Development of a Framework for Monitoring the Long-Term Performance of Perpetual Pavements

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

Perpetual pavements represent a significant investment to an owner who has committed to spending additional dollars at initial construction in order to benefit from potential long-term savings from the enhanced performance of this asset. This makes the monitoring of a perpetual pavement critical to ensure that this asset is optimally preserved and maintained in order to meet the expectations of service for the design life and potentially beyond. This Thesis research involved investigation of methods of completing the long-term monitoring of a perpetual asphalt pavement including the development of a testing protocol using a falling weight deflectometer (FWD) as well as a framework for the monitoring of long term perpetual pavement performance.

The project site used for the research consisted of one perpetual pavement section (with rich bottom mix (RBM)) which was constructed and instrumented at the Capitol Paving Plant in Guelph, Ontario. It was constructed by a consortium that included the Ontario Ministry of Transportation (MTO), Ontario Hot Mix Producers Association (OHMPA), the University of Waterloo Centre for Pavement and Transportation Technology (CPATT), the Natural Science and Engineering Research Council of Canada (NSERC), Stantec Inc., and McAsphalt Industries Limited.

An initial testing program was required to accurately locate the embedded sensors within the test section. This testing program was completed with an array of FWD testing completed within the test section followed by analysis of the response of the embedded sensors to the testing in order to determine their location. This initial testing was successful in determining the embedded sensor locations and the locations were marked in the field for use in future testing programs.

The next step consisted of validation of the performance of the embedded sensors. This involved predicting the expected strains using mechanistic design software (Kenpave) followed by a comparison with the strains recorded with the embedded sensors on the site. A significant discrepancy was found between these results and supplemental testing was completed to attempt to isolate and mitigate the source of the variability. The in-situ resilient modulus values were backcalculated using and the FWD results which were adjusted in order to obtain design deflections similar to the deflections measured using the FWD. The resilient modulus of the asphalt concrete layer was adjusted for temperature and the expected strains recalculated using the mechanistic design software. While the results showed signs of converging, the known sources of variability had been evaluated and the remaining difference between the predicted and calculated strain values were considered to be due to a change in the calibration factor of the gauges.

New calibration factors were calculated for the gauges and the new calibration factors applied to the sensors and checked using the FWD in order to validate the new calibration factors. The additional testing showed that the embedded sensors were now within the tolerance expected for the types of monitoring equipment used at the site and the new calibration factors were considered to be suitable.

Finally, a framework was developed to provide guidance for the long-term monitoring of perpetual pavements using the knowledge and experience gained during the research.

Acknowledgements

I would like to thank my supervisor, Professor Susan Tighe, from the Civil and Environmental Engineering Department at the University of Waterloo for her guidance, help and encouragement throughout my entire Masters studies.

I would also like to thank LVM for providing financial support for this Thesis, allowing me the use of their high tech non destructive testing equipment as well as allowing me to be flexible with my work schedule in order to accomplish my educational goal.

I would like to acknowledge several industry professionals and colleagues who assisted me with this research including: Mohab El-Hakim for his assistance and knowledge of the Test Section; Mark Latyn of Capital Paving Ltd. for his assistance and patience during the field testing; Aleks Kivi of ARA Inc. for his assistance with troublesome monitoring equipment; and my colleagues at LVM for supporting me during this endeavour.

Finally, I would like to thank John Emery, who I consider a mentor, for instilling in me a devotion to quality, a passion for pavements, a joy for teaching and a caring for those less fortunate.

Dedication

I would like to dedicate this Thesis to my wonderful family for all of their support through my pursuit of academic advancement. Specifically, I would like to thank my wife Chrisinne and my three children, Nicholas, Alexandre and Geneviève for all of the support, light and happiness they provided me during this pursuit.

Table of Contents

List of Figures

List of Tables

List of Abbreviations

AADT	Average Annual Daily Traffic
AADTT	Average Annual Daily Truck Traffic
AASHTO	American Association of State Highway and Transportation Officials
APA	Asphalt Pavement Alliance
ASG	Asphalt Strain Gauge
ASTM	American Society for Testing and Materials International
CBR	California Bearing Ratio
CCTV	Closed Circuit Television
CPATT	Centre for Pavement and Transportation Technology
DCP	Dynamic Cone Penetrometer
EOP	Edge of Pavement
EPC	Earth Pressure Cell
ESALs	Equivalent Single Axle Loads
FAA	Federal Aviation Administration
FEL	Fatigue Endurance Limit
FHWA	Federal Highway Administration
FWD	Falling Weight Deflectometer
GPR	Ground Penetrating Radar
GPS	Global Positioning System
HMA	Hot-Mix Asphalt
HWD	High Capacity Falling Weight Deflectometer
IDOT	Illinois Department of Transportation
ITS	Intelligent Transportation System
LTPP	Long Term Pavement Performance program
LWD	Lightweight Deflectometer
MEPD	Mechanistic-Empirical Pavement Design Methods
MEPDG	Mechanistic-Empirical Pavement Design Guide
MERO	Materials Engineering Research Office, Ontario Ministry of Transportation
MET	Method of Equivalent Thickness
MTO	Ontario Ministry of Transportation

NCAT	National Centre for Asphalt Technology
NHS	National Highway System
NMAS	Nominal Maximum Aggregate Size
NSERC	Natural Science and Engineering Research Council of Canada
OHMPA	Ontario Hot Mix Producers Association
OPSS	Ontario Provincial Standard Specification
PDDE	Pavement Deflection Data Exchange
PGAC	Performance Graded Asphalt Cement
RAP	Reclaimed Asphalt Pavement
RBM	Rich Bottom Mix
RWIS	Road Weather Information System
SHM	Structural Health Monitoring
SHRP	Strategic Highway Research Project
SMA	Stone Mastic Asphalt
SP	Superpave
TAC	Transportation Association of Canada
TDC	Top-Down Cracking
VMA	Voids in Mineral Aggregate
WHRP	Wisconsin Highway Research Project
WIM	Weigh-In-Motion

1. INTRODUCTION

1.1. Background

There have been many studies completed that demonstrate the link between the quality of a society's transportation network and its comparative level of economic success and the improved quality of life of the population [Tighe et.al, 2014]. The earliest example of road transportation networks come from the Roman Empire which developed vast paved road networks using locally available materials, mostly for the effective movement of its military. When not in use by the military, these road networks were used by local entrepreneurs and traders to transport their goods to different markets. Even after the demise of the Roman Empire, these road networks left a legacy that promoted growth and prosperity in many parts of Europe for years to come.

For a country as vast as Canada, transportation networks are vital in order to move natural and manufactured goods both across the country and to international markets. Historically, natural and manufactured goods were transported either over water (canoes, and more recently ships) or by rail. However, the transportation dynamics changed in 1962 with the completion of the Trans-Canada Highway which allowed for more flexibility in the timing and quantity of goods being transported. Since this time, the use of highways for the transportation of goods has grown and according to Transport Canada statistics, road transportation is the dominant mode of transport representing 56.5% of all trade between Canada and the United States (225 billion tonne-kilometers in 2010). This demonstrates how vital Canada's road network is to the prosperity of our nation.

The original road networks were designed for lighter loads and lower traffic volumes than those which today use these same networks. As a result, not only has the road network expanded, the pavement structures have also needed to be enhanced resulting in increased consumption of construction materials (premium aggregates and high performance binders, for instance) and infrastructure funding. For instance, Transport Canada statistics indicate that between 2006 and 2010, $17.4 billion dollars were invested in the National Highway System (NHS) by all levels of government as part of planned maintenance, expansion, and improvements.

As traffic levels and the tonnage of goods transported over road networks has grown to levels beyond that which was contemplated by the original highway designers, the thickness of the flexible pavements has also been increased in relation to the empirical design methods used at the time. While the empirical relationships have been extended for higher design traffic loads, it has been shown that these empirical relationships are not generally calibrated for these higher traffic levels and tend to result in an over design of the pavement structural components. This type of design is not socially or economically sustainable and as a result, new ways of designing and maintaining flexible pavements are required such as the perpetual pavement design method.

Recent research and testing of perpetual pavements has shown that there are asphalt concrete thickness limits where critical tensile strains at the bottom of the asphalt concrete layer and compressive strains at the top of the subgrade can be reduced to the point where the distress in the pavement will be limited to the upper layers which are more readily maintained and rehabilitated than the deeper asphalt concrete layers [Thompson & Carpenter, 2006, Willis, 2009, Willis, Timm, 2009b]. In addition, this research has shown that different asphalt concrete mixes placed in strategic locations within the pavement structure are able to prolong the pavement life in ways not previously contemplated.

Increasing the longevity and reliability of our roadway networks is an important component required to improve our economic success, reduce our environmental footprint and improve our standard of living which are the three important components of evaluating sustainability. The use of the perpetual pavement design concept as well as the development of ways to optimally manage and maintain these flexible pavements are key components in developing a sustainable transportation network.

1.2. Scope and Objective

The scope of this Thesis involves the evaluation of the performance of embedded sensors installed in a perpetual pavement test section that was developed with the collaboration of the Ontario Ministry of Transportation (MTO), Ontario Hot Mix Producers Association (OHMPA), and the University of Waterloo Centre for Pavement and Transportation Technology (CPATT) with funding provided by the Natural Science and Engineering Research Council of Canada (NSERC). The evaluation included non-destructive testing in the vicinity of the embedded sensors in order to determine their performance after five years of service. In addition to the field evaluation of the sensors, a literature review was completed to determine: the key performance indicators that need to be monitored during the service life of a perpetual pavement; the types of sensors required to monitor key performance indicators; the expected service life and reliability of pavement sensors; and preventive maintenance options for perpetual pavements.

The objective of this research is to evaluate the benefits of the use of embedded sensors in the monitoring of perpetual pavement sections as well as to develop guidance on the effective operation and monitoring of an instrumented perpetual pavement. At the conclusion of the research a framework is provided for monitoring the long-term performance of a perpetual pavement including: selection of the type and number of sensors to use over the design life;

validation of sensor performance and/or damage during the construction process and initial calibration; validation of the field performance over time and calibration procedures; and sensor abandonment.

1.3. Methodology

In order to meet the scope and objectives of this research project, an investigation and analysis plan was developed to determine the required performance characteristics of the perpetual pavement test section studied. The plan included a prediction of the expected performance characteristics of the pavement based on the design values determined during the initial test site construction as well as a validation of the performance using the sensors that are embedded in the test section and have been in use for the past five years. Finally, the investigation and analysis information were used to develop a framework for using non-destructive test methods to monitor the long-term performance of perpetual pavements. The specific objectives to complete the research objectives were, as summarized in the Figure 1 Flowchart, as follows:

1. Review literature on perpetual pavement design and construction. Review the pavement designs completed for the perpetual pavement test sections as well as the key performance indicators used for the designs.

2. Review literature on sensor installation techniques, performance characteristics of embedded sensors, recommended sensor maintenance and data collection frequency for perpetual pavement sites.

3. Review literature on non-destructive testing techniques for the in-situ determination of long term performance characteristics of perpetual pavements as well as advanced data analysis techniques.

4. Review how the test section was constructed, including the number of sensors used as well as the specific types, the sensor placement and installation, data collection parameters and the expected response of the sensors to different load applications.

5. Complete a detailed pavement condition survey to identify any cracking or other deficiencies that may have developed over the first five years of service of the perpetual pavement test section.

6. Complete non-destructive falling weight deflectometer (FWD) testing at a high frequency in order in to determine the precise location and depth of the sensors embedded in the test section.

7. Complete non-destructive falling weight deflectometer testing at different load levels to evaluate how the different strain levels are developed in the perpetual pavement test section in order to evaluate the performance of the installed sensors.

8. Using the data obtained from the embedded sensors, evaluate the response of the sensors to the applied loads and compare these values to the expected response based on the material characteristics. Analyze any difference between the actual measured and the expected response in order to determine why there is a difference and to calibrate the sensors to the actual loading as necessary.

9. Re-test the site using the new calibration factors in order to validate the new measurements being taken from the embedded sensors.

10. Provide conclusions and recommendations based on the results of the field investigation program on the performance of the sensors after five-years of service as well as any considerations that need to be given to sensor calibration and/or verification over time.

11. Using the results gained from the research, develop a framework for the monitoring of the performance of perpetual pavements using embedded sensors coupled with other non-destructive testing techniques.

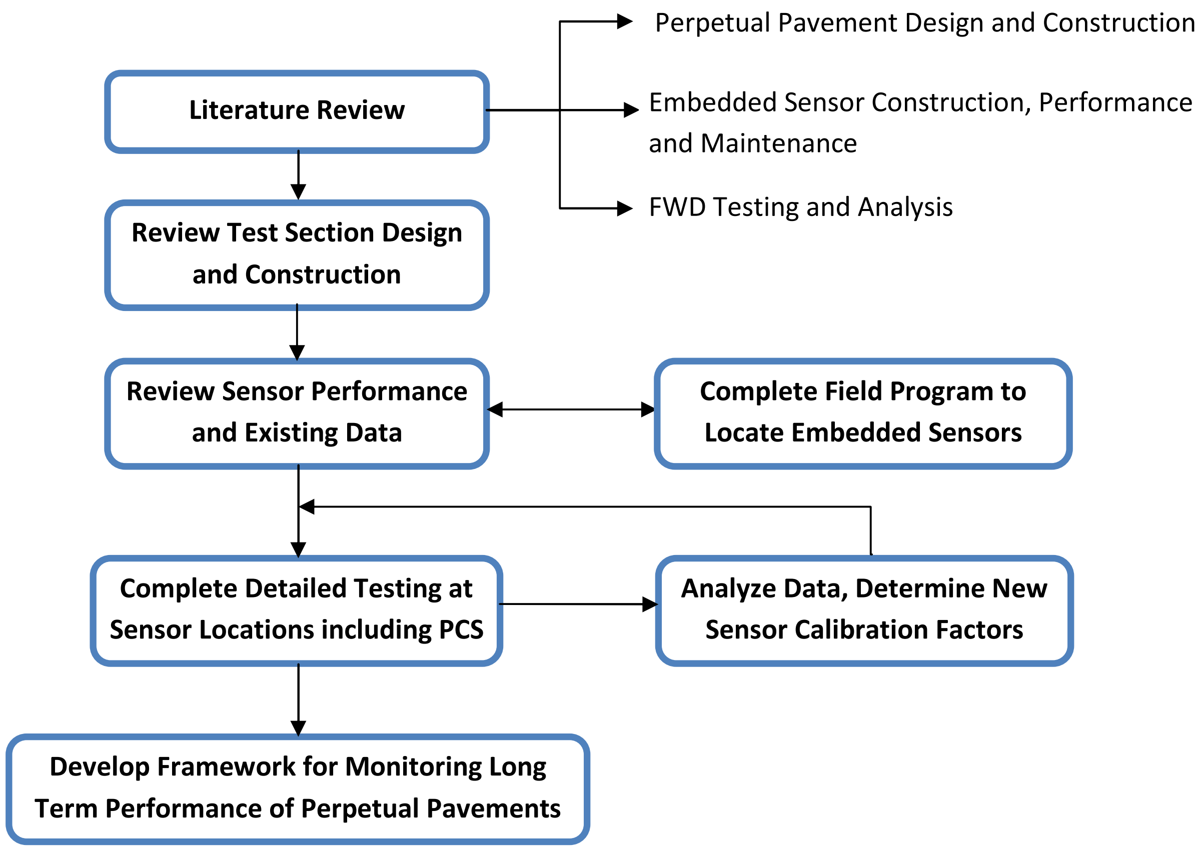


Figure 1 - Research Methodology Flowchart

1.4. Organization of Thesis

This Thesis is comprised of eight Chapters that include complementary figures, tables and equations to provide additional description and clarification for the concepts and analysis that have been presented.

Chapter 1 provides an introduction of the concepts that are being studied as part of this Thesis and explains the scope and objectives of the research project. A methodology is also discussed which outlines the approach which will be used to investigate the objectives.

Chapter 2 consists of an extensive review of the current literature related to perpetual pavements and embedded sensor use. The Chapter begins with a treatment of the state of the practice in perpetual pavement design and discusses the engineering surrounding the key performance parameters that relate to the long term performance of these types of pavement. This is followed by a review of the falling weight deflectometer (FWD) which is a common non-destructive testing technique that can be used to monitor perpetual pavements as well as the engineering used to interpret FWD data. The Chapter also includes a review of embedded sensor technologies that have been used to monitor the key performance parameters for perpetual pavements and discusses how they have been typically used.

Chapter 3 provides details on how the test section was constructed at the Capital Paving Inc. asphalt plant in Guelph Ontario. The details include the location of the test section within the asphalt plant yard, the thickness of the individual pavement structural layers, the types of materials that were used for each structural layer, and a summary of the embedded sensor types and location within the test section.

Chapter 4 consists of an analysis of the data that was obtained from the non destructive testing completed in the test section which was used to determine the precise location of the sensors. A FWD testing program was completed at a very high testing frequency and at three different offsets in order to determine the longitudinal and transverse locations of the sensors as well as their embedment depth. This was achieved by analyzing the response of the embedded strain gauges and earth pressure cell as the FWD testing was completed at different locations.

Chapter 5 presents the results of the verification testing that was completed on the embedded strain gauges. Detailed testing and analysis was completed at the locations of the strain gauges in order to determine if the recorded stresses and strains matched those that would be expected from the loads that were applied to the pavement surface. The difference between the measured and calculated strains and deflections were used to develop revised calibration factors for the gauges.

In Chapter 6, the results and lessons learned from the field testing and analysis of embedded sensor performance were used to develop a framework for monitoring the long term performance of perpetual pavements. The framework includes discussion on the design of instrumented sections, considerations that should be made during construction, and calibration of the sensors while in service.

Chapter 7 contains the conclusions that can be drawn from the research and outlines recommendations on how this research can be applied to future projects as well as recommendations on future complementary research or investigations that could be completed.

2. LITERATURE REVIEW

In order to evaluate the performance of embedded sensors in a perpetual pavement test section, it is first important to understand the key performance requirements of perpetual pavements as well as the expected performance of embedded sensors which are two main topics covered by this literature review. As the focus of the research is the use of the Falling Weight Deflectometer (FWD) in the evaluation of perpetual pavements and embedded sensor performance, a comprehensive review of the operation and application of an FWD has been completed, as well as the scientific methods used to analyse the resulting measured deflection basins and to determine pavement performance characteristics. Finally, the concepts of pavement management have been reviewed in order to evaluate how this research can be applied to the timely rehabilitation and preservation of perpetual pavements.

2.1. The Perpetual Pavement Concept

For the latter part of the 20th Century, most pavements in North America were designed based on experience gained over time at an agency or in accordance with empirical relationships developed from research at pavement test road. The most common empirical design method used in North America is the American Association of State Highway and Transportation Officials (AASHTO) design guides (1961, 1972, 1993) based on the empirical relationships developed from the AASHO Road Test. The most advanced of these design guides is the AASHTO 93 Guide for the Design of Pavement Structures [AASHTO, 1993].

By most accounts, the AASHTO 93 design method, particularly when calibrated, has worked quite well for pavements of limited design life (20 to 25 years) and equivalent single axle loads (ESALs). However, over time, as traffic volumes and heavy vehicle loadings increased on the nation's roads and highways, the design of the asphalt concrete thickness in flexible pavement structures also rapidly increased which exposed a limitation of the AASHTO 93 design method: the empirical models (flexible and rigid) were developed based on the AASHO test road sections' performances for a maximum of 8 million ESALs, which have been used (extrapolated) to design flexible pavements for over 100 million ESALs, resulting in an ever increasing asphalt concrete thickness and overly conservative designs [Newcomb, Buncher & Huddleston, 2001]. There is now a realization by most researchers and practitioners that there is a point with these deep strength flexible (asphalt) pavements where the heaviest loads are "readily accommodated" by the overall pavement structure and any additional asphalt concrete only adds cost.

At the same time as the traffic volumes and vehicle loadings were increasing, research was being completed on mechanistic-empirical pavement design methods (MEPD) involving the computation of a critical (maximum) amount of tensile strain for long-term fatigue performance. The earliest of the reported research projects was completed by Monismith who found that if the tensile strain at the bottom of asphalt concrete was limited to 70 $\mu\varepsilon$, then fatigue (bottom-up) cracking could be mitigated or even prevented [Monismith & McLean, 1972]. Additionally, structural rutting is limited if the vertical compressive strain at the top of the subgrade is limited to 200 $\mu\varepsilon$. While there has been much research into determining the optimal critical tensile strain for flexible pavements [Monismith, 1992, Nishizawa, Shimeno & Sekiguchi, 1996, Wu, Siddique & Gisi (2004), Timm & Newcomb, 2006], the use of critical strains in mechanistic-empirical design along with optimized pavement structural materials allows for pavement designs suitable for high heavy traffic volumes and heavy loads that are more sustainable and use much less asphalt concrete.

The concept of deep strength pavements has evolved into a unified design approach using the mechanistic-empirical concepts initially developed by Monismith and limiting pavement responses such as stresses, strains and deflections to thresholds where pavement distresses will not occur for the design ESALs. This unified approach was first presented by the Asphalt Pavement Alliance (APA) in 2000 under the now common concept of Perpetual Pavement.

The concept of a Perpetual Pavement can best be described as a "long-life pavement that is well designed and constructed that can last indefinitely without deterioration in the structural elements provided it is not overlooked and the appropriate maintenance is carried out" [Ferne, 2006]. In order to last indefinitely, the most critical structural distresses that designers must consider are bottom-up fatigue cracking and structural rutting.

2.1.1. Bottom-up Fatigue Cracking

Bottom-up fatigue cracking occurs when repeated heavy loading causes repeated strains at the bottom of the asphalt layer which are high enough to exceed the fatigue limit where the asphalt concrete is able to withstand the applied strains as shown in Figure 2. According to the Asphalt Pavement Alliance, this is sometimes referred to as the fatigue endurance limit (FEL) [Newcomb, Willis & Timm, 2010]. When the fatigue endurance limit is repeatedly exceeded, bottom-up cracking will occur, and the bottom-up cracking will eventually propagate through the overlying asphalt concrete layers to the surface which then allows water into the pavement structure leading to the acceleration of other pavement distresses.

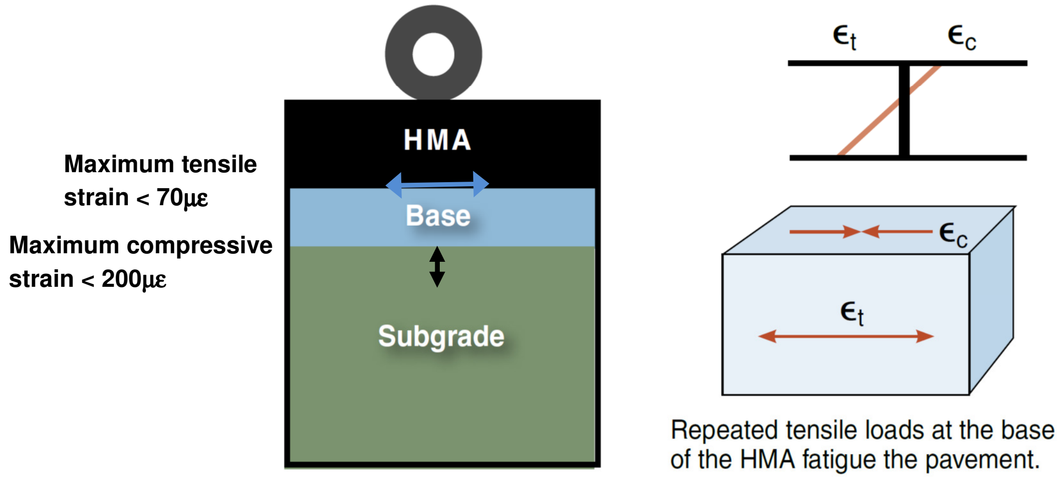


Figure 2 - Fatigue Cracking Schematic [Newcomb, Willis & Timm, 2010]

If the tensile strain at the bottom of the asphalt concrete layer is reduced below the FEL of the mixes, the critical location of distress is relocated to the surface of the asphalt concrete structure where the wearing surface is exposed to environmental degradation and tire/pavement interaction effects which may eventually cause top-down cracking [Mahoney, 2001]. Since this distress begins at the surface and travels downwards, the rehabilitation consists of a relatively inexpensive standard mill and overlay or hot in-place recycling treatment to the depth required to eradicate the top-down cracking. This rehabilitation treatment also serves to renew the pavement frictional characteristics as well as improve the overall ride quality.

Current literature suggests that most pavement engineers agree that limiting tensile strain at the bottom of asphalt concrete structure to 70 με as suggested by Monismith [Monismith & McLean, 1972], will mitigate if not eliminate the development of bottom-up fatigue cracking. However, evaluation of in-service performance of long-life pavements as well as a number of research projects have indicated that this value may be somewhat conservative. In Japan, Nishizawa et. al. completed an analysis if in-service pavements and found that in their experience, fatigue cracking began to appear at strain levels greater than 200 με [Nishizawa, 1997]. Wu et. al. took a different approach [Wu, Siddique & Gisi, 2004]. Using falling-weight deflectometer (FWD) deflection data collected from the Kansas Turnpike perpetual pavement project, strain levels of 96 to 158 $\mu\varepsilon$ were predicted based on the back-calculated stiffness.

Thompson evaluated the test results from over 100 different mixes tested for the Federal Aviation Administration (FAA) and Illinois Department of Transportation (IDOT) tested in the University of Illinois laboratory [Thompson & Carpenter, 2006]. His research demonstrated the concept of a fatigue endurance limit which is described as the break point where the traditional fatigue curve begins to flatten out [Thompson & Carpenter, 2006]. This is shown graphically in Figure 3. The results shown in Figure 3 also indicate that different mixes have different fatigue endurance limits which may explain the different design strain levels obtained through various research. Thompson's research concluded that using a design value of 70 $\mu\varepsilon$ would guarantee extended performance (design confidence of 100 percent which is extremely conservative) however extended fatigue life could be reliably reached at strain levels between 70 and 100 $\mu\varepsilon$.

