intermediate to intense scaling, coarse aggregates are visible at some locations, 0.15 mm (0.006 in) &lt; crack widths &lt; 0.25 mm (0.010 in) were observed mapping the position of rebars.</td>
<td>Loose corroded materials are deposited at the surface and brownish color appeared on the entire surface</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5</td>
<td>Intense scaling, lumps pulled out from specimen’s surfaces and edges, scaling depth &gt; 1.75 mm (0.07 in), scaling area ≥ 125 mm$^2$ (0.20 in$^2$), coarse aggregates near entire edge surfaces are noticeable, crack widths &gt; 1 mm (0.04 in) were observed.</td>
<td>Surface and edge scaling</td>
</tr>
</tbody>
</table>

### Table 6.2 – Crack width due to rebar corrosion in RC beam specimens

<table>
<thead>
<tr>
<th>Exposure duration (months)</th>
<th>Average crack width, mm (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.5% CI</td>
</tr>
<tr>
<td>8</td>
<td>-</td>
</tr>
<tr>
<td>9</td>
<td>-</td>
</tr>
<tr>
<td>10</td>
<td>-</td>
</tr>
<tr>
<td>11</td>
<td>-</td>
</tr>
<tr>
<td>12</td>
<td>&lt; 0.10 (0.004)</td>
</tr>
<tr>
<td>13</td>
<td>0.15 (0.006)</td>
</tr>
<tr>
<td>14</td>
<td>0.18 (0.007)</td>
</tr>
<tr>
<td>15</td>
<td>0.25 (0.010)</td>
</tr>
<tr>
<td>16</td>
<td>0.30 (0.012)</td>
</tr>
</tbody>
</table>
### Table 6.3 – Mechanical properties of RC and SFRC specimens

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>Chloride exposure</th>
<th>Duration (months)</th>
<th>Relative strength ratio ($R$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Compressive strength</td>
</tr>
<tr>
<td></td>
<td></td>
<td>0</td>
<td>1.00</td>
</tr>
<tr>
<td>RC</td>
<td>3.5%</td>
<td>4</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1.01</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>0.79</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>0.71</td>
</tr>
<tr>
<td></td>
<td>10.0%</td>
<td>4</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>0.72</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>0.67</td>
</tr>
<tr>
<td>SFRC</td>
<td>3.5%</td>
<td>4</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1.12</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>1.22</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1.27</td>
</tr>
<tr>
<td></td>
<td>10.0%</td>
<td>4</td>
<td>1.11</td>
</tr>
<tr>
<td></td>
<td></td>
<td>8</td>
<td>1.15</td>
</tr>
<tr>
<td></td>
<td></td>
<td>12</td>
<td>1.33</td>
</tr>
<tr>
<td></td>
<td></td>
<td>16</td>
<td>1.34</td>
</tr>
</tbody>
</table>

$R = (R_c/R_o)$; $R_o$ is the strength without chloride exposure (at 0 month) and $R_c$ is the strength after chloride exposure at various ages.

### Table 6.4 – Service life prediction for RC and SFRC PCTL segments

<table>
<thead>
<tr>
<th>Segment type</th>
<th>RC</th>
<th>SFRC</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>3.5%</td>
<td>10.0%</td>
</tr>
<tr>
<td>$f_t$ (MPa)</td>
<td>7.50</td>
<td>7.50</td>
</tr>
<tr>
<td>$D$ (×10^{-12} m^2/sec)</td>
<td>3.01</td>
<td>4.37</td>
</tr>
<tr>
<td>$\rho_{concrete}$ (ohm-m)</td>
<td>406.13</td>
<td>228.36</td>
</tr>
<tr>
<td>$t_i$ (years)</td>
<td>6.45</td>
<td>4.44</td>
</tr>
<tr>
<td>$t_p$ (years)</td>
<td>38.07</td>
<td>21.41</td>
</tr>
<tr>
<td>$t$ (years)</td>
<td>44.52</td>
<td>25.85</td>
</tr>
</tbody>
</table>

1 MPa = 0.145 ksi
1 m^2/sec = 10.764 ft^2/sec
Figure 6.1 – Cutting of beams from full-scale PCTL segments.

a) Surface degradation  
b) Micro-pores at the surface

Figure 6.2 – Surface degradation of concrete cylinder after 12 months of Cl⁻ exposure.
a) Friedel’s salt compound

b) EDX analysis at “E” point

**Figure 6.3 – SEM image for Friedel’s salt compound.**

a) Hexagonal plates of chloro-aluminate

b) EDX analysis at point “H”

**Figure 6.4 – SEM image of hexagonal plates of chloro-aluminate.**
a) Cubical crystals of chloro-aluminate

b) EDX analysis at “C” point

Figure 6.5 – SEM image of cubical crystals of chloro-aluminate.

a) RC beam specimen

b) SFRC beam specimen

Figure 6.6 – Visual appearance of beam specimens exposed to 10% Cl⁻ solution.
a) Steel fibres and deposition of corroded material

Figure 6.7 – SEM image of SFRC specimen exposed to 10% Cl⁻ solution.

b) EDX at “E” point

Figure 6.8 – Cracks in RC beam specimens after exposure to 10% Cl⁻ solution.
Figure 6.9 – Visual rating of RC and SFRC specimens exposed to 3.5% and 10% Cl$^-$ solutions.

Figure 6.10 – Mass loss of RC and SFRC specimens exposed to 3.5% and 10% Cl$^-$ solutions.
**Figure 6.11** – Mass and diameter loss of bare steel rebar exposed to 3.5% and 10% Cl⁻ solutions.

**Figure 6.12** – Optical microscopic images of fibre corrosion in SFRC specimen after 16 months of 10% Cl⁻ exposure.
**Figure 6.13** – Effect of exposure to 3.5% and 10% Cl⁻ solutions on bond strength of steel rebar to cementitious matrix.

**Figure 6.14** – Rebar corrosion in bond pull-out specimens after exposure to various chloride solutions.
Chapter 7

EFFECT OF STEEL FIBRE LENGTH AND DOSAGE ON MECHANICAL AND DURABILITY PROPERTIES OF ULTRA-HIGH PERFORMANCE CONCRETE*

Ultra-high performance concrete (UHPC) is a new generation of steel fibre-reinforced concrete with superior mechanical and durability properties. However, limited data is available on the influence of the steel fibre length and dosage on UHPC mechanical and durability performance. Therefore, in this chapter, a number of UHPC mixtures with varying steel fibre lengths (8 mm (0.31 in), 12 mm (0.47 in) and 16 mm (0.62 in)) and dosages (1%, 3% and 6% by mixture volume) were tested. Mechanical properties of UHPC including compressive, splitting tensile and flexural strengths were assessed. Moreover, its resistance to chloride ions penetration and mechanical degradation under various chloride exposures (i.e. 3.5% and 10%) was evaluated.

7.1. RESEARCH SIGNIFICANCE

Achieving greater awareness of the mechanical and durability properties of UHPC amongst stakeholders in the construction industry is paramount for its wider implementation. Various researchers studied the mechanical and durability properties of UHPC. Yet, the effect of the steel fibre length and dosage on such properties has not been fully investigated. Furthermore, no previous research has evaluated the durability of UHPC under highly corrosive environments. The main goal of this study is to enhance the existing knowledge regarding the structural and durability properties of UHPC with various steel fibre lengths and dosages cured under similar regime to that used in precast plants. The results should assist in producing precast UHPC structural elements with superior mechanical, durability and sustainability features.

* A version of this chapter was submitted to the Materials and Design Journal (2013).
7.2. EXPERIMENTAL PROGRAM

7.2.1. Material Composition and Proportions

The used fibres were copper coated steel having a constant diameter (0.2 mm (0.0078 in)) and tensile strength greater than 2850 MPa (413.35 ksi). The tested UHPC mixtures consist of portland cement, silica fume, quartz powder, quartz sand and a polycarboxylate based super-plasticizer in addition to steel fibres. The physical and chemical properties of these materials are summarized in Table 7.1. A typical UHPC mixture composition is shown in Table 7.2.

7.2.2. Mixing of UHPC Constituents

A high shear pan mixer (Figure 7.1) with 120 L (32 gal) capacity was used for mixing the UHPC constituents. Quartz sand and silica fume were dry mixed for 3 to 5 minutes. Thereafter, cement and quartz powder were added and mixing resumed for another 3 minutes. Half of the superplasticizer (SP) was mixed with the mixing water and added gradually to the dry mixture and mixing continued for another 3 minutes. Afterwards, the other half of SP was added over 3 minutes of mixing. At the end, steel fibres were added and mixing continued until fibres were fully dispersed. Figure D.1 shows the inside view of mixer during mixing process of UHPC.

7.2.3. Specimen Preparation and Environmental Conditions

UHPC cylindrical and beam specimens were immediately cast after the end of the mixing process. All UHPC specimens were cast in three layers. Each layer was compacted or consolidated using a vibrating table. Afterwards, specimens were transported to an environmental chamber at 45 °C (113 °F) and relative humidity (RH) > 95%. After 5 hours, molds were stripped and UHPC specimens (cylinders and beams) were stored in a curing room at RH ≥ 95% and 20 °C (68 °F) for 5 days. Subsequently, all specimens (beams and cylinders) were stored at ambient laboratory conditions (i.e. 20 ± 2 °C (68 ± 4 °F)). This curing regime was selected to realistically resemble that in industrial precast plants (Recchia et al., 2012).
7.2.4. Mechanical Testing

The compressive strength for the various UHPC mixtures was determined in accordance with ASTM C39 (Standard Test Method for Compressive Strength of Cylindrical Concrete Specimens). The loading rate was 1 MPa/sec (145 psi/sec) in agreement with previous work (Graybeal, 2006). For all tested specimens, the stress and corresponding strain under compression load was recorded. The UHPC stress-strain curve showed almost linear behaviour up to the peak compressive stress (Graybeal, 2006; Hassan et al., 2012). Therefore, the initial slope of the stress-strain curve (up to 85% of maximum stress) was used to evaluate the modulus of elasticity.

The splitting tensile strength was determined according to ASTM C496 (Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens). A displacement control load at a rate of 0.025 mm/min (0.001 in/min) was applied on the UHPC specimens in agreement with previous study (Magureanu et al., 2012).

The flexural performance of UHPC specimens was conducted according to ASTM C1609 (Standard Test Method for Flexural Performance of Fibre-Reinforced Concrete (Using Beam With Third-Point Loading)). A controlled displacement load was applied at a rate of 0.05 mm/min (0.002 in/min) in agreement with previous work (Kazemi and Lubell, 2012). The definitions of first crack and peak loads were mentioned earlier (Chapter 3). Crack width at various loading stages was measured using a crack width ruler (Figure D.2). Tables 7.3-7.5 show the number of UHPC specimens, sizes and coefficient of variance (COV) for the various tests conducted.

7.2.5. Durability Testing

The volume of permeable voids for the UHPC mixtures incorporating various steel fibre dosages (1%, 3% and 6% by mixture volume) were tested in accordance with ASTM C642 (Standard Test Method for Density, Absorption, and Voids in Hardened Concrete). The sorptivity test was performed according to the ASTM C1585 (Standard Test Method for Measurement of Rate of Absorption of Water by Hydraulic Cement Concretes). The rapid chloride ion penetrability test was conducted on UHPC specimens in accordance with ASTM C1202 (Standard Test Method for Electrical Indication of Concrete’s Ability to Resist
Chloride Ion Penetration). The salt ponding test was conducted on UHPC specimens (Figure D.3) according to AASHTO T259 (Standard Method of Test for Resistance of Concrete to Chloride ion Penetration). The salt immersion test was performed (Figure D.4) over six months in accordance with ASTM D870 (Standard Practice for Testing Water Resistance of Coatings using Water Immersion) and ASTM G31-72 (Standard Practice for Laboratory Corrosion Testing of Metals). The sample preparation and details of these tests were mentioned earlier (Sections 5.3 and 6.4).

7.2.6. Micro-structural Analysis

Micro-structural analysis was conducted on thin polished sections and small fragments from selected UHPC specimens using a Hitachi S-4500 scanning electron microscope (SEM). The pore size distribution was examined using a Micromeritics AutoPore IV 9500 Series porosimeter for pressure range up to 414 MPa (60000 psi).

