Evaluating Unbonded Concrete Overlay for Usage on Ontario Residential Streets

by

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Author's Declaration

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners. I understand that my thesis may be made electronically available to the public.

Abstract

Composite pavements have become increasingly common in North America, these pavements usually result from the rehabilitation of Portland cement concrete (PCC) pavements. When a PCC pavement reaches close to the end of its service life, a layer of Hot-Mix Asphalt (HMA) is placed to increase service life. However, with time, increasing traffic loads and other environmental factors, the composite pavements are experiencing heavy distresses. In order to mitigate distresses, routine maintenance must be completed on time. Many municipalities are facing challenges allocating funds due to smaller budgets, therefore they are unable to keep up with routine maintenance. This study looks at the challenges a large municipality faces when majority of the residential roads are composite pavements. This research focused on developing a solution that would allow long-term pavement performance; concrete overlays have been deemed successful in many parts of North America, mainly on highways. This study looks at the design, construction, testing and monitoring of a concrete overlay on a residential road, the first of its kind in Ontario, Canada.

This thesis presents the current knowledge gaps in the construction of residential concrete overlays through a literature review, focusing on the construction process for unbonded and bonded overlays. The trail section has been installed in the City of Hamilton (The City), Ontario, Canada. Preliminary tests and inspections were conducted on the trial section prior to the construction of a concrete overlay.

The research looks at the decision making process for overlay design and material selection. A step-by-step guideline for construction is presented along with the instrumentation installation. Instrumentation installation was a key component of this project to understand the behaviour of the section due to external factors. Fresh properties and hardened properties were tested for during and immediately after construction to monitor the preliminary behaviour of the concrete. Analysis of the concrete overlay was done using the data collected during the first year of service. Non-destructive testing such as Falling Weight Deflectometre, Light Weight Deflectometer, British Pendulum, SurPRO and T2GO were conducted before and after construction to see the effect of installing a concrete overlay. Lastly, Life Cycle Cost Analysis was

completed for two alternatives to determine the most efficient solution for residential composite pavements.

From the analysis and construction completed, it has shown the pavement to be performing very well. Further monitoring throughout the service life will need to be done to confirm the advantages and disadvantages of using concrete overlays. The aim of this research is to provide a feasible long-term solution and provide all the necessary information and data should they choose to install a concrete overlay.

Acknowledgements

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There are many individuals without whom this project would not have been possible. Thank you to Rico Fung from Cement Association of Canada and Alen Keri from Concrete Ontario for their guidance from the early stages of the project and providing constant feedback and support. Thank you to Gary Moore, Dennis Perusin, Michael Becke and Richard Andoga from the City of Hamilton for bringing forth the opportunity to conduct a unique research project and providing all the necessary information throughout this study. A sincere thank you to Steve Tamas and his entire team at Rankin Construction Inc. for construction of the entire project. Appreciation to the City of Hamilton, Cement Association of Canada, Concrete Ontario, Rankin Constructions Inc., Stantec Inc., Peto MacCallum Inc., Lafarge and National Science and Engineering Research Council (NSERC) for funding and sponsoring this project.

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Dedication

This thesis is dedicated to my forever family, thank you for being there with me every step of the way, I will always be grateful for you.

And to my friend Stephen Emslie, may you find eternal peace.

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1 Introduction

1.1 General Statement

In North America, the use of rigid pavements makes up five percent of hard surfaced roads (Atkins, 2003). The number of rigid pavements constructed has steadily increased since it was first used in Canada in 1890 (TAC, 2013). This increase can be attributed to higher traffic, heavier transport loads and the need for durability. Rigid pavements are a combination of aggregate and Portland cement, when the right proportions are used a durable, low maintenance and safe roadway can be constructed. Concrete pavements are designed mainly to accommodate heavy traffic such as highways and main intersections.

Composite pavements are a layered combination of Hot-Mix Asphalt (HMA) and Portland Cement Concrete (PCC). Composite pavements are an effective method due to the PCC providing structural capacity while the HMA acts as the surface layer and provides rider comfort. However, one of the challenges faced with composite pavements is high initial cost and potential failures related to the underlying concrete joints. A composite pavement is mainly used as a rehabilitation method, where a layer of asphalt, known as an overlay, may be placed on top of a distressed layer of concrete (Haung, 2004). Pavement overlays can be concrete or asphalt depending on the rehabilitation need.

Concrete overlays are one of the few rehabilitation methods used on high volume traffic roads experiencing heavy traffic loading. For example, when a typical asphalt pavement is beyond repair, instead of completing a full depth repair, a concrete overlay may be placed as a low maintenance and more cost-effective option (Haung, 2004). Concrete overlays have been used in North America since the 1900s (ACI Committee 325, 2002). The range of thickness for each of the overlays depend on the volume of traffic, climate, and subgrade conditions. Concrete overlays are easy to maintain as they do not experience rutting or shoving which are more common with asphalt overlays; concrete overlays can accommodate increasing traffic loads and have a service life from 20-30 years (Harrington, et al., 2014).

One of the biggest challenges facing Canadian cities and municipalities with composite pavements is: reflective cracking. Reflective cracking occurs when the crack or joint pattern of the underlying layer propagates to the top layer (Haung, 2004). Once the reflective cracks form, other distresses also begin to take place such as alligator cracks. Rehabilitation options for composite pavements include: full depth repair, milling the existing asphalt surface layer and replacing it with a new asphalt overlay. Mill and replace is the most common method, however, if the asphalt is not saw cut accordingly, reflective cracking may appear in the early stages of the pavements service life. Concrete overlays have been successful in many parts of Canada due to high pavement performance and lower future costs. A research study conducted by the University of Waterloo on Bloor St. and Auckland Rd in Toronto, Ontario, found the street to be in excellent condition with minimal surface distresses after 10 years of service (Kivi, et al., 2013).

There are two types of concrete overlays, bonded or unbonded, the type of overlay is mainly dependent on the condition of the underlying layer. Concrete overlays are not commonly used on residential streets due to budget limitations (Harrington, et al., 2014). With increasing traffic demands and use of technology, the public is looking for more innovative, rider and environmentally friendly solutions. Using concrete overlays can be beneficial for residential neighborhoods due to its 25+ years of service life vs. 7-10 years for HMA (Atkins, 2003). However, the benefits of this method will not be known unless through research and trial sections are constructed for further investigation.

The Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo in collaboration with the City of Hamilton, Concrete Association of Canada (CAC), Concrete Ontario and Natural Sciences and Engineering Research Council (NSERC) has taken on the first residential concrete overlay construction in Ontario. In order for concrete overlays to be a viable rehabilitation option for residential neighborhoods and for municipalities to implement them the design, construction and performance of this method needs to be understood. This particular research project examines concrete overlay design with a construction guideline, preliminary test results and cost analysis. The City of Hamilton (The City) herein, has many composite pavements in its Yeoville Neighbourhood, and Jameston Ave. was selected as the trial section for this project.

1.2 Background

Concrete overlays can be either bonded or unbonded. To determine whether the street needs an unbonded or bonded overlay, the top layer must first be milled off. Once the top layer has been milled off, the existing layer must be assessed for structural capacity and the type of overlay is chosen. A distress survey of the underlying layer is performed to assess whether a bonded or unbonded overlay should be constructed.

1.2.1 Bonded Overlay

Bonded overlays have been successfully used in many parts of North America, the main function of a bonded overlay is to act as one monolithic structure with the underlying layer. For a bonded overlay to be constructed, the underlying layer must be in good to excellent condition. A bonded overlay requires the existing pavement to be clear of full depth distresses and any working cracks to be repaired. If there are distresses beyond repair, such as material related distresses and the layer has structurally failed, the pavement is considered to be in poor condition and a bonded overlay cannot be placed. Bonded overlays rely on the underlying layer for flexural support and increases the overall structural capacity, therefore proper bondage is crucial (Harrington, et al., 2014).

Bonded overlay is successful when the underlying pavement has been cleaned and has grooves. Paving should begin immediately after cleaning to prevent any dust or debris in-between layers. If properly bonded, the layers will act monolithically, consequently, the bonded overlay can thinner than an unbonded overlay. If the underlying pavement is PCC, then the joints on the bonded overlay must be matched exactly, otherwise, reflective cracking will immediately propagate to the overlay (Harrington, et al., 2014), Figure 1-1 below shows a typical bonded overlay.

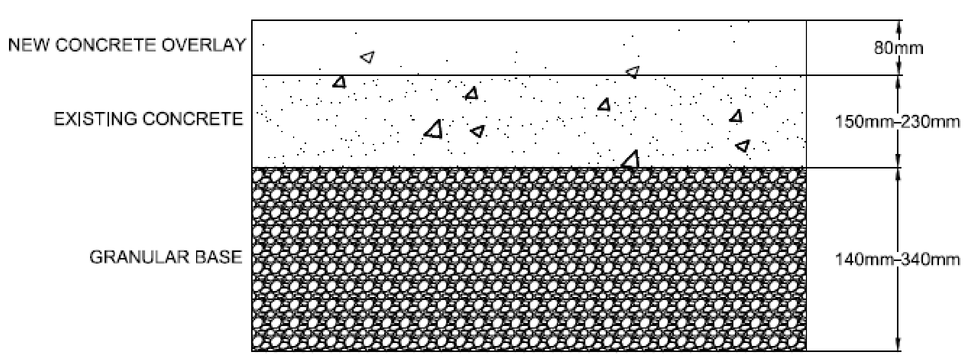

Figure 1-1 Bonded Concrete Overlay (Wafa, et al., 2017)

1.3 Unbonded Concrete Overlay

Unlike bonded overlays, unbonded does not rely on the underlying concrete for flexural support, it acts as a separate entity. However, it does use the layer as a stable base which results in an increased overall structural capacity. The condition of the underlying concrete can range between moderate to significantly deteriorated. A concrete that is significantly deteriorated will have working cracks, spalling, shattered slabs and pumping as some of the distresses. Due to the existing concrete being significantly deteriorated, a separation layer must be placed between the concrete and the overlay to prevent the distresses from propagating and to provide a smooth base for the overlay. The separation layer can either be a thin HMA layer (<25mm) or non-woven fabric such as geotextile (<3mm).

If severe faulting is observed in the concrete layer, a geotextile layer can be used to separate the existing layer with the new layer. The underlying layer should still be cleaned and cold patches should be placed to even the surface layer (Harrington, et al., 2014). Since a separation layer is used in-between, the jointing of the overlay is not required to match the joints of the underlying concrete. Figure 1-2 below shows a typical unbonded concrete overlay with HMA separation layer. The cross section of an unbonded overlay uses HMA as the separation layer, whereas Figure 1-3 displays a cross section with geotextile.

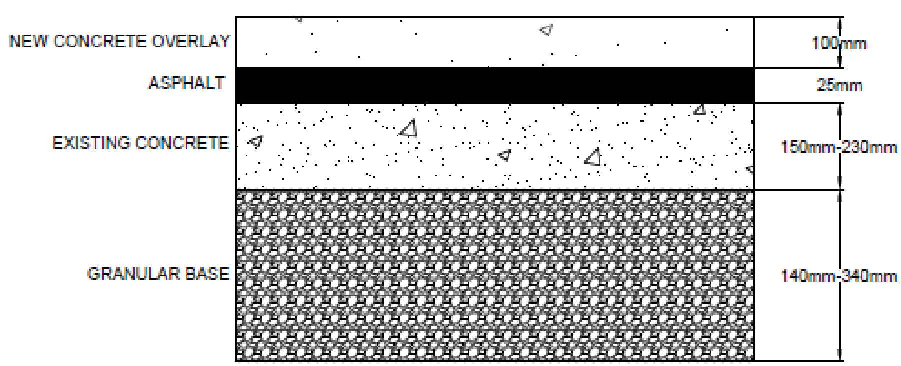

Figure 1-2 Unbonded Concrete Overlay with HMA (Wafa, et al., 2017)

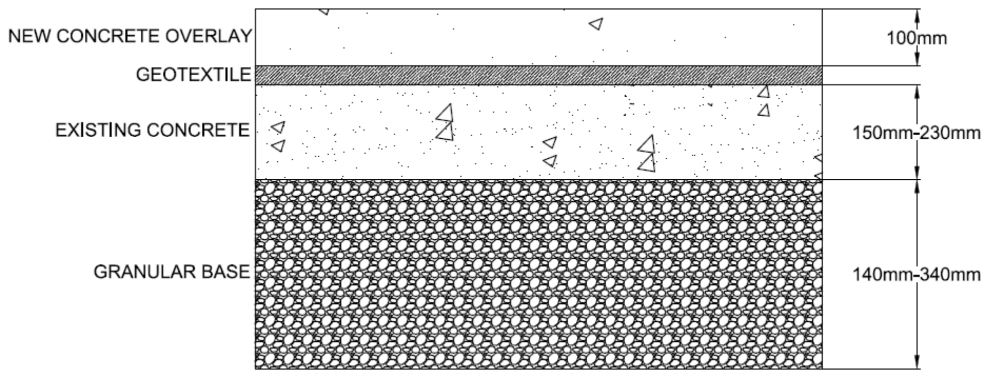

Figure 1-3 Unbonded Concrete Overlay with Geotextile
(Wafa, et al., 2017)

1.4 Research Objective

The City of Hamilton has observed deterioration in its composite pavements. Many of these streets have an existing concrete layer with an HMA surface layer. The HMA layers were originally saw cut and sealed to match the underlying concrete. This allowed the concrete to remain in good condition for the first couple of years. However, as the pavement aged, routine maintenance was limited due to unavailable resources, thus many of the sections exhibit cracking, faulting and joint spalling. The City is looking for a viable long-lasting solution for its residents. A solution that would allow them to lower the need for routine maintenance, lower environmental impacts and increase cost effectiveness.

The City collaborated with partners within the industry to develop the most feasible solution. The main objective of this proposed research is to determine the feasibility of concrete overlays for residential streets in Ontario. The City in partnership with the CPATT aimed to achieve the following objectives from the trial section, Jameston Ave.,Hamilton:

- Design and construct a full-scale trial section with instrumentation using concrete overlays
- Assess the feasibility of concrete overlays as a rehabilitation/maintenance strategy on municipal streets
- Develop a guide that helps determine the type of overlay to be used based on existing conditions

- Evaluate the mechanical responses of the pavement structure under traffic and environmental loading using embedded instrumentation
- Monitor performance of the trial street through various non-destructive testing at various times of the year
- Determine a Life Cycle Cost Analysis (LCCA) based on trial project to be used as a cost and expense guideline
- Provide recommendations and validate the use of concrete overlays for future projects

1.5 Methodology

Several sites within Hamilton were investigated, and it was determined that Jameston Avenue in the Yeoville neighbourhood was the most suitable trial section. The design of the concrete overlay was developed using StreetPave and the National Concrete Pavement Technology Centre's "Guide to Concrete Overlays". Three separate mix designs were brainstormed and developed by CPATT, CAC and Concrete Ontario.

Vibrating strain gauges were placed in each section, the strain data from these sensors were be used to assess the stress and strain relationships due to traffic, temperature and other environmental and structural impacts. FWD test was conducted by Stantec Consulting Ltd. and LWD tests were conducted by CPATT on various surfaces. SurPRO profile machine was used to determine the roughness and compare the difference between a newly constructed road and a year-old road.

The British Pendulum test and T2GO was conducted on the hardened concrete to evaluate the friction. The methodology of this research project is to determine the most cost-effective way to increase the service life for composite pavements in the municipal streets of Ontario for years to come. Figure 1-4 below outlines the research methodology in this thesis for the research project.

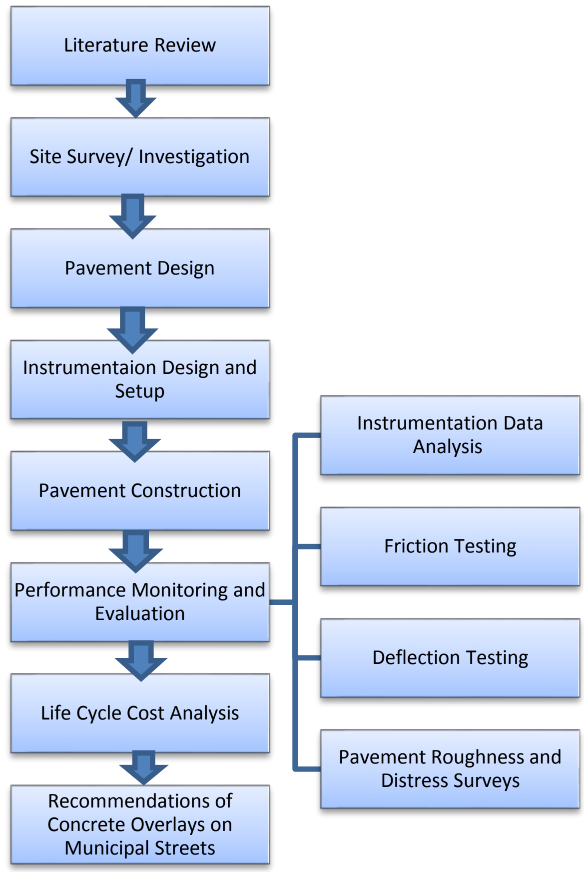

Figure 1-4 Research framework for Jameston Ave. concrete overlay

1.6 Thesis Outline

This thesis contains 6 chapters:

Chapter 1 provides a basic background on concrete overlays and the research framework. Chapter 2 is the literature review, focusing on composite pavements, reflective cracking, design of overlays pavement evaluation and sustainability. Chapter 3 focuses on the design of overlay, from the initial testing done on the composite pavement to the decision process for an unbonded overlay. Chapter 4 outlines the step-by-step construction process that took place to implement the overlay along with the instrumentation installation. Chapter 5 discusses the results collected through testing throughout the year and describes the current behaviour of the overlay. Life cycle cost analysis is also compared between two alternatives. Chapter 6 concludes the project, with challenges faced and recommendation for the future.

2 Literature Review

2.1 Overview of Composite Pavements

A composite pavement is defined as a pavement consisting of two materials, a HMA layer over a PCC layer. Composite pavements are widely used around the world as a rehabilitation method. When a layer of PCC pavement experiences severe distresses and approaches failure, an asphalt layer ranging from 2-6 inches of thickness may be placed on top to increase the longevity of the road (Chen, et al., 2015).

Placing an HMA layer on top allows the pavement to regain strength, increase rider comfort and reduce noise levels, interim making it a cost-effective solution. The underlying PCC layer provides structural support, therefore decreasing the tensile stresses that accumulate underneath the HMA layer. Whereas, the HMA layer acts as a preservation layer for the PCC, preventing water or salt from reaching the concrete, helping it last longer. The asphalt layer can also be an insulating layer, preventing temperature stresses for the underlying concrete (Núñez, et al., 2008).

When developing a composite pavement, various design factors should be taken into consideration, including the thickness of each pavement layer. Using the (AASHTO, 1993) guideline, a mechanistic analysis of composite pavements was conducted. Many research studies have shown that the thickness of the layers control the service life of the pavement. Bottom up fatigue failure in the HMA layer can be significantly decreased when a stiffer base is used; it has been shown that HMA layer can experience compressive strain instead of tensile strain. However, the bottom of the rigid layer accumulates more tensile strain than the granular base, hence when designing the pavement structure, special attention should be paid to the PCC to avoid reflective cracking (Núñez, et al., 2008).

To reduce the tensile strains at the bottom of the concrete layer, the modulus of bedrock should be increased and the thickness of the HMA and PCC layer should also increase. The maximum tensile strain was reduced by 71% once the PCC layer thickness was changed from 40cm to 80cm. When designing the HMA layer, increasing the thickness increased the maximum shear stress as well as the horizontal tensile strain (Cao, et al., 2016).

The interlayer bonding between the layers is also a factor in determining the types of distresses that may result on the composite pavement. Interlayer damages can result between the layers that determine the serviceability of the pavement. The interlayer bond condition is defined using the Goodman model seen below in equation (2-1) (Zhu, et al., 2011). Where τ (Pa) is the shear stress, K is the shear modulus between the layers, and Δu is the displacement between the layers.

$$\tau = K\Delta u \qquad (2-1)$$

Higher shear stress contributes to an increased bond coefficient between the layers. A discrete element method design was conducted to better understand the interaction of aggregate particles due to traffic loads. It was found that mainly tensile stresses contribute to the interlayer damages. Traffic loads contribute the interlayer damages when the bond coefficient is zero (Zhu, et al., 2011).

2.1.1 Reflective Cracking

One of the biggest concerns in composite pavements is the development of reflective cracks. Reflective cracks usually form due to repeated traffic loads, atmospheric temperature changes and other environmental and material factors. These factors initiate a concentrated stress point at the bottom of the underlying concrete layer, which propagate to the top of the overlying asphalt layer (Yoon, et al., 2017).

Figure 2-1 displays the three main causes of reflective cracking, a) is shear stress due to traffic loading, b) is expansion and contraction due to temperature changes and c) is bending and stretching due to slab warping (Liu, et al., 2016).

Visual inspections before any rehabilitation is done can help determine the location where reflective cracks have taken place. When an asymmetric load is applied with respect to an existing crack it causes a vertical movement at the edges, causing large shear stress; this phenomenon is known as slab rocking. Subgrade properties can reduce the stress point by increasing the modulus, as well as placing a stress absorbing layer or sealing existing cracks (Kazimierowicz-Frankowska, 2008).

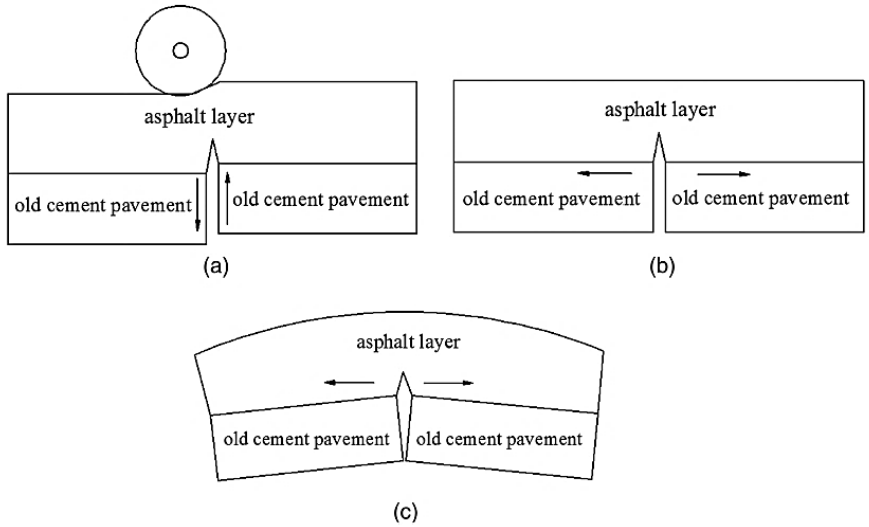

Figure 2-1 Causes of reflective cracks in a composite pavement (Liu, et al., 2016)

2.2 Overview of Concrete Overlays

Existing concrete resurfacing has been documented in the United States since the 1900s, the American Concrete Pavement Association (ACPA) has documented that until 2012, 1,152 concrete overlays have been constructed (Harrington, et al., 2014). The growth of concrete overlay usage has significantly increased in the last three decades. This is due to the transportation community looking for sustainable solutions for aging roadways. Concrete overlays can be constructed at a faster speed since the existing layer does not require removal and the road can be opened within a couple of days. Overlays have minimal maintenance, thin overlays without reinforcement can be easily milled and repaired. They are also cost effective due to providing long service life (ACI Committee 325, 2002).

Concrete resurfacing technology has improved significantly since its first installment in the 1900s. Fibre reinforcement now allows the concrete to be much thinner than before. Research has also show that to achieve a successful overlay pavement, the underlying support should be consistent and interface layer movement should be recognized (CP Tech Centre, 2012). A preservation fix is when the underlying pavement is in good shape and a bonded overlay can be placed. A rehabilitation fix is when the underlying pavement has significant deterioration causing an unbonded overlay to be placed. The main intent of overlay is to increase the structural capacity of the original pavement which deteriorates with time and traffic loading (AASHTO, 1993). Figure 2-2 illustrates the structural capacity loss as developed in the 1993 AASHTO *Design Guide for Pavement Structures*. SC_o is the structural capacity of a new pavement, SC_{eff} is the effective structural capacity once significant time has passed and the pavement has experienced traffic loads. To achieve a certain structural capacity for the future, SC_f , the overlay must have a structural capacity of SC_{ol} (AASHTO, 1993).

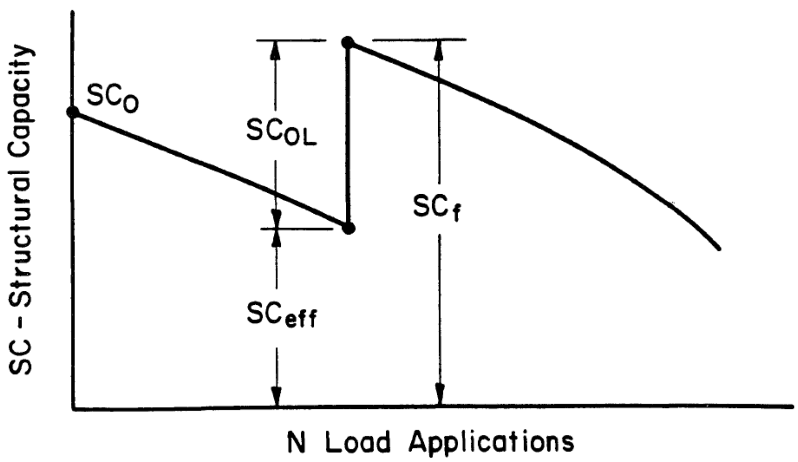

Figure 2-2 Illustration of structural capacity loss over time and with traffic (AASHTO, 1993)

2.2.1 Design of Bonded Overlays

2.2.1.1 Visual Inspections

Bonded overlays are constructed when the underlying concrete is in good to fair condition. To determine whether the concrete is in good or fair condition, certain distresses must be noticed during a visual inspection before an overlay is chosen. When the existing overlay is a composite pavement, HMA over PCC, the surveyor should look for distresses such as a few potholes, thermal cracking, loss of friction and block cracking. If cores have been taken then they should be checked for stripping. If the existing pavement is concrete, the surveyor should look for loss of friction, random cracking, partial depth joint spalling, and surface defects. When minor distresses are reparable in a timely and efficient manner a bonded overlay can be used (Harrington, et al., 2014).