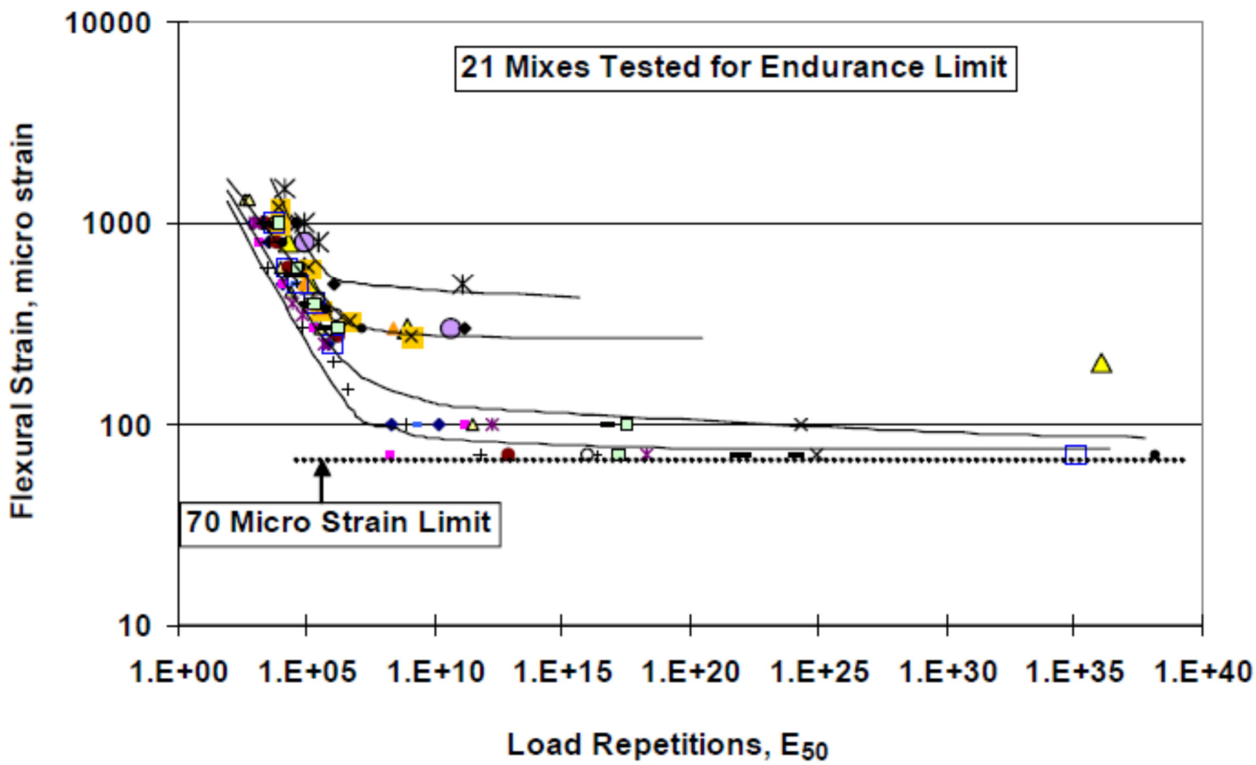


Figure 3 - Strain-Load Relationship Illustrating the Fatigue Endurance Limit [Thompson & Carpenter, 2006]

Testing completed by the National Center for Asphalt Technology (NCAT) by Willis et al. using perpetual pavements constructed at the test track has shown that their pavements are capable of withstanding strain levels in excess of 100 με [Willis, 2009]. Willis and Timm have postulated that the design strain level can be made less conservative by calculating a design strain level at a cumulative percentile of all expected strains over the design life [Willis, Timm, 2009b]. The authors suggest that as an example, the resulting strain ratio for a confidence interval of 95 percent would be 2.45.

2.1.2. Structural Rutting

Structural rutting occurs when the compressive strain caused by repeated heavy loading at the pavement surface exceeds the capacity of the pavement foundation elements (the granular base/subbase and subgrade) which causes large permanent deformation in the entire pavement structure. While structural rutting is considered to be rare in properly designed modern pavement structures, structural rutting requires significant and costly major rehabilitation or even reconstruction to address the principal causes and as a result is considered to be a critical parameter in Perpetual Pavement design. It is commonly accepted amongst pavement engineers that the maximum allowable vertical compressive strain at the top of the subgrade for perpetual pavement design is 200 με.

2.1.3. Perpetual Pavement Design

In order to meet the design structural and fatigue characteristics, each layer of the pavement structure must be thoughtfully designed to work in tandem with the other components throughout the design life. The perpetual pavement design concept is described in detail by the Asphalt Pavement Alliance [Newcomb, Willis & Timm, 2010] with a graphical representation of the APA design concept shown in Figure 4. The design concept can be summarized as follows:

1. The perpetual pavement must be constructed on top of a strong foundation consisting of a sound subgrade that is free of deleterious material and will not be unreasonably affected by the effects of weather and seasonal changes as well as well constructed granular subbase and base materials that protect the subgrade during construction and provide uniform support and positive drainage for the pavement structure.

2. The base of the asphalt concrete layer should be constructed of a fatigue resistant hot-mix asphalt that is strongly resistant to bottom-up cracking. This layer must also be able to withstand the effects of freeze-thaw cycles without cracking due to the thermal stresses.

3. The intermediate layer should be composed of a high modulus, rut-resistant layer that should be able to resist rutting for the duration of the design life.

4. The surface course should be comprised of a high quality, renewable mix that provides excellent skid resistance, low pavement tire noise, good ride quality, and is resistant to top-down cracking.

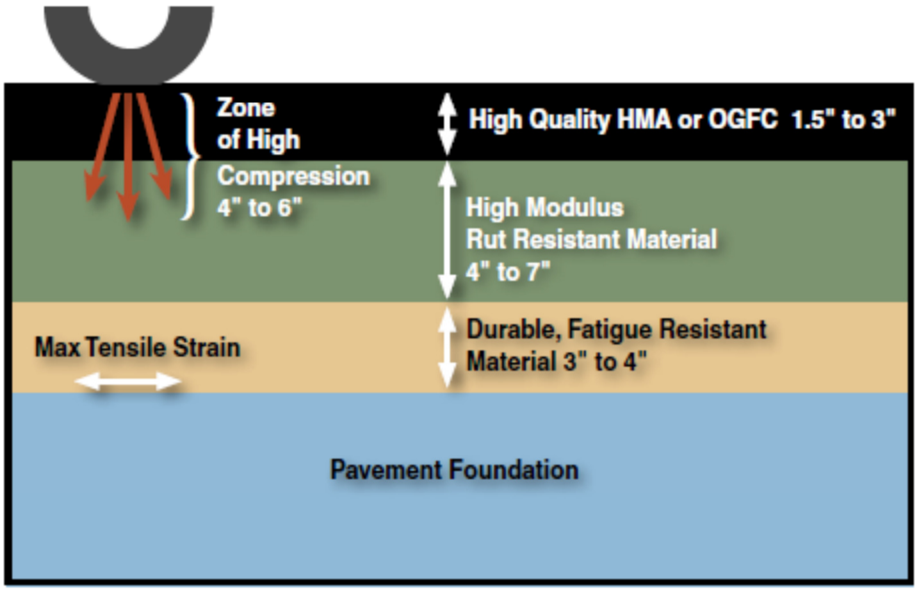


Figure 4 - Perpetual Pavement Design Concept [Newcomb, Buncher & Huddleston, 2001]

Pavement Foundation

The pavement foundation forms the backbone of the pavement structure and it is critical to the performance of a Perpetual Pavement. The foundation is initially required to be able to support construction traffic during placement of the hot-mix asphalt layers and to provide firm resistance to the compactors in order to achieve the in-situ air voids required to meet the design performance requirements of the mixes. After construction, the combined foundation elements then must be designed to provide adequate support throughout the design period, and specifically when the subgrade may be in a seasonally weaker condition, and to mitigate the effects of volumetric changes due to freeze-thaw cycles in cold climates. Given the length of time the pavement is expected to be in service, it is important that designers take into consideration the effects of climate change on the pavement foundation and/or build in sufficient conservatism into the design to ensure that the foundation will be able to withstand climatic effects which may differ from those experienced in the past 50 to 100 years.

The pavement foundation typically consists of a mix of compacted natural or chemically stabilized subgrade, select subgrade material, stabilized granular material, granular subbase and granular base. The combination of the different foundation elements will depend on what is required by the designer to provide the design stiffness during construction and over the design life and to protect the pavement structure from environmental effects such as thaw weakening and non-uniform frost heave.

There are a number of different criteria which are used by different jurisdictions to determine if the native or recompacted native subgrade soils will be able to provide the required stiffness to prevent overstressing of the subgrade soils during construction. The Illinois Department of Transportation (IDOT) has developed a Subgrade Suitability Manual [IDOT, 1982] which has been calibrated to the typically fine grained soils in the State and provides guidance on testing and acceptance of site subsoils for constructability. The manual classifies the subgrade based on different classification tests (CBR for instance) and then provides guidance on the thickness of granular material required for overlay depending on the results of the classification tests. It should be noted that the IDOT procedure determines appropriate remediation action based on three general categories: CBR > 8 - no remedial action required; CBR between 6 and 8 - remedial procedures optional; and CBR < 6 remedial action required. A graphical representation of the IDOT requirements which was reproduced by the APA is provided in Figure 5.

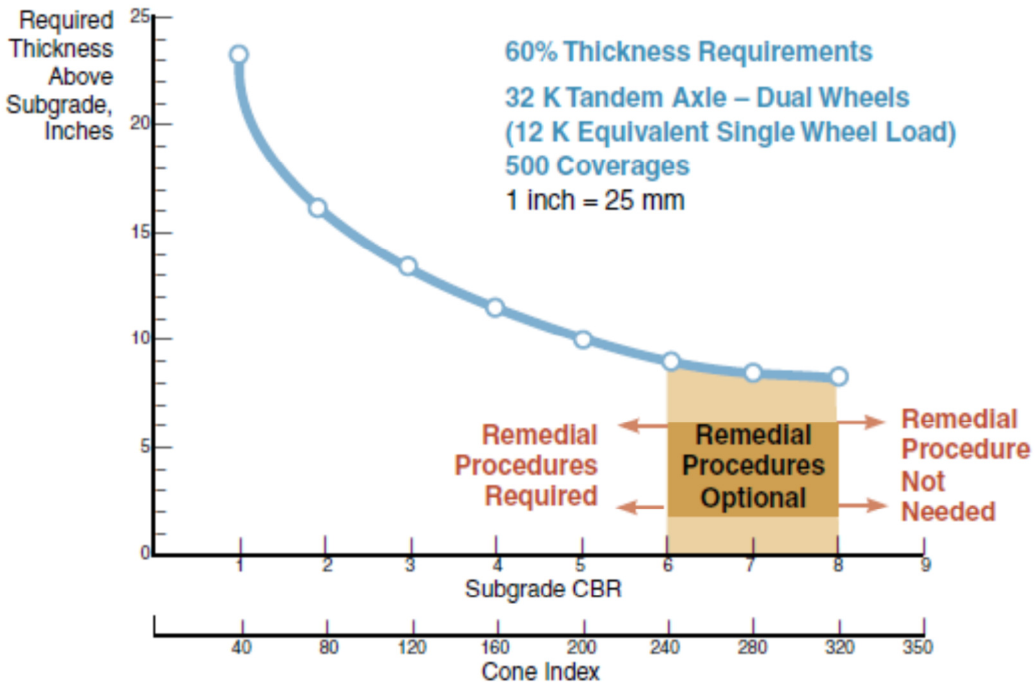


Figure 5 - Illinois Granular Thickness Requirement for Foundation [Newcomb, Willis & Timm, 2010]

Von Quintus recommends that the stiffness of the pavement foundation be determined at the top of the pavement foundation based on a composite resilient modulus of all of the foundation layers. He suggests that in order to achieve constructability, the composite modulus should be greater than 172,000 kPa (25,000 psi) [Von Quintus, 2001]. If this modulus is not achieved, then the susceptible soils should be replaced by high quality granular material, chemically or mechanically stabilized, or improved using geogrid/geofabrics to increase the stiffness prior to construction of the hot-mix asphalt layers.

Another important aspect of the pavement foundation, and more specifically the pavement subgrade is the potential for volumetric changes due to either expansive clays or freeze-thaw cycles in frost susceptible subsoils. For expansive soils, special precautions are required to ensure that the potential effects of expansion on the pavement structure are either eliminated or significantly reduced prior to pavement construction. Options to treat this type of soil include chemical stabilization in conjunction with carefully designed and maintained pavement subdrainage. Moderate to highly frost susceptible soils are also a concern in cold climates and require that the pavement structure be protected from frost related (differential) heaving as well as weakening during the spring season. Typical protection guidance provided by many cold weather agencies involves removal of frost susceptible materials within the design frost penetration depth and replacement with non-frost susceptible material such as select subgrade (typically a sand to sand and gravel sized material) or granular base or subbase material as well as providing enhanced pavement subsurface drainage. As the design frost penetration depth increases, removal of all frost susceptible material becomes impractical and agency guidance suggests that the frost susceptible material be removed to a depth where the frost related movements will be uniform. This depth ranges from 40 to 70 percent of the design frost penetration depth depending on the classification of the roadway [Tighe et.al, 2014].

The upper portions of the perpetual pavement foundation are composed of the granular subbase and granular base layers. The purpose of these layers is to provide uniform support to the upper asphalt concrete layers during the design life and to promote positive drainage of the pavement structure. These layers may consist of unbound natural or crushed quarried aggregate as well as premium recycled aggregate or chemically stabilized materials depending on the requirements of the design. The thickness of these layers typically range from a minimum of 150 mm for high quality subgrades to a maximum of 600 mm for very weak subgrades [Newcomb, Willis & Timm, 2010]. It should be noted however, that for very weak subgrades other forms of strengthening such as mechanical or chemical stabilization or the use of geogrids/geotextiles are typically used as a more sustainable design option rather than using thick granular lifts unless these granular layers are also designed to provide the necessary frost protection to the pavement structure. Research by Ovik et. al has shown that seasonal adjustment factors must be considered in the design for unbound granular layers when used in cold climates [Ovik, Birgisson & Newcomb, 1999]. They showed that during the winter, the unbound granular layers are in a frozen condition and provide supplemental support for the pavement structure (increase factor of 22 times the design strength). During the progression of the thaw in early spring, the unbound granular layers are in their weakest condition which results in reduced support for the pavement structure (decrease factor of 0.65 times the design strength).

Fatigue Resistant Lower Asphalt Concrete Binder Course Layer

The lowest layer in a perpetual pavement structure serves the key function of being highly durable and must be able to resist bottom-up cracking due to the repeated bending loads expected over the design life. The APA recommended approach is the use of what is called a rich bottom mix (RBM) which is designed for optimum durability and fatigue resistance as shown in Figure 6. The RMB can be incorporated into the perpetual pavement design using APA's design software called PerROAD. It should be noted however that the optimization of this layer is still not well understood and there is currently a number of research projects in progress which are working towards defining the mix characteristics which provide the are best suited to perpetual pavement performance.

A rich bottom mix is essentially a binder course hot-mix asphalt layer which is designed using a higher asphalt content which allows the mix to be compacted to a higher density which has been shown to improve both the durability of the mix as well as the fatigue resistance. The additional asphalt cement content has been shown to prevent the formation and growth of bottom-up cracking possibly due to the ability of the mix to heal between strain applications. However it should also be noted that increased fatigue resistance can also be attributed to the higher density that can be achieved with these mixes.

Kassem et al. have found in their research that because this layer will be in long-term contact with water that could be present beneath the asphalt concrete layer, the resistance to the effects of moisture damage (moisture susceptibility) needs to be confirmed for the mix during the mix design process [Kassem, et. al., 2008].

As mentioned previously, the lowest binder course layer must be designed to resist the design fatigue as determined by the fatigue endurance limit (70 $\mu\varepsilon$ or above depending on the approach used). Regardless of the properties of the properties of the mix type used, another approach to lowering the amount of strain at the bottom of the pavement is to control the strains using the thickness of the pavement. This concept, along with the use of RBM is illustrated in Figure 6.

High Modulus Rut Resistant Asphalt Concrete Layer

The intermediate layer of the perpetual pavement should consist of a mix which has similar characteristics of durability as the underlying lower binder course layer but must have the added benefit of being highly resistant to rutting.

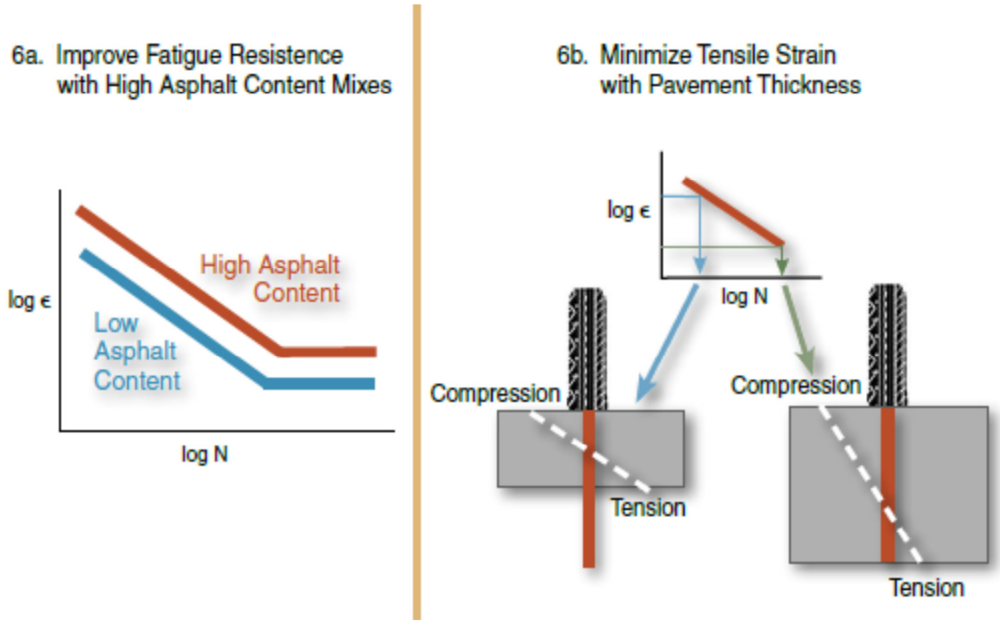


Figure 6 - Resistance to Fatigue in Lower Binder Course Layer [Newcomb, Willis & Timm, 2010]

The most significant portion of the rut resistance in this layer is developed through the internal friction generated in the aggregate skeleton by stone on stone contact in the coarse aggregate. In order to develop the optimal aggregate skeleton for rutting resistance, the mix should be designed using high quality crushed and graded aggregates blended together in accordance with the requirements of the Superpave mix design specification. It should be noted that mixes that are created with a large Nominal Maximum Aggregate Size (NMAS) may be susceptible to segregation and to lower in-place air voids which will cause the mix to have a higher porosity and increase the likelihood of long term moisture susceptibility problems. In these cases, it is recommended that the Bailey Method be used to analyze the mix design and make the necessary adjustments to improve its VMA, in-situ air voids, as well as the overall workability of the mix prior to it being placed [Vavrik, Pine, Carpenter & Bailey, 2008].

As this mix will be subjected to high stresses from wheel loads, the selection of an appropriate performance graded asphalt cement (PGAC) grade, and more specifically the high temperature grade is also an important consideration to prevent shear failure and associated rutting in this layer. Guidance on the design and construction of perpetual pavements indicates that the high temperature grade that is used in the surface course layer should also be used for the intermediate/ rut-resistant layer to assist the mix in withstanding the expected shear forces over the design life. It should be noted however that due to the high temperature gradient in the asphalt concrete layer, the lower temperature grade may be reduced by one grade from the grade used in the surface course layer as determined using software such as LTPPBind.

The highest shear stresses are expected to be developed in the top 100 mm of the asphalt concrete layer in perpetual pavements. Depending on the total design thickness of the intermediate/rut-resistant layer, consideration could be given to using a lower high temperature grade in the lower lifts of the intermediate/ rut-resistant layer if multiple layers are planned to be constructed as determined using software such as LTPPBind. This should only be considered if there will be a significant cost savings in reducing the performance grade of the lower layers in this lift, and if it is considered to be practical from a materials and construction standpoint for the specific site that is being constructed.

Asphalt Concrete Wearing Surface

The main structural functions of the asphalt concrete wearing surface in perpetual pavements are to withstand the high shear forces generated from the design traffic and the associated potential for rutting, as well as to mitigate the development of top-down cracking until the planned subsequent rehabilitation treatment. As this layer is also in contact with vehicle tires as well as the local environmental conditions, this layer is also responsible for providing high quality friction, low amounts of tire-pavement noise and must be able to withstand the weathering and wear effects of the climate and maintenance activities (winter maintenance, for instance). It is also important that the appropriate PGAC grade is selected for this mix that will withstand the design traffic and weather conditions over the design life.

The Asphalt Pavement Alliance suggests that an ideal surface course mix that meets all of these requirements for very high (urban) traffic zones is stone mastic asphalt (SMA) [Newcomb, Willis & Timm, 2010]. The benefits of using stone mastic asphalt mixes include:

- Excellent rutting resistance due to the well developed stone skeleton;
- Excellent stiffness and durability generated from the matrix (combination of polymer modified binder, mineral filler, and fibres);
- Very low permeability;
- Excellent frictional characteristics (beware of early friction issues); and
- Excellent wear resistance.

For lower traffic volumes, and more specifically for lower average annual daily truck traffic (AADTT) volumes, a well designed, dense graded Superpave surface course mixture may be considered more appropriate. Regardless of the mix used, it is recommended that any mix considered for use as the surface course hot-mix asphalt be tested for its rut susceptibility characteristics (Asphalt Pavement Analyzer, for instance) at a minimum prior to use in construction.

2.2. Use of Embedded Sensors to Monitor Pavement Performance

Recent advances in computer technology as well as the design and construction of microelectronic equipment have allowed pavement engineers the ability to measure the stresses, pressures, temperature profiles, moisture condition, deflection profile, as well as wheel wander using embedded sensors rather than solely relying on destructive and/or non-destructive (surface based) monitoring techniques. When used together with calibrated transfer functions, the mechanistic response parameters gathered by these techniques can be used to predict and monitor pavement performance and life.

The design of perpetual pavements is predicated on limiting the amount of tensile strain at the interface between the asphalt concrete and the lower portion of the pavement structure to a certain design level as well as limiting the amount of compressive strain at the interface of the bottom of the pavement structure and subgrade to below a certain design level. The common way to predict pavement performance is the use of theoretical strains and stresses gathered from measured or estimated design traffic information coupled with calibrated transfer functions. Recent technological advancements, which up to this point have been more commonly used by researchers, allow the use of specially designed sensors which are embedded within the pavement structure at strategic locations that are capable of accurately measuring the stresses and strains which are being generated by existing traffic.

The critical location for measuring the amount of horizontal strain in the pavement structure is at the bottom of the asphalt concrete layer and directly below the wheelpath. In this location asphalt strain gauges (ASGs) can be used to measure the load-induced response of the asphalt concrete and to predict the long term fatigue performance of the pavement. An asphalt strain gauge works by measuring the change in electrical resistance of embedded wires as the gauge is expanded (stretched) under loading. The resulting change in resistance results in a change in the measured voltage across the circuit. The change in voltage is multiplied by a gauge factor (which is unique for each gauge) to calculate the corresponding measured strain across the gauge.

Asphalt strain gauges are specially manufactured for the harsh conditions expected during installation. The gauge itself is typically made of special plastics that are both heat and water resistant with a specially shielded core that can handle installation temperatures of up to 200°C which exceeds those that are expected at the time of construction. ASGs are typically constructed using a Wheatstone bridge circuit which is capable of compensating for temperature and lead resistance. They are installed to measure the strains generated in either the longitudinal (the direction of traffic) or transverse direction (or both). Research completed at the NCAT Pavement Test Track (PAVETRACK) and the National Airport Pavement Test Facility provide guidance on the best use sensors as well as the key sensor installation techniques [Timm, 2009, GARG & HAYHOE, 2002].

The most common type of embedded sensor used to measure the vertical compressive stress at the interface between the granular base and the subgrade is using earth pressure cells (EPCs). An EPC is designed to measure the amount of pressure applied to the sensor by the combination of the overlying earth pressure as well as any supplemental static and/or dynamic loads. The EPC's manufactured by RST Instruments work by placing a transducer within two circular, sealed stainless steel plates which are filled with deaired glycol forming a closed hydraulic circuit [RST, 2014]. The stress measured by the transducer is converted to a calibrated signal which can be read using either handheld devices or a datalogger.

There are a couple of drawbacks to using embedded sensors to monitor pavement performance. The first drawback is that sensors are difficult to install. Even with very special care taken during construction, it should be expected that only about 80 percent of the installed sensors will survive the construction process [Timm, 2009]. In addition, sensors such as ASG's sensors should be checked for variability caused by being reoriented (moved from the longitudinal and/or transverse axis) during subsequent overlay with hot-mix asphalt. The sensors also have a finite lifespan. For instance, one ASG manufacturer states that their gauges will provide a lifespan in the range of less than 10^5 to 10^6 repetitions which is far fewer repetitions than the expected lifespan of a typical or perpetual flexible pavement. Once cracking is initiated in the pavement surface, research has shown that the readings measured in embedded sensors can become more variable/less reliable which may reduce or eliminate the applicability of the data from these sensors. Finally, it must be noted that there are costs involved with collecting and monitoring embedded sensors including maintenance on data loggers, batteries, power, and data retrieval (such as wireless data rates) that have to be considered.

2.3. Falling Weight Deflectometer (FWD)

As outlined by Ferne, a perpetual pavement must be well designed and constructed, but will only last indefinitely without deterioration in the structural elements if they are not overlooked and if appropriate maintenance is carried out [Ferne, 2006]. In this regard, the importance of the ongoing evaluation of the pavement surface condition as well as the structural condition during the pavement service life to choose the most appropriate maintenance or preservation treatment cannot be overlooked. The FWD is a non-destructive device which provides accurate, repeatable evaluation of the structural condition of pavement and it has been used to test pavements around the world.

The FWD is an enclosed or trailer mounted device (Figure 7) which is designed to impart a dynamic impulse load to the pavement which is similar in force and load duration to that which would be applied by a moving vehicle at 60 km/h. The impulse load is applied to a loading plate by dropping a weight package on a damping system. The resulting pavement deflection is typically measured by nine vertical displacement transducers (up to 15 transducers can be monitored in some devices) spaced at predetermined longitudinal (and potentially transverse) distances from the loading plate in order to measure the shape of the deflection basin (Figure 8 and 9). The transducers (or geophones) are mounted on a bar that is lowered automatically to the pavement surface with the load plate at each test location. The force applied to the pavement structure is measured by a load cell mounted on the top of the load plate. The loading plate used during most test setups is 300 mm in diameter, but 450 mm diameter plates are also available for use in testing weak pavement structures (when lower test forces are required) or when using high drop loads (to apply a more evenly distributed load). The plates that are typically used are composed of solid steel with special rubber mats attached to the bottom of the plate to provide even load distribution during testing. Segmented load plates are also available which are able to provide a better load distribution when pavement surfaces are particularly uneven.

The air temperature as well as the pavement surface temperature are automatically recorded and captured in the FWD database. Depending on the test procedure used, the asphalt concrete mid-depth temperature is typically recorded manually by the operator on an hourly basis (or more frequently during sudden temperature changes). The mid-depth temperature is used to normalize the recorded deflections to deflection values at a standard temperature (20°C). GPS and differential GPS systems are available to accurately measure the location of each test point.

A microcomputer monitors and controls the complete operations of the FWD from the main vehicle, with the test data stored in a database for later processing. Most FWD systems are capable of outputting the test information into a Pavement Deflection Data Exchange (PDDE) formatted output which is compatible with most pavement design and analysis software. The testing equipment and procedure most commonly followed are in general accordance with ASTM D4694-09 [ASTM, 2009b]. Examples of typical truck and trailer mounted devices is provided in Figure 7.

Figure 7 - Truck and Trailer Mounted FWD Devices [Duclos, 2014; FHWA, 2011]

The selection of the spacing between the deflection sensors varies depending on the agency and the type of pavement being tested. For instance, if the critical parameter being tested is the deflection basin, then the sensors will be spaced in order to provide an accurate representation of the basin based on the applied load and the overall stiffness of the pavement. If load transfer is being tested, the sensors will be spaced in order to evaluate the deflection loss across a longitudinal or transverse joint. The most common sensor spacing used by US agencies is defined in Version 4.1 of the Long-Term Pavement Performance (LTPP) manual for FWD measurements [Schmalzer, 2006]. In Ontario, the typical FWD sensor spacing (seven sensors) for flexible and rigid pavements is outlined in the Ontario Ministry of Transportation Material Engineering Research Office (MERO) Falling Weight Deflectometer Guideline MERO-019 [Chan & Lane, 2005] as shown in Figure 8 and Figure 9 respectively.