7.3. RESULTS AND DISCUSSION

7.3.1. Fresh Properties

It was observed that, within the range of parameters tested, the steel fibre length and dosage have minimum effect on the flowability of the tested UHPC mixtures (Table 7.6). For instance, only 13% reduction in UHPC flowability was observed for mixtures incorporating 6% by mixture volume of 16 mm (0.62 in) fibres compared to that of the mixture without fibre addition. The temperature of the freshly mixed UHPC mixtures ranged between 24 and 28 °C (75 and 82 °F), in agreement with previous studies (Kazemi and Lubell, 2012; Magureanu et al., 2012).

7.3.2. Compressive Strength and Modulus of Elasticity

Figure 7.2 (a) shows typical 28 days compressive stress-strain curve for UHPC incorporating 8 mm (0.31 in) steel fibres at various dosages (1%, 3% and 6% by mixture volume). The initial slope of stress-strain curves for UHPC mixtures with various steel fibre dosages were comparable. Therefore, the compressive stress-strain curves were horizontally offset (Figure 7.2 (a)) in order to avoid their intersection, similar to previous study (Kazemi and Lubell, 2012). The mixture without steel fibre showed a brittle failure with an abrupt drop in load carrying capacity. However, mixtures incorporating steel fibres exhibited
somewhat more ductile behaviour with steadier drop in load carrying capacity (Figure 7.2 (a)). This was attributed to restricting lateral expansion by steel fibres, leading to higher tolerance for axial deformation (Kazemi and Lubell, 2012). The slope of the descending branch of the UHPC compressive stress-strain curves (Figure 7.2 (a)) decreased with higher dosage of steel fibres.

Table 7.6 shows the compressive strength results for the various UHPC mixtures. The 7 days compressive strength and modulus of elasticity were 131 MPa (19 ksi) and 35.6 GPa (5163 ksi) respectively, for the UHPC mixture without fibres. However, after 28 days, the compressive strength and modulus of elasticity increased to 151 MPa (22 ksi) and 42.6 GPa (6179 ksi), respectively. This progression of UHPC compressive strength and modulus of elasticity continued at later age. For instance, an increase in UHPC compressive strength and modulus of elasticity by 6% and 8%, respectively, was observed at 56 days compared to that at 28 days. This was attributed to hydration reaction of pozzolanic cementitious materials at later ages (El-Dieb, 2009).

An increase in compressive strength of 3%, 9% and 13% was observed for mixtures incorporating 1%, 3% and 6% of 8 mm (0.31 in) steel fibres, respectively, compared to that of the mixture without steel fibres. This increase in compressive strength due to steel fibre addition is in agreement with other studies (Bonneau et al., 1997; Soutsos et al., 2005; Lee and Chisholm, 2005). The homogenously distributed fibres restrict the internal material deterioration and crack propagation by absorbing the developed stresses at the fibre’s tip and consequently enhanced compressive strength (Orgass and Klug, 2004). It should be understood that fibre reinforcement is primarily used to enhance the tensile behaviour and toughness characteristics of cementitious systems, and not to increase compressive strength. However, since very high compressive strength is a key feature of UHPC, the improved effect of fibres on compressive strength merits further investigation.

No significant effect of the fibre length on UHPC compressive strength was observed (Table 7.6). For instance, increasing the fibre length from 8 mm (0.31 in) to 16 mm (0.62 in) had induced difference less than 2%, regardless of the fibre dosage.
Furthermore, fibre length and dosage had a minimum effect on the modulus of elasticity of UHPC (i.e. < 2%). It was observed that the cylinder specimens without fibres were severely damaged through a sudden and explosive behaviour, similar to previous work (Hassan et al., 2012). However, specimens incorporating steel fibres did not show any splitting or breakage into pieces after failure (Figure 7.2 (b)). This was ascribed to the effect of the fibres, which did not allow the concrete to explode or break into pieces (El-Dieb, 2009).

### 7.3.3. Splitting Tensile Strength

As expected, the splitting tensile strength increased with the curing time (Table 7.7). The splitting tensile strength of UHPC was considerably affected by the steel fibre addition (Table 7.7). For instance, a 48% increase in the 28-days splitting tensile strength was observed for the mixture incorporating 1% of the 16 mm (0.62 in) steel fibre compared to that of the control mixture without fibres. At a higher fibre dosage of 6%, the 28 days splitting tensile strength was approximately 4 times greater than that of the control mixture. Furthermore, it was observed that the fibre length considerably influenced the splitting tensile strength. For example, the mixture incorporating 3% of the 8 mm (0.31 in) steel fibre showed 16% increase in the 28 days splitting tensile strength compared to that of a similar mixture incorporating the same dosage of the 16 mm (0.62) steel fibre. It was observed that cylinder specimens including steel fibres did not split into two parts due to crack bridging action of fibres and the dense interface between steel fibres and the concrete matrix.

### 7.3.4. Flexural Strength

The flexural behaviour of the UHPC beam specimens incorporating steel fibres and that of the control specimens without fibres were significantly different. For instance, a 37% increase in the peak load was observed for the beam specimens incorporating 1% of the 16 mm (0.62 in) fibres compared to that of the control specimens (Table 7.8). Furthermore, the beam specimens without fibre addition had a brittle failure and sudden drop in load carrying capacity after reaching the peak load.
7.3.4.1. Load-Deflection Behaviour

The initial linear elastic phase (Portion 0C in Figure 7.3 (a)) for the UHPC beams incorporating short fibres (8 mm (0.31 in)) and long fibres (16 mm (0.62 in)) was comparable for all UHPC specimens. However, the short 8 mm (0.31 in) fibres better improved the strain hardening behaviour (Portion CP in Figure 7.3 (a)). It is believed at the same fibre dosage, short fibres better captured the growth of micro-cracks, yielded more stable multiple micro-crack formation (Sheng, 1995; Magureanu et al., 2012).

UHPC beam specimens incorporating short fibres exhibited larger cracking and ultimate deflections compared to that of beam specimens with long fibres (Figure 7.3 (a)). For example, an increase in cracking and ultimate deflections by 27% and 31%, respectively was observed for beams incorporating 3% of 8 mm (0.31 in) fibres compared to that of similar specimens with 16 mm (0.62 in) fibres. It seems that the development of multiple micro-cracks delayed the formation of macro-cracks and led to higher first crack and peak load carrying capacities for the beam specimens incorporating short fibres. For instance, beam specimens incorporating 3% of 8 mm (0.31 in) fibres exhibited 30% and 26% higher first crack and peak loads respectively, compared to that of the beam specimens with 3% of 16 mm (0.62 in) fibres.

The difference between the behaviour of mixtures incorporating short and long fibres increased at higher fibre dosage. For example, beams incorporating 6% of the 8 mm (0.31 in) fibre exhibited 42% and 34% increase in first crack and peak loads, respectively, compared to that of the similar beams with 16 mm (0.62 in) fibres. Higher cracking and peak load capacities of beams incorporating short fibres was attributed to the fibre pinching force applied at the crack tips, which suppressed the propagation and development of cracks (Sheng, 1995; El-Dieb, 2009; Hassan et al., 2012). Furthermore, at the same dosage, more short fibres are available for crack bridging compared to that of the longer fibres, thus increased the load carrying capacity (Sheng, 1995). The increased crack load capacity of UHPC beams incorporating short fibres is beneficial against the penetration of aggressive materials responsible for corrosion of steel fibres.

The length of the softening phase for short fibres (Portion PF in Figure 7.3 (a)) was smaller and exhibited a steeper drop in load carrying capacity since short fibres can pull-out
or de-bonded relatively easier from the matrix because of smaller embedment length. Conversely, the beam specimens incorporating longer fibres (16 mm (0.62 in)) exhibited improved strain softening (post peak) behaviour (Figure 7.3 (a)). Long fibres especially at low fibre concentration (i.e. 1% or 3%) have relatively larger inter-fibre spacing; therefore, they became effective only after the development of macro-cracks. Furthermore, the load-deflection curves for beam specimens incorporating the long fibres exhibited steadier drop in load carrying capacity after the peak load compared to steeper load drop in the case of short fibres. This is due to the increased energy required for de-bonding or pull-out of the longer steel fibres.

Moreover, it was observed that the first crack load and peak load increased with higher fibre dosage. For instance, 51% and 64% increase in the first crack and peak loads were observed for beam specimens incorporating 3% of the 12 mm (0.47 in) steel fibres, respectively, compared to that of similar beam specimens with 1% fibre dosage. At higher dosage, fibres are closely spaced, providing more effective and localized control for the growth of micro-cracks into macro-cracks (Sheng, 1995; El-Dieb, 2009; Kang et al., 2010; Park et al., 2012). This delay of macro-cracks propagation increases the ultimate load carrying capacity.

It was observed that the initial stiffness of the load-deflection curves of UHPC beams was not affected by the fibre dosage (Figure 7.3 (a)). The deflection at crack and ultimate loads increased with higher fibre dosage. For example, 38% and 67% increase in deflection at crack and ultimate loads, respectively, were observed for beam specimens incorporating 6% of the 12 mm (0.47 in) fibres compared to that of similar specimen with 1% of the 12 mm (0.47 in) fibres (Table 7.8).

7.3.4.2. **Toughness**

The toughness of UHPC beams incorporating various fibre lengths and dosages was calculated by estimating the area under the load mid-span deflection curve using ASTM C1609. It was observed that beam specimens incorporating the long 16 mm (0.62 in) fibres exhibited higher toughness compared to that of identical beams with short fibres. For instance, a 20% increase in toughness was observed for the beams incorporating 3% of the 16 mm (0.62 in) fibres compared to that of similar beams with the 8 mm (0.31) fibres. This can
be ascribed to improved post-peak (strain softening) behaviour of the beam specimens incorporating the longer fibres. Moreover, the steel fibre dosage considerably influenced the toughness of UHPC beam specimens. For instance, beam specimens incorporating 3% and 6% of the 16 mm (0.62 in) fibres showed approximately 2.0 and 3.5 times higher toughness than that of beam specimen with 1%, respectively, regardless of the fibre length (Table 7.8).

7.3.4.3. Crack and Failure Pattern

As expected, no cracks were observed during the initial linear elastic loading (Portion 0C in Figure 7.3 (a)). At the end of the linear elastic phase, cracks were initiated in the form of various hairline micro-cracks, barely visible with the naked eye. New micro-cracks in between the previously developed cracks were observed as the load increased. Most of these cracks continued to develop in both directions. Closely spaced cracks developed perpendicular to the flexural stresses on the beam specimens (Figure 7.4 (a)), demonstrating the ability of fibre-reinforced UHPC to redistribute the flexural stresses through several micro-cracks until fibre pull-out or fracture occur (El-Dieb, 2009; Yang et al., 2010).

Fibre pull-out from the matrix began as the stress carried by individual fibres exceeded the ability of the UHPC matrix to hold the fibre (Yang et al., 2010). The pulled-out of fibres increased the stresses in neighboring fibres across cracks, pulling-out more fibres from the matrix. As the load increased on the beam specimens, stresses continued to increase with more formation of cracks (Kang et al., 2010; Park et al., 2012).

At the ultimate load, the highly stressed fibres started pulling-out from the matrix at one particular cross-section, which represented the weakest point (Figures 7.4 (a) and D.5). This was attributed to the localization of maximum strain higher than the strain capacity of concrete matrix (Hassan et al., 2012). Afterwards, the softening branch began with steadier or steeper drop in the load carrying capacity depending on the fibre length and dosage. The failure of UHPC beam specimens was characterized by a local failure of the fibre-matrix bond with significant increase in crack width (Table 7.9).

The crack width after first crack, at peak load and failure load decreased with higher dosage of fibres (Table 7.9). For instance, the average crack width after first crack, at peak load and failure load decreased by 81%, 71% and 50% for beam specimens incorporating 6%
of 8 mm (0.31 in) fibres compared to that of beam specimens with 1% of 8 mm (0.31 in) fibres. Furthermore, a decrease in crack width was observed for beam specimens incorporating short fibres compared to that similar beam specimens made with a similar dosage of the long fibres (Table 7.9).

The average spacing between cracks decreased for beam specimens incorporating short 8 mm (0.31 in) fibres in comparison with that in beam specimens made with longer 16 mm (0.62 in) fibres as shown in Figure 7.5. A similar decreasing trend in average crack spacing was observed at higher fibre dosage compared to that at lower fibre dosage (Figure 7.5).