2.2.1.2 Bonded Concrete Overlay on Composite (HMA over PCC)

When constructing a bonded concrete overlay on a composite pavement, the thickness of the overlay is usually 50-150mm thick. This is due to the underlying layers providing load carrying capacity. The benefit of using a concrete overlay on composite pavements is the elimination of rutting and shoving. The main intent of constructing a bonded overlay is to ensure the underlying layers and the overlay act as one (AASHTO, 1993).

To achieve proper bondage between the asphalt and the concrete, the asphalt layer should be milled a minimum of 50mm to remove any distresses. The surface should be cleaned after milling and concrete should be poured immediately to prevent debris from getting in between the layers. When designing the joints for the overlay, the patter should be small square panels ranging from 0.9 – 2.4 m. This helps reduce curling and warping as well as prevent significant differential movement between the layers; joints should also not fall on wheel paths. Paving can be completed using a slip form method or fixed form. Proper curing is a crucial factor to allow the concrete to gain its ultimate strength capacity, curing compound should be sprayed within 30 minutes (Harrington, et al., 2014).

2.2.1.3 Bonded Concrete Overlay on Concrete

The thickness of a concrete overlay on an existing concrete pavement ranges from 50-125mm. The existing pavement must be in good condition for the overlay to be constructed and to act as a monolithic layer. This type of overlay may be challenging due to the need to match the overlay joints exactly with the existing concrete joints, which can sometimes be overlooked during construction (Harrington, et al., 2014). Slab stabilization and concrete patches may be done to repair the existing concrete from faulting or asphalt patches. Milling may also be done to remove the existing distresses however, shotblasting is recommended to prevent any microcracks or fractures (Ryu, et al., 2009).

Shotblasting allows the overlay to have better bondage with the existing concrete layer. Similar to composite pavement construction, surface must be properly cleaned and conventional concrete practices when placing the overlay. Curing and joint sawing should immediately begin, 30 min after placement and when the concrete will not chip, respectively (Konduru, et al., 2010). Figure 2-3 shows the condition of the pavement improving after installing a concrete overlay.

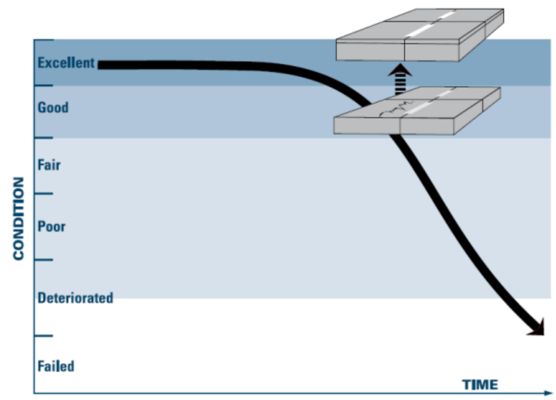

Figure 2-3 Bonded concrete overlay on concrete (Harrington, et al., 2014)

2.2.2 Unbonded Overlay

2.2.2.1 Visual Inspection

Unbonded concrete overlays are constructed when the existing pavement is in poor to deteriorated condition. A visual inspection must be completed to determine the condition of the pavement. When the existing pavement is composite, the surveyor must look for alligator cracks, rutting, slippage and shoving, raveling. Severe structural distresses are also visible when the asphalt is deteriorated, D-cracking and joint deterioration can be seen. If the existing pavement is concrete the surveyor should look for full-depth joint deterioration, working cracks, faulting, and severe structural distresses (Harrington, et al., 2014).

2.2.2.2 Unbonded Concrete Overlay on Composite or Concrete (HMA over PCC)

Unbonded concrete overlays on existing composite pavements range in thickness from 100-280mm. Unbonded concrete is designed as a new pavement since it does not rely on the underlying layers for flexural support., therefore the design is similar for a composite and concrete pavement. The underlying pavements are usually in poor to deteriorated condition, however for an overlay to be placed a uniform base is required (Mack, et al., 2006). Milling of the pavement should be done to remove the distresses, adequate drainage must be confirmed. Full depth repair and concrete patches may be also required for reflective faulting, joint spalling, pumping or severe distresses.

Drainage is a key factor in ensuring the longevity of the pavement, therefore, the existing subgrade should be thoroughly checked, and deeper edge drains placed if necessary. Drainage repairs ensure water does not accumulate between the layers, and cause faulting and pumping. Separation layer of either geotextile or HMA needs to be placed, which allows the overlay to act as a separate entity, joints from the underlying layer are not matched. If the overlay thickness is above 150mm, then the joint spacing should be around 3.6m. If the overlay thickness is below 150mm, then the joint spacing should be around 2.7m (Harrington, et al., 2014).

Joint spacing is crucial in ensuring the concrete does not warp and curl. The surface should be properly cleaned before a separation layer is placed and curing done immediately after placement (Liao, et al., 2012). Figure 2-4 below shows the condition of the original pavement going from poor to excellent after placing an unbonded concrete overlay.

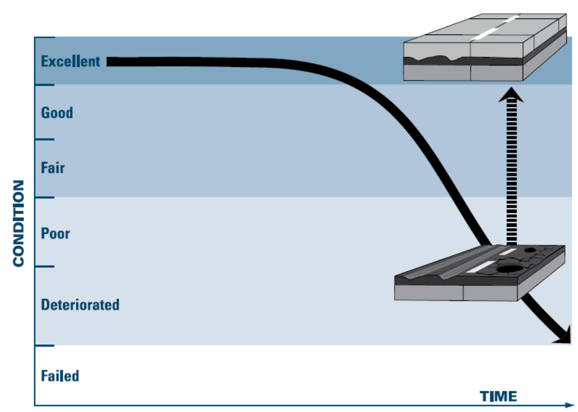

Figure 2-4 Unbonded concrete overlay on composite (Harrington, et al., 2014)

2.2.3 Separation Layers

Separation layers for unbonded concrete overlay can be either HMA or geotextile. Separation layers are also known as: interlayer and stress absorbing layer. The purpose of a separation layer is to create a shear plane that reduces reflective cracks from occurring. It also allows the overlay to act as a separate structural layer. Stress absorbing layers such as asphalt, has been shown to significantly increase the anti-cracking ratio of an unbonded overlay (Li, et al., 2011).

When determining the type of overlay for a pavement, three key factors should be taken into consideration: separation from the underlying pavement, proper bedding for the overlay and drainage. A properly placed separation layer is usually around 25mm in thickness, placed on the existing layer and thoroughly covers any distresses. It does not provide any structural support therefore should not be thicker (AASHTO, 1993). Asphalt layers also fills in any existing cracks and joints in the underlying pavement. A geotextile layer can also be used to meet certain height specifications of the road. When placing geotextile, it should be properly attached to the underlying pavement to prevent any folding during construction (Harrington, et al., 2014).

Intrusion of water between the layers can cause significant damage to the overlay as shown by Harrington & Fick, 2014:

- Longitudinal joint trapping
- Clay subgrade unable to drain
- High water table
- Lack of subdrains

To ensure the pavement has proper, adequate drainage, the cross slope should be designed to direct water towards the edge and into the subdrains. Sealing joints can help prevent water from accumulating in them. Open graded HMA or non-woven geotextile fabric can be used to allow proper drainage (AASHTO, 1993).

Geotextile should be brought to the edge of the pavement to help direct water. Research has found that HMA asphalt acts as a much better layer of stress absorbing material than geotextile, formation of reflective cracks significantly slowed down as the asphalt absorbed flexural tensile stresses and shear stresses (Liu, et al., 2016).

2.2.4 Fibre Reinforcement

The use of fibre reinforcement concrete has significantly increased due to the fibres allowing a thinner overlay design. Concrete fibres can be glass, synthetic or natural, synthetic fibres such as polypropylene are commonly used in concrete overlays, Figure 2-5. They are known to reduce plastic shrinkage, cracking, settlement of aggregates and spalling (Kosmatka, et al., 2011).

Flexural, fatigue resistance and load carrying capacity of concrete increases with the addition of fibres. The concrete overlay design thickness can be reduced by 60mm when fibres are used (Bordelon, et al., 2012). There is no specific design for use of FRC, most designs are based on an experience approach. Studies have shown FRC reduces the crack width by 1mm in comparison to an unreinforced concrete. The FRC was also had lower deflections than the unreinforced concrete. FRC also reduces construction time by avoiding laying down mesh or other reinforcement (Kim, et al., 2015).

Figure 2-5 Polypropylene fibres (CEMEX UK Materials Ltd)

2.3 Pavement Evaluation

Pavement evaluation for concrete overlays are similar to regular concrete pavements. Site evaluations are necessary before construction to determine the type of rehabilitation strategy. Site evaluations are also conducted after construction and during the service life of the pavement to understand its behavior. Various instrumentations can also be implemented into trial field tests to understand the various factors that contribute to the stress strain locations within the pavement.

Lastly, various non-destructive tests such as: Falling Weight Deflectometer (FWD), Light Weight Deflectometer (LWD), SurPRO and frictional tests can be conducted during the service life.

2.3.1 Condition Evaluation

Pavement condition evaluation is the first step in ensuring a long-lasting pavement. Distresses recognized during the survey can help determine repair methods as well as the type of overlay. The Federal Highway Administration (FWHA) has developed *Distress Identification Manual for Long Term Pavement Performance Program* (2003a) which can help municipalities identify the condition of their pavement. The ride quality of a pavement refers to the surface smoothness of the pavement section. The surveyor should look for whether the ride was uncomfortable with frequent bumps or smooth and provide a rating (MTO, 1995).

2.3.2 Instrumentation

Pavements can be hard to monitor until the end of its service life, this is due to safety concerns where once the pavement reaches towards the end of life it may not be structurally capable for traffic. Due to this reason, software such as Mechanistic Empirical Pavement Design Guide (MEPDG) are used to understand the life cycle of the pavement. However, for the software to be successful, historical and present data for traffic loads, cost, temperature, moisture and pressure must be available. Using the various data, a cost-effective and sustainable pavement structure can be designed. With the huge advancement in technology many instrumentations have been designed to successfully collect data and allow a precise prediction. These instruments can be embedded into new or rehabilitated pavements to help future designs.

Placing instrumentation can be a difficult task in the field, there is a significant amount of cost and time associated to embedding them into the pavement. For the instrumentation to function properly, it must be carefully placed without any damages, initial readings must be taken to ensure functionality and constant data collection and monitoring must be done (Willis, 2008). Some of the most common types of instrumentation have been researched by Willis, 2008:

1. Strain Gauges
2. Linearly Variable Differential Transducers (LVDT)
3. Pressure Cells

4. Thermocouples
5. Moisture Probes

Stresses that develop underneath the pavement layers due to environmental and traffic factors can determine where a fatigue crack may take place. Fatigue cracks occur due to repeated traffic and temperature loading (MTO, 1995). To determine whether bottom up reflective cracking will take place on the pavement strain gauges can be installed to better understand the loads (Willis, 2008).

Vertical strain measurements are not as popular as horizontal strain measurements, this is due to difficulty of installing them vertically on the pavement. Instead, LVDTs can be used to measure the pavement deflection due to curling and warping (Willis, 2008).

Pressure cells are usually placed on top of a pavement layer, e.g. subbase to determine the change in stress state due to temperature. They also monitor the change in pressure due to traffic loading. To receive valid data, it is a key component for pressure cells to have a plain uniform level, otherwise inconsistency will be seen during data analysis (Willis, 2008).

Concrete is known for its ability to expand and contract due to change in temperature. Therefore, thermocouples can aid in understanding the relationship between temperature and the stress zone in the pavement layers. Temperature is known to be a large contributor of reflective cracking (Kazimierowicz-Frankowska, 2008). Most strain gauges now come with thermocouples installed within, (Geokon Inc., 2016).

Lastly, installation of moisture probes between layers help understand if the pavement is experiencing any drainage issues. It also is an indicator of pavement strength that may change due to weakened pavement from over saturation. Data retrieved from the moisture proves can also help understand frost depth (Willis, 2008).

2.3.3 Non-destructive testing

Non-destructive testing is a method that involves collecting data without altering or damaging the physical structure of the pavement, allowing this type of testing to be conducted more frequently. Some of the more common non-destructive testing conducted for concrete pavements include (CP Tech Centre, 2012):

1. Falling Weight Deflectometer (FWD)
2. Light Weight Deflectometer (LWD)
3. SurPRO
4. British Pendulum
5. T2Go

FWD tests are one of the most common tests conducted on pavements. The test involves simulating moving wheel loads, where a load is dropped on a portion of the pavement and the deflection of the pavement is measured. This is a useful tool when designing a new pavement since it provides the current state of the layer. FWD can also determine the Load Transfer Efficiency to understand whether the function of joints on a concrete pavement are satisfactory (Grogan, et al., 1998). LWD tests can be used for the same function as FWD, however it is portable allowing testing to occur more often.

SurPRO is a rolling inclometer that determines the longitudinal or transverse profile of the pavement along with the International Roughness Index (IRI). Analysis of the transverse or longitudinal profile can pinpoint locations within the layers or rutting or severe pumping has taken place (International Cybernetics, 2007). The instrumentation is user-friendly and can be conducted on a yearly basis to monitor the condition of the pavement.

British pendulum tests the skid resistance on the surface of the pavement. By testing the site yearly, the change in skid resistance can be determined (ASTM E303-93, 2013). Similar to the BP, T2GO is a rolling device that can be used to determine the coefficient of friction (COF), and can be monitored at various times of the year.

2.4 Sustainability

Sustainability is defined as the “development that meets the needs of the present without compromising the ability of future generations to meet their own needs” (IISD, 2017). Sustainable design has become one of the key components of Civil Engineering. To ensure sustainable development three primary factors must be taken into consideration: 1) environment, 2) society and 3) economics (Harrington, et al., 2014). Using concrete overlays can be beneficial since the existing pavement is preserved and less materials are used. Construction costs are much less in comparison to a newly constructed road; user delays are also reduced. Concrete pavements have a longer service life and require minimum maintenance providing service to the public for many years before rehabilitation is required (TAC, 2013).

2.4.1 Environmental Impact

To properly understand the implication of pavements on the environment a Life Cycle Assessment (LCA) needs to be completed. Concrete and asphalt are the most commonly used materials in pavement structures. A comparison of LCA for both the materials can help a user determine which material maybe more sustainable.

Horvath & Hendrickson, 1998 look at the LCA of 1km of new asphalt pavement and compare to a new 1km of concrete pavement with steel reinforcement. Both pavements experience the same traffic loads and environmental factors. The design materials and cost for both types of pavement were determined and the type of resources needed for construction were also accounted for seen in Table 2-1. The table further shows the total amount of energy required for each type of pavement, asphalt pavement requires much more energy than steel-reinforced concrete.

Environmental outputs into the atmosphere can be categorized into three types: “the Toxic Release Inventory (TRI) chemical emission, hazardous waste and pollutant emissions” (Horvath, et al., 1998). Asphalt roadway was found to produce the most hazardous waste and concrete released more pollutant emissions. Asphalt pavement also release bitumen fumes during construction, due to the various chemicals it can be life-threatening for the human health (TAC, 2013).

Table 2-1 Environmental Effect of 1km Asphalt and Concrete Pavement (Horvath, et al., 1998)

Resource inputs	Unit	Asphalt	Concrete	Ratio (asphalt/concrete)
Electricity	kWh million	**0.1**	**0.1**	1
Coal				
Anthracite coal	metric ton	**0.05**	**0.1**	1
Bituminous coal	metric ton	**30**	**100**	0.3
Total	metric ton	**30**	**100**	0.3
Other Fuels				
Natural gas	metric ton	**70**	**20**	3.5
Liquefied natural gas	metric ton	**9**	**1**	9
Motor gasoline	metric ton	**3**	**6**	0.5
Aviation fuel	metric ton	**0.1**	**0.1**	1
Jet fuel	metric ton	**0.5**	**0.5**	1
Kerosene	metric ton	**0.001**	**0.001**	1
Light fuel oil	metric ton	**7**	**9**	0.8
Heavy fuel oil	metric ton	**3**	**2**	1.5
Liquefied petroleum gas	metric ton	**10**	**2**	5
Total	metric ton	**100**	**40**	2.5
	MJ	**7,000,000**	**5,000,000**	1.4
Ores				
Iron ore	metric ton	**2**	**40**	0.05
Ferroalloy ores	$	**6**	**40**	0.15
Copper ore	metric ton	**4**	**7**	0.6
Bauxite	metric ton	**0.3**	**0.5**	0.6
Gold ore	metric ton	**1**	**6**	0.2
Silver ore	metric ton	**0.02**	**0.1**	0.2
Lead-zinc ore	$	**8**	**100**	0.08
Uranium-vanadium ore	$	**1**	**10**	0.1
Fertilizers				
Ammonium nitrate	$	**5**	**20**	0.25
Ammonium sulfate	$	**5**	**5**	1
Mixed fertilizers	$	**5**	**5**	1
Organic fertilizers	$	**20**	**40**	0.5
Super phosphate	$	**2**	**2**	1
Total	$	**40**	**70**	0.6

Numbers may not sum due to rounding.

To promote sustainable development and reduce emissions and waste, recycling of pavements have become more practical for asphalt and concrete (TAC, 2013). From a survey conducted in the United States, it was found that from the total number of pavements surveyed, 80% get recycled. Research on Recycled Asphalt Pavement (RAP) has been successful, where a minimum of 20% can be used in new constructions (Sanchez-Castillo, 2014).

The percentage of concrete that gets recycled is lower than asphalt, this is due to concrete structures staying in place in the pavement after losing structural capacity. The amount of material saved from recycling asphalt is 2 times the amount of materials saved for concrete (Horvath, et al., 1998). Based on the research completed by Horvath & Hendrickson, 1998, it was found that asphalt pavements are more environmentally friendly if recycled properly.

2.4.2 Concrete Overlay Success in Canada

Concrete overlays have become more popular as a method of rehabilitation in Canada. Particularly in the province of Ontario. The City of Toronto constructed a concrete overlay in one of the major intersections at Bloor St. W and Aukland Rd. The project being in service over 14 years and has shown excellent results as discussed below. Concrete overlays have a long way to go in Canada, however it is these projects that act as guidelines and help teach industry partners and municipalities about the benefit of their usage.

The intersection of Bloor St. W and Aukland Road in Toronto, ON experiences heavy traffic throughout the year. The Average Annual Daily Traffic (AADT) of Bloor St. W is approximately 30 000 vehicles (Kivi, et al., 2013). Heavy traffic loads on the street was causing various distresses that needed constant repairs. The intersection is close to a public transit station; therefore, busses are more frequent along Aukland Road seen in Figure 2-6. A visual inspection conducted in 2003 found that both the streets had rutting and shoving, along with reflective cracking.

To accommodate the daily vehicle traffic and ensure vehicle safety, the City of Toronto practiced the mill and replace strategy on these streets. The layer of asphalt was milled off and a new layer of asphalt was placed on top of concrete. However, the composite pavement did not last as long as expected and required routine maintenance. Routine maintenance would cause constant user delays and increase construction costs; therefore, a more permanent solution was proposed (Kivi, et al., 2013).

Concrete overlay was suggested as a rehabilitation strategy for the intersection. Once the existing asphalt layer was milled off, the condition of the underlying concrete layer was observed. Based on the assessment, an unbonded concrete overlay of 150mm with 25mm HL3 asphalt as the separation layer was designed (Kivi, et al., 2013). The concrete base was sprayed with a tack coat prior to the separation layer being installed.

Vibrating wire strain gauges were embedded in the concrete to monitor the performance of the overlay. The gauges were embedded at various heights and locations along Bloor St. W and Auckland Rd, seven were installed on Bloor St. and five on Aukland Road. The intent of this was to monitor the change in concrete due to environmental and traffic effects. Over the ten-year analysis period, it was found that temperature had the largest impact on the concrete overlay. The largest strain values occurred at higher temperatures while strain decreased as temperatures dropped. In order to validate the concrete overlay design, a key component was monitoring the data and conducting visual surveys every couple of years. A visual inspection conducted in 2013 found the joints to experience minor spalling. Due to the movement and settlement of catchbasins and manholes, some diagonal cracks were visible. Other than that, the street seemed to be in excellent shape with minimal requirement for maintenance.

Figure 2-6 Bloor St. W and Aukland Road Intersection – City of Toronto (Kivi, et al., 2013)

The Bloor St. W and Aukland Rd. project is the first unbonded concrete overlay in Toronto, Ontario. The original condition of the street required major rehabilitation, installation of an unbonded concrete overlay, not only accommodates the daily growing traffic but also has been in service for the past 14 years without any maintenance. The overlay continues to show excellent performance with very minor distresses and is likely to exceed the 25+ years of service life. The project is a good representation of the use of overlays as a rehabilitation strategy on composite pavements. Use of overlays as a rehabilitation method, reduces overall cost and use of materials and has been highlighted through this project at the City of Toronto.

2.5 Chapter Summary

- Composite pavements are common in many parts of North America, they are considered to be the perfect combination of pavement materials. A combination of PCC and HMA where rider comfort and structural capacity can both be achieved.
- Preserving composite pavements poses to be a challenge for the industry as reflective cracking is the main cause of deterioration. Reflective cracking occurs from a buildup of tensile stresses at the bottom of the concrete layer and shear stress due to vehicular loading.
- Concrete overlays can be used to rehabilitate composite pavements, there are two types of overlays unbonded and bonded. Separation layers such as asphalt and geotextile can be used in unbonded overlays.
- Visual inspections are crucial to understand the condition of the underlying layer and determine the type of overlay that should be used. Specific distresses are key indicators of pavement condition and the type of repair needed to ensure a uniform layer is maintained for an overlay.
- Fibre reinforcement provides support from shrinkage cracks, spalling and aggregate settlement. Use of fiber reinforcement has been known to significantly reduce the overall thickness of the overlay.
- Concrete overlays have been successful in Canada; however, they have not been implemented in residential streets. Municipalities require long-term solution to reduce routine maintenance due to the limited budgets. City of Toronto has successfully installed concrete overlay and the overlay has been in service for 14 years without any major distresses.

3 Trial Selection and Preliminary Design

This chapter looks at the research project in depth; providing background information on the selection process for the trial section. The project location is discussed along with visual inspections that were conducted to determine the existing condition of the pavement. A coring report was also conducted and discussed in this chapter to determine the underlying condition of the pavement layers. FWD tests along with LWD tests were conducted on the selected trial section, Jameston Ave., to understand the functionality of the pavement layers and determine repair options.

3.1 Introduction – Jameston Ave.

The City of Hamilton, Ontario has many residential roadways that are composed of composite layers of asphalt over PCC. Many of these streets were first constructed around the 1960s or earlier (City of Hamilton , 1969). The condition of these streets is poor and need major rehabilitation. The City was looking for an innovative solution that would be cost and construction efficient. A solution that could potentially provide theses stress with a longer service life and reduce the need of regular maintenance.

The City had previously rehabilitated these streets using an asphalt mill-and-replace strategy. Where the topmost asphalt layer would be milled off and a new layer of asphalt would be placed. Once the asphalt layer was placed, the asphalt would be saw cut to match the joints of the underlying PCC layer. This design strategy was used to mitigate the effects of reflective cracking. The joints were then sealed with a rubberized sealant to prevent moisture or other debris from damaging the overall pavement. However, with time the sealants began to degrade and were not regularly maintained causing more distresses along the street.

The Centre for Pavement and Transportation Technology (CPATT) at the University of Waterloo (UofW) in partnership with the Cement Association of Canada and Concrete Ontario developed a preliminary proposal for the City of Hamilton to construct a trial concrete overlay. The proposal was aimed to show the benefits and implementation of a concrete overlays as a rehabilitation method. Residential concrete overlays are not common, and this project is the first one of its kind in Ontario, Canada.

If the implementation and behavior of the overlay was deemed to be successful, the City would consider future usage of concrete overlays as a viable option for rehabilitation. Other municipalities facing similar challenges would also gain the necessary knowledge to determine whether they should invest in overlays as a residential street rehabilitation option.

3.2 Project Location

To determine the trial street, various residential composite pavements within the City of Hamilton were observed. The streets were all assessed for: accessibility for residents during construction, construction costs and challenges, traffic, pedestrian safety, condition and feasibility of the individual pavement. The Yeoville neighbourhood in the City was found to have composite pavements that would meet all the criterions for a successful trial street. Within the neighbourhood, Jameston Ave., Hawkridge Ave., and Caledon Ave., were the most suitable pavement options. From the three viable streets, Jameston Ave. was chosen as the trial pavement due to it being the most accessible during construction.

Jameston Ave. is approximately 400m long and 8.5m wide. There are 12 houses on the north side of the road with their driveways facing Jameston Ave. and 8 on the south. The residents in these houses can solely access their driveways through Jameston Ave. This would be problematic during construction; however ample street parking is available for residents on Hawkridge Ave. and Caledon Ave. The street acts as a connector between two arterial roads leading to the highway, it faces heavier traffic and therefore it would be a suitable candidate to fix. Having a concrete overlay would allow for increase in traffic and serviceability.

Figure 3-1 shows the location of Jameston Ave., Hamilton, Ontario. The street is East of Highway 403 and about an hour drive west from Toronto. The street lies between two arterial roads, Upper James St. to the east and West 5th St. to the west of the road. Jameston Ave. is north of the Lincoln M. Alexander Parkway, a collector highway, which can be accessed through Upper James St. There are three blocks within the road and it is intersected by Hawkridge Ave. on the west and Caledon Ave. on the east.