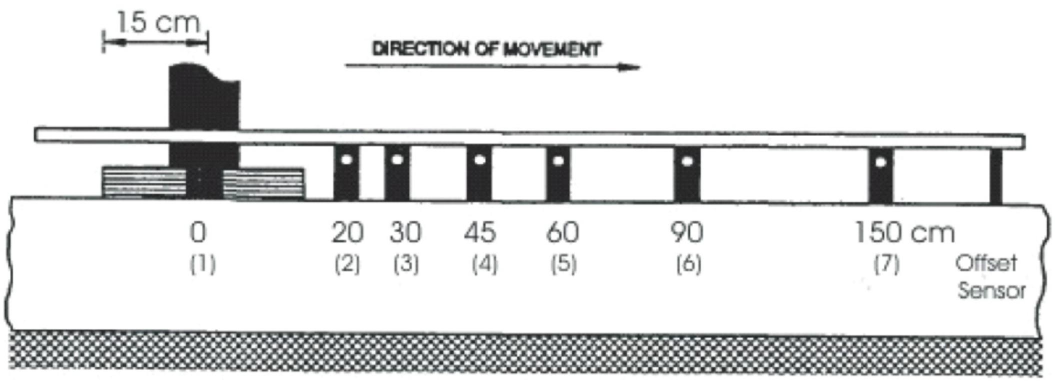


Figure 8 - Typical FWD Sensor Spacing for Flexible Pavement Deflection Test [Chan & Lane, 2005]

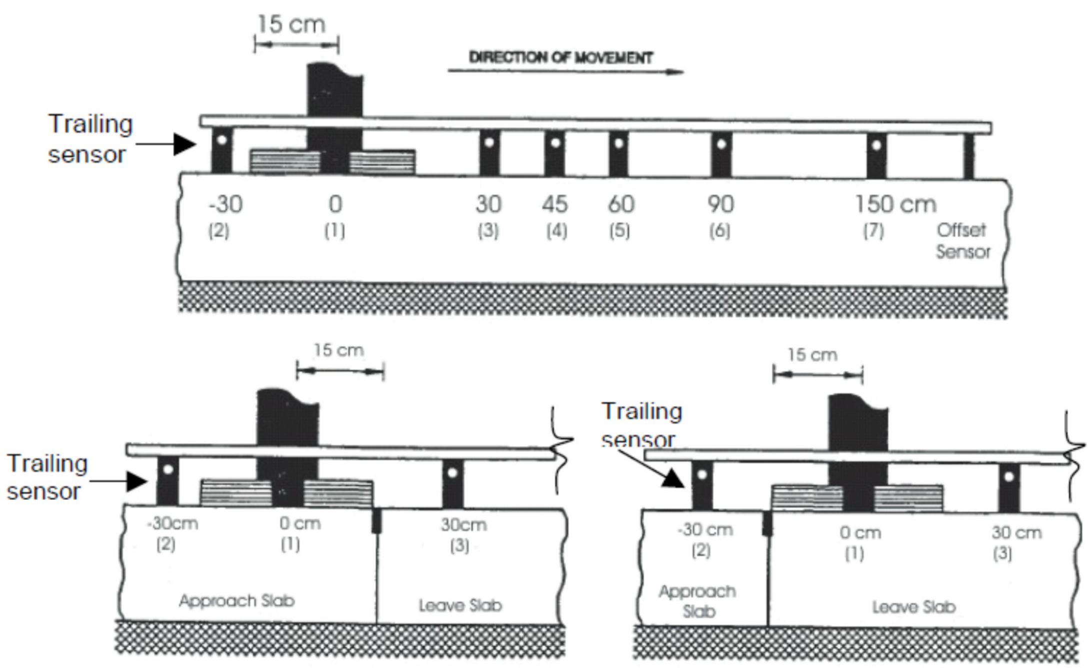


Figure 9 - Typical FWD Sensor Spacing for Rigid Pavement Load Transfer Test [Chan & Lane, 2005]

Similar to the selection of sensor spacing, the selection of target applied load varies from agency to agency depending on specific agency requirements such as the design vehicle loading. For instance, an airport agency that accommodates Boeing 747 aircraft will have much different design loadings than a typical highway agency which is typically only evaluating the effect of a standard 40 kN design load. As a result, by varying the drop height and weight package, a peak force ranging from 10 to 110 kN (240 kN for a Dynatest HWD) can be generated to simulate the wheel load of a wide range of vehicles.

The LTPP testing protocol requires that for flexible pavements, three seating drops be completed followed by four drops at each of the target load levels (26.7, 40, 53.4, and 71.2 kN respectively). It should be noted that this testing frequency is for research and most agencies use fewer drops at different load levels such as MERO which only requires one seating drop and three drops at the target load levels [Chan & Lane, 2005]. However, FWD testing can be performed on many combinations of pavement structures, as well as for many different types of design vehicle loadings and as a result, engineering judgement is required on a project-by-project basis to accommodate for different types of testing.

2.4. Theory of Back-Calculation Using FWD Deflection Data

Deflection measurements, whether using a Benkelman Beam (static) or a falling weight deflectometer (dynamic) are very common for most agencies. There are many agencies which have developed mechanistic-empirical design procedures using the response of the combined pavement structure, also known as the surface modulus, calculated under the load plate (centre deflection) or converting the measured centre deflection response to an equivalent static deflection. The design procedure then uses design charts to calculate the thickness of additional pavement structural materials (typically the overlay of granular or asphalt concrete materials) required to reduce the observed deflection to a lower deflection which is suitable for the design traffic. This method does not consider the performance of the individual pavement layers but rather the pavement system as a whole. Another method to complete a rehabilitation design is to use the deflection data to back-calculate the elastic properties of the individual layers in the pavement structure to use in rehabilitation selection and design.

There are many back-calculation programs (software) that are available, however an analysis completed by SHRP and the FHWA classify the back-calculation software into three main approaches: the equivalent thickness method (Elmod and Bousdef, for instance); the optimization method (Modulus and Wesdef, for instance); and the iterative method (Modcomp and Evercalc, for instance) [Von Qunitus & Simpson, 2002]. The equivalent thickness method is a very common and popular method for back-calculating layer moduli from FWD deflection data and is explained in more detail below.

When evaluating pavement structures, it is usual to assume that the materials used are linear elastic, homogeneous, and isotropic. The equations used to calculate stresses, strains and deflections in such as system (assuming a semi-infinite half space under a point load) were first developed by Boussinesq in 1885 [Boussinesq, 1885]. The Boussinesq equations were integrated in order to more closely simulate the effect of a circular distributed load which would be expected from a vehicle tire at the interface with the pavement surface. The equations can be used to calculate the stress, strain, and displacement, based on a loaded circular area with radius ‘a’ and uniform vertical stress 'σ_0', at a depth 'z' below the centreline of the applied load. The integrated equations are provided in Equations 2.1 to 2.5 [Dynatest, 2006].

$$\sigma_z = \sigma_o\left(1-\frac{1}{\sqrt{1+\left(\frac{a}{z}\right)^2}^3}\right) \quad \text{(Eq 2.1)}$$

$$\sigma_r = \sigma_t = \sigma_o\left(\frac{1+2\mu}{2}-\frac{1+\mu}{\sqrt{1+\left(\frac{a}{z}\right)^2}}+\frac{1}{2\left(\sqrt{1+\left(\frac{a}{z}\right)^2}^3\right)}\right) \quad \text{(Eq 2.2)}$$

$$\epsilon_z = \frac{(1+\mu)\sigma_o}{E}\left(\frac{\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}^3}-(1-2\mu)\left(\frac{\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}}-1\right)\right) \quad \text{(Eq 2.3)}$$

$$\epsilon_r = \frac{(1+\mu)\sigma_o}{2E}\left(\frac{-\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}^3}-(1-2\mu)\left(\frac{\frac{z}{a}}{\sqrt{1+\left(\frac{z}{a}\right)^2}}-1\right)\right) \quad \text{(Eq 2.4)}$$

$$d_z = \frac{(1+\mu)\sigma_o a}{E}\left(\frac{1}{\sqrt{1+\left(\frac{z}{a}\right)^2}^3}+(1-2\mu)\left(\sqrt{1+\left(\frac{z}{a}\right)^2}-\frac{z}{a}\right)\right) \quad \text{(Eq 2.5)}$$

$$R = Ea\left(\frac{\sqrt{1+\frac{z}{a}^2}^5}{(1-\mu^2)\sigma_o\left(1+\frac{3}{2(1-\mu)}x\left(\frac{z}{a}\right)^2\right)}\right) \quad \text{(Eq 2.6)}$$

$$\epsilon_r = z/2R \quad \text{(Eq 2.7)}$$

Where:
- σ_z = Vertical stress at depth z
- ϵ_z = Vertical strain at depth z
- ϵ_r = Horizontal strain
- d_z = Deflection at depth z
- σ_0 = Vertical Stress at surface (MPa)
- E = Elastic modulus (MPa)
- a = Loaded area radius (mm)
- z = Depth below pavement surface (mm)
- μ = Poisson's ratio
- R = Radius of Curvature

If the asphalt concrete layer is homogeneous and isotropic, then the horizontal strain at the bottom of this layer (ε_r), which is one of the critical parameters in performance of perpetual pavements, can be calculated by first calculating the radius of curvature of the plane at the bottom of the layer using Equation 2.6. The horizontal strain can then be calculated using Equation 2.7.

The issue with using Boussinesq's equations for back-calculation is that they assume that the materials below are completely homogeneous which is obviously not the case in the construction of pavement structures which incorporate different material layers with depth. In order to overcome this issue, one can use the transformations developed by Odemark to take a system consisting of linear elastic (isotropic) layers of different moduli and convert it to an equivalent system where the thickness of the layers are altered in such a way that they then have the same stiffness. With the layers homogeneous, the Boussinesq equations now apply. This is known as the Method of Equivalent Thickness (MET) [Dynatest, 2006].

The Boussinesq equations and Odemark's MET were originally intended to calculate the stress, strain, and displacement at various depths in a layered structure where the modulus of the individual layers is known, based on a load applied to the surface. When used to analyze deflection data from a FWD, the process is simply reversed by using the deflections measured at varying distances from an applied load to determine the deflection basin generated from the load and using this information to 'back-calculate' the modulus of the individual layers in the pavement structure.

Assuming that the pavement structural components (hot-mix asphalt, granular base and granular subbase) are homogeneous and isotropic is considered reasonable as these materials are produced from controlled (engineered) sources and their placement in monitored for uniformity during construction. The subgrade however, whether it is composed of layers of fill or native material, will in most cases be layered or will have moduli that vary based on such things as differences in moisture content with depth, or varying overburden pressures. In Boussinesq based back-calculation software, these variables are dealt with by treating them as a non-linearity. While this method is not entirely correct, it does reduce the very large errors that can be found in the resulting back-calculated subgrade modulus values and typically results in very good agreement between measured and calculated deflection basins. The non-linearity of the subgrade is calculated using Equation 2.8.

$$M_{sub} = C\,[\sigma_1/p_a]^n \quad \text{(Eq 2.8)}$$

Where: σ_1 = Major principal stress from FWD loading
p_a = Reference stress taken equal to atmospheric pressure
C and n are constants with C decreasing almost linearly with increase in moisture content and n varying from 0 (for linear elastic material) to -0.5

In order to further improve the accuracy of the Boussinesq method, numerical integration techniques can be used to match the measured deflection basin with a calculated deflection basin. In these methods, a theoretical deflection basin is calculated based on the stress level at the centre of the load plate and the error, which is calculated as the difference between the measured and calculated deflection basins, is then assessed. The moduli of the layers in question are then adjusted slightly based on the inputted convergence criteria with the error then recalculated. If the error in the calculated deflection basins are less than the measured deflection basin, then the calculated deflection basin is taken as the better solution and used in the back-calculation of the layer moduli.

The ability to measure the structural performance of the pavement layers is a valuable tool in the management of pavement assets. The ability to estimate the pavement performance characteristics such as the effective modulus as well as the strain levels at different locations within the pavement can be used to monitor the performance of the important aspects of a perpetual pavement such as the fatigue performance of the lower asphalt layers and the compressive strains at the interface of the granular base and subgrade.

2.5. Evaluation and Management of Perpetual Pavements

The construction of a perpetual pavement represents a substantial investment by an owner and requires not only good design based on quality material characterization as well as good traffic and climate predictions, but also high quality construction ensuring that the pavement structural components meet the expected performance and life of the design. What is not always discussed and possibly overlooked during the design stage but equally as important to the success of a perpetual pavement is the development of an efficient and effective evaluation and management program for the pavement throughout its service life. With an effective pavement management program which includes regular evaluation with some form of systematic pavement preservation, a perpetual pavement can not only have a long service life, but will also enhance safety while meeting or exceeding the service expectations of motorists.

The first component of effective pavement management for a perpetual pavement is the design and implementation of an evaluation program which is capable of assessing/evaluating the key performance characteristics of the pavement in order to provide the information necessary to choose the optimum and most timely maintenance, preservation or rehabilitation strategy for the pavement. Historically, many pavement management systems have relied on visual/surface pavement condition assessment methods to track the methods and rate of deterioration of the pavement in order to develop strategies for the management of the pavement over its life. While visual/surface methods are quite effective for monitoring traditional pavements, the durability and longevity of perpetual pavements relies on preventing the development of cracks and other distress which initiate at the bottom of the asphalt concrete layer. Visual/surface methods are reactive since they record distresses that have already appeared at which point it is too late to prevent. Therefore, in order to plan effective pavement preservation treatments for perpetual pavements, surface cracks (other than top-down cracks) cannot be used as a reliable indicator of structural condition or health of the pavement structure.

While visual/surface methods will still form an important part of pavement management of perpetual pavements to address conditions such as surface texture, smoothness, surface rutting, potholes, top-down cracking etc., the management of perpetual pavements will also need to consider methods that evaluate the key performance indicators of the pavement with depth. One way of accomplishing this would be the use of non-destructive testing methods, and more specifically embedded sensors.

The use of long-lasting and calibrated embedded sensors can form a part of an overall management plan for a perpetual pavement and be especially helpful in planning pavement preservation treatments. Embedded sensors can be used to monitor the key performance indicators for the perpetual pavement, namely the frequency and amplitude of tensile strain at the bottom of the pavement structure as well as the compressive strain at the top of subgrade. This data can then be used to ensure that the in-service performance of the perpetual pavement is meeting the expected performance which was calculated during the design process. If the expected performance is deviating from the planned performance, a rehabilitation technique can be applied in order to obtain the design performance and prevent damage to the deeper, critical portions of the perpetual pavement structure. The effectiveness of the rehabilitation treatment can then be validated using the embedded sensors.

The Transportation Association of Canada (TAC) Pavement Asset Design and Management Guide outlines three types and levels of action for the cost-effective maintenance of pavement infrastructure including: routine maintenance; pavement preservation and rehabilitation [Tighe et.al, 2014]. The timing of these treatments is shown on an illustration of the typical deterioration curve for a pavement in Figure 10. It should be noted that the actual pavement deterioration curve for long-life flexible pavements is expected to be different than the typical deterioration curve provided below, however the general principles still apply. Routine maintenance treatments are usually reactive and are comprised of strategies that are used to address a specific problem such as spray patching of cracks to prevent the ingress of moisture and incompressible material which may accelerate the deterioration of the crack. Pavement preservation techniques occur early on in the service life and consist of well-timed techniques which prevent premature distress and slow the rate of deterioration until the next planned rehabilitation. Rehabilitation consists of techniques that renew or enhance the structural capacity of the pavement as well as improving the functional characteristics such as pavement smoothness and friction.

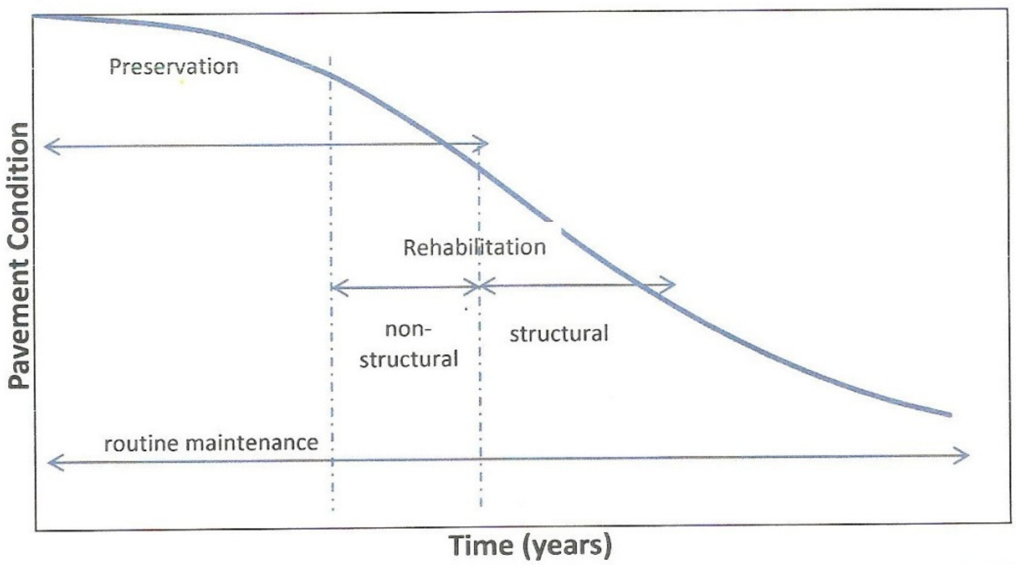


Figure 10 - Typical Pavement Deterioration and Treatment Timing [Tighe et.al, 2014]

2.6. Summary of Key Findings

A review of the key design and performance monitoring parameters for perpetual pavements identified strategies for limiting the tensile strain at the bottom of the asphalt concrete layer as well as the compressive strain at the interface of the granular subbase and subgrade to design levels which will prevent structural rutting and bottom up cracking over the design life and possibly beyond.

One type of technology that can be used to monitor these key performance parameters in perpetual pavements is embedded sensors. Information provided by sensor manufacturers indicates that despite recent advancements in sensor technology, these embedded sensors have relatively short design lives when compared to the design life of a perpetual pavement. As a result, it is common for embedded sensors to be used in accelerated research testing at locations such as the National Centre for Asphalt Technology (NCAT) test track at Auburn University where the design loads are applied in a fraction of the design life. However, there is a gap in the research regarding the long term monitoring of perpetual pavements using this technology.

The scope and objectives of this Thesis were developed in order to address the gap in long term monitoring of perpetual pavements. Considerations include: the best way to complete regular validation of the data which is being received from the sensors; a methodology to calibrate the sensors once they have been installed in the field; and a decision matrix to use when deciding to replace or abandon a sensor. This information was be used to develop a framework for monitoring the long-term performance of perpetual pavements.

3. TEST SECTION DESIGN, CONSTRUCTION AND IN SERVICE MONITORING

This section describes the design of the perpetual flexible pavement test section including the types of embedded sensors used and their location in the pavement structure. The original work to evaluate the performance of perpetual pavements was completed by a consortium that included the Ontario Ministry of Transportation (MTO), Ontario Hot Mix Producers Association (OHMPA), the Centre for Pavement and Transportation Technology (CPATT), Stantec Inc., and McAsphalt Industries Limited with funding consideration provided by the Natural Science and Engineering Research Council of Canada (NSERC). It involved the construction of three flexible pavement structures (one perpetual pavement design with RBM, one perpetual pavement design without RBM, and one conventional flexible pavement) on Highway 401 between Woodstock and Waterloo in southwestern Ontario. In addition to the three test sections constructed on Highway 401, Capital Paving, who were the paving company selected to construct the test sections, agreed to construct and equip a test section at their Guelph Ontario Asphalt Concrete Plant in conjunction with research activities being completed by CPATT.

It should be noted that the design and construction/instrumentation of the Capital Paving Guelph test section was completed and reported by Mohab Y. El-Hakim as part of his Master of Applied Science Thesis [El-Hakim, 2009c], several papers [El-Hakim, Tighe & Galal, 2009a, El-Hakim, Tighe & Galal, 2009b, El-Hakim, Norris & Tighe, 2010, El-Hakim, 2012] as well as his Doctor of Philosophy Thesis [El-Hakim, 2013]. The work that is being completed as part of this Thesis is complementary to the initial work completed by Dr. El-Hakim to develop this site and provides a long term analysis of the performance of the monitoring equipment installed at the site. Additional photographs as well as site installation information has been provided by Dr. El-Hakim for this test section and used to provide a more comprehensive analysis of this site.

3.1. Project Location and Description

The Guelph test section is located at the Capital Paving Inc. asphalt plant yard located on Concession Road 7 in Guelph, Ontario just north of Exit 295 off of Highway 401. The test section was constructed in July of 2009, in conjunction with construction of some of the Highway 401 test sections. The overall test section location is shown in Figure 11.

Figure 11 - Guelph Test Section at Capital Paving Yard [Google Earth, 2014]

Given the length of this site, only one test section could be constructed and instrumented at the Guelph Site. For the purposes of the CPATT research, the perpetual pavement design with rich bottom mix (RBM) was selected as the structure for this site.

The Guelph test site is unique among the test sections that were constructed. As it is located directly adjacent to the asphalt plant weigh station, all of the traffic that passes over the test section is controlled and weighed allowing for a detailed analysis based on vehicle type if preferred. As the weigh scale at the Capital Paving site is a static scale, all of the truck traffic is required to stop to get their loaded weight. After being weighed, the trucks move forward to the ticket room to obtain their weigh ticket. After receiving the weigh ticket, the trucks pick up speed from a standing position and leave the site by passing over the test section pavement.

The truck traffic on this site is therefore similar to an extreme condition where the traffic would be in a congested condition rather than what is typically experienced at the Highway 401 test site where traffic is regularly in the free flow condition. Research using embedded asphalt strain gauges has shown that strain generation can be as much as 2 to 3 times higher than that predicted by layered elastic computer analysis programs when test speeds are lowered to simulate the congested condition and depending on the temperature at the time of testing [Garg & Hayhoe, 2002]. As a result, this site can be used to evaluate the speed/temperature dependence of strain generation in a congested condition. In addition, the slow moving, channelized nature of the traffic on this site makes it safe for researchers to access the site and to test/observe the site conditions making it well suited to educate engineers and other practitioners about the use and benefits of perpetual pavement technologies.

There are some limitations to the setup of sites such as the Guelph Site. One limitation is that the asphalt plant is only in production during the southern Ontario construction season, and as a result, is only subjected to truck loading from April to November of each year. In addition, the trucks that are used to haul the hot-mix asphalt are limited in load carrying capacity and axle configuration and as a result, there is not a wide range of loads/strains that are applied to the test section which can be used in the evaluation. A plan of the Guelph Site Area is provided in Figure 12.

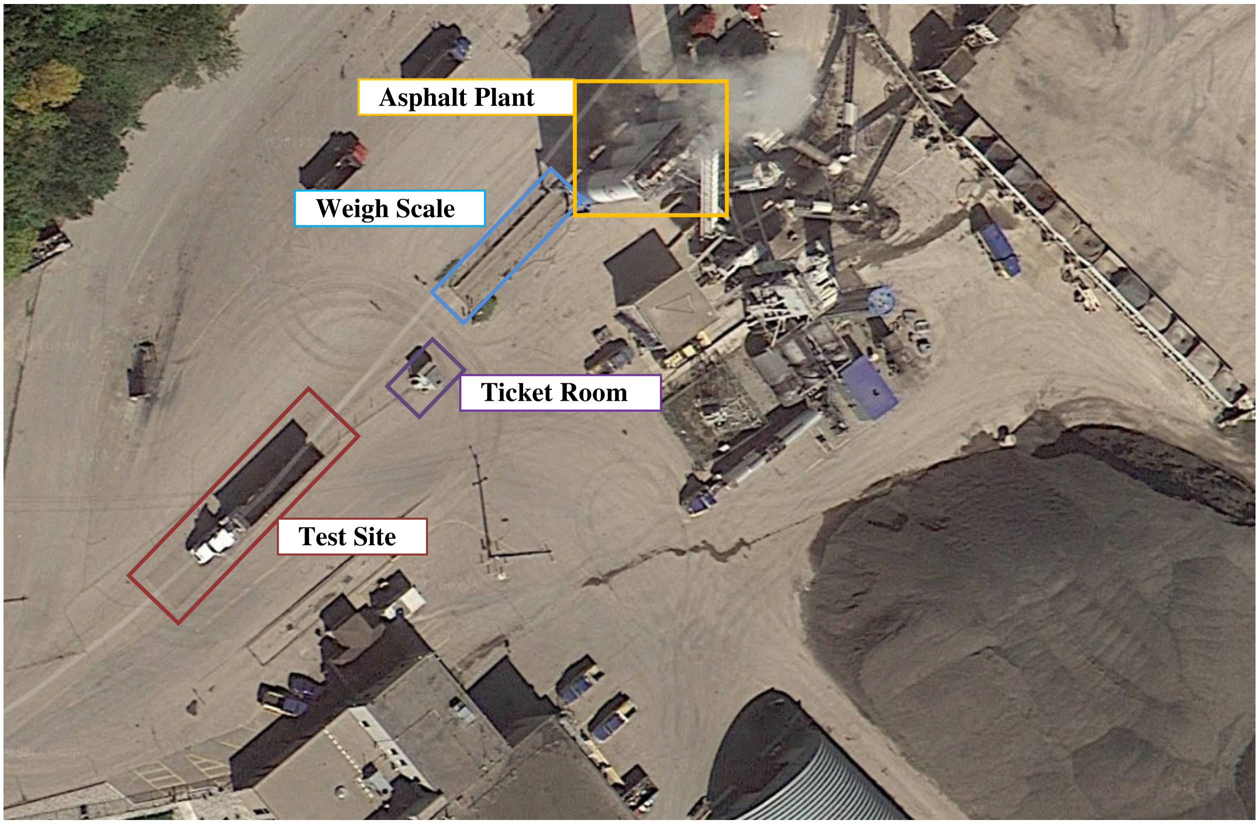


Figure 12 - Plan of the Guelph Test Site Area [Google Earth, 2014]

3.2. Test Section Design and Construction

The original perpetual pavement designs were completed by the MTO with the critical strains evaluated by El-Hakim to validate the perpetual pavement designs [Ponniah, Lane, Marks & Chan, 2009, El-Hakim, 2013]. The designs were evaluated and compared using a number of different analysis programs (MLES, ELSYM5, WESLEA, and MEPDG) using the traffic, weather, and subgrade characteristics from the Woodstock Ontario site. Of all of pavement designs evaluated, the pavement structure incorporating the rich bottom mix (RBM) had the lowest design strains at the bottom of the asphalt concrete structure [El-Hakim, Tighe & Galal, 2009a, El-Hakim, Tighe & Galal, 2009b].

The Guelph test section was constructed using the perpetual pavement design incorporating rich bottom mix (RBM). The RBM design was selected for this site as a RBM contains a slightly higher percentage of asphalt binder which is expected to have superior fatigue endurance compared with traditional mixes based on the design parameters. As the use of RBM results in additional cost, this site allows for further analysis of the cost-effectiveness of using this type of mix in perpetual pavement design. All of the mixes used were designed in accordance with Ontario Provincial Standard Specification (Provincial Oriented) 1151 *Material Specification for Superpave and Stone Mastic Asphalt Mixtures* and constructed in accordance with OPSS.PROV 313 *Construction Specification for Hot Mix Asphalt - End Result*. [OPSS.PROV 313, OPSS.PROV 1151]. The thickness of the each of the design layers is shown in Figure 13.