Visual inspection of the failure surfaces of beam specimens revealed that fibre pull-out was dominant compared to fibre fracture (Figure 7.4 (b)). The fracture surfaces of beam specimens were also analyzed under SEM, indicating very dense micro-structure (Figure 7.6 (a)) and interface between aggregates and the cementitious materials (Figure 7.6 (b)) and intimate contact between the fibre and the matrix (Figure 7.7). This confirms the ability of fibre-reinforced UHPC to transfer stresses through cracks and achieve enhanced toughness (Orgass and Klug, 2004; El-Dieb, 2009).

7.3.5. Permeable Voids

The volume of permeable voids (VPV) in UHPC was less than 4% (Table 7.10), which is lower than the high strength concrete (i.e. 14%) (Roque et al., 2009). This is attributed to the dense micro-structure of UHPC and the improved interfacial zone between the cementitious matrix and aggregates (Figure 7.6). There was no significant influence of steel fibre length on the VPV (Table 7.10). However, the steel fibre dosage in UHPC mixtures reduces the VPV leading to improved durability properties. For instance, 12% and 36% decrease in VPV was observed for the mixtures incorporating 3% and 6% of steel fibres, respectively compared to that of the control mixture without fibres. The steel fibre addition disturbs the continuity of capillary pores and reduced the VPV (Banthia and Bhargava, 2007; Roque et al., 2009). This was confirmed through MIP analysis (Figure 7.8). For instance, UHPC mixtures incorporating 3% of the 8 mm (0.31 in) steel fibre had 17% reduction in the total porosity compared to that of the control UHPC mixture without fibre addition. The lower capillary pore volume in fibre-reinforced UHPC (Figure 7.8) can reduce the capillary suction.
and intrusion of aggressive solutions (e.g. chlorides), leading to improved durability properties (Teichmann and Schmidt, 2004).

7.3.6. Sorptivity Coefficients

Table 7.10 shows the initial and secondary sorptivity coefficient results for UHPC specimens. It was observed that the fibre length had no appreciable effect on the sorptivity coefficients. However, the initial and secondary sorptivity coefficients decreased by 7% and 12%, respectively, due to the addition of 1% of the 8 mm (0.31) steel fibres. This difference was more evident at higher dosage of steel fibres. For instance, the initial and secondary sorptivity coefficients decreased by 26% and 37%, respectively for UHPC mixtures incorporating 6% of the 8 mm (0.31 in) steel fibres compared to that of the control mixture without fibres. This indicates that the addition of steel fibres resulted in relatively less connected pores, leading to denser microstructure and consequently enhanced durability properties. This was verified through MIP analysis (explained earlier). Figure 7.9 (a) shows a typical sorptivity curve for the UHPC mixture incorporating 3% of the 8 mm (0.31 in) steel fibres.

7.3.7. Electrical Resistance of Chloride Ions

The short and dispersed steel fibres did not cause a short circuiting problem during the rapid chloride ion penetrability testing (RCPT) of UHPC specimens. There was also no significant rise in temperature due to electrical heating of the tested UHPC specimens. This is in agreement with previous studies (Graybeal, 2006; Ahlborn et al., 2008; Vaishali and Rao, 2012).

RCPT test results for UHPC specimens are summarized in Table 7.11. All UHPC specimens showed very high resistance to chloride transport and exhibited coulomb values less than 100, indicating negligible ASTM C1202 chloride ion penetrability. RCPT results of UHPC at 28 and 56 days were comparable. The chloride concentration (3%, 3.5% and 10%) had minimum effect on the number of coulombs passed. Furthermore, no significant effect of the fibre length on RCPT results was observed. However, the steel fibre dosage had a significant effect on the passed coulomb values. For instance, the UHPC mixtures incorporating 3% and 6% of the 8 mm (0.62 in) steel fibres exhibited 27 and 35 lower
coulomb values respectively, compared to that of the control mixture without fibre addition. This can be ascribed to the role of steel fibres, which can restrict the formation and growth of plastic and drying shrinkage cracks, resulting in decreased penetrability (Banthia and Bhargava, 2007; Vaishali and Rao, 2012). MIP analysis confirmed that the addition of fibres reduced the porosity in UHPC, leading to improved durability properties (Figure 7.8). Furthermore, SEM analysis (Figures 7.6 and 7.7) showed a dense fibre-matrix and matrix-aggregate interface, indicating lower penetrability properties due to reduced porosity.

7.3.8. Chloride Ions Penetration

Figure 7.9 (b) illustrates the chloride penetration in UHPC specimens versus the depth from the ponding solution. It can be observed that the chloride ions penetration in UHPC was very limited. The maximum chloride content in UHPC specimens was 0.055 kg/m$^3$ (0.0034 lb/ft$^3$), which is lower than the corrosion threshold value of 0.60 kg/m$^3$ (0.037 lb/ft$^3$) (Angst et al., 2009). No significant effect of the testing age (i.e. 90 and 180 days) and the chloride ion concentration (i.e. 3.5% and 10%) was observed on the chloride penetration into UHPC. It was observed that fibres on the UHPC specimen surface were corroded after chloride ions exposure (Figure 7.10 (a)). A thin slice at a depth of 3 mm (0.12 in) from the specimen’s surface was prepared in order to examine signs of corrosion on the embedded fibres. However, there was no evidence of corrosion on the embedded fibres at this depth (Figure 7.10 (b)). This indicates that the corrosion of fibres was limited to the surface. This was attributed to the very low porosity of UHPC specimens, which does not allow the penetration of chloride ions, moisture and oxygen required for the onset of corrosion. Figure 7.11 shows optical and SEM images of a surface corroded fibre. It was observed that the embedded fibres were corroded only to a depth of 1 mm (0.04 in) (Figure 7.11 (b)). Energy dispersive x-ray (EDX) analysis (Figure 7.11 (d)) showed higher peaks of iron and oxygen, which also confirms the corrosion activity of the surface of fibres.

7.3.9. Effect of Chloride Ions Exposure on Mechanical Properties

A white deposit of salt material was observed on the surface of UHPC specimens after chloride ions exposure (Figure 7.12). However, no surface degradation on UHPC specimens was detected (Figure 7.12 (b)). Table 7.12 shows the relative compressive, splitting tensile and flexural strengths of UHPC specimens without fibres and with 3% of the 12 mm (0.47
in) steel fibres after 1, 3, 5 and 6 months of exposure to various chloride ion solutions (i.e. 3.5% and 10%). No degradation of UHPC mechanical properties was observed due to chloride ions exposure. This indicates the superior resistance of UHPC against chloride ions exposure, which should result into more durable and sustainable concrete structures.

7.4. SUMMARY

This study explored the effects of the steel fibre length and dosage on the mechanical and durability properties of UHPC. The following specific conclusions can be drawn from this study.

1) The compressive strength of UHPC slightly increased with steel fibre addition, while the fibre length had insignificant effect on compressive strength. It was also observed that the fibre addition improved the failure pattern from sudden explosive to ductile behaviour.

2) A significant increase in UHPC splitting tensile and flexural strengths was observed with higher dosage of steel fibres. Moreover, the fibre length considerably influenced the peak load carrying capacity and load-deflection behaviour. For instance, UHPC mixtures incorporating short steel fibres achieved higher peak load capacity and exhibited enhanced strain hardening behaviour compared to that of the mixture with a similar dosage of longer fibres.

3) UHPC exhibited improved durability properties owing to its very low porosity and denser micro-structure, which was confirmed through MIP and SEM analyses. There was no significant effect of the fibre length on durability properties of UHPC. However, at higher fibre dosage, UHPC mixtures exhibited relatively improved durability properties. No deterioration of UHPC mechanical properties was observed after chloride exposures.
7.5. REFERENCES


### Table 7.1 – Chemical and physical properties of used materials

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<th>Quartz sand</th>
<th>Quartz powder</th>
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### Table 7.2 – UHPC mixture proportions

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ᵃ Polycarboxylate superplasticizer (% by cement mass)
* 8 mm (0.31 in), 12 mm (0.47 in) and 16 mm (0.62 in) length
# 1%, 3% and 6% by total volume of mixture
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*d = days; C = cylinder; P = prism; 1 mm = 0.04 in; # small fragments*
Table 7.4 – Coefficient of variance for 12 mm (0.47 in) steel fibres

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d = days; C = cylinder; P = prism; 1 mm = 0.04 in; ## small fragments
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d = days; C = cylinder; P = prism; 1 mm = 0.04 in; ## small fragments
### Table 7.6 – Flowability, compressive strength and modulus of elasticity results

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* Flow diameter; d = days; 1 mm = 0.04 in; 1 MPa = 0.145 ksi; 1 GPa = 145 ksi

### Table 7.7 – Splitting tensile strength of UHPC

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<td>12.8</td>
<td>13.5</td>
<td>13.9</td>
<td>14.5</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>16</td>
<td>17.9</td>
<td>18.1</td>
<td>18.9</td>
<td>19.5</td>
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<td>10</td>
<td>6</td>
<td>32.7</td>
<td>33.1</td>
<td>33.8</td>
<td>34.6</td>
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</tbody>
</table>

1 mm = 0.04 in; 1 MPa = 0.145 ksi
### Table 7.8 – Flexural properties of UHPC mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Steel fibre Length (mm)</th>
<th>Steel fibre Dosage (%)</th>
<th>First crack load (kN)</th>
<th>Cracking deflection (mm)</th>
<th>Peak load (kN)</th>
<th>Peak deflection (mm)</th>
<th>Toughness (kN-mm)</th>
</tr>
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<tbody>
<tr>
<td>1</td>
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<td>-</td>
<td>20.07</td>
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<td>20.45</td>
<td>0.47</td>
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</tr>
<tr>
<td>2</td>
<td>1</td>
<td>3</td>
<td>28.68</td>
<td>0.81</td>
<td>32.52</td>
<td>1.35</td>
<td>52</td>
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<td>3</td>
<td>8</td>
<td>3</td>
<td>45.87</td>
<td>0.95</td>
<td>54.32</td>
<td>1.65</td>
<td>108</td>
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<td>4</td>
<td>6</td>
<td>3</td>
<td>76.32</td>
<td>1.10</td>
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<td>181</td>
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<tr>
<td>5</td>
<td>1</td>
<td>26.76</td>
<td>0.68</td>
<td>29.87</td>
<td>1.08</td>
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<tr>
<td>6</td>
<td>12</td>
<td>3</td>
<td>40.43</td>
<td>0.81</td>
<td>49.12</td>
<td>1.40</td>
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<tr>
<td>7</td>
<td>6</td>
<td>3</td>
<td>64.34</td>
<td>0.94</td>
<td>77.54</td>
<td>1.80</td>
<td>201</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>24.76</td>
<td>0.61</td>
<td>27.97</td>
<td>0.95</td>
<td>63</td>
<td></td>
</tr>
<tr>
<td>9</td>
<td>16</td>
<td>3</td>
<td>35.38</td>
<td>0.75</td>
<td>43.26</td>
<td>1.26</td>
<td>130</td>
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<tr>
<td>10</td>
<td>6</td>
<td>53.76</td>
<td>0.88</td>
<td>66.13</td>
<td>1.63</td>
<td>221</td>
<td></td>
</tr>
</tbody>
</table>