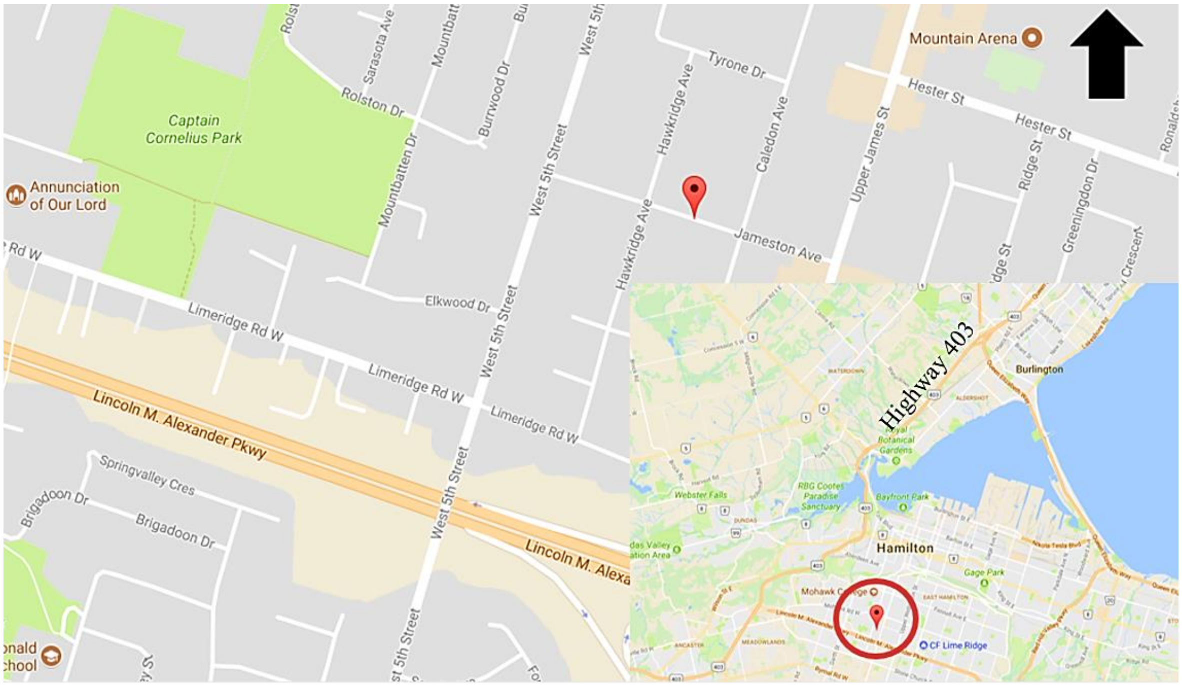

Figure 3-1 Trial section: Jameston Ave, Hamilton, ON (Google , 2017)

Figure 3-2 Section 1-3, Jameston Ave Satellite Image (Google , 2017)

Jameston Ave. was divided into three sections shown in Figure 3-2, Section 1 is between West 5th St. and Hawkridge, Section 2 is between Hawkridge Ave. and Caledon Ave., Section 3 is between Caledon Ave. and Upper James St. Section 1 and 2 mostly comprise of residential homes. A church is located on Section 3 and the driveways to two plazas also exit onto Jameston Ave. as displayed in Figure 3-3. Alexanian Carpet and Floors, a furniture store, is location on the south side of the street, the 2 driveways are used by trucks during deliveries. Goodyear Certified Auto, an automotive shop, has two driveways on the north of Jameston Ave. used by their customers. Finally, the church heavy usage by the community on Sundays.

Figure 3-3 Section 3 driveways for church, furniture store and automotive (Google , 2017)

3.3 Pre-Construction Assessments

Once the street was selected, pre-construction assessments were conducted to understand the current state of the pavement. Part of the assessments include: investigating the site and evaluating the various distresses on the road, conducting FWD and LWD tests and collecting cores. Cores were collected for various composite pavements in Hamilton before the trial street was chosen.

3.3.1 Site Condition Evaluation

During visual inspection of the composite pavement, Jameston Ave, a distinct design was observed. There were two rows of 100mm wide bricks at the edges of the entire length of the pavement seen in Figure 3-4. The structural intent of these bricks at the time of inspection was not understood. However, similar to interlocking pavers used for drainage, the bricks may have been placed for the same intent. The bricks were not routinely maintained, in many parts of the street, some of the bricks are covered with asphalt, which may have taken place during resurfacing (Pickel, et al., 2016). The surface layer of the road was asphalt with transverse saw cuts all along the road.

Figure 3-4 Brick gutter along Jameston Ave. (Pickel, et al., 2016)

Once the transverse saw-cut joints were identified, the distance between joints were measured, the spacing of the joints was approximately 9.5m. The transverse joints were sealed to prevent bottom-up reflective cracking from the underlying layer. Due to the lack of maintenance, the sealant in the transverse joints had significantly deteriorated. This deterioration caused distresses such as alligator cracks to form from the joints as seen in Figure 3-5. These cracks were seen along the entire street.

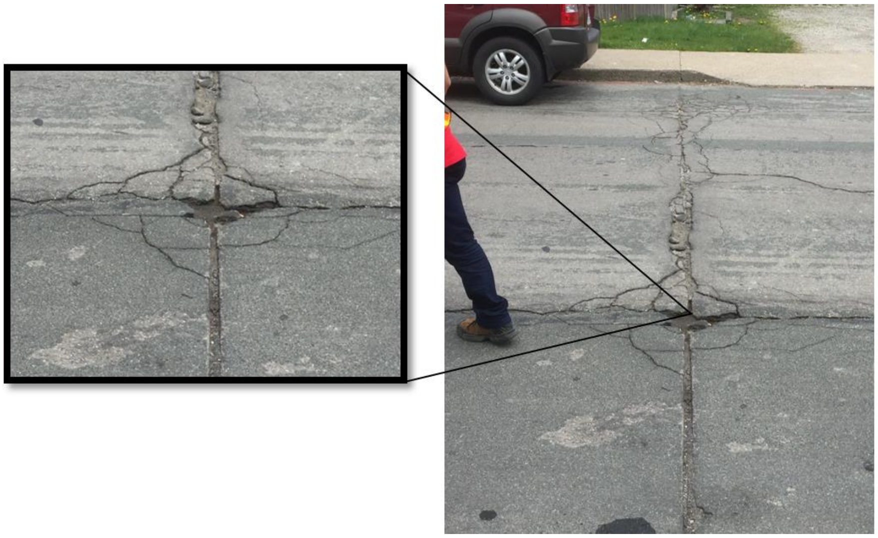

Figure 3-5 Typical deterioration on transverse saw-cut joints (Pickel, et al., 2016)

Since the asphalt layer was not longitudinally saw cut to match the underlying layer, this caused longitudinal cracks to develop Figure 3-6. The longitudinal cracks appeared at the center of the street and at 1/3rd of the width in some sections (Pickel, et al., 2016). The cracks did not run in a straight line, at certain locations they meandered. At the time of inspection, the cause of the cracks was unknown, they could have resulted from the longitudinal joints or distresses underneath. The deterioration along the street did not necessarily indicate the condition of the underlying pavement.

Figure 3-6 Longitudinal cracks along Jameston Ave. (Pickel, et al., 2016)

The City of Hamilton provided elevation drawings of Jameston Ave, Figure 3-8., the ends of the street at Upper James St. and West 5th St. are at a higher elevation. The middle portion of the street (Section 2) is at the lowest elevation. The difference between the highest and lowest point on the street is about 6m. Water is likely to run from the higher elevation points in Section 1 and Section 3 towards the middle low-point, Section 2. Significant amount of deterioration had developed in Section 2, with increased rainfalls and lack of proper drainage, water had accumulated in the lower region (Pickel, et al., 2016). The accumulation of water in Section 2 caused the soil to be overly saturated causing noticeable distortions.

The catch basins settled overtime and voids were visible through the grating, Figure 3-7. At the time of the visual inspection it was not clear how much damage the underlying concrete experienced. The impact of water to the bondage between each layer was also unknown. The sidewalks along the street were in need of repair, sidewalk distresses in Section 2 were more apparent.

Figure 3-7 Settled catch basins in Section 2 (Pickel, et al., 2016)

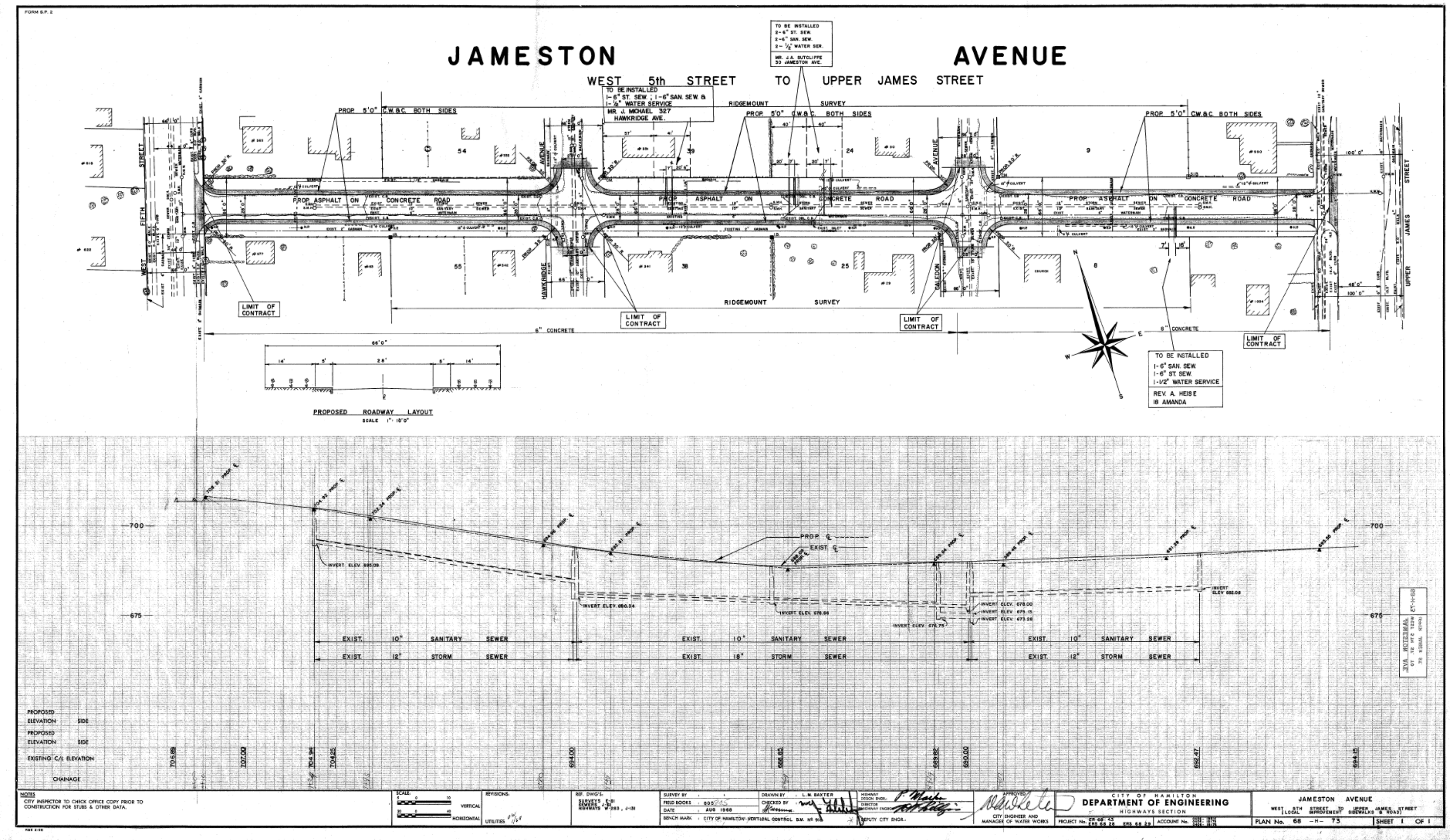

Figure 3-8 Jameston Ave. elevation view : engineering drawing (City of Hamilton , 1969)

3.3.2 Coring Report

A coring report was conducted and complied by Peto MacCallum Ltd., a coring report outlines the pavement structure and can give inside into the current condition of the pavement materials. The coring report was provided to the City and CPATT during the selection process for the trial street. A total of six cores were taken along Jameston Ave., two cores for each section. Table 3-1 shows the thicknesses of each layer along Jameston Ave. The cores were taken starting from Upper James St., where 5D 1-2, 5D 3-4 and 5D 5-6 represent Section 3, Section 2 and Section 1, respectively. The asphalt thickness is consistent throughout the street whereas the concrete thickness ranges significantly from 150mm to 235mm. The lowest concrete thickness is 5D 3-4, Section 2, which maybe be indicative of the original state of the section. The subgrade for most of the street is clay except Section 2, which is topsoil, top soil is known to absorb more water.

Figure 3-9 below shows a typical pavement core from Jameston Ave. Appendix A presents photos of all the cores that were obtained for this project. Most of the cores the asphalt was separated from the concrete. The bottom portion of the concrete core has degraded significantly more than the top. For Section 2, it was not possible to pull out the concrete core for 5D-3, this was due to a layer of sand and gravel existing between the asphalt and concrete layer (Peto MacCallum Ltd., 2015). This was not observed with the other remaining cores.

Table 3-1 Jameston Ave. Pavement Structure (Peto MacCallum Ltd., 2015)

Borehole No.	Asphalt Thickness (mm)			Concrete Thickness (mm)	Granular Thickness (mm)			Subgrade
	Surface	Binder	Total		Base	Subbase	Total	
Jameston Avenue from Upper James Street to West 5th Street								
5D-1	40	35	75	235	150	---	150	CLAY: Brown silty clay, trace sand, low plastic, APL
5D-2	45	40	85	205	260	---	260	CLAY: Brown silty clay, trace sand, low plastic, APL
5D-3	40	40	80	150	140	---	140	TOPSOIL: Dark brown silty clay topsoil, low organic, WTPL
5D-4	40	40	80	170	275	---	275	TOPSOIL: Dark brown silty clay topsoil, low organic, WTPL
5D-5	40	40	80	193	277	---	277	CLAY: Brown silty clay, trace sand, non-slightly plastic, APL
5D-6	45	40	85	175	340	---	340	CLAY: Brown silty clay, trace sand, slightly plastic, WTPL to APL

Figure 3-9 Typical sample core from Jameston Ave.-Section 1 (Peto MacCallum Ltd., 2015)

3.3.3 StreetPave 12 Design

Prior to construction, a preliminary design of concrete overlay, bonded and unbonded, was created for Jameston Ave. The purpose of creating a preliminary design is to determine range in thickness for the overlay. This would allow better preparation for construction when the asphalt in the composite pavement is milled off. To complete the preliminary designs, StreetPave 12, a software designed by the American Concrete Pavement Association (ACPA), was used. This software allows the user to input various design parameters and outputs the minimum thickness that would be required to meet design criterions.

The coring report provided to the City of Hamilton and CPATT was heavily used in this preliminary design. Using the pavement information of the six cores provided for Jameston Ave., an unbonded and bonded concrete overlay was designed for each core. The final design for the overlay can only be determined once the underlying pavement has been assessed, however, having both unbonded and bonded designs completed for each cores will further benefit that process.

3.3.3.1 Pavement Design Requirements

- Each concrete overlay designed is required to meet a performance with 85% reliability.
- The overlays should have 2% cross slope down for proper drainage.
- All designs would be completed for the right wheel path
- The overlays should have a minimum design life of 25 years

3.3.3.2 Pavement Design StreetPave 12

As previously mentioned, StreetPave 12 will provide a preliminary design, therefore many assumptions were made that would be confirmed during construction. The first design variable is AADT, based on the data provided by the City of Hamilton, the AADT for Jameston Ave. was used as 50-800 average daily traffic. From the daily traffic, it was estimated that about 20 trucks per day would be passing through Jameston Ave with a growth rate of 1%. From the coring report it was found that most of Jameston Ave. sits atop of clayey subgrade, with the exception of Section 2 where topsoil was found. California Bearing Ratio (CBR) for clayey subgrade ranges from 3-5%, to be conservative 3% was used in the design (Pickel, et al., 2016).

During the software design process, very minimal information was known about the actual condition of the pavement layers. To emulate the condition of the pavement layers, adjustment factors are required. Adjustment factors can be used to consider any distresses that may not be accounted for during actual design and construction process. There are various adjustment factors for both unbonded and bonded concrete overlay. Joints and crack factor is applied to both bonded and unbonded overlay where fatigue and durability adjustment factors are applied for bonded. To simulate the effect of these factors, the effective thickness of the existing concrete is reduced in the program. Table 3-2 below outlines the type of adjustment factors and the values used during preliminary design.

Table 3-2 Adjustment factors used on StreetPave 12 for Jameston Ave.

Type of Adjustment Factor	Adjustment Factor
Joints and cracks – due to reflecting cracking from a lack of maintenance	0.9
Fatigue – cause of transverse cracks along the road	0.9
Durability factor – cause of spalling and preventing bondage between layers	0.8

The concrete overlay is expected to have a compressive strength of 32MPa and a flexural strength of 3.5 MPa, the use of polypropylene fibres were not taken into account in StreetPave 12. For residential pavement it is expected that approximately 15-25% of the slabs will be cracked and the pavement will reach terminal serviceability at 2.0 (Pickel, et al., 2016).

3.3.3.3 Preliminary Results

Table 3-3 provides a summary of an unbonded and bonded overlay for each of the six cores extracted from Jameston Ave labelled 5D1-6. A worst case scenario has also been presented, where minimum thickness for existing concrete and granular base has been used. For each type of overlay, a design thickness and a required thickness was determined. The required thickness is the one presented by the software StreetPave 12, whereas the design thickness has been rounded up to the nearest 5-10mm.

From the design analysis, it was found that unbonded overlays have a higher thickness than bonded overlays. This is due to unbonded overlays not relying on the existing underlying concrete for flexural support. Case 5D-3 was found to have the highest thickness for an unbonded overlay, this portion of the core is from Section 2 of the pavement. As mentioned in the coring report, Section 2 was found to be the weakest of all the sections due to its underlying topsoil subgrade. The worst-case scenario requires an unbonded thickness of 115mm, which would require a separation layer. Use of a separation layer along with the overlay would significantly increase the height of the pavement.

Across the table, NR (not required) is used for unbonded overlays. This occurred due to StreetPave 12 deeming the existing underlying concrete structurally sound to withhold vehicular load, therefore an unbonded overlay would not be required.

Table 3-3 Preliminary Design Inputs and Results

	Case						
	Worst Case	5D-1	5D-2	5D-3	5D-4	5D-5	5D-6
Existing Concrete Thickness (mm)	130	235	205	150	170	193	175
Existing Granular Base Thickness (mm)	140	150	260	140	275	277	340
CBR (%)	3	3	3	3	3	3	3
AADTT (# of trucks/day)	20	20	20	20	20	20	20
Growth Rate (%)	1	1	1	1	1	1	1
Terminal Serviceability	2	2	2	2	2	2	2
Cracking Factor	0.9	0.9	0.9	0.9	0.9	0.9	0.9
Fatigue Adjustment Factor	0.9	0.9	0.9	0.9	0.9	0.9	0.9
Durability Factor	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Design Life (years)	25	25	25	25	25	25	25
Unbonded Overlay							
Design Thickness (mm)	115	NR	NR	100	50	NR	NR
Required Thickness (mm)	110	NR	NR	90	40	NR	NR
Bonded Overlay							
Design Thickness (mm)	90	15	50	90	50	40	50
Required Thickness (mm)	80	10	50	80	45	35	45

Based on the summary of results from the preliminary design it was determined that Jameston Ave. would be a good trial section for unbonded and bonded overlay design. Once the existing concrete is inspected visually there is a possibility of installing bonded in one section and unbonded in another section to monitor the behavior of two types of overlay. However, if the condition of the underlying concrete is in poor-fair condition and require unbonded overlay only, then various separation layers can be tested to determine the most feasible one in a residential street. If the pavement is in good-excellent condition, then various thickness of bonded overlay can be tested and monitored for performance.

3.3.4 FWD and LWD Testing

Prior to the start of construction, a Falling Weight Deflectometer (FWD) test Figure 3-10, was conducted on the existing composite pavement by Stantec Consulting Ltd. FWD testing serves three important functions when evaluating an existing rigid (concrete) pavement, including providing insight into load transfer efficiency (LTE), differential deflection, and the presence of sub-slab voids. The FWD and LWD tests were completed to understand the strength of the underlying concrete pavement, the LWD testing was done for comparison purposes to determine if it produces similar values as the FWD.

FWD is a non-destructive method of testing where moving wheel loads are simulated. A falling load is dropped on to a plate at a selected portion of the road and the response of the pavement is measured. Using a "Stantec LTPP-FHWA calibrated Dynatest FWD with 9 differential GPS configuration" as seen in Table 3-4 the testing was conducted (Stantec Consulting Ltd. , 2016).

Table 3-4 FWD Sensor Configuration (mm) (Stantec Consulting Ltd. , 2016)

D1	D2	D3	D4	D5	D6	D7	D8	D9
0	300	450	600	900	1200	1500	1800	-300

A seating load of 40 kN was used for the test, the test was conducted on June 2nd, 2016 with an average temperature of 20°C. The testing was conducted on 40 separate locations along the street east to west from Upper James St to W 5th St and then another 40 locations travelling west to east from W 5th St to Upper James St. Three load drops of 40 kN, 55 kN and 70 kN on each pavement location were applied (Stantec Consulting Ltd. , 2016). The 80 test locations were selected randomly along the road, some of them were over distresses such as alligator cracking, while others on good condition pavement. They were conducted in the outer wheel path each of the locations were spray painted and marked with an 'x' for identification purposes.

Figure 3-10 Falling Weight Deflectometer (FWD)

After the testing was conducted at various points along the road, the joints were tested. The vehicle travelled from W 5th St towards Upper James St (west to east) and a total of 40 joints were tested, however not all joints along the wheel path were accessible due to parking on both sides of the road. The load was applied adjacent to the joint, marked with an 'o', three drops were loaded to the west and then to the east of the joint seen below Figure 3-11.

Figure 3-11 FWD Test marking on Jameston Ave.

The analysis from the testing prior to construction is discussed in detail in the analysis portion, Chapter 5. From the testing it was found that most of the transverse joints were not functioning at full capacity. Many of them were below the 70% load transfer efficiency limit and would need immediate repair. It was found that the normalized deflection for the pavement layer was above the maximum capacity, indicating the pavement lacked in stiffness. Lastly, voids were detected underneath Section 2 the pavement layer during testing. Due to the poor results from the FWD testing, it was decided that the pavement needed a long-term rehabilitation solution. Concrete overlay was suggested, selection of the type of overlay would be completed after milling the asphalt layer.

A Light Weight Deflectometer (LWD) is a portable Falling Weight Deflectometer and functions similarly to the FWD machine. It is a portable device and can be used to test the subsoil degree of compaction. A release mechanism holds the falling weight at a constant height, the weight drops and transmits the load onto the pavement seen in Figure 3-12. In compliance with health and safety at the University of Waterloo, hard hats were not required during the testing of LWD. The LWD is a manual procedure where human error is introduced, especially if the weight is not dropped properly. The analysis from the testing prior to construction is discussed in the analysis portion, Chapter 5.

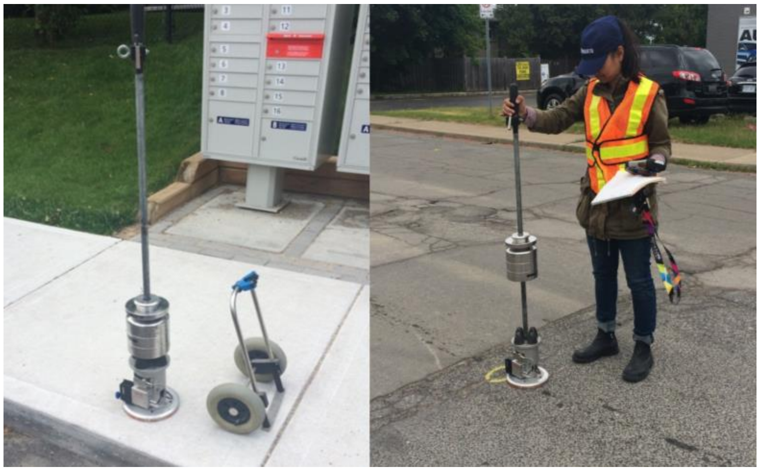

Figure 3-12 Light Weight Deflectometer (LWD) testing on Jameston Ave.

Similar to the FWD, the details of the LWD tests are discussed in the analysis chapter, Chapter 5. The LWD test produced similar deflection results as the FWD tests, which further confirmed a weak pavement layer. Based on both the pavement layer evaluations, concrete overlay was chosen as the rehabilitation method.

3.4 Chapter Summary

- The City of Hamilton needed a more feasible rehabilitation solution for the composite pavements. Jameston Ave. was selected due to accessibility for residents during construction and pedestrian safety. The trial section connects two major arterial roads, therefore would be the perfect candidate to repair.
- The street has been divided into three separate sections, this allows various components of the same street to be tested by having different construction methods. Section 1 and Section 3 are at a higher elevation than Section 2.
- Visual inspection of the existing pavement was conducted, longitudinal cracks all along the road was observed at 1/3rd of the pavements width. Transverse joints are in poor conditions with most of the sealant being deteriorated.
- Section 2 experienced the most distresses along the street with settlement at catchbasins and water accumulation due to lack of drainage.
- Coring report conducted by Peto MacCallum Ltd found the clayey material as the subgrade for Section 1 and 3, Section 2 had topsoil as subgrade layer. The concrete had degraded in the cores for Section 2, gravel and sand was observed in between the layers.
- FWD and LWD tests were conducted on the existing composite pavement. It was found that the transverse joints were functioning at a lower percentage than the minimum requirement of 70%.

4 Construction of Concrete Overlay

4.1 Asphalt Removal

Construction began for the Hamilton overlay on July 25th, 2016. During the week of July 25th, the asphalt was milled off using an asphalt milling machine, the underlying existing concrete pavement was observed for distresses. A micro milling machine was used to completely remove the residual asphalt pavement that varies in thickness between 80-90mm along Jameston Ave. A few of the catch basins and manholes along the road had been patched with concrete, which was also milled off due to visible distresses. Removal of the asphalt layer is necessary in evaluating the existing concrete for joint deterioration and distresses.

4.2 Existing Concrete – Site Evaluation and Design

Once the asphalt layer was removed, on July 25 and 26 of 2016, members of CPATT, CAC and Concrete Ontario visited the site to determine the current performance of the concrete layer. This was done to verify the assumptions made about the condition of the pavement during the original visual inspection. Table 4-1 displays the major distresses observed during the visual inspection.

Table 4-1 Types of distresses and visual observations on concrete (Pickel, et al., 2016)

Type of Distress	Observations
Longitudinal cracks	- Lack of longitudinal joints in the underlying concrete layer resulted in longitudinal cracks to be reflected to the top - The cracks were mainly at 1/3rd of the road, however some appeared along the street at random
Transverse cracks	- Many transverse joints along the road was missing sealant however settlement of joints was not noticed - Transverse cracks formed from the transverse joints, meandering from the joints and expanding

The overall condition of the underlying pavement was poor. Unbonded overlays are chosen as the overlay option when the condition of the street is poor. As mentioned in the literature review, this method would prevent construction delay and reduce rehabilitation costs. Separation layers for each of the sections were chosen based on the individual condition of the street and the City of Hamilton's specifications. Catchbasins and manholes along the street were designed according to the American Concrete Pavement Associations's guide for "Concrete Intersections: A guide for Design and Construction". The catchbasins and manholes were isolated to allow separate lateral movement of the pavement and the structures. Various isolation joint designs are shown from the design guide in Figure 4-1 below. The elevation of the manholes and catchbasins were constructed to match the height of the overlay. The saw cuts for the streets were according to Harrington & Fick's "Guide to concrete Overlays".