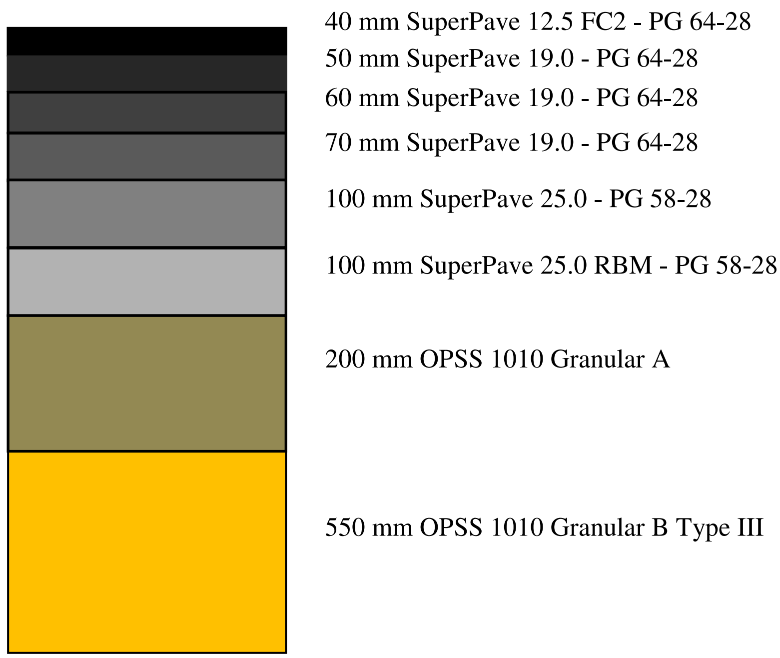


Figure 13 - RBM Perpetual Pavement Design Layer Thicknesses

As shown in Figure 13, the mixes used for the project were all designed in accordance with Superpave design criteria and using performance graded asphalt cement (PGAC) appropriate for the temperatures expected throughout the year, as well as the depth within the pavement structure. The PGAC grade is classified with the first digit referring to the highest temperature that the pavement is expected to encounter during the design life and the lower digit referring to the lowest temperature the pavement is expected to encounter during the design life. The grades typically change in six degree increments, however some specialized asphalt cements are known to change in increments in between these values. The PGAC grade in the upper asphalt concrete layers is often bumped between one and two grades (6 and 12°C) in order to provide additional resistance to rutting during the summer when dealing with either slow moving or heavily loaded traffic. In the OPSS design method, SP stands for Superpave and the numeral after the SP refers to the nominal maximum aggregate size (NMAS) for the mix.

The sensors at the Guelph site were placed to measure the two most critical aspects of the long term performance of a perpetual pavement: vertical compressive strain at the interface with the subgrade that is responsible for structural rutting of the entire pavement; and horizontal tensile strain at the interface with the granular base that is responsible for fatigue cracking.

The vertical strain was measured using an RST earth pressure cell (EPC). After the test section had been excavated to the depth of the pavement structure, the exposed subgrade was first compacted in order to ensure that the interface did not become loosened during excavation. The EPC was then placed at the interface with the surface of the subgrade and covered with moist asphalt sand in order to protect the EPC from damage during overlay with the OPSS Granular B Type III material. The EPC installation procedure at the test site is shown in Figures 14, 15 and 16 with the EPC installation location shown on Figure 20.

Figure 14 - Preparation of the Subgrade for EPC Installation

Figure 15 - Earth Pressure Cell Installation

Figure 16 - Earth Pressure Cell Protection During Subbase Construction

The horizontal strain was measured using six asphalt strain gauges (ASG) installed in pairs at three different locations within the test section. At each location, one ASG was installed at the top of the Granular A material (within the RBM mix) while a second ASG was installed in the same location but at the interface between the RBM and the overlying Superpave SP 25.0 (without RBM) mixture. The ASGs were offset 1.0 m from the edge of pavement so that they were installed within the driver's wheel path of the loaded trucks. The wiring for the strain gauges was placed in 50 mm inside diameter PVC tubes which were installed within a trench leading to a data logger box installed adjacent to the ticket room which provides continuous power to the data logger. The ASGs were oriented to measure strain parallel to the direction of travel. The ASG installation procedure at the test site is shown in Figures 17, 18 and 19 with the ASG and EPC installation locations shown on Figures 20 and 21.

Figure 17 - ASG Installation within Trench Leading to Ticket Room

Figure 18 - ASG Installation at Bottom of RBM

Figure 19 - ASG Installation at Between RBM and SP 25 Layers

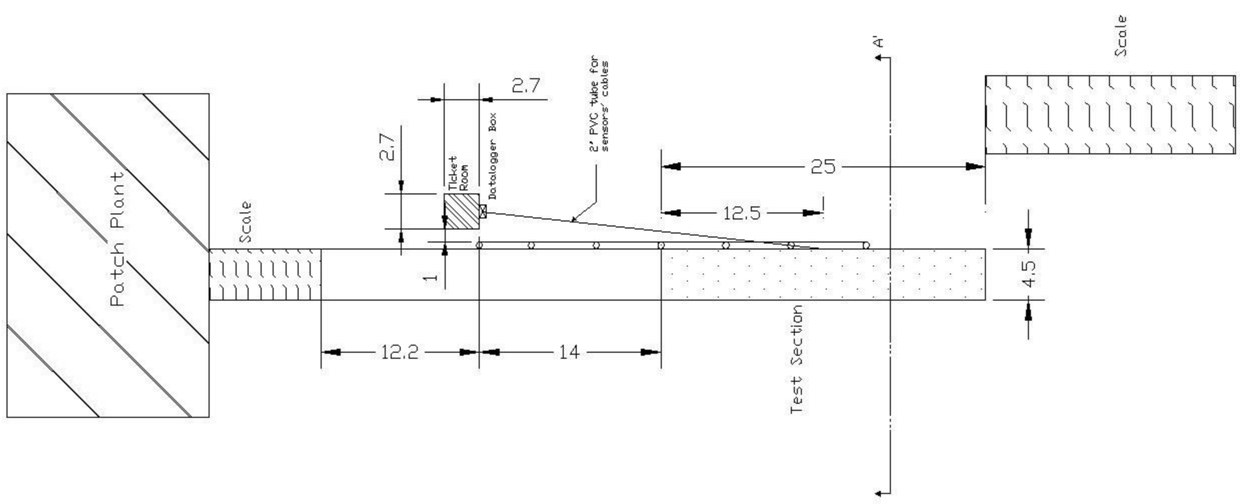


Figure 20 - ASG and EPC Installation Locations (all units in metres)

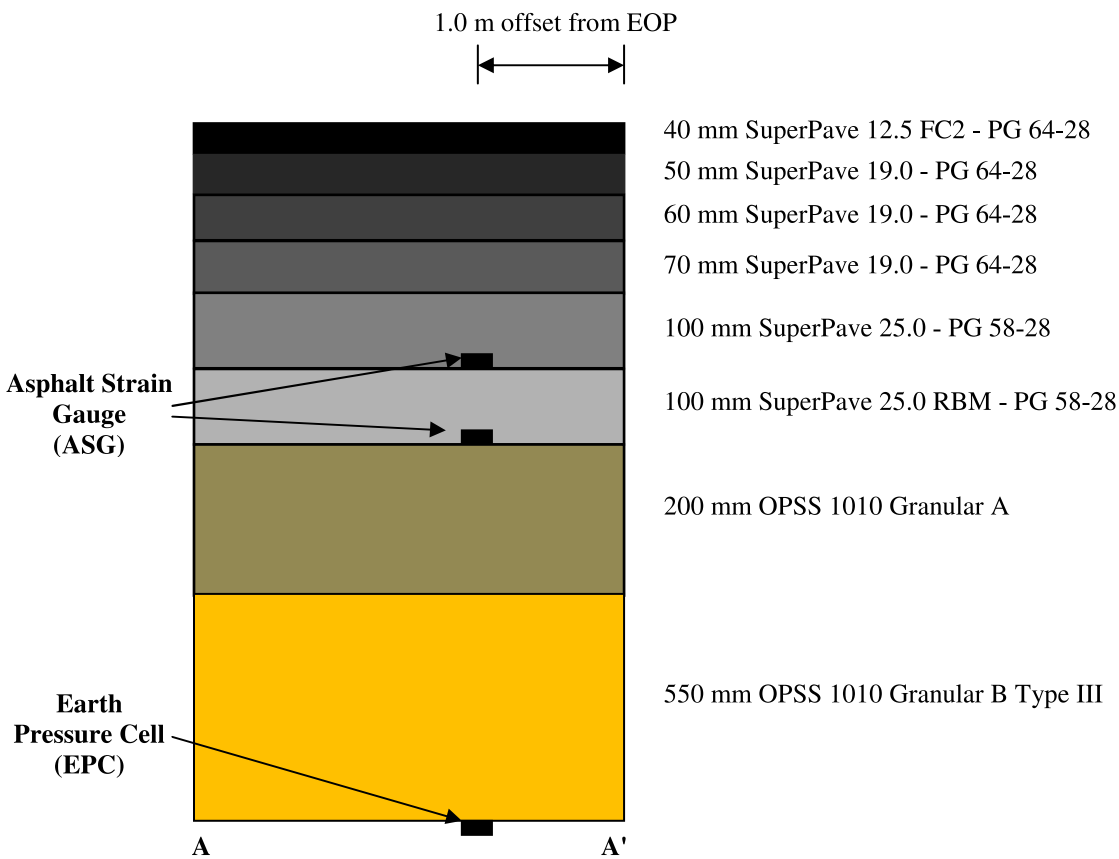


Figure 21 - Sensor Installation Location (Cross Section A-A')

3.3. Instrumentation Selection

The embedded sensors and data logger used for the instrumentation at the Guelph test site were selected based on an extensive literature review of similar projects completed in Minnesota, Virginia and at the National Centre for Asphalt Technology (NCAT) [El-Hakim, 2009c].

Asphalt strain gauges manufactured by Construction Technologies Laboratories (CTL) were selected based on their successful use at many test sites, including several test sections at NCAT. The specific model chosen for use at the Guelph test site was the ASG-152 vibrating wire asphalt strain gauge. The technical specifications of this device are provided in Figure 22.

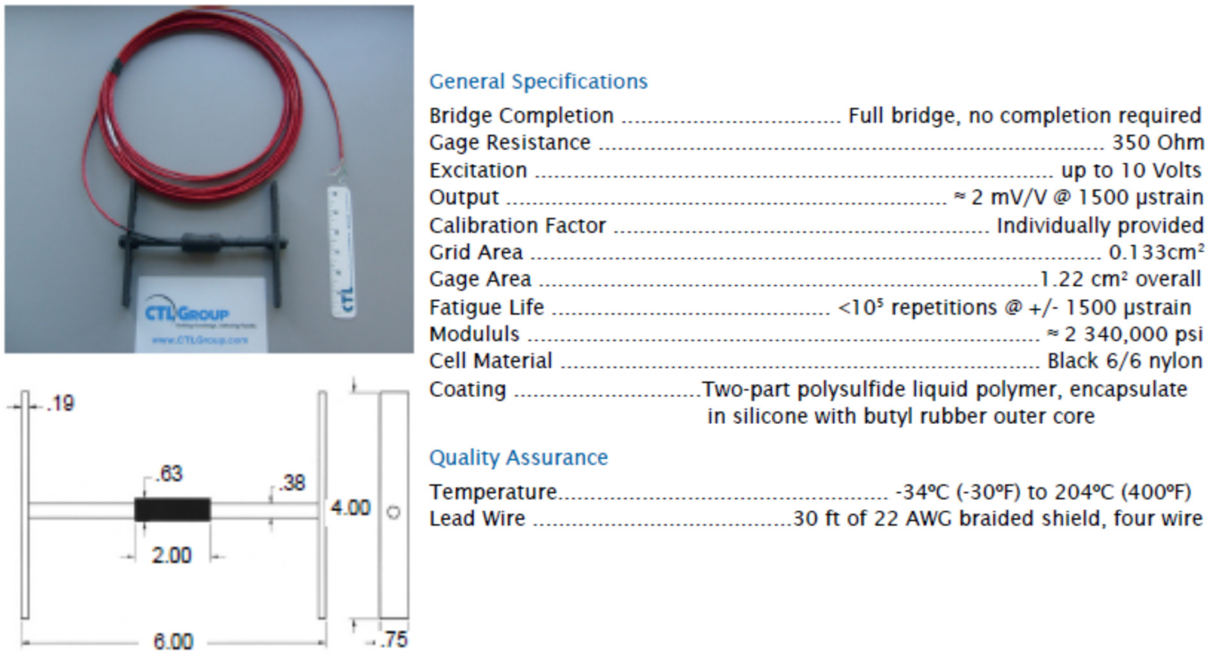
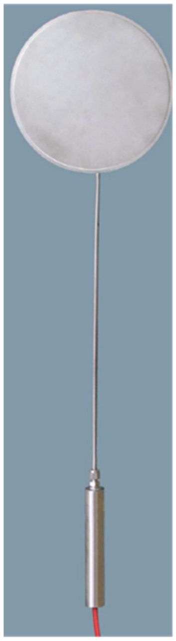


Figure 22 - CTL ASG-152 Product Specifications [CTL, 2008]

The earth pressure cell selected for the test site was a vibrating wire type EPC manufactured by RST Instruments. The specific model selected for the test site was to LPTPC-V total earth pressure cell. The technical specifications of this device are provided in Figure 23.

The data logger used for the test site was the Campbell Scientific CR-1000. The CR-1000 is capable of scanning up to 8 channels (for a double wired installation) at a frequency of 1 Hz which is ideal for the six ASGs and one EPC installed at the test site. A one gigabyte (1 GB) flash memory card adapter was also installed in order to store the collected data until it could be manually downloaded. The data logger box was located directly adjacent to the ticket room at the test site, and as a result, the data logger and sensors could be powered directly from a full time power source. Regardless, a backup battery was initiated order to ensure that test site will be monitored in the event of a power outage. The data logger and field setup are shown in Figure 24.



DESCRIPTION	LPTPC-V
Transducer Type	Vibrating Wire
Range - Standard Calibration	Up to 2.0 MPa (300 psi)
Range - Max Available	20 MPa (3000 psi)
Special Calibration	Up to 3.4 MPa 500 (psi)
Calibrated Accuracy	.15% F.S.
Resolution	0.025% F.S. minimum
Excitation Voltage	5 V sq. Wave
Signal Output	1200 - 2000 Hz
Conductor	2 X #22
Operating Temperature	-29° to +65°C -20° to +150°F

Figure 23 - RST LPTPC-V Product Specifications [RST, 2014]

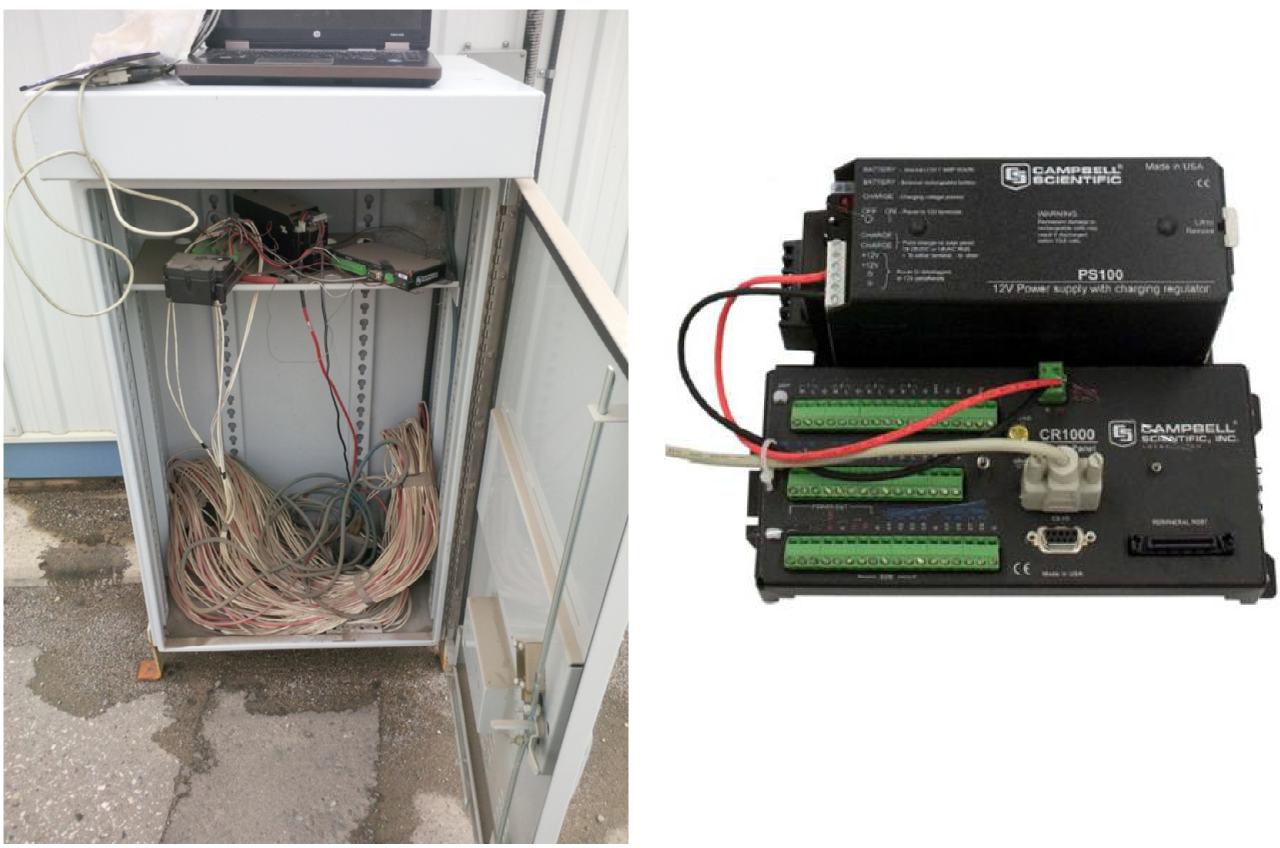


Figure 24 - CR-1000 Data Logger and Field Setup

3.4. FWD Testing of Perpetual Pavements

Research and testing of the various pavement sections at the NCAT test track was completed in 2009 in order to evaluate sources of embedded strain gauge variability. The research evaluated different methods of loading the pavements in order to evaluate the repeatability of results in the embedded asphalt strain gauges (ASGs). One of the outcomes of the research was that the FWD was a good tool for the evaluation of embedded ASG performance as it was able to drop a consistent load, concentrically onto the pavement directly above the gauge. Reliable strain readings were generated from the FWD impact loads and the FWD loading removed material differences and wheel wander variability allowing the evaluator to focus on the precision of the instrument itself [Timm, 2009]. Using the experience gained in the NCAT research, this methodology was applied to the perpetual pavement section at the Guelph Site to evaluate the performance of not only the ASGs but also the earth pressure cell (EPC). The FWD testing procedure is outlined in Figure 25 with each of the testing steps described in more detail below.

The equipment used to complete the FWD testing on the Guelph Site was a Dynatest 8081 ® trailer mounted high capacity falling weight deflectometer (HWD) provided for the investigation by LVM, Division of EnGlobe Corp, of Toronto, Ontario. This device is extremely versatile, capable of applying a wide range of loads from 7 kN up to 240 kN which allows for simulation of large loads such as those applied by the landing gear of an Boeing 747 aircraft. The Dynatest HWD produces a transient, impulse type load of 20-30 ms in duration approximating the effect of a moving wheel load at 60 km/h.

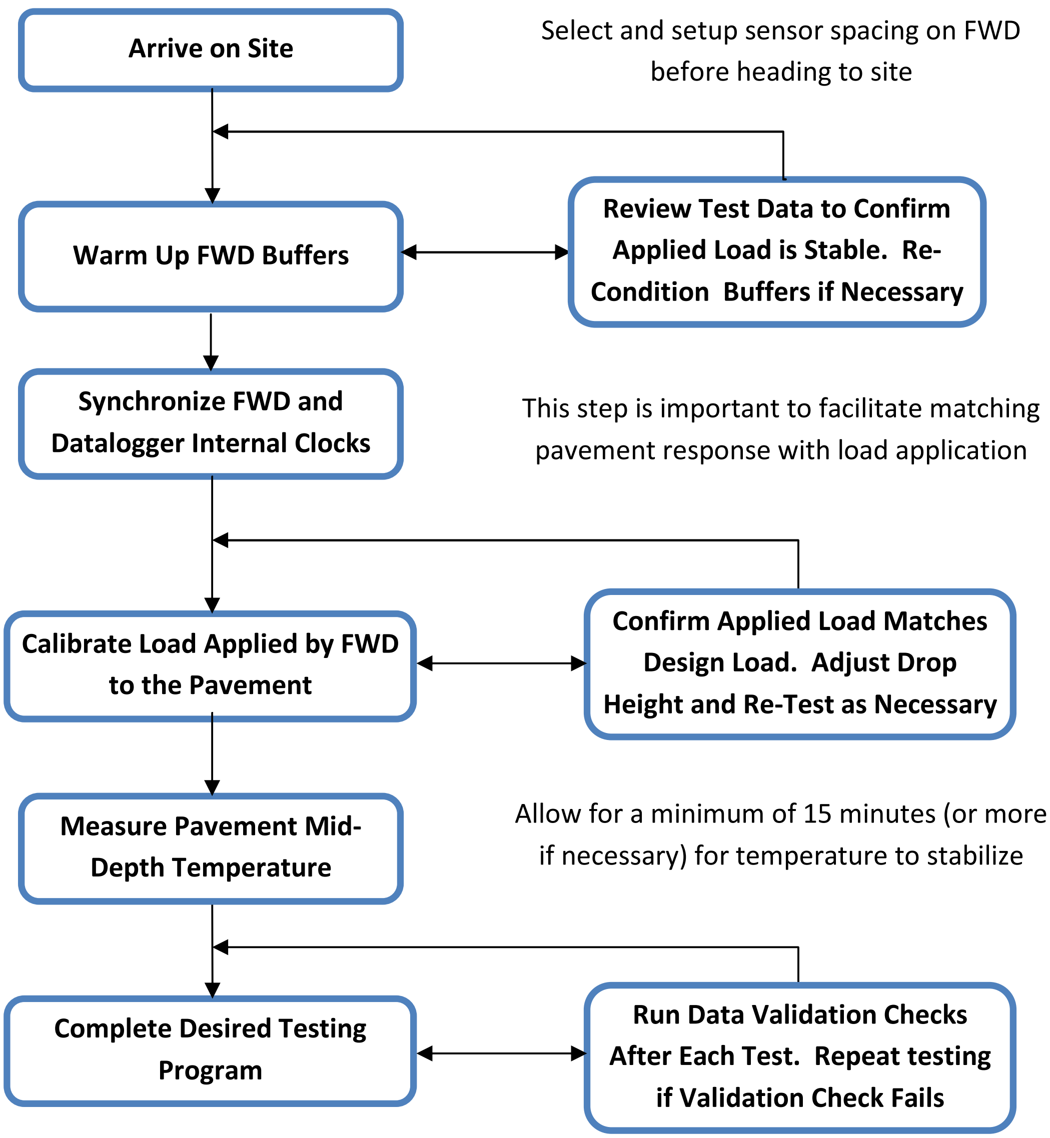


Figure 25 - FWD Testing Procedure for Test Site

After arriving at the test site and before the testing started, a FWD warmup and site initiation procedure was completed in order to warm up the buffers to ensure a repeatable applied load and to calibrate the drop height to the local conditions in order to obtain the planned target loads. The rubber buffers that are used to dampen the applied load from each drop height are sensitive to temperature changes which results in a reduction in the applied load as the temperature of the buffers increases. The buffers are usually cold after travelling to the site and as a result it is good practice to complete a number of warmup drops to condition and warm the buffers prior to the start of testing. In order to prevent the applied load from varying (becoming lower) throughout the testing program and to ensure that the buffers were properly conditioned, 40 warmup drops were used with the resulting load analyzed to ensure that it was not varying outside the normal operating boundaries of +/- (0.18 kN + 0.02*Load).

In order to compare the load and deflection profiles generated during FWD testing to the measured strain, it was important that the clocks of both the FWD and the data logger were synchronized. As a result, while the FWD buffers were being conditioned, the clock on the data logger was updated to match the exact time being used by the FWD equipment.

The impulse load applied by the FWD from a certain drop height will vary from pavement to pavement depending on the overall pavement stiffness. For the perpetual pavement test section, the overall pavement is designed to be stiff and as a result the drop height will have to be increased when compared to a conventional pavement design in order to obtain the desired load and deflection. After the buffers were conditioned, the drop heights were calibrated for the local pavement stiffness by completing trial runs at the planned drop sequences and comparing the measured values against the desired values. The proximity sensors were then adjusted either upwards or downwards and the trial run repeated until the desired load was achieved for each drop sequence.

While the FWD contains automatic sensors to measure and record the air temperature and the pavement surface temperature, the pavement mid-depth temperature must still be taken manually in order to measure the thermal gradient through the bound pavement layers. The manual measurement is taken by drilling a 13 mm diameter hole to the mid-depth of the pavement structure (in this case 210 mm) using a portable hammer drill and tungsten carbide hammering bit. The hole is then cleared using compressed air and filled with 25 mm of standard mineral oil. The hole is then covered and the stabilized temperature of the mineral oil is measured using a calibrated digital temperature probe after a minimum of 15 minutes have passed. The temperature that is measured is considered representative of the mid-depth temperature and this value is input into the FWD testing software and recorded in the testing database for each test set.

Due to the viscoelastic nature of asphalt concrete mixtures, the temperature has a great effect on the resilient modulus which can affect (sometimes greatly) the amount of deflection which is measured during testing. As a result, an accurate measurement of this temperature is critical in order to be able to adjust (nomalize) the measured deflections or backcalculated resilient modulus to a standardized temperature when testing in different seasons. This is important for the testing completed as part of this Thesis as the testing was completed in the spring, summer and fall at air temperatures of 14, 26, and 7°C respectively.

During testing, the data quality was checked using automated validation tools and the test was repeated if any of the quality checks failed. The five quality checks included: roll-off; nondecreasing deflections; overflow; load variation; and deflection variation. A roll-off error occurs when a deflection sensor does not return to zero after the test is complete. It is generally accepted that deflections will decrease as the distance increases from the load plate and as a result, if this condition is not met during testing, the drop results are rejected. An overflow error occurs when the measured deflection exceeds the tolerance of the sensor. Load and deflection variation checks are used to confirm that the load and deflection levels do not exceed a 2 and 1 percent variation respectively between test results, or then those results are rejected.

The selection of load levels are an important consideration for FWD testing. The loads that are selected must be representative of the type of loading that is experienced on a roadway section. However, load selection must take into account the stiffness of the pavement structure that is being measured in order to ensure that a good deflection basin is measured (for stiff pavements a higher load is sometimes required to get reliable deflection measurements from the outer sensors) and the measured deflections are within the geophone range (for soft pavements, the measured deflections can sometimes exceed the 2 mm range of the geophones and the load should be reduced in order to get deflections within range). The 40 kN load is a standard load level that is used because it simulates a typical 8000 kg design load. The 60 kN load was used at the test section as this load level simulates a typical fully loaded dump truck which traverses the test section. The 90 kN load level was selected to determine how the test section responded to an extreme scenario of an overloaded vehicle.

The selection of sensor spacing is important for the overall evaluation of the pavement. The sensors are used to determine the shape of the deflection basin generated from the applied load. The sensors must be placed at radial offsets from the center of the load plate which will provide the best definition of the deflection basin in order to achieve proper modeling of the pavement structure and/or backcalculation of layer parameters. While in theory any sensor locations can be used in deflection testing and the resultant deflection basin can be used in backcalculation analysis, it has been reported that sensor spacing that are either too close together or too far apart may result in significant variations in the back calculation results [Li and White, 2000].