1 mm = 0.04 in; 1 kN = 0.224 kip; 1 kN-mm = 0.009 kip-in

### Table 7.9 – Measured crack width during flexural test of UHPC beam specimens

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Steel fibre Length (mm)</th>
<th>Steel fibre Dosage (%)</th>
<th>Average crack width at First crack load (mm)</th>
<th>Peak load (mm)</th>
<th>Failure point (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1</td>
<td>0.22</td>
<td>0.50</td>
<td>14</td>
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<tr>
<td>3</td>
<td>3</td>
<td>0.08</td>
<td>0.25</td>
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<td>0.04</td>
<td>0.15</td>
<td>7</td>
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</tr>
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<td>5</td>
<td>1</td>
<td>0.32</td>
<td>0.60</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>12</td>
<td>0.12</td>
<td>0.30</td>
<td>13</td>
<td></td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>0.06</td>
<td>0.18</td>
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</tr>
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<td>8</td>
<td>1</td>
<td>0.48</td>
<td>0.73</td>
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</tr>
<tr>
<td>9</td>
<td>16</td>
<td>0.20</td>
<td>0.36</td>
<td>16</td>
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</tr>
<tr>
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<td>6</td>
<td>0.10</td>
<td>0.22</td>
<td>11</td>
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</tr>
</tbody>
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1 mm = 0.04 in
Table 7.10 – VPV and sorptivity coefficient results for UHPC mixtures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Steel fibre Length (mm)</th>
<th>Dosage (%)</th>
<th>VPV* (%)</th>
<th>Initial sorptivity (kg/m²/h₁/₂)</th>
<th>Secondary sorptivity (kg/m²/h₁/₂)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
<td>3.52</td>
<td>0.0631</td>
<td>0.0432</td>
</tr>
<tr>
<td>2</td>
<td>1</td>
<td>6</td>
<td>3.28</td>
<td>0.0582</td>
<td>0.0382</td>
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<tr>
<td>3</td>
<td>8</td>
<td>3</td>
<td>3.11</td>
<td>0.0534</td>
<td>0.0310</td>
</tr>
<tr>
<td>4</td>
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<td>2.25</td>
<td>0.0470</td>
<td>0.0271</td>
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<td></td>
<td>3.30</td>
<td>0.0589</td>
<td>0.0387</td>
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<td>0.0540</td>
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<td>0.0390</td>
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<td>2.29</td>
<td>0.0479</td>
<td>0.0277</td>
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</table>

* VPV = volume of permeable voids

Table 7.11 – Rapid chloride ion penetrability of UHPC mixtures under various chloride ion exposures

<table>
<thead>
<tr>
<th>Mixture</th>
<th>Steel fibre Length (mm) (in)</th>
<th>Dosage (%)</th>
<th>3.0% 28 days</th>
<th>56 days</th>
<th>3.5% 28 days</th>
<th>56 days</th>
<th>10.0% 28 days</th>
<th>56 days</th>
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<tbody>
<tr>
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<td>-</td>
<td>-</td>
<td>71</td>
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<td>80</td>
<td>78</td>
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<td>2</td>
<td>1</td>
<td>60</td>
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<td>59</td>
<td>60</td>
<td>65</td>
<td>65</td>
<td>63</td>
</tr>
<tr>
<td>3</td>
<td>8 (0.31)</td>
<td>45</td>
<td>43</td>
<td>47</td>
<td>44</td>
<td>50</td>
<td>49</td>
<td></td>
</tr>
<tr>
<td>4</td>
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<td>60</td>
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<td>60</td>
<td>67</td>
<td>65</td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>12 (0.47)</td>
<td>47</td>
<td>48</td>
<td>49</td>
<td>48</td>
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<td>70</td>
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<td>52</td>
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Table 7.12 – Mechanical properties of UHPC specimens after various chloride exposures

<table>
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<tr>
<th>UHPFRC mixture</th>
<th>Exposure solution (NaCl)</th>
<th>Exposure duration (Months)</th>
<th>Relative strength factor (R)</th>
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<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Compressive strength</td>
</tr>
<tr>
<td>3.5% No fibre</td>
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</tr>
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<tr>
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<td>5</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>1.12</td>
</tr>
<tr>
<td>10.0%</td>
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<td>1.03</td>
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<tr>
<td></td>
<td></td>
<td>3</td>
<td>1.06</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>1.09</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>1.11</td>
</tr>
<tr>
<td>3% by mixture</td>
<td>3.5%</td>
<td>1</td>
<td>1.04</td>
</tr>
<tr>
<td>volume of 12</td>
<td></td>
<td>3</td>
<td>1.09</td>
</tr>
<tr>
<td>mm (0.62 in)</td>
<td></td>
<td>5</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>1.11</td>
</tr>
<tr>
<td>10.0%</td>
<td></td>
<td>1</td>
<td>1.03</td>
</tr>
<tr>
<td></td>
<td></td>
<td>3</td>
<td>1.07</td>
</tr>
<tr>
<td></td>
<td></td>
<td>5</td>
<td>1.10</td>
</tr>
<tr>
<td></td>
<td></td>
<td>6</td>
<td>1.09</td>
</tr>
</tbody>
</table>

$R = \frac{R_{CI}}{R_{28}}$; $R_{28}$ is the strength after 28 days, $R_{CI}$ is the strength measured after exposure period to chloride ions.
CHAPTER 7

Figure 7.1 – High shear pan mixer for mixing UHPC.

Figure 7.2 – Compressive strength behaviour of UHPC specimens.

a) Typical compressive stress-strain curve for UHPC specimens

b) Cylindrical fibre-reinforced UHPC specimens remained nearly intact after compressive strength testing
Typical load-deflection curve for UHPC beam specimens incorporating 3% of various fibre lengths

Effect of fibre length and dosage on peak load carrying capacity of UHPC

Figure 7.3 – Flexural testing results of UHPC beam specimens.

Localization of strain at a single crack in UHPC beam specimen leading to failure

Failure surface of beam specimen

Figure 7.4 – Flexural failure of UHPC beam specimen.
Figure 7.5 – Average crack spacing for UHPC beam specimens incorporating various fibre dosages.

Figure 7.6 – SEM images of UHPC specimen.

a) Strong microstructure

b) Interface between aggregate and matrix
Figure 7.7 – SEM image of fibre-cementitious intimate contact.

Figure 7.8 – Measured porosity for UHPC mixtures incorporating various fibre lengths and dosages.
a) Typical sorptivity plot for UHPC incorporating 3% of 8 mm (0.31 in) fibres

b) Chloride ion penetration profiles after 180 days exposure to 10% Cl\(^-\) solution

**Figure 7.9 – Sorptivity plot and chloride penetration into various UHPC specimens.**

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a) Surface corrosion of steel fibres after 30 days of salt ponding

b) No corrosion at 3 mm (0.12 in) depth

**Figure 7.10 – Corrosion of surface steel fibres (salt ponding specimen incorporating 6% by mixture volume of 8 mm (0.31 in) steel fibres).**
Figure 7.11 – Optical and SEM images of surface fibre: a) Corrosion of surface fibre, b) Penetration of corrosion in the embedded fibre (Section at S-S), c) SEM image, and d) EDX analysis ‘E’.
a) UHPC specimen after 180 days of salt ponding

b) UHPC specimens after chloride ions immersion test during drying cycle

Figure 7.12 – Deposition of salt material on the surface of UHPC specimens.
STRUCTURAL BEHAVIOUR OF ULTRA-HIGH PERFORMANCE FIBRE-REINFORCED CONCRETE TUNNEL LINING SEGMENTS*

Ultra-high performance fibre-reinforced concrete (UHPFRC) is an emerging advanced material exhibiting superior mechanical and durability properties. However, its application in precast concrete tunnel linings is lagging due to the lack of adequate design provisions. In this chapter, the structural behaviour of UHPFRC tunnel lining segments was evaluated experimentally and validated numerically. Flexural and edge point load tests were conducted on scaled-down UHPFRC tunnel lining segments to evaluate their bending and thrust load resistance. The steel fibre length and dosage in the UHPFRC mixtures were the main studied parameters.

8.1. RESEARCH SIGNIFICANCE

Various studies have examined the flexural behaviour and thrust load performance of conventional reinforced concrete (RC) precast tunnel lining segments. The premature corrosion deterioration of RC PCTL has led to the development of steel fibre-reinforced concrete (SFRC) PCTL segments, thus partially or completely replacing the conventional steel reinforcement. However, the reduced ultimate flexural load capacity of normal SFRC PCTL segments is a concern for tunneling stakeholders, which hampered its wider scale application. Therefore, this study provides an assessment of the mechanical behaviour of ultra-high performance fibre-reinforced concrete (UHPFRC) tunnel lining segments. The results of this study can define a thrust for a new direction in tunneling projects relying on UHPFRC with superior mechanical performance and durability.

* A version of this chapter was submitted to the Tunneling and Underground Space Technology Journal (2013).
8.2. EXPERIMENTAL PROGRAM

8.2.1. Material Composition and Mixing of UHPFRC

The material composition and mixing process of various UHPFRC mixtures used to cast tunnel lining segments were mentioned in previous chapter (Sections 7.2.1 and 7.2.2).

8.2.2. Fabrication of UHPFRC Tunnel Lining Segments

The length and width of the tested tunnel lining segments were 1000 mm (40 in) and 500 mm (20 in) respectively, while their thickness was 100 mm (4 in) (Figure 8.1). These dimensions represent about 1/3rd the size of normally used full-scale precast tunnel lining segments (Hung et al., 2009). The segment mold was placed on a vibratory table and UHPFRC was poured at the center of the mold allowing flow to both sides of the mold. Similar technique for concrete placement into PCTL segment molds was employed at industrial precast plants. After filling the segment mold with UHPFRC, it was immediately moved to an environment chamber at 45 °C (113 °F) and relative humidity (RH) > 95%. After 5 hours of curing, the segments were taken out from their molds and placed in a moist curing room at RH ≥ 95% and 20 ± 3 °C (68 ± 6 °F) for another 5 days. Subsequently, segments were stored under ambient laboratory conditions (i.e. 20 ± 2 °C (68 ± 4 °F)). This curing regime was selected to simulate the curing regime in commercial precast plants for fabricating full-scale PCTL segments (Recchia et al., 2012).

8.3. EXPERIMENTAL PROCEDURE AND SETUP

8.3.1. Flexural Testing

Figure 8.2 shows the experimental setup for the flexural testing of the UHPFRC tunnel lining segments. A reaction frame consisting of two long stiffened W10×49 beams, welded together and fixed to the ground frame with bolts was used for the flexural test. Each tunnel lining segment was simply supported on the reaction frame with a span of 900 mm (35.40 in). A displacement control load at a rate of 0.5 mm/min (0.020 in/min) was applied on a loading beam placed at the extrados face of segment’s mid-span using a load cell actuator having a maximum applied load capacity of 250 kN (56.20 kip). A smooth contact surface underneath the loading beam was ensured by placing a rubber pad in between the loading beam and the segment surface in order to distribute the load uniformly. The mid-span vertical
displacement was measured using three linear variable displacement transducers (LVDTs) placed at the segment’s mid-span. A high speed data acquisition system was utilized for recording all the loading and displacement readings. The crack widths at various loading steps were measured using a crack width ruler. Segments were white painted in order to inspect crack patterns.

### 8.3.2. Thrust Load Testing

The thrust load test was conducted to investigate the effects of tensile splitting stresses typically induced by tunnel boring machines during the installation process of PCTL segments (Burgers et al., 2007). The laboratory set-up for the thrust load test is shown in Figure 8.3. The segment was first placed vertically on the ground and an incremental load at a rate of 2 mm/min (0.078 in/min) was applied. The vertical displacement was measured using two LVDTs attached to the top concrete surface; while concrete splitting in the horizontal direction was monitored using another two LVDTs fastened at the intrados and extrados faces of the segments.

### 8.4. FINITE ELEMENT SIMULATION

A finite element analysis (FEA) using the commercially available software ABAQUS (Version 6.9) was performed in order to verify the experimental findings. The tested tunnel lining segments were modeled using 8-node brick elements (Figure 8.4). The geometry of the modeled tunnel lining segments is shown in Figure 8.1. A concrete damaged plasticity (CDP) ABAQUS built-in model was used in order to capture the UHPFRC behaviour. A softening stress-strain curve was used to describe the development of tensile micro-cracks behaviour (Chen and Graybeal, 2012). UHPFRC compressive stress-strain curves (Figure 7.2 (a)) recorded during experimental testing of cylinders were used as input parameters for the material elastic and plastic properties. Furthermore, the tensile fracture properties of UHPFRC were deduced from the results of beam bending tests (Figure 7.3 (a)) using the AFGC-SETRA (2002) and RILEM TC162-TDF (2003) recommendations (Appendix F) and used as input for the FEA model. The densities for various UHPFRC mixtures ranged from 2250 to 2750 kg/m$^3$ (140 to 165 lb/ft$^3$) and were employed as an input parameter. Moreover, the Poisson’s ratio was assumed equal to 0.18, similar to other study (Chen and Graybeal, 2012).
In order to fully define the concrete behaviour using CDP model, “the dilation angle, ratio of initial equibiaxial compressive yield stress to initial uniaxial compressive yield stress, flow potential eccentricity, viscosity parameter, and the ratio of second stress invariant on the tensile meridian to that on the compressive meridian” were assumed equal to 15°, 1.16, 0.10, 0.00 and 0.67, respectively, similar to previous study (Chen and Graybeal, 2012). The used values of modulus of elasticity for various UHPFRC mixtures were mentioned in Table 7.6. UHPFRC showed nearly linear behaviour up to its peak compressive load (Figure 7.2 (a)); therefore, no stiffness degradation in compression was considered for this stress range in defining the CDP model. A displacement control load was applied at the segment’s mid-span. Pinned end conditions were used as boundary conditions for modeling the experimental behaviour of PCTL segments. Various mesh sizes (structured mesh) were employed initially in order to calibrate the model.