In order to have a successful pavement structure, it was recommended that any distresses along the street be filled and leveled for a uniform surface. Several utility cuts were observed along the road, they exposed granular substance what was previously mentioned in the coring report. It was recommended the granular substance is compacted and filled with concrete. It was also recommended that any asphalt patches be removed, and tack coats should not be applied to allow the overlay to act as a separate entity.

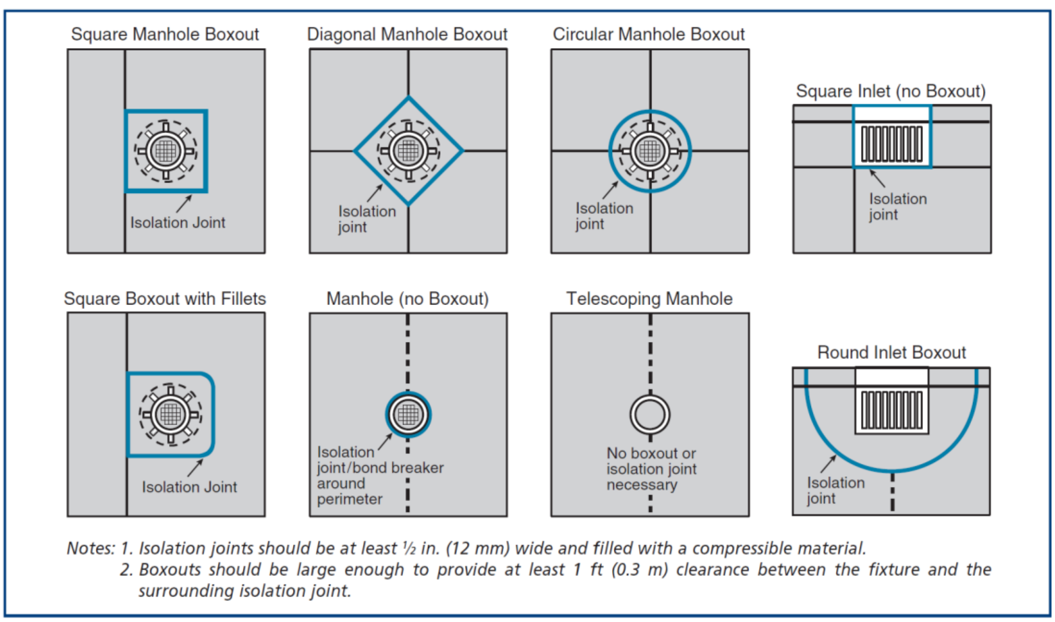

Figure 4-1 Details for boxing out fixtures (ACPA, 2007)

4.2.1 Section 1 – West 5th St. to Hawkridge Ave.

The design for this section was restricted due to the existing sidewalk and curb. The sidewalks and curbs were newly constructed a year before in 2015, removing them would not be economical or environmentally friendly. The concrete base in this section is also thicker than the rest of the street according to the coring report (Peto MacCallum Ltd., 2015). Once the asphalt was milled off, the height from the bottom of the exiting concrete base was measured to the top of the curb. As shown in Figure 4-2, the elevation difference was 225mm. In the City of Hamilton's Public Works Department Standard Road Drawings (RD-103) the specified curb height is 150 mm. In order to meet the specified curb height, the concrete overlay could only be a maximum of 75mm (Pickel, et al., 2016). However, the minimum thickness of a concrete overlay according the Harrington & Fick, 2007 is 100mm. Due to this restriction, it was recommended that a concrete overlay with a thickness of 100mm be used and a geotextile separation layer with a maximum height of 5mm. The City of Hamilton's standard specifications had to be compromised with the curb being 120mm (Pickel, et al., 2016).

Figure 4-2 Height from the bottom of existing concrete to top of curb (Section 1) (Pickel, et al., 2016)

The geotextile thickness was about 3mm and weighed 440 g/m^2 (13 oz/yd^2) (Pickel, et al., 2016). It was recommended to use a white coloured non-woven geotextile to prevent the pavement from becoming overheated, however only black non-woven geotextile was available. The geotextile was recommended to be placed and secured in place with nails once an even base surface had been achieved through repairs. The section has four catchbasins and two manholes, the isolation joints were designed according to ACPA, 2007. For the two manholes, seen in Figure 4-3 below, a rectangular boxout with joints connecting to the corners of the rectangle.

Figure 4-3 Manhole detail for Section 1 (Pickel, et al., 2016)

The driveways in section one protrude onto the roadway, seen in Figure 4-4. Due to this specific design and elevation limitations, the slope was recommended to be saw-cut flush with the vertical outside face of the curb and the overlay elevation matched to the driveway. However, during construction it was determined that no changes should be made to the driveways and the concrete would be poured to keep the original shape. Although, for the driveways on the north side of Section 1, rubberized material was placed along the edge of the sidewalk to allow for expansion and contraction of concrete.

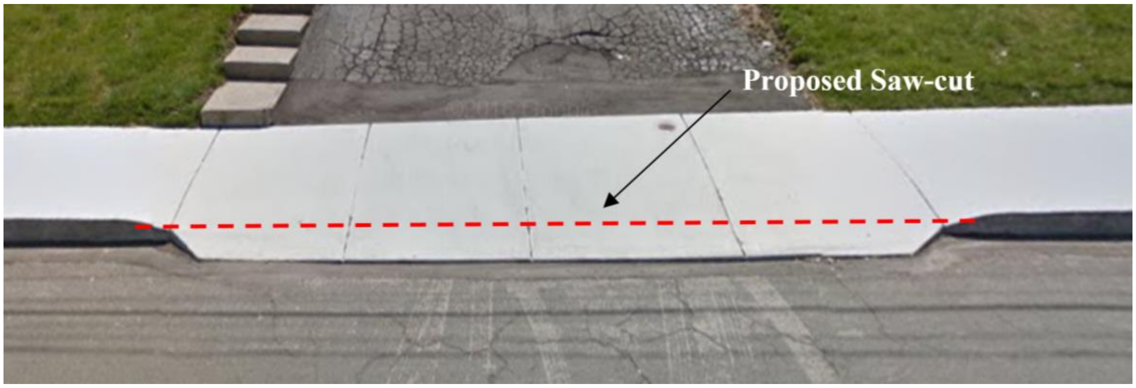

Figure 4-4 Proposed saw-cut for extended driveways Section 1 (Pickel, et al., 2016)

When designing joints for this section, Harrington and Fick's "Guide to Concrete Overlay: Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements" recommend a maximum joint spacing of 1.8m (6ft). For transverse joints it is recommended to not exceed 1.5 times the thickness of the overlay in inches. Therefore, the longitudinal joint spacing was recommended to be 1.5m, however, due to construction efficiency it was decided for the longitudinal joints to be 2.1 m.

The transverse joints were chosen also as 1.5m to match the existing curb and gutter details. It was recommended for the joints for curbs on the north side and south side align with each other, to ensure minimal cracking. During construction it was determined that 1.5m apart longitudinal joints separates the width of the road (8.5m) into 5.6 rows which would be inconvenient to measure. For construction efficiency it was decided for the longitudinal joints to be 2.1 m. This allowed for the joints to be in 4 rows and reduce the number of saw-cuts. Use of polypropylene fibres were suggested for this mix due to thinner thickness and to prevent shrinkage cracks. Table 4-2 below summarizes the design for the entire overlay.

Table 4-2 Summary of unbonded overlay design – Section 1

Section 1	Unbonded Overlay	Note:
Separation Layer	Geotextile	440 g/m^2 weight
Overlay Thickness (mm)	105	The thickness has been based on the design assumptions from Harrington & Fick, 2014
Spacing of Longitudinal Joints (m)	2.1	Longitudinal joints can be a maximum of 1.8m for overlays in thickness less than 125mm. Suggested 1.5m was not construction friendly. Therefore the maximum spacing limit was exceeded and 2.1m was selected
Spacing of Transverse Joints (m)	Spacing to match joints on existing/new curb and gutter, max @1.5	Maximum transverse spacing = 1.5 x overlay thickness in inches = 1.5 x 3.93” = 5.89’ = 1.79m
Saw cut depth (mm)	35	Unbonded overlay – Longitudinal and Transverse joint depths should be one third of the overlay thickness
Saw cut thickness (mm)	3	To eliminate the need for sealing
Concrete	**32 MPa Exposure Class C-2, 0.9 kg/m^3 fibrillated polypropylene fibres (mix 2)**	

4.2.2 Section 2 – Hawkridge Ave. to Caledon Ave.

The condition of the curb and gutters in Section 2 was poor and needed to be newly constructed. This allowed more flexibility for the thickness design and type of separation layer, unlike Section 1. When observing Section 2, the existing seemed to be the worst condition of all three sections. There were more dugouts and loose granular material that needed compaction. Due to the condition of the pavement, it was decided a thicker separation layer would be suitable. Therefore, 25 mm layer of Hot Laid 3 (HL3) high-stability asphalt was selected to separate the overlay from the existing pavement (Pickel, et al., 2016).

The minimum thickness of the overlay is suggested to be 100mm according to Harrington and Fick’s “Guide to Concrete Overlay: Sustainable Solutions for Resurfacing and Rehabilitating Existing Pavements”. With the overlay being 100mm and separation layer being 25mm the sidewalk height was raised to ensure 150mm of difference of curb height from the bottom of the overlay to the top of the curb. To test the difference in behavior due to fibres and non-fibres along the street, Section 2 was selected not to have any polypropylene fibres.

There are four manholes and four catchbasins along Section 2, this section is in the lowest point of elevation. This section was experiences more storm water flow than the rest of the street due to the elevation. Saw-cuts and joints were designed the same way as Section 1, to align with the new concrete curb at 1.5m. Table 4-3 below summarizes the design for Section 2 unbonded overlay.

Table 4-3 Summary of unbonded overlay design – Section 2

Section 2	Unbonded Overlay	Note:
Separation Layer	HL3	25mm HL3 Hot mix asphalt
Overlay Thickness (mm)	100	The thickness has been based on the design assumptions from Harrington & Fick, 2014
Spacing of Longitudinal Joints (m)	2.1	Longitudinal joints can be a maximum of 1.8m for overlays in thickness less than 125mm. Suggested 1.5m was not construction friendly. Therefore the maximum spacing limit was exceeded and 2.1m was selected
Spacing of Transverse Joints (m)	1.5	Maximum transverse spacing = 1.5 x overlay thickness in inches = 1.5 x 3.93" = 5.89' = 1.79m
Saw cut depth (mm)	35	Unbonded overlay – Longitudinal and Transverse joint depths should be one third of the overlay thickness
Saw cut thickness (mm)	3	To eliminate the need for sealing
Concrete	**32 MPa Exposure Class C-2, no fibres (mix 1)**	

4.2.3 Section 3 – Caledon Ave. to Upper James St.

Similar to Section 2, the curb and gutter needed replacement for Section 3. Therefore, it was also recommended to use asphalt as a separation layer; 25 mm layer of Hot Laid 3 (HL3) high-stability asphalt. Section 3 needed to be opened earlier than the rest of the street due to Alexanian's Furniture needing access for delivery trucks. To ensure the street would be structurally capable for traffic, high early strength mix was recommended. The recommended minimum thickness was also 100mm for the concrete overlay in Section 3 and the height of the gutter was adjusted to ensure 150mm height difference.

With Section 2 and Section 3 having the same separation layer, it was recommended to use polypropylene fibres for comparison with Section 2. Polypropylene fibres will be fibrillated and to be added at a dosage of 0.9 kg/m^3 as per the manufacturer's recommendation (Pickel, et al., 2016). Three catchbasins and two manholes were observed on Section 3, the catchbasins are at the lower portion of the street. Similar to the rest of the street, the saw-cuts on the concrete overlay would match new concrete curb saw-cut spacing. Table 4-4 below summarizes the design for Section 3 unbonded overlay.

Table 4-4 Summary of unbonded overlay design – Section 3

Section 3	Unbonded Overlay	Note:
Separation Layer	HL3	25 mm Hot mix asphalt layer
Overlay Thickness (mm)	100	The thickness has been based on the design assumptions from Harrington & Fick, 2014
Spacing of Longitudinal Joints (m)	2.1	Longitudinal joints can be a maximum of 1.8m for overlays in thickness less than 125mm. Suggested 1.5m was not construction friendly. Therefore the maximum spacing limit was exceeded and 2.1m was selected
Spacing of Transverse Joints (m)	1.5	Maximum transverse spacing = 1.5 x overlay thickness in inches = 1.5 x 3.93" = 5.89' = 1.79m
Saw cut depth (mm)	35	Unbonded overlay – Longitudinal and Transverse joint depths should be one third of the overlay thickness
Saw cut thickness (mm)	3	To eliminate the need for sealing
Concrete	**32MPa Exposure Class C-2, 0.9 kg/m^3 fibrillated polypropylene fibres (mix 3)**	

4.3 Concrete Overlay Design

Once the recommendations for each section were made, the final mixes were confirmed and subsequently supplied by Lafarge Inc Table 4-5. Mix 1 without fibre was used in Section 2, mix 2 with fibre was used in Section 1 and Section 3. Finally, mix 3 high early strength was only used in Section 3. In fast track paving, use of high-early-strength mixture allows traffic to open within a few hours after concrete is placed. High-early-strength concrete on Jameston Ave. was used to achieve a specified strength of 20MPa in 24 hours for half of section 3 due to truck traffic near the furniture store Alexanian and weekend church services. The remainder of Section 3 was paved with mix 2. Figure 4-5 displays the plan view of Jameston Ave. with the mix design number for each section.

Table 4-5 Final Mix Designs – Jameston Ave.

	Mix Number (3 mixes submitted)	1	2 With F100 Fibres	3 With F100 Fibres
	Mix Code	RMXC32M511X	RMXC32M511X	RMXN23225XY
SPECIFICATIONS ITEMS as per CSA A23.1 - 2009	Mix Description	CITY 32MPA 20MM C2	CITY 32MPA 20MM C2	CHRONOLIA (20MPA @ 24HR) 32MPA C2 20MM
	Structural Requirements			
	- CSA Exposure Class	C2	C2	C2
	- Maximum W/CM Ratio (if Specified)	0.45	0.45	0.45
	- Minimum Specified Strength	32	32	32
	- Maximum Aggregate Size	20	20	20
	- % SCM	25% Slag 0% Fly	25% Slag 0% Fly	0%
	Durability Requirements			
	- Exposure to Sulphate Attack	normal	normal	normal
	- Alkali Aggregate Reactivity	normal	normal	normal
	- Other	--	BASF F100 FIBRE AT 0.9KG/M3	BASF F100 FIBRE AT 0.9KG/M3
	Architectural Requirements			
	- Colour	--	--	--
	- Other	--	--	--
CONTRACTOR ITEMS	**Slump Range**	75 +/- 25	75 +/- 25	75 +/- 25
	Plastic Air Range	5 - 8%	5 - 8%	5 - 8%
	Strength/Age	32 / 28 days	32 / 28 days	32 / 28 days
	Other			
	Specialty Information			
	- Initial Set (Delay, Normal, Accelerated)	N	N	20MPA @ 24HR
	Method of Placement	Razorback horizontal screed	Razorback horizontal screed	Razorback horizontal screed

Figure 4-5 Mix design for each section on Jameston Ave.

4.4 Instrumentation Implementation

To monitor the change in behavior of the overlay, strain gauges were designed to be installed at the bottom of the overlay layer. This would validate the design of the overlay and determine the main contributing factor for stress points; environmental or traffic. Reflective cracking on composite pavements occur due to excessive tensile stresses underneath the concrete layer. Strain sensors were placed underneath the overlay to monitor the strain readings in relation to the cracks that may appear on the surface of the pavement.

The vibrating wire strain gauges were purchased from Geokon Inc., model number 4202. These are gauges that can be directly embedded into concrete. To determine the strain in concrete, a vibrating steel wire is tensioned between two blocks. When the concrete expands or contracts due to various factors, the blocks move along with the steel wire. The change in this movement is recorded as change in frequency of the wire (Geokon Inc., 2016). Along with the ability to determine the strain of the concrete, these vibrating stain gauges have a thermistor embedded allowing a more accurate temperature reading.

Six strain gauges were installed along Jameston Ave. with 2 sensors in each section. In Section 1, the instrumentation was designed to be in the right wheel path of the lane going eastbound. In Section 2, the instrumentation was in the right wheel path of the lane going westbound. Lastly, in Section 3, the instrumentation was in the right wheel path of the lane going westbound. Below Figure 4-7 illustrates the sensor locations on the street.

The sensors were placed longitudinal direction (parallel to the direction of traffic) and transverse (perpendicular to the direction of traffic) to see the varying strain difference, Table 4-6. This method will allow a comparison between the different types of separation layer and their impact. The strain gauges were approximately 15mm above the base of the separation layer. This is to understand the most critical stress points at the bottom of the section on a thin overlay. The step-by-step installation of the strain gauges are provided below:

1. Prior to installation, the resistance of the strain gauge wires was checked using an ohmmeter.

2. The wires of the strain gauges were covered in black plastic piping to prevent any damages that may occur during installation on the field. Each sensor was labelled for easier identification during installation.
3. On site, four holes per sensor was drilled, for Section 1, the underlying concrete was drilled, and the geotextile was placed on top and cut to match the holes. For Section 2 and 3, holes were drilled into the asphalt separation layer.
4. Tapcon Concrete Anchors © were placed into the drilled holes and 15mm of Styrofoam was cut into small squares.
5. The Styrofoam was placed underneath the sensors and was fastened with a copper wire and attached to the Tapcon Concrete Anchors ©, Figure 4-6.

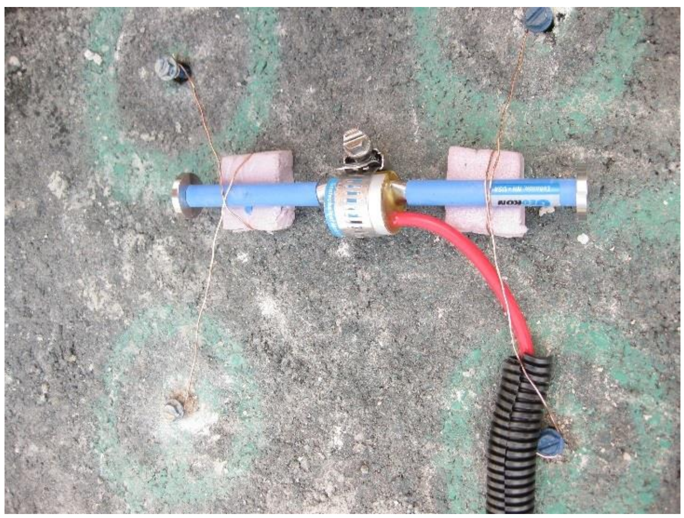

Figure 4-6 Typical Vibrating Wire Strain Gauge along Jameston Ave.

The strain gauges were placed on top of Styrofoam to remain at the lowest portion of the concrete overlay. It was also done due to the overlays not having any reinforcement, this method allowed the concrete to remain embedded in concrete intact without touching the separation layer. During paving it was crucial to ensure the instrumentation does not get damaged during paving, or get moved due to any strong forces. In coordination with the contractor the sensors were placed directly in front of the paver. The encased cables were run across the street through the PVC pipes installed underneath the sidewalk towards the sub-surface mount box. The sensors were covered with a small amount of fresh concrete before the paver went over to prevent any damages, seen in Figure 4-8.

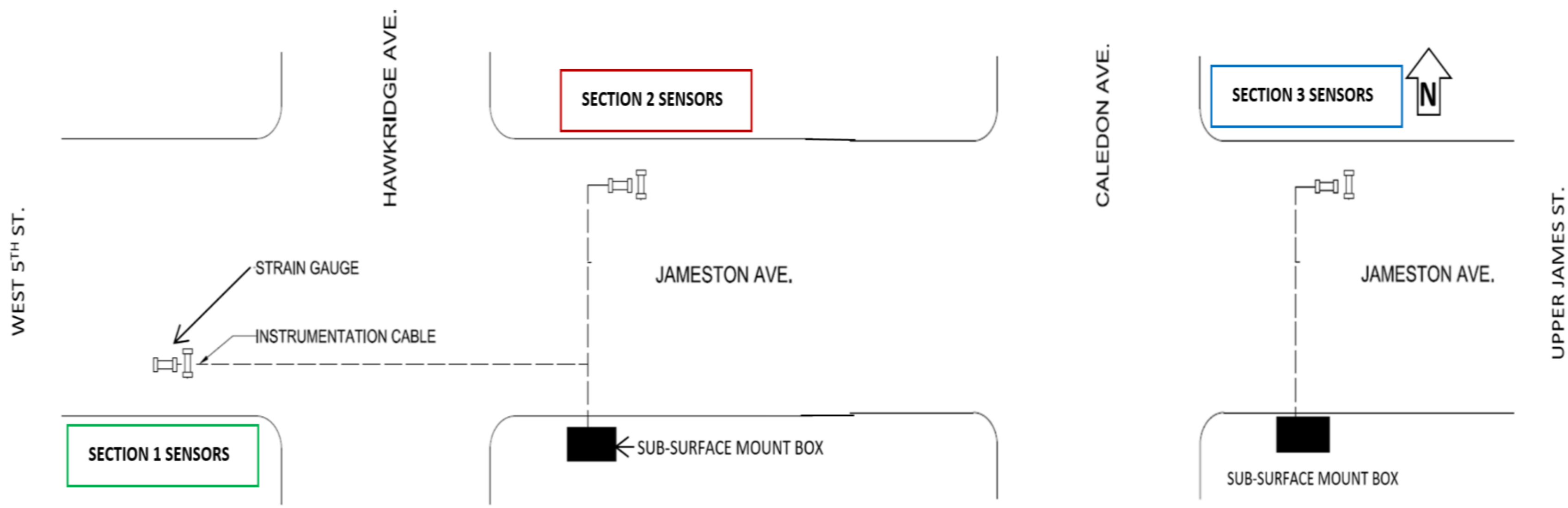

Figure 4-7 Instrumentation layout on Jameston Ave. (Pickel, et al., 2016)

Table 4-6 Orientation and location of Strain Gauges along Jameston Ave.

Section	Strain Gauge Number	Strain location	Orientation
1	S1EB01	15mm above geotextile layer	Transverse
1	S1EB02	15mm above geotextile layer	Longitudinal
2	S2EB03	15mm above asphalt layer	Longitudinal
2	S2EB04	15mm above asphalt layer	Transverse
3	S3WB05	15mm above asphalt layer	Transverse
3	S3WB06	15mm above asphalt layer	Longitudinal

Figure 4-8 Typical sensors being covered using fresh concrete and cable running across the street

4.4.1 Data Acquisition Center

In order to collect the data from the gauges, data loggers were installed in a location where it can be accessed by members of the CPATT team. Based on the overall design involving three test sections, two separate sub-surface mount boxes were installed to accommodate data loggers attached to the various gauges and sensors. The sensor wires from Section 1 and Section 2 run into one sub-surface mount box located at the south-east corner of Jameston Ave. and Hawkridge Ave. The sensor wires for Section 3 runs across the street towards a separate sub-surface mount box located at the south-east corner of Jameston Ave. and Caledon Ave.

The idea of the data acquisition centre was to take the strain readings and temperature readings hourly. The program was designed to take readings every 15 minutes for most of the day, however during rush hour it was programed to record measurements every 5 minutes and also continue to record measurements every 15 minutes. The system observed in Figure 4-9 was set up in March 2017.

Figure 4-9 Sub-surface mountbox with instrumentation box and datalogger

Batteries for the data loggers were replaced once every two months to prevent any loss of data. Members of the CPATT team went out to the site and collected data using a portable laptop through the software "Loggernet: Data Support Software".

Some of the challenges faced with a subsurface mount box in Section 2 and Section 3 was that they are in the lowest elevation point of the section, near a catch basin. This was problematic as there was heavy precipitation in Fall 2016 resulting in water penetrating into the instrumentation box. To remove the water from the instrumentation box, a trench hole was dug to prevent water from accumulating in the sub-surface mount box and gravel was placed for drainage.

Figure 4-10 Trench hole and gravel to allow drainage of water – Section 1

The City of Hamilton created further trenches to direct the flow of water towards the catchbasin on Hawkridge Ave. and Caledon Ave. However, in an attempt to solve the water accumulation issue, damage was caused to the vibrating wires resulting in missing data for 4 months. The screws for the subsurface mount box was also broken during this time. The vibrating wires were re-wired and rebooted during the 4 months on a regular basis. The reboot and change of battery in June had the sensors up and running.

4.5 Construction Timeline

During the week of August 2nd, the sidewalks on section 2 and 3 were removed due to major deterioration, formwork was set up for the new sidewalks. The curb at each intersection was also newly constructed for Section 2 and 3. Once the assessment for all three sections were completed, the entire street was washed and swept with a mechanical sweeper Figure 4-11. All the excess debris were removed. CPATT members conducted LWD and SurPRO testing on the existing concrete and finalized the instrumentation location on all three sections. Location for the sub-surface mount box and sensors were spray painted.

Figure 4-11 Washing and Sweeping on Jameston Ave.

For the week of August 8th, 2016 temperatures in the City of Hamilton almost rose to a high of 33°C and a heat warning was issued (Environment Canada, 2016). Due to the health risks associated with the heat warning as well as potential for rapid curing and drying out of ready-mixed concrete, concrete paving was postponed to the following week. However, sidewalk construction continued, concrete was poured on August 9th, 2016, a joint spacing of 1.5m and a height of 120mm. PVC pipes for the instrumentation were placed before concrete was poured to allow wires from the sensors to run past the sidewalk and into the subsurface mount box location Figure 4-12.

Figure 4-12 PVC pipe below sidewalk for instrumentation

On August 15th, 2016 concrete paving on Jameston Ave. began, construction began at 8:00am. Black geotextile was placed along the Section 1 as the separation layer, geotextile was laid through the intersection to the beginning of Section 2, Figure 4-13.