The deflection response of the pavement is determined by the shape of the deflection basin and the distance of the sensor to the applied load. This phenomenon is shown for a 9 sensor FWD configuration in Figure 26. This figure shows that the deflection response of sensor 1 (at the load plate) is influenced by a composite response of the surface, base, subbase and subgrade. However, sensors 7, 8 and 9 are only influenced by the response of the subgrade. If the response of the subgrade can be isolated by placing the outer sensors far enough away from the applied load, then the subgrade response can be isolated from the response of the upper layers. If the sensors have been properly placed, then this process can be repeated sequentially through the overlying pavement structural layers until the response of each individual layer has been calculated.

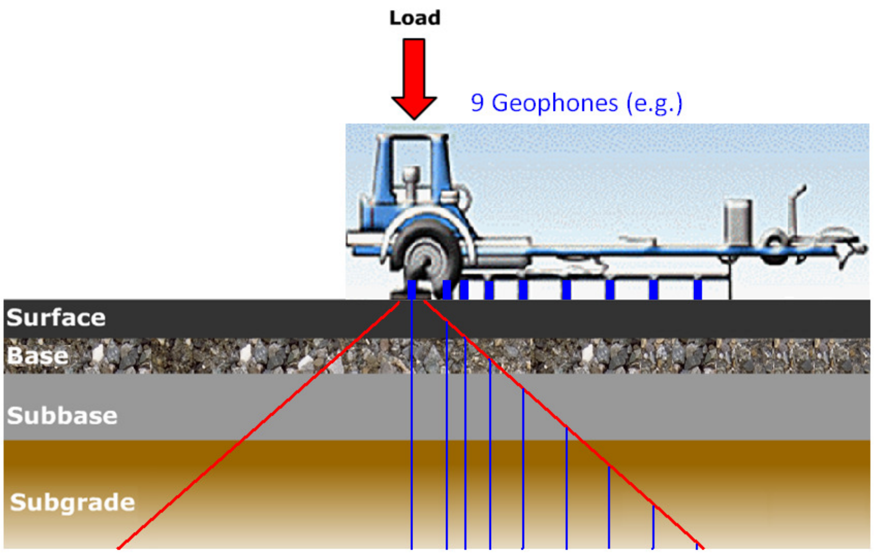


Figure 26 - Effect of FWD Sensor Spacing with Depth [Dynatest, 2014]

The Dynatest 8081 FWD used for this research is equipped with nine sensors that can be adjusted to variable distances from the load plate according to the requirements of the test section. As the focus of the research was the performance of the embedded sensors, an additional sensor was moved closer to the load plate in order to provide a better definition of the deflection basin in the vicinity of this layer. A graphical representation of the sensor spacing is provided in Figure 27.

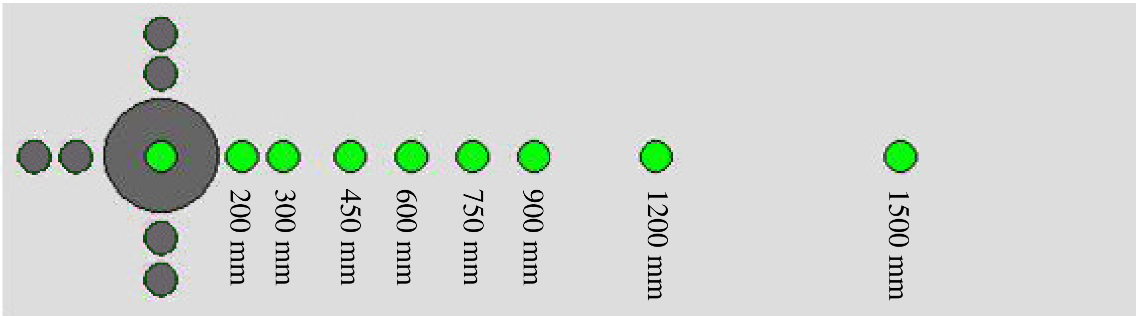


Figure 27 - Sensor Spacing Used in Research (sensor in grey are not used)

3.5. Summary of Findings from Test Site Construction and Instrumentation

Strain gauges were installed in pairs at the site at three separate locations in the drivers' wheel path. At each strain gauge location, one gauge was installed at the interface between the granular base and the bottom asphalt concrete layer (SP 25 RBM) and the other was installed shallower in the pavement structure between the SP 25 RBM and the overlying SP 25. The earth pressure cell was installed in the drivers wheel path at the interface between the subgrade and the granular subbase.

The main issue at the site is that during construction of the test site, it was not anticipated that the location of the embedded sensors would be required in the future and as a result, no permanent location markers were placed in the field. Therefore, the precise longitudinal and transverse offsets of the ASGs could only be estimated in the field based on the location information provided during the initial field trial as well as the photographs that were taken to document the installation [El-Hakim, 2009c]. As a result, a separate testing program was required to first identify the location and depth of all of the embedded sensors as well as the specific channels that were used in the data logger to record the strains generated from the individual ASGs.

4. EMBEDDED SENSOR LOCATING USING FWD

As mentioned previously, no permanent location markers were placed in the field to mark the location of the embedded sensors during construction. Research completed by NCAT suggests that the strain measured by the embedded asphalt strain gauges and earth pressure cells is expected to vary if the load is not applied directly above the embedded sensors [Timm, 2009]. Therefore a testing program was first required to determine the precise longitudinal and transverse offsets of the ASGs and EPC. In addition to issues with the location, the individual strain gauges were not labelled when connected to the data logger, and based on a review of the data that had accumulated in the data logger from the loading that was being applied by the hot-mix asphalt hauling trucks travelling on the testing site, it did not appear that the asphalt strain gauges were installed sequentially and it was not possible to determine how the strain gauges were paired vertically within the pavement structure. As a result, the testing program also needed to be able to identify the depth of all of the embedded sensors as well as the specific channels that were used in the data logger to record the strains generated from the individual ASGs.

The site was initially tested at 30 cm intervals in the vicinity of where the EPC was expected to have been installed. The zero station is located at the west end of the test section and increases as the testing heads east. At each FWD test location, three drops were completed at standard drop loads of 30, 40 and 50 kN The same FWD drops were completed at offsets of 0.80, 1.0 and 1.25 m north of the south edge of test section. The testing layout is shown in Figure 28.

Figure 28 – Plan of the Guelph Test Section [Google Earth, 2014]

4.1. Determining the Location of the Earth Pressure Cell

The response of the EPC was reviewed after each set of drops. The peak pressure response from all three runs (offsets of 0.8, 1.0 and 1.25 m) were then plotted and analyzed to determine the location of the EPC in the test section. The response of the EPC to the peak FWD drop at each of the testing locations was plotted as shown in Figure 29.

The results of the testing showed that for all three of the tested offsets, the peak pressure response of the EPC was measured near Station 8.9 m. The highest pressure response of the EPC was measured during the testing completed along the 0.8 m offset. The testing shows a clear trend of decreasing pressure as the offset is increased (i.e. 1.25 m offset has significantly lower results) and as a result, it was concluded that the EPC was most likely installed at the 0.8 m offset location. It should be noted that the EPC was expected to be found at a 1.0 m offset based on the notes provided during the initial installation. It is possible that the EPC may have been moved slightly during construction (construction activities may have pulled on the data cable of the EPC causing it to shift slightly, for instance) which may have resulted in this slightly different location. Regardless, the Station and Offset where the peak pressure was measured by the EPC were marked out in the field as the embedded sensor location.

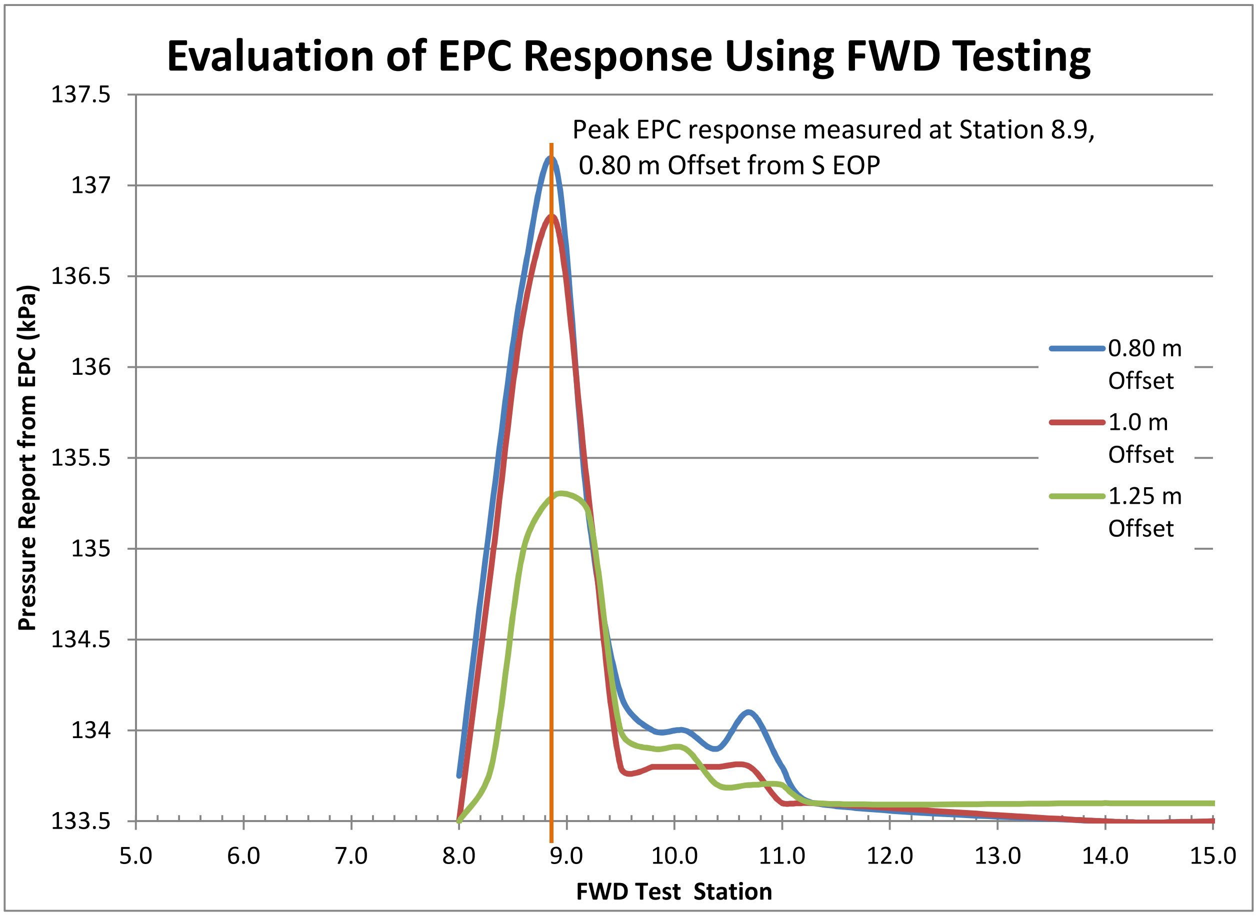


Figure 29 - Evaluation of EPC Response at Different Stations and Offsets Using FWD

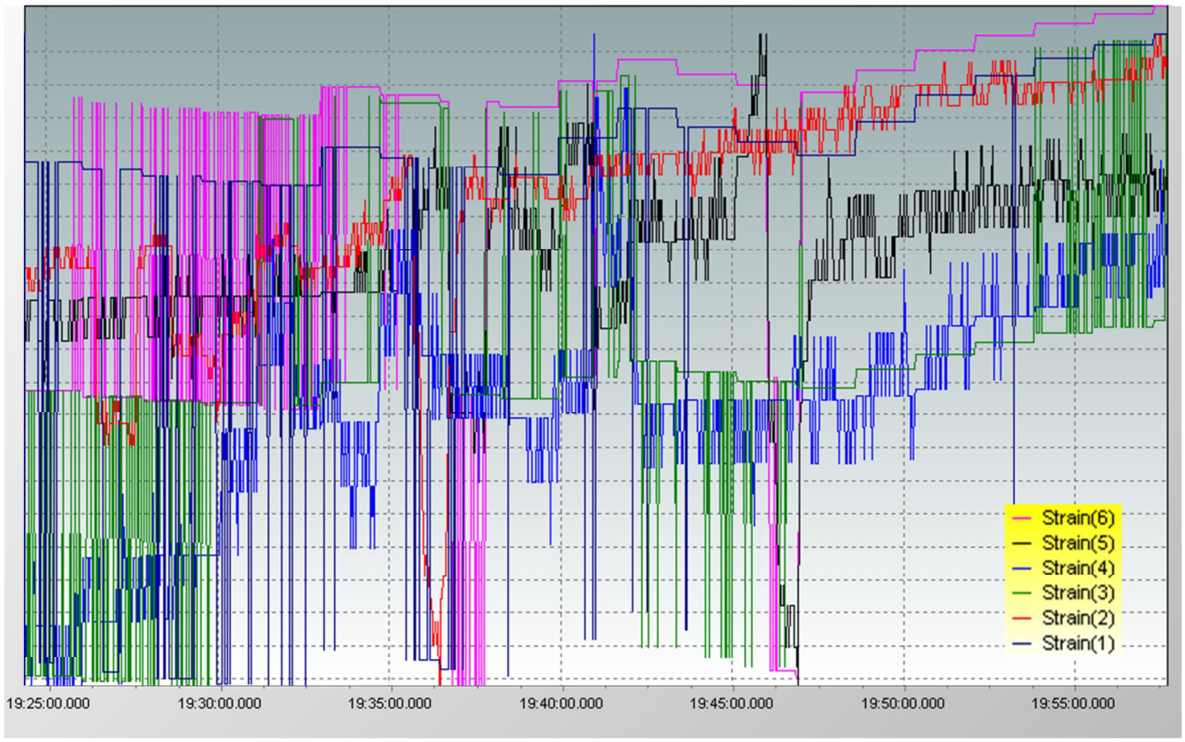
4.2. Determining the Location of the Asphalt Strain Gauges

In order to determine the location, relative depth and sensor position of each of the six embedded ASGs, a similar testing program was completed in the vicinity of where the ASGs were expected to have been installed, with testing completed at 1 m intervals. In order to determine if the ASGs were installed at the same offset as the EPC, the FWD testing was completed at the same three offsets as the EPC testing (i.e. 0.8, 1.0 and 1.25 m north of the south edge of pavement as shown in Figure 28). For the strain gauge testing at each FWD test location, four drops were completed at drop loads of 40, 60, 90 and 90 kN. The response of the ASGs was reviewed after each set of drops. A detailed review of the ASG responses did not show any discernable differences in the response of the ASGs when testing along the three offsets and as a result, the response generated at the 0.80 m offset was used for the data analysis assuming that the ASGs were installed at the same transverse offset as the EPC. This result was somewhat surprising given that NCAT had reported that the strain gauge result was sensitive to wheel wander. However, the FWD applies a circular load on a 300 mm diameter plate which is much wider than that of a typical tire which may explain why the ASGs were not sensitive to the slight changes in offset.

After the testing pattern was completed, the data was downloaded from the data logger and analyzed in order to determine the response of the individual ASGs at each 1 m interval. The first analysis consisted of determining which of the ASGs were paired together at the different stations in the test section. The ASG response to the testing completed at each 1 m interval was analyzed using the Campbell Scientific View Pro (Version 4.2) software which has expanded capabilities for reviewing, plotting, and comparing data collected using data loggers.

The test section construction notes which were reported for the initial construction indicated that the ASGs were installed in pairs at two different depths within the asphalt concrete. The uppermost sensor was installed between the interface of the SP 25 binder course and the underlying SP 25 (RBM) while the lower sensor was installed at the interface between the SP 25 (RBM) and the underlying Granular A base. However, while six strain gauges were installed, the location of the individual gauges was not documented and was not obvious based on a review of the initial test data. An example of the initial response during the initial testing phase is provided in Figure 30.

Relative Strain Change



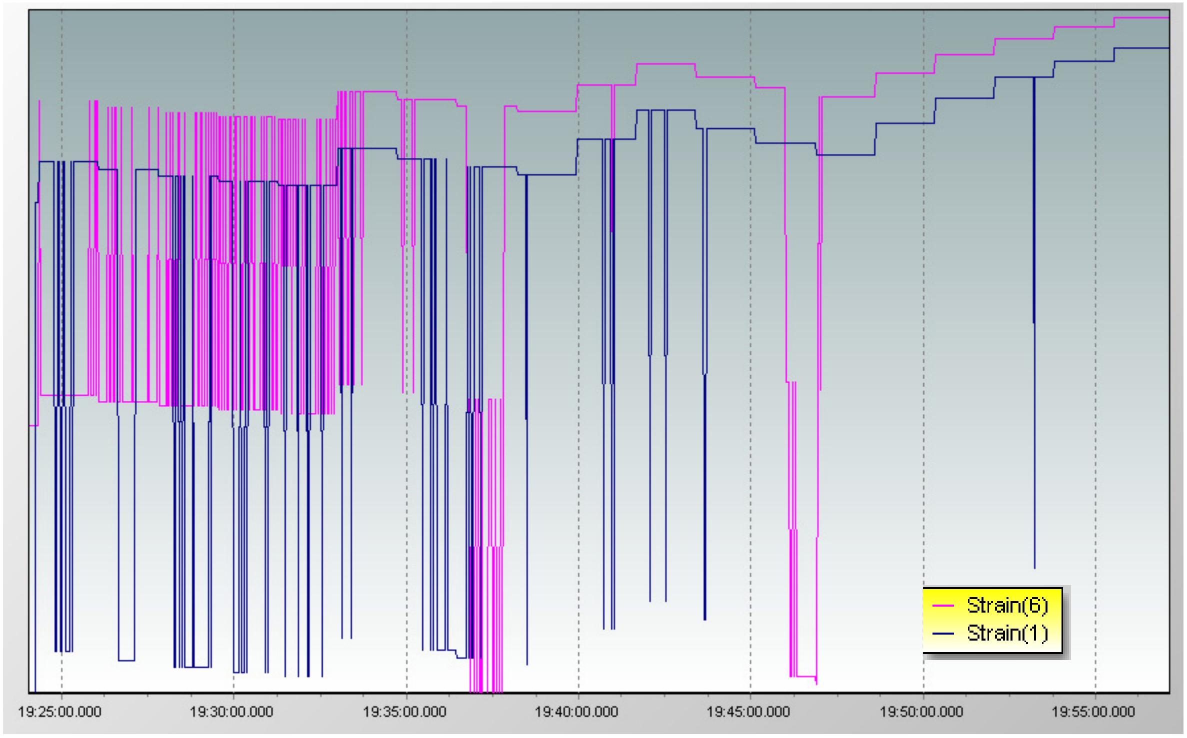
Testing Time (hh:mm:ss)

Figure 30 - Response of All Six Strain Gauges to the Initial FWD Testing

4.2.1. Determining the ASG Pairs Installed in the Test Section

In order to aid with the future detailed analysis which was planned for the site, a thorough analysis of the ASG response was completed to locate the sensors as accurately as possible. The analysis consisted of taking one pair of sensors at a time and comparing the measured strain values from each sensor pair using the same time and strain scale. After comparing all of the possible ASG pair combinations, clear trends emerged where the strain response from the sensor pairs matched but had slightly different amplitudes which would be expected given that the ASG pairs were installed at the same location but at different depths. The View Pro plots with the final sensor pairings are shown in Figures 31 to 33.

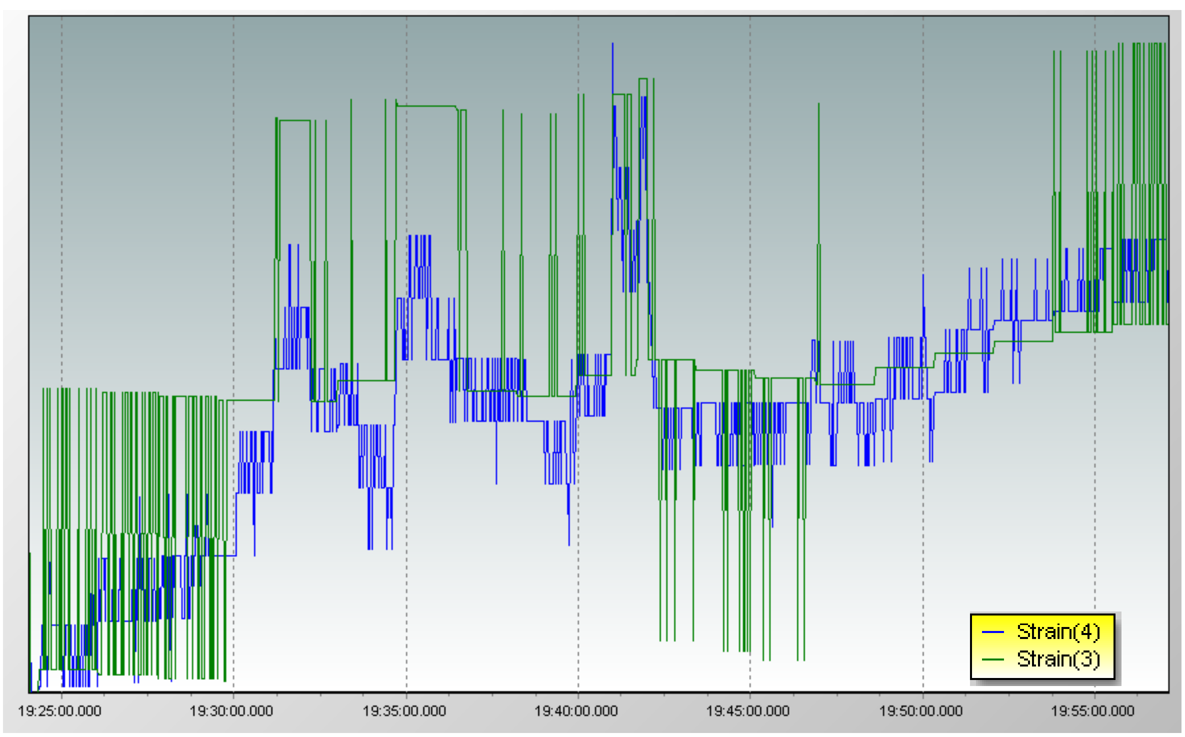
Relative Strain Change



Testing Time (hh:mm:ss)

Figure 31 - Comparison of Response of ASG 1 and 6 to FWD Loading

Relative Strain Change



Testing Duration Time

Figure 32 - Comparison of Response of ASG 3 and 4 to FWD Loading

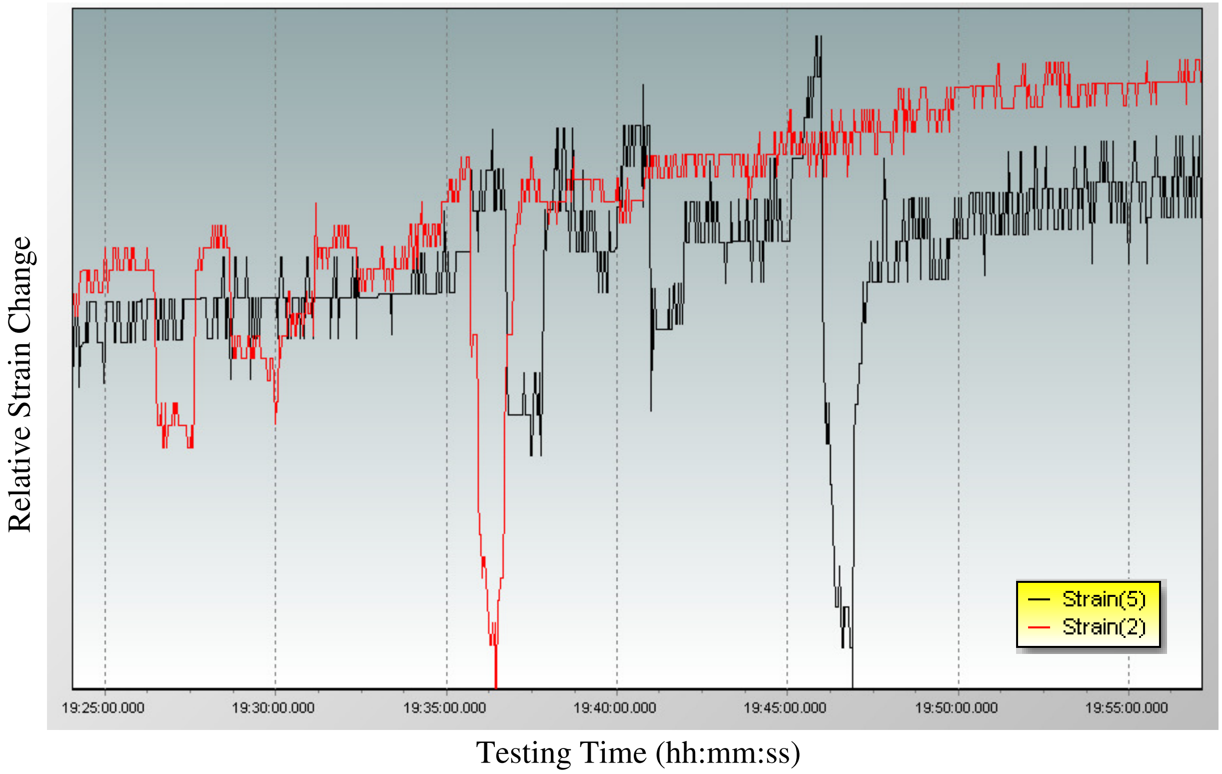


Figure 33 - Comparison of Response of ASG 2 and 5 to FWD Loading

Figure 31 shows that the response of ASG 1 and ASG 6 match very well but at a slightly different amplitude, with the response of the gauges very similar based on the applied FWD loading. The magnitude of the strain changes is larger in ASG 1 than in ASG 6. These strain profiles do not show a lot of signal noise in either of the gauges which is an indication that the gauges are working well.

In Figure 32, the response of ASG 3 and ASG 4 is similar. There is a noticeable amount of signal noise is ASG 4 as compared to ASG 3 and this could be an indication of a problem with this gauge or that this gauge is nearing the end of its service life. The magnitude of the strain change is larger in ASG 3 than in ASG 4.

In Figure 33, the response of ASG 2 and ASG 5 is similar however it would be very difficult to conclude that they were paired based on the response observed in the Figure due to the significant signal noise and out of sync strain responses except for the fact that they were the only two strain gauges left after the first two pairings. While not obvious from this Figure, the magnitude of the strain change in ASG 2 is much larger than ASG 5 but this effect was muted due to the scaling required to have both profiles on a similarly scaled vertical axis.

The response of Asphalt Strain Gauges 2, 4 and 5 show varying amounts of signal noise which is an indication of either a potential performance issue with the gauge or that the gauge is nearing or beyond its service life. As a result, the response of these gauges to the detailed FWD testing plan should be analyzed further to ensure that these gauges are providing accurate and repeatable results before using them for the analysis. In addition to the signal noise, the values of the strain kept increasing during the FWD testing. This increase in strain was also noted during the Marquette Interchange research project and was attributed to the temporary accumulation of strain in the pavement structure due to the rapid succession of tests being completed [Hornyak, Crovetti, Newman & Schabeiski, 2007].

4.2.2. Determining the ASG Station Locations

The second analysis consisted of determining the station where the ASG pairs were installed. The station was determined by first calculating the absolute measured difference between the maximum and minimum strain readings (peak strain) measured in the individual ASGs during the FWD testing at each 1 m increment. This peak strain was then plotted at each testing station and the highest measured peak strain was used to determine the station where the sensor was installed. As the difference in strain recorded in the datalogger was relatively small for the FWD testing loads and frequency used during the sensor locating process, the measured strain was calculated as a percentage of the increase observed from the testing in order to accentuate the peak strain graphically. The analysis confirms the assumption of strain gauge pairings which was completed previously and shows that the first ASG pair was installed at 9 m, ASG pair two at 15.1 m and ASG pair three at 21 m east of the west end of the test section. The analysis is shown graphically with the assumed ASG station selected based on peak strain increase highlighted using a circle in Figure 34. It should be noted that in order to show how the peak strains were located, the strain gauge results were plotted on two separate vertical axes with ASG 1, 3 and 6 plotted on the primary axis (1-14) and ASG 2, 4, 5 plotted on the secondary axis (0-25).