8.5. RESULTS AND DISCUSSION

8.5.1. Flexural Performance of UHPFRC Tunnel Lining Segments

All three LVDTs installed at the segment’s mid-span showed comparable readings, indicating the absence of torsion effects (Moccihino et al., 2010). Therefore, their average values were considered for further analysis. Table 8.1 shows the crack and peak load carrying capacities of the UHPFRC tunnel lining segments. The self-weight of the loading beam (placed in between the lining segment and load cell actuator) was added in the actual load carrying capacity of the tunnel lining segments (Caratelli et al., 2011).

It was observed that the flexural performance of the tunnel lining segments with and without steel fibres was significantly different. For instance, a 42% increase in peak load was observed for the segment incorporating 1% of the 16 mm (0.62 in) fibre compared to that of the control segment without fibre addition. Furthermore, the segment without fibre addition did not show strain hardening or softening phases due to its brittle failure.

8.5.1.1. Load-Displacement Response

Figure 8.5 shows typical load-displacement responses for UHPFRC tunnel lining segments. The load-displacement response of the UHPFRC lining segments can be classified into three phases: elastic region (Portion 0C in Figure 8.5), strain hardening (Portion CP in Figure 8.5)
and strain softening phase (Portion PF in Figure 8.5). Initially, the load-displacement curve exhibited a linear portion up to the cracking point (Point C in Figure 8.5). Thereafter, the curve slope changed, indicating a reduction in stiffness due to the development of micro-cracks until the peak load (Point P in Figure 8.5). At the peak load, fibres began to pull-out from the matrix along with the development of more cracks as stresses increased. The softening phase began afterwards and its slope continuously changed as more fibres were pulled-out or fractured, along with a drop in the load carrying capacity. Tickling sound was detected during the softening phase due to fibres pull-out or fracture. The failure of segments was through the propagation of a single macro-crack (Figure 8.6 (a)).

The initial linear elastic phase (Portion 0C in Figure 8.5) for segments incorporating the short (8 mm (0.31 in)) and long (16 mm (0.62 in)) fibres were comparable, exhibiting comparable initial stiffness. However, segments incorporating the short fibres achieved superior strain hardening behaviour (Figure 8.5) due to the formation of multiple micro-cracks. The short fibres better capture micro-cracks allowing the stable formation of multiple micro-cracks rather than a single macro-crack (Sheng, 1995). The lining segments incorporating the short fibres also exhibited higher displacement at the cracking and ultimate loads compared to that of the segments made with the long fibres. For example, 54% and 63% increase in cracking and ultimate displacements, respectively, was observed for the segment incorporating 3% of the 8 mm (0.31 in) fibres compared to that of a similar segment made with the 16 mm (0.62 in) fibres (Table 8.1).

Moreover, the segment incorporating 3% of the 8 mm (0.31 in) fibres exhibited approximately 28% higher for both the first crack and peak loads compared to that of the segment with 3% of the 16 mm (0.62 in) fibres. This can be ascribed to fact that the short fibres were more closely spaced compared to the long fibres. Hence, it interfered better with cracks at the micro-level, thus delaying the formation of unstable macro-cracks leading to an enhancement for the first crack and ultimate load carrying capacities (Sheng, 1995). The higher the fibre content, the more obvious were the difference between short and long fibre mixtures. The improved crack resistance behaviour of UHPFRC lining segment is beneficial to mitigate the penetration of aggressive species responsible for the corrosion and oxidation of steel fibres.
It was observed that the length of the softening phase for tested specimens made with the short fibres was smaller and exhibited a steeper drop in load carrying capacity (Figure 8.5) since short fibres can easily be de-bonded from the matrix because of their smaller embedment length. Conversely, UHPFRC lining segments incorporating long fibres had improved strain softening (post-peak) behaviour (Figure 8.5). Long fibres, especially at low fibre content (i.e. 1% or 3%), have relatively larger inter-fibre spacing; therefore, they were only effective after the development of macro-cracks. Furthermore, the load-displacement curve for segments incorporating long fibres exhibited steadier load drop after reaching the peak load unlike the steeper load drop in the case of similar segments made with short fibres. This is due to the increase in the energy required for de-bonding the longer steel fibres.

Moreover, increasing the fibre content resulted in higher first crack load and peak load capacity regardless of the fibre length (Table 8.1). For instance, for the segment incorporating the 12 mm (0.47 in) steel fibres, increasing the fibre content from 1% to 3% induced 56% and 67% increase in first crack load and peak load, respectively. This can be attributed to the fact that at higher fibre content, fibres are more closely spaced, thus better inhibiting the localization of micro-cracks into macro-cracks (Park et al., 2012; Kang et al., 2010). This delay of macro-crack propagation increases the load carrying capacity and the associated ultimate strain (Kang et al., 2010).

It was observed that the initial stiffness of the lining segments was not affected by the fibre dosage. However, the strain hardening and softening phases increased at higher fibre dosage. Furthermore, the displacements at crack load and ultimate load were a function of the fibre content (Table 8.1). For example, the segment incorporating 6% of the 12 mm (0.47 in) fibres, the displacements at crack load and ultimate load increased by approximately 24% compared to that of the control segment with only 1% fibre content.

8.5.1.2. Toughness

The toughness of the UHPFRC tunnel lining segments incorporating fibres with various lengths and dosages was calculated based on the total area under the load-displacement curves, similar to previous study (Poh et al., 2005). The steel fibre length had an insignificant effect on the toughness of the UHPFRC segments as shown in Figure 8.7. Conversely, the steel fibre content had considerably influenced the toughness of UHPFRC segments. For
instance, segments incorporating 3% and 6% fibre content achieved 2.15 and 3.60 higher toughness than that of similar segments with 1% fibre, respectively. Thus, lining segments with higher fibre content can provide higher ductility, leading to enhanced energy absorption capacity (King and Alder, 2001; Poh et al., 2005).

8.5.1.3. Cracking Pattern

All tunnel lining segments failed in flexure and exhibited tensile cracks. The fibre pull-out mechanism was dominant (Figure 8.6 (b)). No cracks were observed during the initial elastic linear loading. The first crack, barely visible with the naked eye, was observed at the mid-span of the segment’s intrados face. Cracks were first initiated in the form of various thin micro-cracks (Figures 8.8 and 8.9). New micro-cracks developed in between the previously developed cracks as the load increased. With increasing load, most cracks continued to propagate in both directions as well as towards the upper face of segments. Closely spaced cracks developed perpendicular to flexural stresses on the segment’s intrados face (Figure 8.6 and 8.9), demonstrating the ability of UHPFRC to redistribute tensile stresses (Yang et al., 2010; Kang et al., 2010).

Tensile failure of the segments initiated as steel fibres progressively pulled-out from the matrix. Fibre pull-out began as the load carried by individual fibres exceeded the ability of the matrix to hold them (Yang et al., 2010). Fibre pull-out increased the stresses in neighbouring fibres, further pulling-out more fibres from the matrix (Sheng, 1995; Rivaz, 2008). As the load applied to the segment increased, stresses continued to increase, leading to more formation of cracks. At the ultimate load, the highly stressed fibres started pulling-out from the matrix at a localized cross-section (Figures 8.6 (a) and 8.9 (b)) with a significant increase in crack width (Table 8.2). Afterwards, the softening branch began with steeper drop in load carrying capacity as the crack width increased.

The crack width decreased for segments incorporating shorter fibres compared to that of segments with longer fibres. For example, the segment incorporating 3% of the 8 mm (0.31 in) fibre exhibited approximately 50%, 30% and 40% decrease in average crack width after first crack, at peak load and failure load respectively, compared to that of a similar segment with the 16 mm (0.62 in) fibre (Table 8.2).
Moreover, the crack width after first crack, at peak load and failure load decreased at higher fibre content (Table 8.2). For instance, increasing the fibre content from 1% to 6% decreased the average crack width by 81%, 70% and 40% after first crack, at peak load and failure load, respectively for segments incorporating the 8 mm (0.31 in) fibre.

Lining segments incorporating the short 8 mm (0.31 in) fibres exhibited more stable multiple micro-cracks compared to that of segments with long 16 mm (0.62 in) fibres which tended to develop macro-cracks (Figure 8.8). Furthermore, an increase in the number of micro-cracks was observed at higher fibre content. The average spacing between cracks decreased for segments incorporating the short fibres compared to that for segments made with long fibres (Figure 8.10 (a)). Segments incorporating short fibres showed smaller crack spacing and crack width due to shorter development length for the transfer of concrete tensile strength at a particular section compared to that of the segments with longer fibres (Schumacher et al., 2009).

8.5.1.4. Rule of Mixtures

From the experimental results, it can be concluded that the peak load carrying capacity of UHPFRC tunnel lining segments depends on the steel fibre length and dosage (Figure 8.10 (b)). Therefore, the following equation was proposed to model the peak load carrying capacity of UHPFRC lining segments using the rule of mixtures (Shah and Rangan, 1971; Naaman, 1972; Swamy et al., 1974).

\[ P_{\text{max}} = P_0 + Av_f \]  

Eq. 8.1

Where, \( P_{\text{max}} \) is the peak load carried by the UHPFRC tunnel lining segment, \( v_f \) is the volume concentration of fibres, \( P_0 \) is the peak load carrying capacity of the tunnel lining segment without fibre reinforcement, and \( A \) is a constant depending on the fibre aspect ratio (length \((l_f)/\)diameter \((d_f))\). The constant \( A \) and \((l_f/d_f)\) can be linearly related as shown in Figure 8.11 (a) and formulated as follows based on experimental results.

\[ A = 14.96 - 0.092 \left( \frac{l_f}{d_f} \right) \]  

Eq. 8.2
Figure 8.11 (b) shows the relationship between the predicted peak load \( (P_{\text{pred}}) \) based on Eq. 8.1 and the experimental peak load \( (P_{\text{exp}}) \) of UHPFRC tunnel lining segments. The ratio \( P_{\text{pred}}/P_{\text{exp}} \) was 0.99, indicating a good correlation between the predicted and experimental peak load values and confirming the applicability of rule of mixtures to UHPFRC tunnel lining segments.

8.5.2. Thrust Resistance of Tunnel Lining Segments

The thrust load resistance test was conducted on tunnel lining segments in order to evaluate the effect of concentrated loads. Figure 8.12 shows the load-displacement curve for thrust load applied to the UHPFRC segments. It can be observed that the segments with and without fibres exhibited comparable load-displacement behaviour. However, the segment without fibres had some cracking and chipping under the concentrated load (Figure 8.13). The lining segment with 3% of 8 mm (0.31 in) steel fibres showed no cracking and spalling. The results demonstrate the ability of UHPFRC tunnel lining segment to carry typical thrust load induced by TBM.

8.6. NUMERICAL VALIDATION

Table 8.3 shows the finite element predicted \( (P_{\text{fea}}) \) versus the corresponding experimental \( (P_{\text{exp}}) \) results for the tested UHPFRC tunnel lining segments. The average ratio \( P_{\text{exp}}/P_{\text{fea}} \) was 0.93, indicating a reasonable correlation between experimental results and finite element predictions. Figure 8.14 exhibits the post-processing (stress contours) of the finite element ABAQUS software. The FEA model for the tunnel lining segments was calibrated through various mesh sizes. It was observed that the mesh size of 50 mm (2 in) and 25 mm (1 in) global seeds had comparable results; however, the 50 mm (2 in) global seeds required less execution time in order to complete the analysis.

Furthermore, normal steel fibre-reinforced concrete (SFRC) and conventional rebar reinforced concrete (RC) segments were modeled and compared with UHPFRC segments. The experimental procedure, testing and dimensioning of the full-scale SFRC and RC tunnel lining segments were presented in Chapter 3. For comparison purposes, the geometrical properties of UHPFRC segments were modeled using the same dimensions as the full-scale SFRC and RC segments. Details for constitutive material modeling parameters for the SFRC
and RC (Appendix F) can be found elsewhere (Abbas, 2010; Ahn, 2011; Blazejowski, 2012).