Figure 4-13 Placement of geotextile on Section 1

The geotextile was cut out at manhole and catch basin locations and along the sidewalk to accommodate the extended driveways, seen in Figure 4-14. The geotextile was not attached to the underlying concrete pavement therefore it moved out of place when construction vehicles including concrete trucks traveled along the section. The movement due to vehicles also caused tears on the geotextile along the section.

Figure 4-14 Geotextile cutout for manholes and catchbasins

Since the overlay on Jameston Ave. was to be unbonded, not many repairs on the existing concrete base was made therefore the geotextile took the shape of deformities along the road. Section 1 was a mixture with 0.9 kg/m^3 fibrillated polypropylene fibres. The first mixing truck that arrived on site did not meet the 75±25mm slump requirement and was rejected, the subsequent trucks on site passed slump and air void requirements. A razorback was used for paving followed by a transverse broom finish, Figure 4-15 .

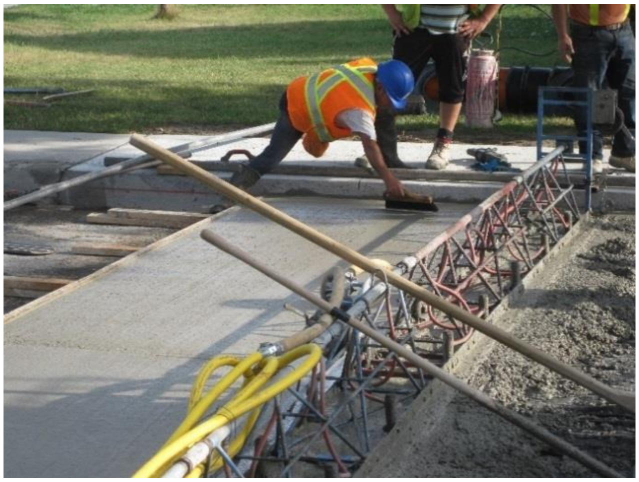

Figure 4-15 Razorback horizontal screed and broom finish

To ensure the geotextile was uniform along the road, a worker oversaw smoothing out the layer as the concrete was being poured. The paving was slow due to the razorback being reversed, once corrected progress was made. A white pigmented curing compound was sprayed subsequently following the broom finish. This white compound protected the concrete from excessive evaporation as well as reflected sun's rays to keep the concrete cooler and prevent heat buildup. It allows the applicator to visibly see whether the proper concrete coverage had been achieved. Best application practice for the curing compound was not followed, many patches along the section were visible seen in Figure 4-16

Figure 4-16 Uneven placement of curing compound

Paving concluded at 5:00 pm on August 15th and a temporary butt joint was constructed, paving stopped at about 2/3 of the section length away from the intersection. Using a compact roller, 25mm of HMA was placed as the second separation layer from 2 to the end of Section 3, paving of HMA started at 11:00am on August 15th Figure 4-17.

Figure 4-17 Placement of HMA separation layer

The asphalt separation layer was crowned to the existing crossfall of 2%. To prevent early cracking, soff-cut of concrete began 5-6 hours after paving. Joint spacing in Section 1 between West 5th St. and Hawkridge Ave. varied, 3.0 m transverse joint spacing before the first set of manholes on Jameston Ave. and 1.5m spacing after the manhole and for the remainder of Jameston Ave seen in Figure 4-18.

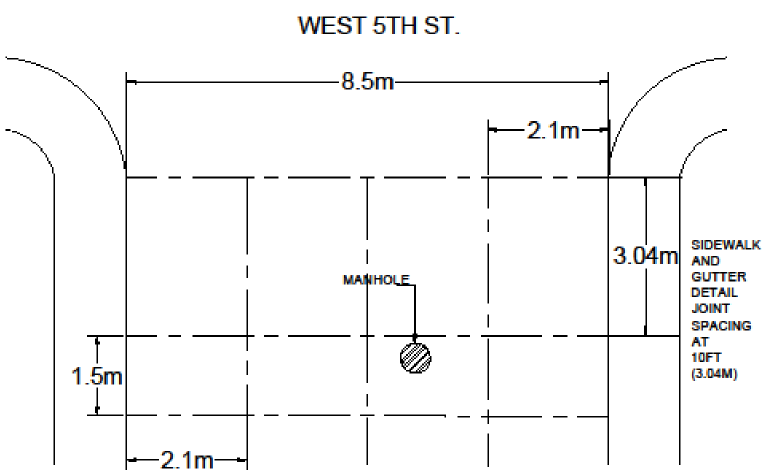

Figure 4-18 Transverse and longitudinal joint spacing

CPATT members conducted slump and air void testing throughout the day and cast cylinders for compressive and tensile testing in accordance with OPSS. Surpro testing on the asphalt separation layer and set up of sensor locations were also completed.

Due to extreme heat alert issued on August 16th, concrete was not poured. Construction continued August 17th at 7:30am. The first set of vibrating wire gages were installed Figure 4-19, paving continued from the end of Section 1 through the intersection of Jameston Ave and Hawkridge Ave. A new batch of concrete containing no polypropylene fibres were sent to the construction site for the beginning of section 2. Once paving started, the second pair of vibrating wires were placed in Section 2.

Figure 4-19 Installation of vibrating wire strain gauges (Section 1)

As paving continued for the rest of the section, like section 1, curing compound was applied and soff cutting began 4-5 hours later in accordance with OPSS. CPATT members conducted slump and air void tests as well as casted cylinders for compressive and tensile testing. Paving stopped on the 17th at 5:30pm about 30m away from the Caledon Ave. However, soff cutting continued later into that evening of August 17th, 2016, Figure 4-20.

The final day of paving began on August 18th at 7:30am, the no polypropylene mix was continued from Section 2 through the intersection of Jameston Ave and Caledon Ave. Polypropylene fibres were used again for Section 3, the last set of sensors were installed by the CPATT members. Paving continued on Section 3 until the first driveway for the New Testament Baptist Church, once reaching the beginning of the driveway the mix was switched to a high early strength concrete with polypropylene fibres Figure 4-21.

Figure 4-20 Soff-cutting of longitudinal and transverse joints

Figure 4-21 Use of high early strength concrete on Section 3

Similar to Section 1 and 2, CPATT members conducted slump and air void tests and casted cylinders for compressive and indirect tensile strength testing. The concrete was cured as well as soff cutting began similarly to the other sections. Paving completed on August 18th at 5:30pm, however soff cutting continued.

On August 19th, paving on Hawkridge Ave. and Caledon Ave. was completed to create the transition from the concrete overlay on Jameston Ave to the adjoining streets. On August 20th, the manholes and catch basin were placed, as well concrete poured in the box outs (Figure 22). Lastly, the sub-surface mount boxes were installed.

Figure 4-22 Concrete Poured in box-outs

4.6 Fresh Properties

During construction, various tests and observations were made to ensure the concrete was in suitable condition to be opened to the public and will perform well. Initial pavement assessments were conducted to document any early cracking or distresses. Air void and slump tests were done to ensure the construction process and quality of concrete was satisfactory. Compressive and indirect tensile strength tests were completed in the CPATT lab at 3, 7 and 28 days. SurPRO profiling test was done on the various layers of the pavement to monitor the thickness as well as the indirect roughness index (IRI). The British Pendulum Friction test as well as T2GO test were conducted to ensure the initial friction of Jameston Ave. was satisfactory.

4.6.1 Air Content

It is crucial to maintain the quality of materials during construction; therefore, air content and slump of the ready-mixed concrete was frequently tested. The air content was measured using the pressure method (CSA A23.2-4C), where the applied pressure compresses the air within the concrete sample, including the pores of aggregate, Figure 4-23. Too much air can decrease the density and increase workability. Variations in air content can be expected with variations in aggregate proportions and gradations, mixing time, temperature, and slump. Consistency in batching is needed to maintain adequate control. Therefore, it was crucial to maintain the air content range between 6-8% for this mix. The resistance of hardened concrete to freezing and thawing in moist condition is significantly improved using entrained air.

Figure 4-23 Pressure method testing on Jameston Ave.

4.6.2 Slump Test

The slump test is the most generally accepted method used to measure the consistency of concrete and it was tested using CSA A23.2-5C. The concrete was placed into the conical mould in three separate layers and rodded 25 times. A high slump value is indicative of a more fluid concrete. Air content can also increase as slump increases; therefore, it was crucial to maintain the slump between 75±25mm as seen on Figure 4-24. The slump and air content for each section was also collected by Lafarge.

Figure 4-24 Conducting slump test on Jameston Ave.

Table 4-7 through to Table 4-9 below outlines the air content and slump for all three sections, in truck numbers marked with * indicate high-early strength concrete mix. Based on the numbers displayed the slump and air content percentage were maintained. As previously mentioned, high-early strength concrete mix was used to open up Section 3 earlier as the furniture store and church needed access to the driveways.

Table 4-7 Slump and air content of truck load on Section 1

Truck Number	Slump (mm)	Air Content (%)	Concrete Temp ◦C	Air Temp ◦C
1	80	7.8	26	18
2	90	6	26	20
3	85	6.4	27	24
4	90	6.4	28	25
5	90	7.2	27	25

Table 4-8 Slump and air content of truck load on Section 2

Truck Number	Slump (mm)	Air Content (%)	Concrete Temp ◦C	Air Temp ◦C
1	100	7	23	20
2	85	7	24	20
3	80	5.8	26	20
4	100	7	26	22
5	85	6.4	27	24
6	115	6.3	25	25
7	115	7.6	25	27
8	95	7.0	26	28
9	90	6.6	26	28
10	120	7.0	25	28
11	100	6.8	25	27
12	100	6.6	25	28

Table 4-9 Slump and air content of truck load on Section 3

Truck Number	Slump (mm)	Air Content (%)	Concrete Temp ◦C	Air Temp ◦C
1	80	6	25	20
2	90	6.2	24	21
3	100	6.2	25	21
4	90	6.5	25	23
5	70	6.8	24	26
6	90	7.5	25	27
7	80	7.2	25	26
8	95	5.8	25	27
9*	105	6.5	25	27
10*	110	6.8	23	27
11*	135	6.8	23	27
12*	100	6.0	23	27
13*	115	7.0	23	27

4.7 Hardened Properties

Compressive strength and in-direct tensile strength tests determine the hardened properties of concrete. Using the test results the strength of the concrete can be monitored. In the case of pavement design it is crucial to test the compressive and tensile strength of the concrete; it is an indication of when the road is safe enough to be opened to the public. The testing was completed every 3, 7 and 28 days to ensure proper hydration of concrete. The sections below further discuss the procedure that was followed and analysis of test results.

4.7.1 Compressive Strength and In-Direct Tensile Strength

Pre-moulded specimens for field and laboratory strength testing was made and cured in accordance with CSA A23.2-3C. Using cylinders of 100 x 200 mm, about 40 were cast each day of construction. Since the slump was greater than 40mm, the test cylinders were rodded and filled in three equal layers. The strength of the test specimen can be greatly affected by changes in temperature, exposure to drying and jostling. Therefore, to ensure minimum changes, they were cast in coolers and placed on even surface to prevent subsequent movement. The samples were then brought into the CPATT lab after 24 hours and de-moulded. They were placed in moist curing room in accordance to CSA A23.2-3C. All test specimen was end ground to ensure accuracy, seen in Figure 4-25.

Figure 4-25 End grinding of concrete cylinders

Compressive and Splitting Tensile Strength tests were conducted per CSA A23.2-9C and 13C respectively. The concrete cylinders were tested at 3 days, 7 days and 28 days after casting. Based on the testing conducted in the CPATT lab it was found that the average of all compressive strength tests was equal or exceeded the specified 28-day strength, which was to be at least 32MPa. The 3-day testing was done to determine if the road could be opened and withstand traffic; compressive strength of 13 MPa. The 7-day cylinders monitor early strength gain.

All three mixtures performed well, the mixture for Section 3 with high-early strength concrete and polypropylene fibres had the highest average compressive strength for all three days of testing. The mixture in Section 2 with no-fibre, had the lowest compressive strength as expected. The addition of fibres is known to add reinforcement and strength to the concrete. The average compressive strength results for all three sections can be seen in Figure 4-26.

Tensile strength is also an important property to determine as concrete is known be highly vulnerable to tensile cracking. The polypropylene fibres were added into the mixtures to aid in the strength of concrete. However, based on the testing conducted, it seems that mixture in Section 2 without fibre had the highest tensile strength at 28 days, this may require further investigation. However, mix in Section 3 for the high early strength with fibre, is the highest for both 3 days and 7 days in comparison to the other mixes as expected. The average tensile strength can be seen in below in Figure 4-27. The mixture performed well based on the observations made from the split cylinder samples, on one side seems to be rough while the other side has depressions indicating the aggregate is strong, Figure 4-28.

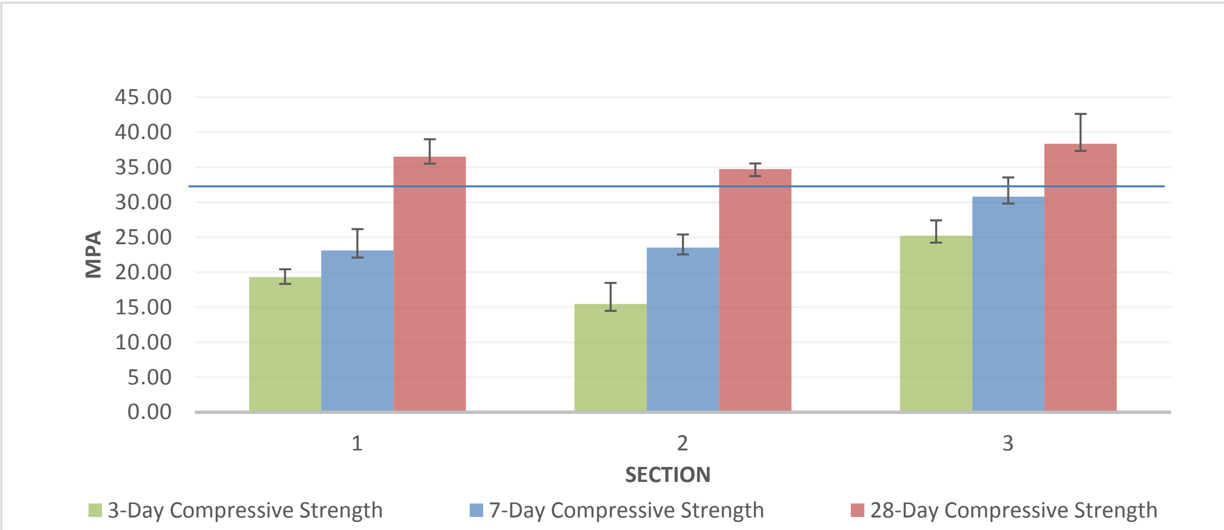

Figure 4-26 Average compressive strength for Section 1-3

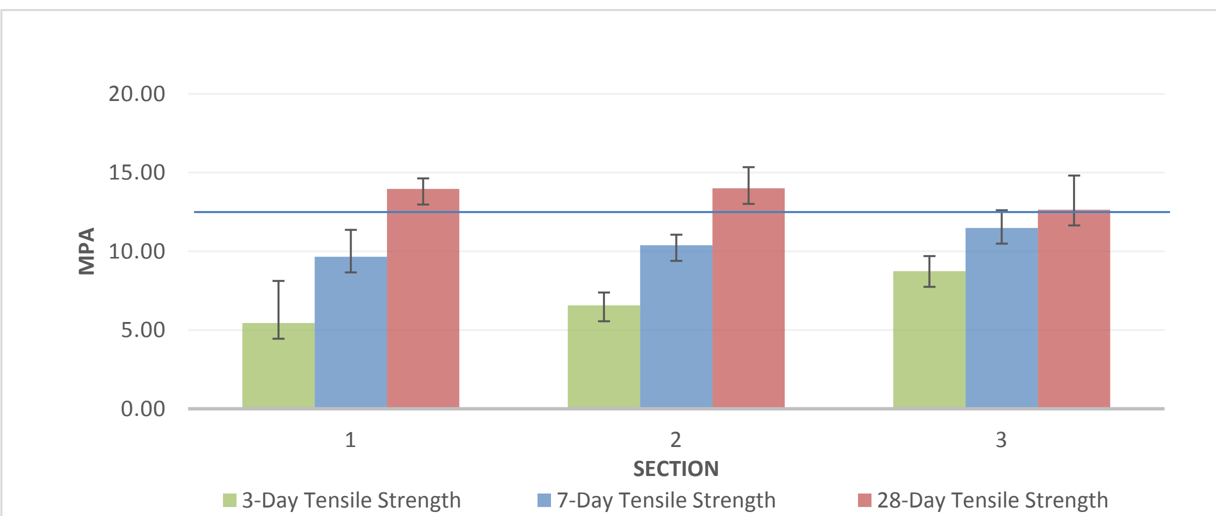

Figure 4-27 Average in-direct tensile strength for Section 1-3

Figure 4-28 Split cylinder after tensile testing

4.8 Chapter Summary

- Visual inspection of the underlying concrete was examined once the asphalt layer was milled off. From the inspection it was found that the condition of the base was poor and had experienced a lot of longitudinal and transverse cracks.
- Based on the visual inspection, it was determined that an unbonded overlay would be the best solution for Jameston Ave. Each section was designed according to guidelines.
- Section 1 was designed to use a geotextile separation layer due to curb and sidewalk height limitations. In order to meet the City of Hamilton's guidelines, the separation layer had to be thinner as well as the thickness of the overlay. Use of polypropylene fibers would be used as reinforcement for the overlay.
- Section 2 was designed to use an asphalt separation layer. The curb and sidewalk needed to be re-constructed allowing the use of thicker separation layer and concrete overlay, polypropylene fibers were not used for this section.
- Section 3 was designed to also use an asphalt separation layer the design of the overlay was very similar to Section 2. However, fibers were used for Section 3 and high-early strength mix used for the second half of Section 3.

- Challenges were experienced during placement of the geotextile and use of a horizontal razorback screed. Proper practice of spraying curing compound should be followed for future projects. The concrete should be cut within 3-5 hours after placement to ensure clean saw-cuts.
- Two vibrating wire sensors were placed per section, one longitudinally and one transverse to better understand the behaviour of the concrete.
- Fresh and hardened concrete properties indicated the overlay to be behaving above the minimum requirement. Test results from 7-day compressive and tensile strength allowed the street to be opened to the public.
- Visual inspection conducted immediately after construction showed the street to be free of distresses. A transverse crack has been noted in Section one, protruding from one of the catch basins, Figure 4-29.

Figure 4-29 Transverse crack in Section 1 close to catchbasin

5 Unbonded Concrete Overlay Evaluation

Once the road was constructed the street was assessed various times of the year to monitor performance. Before the road was opened for traffic, non-destructive tests such as British Pendulum, T2Go and SurPRO. A year after the street was opened to the traffic, FWD, SurPRO, British Pendulum and T2GO were conducted again compare any noticeable changes within the year. Sensor data that was collected throughout the year was analyzed. Lastly, visual condition was done on the pavement to validate pavement design.

5.1 Vibrating Wire Strain Gauges Evaluation

As previously mentioned in Chapter 3, the data was collected in an hourly basis throughout the course of eight months. The vibrating strain gauges collected the raw stress values and the concrete temperature. Sensors S1EB01-S1EB04 did not function for about three months between April to July. This was due to drainage issues where water was accumulating in the subsurface mount box, in an attempt to fix the water issues, damages were made to the sensor wires causing no collection of data during that period. The sensors were fixed again in July and began to collect normally, with another outage in mid-August to beginning of September. The sensors are currently functioning very well. Sensors S3WB05 and S3WB06 also experienced the same outage however, they were able to be fixed at a much earlier date, no data was recorded between end of April to beginning of June, they have not experienced any outages since.

Quality control check was completed each time the data was downloaded from the datalogger. The initial strain values were recorded and used as the baseline to determine the strain of the concrete. To determine the actual strain of concrete, the measurement and correction for temperature effects were considered. Using Geokon Inc, 2016 instrumentation manual, the actual strain of concrete was calculated using Eq. (4-1). The concrete is restrained which only allows the vibrating wire to expand, if the concrete was free the coefficient of expansion of concrete would be used as well.

$$\mu_{actual} = (R_1 - R_0)B + (T_1 - T_o)C_1 \qquad (4-1)$$

Where:

μ_{actual}= actual strain

R_1= Current stress reading

R_0 = initial stress reading

T_1= Current temperature

T_o= Initial temperature

B = Batch Calibration Factor (0.975)

C_1= Coefficient of expansion of Steel

The data set was then checked again to remove any erroneous outliers before analysis was completed. A negative strain value indicated the concrete was in compression while a positive value indicated tension for the concrete. Figure 5-1shows the strain for all six sensors on Jameston Ave. from March 2017 to Nov 2017.

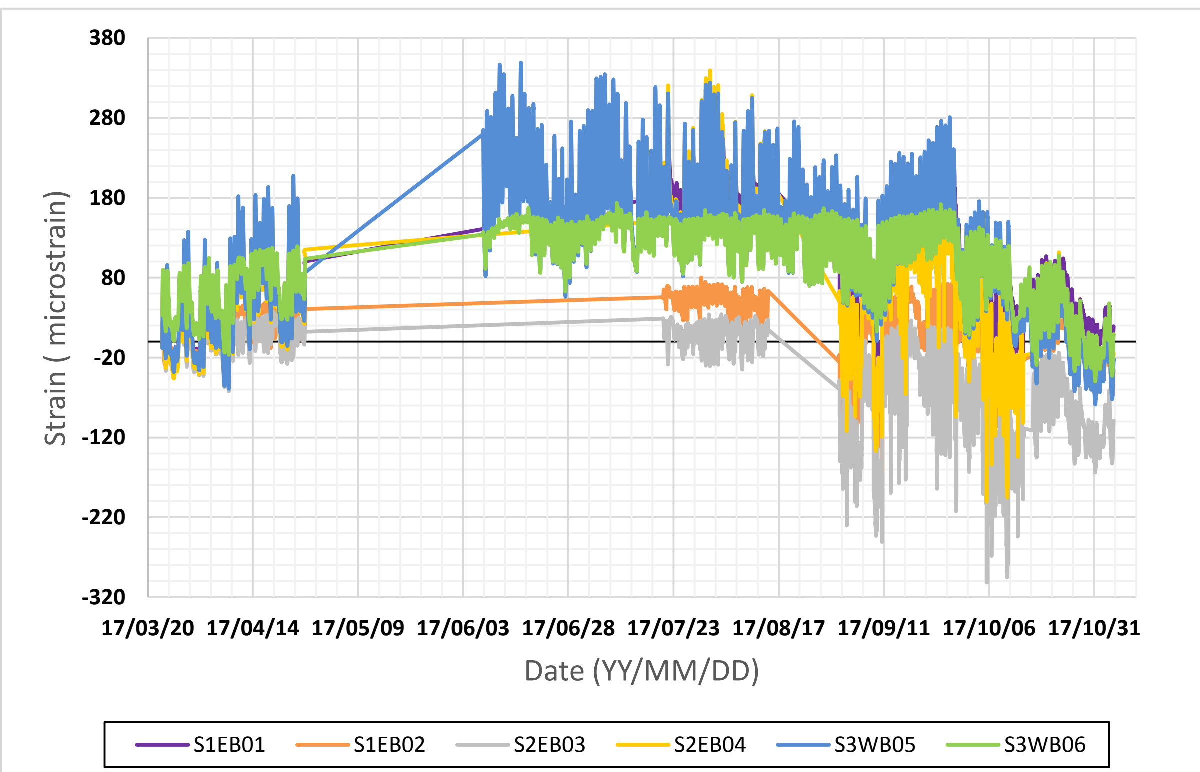

Figure 5-1 Strain for 6 vibrating strain gauges along Jameston Ave. (March-Nov)

5.1.1 Behaviour of Overlay due to Temperature

As previously discussed in the literature review, various factors contribute to the stress points along a pavement layer, when stress accumulates it can cause reflective cracking. Temperature is a key component to the cause of stress on pavement layers, and this pattern is seen throughout the data for Jameston Ave. Based on historical data, it was found that at 2am the temperature is the lowest and at 2pm the temperature is the highest (Xia, et al., 2008). Similarly, when observing the change in strain vs. the temperature in Figure 5-2, it was seen that on a typical summer day, the temperature is the lowest around 7am in the morning, it began decreasing from 12am. The temperature is the highest around 4:30-5pm, it began steadily increasing from 11am.

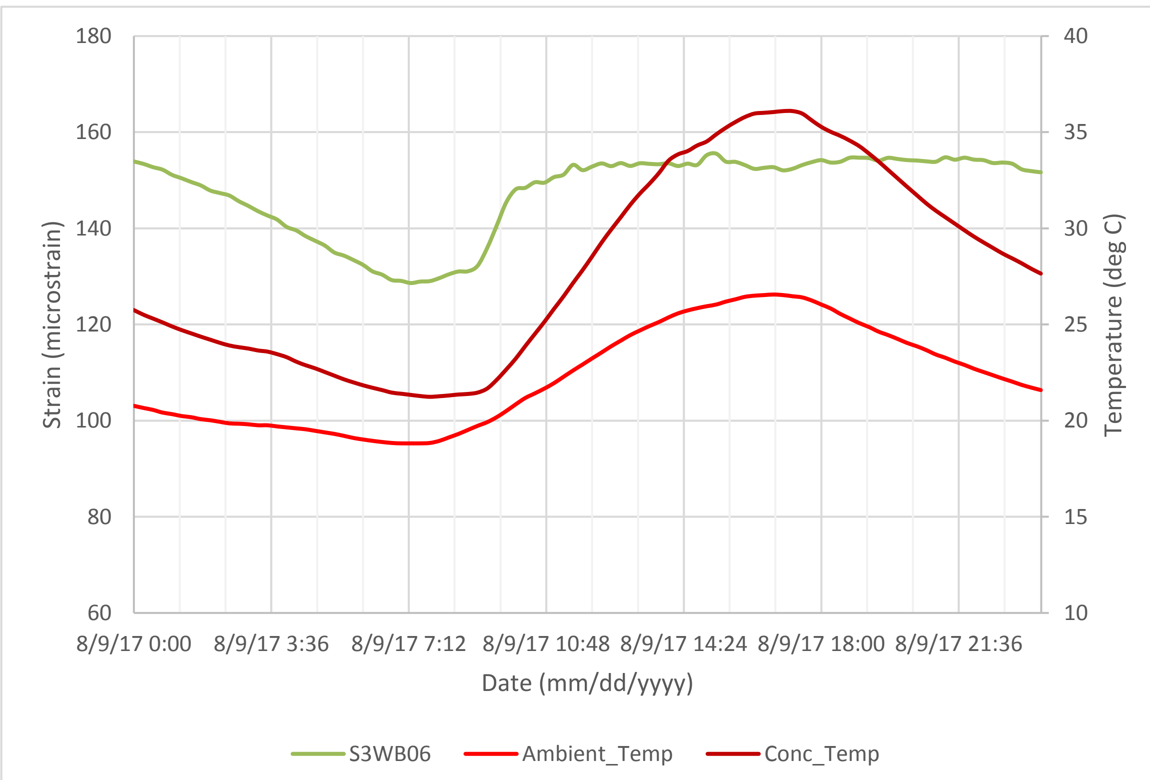

Figure 5-2 Change in strain vs. temperature for 24 hours (August 9th, 17)

When the ambient temperature increases due to solar radiation, the concrete temperature slowly begins to rise as well. The concrete temperature is higher since the gauge is embedded into the concrete in between layers, preventing heat from escaping; the separation layer for strain gauge S3WB06 is asphalt which absorbs more heat due to its dark hue, along with sensors in Section 2 also with asphalt and Section 1 with black non-woven geotextile. The temperature increase causes the concrete to expand and increase in strain, indicating tension. As the ambient temperature decreases at night, the concrete temperature also decreases but remains to be higher than the ambient temperature. Concrete begins to contract with decreasing temperatures causing more compression seen around 7am. The stresses caused throughout the day would not accumulate if the concrete was free, however it is restricted by other slabs and the sidewalk causing buildup.