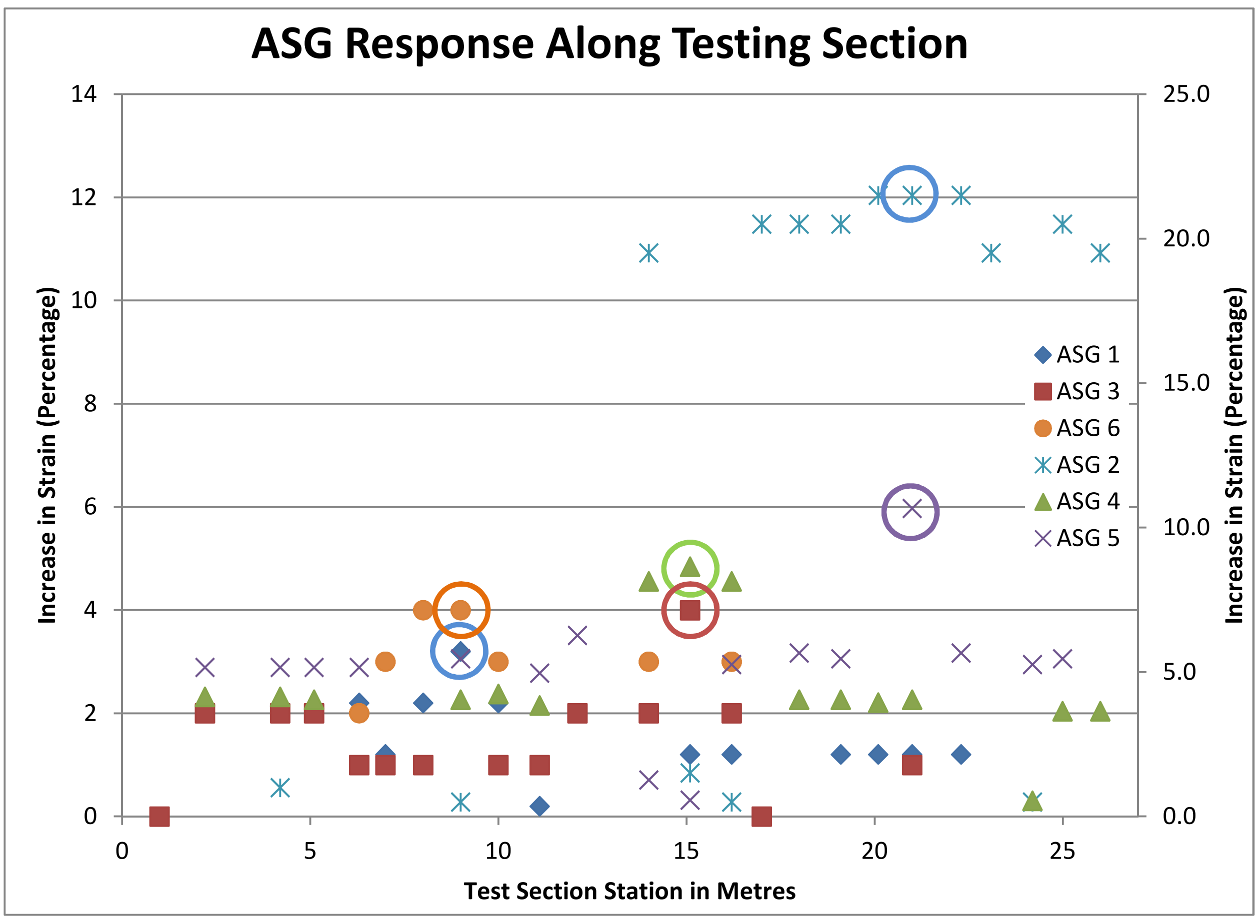


Figure 34 - Evaluation of ASG Response at Different Stations Using FWD

4.2.3. Determining the Depth Where the ASG was Installed

The third and final analysis consisted of determining the depth where the individual ASGs were installed in the pavement structure. Using the calculated ASG stations from the previous analysis, the absolute measured difference between the maximum and minimum strain readings (peak strain) was plotted for each ASG which typically corresponded to one of the 90 kN FWD drops. As shown in Figure 2, the asphalt concrete is expected to be in compression for the first portion of its thickness and then transition into tension with increasing depth. As a result, in each pair, the ASG with the lower measured strain was assumed to be at a shallower depth within the pavement structure. Based on the measured strain, ASG 1 and 3 had the lower measured strains and have been installed at the interface of the SP 25 and underlying SP 25 RBM. The corresponding ASGs 6 and 4 have higher measured strains and have been installed at the interface of the SP 25 RBM and the underlying granular base material.

For ASG pair 2 and 5, the highest peak measured strain was found in ASG 2. The reading however is quite high and is considered out of range for the loads which were applied to the pavement and therefore it is not possible to conclude its depth in the pavement structure. The strain reading in ASG 5 was the lowest of the pair of gauges tested at this station. However it is noted that the recorded strain is more typical of the readings obtained from the other ASGs installed at the interface of the SP 25 RBM and the underlying granular base material. Given that the strain readings from ASG 2 are not considered to be reliable, and that it is possible that ASG 2 may have been installed at the interface of the SP 25 and underlying SP 25 RBM, it is expected that ASG 5 has actually been installed at the bottom of the asphalt concrete and the recorded readings are indicative of the strain measured at the interface of the SP 25 RBM and the underlying granular base material. Previous results have shown that both of these gauges have significant signal noise and it is not believed that the results that have been obtained can be considered to be reliable for these gauges. The peak strain (in microstrain) measured at each ASG location has been plotted against the test section station and is provided in Figure 35.

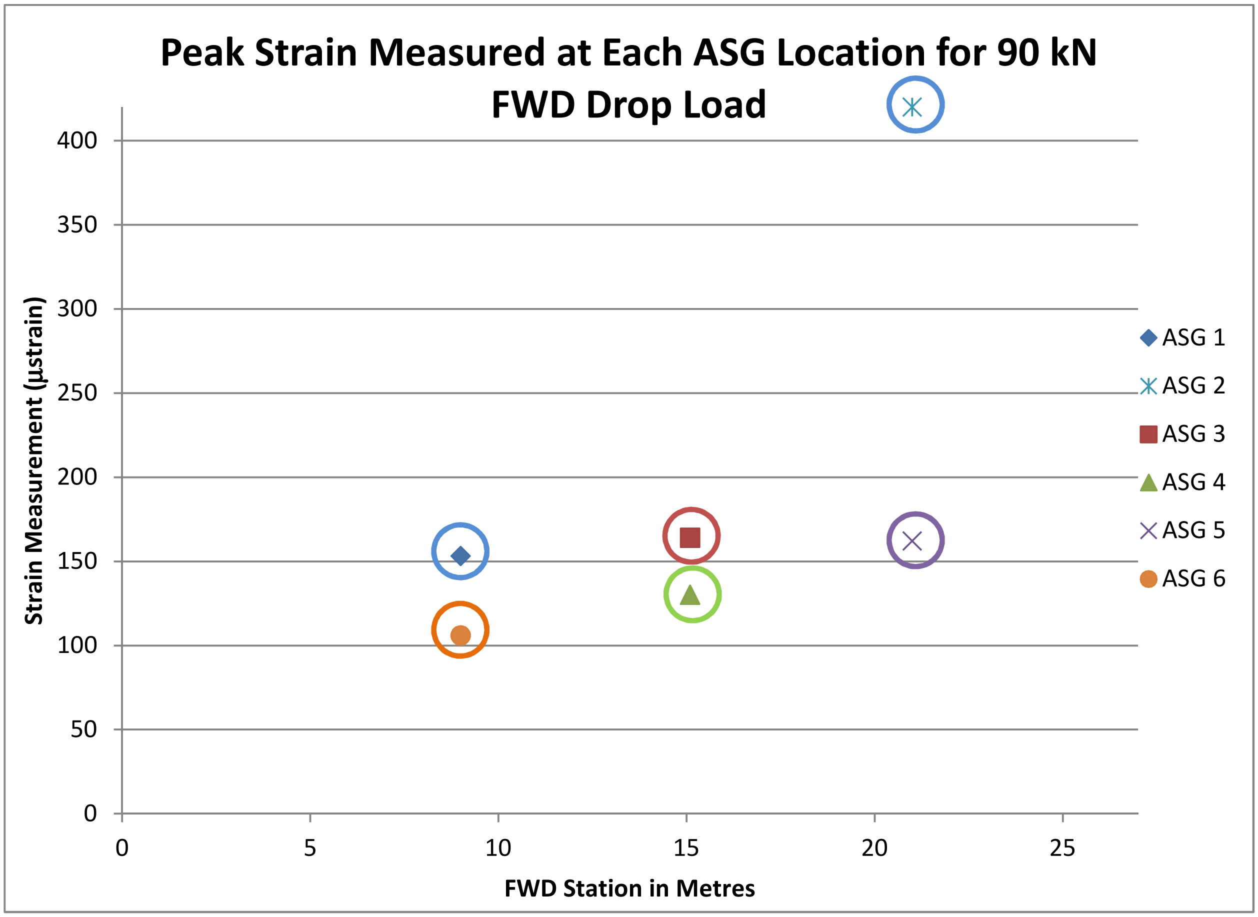


Figure 35 - Evaluation of ASG Depth in Pavement Structure

A diagnostic check was made on ASG 2 and ASG 5 to see if there was some factor in the test section or ASG setup which could explain the relatively high measured strains compared to the remainder of the test section. The diagnostic check included a visual evaluation of the testing section to see if there were any anomalies such as cracking or other distress which may affect the readings as well as checking the readings recorded in other data logger channels in order to eliminate the possibility that the out of range readings were being cause by issues with the installation or the particular datalogger channel. The visual evaluation did not identify any distress which may be causing the out of range readings. Similarly, ASG 2 and ASG 5 provided the same results in different datalogger channels and as a result it has been concluded that the out of range readings must be due to either an issue during the installation/test site construction or simply a gauge malfunction due to age. On the Marquette Interchange Project, the researchers used a portable signal generator to generate a sinusoidal signal in the strain gauges to check the resulting signal generated by the gauges as well as the acquisition rate of the datalogger. Unfortunately while this test could have proved useful for this research, this type of equipment was not available for use on this site [Hornyak & Crovetti, 2008]. With the analysis that was completed, the strain measurements gathered by ASG 2 and ASG 5 are not considered to be reliable and have been excluded from further analysis.

4.3. Summary of Findings

The testing that was completed at the site showed that it is possible to determine the location of embedded earth pressure cells and asphalt strain gauges in a pavement structure using the FWD. This process however is challenging and time consuming and could have been even more difficult if multiple gauges were installed at the same station and depth with either a slight offset or a difference in orientation like some test setups. Accurate sensor locating is made even more challenging when the gauges in a test site are not functioning properly, and are either providing non-repeatable results or results with a lot of signal noise.

Some lessons that were learned from this testing is that the testing and calibration of embedded sensors can be made much easier if the sensor locations are accurately surveyed and/or permanently marked in the field so that they can be easily found in the future. In addition, the cables which are connected from the embedded sensors into the data logger should be properly labelled using durable markers inside the data logger box which are capable of withstanding the range of climatic conditions that are expected for the site. In order to provide redundancy to the labelling of the embedded sensors in the datalogger channels, it is suggested that a comment be inserted into the source code for the data logging software which links the channel that is being monitored to the location of the embedded sensor that is being monitored (either x, y, z coordinates or a written description of the location) to provide a backup of the location information in the case that the durable markers become unusable over time.

5. VERIFICATION OF EMBEDDED SENSOR PERFORMANCE USING A FALLING WEIGHT DEFLECTOMETER

The application of embedded pavement instrumentation has a long history of use in transportation infrastructure and particularly with regard to the traffic and transportation sector. One example is the inductive loop detector which has been in use for detecting, counting, and evaluating the speed of vehicles since the early 1930s [Klein, Mills & Gibson, 2006]. More recently, pavements have begun to be instrumented with more advanced sensors to support Intelligent Transportation System (ITS) applications. These include sensors for Weigh-In-Motion (WIM) which are used to measure vehicle speed, vehicle classification, axle load, gross vehicle weight (to name a few) and Road Weather Information Systems (RWIS) which can be used to measure real-time atmospheric, pavement, water level conditions, and visibility on the roadway [Quinley, 2010].

While pavement instrumentation has been used commercially to measure many different pavement infrastructure performance measures, the use of embedded sensors to monitor in-situ pavement material performance has been primarily utilized in research on either full scale test sites or as part of an accelerated pavement testing project. Given the ever rising cost of our transportation infrastructure, it is foreseeable that embedded sensors will become an important tool to monitor pavement material performance characteristics and quantitatively measure pavement system response to loading in order to monitor the health of the pavement and to schedule optimally timed maintenance as part of an advanced infrastructure management system. This is especially true when dealing with the management of perpetual pavements which represent a significant investment to an owner and where distresses prevention requires good quality data on the mechanical response of the pavement to prevent cracking that initiates from the bottom of the pavement structure which is costly to remediate.

One of the main reasons that embedded sensors have not been used more frequently in applied pavement management is due to their current technical limitations. Like many pavement sensors, embedded sensors generally have a limited lifespan, which is far less than the life of the pavement. While traffic loop and WIM sensors are installed at or near the surface of the pavement and are relatively easy and cost effective to replace, pavement vertical and horizontal strain sensors are installed in the pavement structure and are expensive and difficult to replace. In addition, the process of retrofitting strain gauges in existing pavements tends to alter how the stresses are generated and measured and have typically resulted in unreliable strain data. Recent advancements in sensor technology such as the use of fibre optic strain sensing techniques and self-powered piezoelectric transducers are promising to overcome these limitations in the lifespan of the sensors and facilitate rehabilitation of the sensors when required [Doré, 2013, FHWA 2013].

Embedded sensors can already play an important role in monitoring the health of perpetual pavements given that they are relatively inexpensive when compared to the unit costs of construction. Firstly, if installed during the construction of the perpetual pavement, they can be used to measure the strains post-construction to ensure that the pavements are performing in accordance with the design. If there is a deficiency in the performance after construction, then a rehabilitation treatment can be designed to bring the performance within the design. Similarly, the installed sensors can then be used to gather good quality data on the mechanical response of an existing pavement during the initial service period which is critical to the success of a perpetual pavement. Once the lifespan of gauges has been improved, then long term health monitoring of the perpetual pavement will become possible as well as evaluating changes to the trafficking of the pavement as well as the effects of climate change.

Whether embedded sensors are used for short term or long term monitoring, they require calibration and validation just like any other pavement inspection or testing tool and a quality control program should be developed to ensure the accuracy and reliability of the data which are being gathered in the system. However, this cannot be done using traditional laboratory methods given that these sensors are embedded deep within the pavement structure and not accessible. Sensors that have recently been installed require testing to determine if they were installed correctly as gauge orientation is sometimes changed or offset when overlaying the next hot-mix asphalt lift. In addition, sensors installed in hot-mix asphalt lifts are subject to very harsh thermal and load related stresses that leads to frequent sensor damage and loss and the function of the sensors should be verified before relying on the data. If sensors are used over the short or long term, their function, accuracy, and repeatability should be periodically checked to ensure that they are producing reliable results.

There are a number of potential ways embedded sensors could potentially be validated. One method would be the use of a vehicle with a calibrated load which travels amongst live traffic. The difficulty with using a calibrated vehicle in live traffic is that it can be difficult to match the sensor response with the loaded vehicle without having all of the systems in synchronization in terms of their time and without having supplementary systems such as CCTV and/or subcentimetre accuracy GPS. It addition, it is difficult to ensure that the vehicle travels directly above the sensor at highway speed (wander) which will affect the measured strain even after taking the time and expense to carefully weigh the test vehicle and control the tire/pavement contact pressure. There are other sensors available on the market that can be installed to measure wheel wander which could then be used to adjust the results obtained from live traffic. These sensors will add cost to the monitoring project as they too will have to be monitored and regularly calibrated in order to ensure that they provide reliable results. These sensors also have a short functional life when compared to the expected life of a perpetual pavement and will require numerous replacements over the life which may not justify the cost unless they are used in conjunction with other monitoring being completed on the site.

Another method would be to use a plate load test. While quite accurate, the plate load is a very time consuming and costly test to perform which will require extensive traffic control and lane closure time (and associated traffic disruption). One non destructive test method which has been used for many years for pavement performance monitoring and can overcome these technical issues is the falling weight deflectometer.

The falling weight deflectometer has been in regular use for non destructive testing in North America since the early eighties. The load that is produced from each FWD drop is measured by a highly accurate and calibrated load cell which then records the imparted load, time as well as a number of other pieces of information from the test into a testing database in an onboard computer. In addition, the FWD can be positioned to precisely above the location of the sensor in order to prevent the effects of wander in the calculation. The FWD also simulates the dynamic action of traffic unlike the plate load test and the test can be completed quite rapidly, taking 1 to 2 minutes per test point depending on the desired number of test drops. Due to its versatility, the FWD can be used to validate sensor construction but can also be used to validate the as constructed performance of the perpetual pavement section either before it has been commissioned to traffic or while it is in service.

Validation of sensor construction is very important to ensure that the sensor alignment was not changed during construction of the overlying layers and the sensor was not damaged from the heat of hot-mix asphalt placement or the stress of the compactive effort. The FWD can be used to validate the sensor after the first lift of asphalt concrete has been placed over the sensor and before it is too late to repair or replace a damaged sensor. Using relatively light loads, and as long as the stiffness and temperature of the overlying layer is well known, the recorded load and be readily converted into a horizontal strain at the bottom of the pavement layer in order to validate the readings being recorded by the embedded strain sensors. After all of the asphalt concrete layers have been constructed, the FWD can return to the location of the asphalt strain gauges and complete the testing program again in order to determine the resilient modulus of the composite of the bound layers in the perpetual pavement structure to validate the design.

The second problem, and the one explored in detail in this Thesis is the calibration of in-service sensors to ensure that the data that they are gathering is reliable and can be used in the management of the pavement. For pavement sections that have had validation testing using a FWD, the sensor calibration would be relatively straightforward. Using the as-constructed thickness of the pavement structure, the FWD could be used to back-calculate the effective stiffness after aging of the composite asphalt concrete layers on the site. Then using the asphalt concrete temperature, the FWD could then be used to apply a known load directly on top of the sensors and the measured strain compared to the anticipated strain and the strain gauge calibration factor (gain) adjusted as necessary to obtain the correct strain measurement.

5.1. Prediction of Tensile Strains in Test Section

Prior to completing field falling weight deflectometer testing in the test section, the expected response in terms of deflection and strain were predicted in order to provide a quick quality check during testing to determine if the measured deflections were in line with the expected deflections and to ensure that the embedded sensor response was in a reasonable range given that they have been in service for approximately four years. The strains and deflections were predicted using the detailed material properties testing of the individual pavement layers completed as part of the design and construction of the test site which was input into computer software capable of calculating the stresses and strains in an elastic multilayer system.

The resilient modulus of each of the hot-mix asphalt layers was determined in the laboratory prior to placement in the test section. A full description of the sample preparation and test results has been published by El-Hakim [El-Hakim, 2013]. Test samples were created from each mix using a gyratory compactor with the samples trimmed to meet the dimensions required for test specimens. The samples were then tested in accordance with the requirements of ASTM D7369-09 *Standard Test Method for Determining the Resilient Modulus of Bituminous Mixtures by Indirect Tension Test* [ASTM, 2009]. After testing was completed, the data from any of the samples which did not meet the criteria of the ASTM standard were excluded in accordance with the requirements of the standard. A minimum of four samples were used to calculate the design resilient modulus of each mix. The results of the testing at 21°C are summarized in Table 1.

Table 1 – Summary of Resilient Modulus Values of Mixes Used in the Test Section

MIX TYPE TESTED	AVERAGE RESILIENT MODULUS (MPA)	STANDARD DEVIATION
Superpave 12.5 Surface Course	2300	163
Superpave 19 Binder Course	2176	162
Superpave 25 RAP Lower Binder Course	2962	50
Superpave 25 RBM	2867	137

Note – Four samples were used to calculate the design resilient modulus

The software used to predict the deflections and strains based on the design pavement parameters was the mechanistic design software called Kenpave. Kenpave was developed by Dr. Yang H. Huang of the University of Kentucky and provides the capability of completing a mechanistic analysis of both flexible and rigid pavements [Huang, 2003]. The KENLAYER portion of the Kenpave software is used to provide a solution for an elastic multilayer system under a circular loaded area which is precisely the type of loading that was applied with the falling weight deflectometer. While the Kenpave software is capable of analyzing systems under single, dual, dual-tandem or dual-tridem wheel loadings, including the possibility of calculating the accumulated damage over up to twelve different periods (weather/temperature conditions), the solution for a single wheel loading in a single period was considered to accurately reflect the conditions encountered in the test section by the application of a load to a circular load plate by the FWD.

There are a number of different parameters that are required in order to complete the analysis using KENLAYER. The key components of the analysis require the definition of the number of layers, their thicknesses and design parameters, the types of loads that will be applied as well as the points within the pavement structure that are supposed to be analyzed. The key parameters used in the prediction of the deflection and tensile strains in the test section are summarized in Table 2.

Table 2 – Summary of Key Design Parameters used in KENLAYER Analysis

DESIGN PARAMETER	RESPONSE	VALUES
Number of Periods	1	Fall
Number of Load Groups	3	530, 800, 1180 (kPa)
Number of Layers	6	4 HMA Layers, Granular Base, Subgrade
Number of Coordinates for Analysis (cm)	4	0 (surface), 32 (ASG shallow), 42 (ASG deep), 117 (EPC)
Material Properties for Each Layer (kPa/Poisson Ratio) at 21°C		2,300,000/0.35; 2,176,000/0.35; 2,962,000/0.35; 2,867,000/0.35; 250,000/0.30; 30,000/0.45

The results of the KENLAYER analysis provide a number of different calculations including the deflection (vertical displacement), vertical stress, radial stress (tensile strain), tangential stress and shear stress. These calculations are provided for each load group analyzed and for each coordinate which is being analyzed. However, in order to evaluate the response of the materials and sensors at the test section, there are only four key parameters that need to be evaluated which are: the vertical deflection at the surface; the tensile strain at the shallow asphalt strain gauge placed between the lower binder course (SP 25 RAP) and the rich bottom mix binder course (SP 25 RBM) (ASG shallow placed at 32 cm depth in the pavement structure); the tensile strain at the deep asphalt strain gauge placed between the rich bottom mix binder course (SP 25 RBM) and underlying granular base (ASG deep placed at 42 cm depth in the pavement structure); and the vertical stress at earth pressure cell installed at the interface between the granular base and the subgrade (EPC installed at 117 cm depth in the pavement structure). The predicted parameters based on the design inputs for the site are summarized in Table 3.

The calculated deflections and vertical and horizontal strain values are based on the design resilient modulus values which were determined at a design temperature of 21°C.

Table 3 – Summary of Results from KENLAYER Analysis

DESIGN PARAMETER	VALUES
Vertical Deflection at Surface (μm)	40 kN – 29.25 60 kN – 44.15 90 kN – 65.12
Tensile Strain ASG Shallow (microstrain)	40 kN – 27.75 60 kN – 41.89 90 kN – 61.79
Tensile Strain ASG Deep (microstrain)	40 kN – 53.57 60 kN – 80.86 90 kN – 119.30
Vertical Stress at EPC (microstrain)	40 kN – 46.52 60 kN – 70.22 90 kN – 103.60

Note that negative values for strain in the KENLAYER output have been reported as positive values in the table above

5.2. Analysis of Tensile Strains From Embedded Sensors in Test Section

Research by Timm et al. at the NCAT Test Track has shown that the performance of pavement sensors can be affected by pavement distress with cracked sections found to display less repeatable strain results [Timm, 2009]. As a result, a detailed pavement condition inspection of the test section pavement was completed prior to the start of falling weight deflectometer testing to ensure that existing distress would not affect the testing results. The pavement condition inspection showed that the test section pavement was generally in excellent condition after five years of service with no visually detectable cracks, distress or other discontinuities in the pavement surface. The only observable distress was a low severity longitudinal crack that ran symmetrically around the entire perimeter of the test section approximately 65 cm from the edge of the paving. It is speculated due to the symmetry of this distress that it is caused by some detail from the construction of the test section (step joint for instance). In addition to the perimeter crack, there were some longitudinal cracks that started outside of the test section and propagated up to and intersecting with the edge of the perimeter crack. These distresses were likely pre-existing distresses in the pavement outside of the test section which progressed up to the discontinuity (perimeter crack) inside of the test section but could not penetrate the perpetual pavement structure. As the observed distresses are at the outermost edge of the test section, the distresses are not expected to adversely affect the results. Photographs of the site conditions are shown in Figures 36 and 37.

Figure 36 - Photographs of Test Section Condition

Figure 37 - Photographs of Crack Initiated Outside the Test Section and Ending at the Location of the Perimeter Crack

After arriving at the test site and before the testing started, the same FWD warmup and site initiation procedure as described in Section 3 was completed in order to warm up the buffers to ensure a repeatable applied load and to calibrate the drop height to the local conditions to obtain the planned target loads. During the warmup the pavement mid-depth temperature was gathered manually using the same drill hole that was drilled previously into the test section. The hole was first cleared using compressed air and filled with 25 mm of mineral oil. The hole was then covered and the stabilized temperature was measured using a calibrated digital temperature probe after a minimum of 15 minutes had passed.

The research FWD testing consisted of 33 drops at each test location consisting of three seating drops at 30 kN, 10 drops at 40 kN, 10 drops at 60 kN and 10 drops at 90kN. The data from the three seating drops was not recorded, while the deflection profiles from the other 30 drops were recorded in the testing database including one time history from each load level. During testing, the data quality was checked using automated validation tools and the test was repeated if any of the quality checks failed. The five quality checks included: roll-off; nondecreasing deflections; overflow; load variation; and deflection variation.

The recorded deflection results from each of the FWD drop levels were compared against the amount of deflection predicted using the Kenpave analysis in order to evaluate the difference. The results of the comparison are shown in Figure 38. The results show that the measured deflection was between 6 (at the 40 kN load level) and 9 (at the 90 kN load level) microns greater than the deflections predicted using Kenpave. In addition, when grouping the deflections by load level, the deviation was observed to be fairly consistent by load level and the deviation increased as the load level increased. As the measured deflections were greater than the predicted deflections, this would indicate that the resilient modulus of the pavement structural components is lower than those used in the prediction calculation. This observation implies that further analysis will be required to determine which layer or layers are responsible for this reduced overall modulus and new moduli will have to be calculated in order to facilitate further analysis.