**Figure 8.15** shows a comparison between full-scale conventional RC, SFRC and UHPFRC tunnel lining segments. It can be observed that the load carrying capacity of SFRC is lower than that of conventional RC and UHPFRC tunnel lining segments. No significant difference in peak load carrying capacity between the conventional RC and UHPFRC lining segments was observed. However, the slope of the load-displacement curve for RC segment continuously decreased, indicating the development of early cracks which further led to internal material degradation. For UHPFRC segment, the load-displacement curve is almost linear up to the peak load capacity and exhibited higher stiffness compared to that of the RC segment. This indicates that the initial cracking load for UHPFRC segment is much higher than that of control RC segment (**Figure 8.15**).

The higher cracking resistance of UHPFRC can lead to more durable tunnel lining segment compared to that of the conventional RC segments. Therefore, it can be concluded that UHPFRC is a viable and durable alternative for conventional RC in tunneling lining applications, possibly leading to more economical construction due to the elimination of conventional steel rebar cages and their corrosion-related maintenance and repair costs.

### 8.7. Moment Capacity of Tunnel Lining Segments

The nominal moment capacity \( (M_n) \) of RC, SFRC and UHPFRC PCTL segments can be estimated using the Hung *et al.* (2009), ACI 544.4R (Reapproved 2009), Sturwald and Fehling (2012) methods respectively, as follows.

\[
M_n = A_s f_y \left( \frac{d_s - a}{2} \right) \quad \text{RC segment} \quad \text{Eq. 8.3}
\]

\[
M_n = \sigma_t b (h - e) \left( \frac{h}{2} + \frac{e}{2} - \frac{a}{2} \right) \quad \text{SFRC segment} \quad \text{Eq. 8.4}
\]

\[
M_n = \lambda \sigma_t b k (h - c) \left( h + \frac{k(h-c)}{2} - \frac{c}{3} \right) \quad \text{UHPFRC segment} \quad \text{Eq. 8.5}
\]

Where, \( A_s \) is the cross-sectional area of reinforcing steel rebar (mm\(^2\)), \( f_y \) is the yield strength of steel reinforcement (MPa), \( a \) is the depth of rectangular stress diagram (mm); \( d_s \) is the
effective depth (mm), \(\sigma_t\) is the tensile stress of fibre-reinforced concrete depending on the fibre content and aspect ratio (MPa), \(c\) is the distance from the extreme compression fibre to neutral axis (mm), \(h\) is the segment thickness (mm), \(b\) is the segment width (mm), \(\lambda\) and \(k\) are the tensile stress block parameters and \(e\) is the distance from extreme compression fibre to top of tensile stress block of fibre-reinforced concrete (mm). Detailed descriptions for these parameters are mentioned in Appendix G.

Using the properties of full-scale conventional RC, SFRC and UHPFRC tunnel lining segments, the moment capacity was calculated (Table 8.4). It was observed that the experimental and finite element moment capacity of RC, SFRC and UHPFRC segments were higher than the moment capacity predicted using Eqs. 8.3-8.5, indicating the adequacy of the strength of the tested segments. Therefore, it can be concluded that the tested full-scale segments satisfy the design criteria.

8.8. SUMMARY

A study was conducted on the mechanical performance of tunnel lining segments made with UHPFRC mixtures incorporating various steel fibre lengths (8 mm (0.31 in), 12 mm (0.47 in) and 16 mm (0.62 in)) and dosages (1%, 3% and 6% by mixture volume). It can be concluded that the load-displacement curve of UHPFRC lining segments was greatly influenced by the steel fibre length and dosage. Lining segments incorporating short fibres showed an improved strain hardening phase by exhibiting multiple micro-cracks; however, a steeper drop in load carrying capacity was observed after peak load. Conversely, tunnel lining segments incorporating long fibres showed more stable post-peak (strain softening) behaviour by exhibiting a steadier drop in load carrying capacity.

Moreover, the lining segments incorporating the short 8 mm (0.31 in) fibres exhibited higher crack and peak loads in comparison with similar segments made with long 16 mm (0.62 in) fibres. Results indicated that the load carrying capacity of UHPFRC lining segments linearly increased with higher steel fibre content. The rule of mixtures was applicable to the UHPFRC tunnel lining segments. Furthermore, tunnel lining segments incorporating higher dosage of short steel fibres exhibited improved cracking patterns. The thrust load test showed comparable load-displacement behaviour for both the lining segments with and without fibres.
The experimental results were confirmed through finite element analysis. Furthermore, conventional RC and SFRC lining segments were modeled using ABAQUS and results were compared with those of UHPFRC tunnel lining segments. The conventional RC and UHPFRC showed a comparable peak load carrying capacity. However, the UHPFRC lining segments exhibited enhanced stiffness owing to its negligible internal material degradation, thus leading to much higher cracking strength compared to that of the RC segments. Therefore, it can be concluded that the application of UHPFRC in precast tunnel linings can be considered as a strong contender compared to that of conventional RC.
8.9. REFERENCES


Table 8.1 – Flexural results of UHPFRC tunnel lining segments

<table>
<thead>
<tr>
<th>Segment</th>
<th>Steel fibre Length mm (in)</th>
<th>Dosage %</th>
<th>First crack load kN (kip)</th>
<th>Cracking displacement mm (in)</th>
<th>Peak load kN (kip)</th>
<th>Peak displacement mm (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
<td>17.10 (3.84)</td>
<td>0.70 (0.027)</td>
<td>17.55 (3.94)</td>
<td>0.72 (0.028)</td>
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<td>2</td>
<td>8 (0.31)</td>
<td>1</td>
<td>26.87 (6.04)</td>
<td>1.87 (0.073)</td>
<td>29.45 (6.62)</td>
<td>4.10 (0.16)</td>
</tr>
<tr>
<td>3</td>
<td>3</td>
<td>6</td>
<td>43.86 (9.86)</td>
<td>2.17 (0.085)</td>
<td>51.21 (11.51)</td>
<td>4.74 (0.19)</td>
</tr>
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<td>4</td>
<td>6</td>
<td>3</td>
<td>74.20 (16.68)</td>
<td>2.25 (0.088)</td>
<td>85.30 (19.17)</td>
<td>4.84 (0.20)</td>
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<tr>
<td>5</td>
<td>12 (0.47)</td>
<td>3</td>
<td>24.67 (5.54)</td>
<td>1.52 (0.060)</td>
<td>27.45 (6.17)</td>
<td>3.15 (0.12)</td>
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<td>6</td>
<td>6</td>
<td>6</td>
<td>38.57 (8.67)</td>
<td>1.80 (0.070)</td>
<td>45.87 (10.31)</td>
<td>3.80 (0.14)</td>
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<tr>
<td>7</td>
<td>6</td>
<td>3</td>
<td>62.34 (14.01)</td>
<td>1.87 (0.073)</td>
<td>75.10 (16.88)</td>
<td>3.90 (0.15)</td>
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<td>8</td>
<td>16 (0.62)</td>
<td>3</td>
<td>22.78 (5.12)</td>
<td>1.15 (0.042)</td>
<td>25.05 (5.62)</td>
<td>2.20 (0.08)</td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>6</td>
<td>33.87 (7.61)</td>
<td>1.41 (0.055)</td>
<td>40.56 (9.12)</td>
<td>2.90 (0.11)</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>6</td>
<td>53.50 (12.02)</td>
<td>1.47 (0.057)</td>
<td>62.92 (14.14)</td>
<td>2.97 (0.12)</td>
</tr>
</tbody>
</table>

Table 8.2 – Crack width results of tested UHPFRC tunnel lining segments

<table>
<thead>
<tr>
<th>Segment</th>
<th>Steel fibres Length mm (in)</th>
<th>Dosage %</th>
<th>Average crack width at First crack load mm (in)</th>
<th>Peak load mm (in)</th>
<th>Failure point mm (in)</th>
</tr>
</thead>
<tbody>
<tr>
<td>2</td>
<td>1</td>
<td>1</td>
<td>0.27 (0.010)</td>
<td>0.55 (0.022)</td>
<td>15 (0.60)</td>
</tr>
<tr>
<td>3</td>
<td>8 (0.31)</td>
<td>3</td>
<td>0.10 (0.004)</td>
<td>0.27 (0.011)</td>
<td>11 (0.43)</td>
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<tr>
<td>4</td>
<td>6</td>
<td>6</td>
<td>0.05 (0.002)</td>
<td>0.16 (0.006)</td>
<td>9 (0.35)</td>
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<tr>
<td>5</td>
<td>1</td>
<td>1</td>
<td>0.40 (0.015)</td>
<td>0.70 (0.027)</td>
<td>20 (0.79)</td>
</tr>
<tr>
<td>6</td>
<td>12 (0.47)</td>
<td>3</td>
<td>0.15 (0.006)</td>
<td>0.35 (0.014)</td>
<td>15 (0.60)</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>6</td>
<td>0.08 (0.003)</td>
<td>0.21 (0.008)</td>
<td>12 (0.47)</td>
</tr>
<tr>
<td>8</td>
<td>16 (0.62)</td>
<td>3</td>
<td>0.55 (0.022)</td>
<td>0.80 (0.031)</td>
<td>25 (0.98)</td>
</tr>
<tr>
<td>9</td>
<td>6</td>
<td>6</td>
<td>0.10 (0.004)</td>
<td>0.24 (0.009)</td>
<td>15 (0.60)</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>6</td>
<td>0.20 (0.008)</td>
<td>0.40 (0.015)</td>
<td>19 (0.75)</td>
</tr>
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</table>
Table 8.3 – Comparison between experimental and finite element predicted peak load results for 1/3\textsuperscript{rd} scaled-down UHPFRC segments

<table>
<thead>
<tr>
<th>Segment</th>
<th>Steel fibres</th>
<th>Peak load</th>
<th>$P_{exp}/P_{fea}$</th>
<th>Average ($P_{exp}/P_{fea}$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Length mm (in)</td>
<td>Dosage %</td>
<td>Experimental kN (kip)</td>
<td>Finite element analysis kN (kip)</td>
</tr>
<tr>
<td>1</td>
<td>-</td>
<td>-</td>
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<td>20 (4.5)</td>
</tr>
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<td>1</td>
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<td>33 (7.4)</td>
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<td>6</td>
<td>85 (19.1)</td>
<td>87 (19.5)</td>
<td>0.98</td>
</tr>
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<td>1</td>
<td>27 (6.0)</td>
<td>32 (7.2)</td>
<td>0.86</td>
</tr>
<tr>
<td>6</td>
<td>12 (0.47)</td>
<td>46 (10.3)</td>
<td>50 (11.2)</td>
<td>0.92</td>
</tr>
<tr>
<td>7</td>
<td>6</td>
<td>75 (16.8)</td>
<td>77 (17.3)</td>
<td>0.98</td>
</tr>
<tr>
<td>8</td>
<td>1</td>
<td>25 (5.6)</td>
<td>27 (6.0)</td>
<td>0.93</td>
</tr>
<tr>
<td>9</td>
<td>16 (0.62)</td>
<td>41 (9.2)</td>
<td>42 (9.4)</td>
<td>0.97</td>
</tr>
<tr>
<td>10</td>
<td>6</td>
<td>63 (14.1)</td>
<td>68 (15.2)</td>
<td>0.93</td>
</tr>
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</table>

Table 8.4 – Comparison between experimental/finite element predicted full-scale RC, SFRC and UHPFRC results and design moment capacity of PCTL segments

<table>
<thead>
<tr>
<th>PCTL segment</th>
<th>Ultimate moment capacity (kN-m)</th>
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<th></th>
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</thead>
<tbody>
<tr>
<td></td>
<td>Experimental</td>
<td>Design/Sectional capacity</td>
<td></td>
</tr>
<tr>
<td>RC</td>
<td>171</td>
<td>137</td>
<td></td>
</tr>
<tr>
<td>SFRC</td>
<td>81</td>
<td>45</td>
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</tr>
<tr>
<td>UHPFRC</td>
<td>167*</td>
<td>135</td>
<td></td>
</tr>
</tbody>
</table>

1 kN-m = 0.74 kip-ft

* Finite element predicted
1 in = 25.4 mm

Figure 8.1 – Dimensions of tested UHPFRC tunnel lining segments.
Figure 8.2 – Experimental setup for flexural testing of UHPFRC tunnel lining segments.