Concrete goes into compression as the temperature drops and tension as the temperature increases. To confirm the theory, the street was monitored during different seasons. It was found that during the spring as the temperature fluctuates, the strain follows suit Figure 5-3. The maximum tensile strain in the spring remains below the tensile threshold of 200 microstrain (Kim, et al., 2015). Sensor S2EB03 however decreased in strain as the temperature increased for a portion of the test sample, on the 12th of April however, it increased in strain with the temperature.

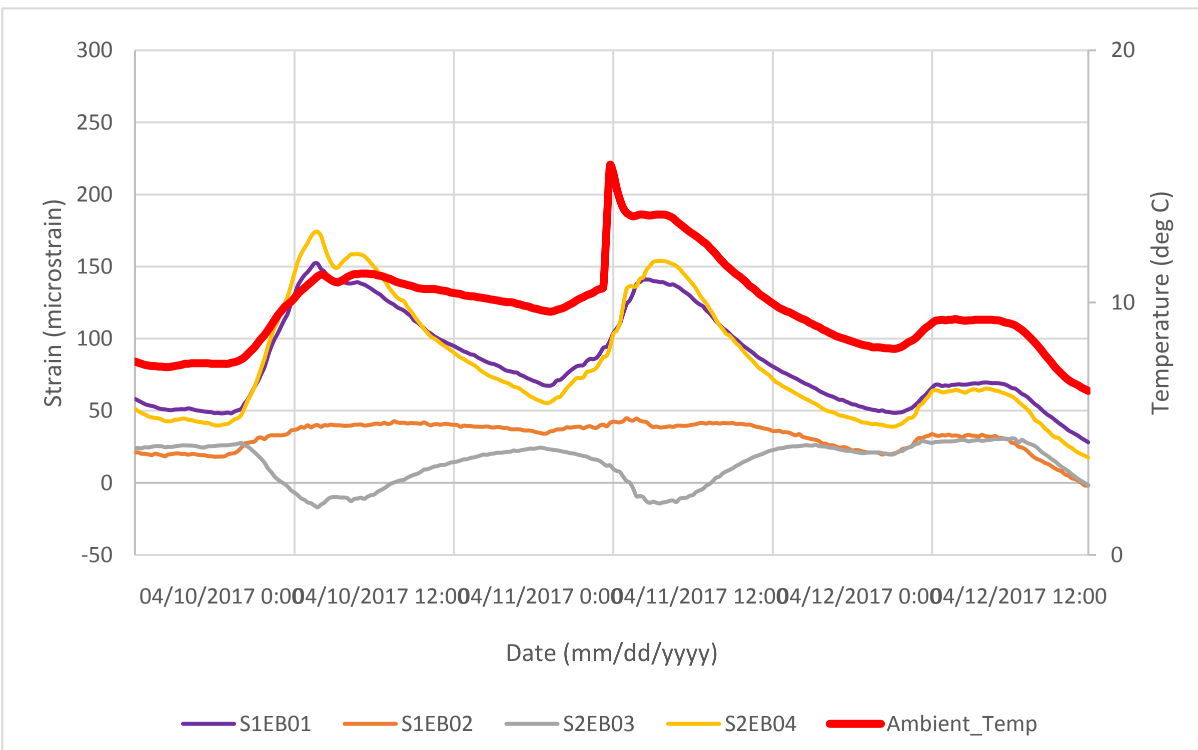

Figure 5-3 Change in strain vs. ambient temperature – Spring (April 10th-12th)

Sensor S2EB03 and S2EB04 with asphalt separation layer experienced higher concrete temperatures than Sensor S1EB01 and S1EB02. However, the sensors in asphalt had higher tensile strengths, which could be an indication of the type of separation layer used. The geotextile is not as efficient being a stress absorbing layer as asphalt. This could also be due to the geotextile taking the shape of the unrepaired distresses on the existing concrete.

During the summer season, the change in strain was the highest as expected due to higher ambient temperatures Figure 5-4. It also caused the separation layers to absorb more heat and stay heated for longer hours. For all four sensors S1EB01-S2EB04, the strain intensified with temperatures reaching a maximum of 33 deg C (geotextile separation layer sensors) and 40 deg C (asphalt separation layers). The strain gauges on top of geotextile exceeded the tensile threshold of 200 microstrains, this could be a cause of concern in the future and the section should be monitored.

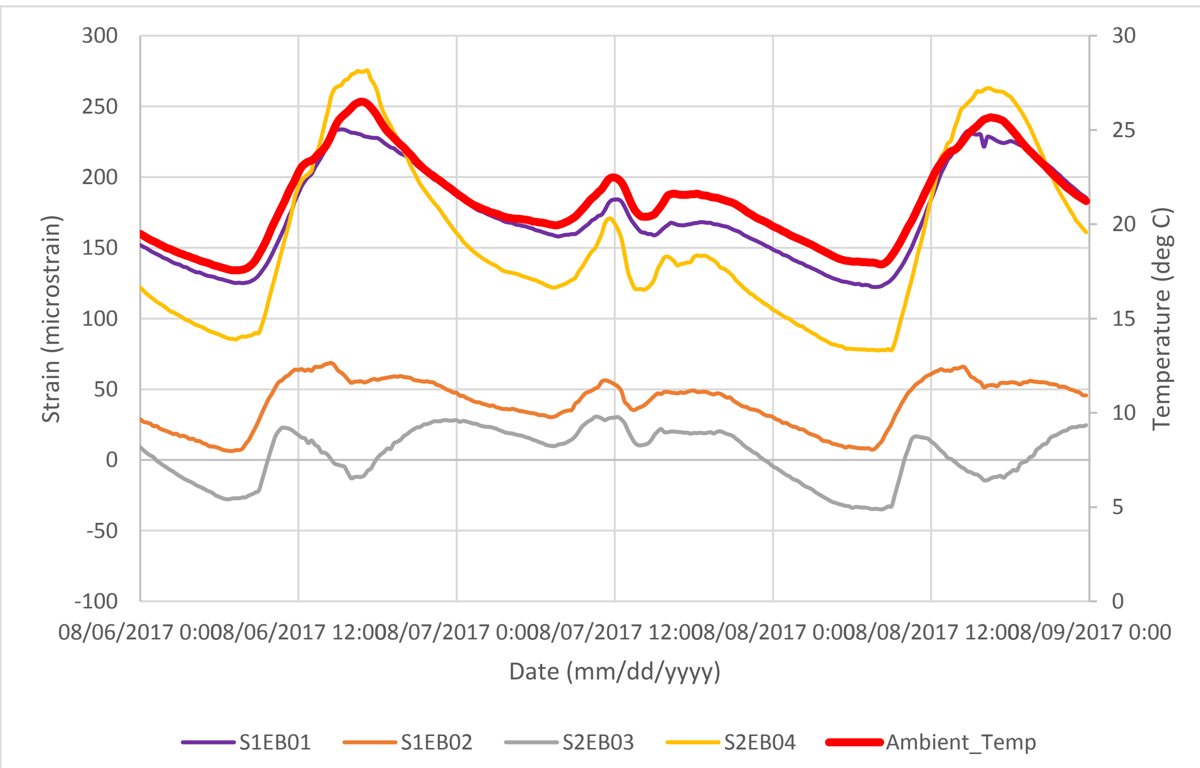

Figure 5-4 Change in strain vs. ambient temperature – Summer (August 6th – 8th)

The temperature became the lowest during the fall, with a drop-in temperature the tensile strain began trending towards compression. Both the sensors in Section 2 (asphalt separation layer) and one sensor in the geotextile layer (S2EB04) experienced compression during the fall. It is predicted that the strain will continue to decline with temperature for the remainder of winter and begin increasing in spring.

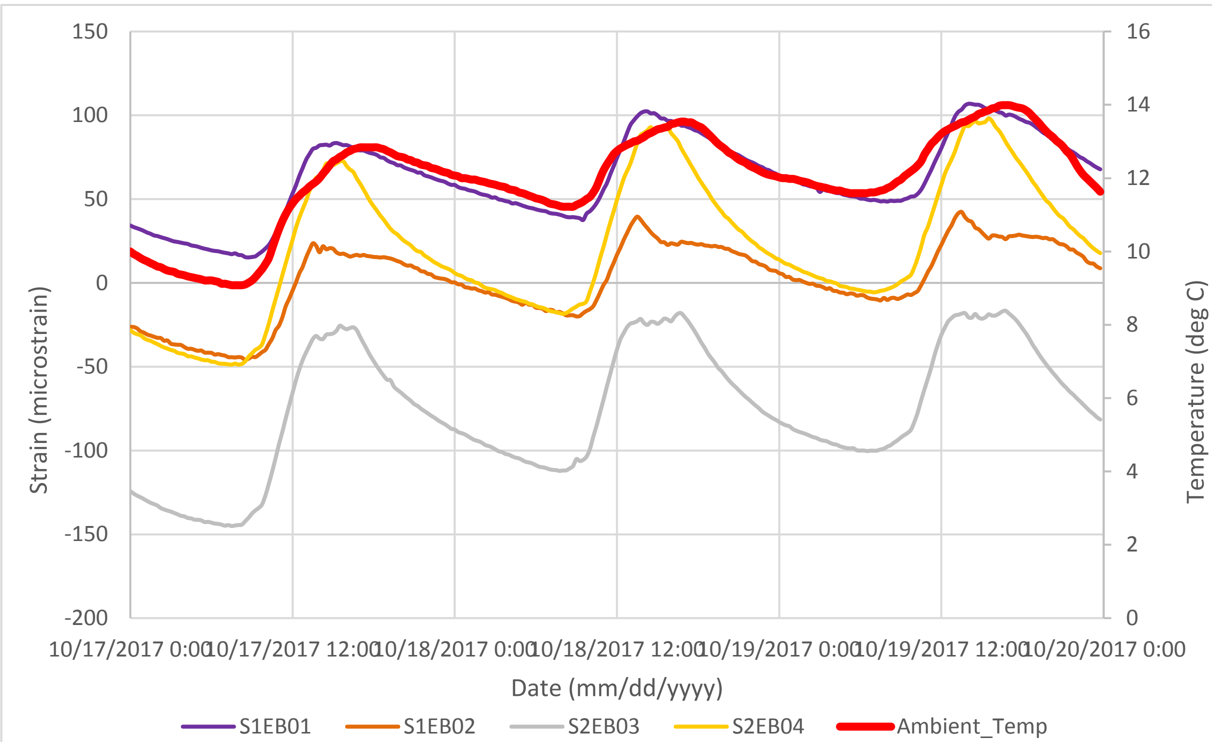

Figure 5-5 Change in strain vs. ambient temperature – Fall – (October 17th -19th)

When observing the range of strains for the duration of 8 months, the sensors placed in a transverse orientation (S1EB01, S2EB04, S3WB05) experience more tensile strain than longitudinal sensors (S1EB02, S2EB03, S3WB06) seen in Figure 5-6. This is potentially due to the transversely oriented sensors being more restricted due to the sidewalk than the longitudinal sensors. Section 3 has the highest tensile strain for both longitudinal and transverse orientation. Traffic, load transfer deficiency, temperature could be contributing to the higher values.

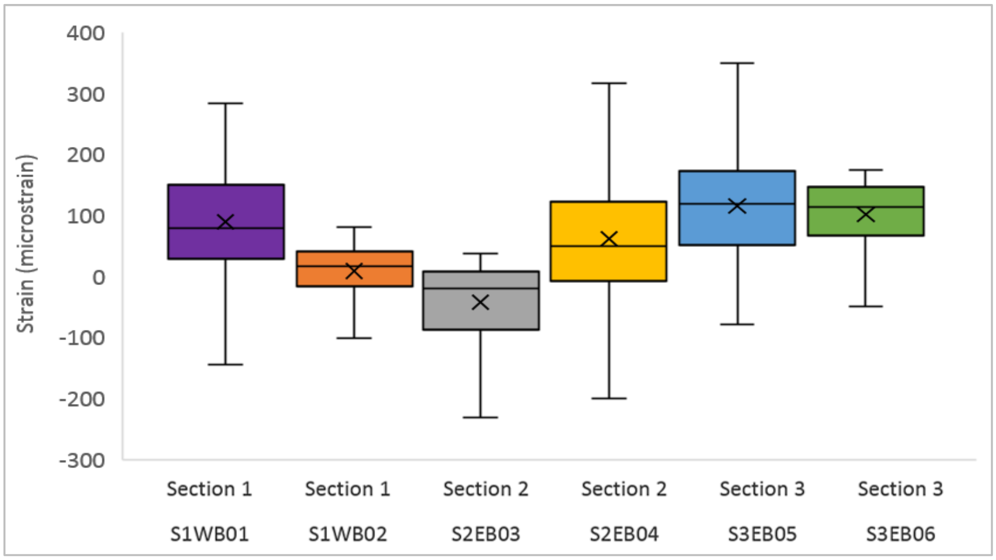

Figure 5-6 Range of strain on Jameston Ave. for 6 sensors

To better understand how heavily temperature impacts the measured strain, a regression analysis was completed to see the relationship between the two variables. Figure 5-7 below shows a directly proportional outcome, whereas the strain increases the temperature does as well. In can be seen from Table 5-1 that sensors that have been placed transversely have a very high correlation to temperature, ranging from 0.91 – 0.97. Strain gauges placed in a longitudinal orientation seem to have a varying range in correlation from 0.24(poor) to 0.52(fair) to 0.83(good). The temperature may not be the main cause of stress for the longitudinally placed sensors. This indicates concrete expanding and contracting more freely in the longitudinal orientation, parallel to the direction of traffic and causing less tensile stresses.

Table 5-1 Strain vs. temperature regression analysis

Section	Strain Gauge Number	Orientation	Linear Regression Equation	R^2
1	S1EB01	Transverse	Ɛ=10.36T - 61.85	0.97
1	S1EB02	Longitudinal	Ɛ=4.64T - 62.33	0.57
2	S2EB03	Longitudinal	Ɛ=3.86T - 106.48	0.24
2	S2EB04	Transverse	Ɛ=10.51T - 111.24.	0.91
3	S3WB05	Transverse	Ɛ=10.59T -105.53	0.93
3	S3WB06	Longitudinal	Ɛ=5.79T - 20.86	0.83

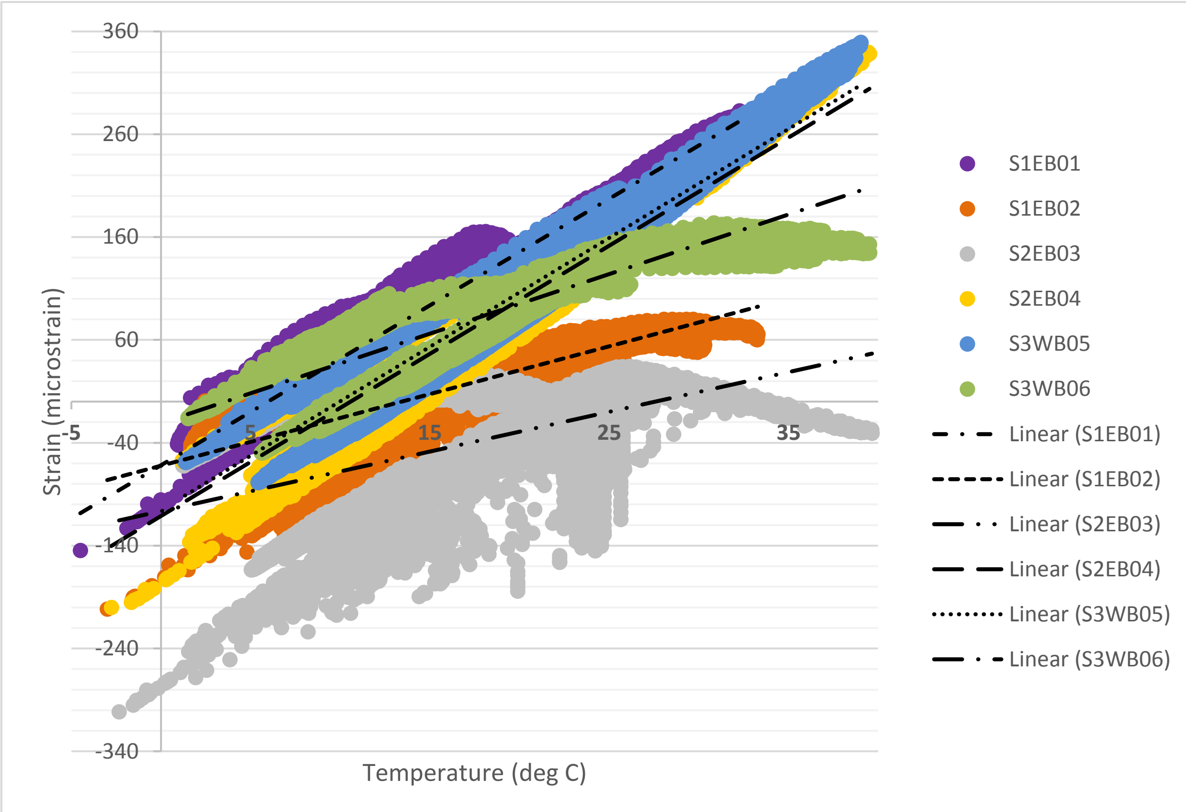

Figure 5-7 Strain vs. temperature relationship – Jameston Ave.

5.2 FWD and LWD Testing Results

Falling weight deflectometer (FWD) and Light weight deflectometer (LWD) were conducted at various times throughout the year. This was done to compare the condition of the road before any rehabilitation took place and after the rehabilitation. Through regular data collection, a better understanding of the pavement's behaviour with each underlying layer can be observed. The FWD tests for each year gave insight into the Load Transfer Efficiency (LTE), mid-slab deflection, modulus subgrade reaction and void analysis. FWD tests require the street to be empty and needs to be closed, therefore LWD testing was suggested since it requires minimum preparation and can be done manually. If the results for FWD and LWD proved to be similar, then the LWD would be more heavily practiced as it would save extra costs and time.

A year after the newly constructed road was opened to the public, FWD testing was conducted. The test was conducted the same way as the testing prior to construction, with the only exception being, the load was also dropped in the mid-slab and right outer wheel path. A total of 61 drops in each direction were conducted for the slab joints to determine LTE (Stantec Consulting Ltd., 2017).

5.2.1 Falling Weight Deflectometer Results

The FWD test was originally conducted prior to construction, to determine the condition of the existing composite pavement. Once the road was opened to traffic, the FWD testing was completed again a year after construction. The purpose of conducting the Falling Weight Deflectometer (FWD) test is to understand the functionality of the pavement. It tests the pavement to verify whether the joints are able to transfer the load (Load Transfer Efficiency), how much the layer is deflecting and whether significant voids exist underneath the concrete pavement layer.

The maximum normalized deflection D1, indicates the overall strength of the pavement. If the deflection is high, it indicates the overall pavement is weak or thin whereas a low deflection value indicates a strong pavement structure. A general guideline is that the differential deflection less than 100 µm would indicate good performance for a composite pavement, like Jameston Ave. The testing was first completed westbound travelling from Upper James St. to West 5th St where three drops were made per slab for mid-slab testing. The normalized deflection can indicate the stiffness of the pavement from the surface layer to the subgrade. Originally before construction began, the deflections ranged from 50 µm to 350 µm, for the test machine travelling westbound. This deemed to be problematic for the existing pavement layer; lack of stiffness can cause the pavement to settle. The testing was done continuously for the entire road the section divisions can be seen in Table 5-2 .

Table 5-2 FWD Section divisions

Section 1	0+000 to 0+130
Section 2	0+130 to 0+265
Section 3	0+265 to 0+400

Section 2 showed to have least amount of stiffness prior to construction, this is primarily due to the subgrade soil being topsoil and the section experiencing more water saturation since it is at the lowest point in the street. Section 1, also had similar deflections, mainly as the section got closer to section 2. Cores were not taken at the intersection; therefore, it is possible the subgrade soil is also topsoil causing higher deflections.

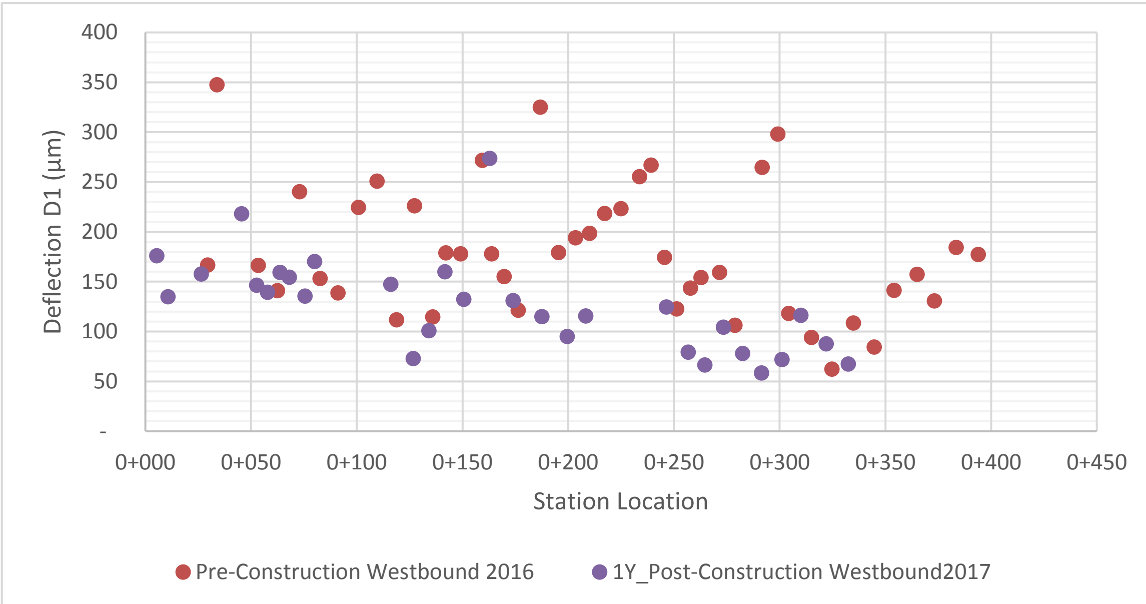

Figure 5-8 Comparison of maximum normalized deflections (D1) WB-2016 vs.2017

The normalized deflection decreased significantly once the overlay was installed and had experienced one-year of traffic. The deflection for Section 2 decreased the most, addition of a 25mm separation layer and concrete as the overlay, lowered the deflection at the bottom of the subgrade. Similar deflections can be seen from the FWD test for vehicle travelling eastbound Figure 5-9. The highest range of deflection like the westbound direction occurred in Section 2. However, the overall deflection eastbound is lower than the overall deflection west bound pre-construction.

Lowered deflection is beneficial for the pavement as it indicates the pavement is less likely to deform with increasing traffic loads. The likelihood of the pavement experiencing curling is also reduced due to a decrease in deflection. Section 3 had the lowest deflection values, this may be a contributing factor from the use of polypropylene fibres and having a 25mm HMA separation layer. The intent of using polypropylene fibres were to increase strength and reduce shrinkage cracks, however they are also known to reduce deflection of the concrete. Section 2 did not have any fibers, this was done to compare the behaviour of each overlay with fibres and without fibres. However, the deflection values for Section 2 was lower than Section 1, which is due to the use of geotextiles as a separation layer over HMA. HMA is known to provide a stronger support and separate the overlay better than geotextiles, especially if the underlying concrete was not properly rehabilitated.

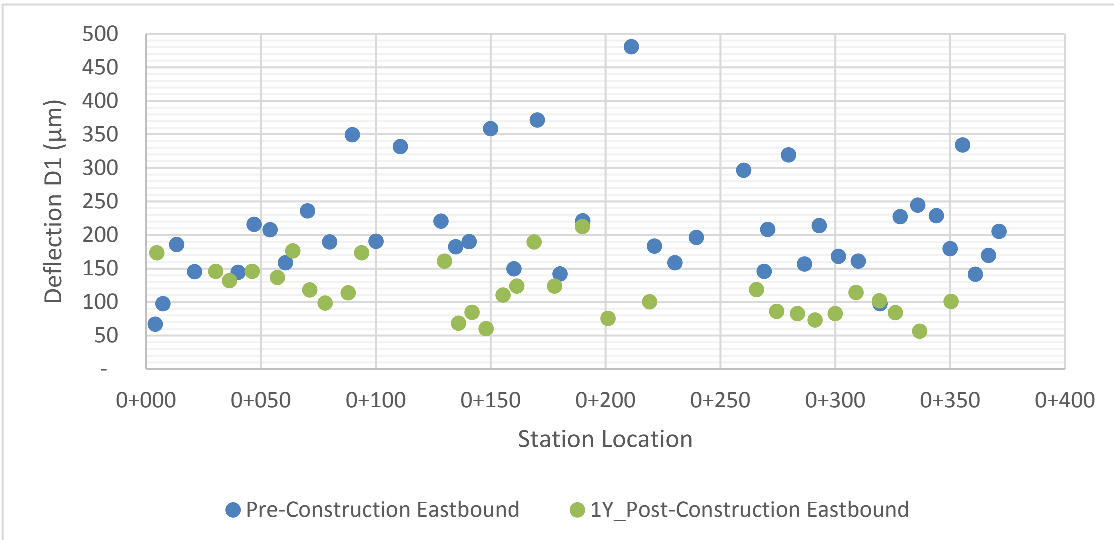

Figure 5-9 Comparison of maximum normalized deflection (D1) EB – 2016 vs. 2017

The varying range in each section for the year 2016 and 2017 can be seen in Figure 5-10. Before the installation of concrete overlay, great variability can be seen in the composite pavement. Westbound lanes have significantly reduced in range since construction of overlay and have better consistency. Deflections for Section 2 remain to have the greatest variability. Whereas deflections for Section 1 is the highest, indicating a less stiff road. This again maybe the cause of using thin separation layer such as 3mm non-woven geotextile which does not provide the same stiffness as 25mm of HMA asphalt. Section 3 the deflection variability reduced significantly, indicating the benefit of using HMA asphalt as separation layer and fibres. FWD test allowed a positive understanding of using the different types of separation layers and use of fibers. A great positive change can be seen in the behaviour of the pavement from composite to concrete overlay.