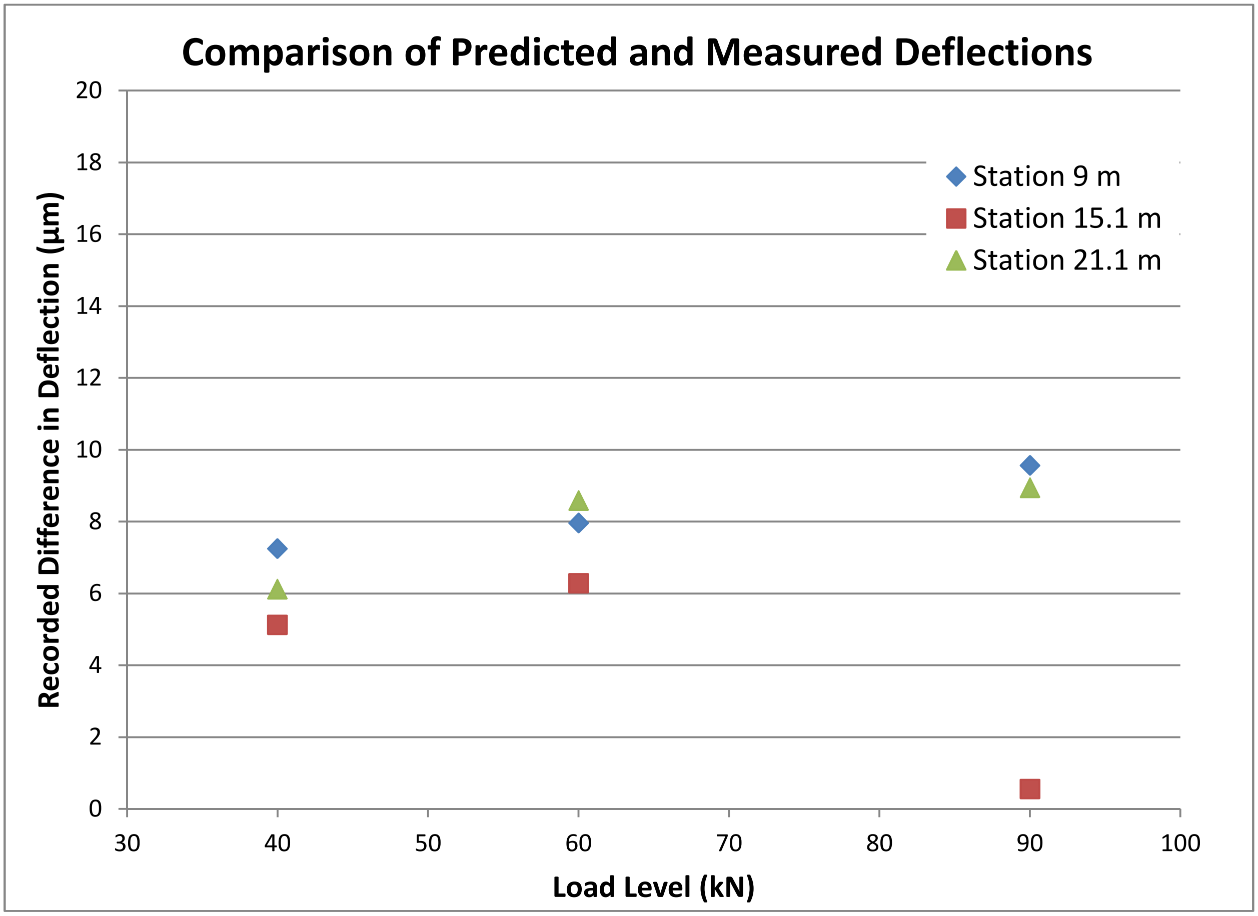


Figure 38 - Comparison Between the Predicted and Measured Deflections

The measured strains from the four reliable sensors were also compared against the strain levels predicted using the Kenpave analysis in order to evaluate the difference. The results of the strain comparison are shown in Figure 39. It was observed that the measured strain values were generally higher than the values which were predicted in the Kenpave analysis. This result corroborates the results of the pavement deflection analysis indicating that the resilient moduli of the pavement structural components must be lower than those that were calculated at the time of the design. The figure shows that the difference between the predicted and measured ASG response is highest in the upper portion of the pavement structure (shallow ASG) and becomes less pronounced in the deeper ASG. What is also shown in Figure 39 is that the measured strain in the EPC is also higher than the predicted strain, and in the exact same proportion as the lower ASG. This would indicate that the greatest difference in the strain appears to be in the upper portion of the asphalt concrete and that the granular materials and subgrade are performing as designed given that there is no continued reduction in the strain between the lower ASG and the EPC.

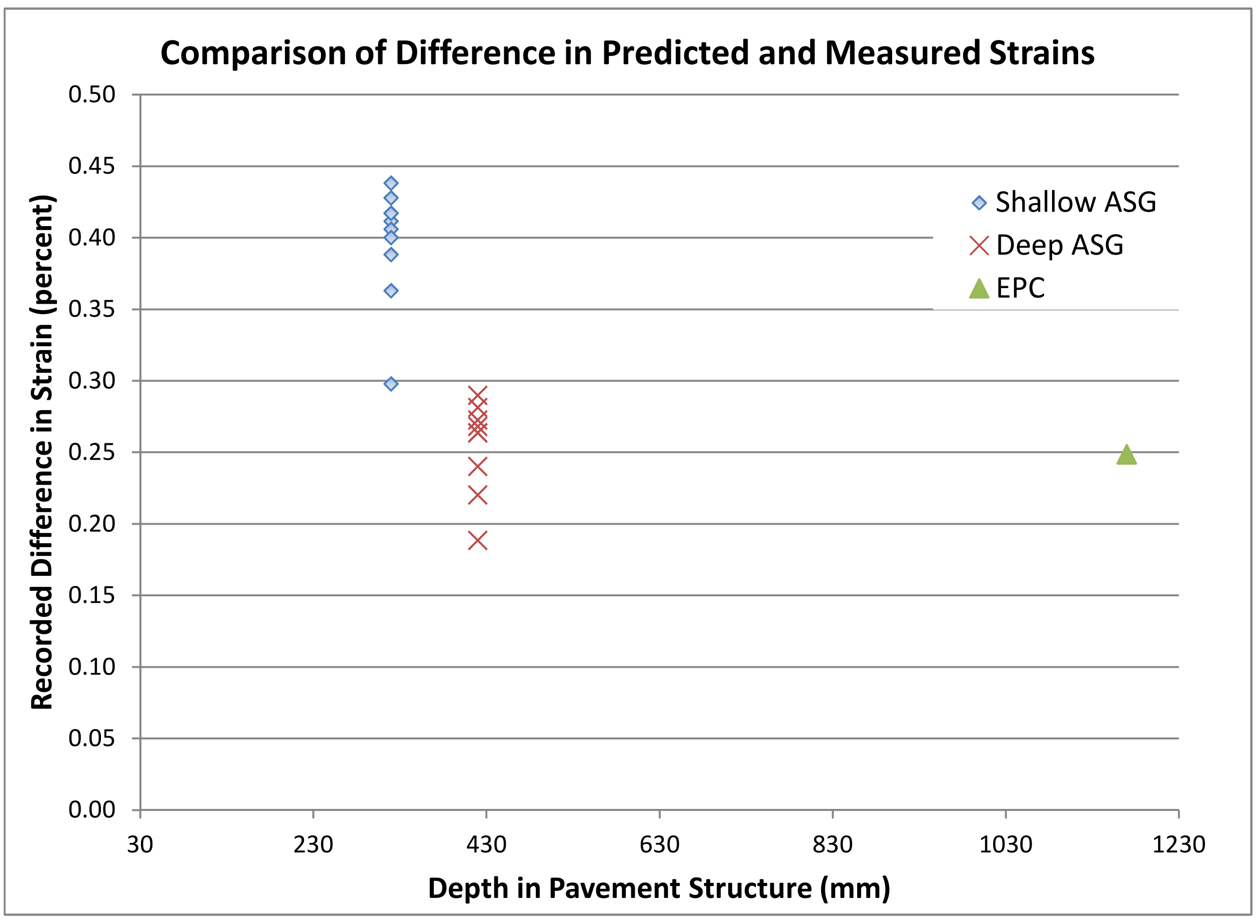


Figure 39 - Comparison Between the Predicted and Measured Strains in the ASGs and EPC

5.3. Back-calculation Analysis Using FWD Deflections

The results of the field testing show that the strains measured using the embedded sensors and the deflections measured using the falling weight deflectometer at the test site do not match those which were predicted using Kenpave with the resilient modulus values obtained during laboratory testing at the design phase. The measured strains and deflections were consistently higher than those predicted indicating that the resilient moduli of the pavement structural components were, or are now, lower than when they were designed. It is possible that the resilient modulus of any one or any combination of the pavement structural components has changed and as a result, the challenge is to isolate the affected layers which can be done using the embedded sensors.

The analysis shown in Figure 39 indicates that the upper layers of the pavement structure show the greatest difference between the measured and predicted performance, while there is no difference in the loss of performance below the bottom of the asphalt concrete layer. This would indicate that the main loss of performance is in the asphalt concrete.

In order to confirm this premise, the resilient moduli of the granular base and subgrade was backcalculated from the FWD data using the Elmod 6.0® computer analysis software [Dynatest, 2014]. The Elmod 6.0 software uses the equivalent thickness method for backcalculation based on the Boussinesq equations and Odemark's transformations (Method of Equivalent Thickness (MET)). The backcalculation inputs that are required are the load and deflection values obtained from the falling weight deflectometer and the precise thickness of the pavement structural components. The backcalculation is then completed iteratively by the software using a deflection basin fit method which employs numerical integration techniques to match the measured deflection basin with a calculated deflection basin. During each iteration, a theoretical deflection basin is calculated based on the stress level at the centre of the load plate. The error, which is calculated as the difference between the measured and calculated deflection basins, is then assessed and the moduli of the layers in question are then adjusted slightly based on the inputted convergence criteria with the error then recalculated. If the error of the calculated deflection basins is less than the measured deflection basin, then the calculated deflection basin is taken as the better solution and used in the back-calculation of the layer moduli. The backcalculation for the test section used the minimize difference function when calculating the theoretical basin and the calculation was completed with a specified maximum offset of 10%. The backcalculated moduli are based on deflection data that has been normalized from the test temperature to a standard temperature of 21°C. The results of the backcalculation are provided in Table 4.

Table 4 – Summary of Backcalculated Granular Base and Subgrade Values

DESIGN PARAMETER	BACKCALCULATED VALUE (MPA)	STANDARD DEVIATION (MPA)
Granular Base Resilient Modulus	243	8
Subgrade Resilient Modulus	29	4

The backcalculated resilient modulus values that were obtained for the granular base were multiplied by a factor of 0.62 in order to get the equivalent laboratory design resilient modulus value in accordance with MTO procedures [MTO, 2012]. Similarly, the backcalculated resilient modulus values for the subgrade were multiplied by a factor of 0.35 in accordance with the same design procedures in order to obtain the equivalent laboratory design resilient modulus for the subgrade [MTO, 2012]. As the backcalculated resilient modulus values for the granular base and subgrade are similar to the ones used in the original design the supposition that the loss of resilient modulus is confined to the asphalt concrete layers is confirmed.

As pavements age and become distressed, the resilient modulus of the asphalt concrete becomes lower. This loss of pavement life is quantified in the Mechanistic-Empirical Design Guide as the damage factor which is used in the MEPDG analysis to determine the field-damaged dynamic modulus master curve. For a perpetual pavement, the principal of the design is to limit the distress to the surface course asphalt concrete which is readily resurfaced. A limitation of the backcalculation method to determining the resilient modulus is that it cannot be reliably done for thin layers and specifically at the surface of the pavement. As a result, in order to confirm that the reduction in modulus is in the asphalt concrete layer, the effective resilient modulus of the entire asphalt concrete layer must be back-calculated and compared to a weighted average modulus based on the design.

The resilient modulus of a composite of all of the asphalt concrete layers was backcalculated using the Elmod 6.0 software. The same analysis parameters were used as the previous analysis except the modulus of the granular base and subgrade were fixed to their respective design values. The resilient modulus value was normalized to a temperature of 21°C in order to compare the results to the design resilient modulus value. In addition, the resilient modulus at the test temperature is provided as this is the resilient modulus that relates to the deflections and strains that were measured at the time of testing. The results of the backcalculation are summarized in Table 5.

Table 5 – Summary of Backcalculated Asphalt Concrete Values at 21°C

DESIGN PARAMETER	BACKCALCULATED VALUE (MPA)	STANDARD DEVIATION (MPA)
Normalized Composite Asphalt Concrete Resilient Modulus	2,398	79
Composite Asphalt Concrete Resilient Modulus at Tested Temperature	2,129	66

The weighted average resilient modulus of all asphalt concrete layers based on the design parameters was 2,539 MPa

Using the new backcalculated resilient modulus value of the asphalt concrete, the predicted deflections and strains were calculated in the Kenpave software in order to determine if there was still a difference between the measured and the predicted values. After adjusting the asphalt concrete modulus using the backcalculated value, there is still a difference between the measured and predicted deflections and strain values, however this difference has narrowed slightly. The only remaining variable that has not been explored is the calibration factors of the gauges and as a result, this is considered the most likely reason for the remaining difference. In order to evaluate this hypothesis, the calibration factors for the embedded sensors was adjusted to account for this apparent error and the site re-tested to validate the revised calibration factors. A summary of the difference is provided in Table 6.

Table 6 – Comparison of Measured versus Strain Values Predicted Using Backcalculated Asphalt Concrete Modulus

DESIGN PARAMETER	PREDICTED VALUE AT 90 kN LOAD LEVEL	MEASURED VALUE	DIFFERENCE MEASURED/PERCENT
Average Deflection at the Pavement Surface (mm)	0.684	0.713	0.029/4.0
Tensile Strain ASG 6 (microstrain)	79.68	104.7	25.02/31.4
Tensile Strain ASG 4 (microstrain)	79.68	106.4	26.72/33.5
Tensile Strain ASG 1 (microstrain)	128.10	152.7	24.6/19.0
Tensile Strain ASG 3 (microstrain)	128.10	155.0	26.9/21.0

5.4. Calibration of Strain Sensors

Simply put, the way that embedded strain sensors work is by emitting a voltage (in millivolts) based on the microscopic amount of movement that is generated in the pavement structure when loads are applied. The resulting strain values that are generated are linearly varying and are simply multiplied by the gauge's calibration factor and adjusted by a constant (gauge offset to zero) for a gauge to provide the actual amount of strain which is being generated.

For the asphalt concrete strain gauges used at the site, the initial calibration factor was completed by the gauge manufacturer prior to them being installed. The laboratory calibration consists of placing the gauge in a specially fabricated bracket which is fitted with a precision extensometer. Weights are then hung from the bracket and the amount of movement measured by the precision extensometer is related to the measured voltage change for different weights. The gauge is then rotated 180° and the testing completed again to determine the calibration factor in the opposite orientation. This additional step is completed in order to account for slight misalignments and bending of the gauges. A typical example of the results of this calibration procedure is shown in Figure 40.

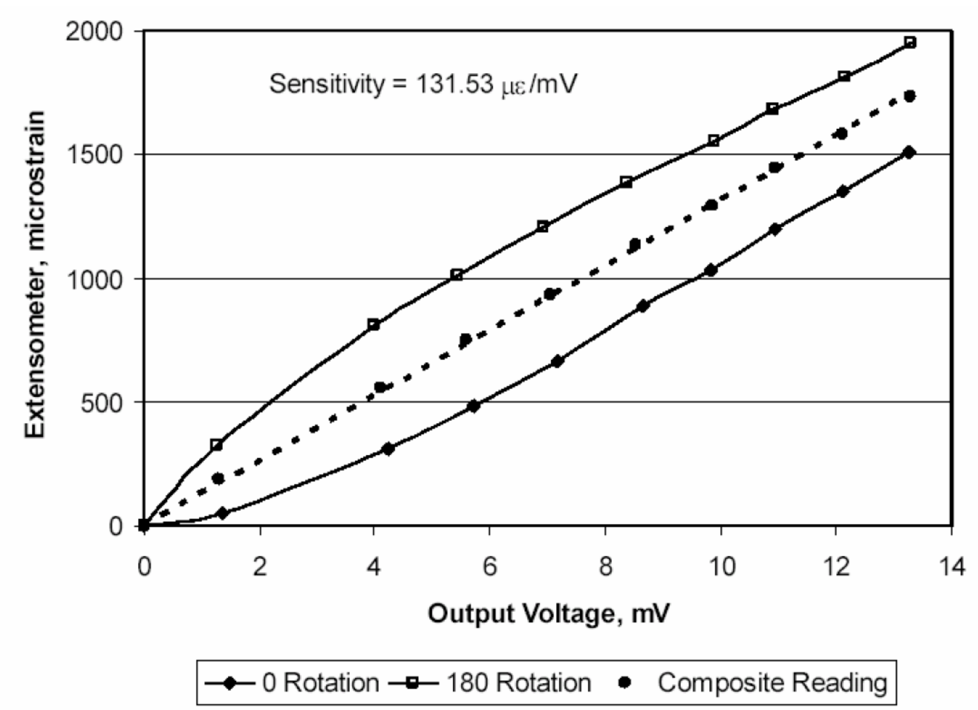


Figure 40 - Typical CTL ASG Calibration Result [Hornyak & Crovetti, 2008]

Once the strain gauges have been installed at a site, a detailed laboratory calibration is no longer possible. As a result, in order to recalibrate the embedded asphalt strain gauges, a new calibration factor must be calculated in order to eliminate the calculated error in the gauges after the other variables that can affect the ASG readings have been taken into account. For the test section, the new calibration factor for each gauge was calculated assuming that the difference between the average of the measured strain and the calculated strain was the error and the calibration factors adjusted to account for this error. The existing and the new calibration factors that were calculated for each of the gauges are provided in Table 7.

Table 7 – Embedded Sensor Calibration Factors Used at Test Site

EMBEDDED SENSOR	ORIGINAL CALIBRATION FACTOR με/mV	ADJUSTMENT RATIO	NEW CALIBRATION FACTOR με/mV
Asphalt Strain Gauge 1	126.64	0.8326	105.44
Asphalt Strain Gauge 3	129.17	0.8326	107.52
Asphalt Strain Gauge 4	129.70	0.7549	97.91
Asphalt Strain Gauge 6	131.84	0.7549	99.52

A detailed asphalt strain gauge calibration study was completed by the Transportation Research Centre at Marquette University to determine if there was a difference between the calibration factors provided by the manufacturers of asphalt strain gauges and the results obtained using the calibration equipment in their laboratory [Hornyak & Crovetti, 2008]. For CTL brand asphalt strain gauges, an average adjustment ratio of 0.93, with a range of 0.83 to 1.1 was recorded as part of this research as illustrated in Figure 41.

The difference between the CTL calibration factors for the gauges provided for the Guelph test site and the factors calculated as part of this research are similar to the results obtained in the Marquette experiments. As with the Marquette experiment, the Guelph test site gauges were found to overestimate the amount of strain that was expected to be measured. The gauges that were installed at the lower interface had an adjustment ratio (0.83) at the lower range of those measured for the Marquette site (0.83 to 1.1) which are considered by the researchers to be within the range of measurement error of the calibration procedure [Timm 2009]. The gauges that were installed shallower in the Guelph test section had an adjustment ratio of approximately 0.76 which is beyond the range which was reported in the Marquette experiment.

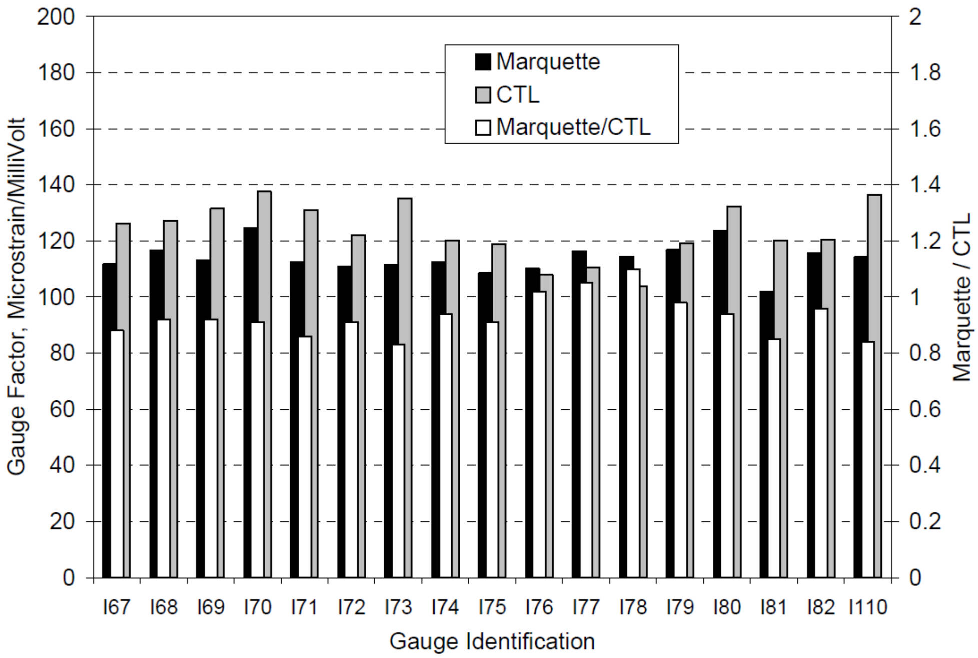


Figure 41 - Comparison of WHRP and CTA ASG Calibration Results [Hornyak & Crovetti, 2008]

The adjustment ratio of 0.91 in the Marquette experiment is explained to be caused by a mix of differences in the gauge factors stated by the manufacturer and construction flaws and errors during the calibration process [Hornyak & Crovetti, 2008]. The additional increase in the measured strain (lower adjustment ratios) of the in-service asphalt strain gauges at the Guelph Site compared to the laboratory calibrated strain gauges are attributed to defects that could have taken place during installation of the sensors in the test section or the effects of wear and age on sensors that have been in service for five years. These factors should be checked experimentally to determine the cause of the increase in adjustment ratio post construction.

5.5. FWD Testing of the Test Section Using Re-Calibrated Embedded Sensors

The revised strain gauge calibration factors were input into the data logging computer program and loaded into the Campbell Scientific datalogger program at the test site in order to determine if the re-calibrated gauges measured strains which are equivalent to those which have been predicted.

The validation testing of the site consisted of FWD testing of ten drops each at 40 kN, 60 kN and 90kN directly above the strain gauges as had been done during the calibration process. The data from the 30 drops was recorded in the testing database including one complete time history for one of the 90 kN load levels from each drop set. The pavement mid-depth temperature was again gathered manually using the same drill hole that was drilled previously in the test section. It should be noted that the temperature during the validation process was cooler than during the calibration process and the in-situ resilient modulus and the predicted strains had to be adjusted to reflect the increase in modulus of the asphalt concrete layer due to the colder temperature. The strain measurements gathered by the datalogger were downloaded and analyzed to compare the measured strains to the predicted strains. The response of the strain gauges using the new gauge factors is shown in Table 8.

Table 8 – Comparison of Measured versus Predicted Strain Value after Calibration

DESIGN PARAMETER		PREDICTED VALUE (με)	AVERAGE VALUE (με)	RANGE (STANDARD DEVIATION) (με)
Tensile Strain ASG (Shallow)	40 kN	30.31	34.5	29.7 - 38.4 (3.1)
	60 kN	45.75	49.2	40.9 - 56.3 (4.3)
	90 kN	67.49	68.9	65.0 - 76.1 (2.9)
Tensile Strain ASG (Deep)	40 kN	53.06	51.8	45.7 - 59.7 (4.1)
	60 kN	80.1	84.5	81.8 - 89.1 (2.0)
	90 kN	118.1	117.8	115.8 - 120.8 (1.8)

n = 10 for each test point

The average values of strain which were measured in the asphalt strain gauges correlate well with the predicted strains using the new calibration factors with much less error than was recorded using the previous calibration. As a result, the new calibration factor is considered to successfully improve the measurement of the strain by the strain gauges.

5.6. Summary of Findings

The research completed in this chapter consisted of first predicting the expected tensile strains at the depth of the embedded sensors in the test section followed by verification of the strains that were being measured in the embedded sensors using the falling weight deflectometer. The strains that were measured in the embedded sensors were then compared against the predicted strains. As the measured strains were significantly higher than the predicted strains, additional study was required in order to evaluate the cause of this discrepancy.

The first component of this research was to determine if there was a material difference in the pavement structural components that could explain why higher than expected strains were being measured. This was completed by backcalculating the effective modulus of each of the pavement layers using the deflection data obtained by the FWD testing. The backcalculation found that the modulus of the subgrade and granular materials were similar to those used in the design, however the modulus of the asphalt concrete layers was somewhat lower than the design. The new asphalt concrete modulus was used to calculate new design strains and these new predicted values were compared to the values that were being measured by the embedded strain gauges. While the difference was reduced after this step, the embedded sensors were still measuring strains greater than what was predicted.

The remaining difference between the calculated and measured strain was attributed to an issue with the calibration of the embedded sensors. Using the remaining difference, new strain gauge calibration factors were determined and then validated by additional testing in the field. The site was re-tested using the FWD and new strains were measured. The new strains that were measured were considered to be within the standard error of asphalt strain gauges validating the use of the new gauge calibration factors.

6. FRAMEWORK FOR MONITORING THE LONG TERM PERFORMANCE OF PERPETUAL PAVEMENTS

Perpetual pavements represent a significant investment to an owner who has committed to spending a premium at the beginning of construction of a pavement asset in order to benefit from potential long-term savings from the enhanced performance of this asset. This makes the monitoring of perpetual pavements even more important to ensure that the asset is optimally maintained in order to meet the performance expectations for the design life and potentially beyond.

In structural engineering, the process of monitoring a system's performance has been given the term structural health monitoring (SHM). This process involves the use of sensors and other non-destructive testing techniques to monitor the ability of the structure to meet its performance criteria as it ages and after extreme events such as collisions and earthquakes, and allows for the planning and application of preservation treatments which are used to ensure the structure meets the performance expectations of the design. Embedded sensors can be used in a similar manner in flexible pavements in order to monitor the structural health of the different pavement structural components.

The monitoring of perpetual pavements requires carefully planned and reliable monitoring which is completed at strategic locations in the pavement structure. The optimal setup for a monitoring program has not been studied as part of this research as it has already been studied in detail in prior research projects that have been completed at accelerated testing facilities and research sites [Timm, Priest, & McEwen, 2004, Hammons, Timm & Greene, 2007, Hornyak, Crovetti, Newman & Schabeiski, 2007, Timm, 2009]. However, supplemental considerations have been developed as part of this research that could be completed during the initial design and construction stages that may facilitate the calibration and verification of embedded sensors in the future.

6.1. Lessons Learned from the Guelph Testing Site

There were a number of lessons learned when evaluating the Guelph Test Site that should be shared in order to advance the knowledge of monitoring a perpetual flexible pavement test site using non-destructive testing equipment such as an FWD and embedded sensors.

The first issue, and one that has been studied in detail in this research is the gauge locating. It is recommended that at future monitoring sites, the gauges should be located using multiple redundant processes in order to facilitate locating them in the future when required. Firstly, it is recommended that a permanent marker (such as a nail, or something similar) be placed a precisely measured offset away from the sensor location after the final lift of asphalt concrete has been placed. This permanent marker will have to be replaced after any surface rehabilitation has been completed which removes this permanent marker. The location should also be surveyed using accurate surveying equipment (such as total station) to provide a backup in case the permanent marker is lost or damaged. The surveyed location should input into a site maintenance log. A copy of the log should be stored in the datalogger box. In addition, the location can be stored as a comment in the datalogger software.

It is recommended to label both ends of the embedded sensors using a durable, weather resistant labelling material with an identification code which relates to the sensor's location within the monitoring section. In addition, placing the labels at both ends of the gauge facilitates the installation process. It is important to maintain a database of the manufacturers' identification codes and records (such as calibration factors) for each of the sensors in order to facilitate future validation and calibration efforts. A copy of the database information can be stored in the datalogger box. In addition, the identification codes and calibration factors can be stored as comments in the datalogger software.