Figure 8.3 – Experimental setup for thrust load testing of UHPFRC tunnel lining segments.
a) Tunnel lining segment model  

b) 8-node brick element

**Figure 8.4 – Finite element modeling of tunnel lining segment.**

![Finite element modeling of tunnel lining segment.](image)

**Figure 8.5 – Typical load-displacement curve for UHPFRC tunnel lining segments.**

![Load-displacement curve for UHPFRC tunnel lining segments.](image)

C = Crack load
P = Peak load
F = Failure point

3% of 8 mm (0.31 in) fiber
3% of 16 mm (0.62 in) fiber
Figure 8.6 – Cracking and failure surface of tunnel lining segment.

Figure 8.7 – Toughness of UHPFRC lining segments for various fibre lengths and dosages.
a) Segment made with 3% of 8 mm (0.31 in) fibre

b) Segment made with 3% of 16 mm (0.62 in) fibre

Figure 8.8 – Schematic representation of cracking in UHPFRC tunnel lining segments.
Figure 8.9 – Cracking pattern in tunnel lining segment incorporating 6% of the 16 mm (0.62 in) steel fibres.
CHAPTER 8

Effect of fibre length and dosage on crack spacing and peak load carrying capacity of UHPFRC tunnel lining segments.

Figure 8.10 – Effect of fibre length and dosage on crack spacing and peak load carrying capacity of UHPFRC tunnel lining segments.

Rule of mixtures for predicting the peak load capacity of UHPFRC segments.

Figure 8.11 – Rule of mixtures for predicting the peak load capacity of UHPFRC segments.
Figure 8.12 – Thrust load results for UHPFRC tunnel lining segments.

Figure 8.13 – Cracking of tunnel lining segment without fibre addition during thrust load testing.
Figure 8.14 – Finite element post-processing (stress contours) of UHPFRC tunnel lining segment.

Figure 8.15 – Comparison of full-scale conventional RC, SFRC and UHPFRC tunnel lining segments.
SUMMARY, CONCLUSIONS AND RECOMMENDATIONS

A comparative study of the mechanical/structural and durability behaviour of conventional RC, SFRC and UHPFRC tunnel lining segments was conducted. Based on the experimental results, it can be concluded that UHPFRC tunnel lining segments can potentially achieve higher peak load carrying capacity and better resistance to chloride ions (Cl\(^-\)) penetration than that of conventional RC and SFRC segments. Furthermore, it was observed that the mechanical properties of RC specimens were adversely affected by the Cl\(^-\) exposure. It was found that the higher concentration of chloride ions (i.e. 10% Cl\(^-\)) was more detrimental to RC segments compared to the lower concentration (i.e. 3.5% Cl\(^-\)). Conversely, SFRC and UHPFRC specimens did not show any significant reduction in mechanical properties under various corrosive environments, albeit their visual appearance was adversely affected due to corrosion of surface steel fibres.

9.1. STRUCTURAL PERFORMANCE OF TUNNEL LINING SEGMENTS

- It was observed that the peak load carrying capacity of full-scale conventional steel rebar reinforced concrete (RC) PCTL segments was higher under both static and cyclic loads compared to that of SFRC lining segments. However, SFRC segments exhibited improved initial cracking due to minimum internal material disturbances. Full-scale RC segments showed a sudden drop in their load carrying capacity; while, SFRC segments demonstrated a steadier drop in load carrying capacity due to the crack bridging effect of steel fibres.

- Based on the properties of the full-scale tunnel lining segments and the site conditions (geotechnical properties) where segments were installed, it was found that both the RC and SFRC satisfied the design criteria for flexural, thrust, settlement and punching loads. Therefore, due to its higher cracking load, SFRC can be considered as a suitable alternative where the governing design criterion is the initial cracking load. Yet, RC segments have larger reserve load and toughness.
The complete replacement of conventional RC with SFRC is not always a feasible option due to the lower strength properties of normal SFRC. Therefore, in this study, an UHPFRC material was evaluated for manufacturing tunnel lining segments. It was found that UHPFRC exhibited enhanced mechanical and durability properties due to its improved micro-structure.

Reduced-scale UHPFRC tunnel lining segments were cast in the laboratory and cured similar to the full-scale RC and SFRC lining segments. It was observed that the steel fibre length significantly influenced the peak load carrying capacity of UHPFRC tunnel lining segments. For instance, lining segments incorporating short steel fibres exhibited higher peak load carrying capacity compared to that of the lining segments with longer fibres at the same fibre dosage. Furthermore, the peak load carrying capacity of UHPFRC tunnel lining segments increased linearly with higher dosage of steel fibres.

Finite element analysis (FEA) using commercially available software ABAQUS was conducted, which confirmed the experimental behaviour of UHPFRC lining segments. Furthermore, full-scale conventional RC and SFRC lining segments were modeled and compared with that made of UHPFRC. FEA results showed that the peak load carrying capacity of UHPFRC can be comparable with that of conventional RC lining segments. Therefore, the discrepancies of normal SFRC can be eliminated using UHPFRC.

It can be concluded that the complete replacement of the conventional steel rebar cage in PCTL segments can be made using UHPFRC owing to its higher mechanical properties. Therefore, UHPFRC can result in economical manufacturing of PCTL segments due to the removal of costly and laborious manufacturing of conventional reinforcing steel cages. Furthermore, the high strength properties of UHPFRC allow decreasing the cross-sectional dimensions of PCTL segments, which can further reduce the initial manufacturing cost.

9.2. DURABILITY PERFORMANCE OF TUNNEL LINING SEGMENTS

Durability tests were conducted on cylindrical cores and sawed beam specimens retrieved from full-scale RC and SFRC lining segments. Furthermore, UHPFRC specimens were cast in the laboratory and evaluated for their chloride ions penetration resistance and mechanical
degradation under various chloride exposure conditions. The following specific conclusions can be drawn from this study:

- SFRC specimens exhibited lower initial and secondary sorptivity coefficient for both the external (intrados and extrados faces) and internal (middle) specimens, compared to that of RC specimens.

- SFRC specimens showed lower Cl\(^-\) penetration at each depth and exhibited lower chloride diffusion coefficient compared to that of the RC specimens. Furthermore, it was observed that the chloride diffusion coefficient decreased as the exposure age/period increased.

- SFRC specimens exhibited lower coulomb values using the ASTM C1202 RCPT test compared to that of RC specimens for both the external and internal specimens at all exposure solutions (i.e. 3%, 3.5% and 10%).

- SFRC specimens showed surface corrosion of steel fibres. However, no signs of corrosion on embedded steel fibres were observed even after 16 months of accelerated Cl\(^-\) exposure, which was confirmed through SEM and optical microscope analysis. Furthermore, no concrete spalling, chipping or bursting was observed due to corrosion of fibres in SFRC specimens. On the other hand, significant surface degradation including crack development was observed in RC specimens.

- Test results on bare steel rebar showed that the rebar mass loss increased as the concentration of chloride ions and exposure period increased.

- Initially, an increase in mechanical properties was observed for RC specimens for both the 3.5% and 10% Cl\(^-\) exposure. This was attributed to the advancing hydration of cement and filling of micro-pores with salt crystals. However, after 8 months of Cl\(^-\) exposure, a reduction in strength properties was observed for RC specimens. This was attributed to mass loss due to leaching of calcium ions and formation of chloro-aluminate crystals, leading to increased porosity. Conversely, SFRC showed no reduction in mechanical properties after 16 months of Cl\(^-\) exposure. Interestingly, a progressive increase in SFRC strength properties was observed with increased Cl\(^-\) exposure duration.
• An increase in rebar bond strength was observed at early-age (up to 8 months) of Cl⁻ exposure due to increased strength properties of concrete and increased friction at the rebar interface due to initial formation of corrosion products. However, at later ages (12 months or more) of Cl⁻ exposure, more formation of corrosion products at the rebar surface weakened the contact between steel and concrete, thus leading to reducing the bond strength.

• The service life of conventional RC and SFRC PCTL segments was estimated based on the chloride diffusion coefficient and concrete resistivity properties. It was found that SFRC segments can exhibit higher service life when subjected to corrosive environments compared to that of similar RC segments. Moreover, it was concluded that higher Cl⁻ concentration has adverse effects on the service lives of PCTL segments.

• It was observed that the UHPFRC tunnel lining segments exhibited enhanced durability properties compared to that of conventional RC and SFRC lining segments. This was attributed to its denser micro-structure and stronger bond between the aggregates, cementitious material and steel fibres, which was confirmed through SEM analysis. Furthermore, no degradation in UHPFRC mechanical properties was observed after exposure to various corrosive environments.

Based on the structural and durability results of the tested precast tunnel lining segments, it can be concluded that UHPFRC has a great potential for the industrial fabrication of PCTL segments compared to that of conventional RC and SFRC. UHPFRC is an attractive option for the complete substitution of conventional reinforcing steel rebar cages in PCTL segments, leading to more economical and sustainable tunnel construction. This study made an effort to provide a confidence level to tunneling stakeholders in order to tackle this issue of costly corrosion deterioration in conventional RC segments by utilizing UHPFRC taking into account the following recommendations.

9.3. RECOMMENDATIONS FOR FUTURE RESEARCH

1. Flexural and thrust load testing of full-scale UHPFRC PCTL under static and cyclic load is recommended for future studies in order to simulate real field conditions.
Such full-scale testing should facilitate the development of design codes for UHPFRC tunnel lining segments.

2. Extensive research is required on the fire behaviour of UHPFRC tunnel lining segments. Furthermore, various fire protection materials or thin sprays on UHPFRC precast tunnel lining segments need to be evaluated in order to reduce cracking and spalling of concrete during fire events.

3. In the present thesis, full-scale tests were conducted on conventional steel rebar reinforced and steel fibre-reinforced PCTL segments in order to evaluate their mechanical behaviour. Therefore, the combined effect of conventional steel rebar and steel fibre reinforcements with various dosages in full-scale tunnel lining segments should be explored in future studies.

4. The durability properties of tunnel lining segments were experimentally evaluated in the current thesis. A detailed numerical analysis is required in order to evaluate and verify the chloride diffusion coefficients and mechanical degradation of lining segments under various corrosive environments.

5. Full-scale conventional RC and SFRC lining segments identical to the ones tested in this thesis were already installed at a subway tunnel project in Canada. Detailed field monitoring and inspection of these installed PCTL segments with regard to its corrosion behaviour are required, which should enhance the understanding of real-world field exposure behaviour.

6. The application of fibre-reinforced polymer (FRP) bar cages for the fabrication of PCTL segments should be investigated in future studies for mitigating the corrosion problems.
Table A.1 – Geotechnical properties at job site where full-scale tested PCTL segments were installed

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>Diameter of tunnel, m (ft)</td>
<td>5.6 (18.60)</td>
</tr>
<tr>
<td>Depth of tunnel lining spring line below the ground surface, m (ft)</td>
<td>15.4 (50.52)</td>
</tr>
<tr>
<td>Vertical modulus of subgrade reaction, MN/m$^3$ (kip/ft$^3$)</td>
<td>30 (191.00)</td>
</tr>
<tr>
<td>Horizontal modulus of subgrade reaction, MN/m$^3$ (kip/ft$^3$)</td>
<td>9 (57.30)</td>
</tr>
<tr>
<td>Unit weight of soil, kN/m$^3$ (kip/ft$^3$)</td>
<td>21 (0.13)</td>
</tr>
<tr>
<td>Earth pressure coefficient at rest</td>
<td>0.80 to 1.10</td>
</tr>
<tr>
<td>Young's modulus of soil, MPa (ksi)</td>
<td>40 to 80 (5.80 to 11.60)</td>
</tr>
<tr>
<td>Poisson's ratio of soil</td>
<td>0.40</td>
</tr>
<tr>
<td>Free field shear strain</td>
<td>0.005% to 0.007%</td>
</tr>
<tr>
<td>Ground shear wave velocity, m/s (ft/s)</td>
<td>455 (1492.78)</td>
</tr>
</tbody>
</table>
Figure B.1 – Concrete pouring into the segment mold using overhead bucket crane at precast plant.
APPENDIX B

Figure B.2 – Covering the segment molds with plastic sheet during steam curing.