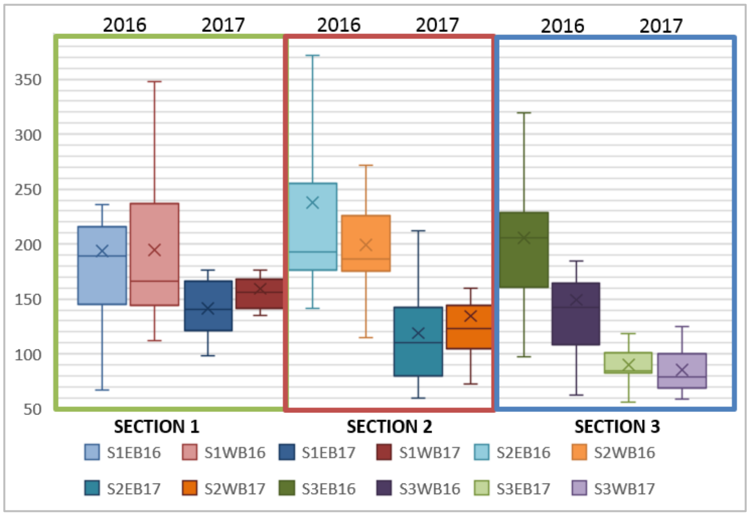

Figure 5-10 Comparison of maximum normalized deflection for Jameston Ave. (2016 vs. 2017)

FWD testing involves the application of a dynamic impact load adjacent to a concrete crack or joint. The deflections of both the loaded and the unloaded sides of the joint are measured by evenly spaced sensors. A ratio is then calculated based on the two deflections. A ratio of 100% indicates full load transfer, while a ratio of 0% indicates no interaction between the two sides of the joint. An LTE value of 70%+ indicates that a given joint is functioning well. The testing for the pavement before construction and after construction were only completed with the vehicle travelling eastbound for LTE, due to time restraints.

Load Transfer Efficiency is calculated with the AASTHO 1993 equation using deflection on either side of the joint or crack as follows:

$$\text{LTE} = \frac{d_u}{d_l}\text{ x }100 \qquad (4-2)$$

where,

LTE = Load transfer Efficiency (%)

du= deflection of the unloaded side of the joint (μm)

dl = deflection of the loaded side of the joint (μm)

The average LTE was found to be less than 70% for the composite layer before any construction took place. The lack of joint functionality aided in deciding to remove the asphalt layer and rehabilitate using a concrete overlay. Before construction it was found that in Section 1 and Section 2, the LTE is the lowest when leaving and the lowest for Section 3 when approaching. The transverse joints were faulting, and the sealant was missing when this testing was conducted, indicating the slabs were not able to transfer the load as efficiently. For many concrete pavements carrying a heavier load, steel dowel bars are placed to help transfer the load. In residential streets with a lower traffic load, dowel bars are not placed. In the case of Jameston Ave., dowel bars were not placed, and the street was tested for LTE to ensure proper functionality.

When an unbonded concrete overlay was installed the LTE significantly increased for the entire road. Particularly, it increased for leaving in Section 1 and 2 and approaching in Section 3. In Section 1, the LTE reached a maximum of 99%, indicating great joint functionality. Section 3 had the lowest LTE for the overlay, it experiences the heaviest traffic on the road due to both plaza's using the road for delivery. Nonetheless, the overall LTE for the entire street is 82% indicating excellent functionality.

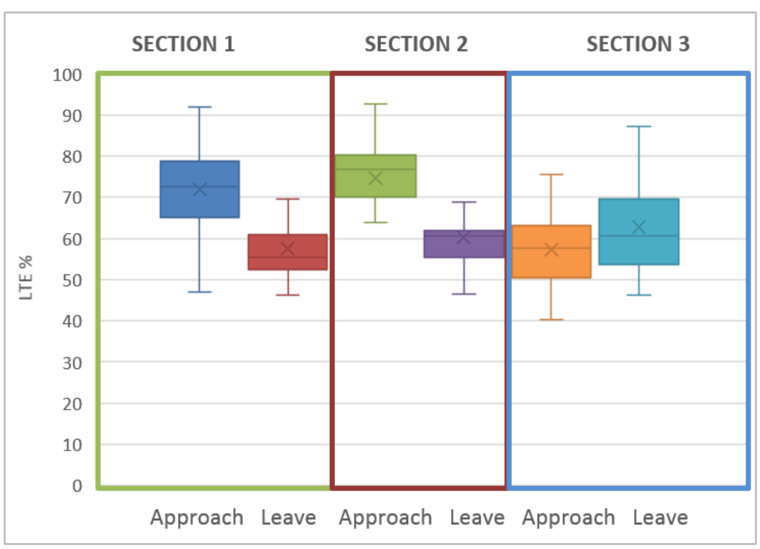

Figure 5-11 Variability in load transver efficiency of Jameston Ave. before construction 2016

According to (Wadkar, et al., 2011), it was found that longitudinal joints can help reduce load slab stresses, therefore allowing the transverse joints to function for a longer period when transferring the load. In the case of Jameston Ave, longitudinal joints did not exist originally however, was implemented during overlay construction.

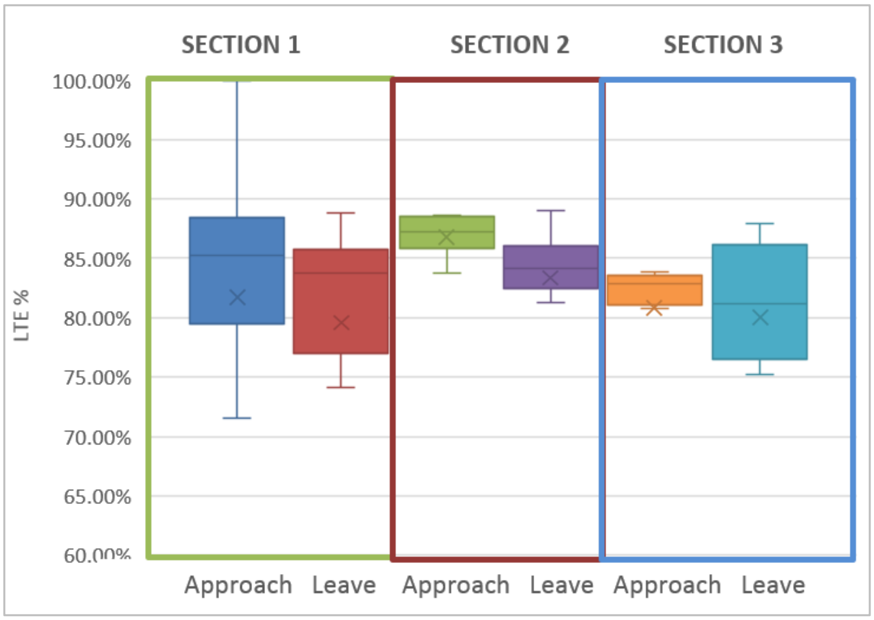

Figure 5-12 Variability in load transver efficiency of Jameston Ave. 1Y after construction 2017

The modulus of the subgrade reaction (Kstatic) was also back calculated from the FWD data prior to construction and a year after construction as seen in Table 5-3 . The modulus of subgrade reaction is an indication of the support underneath the pavement layer. A weak pavement support is less than 15 (Mpa/m) where as a very strong pavement support is (270Mpa/m). Initially, the pavement in both direction could be considered weaker, after installation of the concrete overlay it has become a stiffer pavement with better underlying support. A year after construction, Section 3 again had the highest Kstatic values, whereas Section 1, had the lowest value. This again is due to the use of 3mm geotextile as the separation layer.

Table 5-3 Summary of modulus of subgrade reaction (K_{static} – average) (Stantec Consulting Ltd., 2017)

Road Section	Year of Testing	Direction	K_{static} Average (MPa/m)
Jameston Ave	2016 (pre-construction)	E	78.6
		W	93.1
Jameston Ave	2017 (1 year post construction)	E	113.7
		W	122.6

The effective slab thickness (D_{eff}) indicates the thickness of current pavement if it was to be newly constructed as concrete pavement (Stantec Consulting Ltd., 2017). Originally prior to construction the thickness on the east and west side was 131mm and 159mm respectively, . With the installation of concrete overlay the effective slab thickness increased for east and west to 207mm and 189mm, respectively. This is a great indication of the amount of concrete the use of overlay as a rehabilitation method is saving. It is performing as efficiently and carrying the same load capacities of a concrete pavement with average thickness of almost 200mm.

Table 5-4 Summary of effective slab thickness (D_{eff} – average) (Stantec Consulting Ltd., 2017)

Road Section	Year of Testing	Direction	D_{EFF} Average (mm)
Jameston Ave	2016 (pre-construction)	E	131
		W	159
Jameston Ave	2017 (1 year post construction)	E	207
		W	189

5.2.2 Light Weight Deflectometer Results

Similar to the FWD, the LWD was tested prior to construction to see the deflections caused on the pavement. Similar deflection results between the FWD and LWD would lower costs during testing and allow the LWD to be used more frequently. Figure 5-13 below compares the LWD eastbound prior to construction and FWD deflection prior to construction.

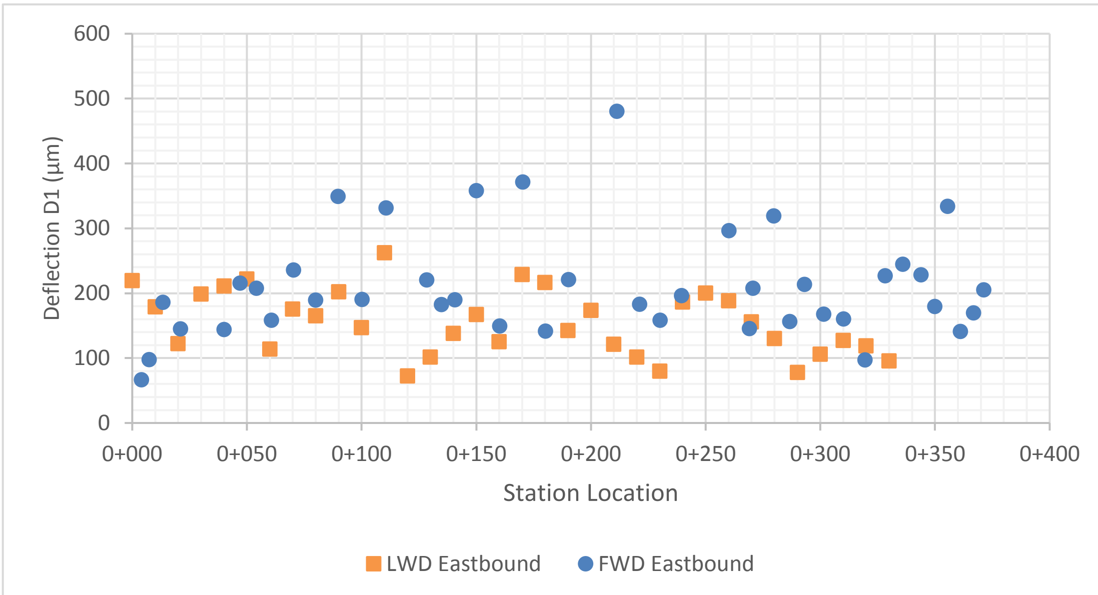

Figure 5-13 Comparision of normalized deflections (D1) for LWD and FWD pre-construction

The testing was only conducted eastbound and more drops were collected for the FWD testing. It is evident that the LWD deflection values are in the same region as the FWD deflections. The LWD is manual and can be used more frequently to understand the strength of the concrete, this can be mainly done if the project budget is low, and using LWD will save costs.

5.3 SurPRO IRI Evaluation

The CPATT SurPRO 4000 is a Class 1 multipurpose walking profiler manufactured by ICC. The purpose of this machine is to collect pavement profile data and determine pavement roughness. Data is collected by pushing the instrumentation, using its handle and walking along the road in a straight line maintaining constant speed, as seen in Figure 5-14. To determine the profile the SurPRO uses inclinometers. The inclinometers act similar to accelerometers, where the data is collected by measuring the incline, distance and the change in elevation from the original elevation. The International Roughness Index (IRI) is the roughness index obtained from measured longitudinal road profiles. It is a functional property that determines the smoothness and rider comfort. IRI values measures the deviation of a test section from a true planar pavement surface. A value of zero (m/km) indicates a perfectly smooth surface that is not causing any rider distresses or creating high stresses to the vehicles. Higher values of IRI require lower vehicle speed and indicate certain distresses that may be existing on the pavement according to ASTM E1926-08, Figure 5-15 .

Figure 5-14 Testing on Jameston Ave. using a SurPRO profiler

The SurPRO from CPATT was calibrated as a closed loop prior to data collection, 50m was measured and spray painted for the calibration. Data was collected as longitudinal profiles, three runs were done in each section; at north, south and center of the pavement. Before testing began, it was made sure the pavement was clear of debris and to prevent meandering, the testing followed the longitudinal saw-cut lines. The data was collected at speeds below 3 km/h. The device was aligned with the starting line and slowly accelerated to avoid erroneous readings and gradually brought to a complete stop at a point beyond the end mark.

The testing was conducted on the existing concrete, after the asphalt layer was milled off. It was then tested again immediately after construction, before the road was opened to the public. Lastly, it was tested a year later after the road has been opened to the public. This was done to see the change in roughness for the three stages of construction. This functionality factor would indicate whether the pavement provides sufficient rider comfort in accordance to the ASTM guideline.

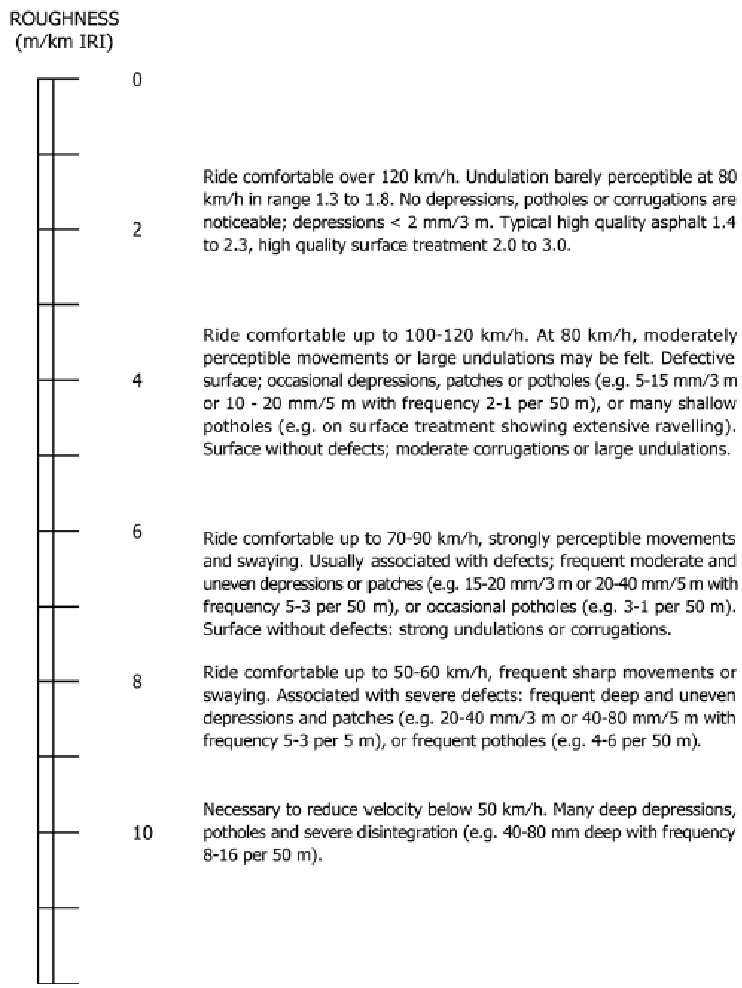

Figure 5-15 Road roughness estimation scale for paved roads with asphaltic concrete or surface treatment (ASTM International, 2008)

Originally on the distressed existing concrete the IRI values were high, the road was not suitable for rider comfort. Figure 5-16 shows the IRI for Section 2 getting to a value of 32mm/m. The center of the section experienced the highest IRI values due to two manholes and severe distresses due to the settlement of concrete, discussed in Chapter 3. The purpose of this portion of the test was to see the change in IRI value when the overlay is newly constructed and compare the difference.

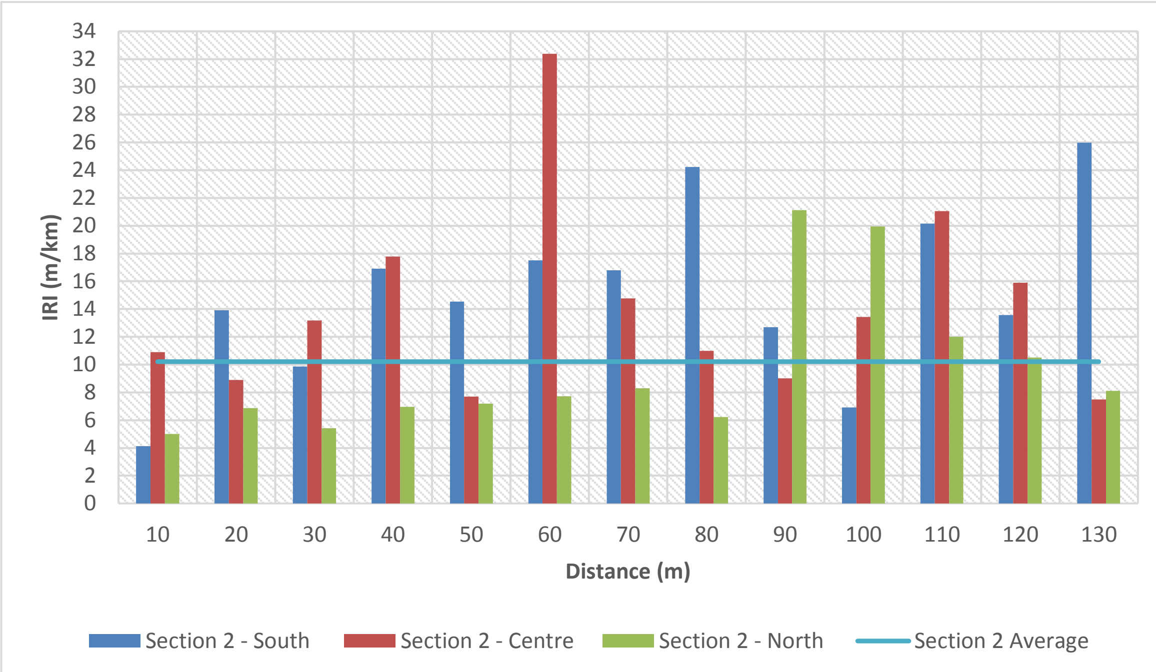

Figure 5-16 Section 2 IRI existing concrete before construction

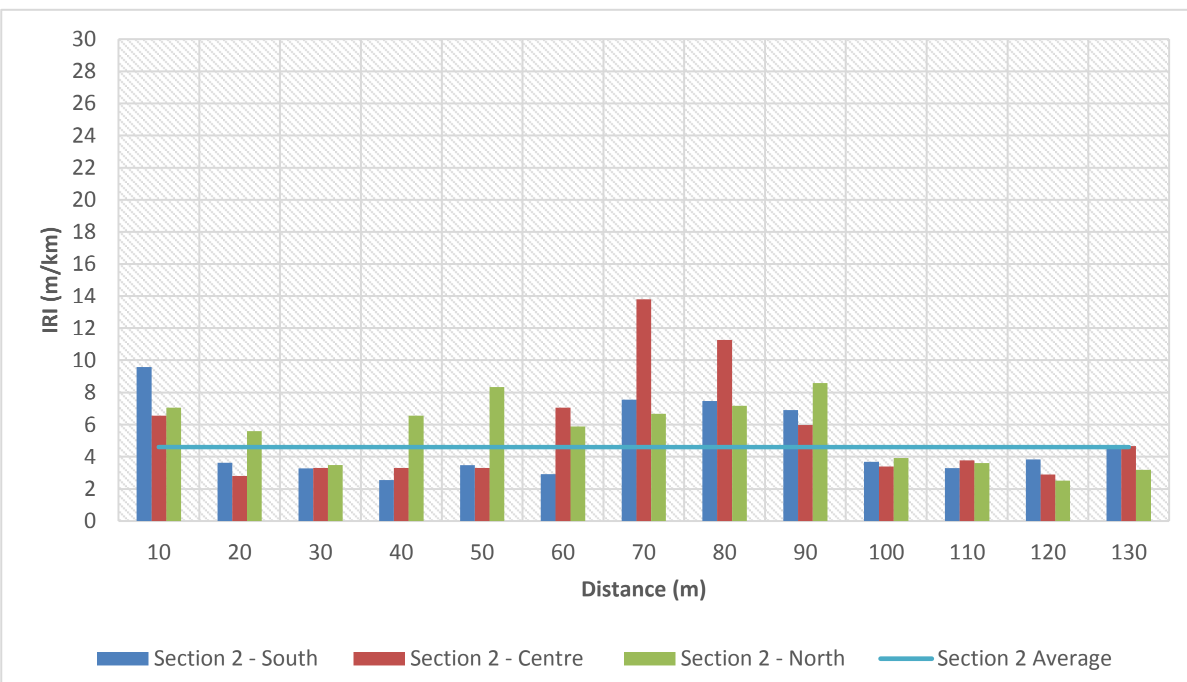

Figure 5-17 Section 2 IRI new concrete before opening road

When the road was newly constructed the IRI values decreased by a half from the existing underlying concrete. However, in the middle of the street the IRI values are still higher due to manholes in the center, seen in Figure 5-17. All three sections of the road were tested however, Section 2 is presented as it showed the best representation of the change in roughness. The average IRI changed to 4.5 m/km as oppose to 10m/km prior to construction. It can be concluded that the road is a safe residential road where ride comfort can still be achieved with 80km/h, even though the road is designed for 50km/h.

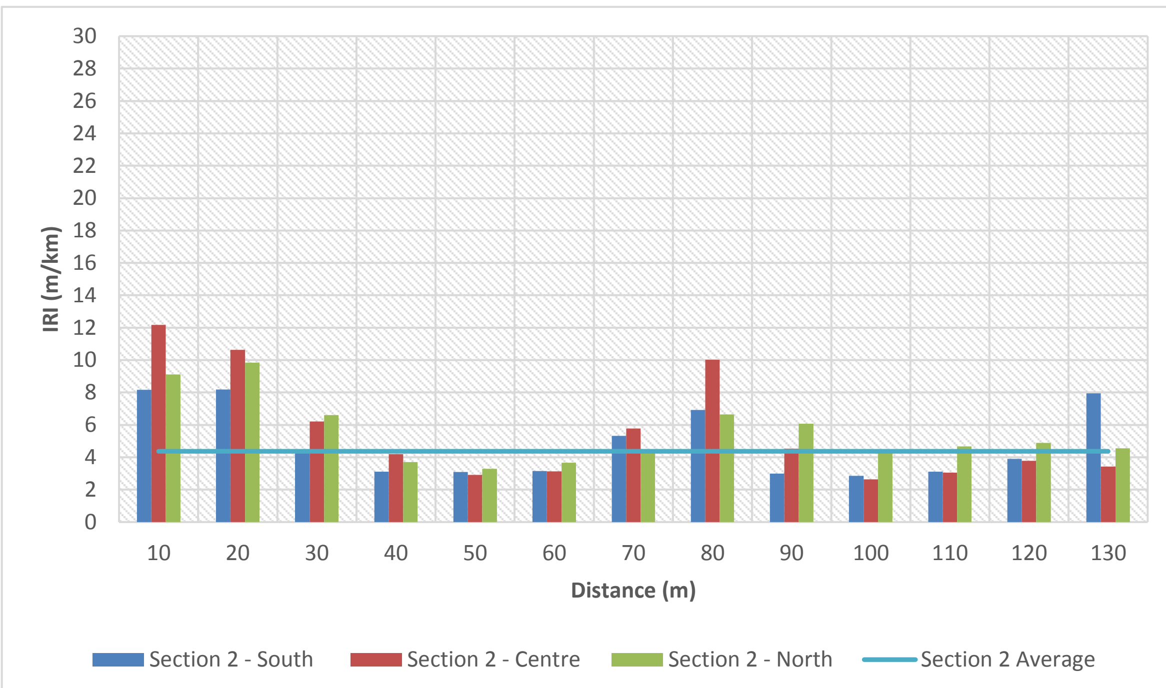

Figure 5-18 Section 2 IRI concrete overlay after 1Y of traffic

A year after the concrete was constructed it was found that the IRI values increased at the beginning stage, this could be due to transverse joints at the start of testing. The middle of the section still has the highest IRI values, otherwise the average without any outliers is similar to when the road was first constructed, with a little increase in IRI values. This is expected as with time the road will eventually not be smooth enough for rider comfort due to distresses, however it is well above average currently.

5.3.1 British Pendulum Testing

To determine the performance of overlay, after construction, friction tests were done on various sections along the road. The friction tests included British Pendulum Friction Test and T2GO friction tests. The British Pendulum Tester is a dynamic pendulum impact-type tester used to measure the energy loss when a rubber slider edge is propelled over a test surface, Figure 5-19. The BPN values represent the frictional properties obtained with the apparatus. Six locations on each section were chosen, three in each direction, eastbound and westbound. A label of S1WB01 indicates: section 1, westbound, sample location 1. A BPN value of >54 indicates good slip resistance. The freshly hardened concrete on Jameston Ave. performed above the required BPN value seen in Figure 5-21. To determine the yearly performance of the overlay, tests should be conducted at various times of the year.