A properly functioning moisture probe as well as an understanding of the change in resilient modulus of the site subgrade materials with moisture would be helpful in order to more accurately determine the instantaneous resilient modulus of the subgrade. Typical backcalculation procedures for subgrade materials require the use of factors in order to determine the spring (soaked) modulus which is used for design. Converting these values to reflect the expected modulus in different seasons can introduce variability into the calculation that can be overcome by an understanding of the behaviour of the site subgrade material which will improve the future validation and calibration process by removing this uncertainty.

A temperature probe which is installed in the asphalt concrete would be useful in order to determine the temperature profile of the perpetual pavement and could be incorporated as part of a weather station used to monitor the site. The resilient modulus of the asphalt concrete will vary depending on the temperature of the material. In addition, the asphalt concrete layer of a perpetual pavement is quite thick when compared to other flexible pavement designs. When combined, these two factors can introduce variability into the validation and calibration process that can be reduced by the presence of this type of sensor.

While a relatively small detail, it is important to synchronize the clocks of the falling weight deflectometer and the datalogger prior to beginning the testing. This synchronization makes the comparison of deflections and strains easier and reduces the processing time for the sensor validation and calibration process.

There are limitations when comparing predicted and measured responses in pavement structures. The analytical models used to predict strains and deflections make assumptions in order to simplify the nature of these sometimes complex materials. While these simplifications are generally accepted and used in practice, they occasionally result in significant errors. Similarly, the direct measurement of pavement layer performance has certain limitations. For instance, the installation of a strain gauge can introduce a discontinuity into the pavement structure that can alter how the material is behaving. Harmonizing these differences is important to the accuracy of the validation and calibration process.

In order to overcome these differences, it is recommended that FWD testing be completed at the embedded sensor locations prior to the pavement being put into service. This will provide an opportunity to correlate the theoretical and measured responses and this relationship can be used when completing future testing at the site, and to verify the design.

Consideration could also be given to completing correlation testing after the construction of each layer. Pavement testing facilities have reported using different equipment to monitor the gauge performance during construction including the use of a Marshall hammer as well as simply monitoring the real time response of the sensor during compaction and other construction activities. This could also be an opportunity to complete deflection testing using a FWD or perhaps even a light-weight deflectometer (for the initial, thin layers) which can be used to give an early indication of both gauge performance and potential issues with material performance.

6.2. Protocol for Testing and Analysis of Embedded Sensors

In order to have an efficient and effective long-term monitoring program, there are a number of considerations that should be made at the time of initial construction that can not only speed up the process of sensor testing and calibration where required, but also improve the accuracy of the results while providing additional insight into the field performance of the perpetual pavement site for a minimal additional cost.

The first important component to consider is the in-situ modulus of the roadway subgrade. The in-situ modulus of subgrade soils are known to vary depending on the moisture content of the soil with cohesive soils being more sensitive to moisture content variations than non-cohesive soils [Shalaby & Soliman, 2010]. In order to provide the most accurate characterization of the subgrade soils at the instrumented site as well as the perpetual pavement site in general, the subgrade soils should be tested to determine the change in resilient modulus with moisture content. While there have been studies that have been completed which can be used to relate grain size analyses to soil resilient modulus, as a minimum the subgrade moisture sensitivity should be tested for the soils at the location of the embedded sensors to provide calibration to local soils and conditions. The test that is used to determine the soil resilient modulus is the repeated load test, which should be completed at a number of different moisture contents in order to determine a 'master curve' for the response of the soils to different conditions [Shalaby & Soliman, 2010].

In conjunction with the determination of subgrade moisture sensitivity, the test site should be instrumented with a moisture probe in order to provide an evaluation of the change in moisture content over time. Not only will this information be useful in order to determine the instantaneous moisture condition of the subgrade and the effective resilient modulus, this information can be used to determine the moisture history of the subgrade which can be compared against the assumptions made during the design (the design seasons) in order to validate the design assumptions or to determine if preventative maintenance will be required in order to mitigate issues that may arise due to a difference between the assumed and in-service moisture conditions in the subgrade.

The resilient modulus of the in-place materials should be tested after each layer (subgrade, granular subbase/base, and for each successive hot-mix asphalt lift) is constructed in order to determine the as constructed resilient modulus of the pavement layers which can be used to compare against those used in design. In addition, if this testing is completed at the monitoring site, it will provide values of the in-situ resilient modulus for the materials that were placed which can be used to compare against the values which are being determined from future testing which can identify testing issues or material changes that have occurred in particular layers that may be affecting performance.

The in-situ resilient modulus testing can be completed by either a standard falling weight deflectometer, or portable equipment such as a dynamic cone penetrometer (DCP) or a light-weight deflectometer (LWD) could also be used for the unbound materials or the initial (thin) asphalt concrete layers for the site. It should be noted that many procedures that are used to backcalculate asphalt concrete resilient moduli have difficulty in accurately measuring the resilient modulus of thin asphalt concrete lifts (<75 mm). This shortcoming could be overcome if the resilient modulus and thickness of the underlying layers are accurately known.

When FWD or LWD testing is completed as part of the overall Quality Control/Quality Assurance program for the construction of the Perpetual Flexible Pavement, the values obtained throughout the site can be not only be calibrated to the values obtained at the monitoring site, but can also be used to make adjustments to the design to reflect localized deficiencies before the entire pavement has been constructed.

Another benefit of completing the FWD/LWD testing of each pavement layer is that the embedded sensor performance can be evaluated during construction and if any of the gauges fail after the next layer is overlain, they can be replaced before successive layers are constructed and it becomes prohibitive to do so (the potential to damage embedded sensors due to construction is expected to be reduced as each successive layer is placed).

After the pavement structure has been constructed and before the perpetual pavement is put into service, it is suggested that ground penetrating radar (GPR) testing is completed in order to get a continuous profile of the constructed pavement structure thicknesses. This GPR survey could be used to find locally deficient areas of thickness which can be corrected prior to the pavement being put into service (coring can be done to verify deficient areas identified by the GPR) and the accurate profile data can be used with the other information gathered to better relate overall site performance to those observed at the monitoring site.

Most testing equipment is required to be calibrated on an annual basis. However, structural evaluations of pavements are more often required to be completed every two years (or more depending on the agency). If the equipment is locally available to complete calibration of the monitoring site on an annual basis, then this would be ideal however it will more likely be completed in conjunction with other testing being completed for the perpetual pavement. Regardless, if observable changes in the performance of a site are noted in the sensor data, this calibration process should be advanced in order to investigate the cause of the change.

As more experience is gained in the testing and maintenance of embedded sensors and the associated required infrastructure, a better recommendation on the calibration frequency will be able to be developed.

6.2.1. Check Historic Data for Anomalies

The most straightforward and easily accessible component of the long term monitoring of perpetual pavements is the historic data from the monitoring site. In this regard, the first step in the embedded sensor testing protocol is to evaluate the embedded sensors for anomalies which may indicate either performance issues at the site or performance issues with the monitoring equipment.

This step should involve reviewing the historic monitoring data to analyze trends to see if the recorded strains are increasing or decreasing over time which may indicate a change in the performance of the pavement materials or that the embedded sensor are becoming out of calibration.

The individual strain events can also be checked and compared against each other to see if the response of the gauges is changing over time and that the magnitudes are the same or within the expected range. The strain events can also be checked to see if the signal noise is increasing which may indicate an issue with the gauge or the wiring to/inside of the datalogging unit.

It is also recommended that the response of the individual sensors be compared against other similarly aligned sensors to see if they are recording similar strain events or if there is a trend of missing event data.

6.2.2. Check the Electrical Function of Monitoring Equipment

Once on site, one of the first checks that should be completed is that the data acquisition system is functioning properly and that there are no hardware related malfunctions that are causing errors in the data. A detailed treatment of this process was completed on the Marquette

Interchange Perpetual Pavement Instrumentation Project - Phase II and is summarized below [Hornyak & Crovetti, 2008].

The individual gauges should be checked for their individual electrical resistance to verify that they are operating properly and that the resistance that they are producing is within the range outlined by the manufacturer. Failure of this test may indicate that there are broken or corroded leads.

Strain gauges are known to drift with time and age and may result in the recorded traces being improper or incomplete as shown in Figure 42 [Timm, 2009]. If this is occurring, the resistance of the strain gauges can be adjusted in order to bring them back into a range where the entire trace is recorded.

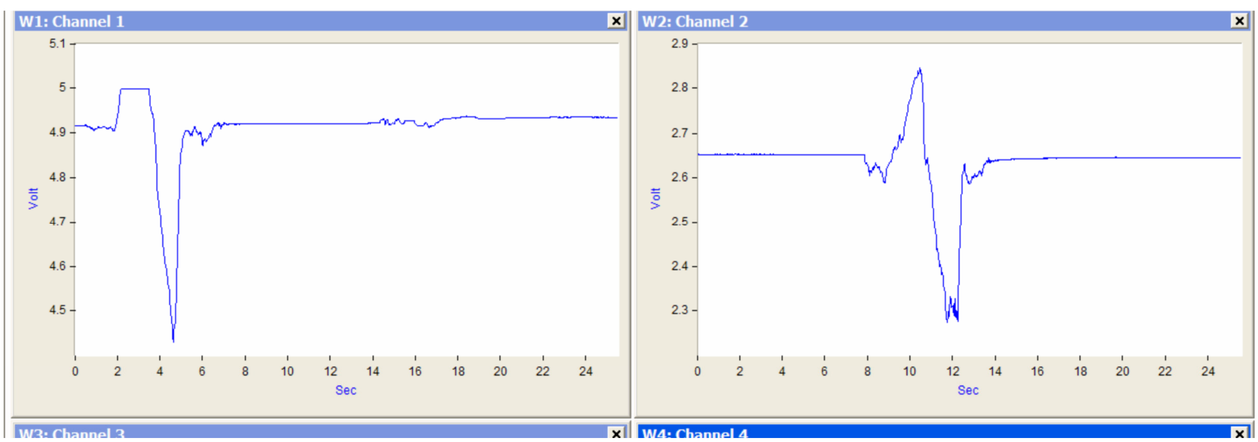


Figure 42 - Adjusting the Resistance of the Gauge to Improve Signal Quality

Next, the functioning of the datalogger should be checked using a strain gauge simulator. A strain gauge simulator is a tool that generates an electrical signal which simulates the signal provided by a strain gauge. It can be used to simulate strains up to about 10,000 με. The data which is recorded using the strain gauge simulator can be processed and compared against the data being obtained from the site to verify if the datalogger is measuring correctly and accurately.

The last recommended monitoring equipment check used in the Marquette Interchange project is the acquisition rate and sequencing/timing of the acquisition software. This is completed by connecting a signal generator to the datalogger and producing a constant signal to verify the performance of the datalogger. If the appropriate number of signals is not recorded by the datalogger, then this means that there is either a hardware or software issue with the unit that requires maintenance.

6.2.3. Complete Evaluation of Deflection Using FWD

Falling weight deflectometer testing provides three valuable parameters that can be used to evaluate the performance of the monitoring site. Firstly, when testing is completed directly above a sensor, the deflection that is generated can be compared against a design deflections predicted using mechanistic design software such as KENPAVE. Secondly, the FWD produces an impulse load that is accurately measured, repeatable, and free from the effects of wander that might be generated from other tests. Finally, the deflection basin measured using the FWD can be used to backcalculate the in-situ modulus values of the pavement structural layers in order to compare against those used during design which provides additional data that can be used to validate sensor performance.

It is recommended that the load levels and sensors spacing as well as the warmup and data validation procedures in the LTPP Protocol for FWD testing be used for the evaluation of the monitoring site [Schmalzer, 2006]. In addition, the FWD testing should consist of 34 drops at each test location with one seating drop, 10 drops at the three standard load levels and the addition of 4 drops at a higher load level (90kN to 240kN depending on the capabilities of the testing unit). The drops at the standard levels will evaluate the performance of the gauges at the normal traffic spectrum, and the additional higher load level will be used to evaluate the performance of the gauges at a higher range of the gauge performance which will provide a better validation of the existing calibration factors used. In order to more easily match the tested deflections with the measured strains, it is important that the clocks on the datalogger and FWD are synchronized prior to beginning the testing.

While this testing program consists of a considerable number of drops, it is not expected to take more than five to ten minutes at any particular test location and is more than justified when considering the cost of mobilizing the equipment and providing traffic control for a site that would be required if the equipment had to return to the site to re-test. The additional drops also allow for better statistical analysis of the results. Many FWDs have the capability of having four pre-programmed drop heights (or more) and as a result, this testing pattern should not require additional time for setup and significantly reduce the productivity of the testing if it is being completed in conjunction with overall structural monitoring of the perpetual pavement.

After completing the FWD testing, the first evaluation that should be completed is a comparison of the deflection measured on site with the deflection that is predicted based on the individual properties of the pavement structural components. If the deflections match, then this means that the design assumptions are valid and that the predicted strains should also be valid. If the deflections differ, then additional analysis will be required in order to determine which layer (or combination of layers) is performing differently than designed.

The first step to determine the performance of the individual layers would be to backcalculate the resilient modulus of the pavement structural components using the deflection basins that were recorded during the FWD testing. It should be noted that backcalculated resilient modulus values for the subgrade and granular base are not directly equivalent to those which have been determined in the laboratory and are required to be multiplied by correction factors in order to be compared [MTO, 2012]. Fortunately, this process is well established and standard correction factors are available in many jurisdictions.

Another important aspect to note when using backcalculated subgrade resilient moduli in particular, is that the factor which is applied is used to obtain a resilient modulus for a subgrade in the spring (or soaked) condition. As a result, the designer should compare the backcalculated subgrade resilient modulus to the design spring value in order to determine if there has been any change since construction. The designer should then use the resilient modulus of the subgrade at the current moisture condition (from standard material correlations or determined specifically for the site (preferred)) in order to calculate the design deflection and strains.

The resilient moduli of the asphalt concrete layers are very sensitive to temperature and the designer must ensure that the resilient modulus related to the asphalt concrete temperature at the time of testing is used to calculate the design deflections and not a standard temperature such as 21°C or else it will be difficult to match the design deflections with the measured deflection values.

When the measured deflections correspond to the design deflections, then the design strain values can be considered as valid when evaluating the embedded sensor performance.

6.2.4. Analyze Predicted and Calculated Strains

The design strains which were predicted from the previous step should be compared against those which were measured using the synchronized data. It should be noted that CTL allows a variation of up to 5% of the calibration factors for their gauges and this can be used as a guide to determine if the predicted and measured results are within the expected tolerances [Willis & Timm, 2009b]. If the results are outside of this tolerance, then an evaluation should be completed to determine a new calibration factor for the embedded sensor(s).

6.2.5. Determine New Calibration Factors (If Required)

If it is determined that new calibration factors may be required, the first step would be to plot the measured voltage from the strain measurement (in mV) against the predicted strain value on the same plot as the original calibration. A liner regression should be completed on the new strain values and the equation of the new calibration compared to the original calibration equation to evaluate whether the difference noted is as a result of a new zero offset or a change in the slope of the calibration equation. An example of a CTL strain gauge calibration factor is shown in Figure 43.

For perpetual pavement based calibration, the amount of strain that is generated is generally quite low which presents challenges when calibrating strain gauges such as those from CTL. As can be seen from Figure 43, the CTL brand gauges are designed to measure strains up to 1500$\mu\varepsilon$. By contrast, a 240 kN load on a 300 mm diameter circular plate is expected to produce strains in the range of 300 to 400$\mu\varepsilon$ for the pavement structure used at the Guelph Site, for example. As a consequence small errors in the calibration factor may result in large errors at high strain levels. By the same token however, if the new calibration factors are shown to reliably produce results in the lower testing range (0 to 150 με) then this should not be an issue as it would not be expected to observe strains at those elevated levels.

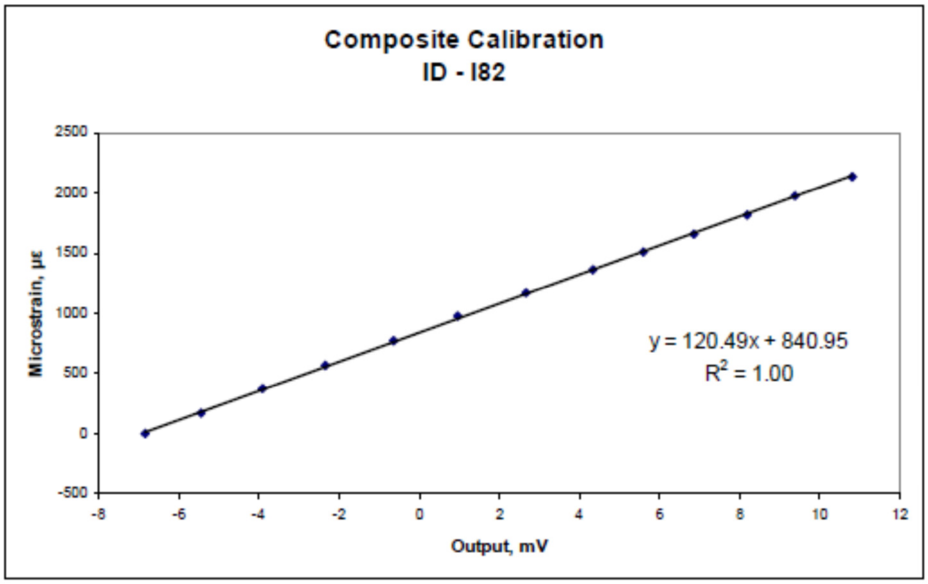


Figure 43 - Example CTL Calibration Factor [Hornyak, Crovetti, Newman & Schabeiski, 2007]

6.2.6. Validate New Calibration Factors (If Required)

The new calibration factors should be validated in the test section prior to implementing them in the monitoring program. This should be completed by updating the monitoring software with the new calibration factors and completing Steps 3 and 4 to ensure that the new factors are within the 5 percent tolerance expected for this type of monitoring equipment.

6.2.7. Decommission or Replace Sensors and Re-Calibrate if Necessary

If a sensor fails the electrical functioning test or cannot produce reliable strain results, then the sensor should be decommissioned and a decision made to determine if it should be replaced. The decision should be based on: the amount of time until the next maintenance cycle is planned (it may be more efficient and cost effective to replace the strain gauges at this time); the number of remaining functioning sensors; and an evaluation of the historic monitoring data to determine if these sensors are even necessary (if strain levels are so low as to not warrant further monitoring).

If the strain gauges are replaced, they should undergo the same testing and validation procedures outlined in this framework prior to being put into service.

A flowchart showing the key components of the Framework and their interdependencies is provided in Figure 44.

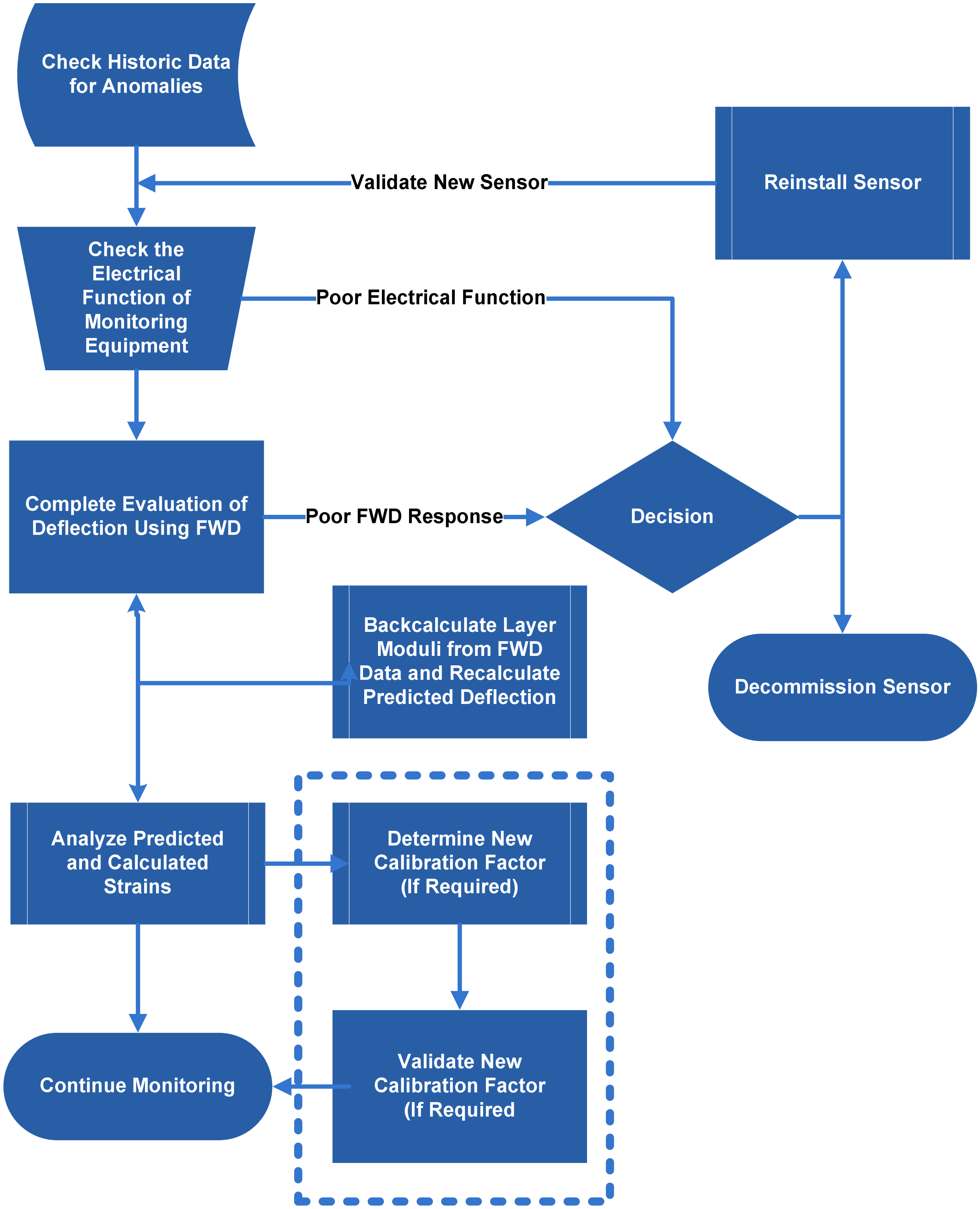


Figure 44 - Flowchart for Testing and Analysis of Embedded Sensors

7. CONCLUSIONS AND RECOMMENDATIONS

This research consists of an evaluation of a perpetual flexible pavement test site instrumented with embedded asphalt strain gauges and an earth pressure cell in order to validate their performance after several years of service and to evaluate a process of calibration of embedded sensors as they age. In addition to the initial analysis, a secondary testing program was completed in order to determine the location of the embedded sensors within the test section as well as the relative depth that they were installed within the pavement structure. The research showed that the falling weight deflectometer (FWD) is a useful tool in evaluating the performance of embedded sensors and also identified testing and analysis that could be completed in order to improve the accuracy of the FWD based analysis and make the evaluation process more efficient.

There are a number of benefits to using the FWD for embedded sensor verification and calibration. The FWD applies a known load to a circular load plate and can accurately measure the deflections that are generated from this applied load. The circular load plate simulates the type of load that is used in mechanistic design software and eliminates one of the simplifications used when modelling typical pavement/tire interfaces. The circular load, when tested directly above a sensor, also eliminates the error attributable to misalignment of a strain gauge that could be caused during construction due to the fact that the FWD load is applied radially from the load plate and does not have a directional bias. As the FWD load can be placed in the exact position that is desired, it will eliminate the error which can be caused by wander, which may occur when the load is being applied by a moving vehicle. The deflection basins that are measured during a FWD test can also be used to backcalculate the resilient modulus values of the in-situ materials, potentially preventing the need for destructive testing (slabs and laboratory resilient modulus testing) if other methods of non-destructive evaluation of the in-situ properties are not available.

An accurate backcalculation of a composite resilient modulus of the asphalt concrete layer from the FWD deflection basin information was found to be greatly simplified when the resilient modulus of the granular materials and subgrade are well known. The research noted an opportunity to include additional analysis of the variability of the site subgrade material to moisture during the design phase, as well as the addition of moisture probes in the subgrade which can both improve the results of the backcalculation and prediction of compressive and tensile strains at the site.

Using the results of the research at the test site, it is considered possible to calculate new calibration factors for embedded sensors by comparing the deflections and strains generated from the FWD testing to those that are predicted using mechanistic design software. The new calibration factors were validated in the field and were found to be within 5 percent of the design value which is within the tolerance of the gauges used on the site. It should be noted however that this hypothesis has only been studied at this one site. Additional testing of other sites is recommended to demonstrate that this procedure can be completed under a variety of site conditions and for a variety of monitoring equipment.

7.1. Recommendations

While the FWD can be used to eliminate a number of issues that can cause variability in the testing of monitoring equipment, there are still some areas where variability can exist which can create a difference between the measured and predicted strains. For one, the installation of the strain gauge can introduce a discontinuity into the pavement structure that can alter how the material is behaving. In addition, the prediction of tensile strains in most mechanistic analysis assumes that each layer is homogeneous, isotropic, and linearly elastic which is not really the case. It is recommended that these differences be evaluated after construction has been completed and before the road has been opened to traffic to allow for these differences to be investigated and the FWD load/deflections to be correlated to the strains generated in the constructed pavement structure. This correlation can be used in the future to quantify the variability observed when comparing the two processes which will provide more accurate results in the future validation and calibration of the embedded sensors.

It is recommended that the correlation of load/deflection to pavement structural response be determined not only after construction of the perpetual pavement is completed, but after each layer (subgrade, granular base, and each individual asphalt concrete layer) has been constructed. This testing could easily be completed using a FWD or even a LWD for some of the thin (initial) layers that do not require large loads to generate a response. This additional testing can further correlate load/deflection response with each successive material layer that is constructed which is expected to provide more information on which layers are contributing disproportionately to the variability and where further destructive or non-destructive testing may be warranted to investigate the source of the variability. This testing also allows for a “check” on the design at each stage of pavement construction.

The frequency of embedded sensor validation and calibration discussed is based on general laboratory procedures and field practices used with other non-destructive equipment. The in-situ testing of embedded sensors is a relatively new process and it is recommended that this process should continue to be studied and the frequency adjusted as more experience is gained in this field.

7.2. Opportunities for Further Research

Opportunities for further research and investigation were identified during this research. These opportunities can be summarized as follows.

1. Investigate the possibility of placing a strain gauge at the bottom of the upper binder course layer (approximately 90 mm depth for the pavement structure used at the Guelph Site) to determine the predictability of the performance proportionally to the gauges installed at the interface with the underlying granular material. A gauge installed at this depth could more readily be replaced for the long term monitoring of a perpetual pavement and could be completed without disturbing the lower layers and affecting the evaluation of their performance as they age.
2. The strains and deflections predicted using the mechanistic design software were slightly different when using a weighted average (composite) modulus of the entire asphalt concrete layer rather than the moduli of the different layers individually. The use of a composite modulus is required when using moduli backcalculated from FWD deflection data and is one of the limits of the backcalculation process. As a result, this introduces another form of variability when comparing measured and predicted deflections and strains that should be further evaluated.
3. When using a weighted average (composite) modulus of the asphalt concrete layer in the mechanistic design software, the resilient modulus was adjusted for temperature based on a mid-depth temperature. The modulus/temperature relationship is known to vary somewhat from mix to mix based on the properties of the materials used in the design. In addition, given that the asphalt concrete layer in a perpetual pavement is thicker than typical designs, a temperature gradient (differential) can form between the top and the

bottom of the combined layers. It is recommended that research be completed to develop an appropriate correction factor for a composite pavement structure with varying temperature gradients.

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