Figure B.3 – Demolding process of PCTL segment at fabrication plant.
Figure B.4 – Surface treatment process of PCTL segment at fabrication plant.

Figure B.5 – Storage of tunnel lining segments at precast plant.
Figure B.6 – Another view for experimental setup for flexural testing of lining segment.

Figure B.7 – Failure of full-scale tunnel lining segment.
Figure B.8 – Internal material disturbances in RC lining segment.
Figure B.9 – Cracking in SFRC lining segments.
Figure B.10 – Another view for experimental setup for punching test of lining segment.
Appendix C

Figure C.1 – Coring operation from full-scale PCTL segments at precast plant site.
Figure C.2 – Retrieved cores from full-scale PCTL segments.

Figure C.3 – Water sorptivity test setup.
Figure C.4 – Rapid chloride ion penetrability and salt immersion tests specimens.

Figure C.5 – Rapid chloride ion penetrability test setup.
Figure C.6 – Specimens sawed cut from full-scale PCTL segments.

Figure C.7 – Cutting prisms from full-scale PCTL segments at precast plant site.
Figure C.8 – Rebar pull-out bond test specimens.

Figure C.9 – SFRC and RC core specimens immersed in chloride solution (Day 1)
Figure C.10 – Brownish color appeared in chloride solution due to corrosion of fibres in SFRC specimens after one week.

Figure C.11 – Core specimens exposed to various chloride exposures inside a walk-in environmental chamber.
Figure C.12 – Scanning electron microscope apparatus.

Figure C.13 – Salt deposition on beam specimens after chloride exposure.
Figure C.14 – Corrosion of rebar after exposure to chloride solution.

Figure C.15 – Cracking in RC specimens due to rebar corrosion.
Figure C.16 – Corrosion of surface steel fibres in SFRC specimens after 4 months of 3.5% Cl− exposure.

Figure C.17 – Chloride penetration inside the hardened concrete specimen confirmed through scanning electron microscopy analysis.
Appendix D

Figure D.1 – Inside view of mixer during mixing process of UHPFRC ingredients.

Figure D.2 – Crack measurement using crack width rule.
Figure D.3 – Salt ponding test on UHPFRC specimens.

Figure D.4 – Salt immersion test for UHPFRC cylinder and beam specimens.
Figure D.5 – Crack localization and steel fibres bridging the crack in UHPFRC specimen.
Appendix E

Table E.1 – Strength range for various concretes

<table>
<thead>
<tr>
<th>Concrete type</th>
<th>NSC</th>
<th>HSC</th>
<th>UHSC</th>
<th>UHPC</th>
</tr>
</thead>
<tbody>
<tr>
<td>Compressive strength (MPa)*</td>
<td>&lt; 50</td>
<td>50 - 100</td>
<td>100 - 150</td>
<td>&gt; 150</td>
</tr>
</tbody>
</table>

NSC = normal strength concrete
HSC = high strength concrete
UHSC = ultra-high strength concrete
UHPC = ultra-high performance concrete

* Data taken from http://www.ce.berkeley.edu/~paulmont/241/HSC.pdf

Table E.2 – Various trial mixtures for ultra-high performance concrete

<table>
<thead>
<tr>
<th>Quantities (kg)</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cement</td>
<td>700</td>
<td>650</td>
<td>800</td>
<td>1000</td>
<td>1000</td>
<td>750</td>
<td>950</td>
</tr>
<tr>
<td>Silica fume</td>
<td>140</td>
<td>163</td>
<td>200</td>
<td>250</td>
<td>250</td>
<td>150</td>
<td>190</td>
</tr>
<tr>
<td>Quartz powder</td>
<td>210</td>
<td>163</td>
<td>200</td>
<td>300</td>
<td>300</td>
<td>225</td>
<td>285</td>
</tr>
<tr>
<td>Quartz sand</td>
<td>840</td>
<td>780</td>
<td>960</td>
<td>1200</td>
<td>1200</td>
<td>900</td>
<td>1140</td>
</tr>
<tr>
<td>Water</td>
<td>154</td>
<td>143</td>
<td>184</td>
<td>230</td>
<td>250</td>
<td>173</td>
<td>228</td>
</tr>
<tr>
<td>SP</td>
<td>25</td>
<td>23</td>
<td>32</td>
<td>40</td>
<td>40</td>
<td>26</td>
<td>19</td>
</tr>
</tbody>
</table>
Appendix F

F.1. MATERIAL MODELING FOR FINITE ELEMENT ANALYSIS

The compressive stress-strain equation used for the modeling of plain concrete was adopted from Collins and Mitchell (1997) model as follow (Eq. E.1).

\[
\frac{f_c}{f'_c} = \frac{n(\varepsilon_c)}{n-1+(\frac{\varepsilon_c}{\varepsilon_0})^nk}
\]

Eq. E.1

Where, \( f_c \) is the compressive stress; \( f'_c \) is the maximum compressive stress (compressive strength); \( \varepsilon_o \) is the concrete strain at peak compressive stress \( \left( \varepsilon_o = \frac{f'_c}{E_c}\left(\frac{n}{n-1}\right) \right) \); \( n \) is the fitting factor \( \left( n = 0.8 + \frac{f'_c}{17}\right) \); \( k \) is the stress decay factor; \( E_c \) is the modulus of elasticity \( \left( E_c = \left(3300\sqrt{f'_c} + 6900\right)\frac{\lambda_c}{2300}\right) \) and \( \lambda_c \) is the concrete density.

Carreira and Chu (1985) model was used in order to capture the stress-strain behaviour of SFRC (Eq. E.2).

\[
f_c = \frac{f'_c \beta(\varepsilon_c)}{\beta-1+(\frac{\varepsilon_c}{\varepsilon_0})^\beta}
\]

Eq. E.2

Where, \( \beta \) is the fibre factor depending on fibre volume concentration \( (\nu_f) \) \( (\beta = (0.0536 - 0.575\nu_f)f'_c) \) and \( (\varepsilon_o = 0.00076 + \sqrt{(0.626f'_c - 4.33) \times 10^{-7}}) \) for SFRC.
F.2. TENSILE FRACTURE PROPERTIES FOR FINITE ELEMENT ANALYSIS

The experimental beam bending test (Table 3.3) was used in order to capture the tensile fracture properties of SFRC. RILEM TC162-TDF (2003) recommendations was employed in order to model the softening response of SFRC. The stresses and corresponding strains (Figure E.1) can be calculated using the following equations.

\[ \sigma_1 = C_1(1.6 - \frac{h}{L})f_t; \quad \epsilon_1 = \frac{\sigma_1}{E_c} \] \hspace{1cm} \text{Eq. E.3}

\[ \sigma_2 = C_2 f_R k_h; \quad \epsilon_2 = \epsilon_1 + 0.01\% \] \hspace{1cm} \text{Eq. E.5}

\[ \sigma_3 = C_3 f_R k_h; \quad \epsilon_3 = 2.5\% \] \hspace{1cm} \text{Eq. E.6}

\[ f_{Ri} = \frac{3f_{Ri}L}{2bh^2} \] \hspace{1cm} \text{Eq. E.7}

Where, \( \sigma_i \) are the post-cracking stresses and \( \epsilon_i \) are the corresponding strains; \( f_{Ri} \) is the residual flexural strength; \( C_i \) is the stress coefficients; \( f_t \) is the flexural tensile strength; \( k_h \) is the size factor; \( L \) is the beam span length and \( b \) and \( h \) are the beam width and height, respectively.

AFGC-SETRA (2002) recommendations were used for modeling the tension behaviour of UHPFRC. Figure E.2 shows typical stress-strain curve for UHPFRC. The stresses and corresponding strains can be calculated as follows.

\[ \epsilon_{el} = \frac{f_{el}}{E} \] \hspace{1cm} \text{Eq. E.8}

\[ \epsilon_{0.3} = \frac{W_{0.3}}{l_c} + \epsilon_{el}; \quad f_c = \frac{\sigma(W_{0.3})}{K} \] \hspace{1cm} \text{Eq. E.9}

\[ \epsilon_{1\%} = \frac{W_{1\%}}{l_c} + \epsilon_{el}; \quad f_{1\%} = \frac{\sigma(W_{1\%})}{K} \] \hspace{1cm} \text{Eq. E.10}

\[ l_c = \frac{2}{3}h \] \hspace{1cm} \text{Eq. E.11}

\[ \epsilon_u = \frac{l_f}{4l_c} \] \hspace{1cm} \text{Eq. E.12}
Where, $f_{ct}$ is the tensile stress at concrete cracking, $f_c$ is the stress at 0.3 mm (0.012 in) crack width, $E$ is the young’s modulus, $l_c$ is the characteristics length (0.67$h$ for rectangular cross-section), $K$ is an orientation factor for steel fibres, $f_{1\%}$ is the stress at 0.01$h$ crack width (where $h$ is the height of the specimen).
F.3. REFERENCES


Figure F.1 – Stress-strain curve (RILEM TC162-TD, 2003).

Figure F.2 – Stress-strain curve (AFGC-SETRA, 2002).
Appendix G

G.1. CONVENTIONAL REINFORCED CONCRETE (RC) PCTL SEGMENTS

The nominal moment capacity \( M_n \) of RC PCTL segments can be determined using Hung et al. (2009) as follows.

\[
M_n = A_s f_y \left( d_s - \frac{a}{2} \right) \tag{Eq. G.1}
\]

\[
c = \frac{A_s f_y}{0.85 f'_c \beta_1 b} \tag{Eq. G.2}
\]

Where,

\[ A_s \] = cross-sectional area of steel rebar

\[ f_y \] = yield strength of steel reinforcement

\[ d_s \] = effective depth

\[ a \] = depth of rectangular stress diagram

\[ c \] = distance from extreme compression fibre to neutral axis

\[ f'_c \] = compressive strength of concrete

\[ b \] = width of segment
G.2. STEEL FIBRE-REINFORCED CONCRETE (SFRC) PCTL SEGMENTS

The ultimate moment capacity of SFRC segments can be estimated using ACI 544 (2009) as follows (Figure G.2):

\[ M_n = \sigma_t b(h - e) \left( \frac{h}{2} + \frac{e}{2} - \frac{a}{2} \right) \]  
\[ \text{Eq. G.3} \]

\[ e = (\varepsilon_{sf} + \varepsilon_c) \left( \frac{c}{\varepsilon_c} \right) \]  
\[ \text{Eq. G.4} \]

\[ c = \frac{\sigma_t(h-e)}{0.85f'_c\beta_1} \]  
\[ \text{Eq. G.5} \]

\[ \sigma_t = 0.00772\nu_f \left( \frac{lf}{df} \right) F_{be} \]  
\[ \text{Eq. G.6} \]

Where,

\( \sigma_t \) = tensile stress of fibre reinforced concrete

\( h \) = segment thickness

\( e \) = distance from extreme compression fibre to top of tensile stress block of fibre reinforced concrete

\( \varepsilon_{sf} \) = tensile strain in steel fibres based on fibre stress during pullout

\( \varepsilon_c \) = concrete compressive strain

\( \nu_f \) = volume concentration of steel fibres

\( l_f \) = length of steel fibre

\( d_f \) = diameter of steel fibre

\( F_{be} \) = steel fiber bond efficiency factor

\( \beta_1 \) = stress block parameter
G.3. ULTRA-HIGH PERFORMANCE FIBRE-REINFORCED CONCRETE (UHPFRC) PCTL SEGMENTS

The moment capacity of UHPFRC segments can be estimated using Sturwald and Fehling (2012) as follows (Figure G.3):

\[ M_n = \lambda \sigma_t b k (h - c) \left( h + \frac{k(h-c)}{2} - \frac{c}{3} \right) \]  \hspace{1cm} \text{Eq. G.7}

\[ c = \frac{2k\lambda \sigma_t (h-2)}{f_c^t} \]  \hspace{1cm} \text{Eq. G.8}

Where, \( k \) and \( \lambda \) are the stress block parameters.
G.4. REFERENCES


Figure G.1 – Cross-sectional dimensions of full-scale RC-PCTL segments.

Figure G.2 – Stress-strain diagram for SFRC (ACI 544, 2009).
Figure G.3 – Stress-strain diagram for UHPFRC (Sturwald and Fehling, 2012).
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