Once the test section was chosen, it was brushed to remove any loose debris. The mechanism was levelled to ensure the pendulum swings correctly onto the scale. The location was sprayed with water as the test determines skid resistance in wet conditions and emulates wheel on wet pavements. The pendulum was swung 4 times in each location to get a good overall average of the street seen in Figure 5-20.

Figure 5-19 British Pendulum Test Apparatus

Figure 5-20 British Pendulum testing on Jameston Ave.

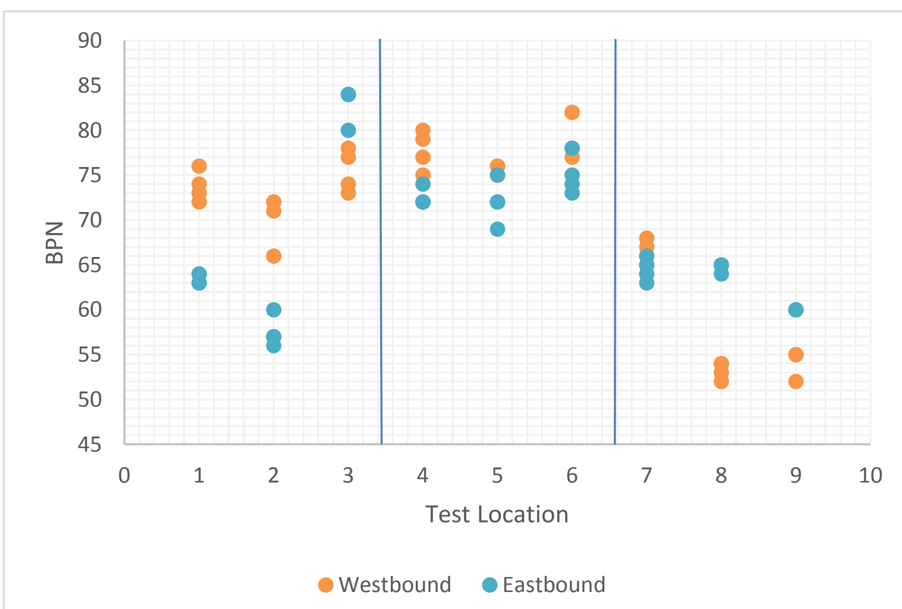

Figure 5-21 British Pendulum results for newly constructed Jameston Ave.

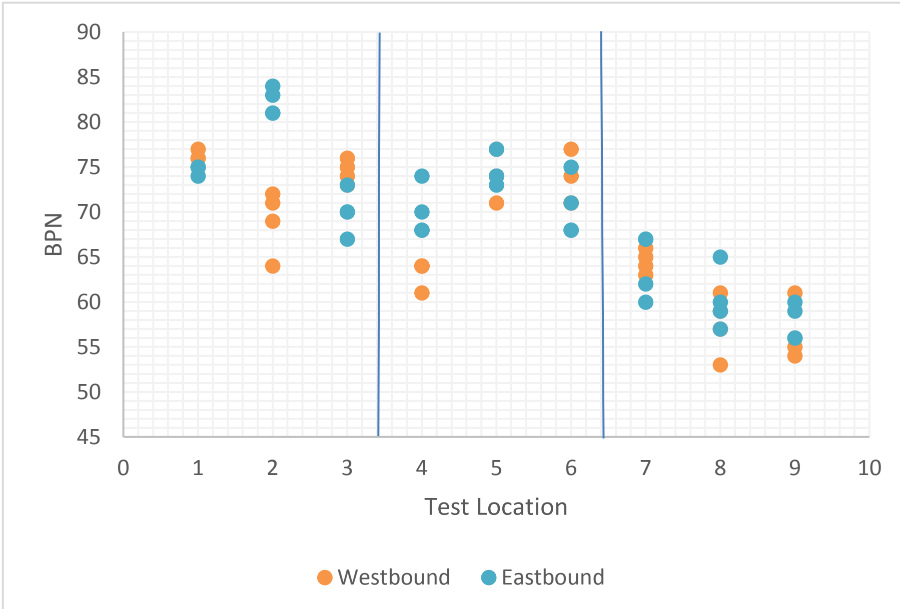

Figure 5-22 British Pendulum results for 1Y old Jameston Ave.

Test locations 1-3 is Section 1, 4-6 is Section 2 and 7-9 is Section 3. There was not a significant change in the BPN values in a year. Most of the values did decrease slightly which is expected due to the wear and tear of the road due to traffic, however it is still well above the threshold for a road with good slip resistance Figure 5-22. The only anomaly exists in the second test location where it shows the BPN value increased, although it was made sure the same exact location was tested, it is possible the testing occurred slightly away from the original section. This newer part of the section could have a better texture due to the manual broom finish as opposed to the original location.

5.3.2 T2GO Test Results

The coefficient of friction of a pavement surface was measured using a device called the T2GO, made by ASFT (Airport Surface Friction Tester, Sweden). T2Go is another mechanism like a wheelbarrow that runs over the section being tested Figure 5-23. It collects friction values and is a good indicator of pavement performance. Like the British Pendulum test, T2Go was conducted on 6 locations on each section, a friction value greater than 0.5 indicates good braking and skid resistance. The testing for T2GO was conducted right after construction (before the road opened) and a year the road has been open to daily traffic.

Figure 5-23 T2GO testing on Jameston Ave. concrete overlay

Similar to the BP the T2GO showed great success as well a year after the road has been opened to daily traffic. The road has gone through one freeze thaw cycle and has not experience a significant decrease in skid resistance. Initially after construction the T2GO friction values ranged from a high of 0.84 to a low of 0.5, Figure 5-24. A year after construction the range in values remained the same as seen in Figure 5-25. Both BP test and T2GO test found section three (locations 8, 9 and 10) to have the lowest skid resistance.

Higher values are beneficial particularly during the winter season, as black ice formation is likely, a pavement with higher skid resistance will allow the vehicle to control speed and come to a halt. It is crucial to ensure the skid resistance remains above the 0.5 threshold, which can be done by diamond grinding the concrete when the values drop 0.5. Similar to the BP test results, any increase in value from the new constructed concrete to a year-old concrete may have been due to the slight location change, where the broom finish texture has deeper grooves. From the BP and T2GO test results it was seen that Section 3 had the lowest values, this is due to the section having a noticeably lower amount of texture from the broom finish.

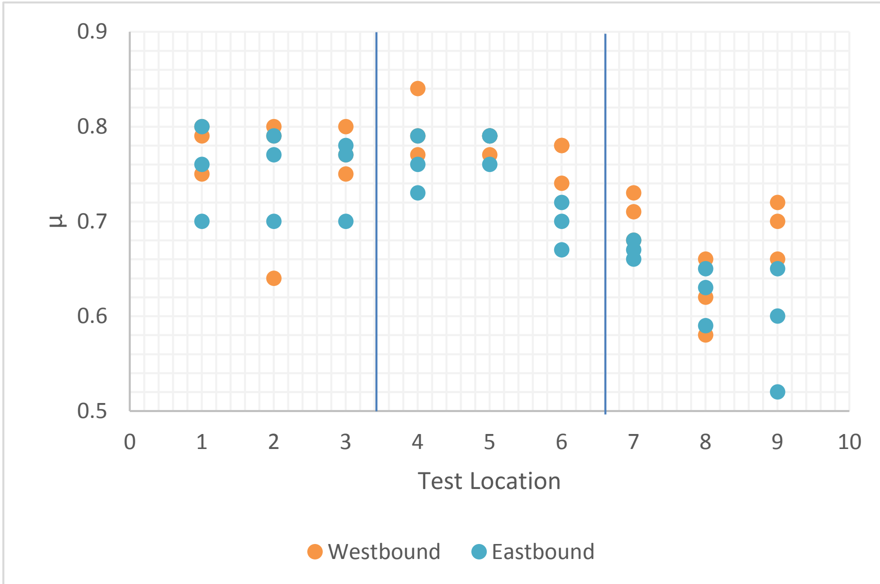

Figure 5-24 T2GO results for newly constructed Jameston Ave.

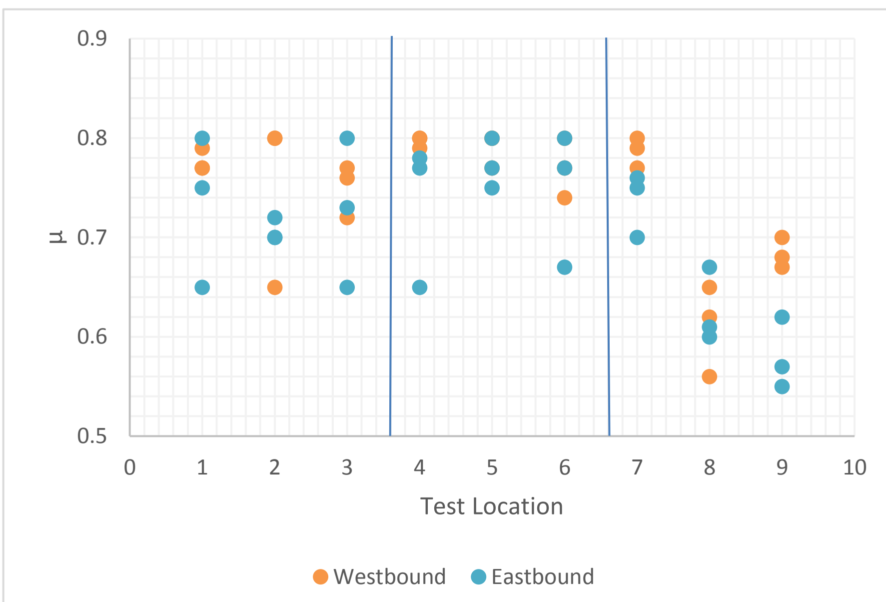

Figure 5-25 T2GO results for 1Y old Jameston Ave.

5.4 Current Pavement Condition

To determine the current condition of the pavement, a visual inspection was also conducted along with the non-destructive tests such as, FWD, SurPRO, T2GO and British Pendulum. A visual inspection can be a great indicator for the type of distresses the pavement is experiencing along with predicting future behaviour of the pavement. Overall, Jameston Ave. is in great condition, proving to be a long-term rehabilitation method for composite residential streets. Each section along the pavement experienced some slight distresses, discussed below.

5.4.1 Section 1

Section 1 is in great condition a year after the road has been opened for service. The joints have not experienced any joint-spalling, Figure 5-26 below shows a typical joint in Section 1. A moderate transverse crack formed initially from one of the catch basins right after construction was completed. The transverse crack has not expanded since and may be due to thermal expansion or contraction, seen in Figure 5-27.

Figure 5-26 Typical Joint in Section 1 – 1Y after construction

Figure 5-27 Moderate transverse crack – Section 1

5.4.2 Section 2

Section 2 experienced the most distresses amongst all the other sections on the street. The visual inspection found some slight joint spalling, d-cracking and longitudinal cracks. The cause of these distresses maybe due to the lack of fibres in the concrete mix, also, the underlying subgrade is top soil which is still prone to settlement. This portion of the street is also sitting at the lowest elevation, experiencing more moisture in-between the pavement layers than expected.

Figure 5-28 Moderate longitudinal joint spalling – Section 2 – 1Y after construction

The joint spalling in Section 2 occurred at the lowest elevation of the section. The joints on Jameston Ave. were not sealed since they are thin soff-cut joints, however, it is likely loose debris may enter the joints preventing expansion and contraction of the concrete. The joint spalling occurred closer to the curb edges, where it was noticed the saw-cuts were not as thick and may be preventing the slab movement, Figure 5-28.

Figure 5-29 Moderate d-cracking in Section 2 – 1Y after construction

D-cracking was observed in Section 2 at the lowest point of elevation, Figure 5-29. This took place after the street went through one freeze-thaw cycle, d-cracking can occur when the lowest part of the pavement is critically saturated. This portion of the section prior to construction also had the highest distresses and experience all the run-down of water from the higher elevated sections. The cracking is moderate, however is likely to get worse as it experiences more freeze-thaw cycles.

Figure 5-30 Moderate longitudinal crack – Section 2 – 1Y after construction

A longitudinal crack was found near the end of Section 2, approaching Section 3. The cause of this crack may be due to late saw-cutting. Soff-cutting the section took approximately 7-8 hours, therefore by the time the soff-cutting took place at the end of the section it may have been later than expected causing longitudinal crack to form.

5.4.3 Section 3

Based on the visual inspection conducted in Section 3, it did not experience many distresses. There was some joint spalling, close to the curb as noted for Section 2. There was also a longitudinal crack that developed with some meandering. The section had joints that were in excellent shape and the overall condition of the section was very good with very little distresses. This part of the section experiences the highest volume of traffic; therefore, it was crucial to see the condition of the section a year after construction. Similar to Section 2, the joint spalling took place at the edge of the curb, where the joints were not as wide as the rest of the street. This may have prevented movement for the concrete, causing the spalling, Figure 5-31.

Figure 5-31 Moderate longitudinal joint spalling – Section 3 – 1Y after construction

A longitudinal crack occurred midway between the longitudinal joints. There is a slight meandering of the crack. The cause of this crack can be attributed to the spacing of the longitudinal joints. Originally, it was suggested during the design stages that the longitudinal joint have a spacing of 1.5m, however the longitudinal spacing was increased to prevent the number of soff-cuts. However, the formation of this longitudinal joint may be due to thermal changes, where the lack of expansion and contraction in that specific area caused the cracking.

Figure 5-32 Longitudinal cracking with slight meandering – Section 3 – 1Y after construction

5.5 Life Cycle Cost Analysis (LCCA)

Life cycle cost analysis is the process of determining the total economic worth of a project by taking into account its initial costs and discounted future costs (TAC, 2013). Discounted future costs for a pavement can include maintenance, rehabilitation, resurfacing and restoration costs throughout its service life. One of the main challenges and concern for municipalities is budgeting projects with high initial costs. A LCCA can help compare project alternatives by presenting the net present value of each alternative and ultimately, aid in the decision-making process. It allows industry partners and municipalities to be more informed and make better investment decisions.

Jameston Ave. is the first residential unbonded concrete overlay in Ontario. For this trial section to be a complete guideline for industry users, cost is a crucial component. The City of Hamilton is interested in seeing two alternatives, the use of concrete overlay vs. mill-and replace of asphalt through the analysis period. The analysis period for this LCCA was selected to be 25 years instead of the recommended 35-year analysis period. According to (Lamptey, et al., 2005), the analysis period can be shorter than the recommended if analysis period is buying time until total reconstruction. The pavement is already existing, there for the initial cost is a rehabilitation procedure.

Figure 5-33 (Alternative A) below outlines the time-based strategies of existing composite pavements, with major maintenance and concrete pavement restoration occurring at year 8 and 17, respectively. Similarly. Figure 5-34 (Alternative B) outlines the time-based strategies of existing composite pavements, with routine maintenance and rehabilitation occurring at every 3 and 9 years, respectively. Alternative A, is the project conducted at Jameston Ave. the pavement will require cleaning of all joints in year 8, however sealing will not be required as the original pavement was not sealed. Alternative B, is the current practice for the City of Hamilton on all of its composite pavements. The existing asphalt layer is milled off and a new layer of asphalt is placed on top of the concrete base. This method requires crack sealing every three years to prevent reflective cracking and replacement of the asphalt overlay every 9 years.

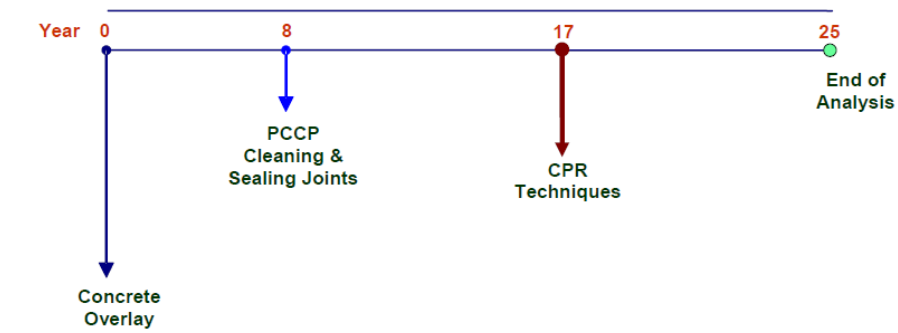

Figure 5-33 Time-based alternative for existing compostive pavement – concrete overlay (Lamptey, et al., 2005)

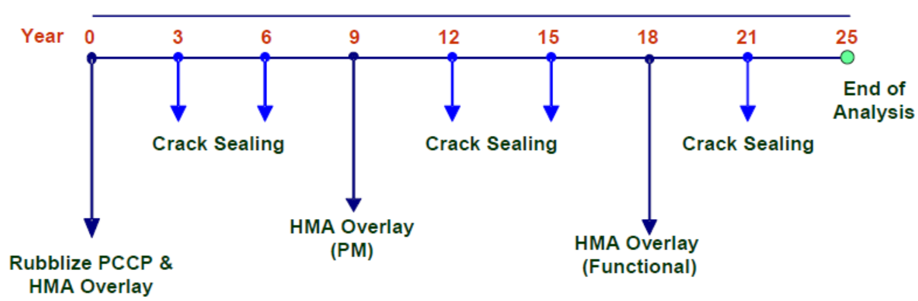

Figure 5-34 Time-based alternative for existing compositve pavement – HMA overlay (Lamptey, et al., 2005)

From the time-based alternatives, a cost comparison between the two alternatives were completed. Table 5-5 displays the net present value of Alternative A, outlining the present worth factor, and the agency costs. The agency costs in this scenario did not include user delay costs, which would require a measure of various travel routes during construction. The values were calculated based on the unit prices provided by the City of Hamilton. The initial cost of the concrete construction includes the use of Geotextile and HMA as the separation layers. Table 5-6 displays the NPV for alternative B, similarly, the unit costs were provided by the City of Hamilton from their current database. The initial cost of the mill-and-replace strategy assumes that an urban local street is resurfaced 80mm and 10% of the curb and s/w are repaired. The major rehabilitation in year 9 is due to the pavement being stripped to the concrete base and 5% of the base is repaired.

Table 5-5 Net-present value for alternative A – concrete overlay

Discount Rate	4.00%		
Alternative A	**Initial**	**Maintenance**	**CPR Techniques**
Year >>>	0	8	17
Agency Costs	$347,248.68	$30,000.00	$83,756.00
Present Worth Factor		0.73069	0.513373
Agency Costs (PW)	$347,248.68	$21,920.70	$42,998.07
Total NPV (Agency Cost)	$412,167.45		

Based on this preliminary, LCCA unbonded overlays for a 400m pavement section is less costly than the procedure of mill and replacement. Unbonded concrete overlays can last longer than 25+ years without any routine maintenance. Routine maintenance and replacement of HMA layer is the cause of higher cost in the mill-and-replace strategy. To provide a better in-depth analysis further information from the City of Hamilton providing maintenance unit prices would be required. Constructing a unbonded concrete overlay can reduce the use of materials that would be required during the rehabilitation process every nine years and sealant every three years.

Table 5-6 Net present value for Alternative B – HMA overlay

Alternative B	**Initial**	**Maint.**	**Maint.**	**Rehab**	**Maint.**	**Maint.**	**Rehab**	**Maint.**
Year>>>	0	3	6	9	12	15	18	21
Agency Costs	$188,000	$6,000	$6,000	$320,000	$6,000	$6,000	$188,000	$6,000
Present Worth Factor		0.88899	0.79031	0.70258	0.62459	0.55526	0.49362	0.43883
Agency Costs (PW)	$188,000	$5,333.90	$4,741.8	$224,827.8	$3,747.5	$3,331.5	$92,802.0	$2,633.0
Total NPV (Agency Cost)	$525,417.91							

5.6 Chapter 5 Summary

- Construction for the overlay was completed August 2016, monitoring and testing of the street began immediately after. Data collected from the vibrating wire strain gauges indicated temperature to be the main contributing factor of strain. With increase in temperature, the strains increased. Section 2 experienced the highest strain values, whereas Section 1 had the lowest strain values. However, the strains experienced by the overlay thus far is well below the threshold for flexural cracks.
- FWD tests were conducted prior to construction and a year after construction, it was found that the LTE values had increased significantly after the installation of concrete overlay. The pavement now has a strong base and is stiffer in comparison to the composite pavement prior to construction.
- SurPRO test results showed decrease in IRI values, indicating safer and smoother pavement. This will provide great rider safety for years to come and allow the street to function above capacity without any routine maintenance.
- Friction and skid resistance test results from BP and T2GO show a slight decrease however that may be due to not testing at the exact same locations. Manual broom finish had more texture in some parts of the street than others.
- LCCA shows the overlay method to be almost 20% cheaper than the original asphalt mill-and-replace strategy that was being followed by the City of Hamilton.
- Finally, from visual inspection all sections are behaving well however, Section 2 will require monitoring as it has experienced the most distresses in a year. Correlation between the use of fibers and distresses can be made once further testing is completed.

6 Conclusion

Rigid pavements can be a long-term solution due to their durability and lack of maintenance, however they are uncommon for residential streets. The City of Hamilton wanted to minimize cost of the routine maintenance and find an innovative solution that would be beneficial for municipalities throughout Canada. The trial concrete overlay on Jameston Ave., Hamilton, Ontario was opened to the public on August 2016. It has been in service over a year and has been performing well above expectations. This was the first of its kind in Ontario, Canada and the design and construction crew faced many challenges. Construction for the street took place during the summer with high temperatures, best practices were maintained by spraying the street with water before placement of concrete. The construction crew faced challenges when placing the geotextile separation layer, for future projects, the textile should be uniformly placed and curing compound should be sprayed evenly. Placing of the geotextile was a challenge as moving trucks caused tears and it took the shape of deformities. Concrete overlays can be constructed like conventional concrete pavements using razorback paving equipment, allowing for fast and easy paving.

This trial project has successfully shown that concrete overlays are capable of carrying daily residential traffic loads. Although some surface distresses can be seen after a year of service, through all the non-destructive test and sensor data, the pavement is performing well above the required threshold. The sensors indicate the strains are still below flexural failure although they will need to be collected and analyzed regularly for the future. The strain gauge data shows that the concrete strains generally remain compressive at the top of the pavement. FWD tests show the pavement to have higher structural capacity after construction. Deflection values have significantly decreased; a concrete overlay constructed on Jameston Ave. is equivalent to a 200mm thick conventional jointed plain concrete pavement. This reduces the use of concrete material by almost 20%.

The objective of this thesis was to act as a guideline for municipalities, beginning with the preliminary design for concrete overlays. Completing analysis for the pavement structure using core samples and determining the most optimal materials for construction. The goal of the thesis was to outline construction procedure and highlight the challenges faced and present the results that were collected after a year of service. From the analysis and evaluations completed, this project can be considered as economically, environmentally and socially beneficial. Originally the City of Hamilton was spending a lot of time and expenditure on the process of milling the asphalt layer of the composite pavement and placing a new one. Due to the underlying layer being concrete, the asphalt had to be cut and sealed, these saw cuts required sealant replacements every 3 years. From the LCCA it was found that the total cost of the concrete overlay vs. mill-and-replace strategy is much more cost efficient. Minimal distresses were seen throughout the pavement structure and low variability in strain values indicate lower potential for fatigue failure. The pavement did not require any reinforcement or placement of dowel bars and yet the LTE values were well above the 70% requirement. This can reduce the amount of material required for construction and still prove to be as promising as a regular asphalt pavement.

Although the pavement has proven well, it has only been monitored for a year. Therefore, the susceptibility to loss of structural capacity may still occur. To understand the behaviour of the overlay and for municipalities or any other industry partners to implement this long-term solution, monitoring and evaluation should continue for the entire design life.

6.1 Recommendations

As previously mentioned the scope of this thesis is limited due to only a year of service. Continued monitoring and evaluation of the Jameston Ave. test section are required to better understand the long-term performance of concrete overlays as a rehabilitation method for municipalities. Additional sensor data collection and analysis, visual inspection, deflection testing and traffic data collection are required to perform a more thorough evaluation.

Future design practices for concrete overlay should consider the moisture levels of the pavement. Drainage is a key component to monitor; moisture buildup between the pavement layers can heavily impact the structural capacity of the pavement. One of the challenges faced on Jameston Ave. was the drainage issue for Section 2, where water runoff from the higher sections accumulate and weaken the pavement structure causing settlement. The subgrade of the pavement should also be taken into consideration during the preliminary stages, if the material is a topsoil it is likely to be more saturated and unable to take the traffic load. For future research projects moisture probes should be installed. When constructing future concrete overlays, time should be given to ensure the underlying concrete layer is uniform for the placement of a separation layer. If the existing concrete consists of any distresses they should be thoroughly repaired before placement. BP and T2GO test should continue every couple of years to monitor the friction values and determine the safety of the road. Furthermore, SurPRO tests should continue to monitor any increases which would indicate the need for repair and potentially a rider safety concern. LWD testing should be conducted on the pavement with FWD to ensure similar results.

Finite element modelling should be performed to determine the effects of vehicular and thermal loading. Testing with heavy truck loads should be conducted on Jameston Ave. to determine the impact of vehicular load, especially to understand the impact of increasing traffic loads. Modelling will help determine the main cause of stress on the pavement, whether vehicular or thermal. However, this will not be possible without the field data and if collected on regular interval it can be compared with the finite element modeling analysis. These results can help optimize design and increase construction efficiency for future projects. Canadian climate has a significant impact on our pavement, and developing long-term designs for colder climates is crucial. Best practices for concrete overlay in Canada can be designed with the data collected and visual evaluations conducted throughout the service life of the pavement. There are many knowledge gaps in the practice of concrete overlays that have yet to be researched. One of the main points of research will be the maintenance and rehabilitation that may be required for a concrete overlay in a Canadian climate. The end-of-life stage of concrete overlays will also need to be studied. The performance of various types of separation layers should also be considered in further detail. The use of polypropylene fibers and the impact it had on Jameston Ave. is unknown and can be understood with time. Finally, a complete detailed lifecycle cost analysis must be completed in order to better evaluate other long-term benefits, agency costs should be determined for a much more accurate analysis.

7 References

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8 APPENDIX A

Core Photographs

BH 5D-1, DEPTH 0 - 310 mm

BH 5D-2, DEPTH 0 - 290 mm

BH 5D-3, DEPTH 0 - 80 mm

BH 5D-4, DEPTH 0 - 250 mm